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Building Practices for Disaster Mitigation

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ABSTRACT

The National Workshop on Building Practices for Disaster Mitigation was concerned with earthquakes, extreme winds, and similar dynamic hazards. These proceedings present recommendations derived at the workshop and addressed to policy makers in government and industry as well as practitioners in engineering, architecture, land use planning, and the earth and meteorological sciences. The recommendations evaluate current building practices, define opportunities for improving current practice from documented research findings, and recommend research to fill gaps in knowledge. Recommendations are made for implementation of improved practices at professional and policy levels. The objectives include avoidance of human suffering, reduction of property loss, and maintenance of vital function in buildings under conditions threatening disaster. Fifteen review articles were prepared by knowledgeable individuals in the professions and research disciplines to define the state of the art in disaster mitigation and to guide discussions at the workshop; the articles are included in the proceedings.

Key Words: Building; earthquakes; hazards; land use; natural disasters; structural engineering; wind effects.

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BUILDING PRACTICES FOR DISASTER MITIGATION

INTRODUCTION

Natural Disasters

Property losses from natural disasters have averaged approximately one billion dollars per year in the United States. This is the most easily measured but perhaps the least important part of the losses. Losses from human suffering, disruption of productive activities and expenditures in disaster relief are difficult to quantify but magnify disaster losses many fold.

The Disaster Preparedness Study, published in January 1972 by the Office of Emergency Preparedness in response to Public Law 91-606, states: "Land-use and construction regulations containing strong disaster mitigation features can in the long run alleviate losses caused by natural disasters." Disaster mitigation denotes preventive measures to reduce the damages caused by extreme environments such as earthquakes and high winds and therefore to avoid the chain reaction of failures which is called disaster. Buildings support and shelter most human activities. Thus, preventing building collapse mitigates most human suffering, preventing building damage greatly reduces property losses, and continuing the functionality of buildings supports emergency activities and prompt return of normal economic and social functions. These benefits require consideration of disaster hazards in the practices of land use planning, building design, construction, operation and maintenance.

Any building site is subjected to several types of natural hazards. These may include earthquake and extreme winds as well as floods, landslides, storm surges and conflagrations. All substantial risks should be considered in design and use of a building. The National Workshop on Building Practices for Disaster Mitigation held August 28 to September 1, 1972, focused on prompt and effective approaches to disaster mitigation.

Earthquakes and extreme winds, as well as explosions and some accidents, produce closely related dynamic loads and mobilize similar resistance mechanisms. Integrated attention to these hazards costs little more than treatment of earthquakes alone and provides substantially increased benefits. The building practices discussed at the workshop focus on these hazards. Although not dealt with explicitly, flooding, landsliding, and storm surge hazards also can be effectively mitigated by the land use planning and control practices discussed.

The measures for disaster mitigation treated at the workshop can be subdivided into Policies and Practices. Policies comprise executive or legislative actions of public authorities as well as the regulations of private organizations such as lending and insurance companies. Practices denote the implementation of policies through the activities of design professions, builders, and regulatory authorities. This report addresses the building practices necessary for effective mitigation of the selected group of disaster hazards and the policies needed to authorize their implementation.

National Workshop on Building Practices for Disaster Mitigation

This report, which constitutes the proceedings of the National Workshop, reviews current building practices in the light of disaster mitigation and recommends action for four distinct audiences.

1. Policy makers in Federal, State and local governments, to provide guidance for laws, regulations, policies, and programs for

mitigation of hazards of natural disasters.

2. Practitioners, for guidance on the best current practices for decisions which must be made at once.

3. Standards writers and those responsible for developing recommended practices, for guidance on the deficiencies in present practices which can be resolved by developments from well-documented research findings.

4. Researchers, for guidance as to the unknowns which most severely constrain improvements in building practices.

The recommendations were prepared at the workshop which brought together the diversity of knowledgeable professional and regional interests required for a balanced view of building practices for disaster mitigation. Most of the participants, who are listed in Appendix C, are practitioners in planning, design and supervision of construction because these are critical steps in building for disaster resistance. However, some are researchers who increase the level of knowledge about the loadings causing disasters and about the response of buildings to these loadings. Most are engineers because the topics are strongly involved with the physical response of buildings in disasters and with design approaches to control of this response. Physical scientists, architects, planners, social scientists, and public administrators also provided their perspectives of disaster mitigation.

The discussions of the workshop were based upon review articles which summarize the state of practice and research knowledge in the principal areas of building practice. The review articles are contained in Appendix B.

The review article "Values and Costs" is by Howard Kunreuther, an economist, who has worked on the economics of natural disasters. The article points out that acceptable levels of risk of disaster losses depend upon the balance between the value of lesser risk and the cost of obtaining it. Potential losses include losses of physical property, indirect economic losses, failure of buildings to perform functions of importance to the community, and human suffering. The review describes approaches to a detailed evaluation of these losses in terms which would improve the ability of decision-making bodies to define what constitutes acceptable risk and to assess accordingly present and future codes. The economic relations between disaster relief and disaster mitigation policies also are developed.

The review article "Approaches to Implementation" is by Paul Baseler and reflects his career experience in the building regulatory system. Better building practices are effective only when improved procedures for land use planning, siting, design, construction, and operation are effectively implemented. The review evaluates approaches to the formulation, enactment, and enforcement of land use and building regulations. Actions are considered for many pertinent roles, such as legislative, professional planning and design, construction management and labor, industrial, and governmental. Organizations which may play important roles in improving building practices are identified, their potential contributions described, and approaches to their participation recommended. Specific attention is given to resolution of potential conflicts and impediments to implementation of improved building practices.

The review articles by Neville Donovan, a foundation engineer, "Earthquake Hazards for Buildings," and by S. T. Algermissen, a seismologist, "The Problem of Seismic Zoning," describe our knowledge of the environment to which earthquakes may subject buildings. They consider effects of ground shaking and permanent relative displacements arising from faulting, subsidence, and sliding. Practices accounting for expected amplitude and duration of shaking as functions of recurrence interval, geographical location, overburden characteristics, and topography are reviewed in light of research knowledge. The former review also treats soil-foundation-structure interaction as a function of the dynamic response characteristics of the structure.

The review article "Wind Hazards for Buildings," by Joseph Vellozzi and John Healey, structural engineers, deals with the effects of geographical location and local site conditions on extreme winds from hurricanes, thunderstorms, tornadoes, and extratropical storms. Attention is given to: wind speeds as a function of recurrence interval, structure of wind turbulence, variation of speed with height above ground, height of the planetary boundary layer, lapse rates and thermal winds. The aerodynamic interactions of buildings and building elements with the wind are reviewed with attention to overall and local external pressure coefficients for various body shapes, and effects of openings on interior pressures. Current practices for design, including use of wind tunnel tests, are related to documented research findings.

The review article "Abnormal Loading on Buildings and Progressive Collapse" by Norman Somes, a research structural engineer, considers loads from explosions, collisions of vehicles and aircraft, unexpected failure of members and other causes which may lead to general structural failure and disastrous loss of function, property or life. Procedures and criteria accounting for these loads in current building practices are evaluated in light of experiences with these loads and documented research knowledge.

The review article "Procedures and Criteria for Land-Use Planning" is prepared by a team of planners, public administrators, geologists and engineers. It relates natural disaster hazards to site characteristics, building uses, and community planning. Examples include soil conditions which amplify earthquake motions at the natural frequencies of the building, the presence of potentially active faults, susceptibility to subsidence or landsliding, and topography leading to severe wind exposure or exposure to storm surge or moving flood water. Procedures and criteria for land use planning can forbid potentially hazardous uses or can require special efforts to mitigate site hazards. The review of current practices characterizes the approaches taken by current criteria and practices and evaluates their physical, economic, and social effectiveness.

The review article by Ernest Hillman, Jr., structural engineer, and Arthur Mann, architect, "Architectural Approaches to Hazard Mitigation" discusses how building forms, spatial distributions, and interior designs can be adapted to natural disaster hazards as part of the design to provide a functional environment in harmony with the natural environment. The review evaluates organizations of the design team for synthesis of external and internal forms which provide effective function and benign interactions with earthquakes, extreme winds, explosions and similar causes of disasters. Methods are noted for encouraging architectural approaches to disaster mitigation at the programming and schematic design stages.

Review articles "Procedures and Criteria for Earthquake Resistant Design" are presented by C. W. Pinkham, structural engineer, and by N. M. Newmark and W. J. Hall, research structural engineers. Experiences in recent earthquakes have provided a laboratory for evaluation of present procedures and criteria for earthquake resistant design. Design procedures and the criteria used with them are reviewed for consistency and effectiveness in accounting for loading magnitude, structural behavior, and the desired limitation of damage. The reviews consider the degree to which procedures and criteria tend to reinforce the designer's appreciation for the actual loading and response characteristics and assist him in understanding limitations of the loading and response models and effects of "non-structural" elements on structural behavior.

The review article "Procedures and Criteria for Wind Resistant Design" by Joseph Vellozzi and John Healey, structural engineers, is based on recent experiences with buildings subjected to severe winds as well as theoretical and laboratory studies. Design procedures and the criteria used with them, are reviewed as noted above for earthquake-resistant design. Design approaches are considered for hurricanes, tornadoes, thunderstorms, and extratropical storms.

The review article "Criteria for Building Services and Furnishings" is prepared by J. Marx Ayres and Tseng Yao Sun, mechanical engineers. Failures of equipment and services, such as elevators, luminaires, and piping, and furnishings such as suspended ceilings and shelving often cause loss of vital function, extensive property loss, and severe hazards to life. The review defines mechanisms of failure induced by earthquakes, extreme winds, explosions and similar causes, evaluates procedures and criteria for avoidance of these failures, and discusses means for effective implementation. Specific attention is given to both mechanisms which can be accounted for in criteria for non-structural systems and those which require consideration paralleling that of the primary structural system.

The review article "Behavior of Structural Elements" by Boris Bresler, research structural engineer, evaluates design and construction procedures and criteria intended to assure adequate strength and ductility in structural members and connections. Consideration is given to expression of resistance functions (relations of generalized force to corresponding generalized displacement) for repeated and reversed loadings as dependent on strain history, rate of strain, and mechanism of failure. Simplified resistance expressions for practical use in design are evaluated for range of validity and the extent to which they illuminate the parameters and mechanisms most strongly affecting the element behavior.

The review article "Behavior of Structural Systems under Dynamic Loads" by Roland L. Sharpe, Garrison Kost, and James Lord, structural engineers, evaluates methods for the analysis of the response of structure to the dynamic loads which threaten damage in natural disasters. It presents first a general review of analytical methods followed by a discussion of structural configurations and modeling procedures. Applications of the analytical methods are then presented, and many of the factors to be considered in the dynamic analysis and design of a structure are discussed. As equipment and systems supported in structures are often important to public safety, equipment-structure interaction and suggested analytical techniques are also discussed. Finally, present philosophy in practice and trends in design are reviewed together with possible simplified design techniques.

The review article "Survey and Evaluation of Existing Buildings" is prepared by Frank E. McClure, structural engineer. Buildings constructed with earlier building practices require special evaluation of hazards from earthquakes, extreme winds, and similar loadings in order to avoid future disasters. In emergency and post emergency situations, evaluations of deteriorated and damaged buildings are needed to determine whether they are safe for immediate use and whether repairs are economically feasible. Present procedures and criteria are reviewed and evaluated; recommendations are given for public policies and professional practices in survey and evaluation, as well as needs for research on methods of strengthening and repair.

There is a strong emphasis on earthquakes and Californian experiences in the review articles. Is this emphasis on earthquakes reasonable when property losses from wind damage to roofs, on the average, exceed earthquake losses, and when lives lost in tornadoes exceed those in earthquakes? Likewise, is the emphasis on Californian experiences valid in light of the fact that substantial risks of great earthquakes exist for areas east of the Rockies? It occurs because recent experience has shown severe inadequacies in the widely accepted design practices for earthquake resistance and because the building community in California has worked most effectively on these problems. However, the discussions and recommendations adopt a national view in identifying best practices for earthquake and extreme winds, in recommending development of improved practices, and in calling for effective focusing of research efforts.

Better building practices applied nationwide are needed for reduction of wind losses and to end the potentially catastrophic neglect of earthquake hazards in most of the country.

The recommendations were developed in two days of subcommittee meetings following two days of intensive discussion of the problems as shown in the workshop program in Appendix A. The following subcommittees took an interdisciplinary view of policies and practices for mitigation of specific hazards:

1. Implementation of Earthquake Hazard Reduction.

2. Formulation of Earthquake Hazard Reduction Practices.

3. Implementation of Wind Hazard Reduction Practices.

4. Formulation of Wind Hazard Reduction Practices.

On the fourth day, the subcommittees reviewed and commented upon each others' preliminary reports. The second draft reports of the subcommitee, which contained recommendations and comments, were discussed on the fifth day of the workshop. The editors subsequently assembled the recommendations and commentaries into the form presented here. This was followed by a final review by all participants.

<u>Cooperative Federal Program on Building Practices for Disaster Mitigation</u>

Federal concern with building practices for disaster mitigation arises from major statutory responsibilities. Approximately 37 percent of all new construction is directly or indirectly Federally supported; good practices protect these investments and responsibilities. The Federal government is obligated by PL 91-606 to pay for repair or replacement of all public facilities damaged in natural disasters; private nonprofit hospitals recently have been added to this responsibility. Forgivable amounts of Federal loans to private citizens for repair of disaster damages has approached $200 million for the San Fernando earthquake of 1971.

However, Federal support for these building practices has not been consistent with the potential benefits. Millions of Federal dollars have been spent each year in research on basic mechanisms of earthquake and storms and on the response of structures, but practicing engineers have generally been left to synthesize this knowledge and develop improved building practices with their own funds in their spare time. This gap between research and practice has long been evident to both practitioners and researchers.

The Cooperative Federal Program in Building Practices for Disaster Mitigation was initiated in the spring of 1972 by the National Science Foundation and the National Bureau of Standards, following planning throughout the winter in conjunction with the Office of Emergency Preparedness and the Department of Housing and Urban Development. The program seeks to integrate extensive activities and resources in Federal agencies and work with professional organizations, private practitioners, and state and local governments in support of improved building practices.

The overall objectives of the program are: (1) to synthesize current knowledge and develop improved building practices for insuring the safety of new and existing buildings with substantial human occupancy and (2) to make this knowledge available in usable form to assist state and local officials in effecting land-use planning and building regulation to mitigate the effects of natural disasters.

This report is the first major output of the program.

Future Actions

The recommendations of the workshop call for substantial efforts in development of improved practices and in important research. A greater effort, if all costs are counted, is required to implement best current practices and future improvements. Design engineers accustomed only to concepts of static design for strength must become familiar with dynamic structural responses and the parameters affecting the energy absorbing abilities of buildings. Construction personnel and building inspectors must understand how seemingly minor factors, such as connection details, vitally affect structural resistance. Then, good practice will be meaningful rather than a seemingly arbitrary nuisance. Administrators, planning groups, legislative bodies, and the using public must be made aware of the costs and potential values of measures for disaster mitigation so they may make informed decisions on policies affecting disaster hazards.

Activities in implementation of the workshop recommendations have begun. The Defense Civil Preparedness Agency has joined the Cooperative Federal Program to support development of methodology and criteria for evaluating the safety of existing buildings. Discussions of further activities and participation of additional agencies are proceeding.

WORKSHOP RECOMMENDATIONS

The recommendations contained herein represent the consensus of the participants in the workshop, but do not constitute an individual endorsement by any participant or his organization.

A. POLICIES FOR DISASTER MITIGATION

The number of groups involved in the building process is large and their interests are varied. All these groups, however, must be involved in any implementation program to effectively mitigate losses from future natural disasters. Such an implementation program must consider changes that will effect current practices in design and construction, changes that will give new tools to policy makers and political jurisdictions so that risks can be assessed in policy making, and changes that will effect the many regulatory systems so that new policies can be carried out and the public protected. The overall objective of this implementation program is the mitigation of future losses of life and property.

Actions taken by governmental bodies at the Federal, State, and local levels, as well as by the professional, financial and insurance communities can play a significant role in achieving disaster mitigation through improved land-use planning, building design, and construction practices. The Federal government through its own direct construction programs and through its many grants, loans, insurance assistance and regulatory programs can lead the way towards implementing practices which recognize risks of natural disasters and plan for mitigating losses. However, it must be recognized that State governments have the constitutional responsibility in this field of health and safety and they in conjunction with local governments must take action to implement disaster mitigation programs with the Federal government providing support and assistance when needed. Governmental bodies at all three levels require the active participation and expertise of a wide spectrum of disciplines in design and engineering, the physical and social sciences, and the banking and insurance industries.

These recommendations were formulated to define the changes necessary to implement a national program for mitigating losses from natural disasters. While few are exclusive to a particular group, they have been divided for convenience into those requiring prime action at governmental levels, by the insurance industry, and by the professional design and construction community.

Actions at Governmental Levels

Land use plans and building regulatory measures based on a consideration of the risks associated with natural disasters provide an effective means for hazard mitigation. Comprehensive land-use plans (general plans, master plans) should include hazard mitigation sections similar in content to the "Seismic Safety Element" for earthquakes included in the California General Plan Law enacted in 1971. (Section 65302 of the Government Code [California]) Such measures will assure that the citizenry and their elected representatives are informed of the risks associated with all aspects of natural disasters.

RECOMMENDATION A1: LAND USE PLANNING

A NATIONAL PROGRAM SHOULD BE ESTABLISHED TO COORDINATE LAND USE PLANNING FOR DISASTER

MITIGATION. IT SHOULD BE DIRECTED TOWARD STATE AND LOCAL GOVERNMENT IMPLEMENTATION.

An effective system of planning for disaster mitigation must consider land use planning at the State, regional, and local levels. Planning should be undertaken by qualified professionals. Plans must be presented in a manner easily understood and used by all parties. A national program in this area could provide coordination of the planning processes and assure reasonable consistency among the States. The establishment of a clearinghouse for the dissemination of design criteria and standards for land use planning should be included in the program.

RECOMMENDATION A2: STATE AUTHORITY FOR REGULATIONS

STATE GOVERNMENTS SHOULD ESTABLISH MINIMUM RULES AND REGULATIONS TO PROVIDE FOR THE SAFETY OF THE PUBLIC FROM THE HAZARDS OF NATURAL DISASTERS. THEY SHOULD REQUIRE INCORPORATION OF THESE PROVISIONS IN LOCAL LAND USE PLANNING AND BUILDING REGULATORY MEASURES.

State governments in exercising their constitutional authority for the promulgation of codes should develop provisions which recognize the hazards of natural disasters. Since such disasters usually occur over a geographical area encompassing several local governmental jurisdictions and the effects in one locality may influence neighboring communities; comprehensive area-wide planning to mitigate these effects is necessary. Local land use plans and building codes should incorporate provisions dealing with natural disasters and other hazards. These provisions should be applicable not only to structures with substantial human occupancy but to all potentially hazardous facilities. States should require coordination of codes and ordinances in comprehensive plans to assure consistent area wide standards. The States should also consider the protection of vital emergency services required during and after the occurrence of natural disasters in setting minimum standards with respect to land use planning, building codes and emergency planning.

RECOMMENDATION A3: EXPLICIT DEGREE OF RISK

CODES, STANDARDS AND CONTRACTS FOR ARCHITECTURAL AND ENGINEERING SERVICES SHOULD CONTAIN LANGUAGE SETTING FORTH THE DEGREE OF RISK ACCEPTED IN THE DESIGN PROCESS.

The building codes now in effect in earthquake-prone areas are not intended to prevent damage to buildings during major earthquakes. Moreover, good current practice as described in the workshop papers implies a risk of damage. This risk is not well recognized by the public nor by most building owners, and is not clearly stated in architectural and engineering contracts. Accordingly, while it is generally held that it usually is not feasible to design buildings so as to have no damage during natural disasters when extreme loads are experienced, the engineer and architect find themselves open to criticism and even lawsuits if damage occurs. Because the situation is not openly recognized, the architect or engineer is seldom able to discuss the risk, and possible steps to reduce the risk, with his client. Often, risk of damage can be significantly decreased by improved practices, sometimes at little increase in construction cost, but the necessary engineering studies are not considered nor included within the usual services. Studies to achieve a lesser risk should be recognized as an important additional service.

RECOMMENDATION A4: SAFETY OF EXISTING BUILDINGS

LAND USE REGULATIONS AND BUILDING REGULATORY MEASURES SHOULD INCLUDE PROVISIONS FOR EVALUATION OF THE SAFETY OF EXISTING BUILDINGS AGAINST NATURAL HAZARDS AND REHABILITATION OF UNSAFE BUILDINGS.

In addition to insuring the adequacy of new construction, it is important to ascertain the safety of the large inventory of existing buildings. The City of Long Beach, California, Ordinance C-4950, "Earthquake Hazard Regulations for Rehabilitation of Existing Structures Within the City" serves as a useful guideline in this respect. It also sets standards for rehabilitation of existing unsafe buildings.

RECOMMENDATION A5: BUILDING REGULATORY PERSONNEL

STATE GOVERNMENTS SHOULD ESTABLISH CRITERIA AND PROGRAMS FOR QUALIFICATION AND CERTIFICATION OF BUILDING REGULATORY PERSONNEL.

Improved inspection and enforcement of building regulations are essential to implementation of practices for disaster mitigation. The skills needed for various activities of the regulatory bodies should be identified. Professional degree programs, two-year technology programs, trade school courses, continuing education seminars, and in-service training should be employed to upgrade current personnel. Salaries should be reviewed to permit retention and attraction of properly qualified personnel.

RECOMMENDATION A6: REVIEW BOARDS

LEGISLATIVE BODIES ADOPTING BUILDING CODES AND LAND USE REGULATIONS SHOULD CREATE BY STATUTE "REVIEW BOARDS" TO HEAR REQUESTS FOR VARIANCES AND APPEALS FROM DECISIONS OF REGULATORY OFFICIALS.

The "Review Board" would provide a means for obtaining consistent procedures for the review of investigations, design criteria, and plans which are not within the scope of existing regulations. This review procedure should assure that appropriate levels of disaster protection, consistent with the risk, are attained where exceptions are granted. The legislation should define the authority of the "Review Board" and the professional qualifications of its members. Professional societies should advise in the appointment of board members.

RECOMMENDATION A7: BUILDING OPERATORS

STATE GOVERNMENTS SHOULD REQUIRE THAT BUILDING OPERATORS BE TRAINED FOR DISASTER PREPAREDNESS AND OPERATIONS.

Building operators should be trained to help mitigate disasters by preventive maintenance procedures, keeping as-built record drawings updated, and following through on routine safety tests of mechanical, electrical, elevator, and fire protection systems. They should be familiar with their responsibilities in state and local disaster plans.

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Governmental programs can provide appropriate incentives for improved practices for disaster mitigation.

RECOMMENDATION A8: FEDERAL DISASTER ASSISTANCE AS STIMULUS TO MITIGATION

THE FEDERAL DISASTER ASSISTANCE PROGRAM SHOULD BE REFORMULATED TO PROVIDE INCENTIVES FOR PREVENTIVE DISASTER MITIGATION EFFORTS AND TO REDUCE POST-DISASTER INCENTIVES FOR IGNORING THE RISKS ASSOCIATED WITH A SPECIFIC STRUCTURE AND SITE.

The Federal Disaster Assistance Program, as presently constituted, provides liberal post-disaster benefits in such form as temporary housing, low cost loans and employment compensation to individuals, and rehabilitation of public facilities. Individuals and communities have come to rely on governmental help after a disaster, rather than to develop an awareness of disasters and take feasible preventive measures. In essence, the program is weakening preparedness and mitigation efforts. While some governmental aid is clearly indicated on compassionate grounds, the assistance program needs to be re-evaluated so that it provides a stimulus and not an impediment to predisaster mitigation efforts by individuals and communities.

RECOMMENDATION A9: HIGHER STANDARDS IN POST-DISASTER RESTORATION

THE FEDERAL DISASTER ASSISTANCE PROGRAM SHOULD REQUIRE THAT POST-DISASTER REHABILITATION AND REPLACEMENTS TO DAMAGED FACILITIES BE DESIGNED AND CONSTRUCTED WITH A VIEW TOWARDS MITIGATING FUTURE LOSSES.

The Federal Disaster Assistance Program currently in effect calls for the rehabilitation and replacement of buildings damaged or destroyed by a declared disaster in accordance with existing land use and codes. Speed in providing aid is stressed. Consequently, structures are rebuilt on the same sites under codes that may not reflect the latest technological advances. Such structures are therefore exposed to the same pre-disaster risks and may have to be rebuilt at public expense in the case of a new disaster. This condition should be corrected in re-evaluating the Federal Disaster Assistance Program.

RECOMMENDATION A10: TAX RELIEF

TAX REGULATIONS SHOULD BE REVIEWED TO IDENTIFY FEATURES THAT WOULD PROVIDE EQUITABLE TAX RELIEF TO ENCOURAGE INSURANCE FOR LOSSES IN NATURAL DISASTERS.

For infrequent hazards such as earthquakes, the tax regulations could provide for a longer tax averaging period in order to build larger insurance reserves for these infrequent natural hazards.

RECOMMENDATION A11: INSURANCE REQUIREMENTS

THE BANKING REGULATORY AGENCIES SHOULD REQUEST

THAT THE AMOUNT OF MONEY LOANED OR GUARANTEED BE INSURED AGAINST NATURAL DISASTERS.

Such actions at the Federal and State level regarding insurance would provide far-reaching incentives for disaster mitigation. They could be accomplished within existing, already legislated programs to stimulate owners and local communities to upgrade building practices and lower insurance premiums.

RECOMMENDATION A12: EFFECTIVE MITIGATION IN FEDERALLY ASSISTED PROJECTS

THE FEDERAL GOVERNMENT SHOULD EMPLOY ITS FINANCIAL ASSISTANCE AND LOAN INSURANCE PROGRAMS TO ENCOURAGE ENACTMENT OF EFFECTIVE LAND USE PLANNING REGULATIONS AND BUILDING CODES, AND THEIR STRICT ENFORCEMENT. APPLICATIONS FOR FEDERAL FUNDS SHOULD BE REVIEWED TO INSURE THAT CONSIDERATION HAS BEEN GIVEN TO RISKS FROM NATURAL DISASTERS.

Many local programs are dependent on financial assistance from the Federal government. About forty percent of the building activities of the country are influenced by the Federal government. A condition for granting such assistance should be the adoption of hazard mitigation measures, as recently prescribed by HUD in regard to flood insurance protection in those communities which have joined the National Flood Insurance Program. This policy could be extended to withholding of Federal assistance from State and local communities (after notification to comply) if disaster mitigation elements are not enacted as part of the comprehensive planning process.

The Federal guidelines for evaluation, review, and coordination of Federal and Federally assisted programs and projects, as issued by the Office of Management and Budget, Executive Office of the President (Circular #A-95), require that applications for Federal assistance be coordinated by an officially designated State or Metropolitan/Regional Clearinghouse. The Clearinghouse is charged with the responsibility for review and comment on the application to ascertain whether or not the project is consistent with area wide plans and programs, and that overlap and duplication in the planning process has been minimized. The Clearinghouse should expand its review to insure that the siting and design of the project have considered the risks from natural disasters.

RECOMMENDATION A13: STATE AND LOCAL TAX INCENTIVES

EFFECTIVE TAX AND OTHER INCENTIVES AT THE STATE AND LOCAL GOVERNMENTAL LEVELS SHOULD BE DEVELOPED TO ENCOURAGE DISASTER MITIGATION IN PLANNING AND CONSTRUCTION.

Such incentives or penalties may be included in the tax systems, rather than allowing the continuance of the present system where the improvement of property with a view toward mitigating losses from natural disasters usually increases the tax burden.

Governmental organizations can provide a strong stimulus to disaster mitigation by setting an example through their own actions. While some agencies have initiated such policies, more positive moves in this direction are feasible and should be undertaken.

RECOMMENDATION A14: HAZARD MITIGATION IN NEW FEDERAL BUILDING

THE FEDERAL GOVERNMENT SHOULD INCORPORATE HAZARD MITIGATION CONSIDERATIONS IN ITS BUILDING AND LEASING OPERATIONS.

It is feasible and most cost effective to initiate preventive measures prior to occupying new facilities. Buildings for Federal use should be individually investigated to assess their anticipated performance in natural disasters. Evaluation reports must be prepared on a consistent basis using nationally established guidelines.

RECOMMENDATION A15: CORRECT DEFICIENCIES IN GOVERNMENT BUILDINGS

FEDERAL, STATE, AND LOCAL GOVERNMENTS SHOULD IMMEDIATELY COMMENCE PROGRAMS TO CORRECT DEFICIENCIES IN BUILDINGS THEY OCCUPY.

Many existing structures pose potential hazards in the event of a natural disaster. Remedial action by government agencies for structures they are responsible for or occupying would considerably mitigate future losses. Vital facilities necessary for providing immediate post-disaster relief such as hospitals, emergency communication centers, schools and other shelters, etc. should be given high priority.

RECOMMENDATION A16: ELIMINATE UNSAFE BUILDINGS IN URBAN RENEWAL

THE ELIMINATION OF OLD AND POORLY MAINTAINED BUILDINGS POSING SUBSTANTIAL HAZARDS IN THE EVENT OF A NATURAL DISASTER SHOULD BE ONE OF THE IMPORTANT FACTORS CONSIDERED IN URBAN RENEWAL PROGRAMS.

Urban renewal programs provide an excellent opportunity to abate clearly hazardous buildings in the current inventory.

RECOMMENDATION A17: LOCAL DISASTER EMERGENCY PLANS

LOCAL GOVERNMENTS SHOULD DEVELOP COMPREHENSIVE DISASTER PREPAREDNESS PLANS WHICH INCLUDE MEANS FOR PROVIDING LIFE SUPPORT ACTIVITIES DURING AND FOLLOWING A NATURAL DISASTER, REPAIR AND REHABILITATION OF DAMAGED FACILITIES.

Local governments have a responsibility to provide emergency services including supplies, electric power, sites for debris disposal, etc. following a disaster. Contingency plans considering sources of supply and logistical problems must be made prior to occurrence of a disaster. Means must be established to insure that post disaster repair and remedial work is carried out in accordance with accepted standards and certified by local building authorities. The review article by McClure describes some of the provisions already in existence for rehabilitation following an earthquake.

RECOMMENDATION A18: FEDERAL AND STATE ASSISTANCE FOR DISASTER PREPAREDNESS

FEDERAL AND/OR STATE ASSISTANCE SHOULD BE PROVIDED TO LOCAL GOVERNMENTS FOR DEVELOPING AND IMPLEMENTING DISASTER PREPAREDNESS PLANS AND CONDUCTING POST DISASTER DAMAGE ASSESSMENTS.

Both financial and nonfinancial assistance should be provided. The identification and acquisition of sites and equipment for use in the event of a disaster, funds to permit building inspection and consulting engineering services for damage assessment, and coordination of disaster response activities between local jurisdictions fall in these categories. A re-evaluation of funding allocated to civil preparedness programs should be made to assure most effective use.

An effective program of disaster mitigation requires a data bank of documented information and appropriate standards and criteria for building practices.

RECOMMENDATION A19: SURVEY EXISTING BUILDINGS

A SURVEY SHOULD BE MADE OF THE INVENTORY OF EXISTING BUILDINGS TO ESTABLISH THE POTENTIAL LOSSES IN NATURAL DISASTERS. FEASIBLE ALTERNATIVES, INCLUDING REHABILITATION OR REMOVAL, SHOULD BE DEVELOPED FOR REDUCING RISKS FROM HAZARDOUS STRUCTURES IDENTIFIED IN THE SURVEY.

Hazard mitigation for new construction can be achieved through use of up to date standards and specifications. However, many existing structures were built prior to incorporation of hazard mitigation provisions in codes or in accordance with codes which have since been updated. It is essential to identify seriously hazardous structures and to take appropriate remedial action. The survey results should be presented in a form clearly indicating the probabilities of losses. Specific attention should be given to life safety hazards posed by nonstructural elements. Results of the survey, analyses, and evaluations, and recommended remedial actions must be effectively put before the appropriate officials, in both general government and building control fields, public utilities corporations, the general public and the media. The presentation to each should be tailored to their responsibilities and interests.

RECOMMENDATION A20: POST DISASTER SURVEYS

A COORDINATED PROGRAM INVOLVING FEDERAL, STATE, AND PROFESSIONAL ORGANIZATIONS SHOULD BE ESTABLISHED FOR CONDUCTING POST DISASTER SURVEYS.

The occurrence of a natural disaster presents an excellent opportunity to gain important engineering, scientific and social data. The engineering and scientific data are useful for developing improved disaster mitigation practices and policies. The social data will help define losses of productivity, and tax base, and psychological and economic impacts on individuals and communities. There has been insufficient coordination, in the past, between post event investigators. Contingency plans and procedures should be established to collect in a comprehensive, non-duplicative manner the information made available by the occurrence of a natural disaster. Such plans should include provisions for the development of base line information prior to the occurrence of a disaster.

RECOMMENDATION A21: BENEFIT COST STUDIES

THE FEDERAL GOVERNMENT SHOULD INITIATE STUDIES AND PROGRAMS TO DEVELOP AND COLLECT DATA CONCERNING THE MANY LESS IMMEDIATE AND OFTEN INTANGIBLE COSTS OF DISASTERS: LOSS OF PRODUCTIVITY, LOSS OF TAX BASE, PSYCHOLOGICAL AND ECONOMIC IMPACTS ON INDIVIDUALS AND COMMUNITY, ETC.

Benefit-cost studies can provide valuable data inputs for developing and implementing a hazard reduction program. It may not be economically feasible to mitigate completely the effects of natural hazards. Hence it may be necessary to decide how much of our resources should be devoted to mitigating natural disasters as compared to other hazards, and for choosing the most effective disaster mitigation methods.

Benefit-cost studies themselves do not make decisions. They are a tool for digesting many diverse facts and for portraying implications of alternative strategies. The usefulness of such studies is related directly to the validity and completeness of the input. The efforts noted must be launched immediately to provide the input necessary for studies that should be made in the near future.

In initiating the recommended studies and programs, it is desirable that feasibility studies be made prior to commencing the major activity. The feasibility studies should ascertain the type of data to be collected and generated, and the use of the data. It is essential, however, that initial efforts using simple (and possibly crude) measures of the hazards to buildings be completed as soon as possible. These would be continually required as more detailed compilations and methodologies are undertaken and completed.

Kunreuther in his review article presents an extended discussion of the usefulness of benefit-cost studies, of perspectives that must be kept in mind, and of steps required to implement such studies.

RECOMMENDATION A22: TECHNOLOGY FOR DISASTER MITIGATION

THE FEDERAL GOVERNMENT SHOULD ESTABLISH A NATIONAL FOCAL POINT AND INFORMATION BANK TO PROVIDE STATE AND LOCAL PLANNING AND REGULATORY BODIES AND THE DESIGN PROFESSIONS WITH NEEDED INFORMATION RELATED TO DISASTER MITIGATION.

The national program would use multi-professional expertise to develop recommendations dealing with:

a. Standard approaches to establishing levels of risk for design.

b. Design concepts for disaster mitigation.

c. Coordination of disaster research and investigations.

d. The use of a referral system to data and information.

The bank should contain the latest information on disaster mitigation. Critical information would then be readily available to those engaged in planning for disasters. The bank should contain:

a. A referral system to data centers containing technical-physical data necessary for planning (e.g., geologic, hydrologic, seismic, meteorologic, etc.).

b. Information pertaining to the latest and most comprehensive ordinances and regulations.

c. Information necessary for land use planning.

Continued effort must be put forth in developing better decision making tools which inform the legislator and the citizen of risk alternatives so that decisions pertaining to disaster mitigation can be made by the appropriate level of government. The tools will help achieve a balance between investments providing safety from disaster and those providing other social benefits for the community.

RECOMMENDATION A23: SPECIAL PROFESSIONAL SKILLS

A NATIONAL FOCAL POINT SHOULD BE ESTABLISHED TO ASSIST THE STATE PROFESSIONAL LICENSING AUTHORITIES IN PREPARING AND MAINTAINING GUIDELINES DEFINING SPECIAL SKILLS FOR THE PROFESSIONS INVOLVED IN DISASTER MITIGATION DESIGN AND CONSTRUCTION PRACTICES. SUCH A FOCAL POINT SHOULD UTILIZE CONTINUING INPUT FROM TECHNICAL REVIEW BOARDS COMPOSED OF REPRESENTATIVES FROM THE RELATED PROFESSIONAL SOCIETIES.

It is interesting to note that, in some professional areas (e.g. medicine), there are standards for peer groups to evaluate the competence or ability of professionals to perform specialized services. It is essential that guides, prepared by a nationally recognized objective group with continuing input from peer professional groups, be available to the States for use in formulating standards for professional practices in areas where there is a potential for exposure to natural hazards.

Actions by the Insurance Industry

The increasing demands for negative incentives for disaster mitigation provided by Federal and local governments (through low interest loans and forgiveness clauses) can be curtailed through an effective nationwide disaster insurance program. Such a program can also stimulate improved building design and construction to mitigate future losses.

RECOMMENDATION A24: DISASTER INSURANCE PROGRAM

INSURANCE COMPANIES SHOULD ESTABLISH A NATIONWIDE DISASTER INSURANCE PROGRAM WHICH IS CONDUCIVE TO DISASTER MITIGATION. A POTENTIALLY VALUABLE FEATURE WOULD BE A DIFFERENTIAL RATE STRUCTURE WHICH RECOGNIZES DISASTER RISKS AND ENCOURAGES IMPROVEMENTS IN LAND USE PLANNING, DESIGN, CONSTRUCTION, MAINTENANCE, AND RESTORATION.

Risks of earthquakes, extreme winds and similar dynamic hazards are broadly distributed and annual losses are generally less than the fire loss. An insurance program is practicable and would assign costs to the potential beneficiaries. The appropriate Federal role as reinsurer or administrator merits consideration. Thus, it provides an incentive for improved practices for disaster mitigation in contrast to the negative incentives of disaster relief measures. Differential rate setting for fire insurance has been effective in mitigating fire losses by providing incentive for improvement of fire defenses, and should be similarly effective for natural disasters. Insurance payments for restoration or repair following a disaster should permit modifications which upgrade the building, thus mitigating future losses. Rates can reflect risks of extreme loads, and the qualities of: land use regulations, building codes, local enforcement practices, mitigating design features, and implementation in construction. The methodologies for probable loss studies, essential for rate setting, are feasible and may be developed with inputs from the inventories of existing buildings and benefit cost studies cited above.

Actions by the Design and Construction Community

Improvements in the design process, including the factors to be considered by the professional in planning and executing the design and the subsequent construction of the facilities in accordance with the design, can play a significant role in mitigating losses from natural disasters. It is essential that the natural hazards be recognized in the planning, design and construction processes.

RECOMMENDATION A25: PROFESSIONAL DESIGN TEAM

AGREEMENTS FOR PROFESSIONAL DESIGN SERVICES SHOULD CALL FOR AN INTERDISCIPLINARY DESIGN TEAM WITH THE EXPERTISE REQUIRED TO DEVELOP DESIGNS TO THE PERFORMANCE LEVELS NEEDED FOR DISASTER MITIGATION. THE SERVICES OF THE DESIGN TEAM SHOULD INCLUDE SURVEILLANCE OF CONSTRUCTION TO ASSURE IMPLEMENTATION OF THE PLANS AND SPECIFICATIONS AS WELL AS RECOGNITION OF UNFORESEEN CIRCUMSTANCES DISCLOSED DURING CONSTRUCTION. THE DESIGN TEAM SHOULD CERTIFY COMPLIANCE OF THE DESIGN AND CONSTRUCTION DOCUMENTS WITH APPLICABLE CODES AND REGULATIONS AND THAT ALL PROFESSIONAL WORK WAS AT LEAST IN ACCORD WITH STANDARD PRACTICES.

Design for mitigation of disaster hazards provides buildings which will perform properly in an extreme environment. The acceptance of finite risks of loss of function, property damage, and human suffering requires professional inputs from a number of disciplines to fit the various aspects of building design to its site and occupancy conditions. For instance, a major building or development in a seismic area requires an engineering geologist, foundation engineer, engineering seismologist, civil engineer, and a planner or architect for site evaluation and hazard identification. The design and surveillance of construction require an architect, structural engineer, mechanical engineer, electrical engineer, foundation engineer, inspector, and testing laboratory engineer. Each of these should have recognized authority in his area of the design and surveillance as indicated by his certification of the design and construction documents. The needs for these services and their complexity should be recognized in the agreement for remuneration for professional services. The effectiveness of surveillance of construction by the design team is well documented by successes of the Field Act in California and Chapter 70 of the Uniform Building Code. However, this surveillance should not lessen the contractor's responsibility for quality control and compliance with plans and specifications. The duties and their limitations of all members of the design and construction teams should be clearly understood by all parties. Certification of compliance by the contractor that all work was in accordance with the construction documents, and applicable codes and regulations should be required.

B. PRACTICES FOR DISASTER MITIGATION

Immediate and long range efforts by professional practitioners, standards writers, and researchers are required to provide an integrated development of building practices to mitigate the effects of natural disasters. The recommendations in this section are directed to these three groups. Professional practitioners, including land-use planners, architects, engineers and geologists, can implement immediately best current practices in present activities for design and analysis of new structures and the evaluation and strengthening of existing structures. Standards writers can effect improvements in practices by updating standards to the level of current knowledge and including new information as it becomes available from accepted research findings. Standards generating organizations, both private and public, will be major participants in this development work. Researchers can substantially improve hazard mitigation capabilities by providing new information to fill gaps in existing knowledge.

Actions by Professional Practitioners

Design loads used in current practice for extreme environments should reflect all usable information on factors affecting the response of the buildings.

RECOMMENDATION B1: SEISMICITY STUDIES

SEISMICITY STUDIES SHOULD BE CONDUCTED FOR IMPORTANT STRUCTURES LOCATED IN SEISMIC ZONES TO DETERMINE THE GROUND MOTIONS TO BE EXPECTED AND THE INFLUENCE OF LOCAL SOIL CONDITIONS ON THESE MOTIONS.

As pointed out in the review article by Donovan, both the general geographic region and the local site conditions should be considered in evaluating the effects of earthquakes on structures. Regional data on geologic conditions and the available seismic record may be used to estimate expected magnitudes and recurrence intervals. Local soil conditions may significantly influence wave propagation and surface motions.

RECOMMENDATION B2: MINIMUM DESIGN LOADS FOR EARTHQUAKES

MINIMUM DESIGN LOADS FOR EARTHQUAKES SHOULD BE BASED ON THE SEISMIC RISK MAP CONTAINED IN THE 1970 UNIFORM BUILDING CODE BUT WITH AREAS PRESENTLY DESIGNATED AS ZONE 0 CHANGED TO ZONE 1.

This recommendation should be considered as an interim measure for disaster mitigation since knowledge obtained from recent earthquakes can be used to update existing codes. Since historical earthquake data is limited and existing risk maps are not based on recurrence interval, it is reasonable, considering the uncertainties involved, to provide some protection for all structures and upgrade Zone 0 to Zone 1. In general, designs with adequate wind resistance will meet Zone 1 seismic requirements providing that major structural components are adequately interconnected, i.e., roof to walls and walls to foundation.

RECOMMENDATION B3: DESIGN WIND LOADS

DESIGN LOADS FOR WIND SHOULD BE BASED ON THE 1972 EDITION OF THE AMERICAN NATIONAL STANDARD A-58.1, "BUILDING CODE REQUIREMENTS FOR MINIMUM DESIGN LOADS IN BUILDINGS AND OTHER STRUCTURES."

This new standard incorporates a number of aspects of wind loading not included in other standards. For example, the use of alternate recurrence intervals (50 yr wind or 100 yr wind) in determining the wind loading permits selection of the risk accepted in the design. In addition, design loads are based on the velocity and duration of the wind (gust factors) as well as characteristics of the structure (shape factor) and site (site roughness effects). Maps indicating extreme wind characteristics permit consideration of forces produced by hurricanes and extra-tropical storms. Since these loadings account for recurrence interval, the traditional 33 percent increase in allowable stress should be used only where it is justified by the load-duration sensitivity of material resistance as it is with strength of timber elements. Although detailed wind speed data are not available and code provisions do not cover tornadoes, their effects should be considered for important structures in tornado prone areas. Until sufficient data are obtained to develop standards, considerable protection can be achieved using appropriate detailing procedures to insure that all elements of the structure are adequately interconnected. Details for seismic loading from the Uniform Building Code or wind detailing requirements from the South Florida Building Code may be used as guidelines.

It is important that procedures used in the analysis of structures subjected to natural hazards reflect the dynamic characteristics of the loading and system response.

RECOMMENDATION B4: DYNAMIC ANALYSIS

EARTHQUAKE AND EXTREME WIND RESPONSE ANALYSES FOR STRUCTURES OF UNUSUAL SHAPE OR STRUCTURAL CHARACTERISTICS SHOULD INCLUDE A DYNAMIC ANALYSIS.

Current practice as well as code provisions for both earthquakes and extreme winds are based on a quasi-static approach. In some cases, standards and codes identify structures requiring a more refined analysis. The ANSI Standard, for example, identifies wind sensitive structures for which gust action and dynamic response are important as buildings taller than 200 feet and having a height to least width ratio greater than 4. Until firm guidelines are established, designers should select structures in this category on the basis of familiarity with the assumptions and limitations of the quasi-static analyses used in current practice. Code provisions which would identify structures requiring dynamic analysis for earthquake design are discussed in the review article by Pinkham and specified in a recent change to the City of Los Angeles Building Code.

RECOMMENDATION B5: SOIL-STRUCTURE INTERACTION

INTERACTION OF THE SOIL-STRUCTURE SYSTEM SHOULD BE INCLUDED IN THE SEISMIC ANALYSIS OF IMPORTANT STRUCTURES.

Consideration should be given to the soil effects on the natural frequency and the damping of the soil-foundation-structure system as discussed in the review articles by Sharpe, Kost, and Lord, by Newmark and Hall, and in the cited references.

Consideration should be given in the planning stages and in design to the real nature of the potential hazards and their total effect on buildings.

RECOMMENDATION B6: STRENGTH, DUCTILITY, AND DAMPING

DESIGNS FOR EARTHQUAKES, EXTREME WINDS AND EXTREME LOCAL LOADS SHOULD CONSIDER THE DYNAMIC RESPONSE PARAMETERS OF THE STRUCTURE AND THE NEED FOR PROVIDING ENERGY ABSORPTION CAPABILITIES AS WELL AS ADEQUATE STRENGTH. COMMENTARIES FOR CURRENT STANDARDS AND SPECIFICATIONS SHOULD BE USED TO INSURE PROPER INTERPRETATION AND USE OF DESIGN CRITERIA IN THIS REGARD.

Although current practice involves the use of equivalent static loads, the response of structures to earthquakes and extreme winds is related to the dynamic characteristics of the individual components and the overall system. Material properties under dynamic loads, repeated excursions into the inelastic range and large amplitude displacements are involved. Due consideration should be given to those factors not usually considered in designs for dead and live load. Particular attention should be given to the structural configuration to insure provision of adequate stiffness, ductility, and continuity. Failures encountered in previous natural disasters and progressive collapses indicate the importance of connection details. Designs for connections should insure adequate resistance to the induced forces and transferal to all load carrying members throughout the structure.

RECOMMENDATION B7: MINIMUM DETAILS

DESIGN CRITERIA FOR STRUCTURES FOR WHICH PROFESSIONAL DESIGN ATTENTION IS NOT REQUIRED SHOULD INCLUDE STANDARD MINIMUM DETAILS SELECTED ON THE BASIS OF THEIR PROVEN ABILITY TO WITHSTAND EXTREME LOADS.

The type of damage occurring in non-engineered structures is discussed by Sharpe, Kost and Lord. Surveys following natural disasters, for example inspections of damage done by Hurricane Celia in 1970, indicate that a major portion of the damage could have been prevented had proven structural details been used in the affected buildings. When a detailed analysis is not economical, minimization of life loss and property damage must be achieved by eliminating potentially hazardous conditions, providing anchorage of various components and adequate connection details. Anchorage requirements in the Uniform Building Code and the South Florida Building Codes should be employed. These requirements should be reviewed using the new knowledge obtained from analysis of the damage to dwellings caused by the 1971 San Fernando earthquake. Recommended framing and details for wind damage mitigation in wood frame houses are also available in Research Paper FPL 33 of the U.S. Department of Agriculture.

RECOMMENDATION B8: HAZARDS FROM ADJACENT SITES

LAND-USE PLANNERS AND BUILDING DESIGNERS SHOULD CONSIDER THE HAZARDS ARISING FROM DAMAGE ON THE SITE OR ADJACENT SITES IN A NATURAL DISASTER.

Failures of dams, standpipes, gas lines, other "life-lines," etc. can produce direct hazards or losses of vital services. Debris from adjacent weaker buildings can be dangerous in earthquakes or winds. Both secondary hazards and loss of services involving disruption of life support systems must be considered for the site involved.

RECOMMENDATION B9: HAZARDS DURING CONSTRUCTION

THE EFFECTS OF WIND AND EARTHQUAKE LOADING ON PARTIALLY COMPLETED STRUCTURES SHOULD BE CONSIDERED IN ERECTION PROCEDURES.

Although the life safety hazard usually is low, probably the greatest incidence of wind induced failures occurs in partially completed structures. For both wind and earthquake, such failures provide a significant hazard to adjacent structures. The response of these structures to wind effects should be established and provisions made for adequate bracing to prevent failure. Seismic loading during construction also should be given more attention.

The life safety hazard and dollar damage associated with failures of building furnishings and equipment have only recently been evaluated. Considerable mitigation of disaster effects can be achieved using current information.

RECOMMENDATION B10: DESIGN OF FURNISHINGS AND EQUIPMENT FOR DISASTER CONDITIONS

THE DESIGN OF MECHANICAL AND ELECTRICAL EQUIPMENT AND OFFICE FURNISHINGS TO WITHSTAND EARTHQUAKE AND EXTREME WIND LOADS SHOULD BE BASED ON PROVIDING ADEQUATE BRACING AND CONNECTION TO THE SUPPORTING STRUCTURE. THE LOCATION OF SUCH EQUIPMENT WITHIN THE STRUCTURE SHOULD BE SELECTED TO MINIMIZE THE INDUCED FORCES AND CONSEQUENCES OF FAILURE.

Many piping, equipment, and elevator failures result from failure of their supports and connections. Conflicting requirements for vibration isolation and adequate tie down provisions are discussed in the review article by Ayres and Sun. Possible approaches to equipment analysis and design are also presented by Sharpe, Kost, and Lord. Several solutions are given based on a consideration of the dynamic nature of the forces produced in these elements. Tendencies for damage can be reduced by properly locating equipment within the structure, such as avoiding heavy equipment on top floors of tall buildings. Design standards for elevators and their supporting structures must be developed to mitigate loss of these vital lifelines.

Consideration should be given to special hazards related to the non-permanent nature of housing units such as mobile homes.

RECOMMENDATION B11: TIE DOWNS FOR MOBILE HOMES

MOBILE HOMES SHOULD BE TIED DOWN TO RESIST EARTHQUAKE AND WIND LOADINGS. TIE DOWNS TO RESIST WIND LOADING SHOULD BE PROVIDED IN ACCORDANCE WITH REQUIREMENTS INCLUDED IN THE DCPA PUBLICATION TR 75 "PROTECTING MOBILE HOMES FROM HIGH WINDS."

Mobile homes and certain other non-permanent structures are subjected to displacement from strong wind conditions and earthquakes. Design procedures and criteria should provide for adequate support, anchorage and stability of these units. While mobile home construction standards have been developed and disseminated through ANSI A-119.1, they appear neither to have been accepted nor implemented by the industry at large nor do they seem adequate for hurricane wind frequently encountered in many sections of the United States. The ANSI A-119.1 should be modified to include the 1972 provisions on wind loading of ANSI A-58.1. Tie down requirements for wind loading included in the DCPA publication appear to be the best practical procedure which can be implemented at this time for existing units. These provisions may be used as guidelines to develop requirements for earthquake loadings.

Actions by Standards Writers

Design loadings giving a better measure of disaster hazards should be developed by standards writing bodies on the basis of up to date research knowledge.

RECOMMENDATION B12: UPDATING SEISMIC CODES

MODEL CODE PROVISIONS AND COMMENTARY FOR SEISMIC DESIGN SHOULD BE PREPARED ON A TOP PRIORITY BASIS TO BRING THE MINIMUM LEVEL OF PRACTICE INTO LINE WITH CURRENT STATE OF KNOWLEDGE AND ANALYTICAL TECHNIQUES. THE OBJECTIVES OF THE MODEL CODE PROVISIONS SHOULD BE TO MINIMIZE LOSS OF LIFE AND PROPERTY DAMAGE AND TO MAINTAIN VITAL FUNCTIONS. THE CODE PROVISIONS SHOULD BE WRITTEN SO THAT THEY CAN BE APPLIED TO ALL AREAS OF THE UNITED STATES.

The last major revision in seismic code provisions occurred in the early 1960's. Since that time there have been significant advances in knowledge of the response of building systems to seismic ground motion and correspondingly in analytical techniques and design procedures.

The development of the model code provisions should be undertaken by a small qualified group with the guidance and advice of a broad-based professional group. During the development of the code provisions, major interim recommendations should arise which can be incorporated in current practice.

RECOMMENDATION B13: RECURRENCE INTERVAL

DESIGN LOADING INTENSITIES SHOULD BE SPECIFICALLY RELATED TO RECURRENCE INTERVAL.

Consistent treatment of loadings occurring singly or in combination, requires consideration of their recurrence interval. Although improvements in scientific knowledge may lead to future modifications in probability-based risk maps, the best available information should be presented to designers in formats suitable for use in decision making. Available evidence suggests that tornado wind forces are feasible to design for, with the possible exception of winds near the funnels of the largest tornadoes.

RECOMMENDATION B14: SITE EFFECTS

RELIABLE STANDARD APPROACHES SHOULD BE DEVELOPED TO ACCOUNT FOR LOCAL SITE EFFECTS ON THE INTENSITIES OF EARTHQUAKES AND WIND LOADINGS.

The forces induced by earthquake ground shaking have been observed to be influenced by the natural frequencies of vibration of the building and surficial soil at the site. Donovan suggests approaches to such standards in his review article. Wind loadings in ANSI A58.1 1972 account for site roughness effects but there is need to better define shielding and channeling effects of topography and adjacent buildings. Power spectra for turbulence need better definition for boundary layers representative of open country and urban conditions, and for storm conditions including tornadoes, hurricanes, thunderstorms, and extra-tropical storms.

RECOMMENDATION B15: LOADING DURATION

LOADING STANDARDS FOR DYNAMIC LOADS SUCH AS EARTHQUAKES, WINDS, AND EXTREME LOCAL LOADS SHOULD DEFINE LOADING DURATION.

Gust factors in the ANSI A58.1 standard relate design wind intensity to the gust duration to which an element is sensitive. Improved statistical treatment of wind velocity as a function of duration would use a base averaging period of 10 to 30 minutes in place of the current fastest mile of wind. The damage potential of an earthquake depends markedly on the duration of strong shaking; expressions of duration should accompany measures of loading intensity such as acceleration, spectral velocity, and spectral displacement.

RECOMMENDATION B16: ELEMENT CHARACTERISTICS

STANDARD PROCEDURES SHOULD BE DEVELOPED TO RELATE DESIGN LOADINGS TO THE CHARACTERISTICS OF THE LOADED ELEMENT.

Research has shown that wind forces on individual elements such as roof panels are markedly different from the forces affecting the main lateral force system of the building. Similarly, earthquake-induced forces on mechanical and electrical equipment and furnishings differ substantially from those induced in the lateral force system of the building. Potential payoffs are large because these elements are responsible for large parts of both building costs and disaster losses.

RECOMMENDATION B17: DEBRIS LOADS

STANDARDS FOR DEBRIS LOADINGS SHOULD BE MADE AVAILABLE TO DESIGNERS.

Standard procedures are needed to define debris loading as a function of location and site conditions. Impacts of airborne debris are responsible for much of the wind damage to building exteriors. Debris loading may be a major factor in progressive collapse. Waterborne debris from storm surges or flooding may produce impact damage or increased drag loading by impeding flow of water.

RECOMMENDATION B18: FIRE HAZARDS

FIRE HAZARDS ARISING FROM WIND STORM AND EARTHQUAKE DAMAGE SHOULD BE DEFINED.

Substantial property damage is considered acceptable under extreme earthquake and wind loadings (recurrence intervals exceeding about 100 years). The philosophy is to accept property loss but prevent structural collapse leading to severe hazards to life. However, damage to services and furnishings tend to increase fire hazards; risks of life loss in fire must be defined as a function of damage to structure services and furnishings and the height of the structure to allow consistent life safety provisions (it is easier to evacuate a one-story house than a sixty-story office building). Major fires following earthquakes must be considered a real possibility.

Analytical models and procedures are used in design to predict the response of buildings to extreme loads. Standards are needed to guide designers and regulatory officials in the selection and use of analytical techniques.

RECOMMENDATION B19: ANALYTICAL MODELLING

STANDARD PRACTICES SHOULD BE DEFINED FOR ANALYTICAL MODELLING OF THE DYNAMIC CHARACTERISTICS OF BUILDINGS AND FOR ANALYTICAL PREDICTION OF BUILDING RESPONSE TO EARTHQUAKES AND EXTREME WINDS. ANALYSES, WHETHER INVOLVING EQUIVALENT STATIC FORCES OR DYNAMIC FORCES, SHOULD ACCOUNT FOR LATERAL, VERTICAL AND TORSIONAL COMPONENTS OF LOAD AS WELL AS STRUCTURE-FOUNDATION-SOIL INTERACTION, DAMPING AS A FUNCTION OF EXPECTED DEFORMATIONS, PARTICIPATION OF NONSTRUCTURAL ELEMENTS, OVERTURNING AND INSTABILITY EFFECTS.

Analytical methods are available for prediction of dynamic structural response including effects of soil-structure interaction and repeated and reversed inelastic deformations. However, designers need guidance on reliable approaches which account for mass, stiffness, damping, cracking, reversed and repeated inelastic deformation, the gravity loads working through lateral deflections, and soil-structure interactions. Results of laboratory model and field test experience can confirm analytical models. Parameter studies can verify the range of applicability of approximations in analysis. The torsional effects of wind and earthquake loadings and the cross-wind response of tall, slender buildings merit fuller investigation. The changes in stiffness and resistance which result from repeated and reversed inelastic deformation should be accounted for in analyzing earthquake effects on unsymmetrical or unconventional buildings.

RECOMMENDATION B20: MODEL STUDIES

STANDARD PROCEDURES SHOULD BE DEVELOPED FOR MODEL STUDIES OF THE RESPONSE OF BUILDINGS TO DYNAMIC LOADS.

Model tests in wind tunnels are widely used in design of important buildings for wind effects. However, there is great need for calibrating different wind tunnel test procedures through tests of a standard model and by comparison with full-scale effects. Since the turbulent wind structure appears to be dependent on wind velocity, the full-scale test must measure wind forces generated by the extreme design wind storm. Model tests under simulated earthquake loadings are beginning as shaking tables become available. Requirements for standardized earthquake model test procedures, derived by comparison to full-scale results, exist for tests of both structural components and elements of the furnishings, mechanical, and electrical systems.

Planning and design criteria provide the designer and regulatory official with minimum standards for the design loadings, analytical procedures, and the strength and ductility to be provided in the design. Improvements giving more explicit indication of required performance and technically better standards promote mitigation of disaster losses and also economies in initial cost where current standards are unduly restrictive.

RECOMMENDATION B21: EXPLICIT LEVELS OF PERFORMANCE

PLANNING AND DESIGN CRITERIA SHOULD STATE EXPLICIT LEVELS OF DESIRED PERFORMANCE IN TERMS OF LIFE SAFETY, PROTECTION OF PROPERTY, AND MAINTENANCE OF VITAL FUNCTIONS. PRESCRIPTIVE CRITERIA, CALIBRATED TO PROVIDE THIS PERFORMANCE, SHOULD BE GIVEN FOR ORDINARY TYPES AND MATERIALS OF DESIGNED CONSTRUCTION. STANDARD DETAILS, ASSURING THE REQUIRED LEVELS OF PERFORMANCE, SHOULD BE PROVIDED FOR SMALL BUILDINGS FOR WHICH PROFESSIONAL DESIGN ATTENTION IS NOT REQUIRED.

In the interest of simplifying the legal provisions of building codes, achieving applicability in all areas of the United States, and at the same time providing for acceptance of improved practices, efforts should be made to move in the direction of performance criteria for the strength and serviceability of buildings under earthquake and wind loads. Performance criteria can be formulated in terms of the recurrence interval of loading and reliability of resistance (including stiffness, strength, and ductility) desired for each limit state or class of limit states. Recurrence interval of loading and reliability should be consistent with the consequences of failure, that is, be greater for limit states involving hazard to life than for those causing property loss or interruption of function. It is recognized that substantial effort is required to define acceptable levels of performance by direct consideration of total life cycle costs including social factors, or by calibration with accepted good practice.

Prescriptive design criteria will continue to be used for well documented types and materials of construction. However, as performance calibration of prescriptive design criteria become better defined, codes will not need to express these detailed design standards.

Acceptable details will continue to be required for some one and two story buildings which can be accepted safely and economically without professional participation in design.

There is a national need for a new comprehensive lateral force code. Present code-referenced standards using equivalent static loadings and alternative approaches using some form of dynamic analysis should be reviewed and evaluated. The more effective approaches for widely used types of structures and materials of construction should be developed into design standards suitable for code reference.

RECOMMENDATION B22: LOAD FACTORS

CONSISTENT LOAD FACTORS SHOULD BE ESTABLISHED FOR THE COMBINED CONDITIONS OF LOADING TO BE CONSIDERED IN DESIGN, AND THESE SHOULD BE APPLIED UNIFORMLY TO ALL TYPES AND MATERIALS OF CONSTRUCTION. VARIABILITY FACTORS PROVIDING CONSISTENT RELIABILITY IN STRENGTH AND SERVICEABILITY SHOULD BE ESTABLISHED FOR THE RESISTANCE OF VARIOUS MATERIALS, COMBINATIONS OF MATERIALS, AND TYPES OF CONSTRUCTION.

Load factors for combinations of dead, live, snow, wind, earthquake, etc. loads should reflect consistently the probability of occurrence of the combination. For instance, all factored load combinations for strength might define an intensity of loading with one specific recurrence interval. The factors expressing variability of resistance should account consistently for the uncertainties in resistance associated with the various types of construction and materials. It is recognized that this recommendation requires modification of existing specifications, developed independently for different materials, which have variously adjusted local factors and variability factors to achieve the desired overall reliability. However, the recommended uniformity provides simplicity in code language, eases the acceptance of improved practice, and makes the level of risk in specific designs more evident to decision makers.

RECOMMENDATION B23: RELIABILITY OF DYNAMIC ANALYSES

THE USE OF DYNAMIC MATHEMATICAL OR MODEL ANALYSES IN DESIGN FOR EARTHQUAKE AND WIND LOADS SHOULD BE ENCOURAGED BY ALLOWING ECONOMIES IN PROPORTIONING CONSISTENT WITH THE INCREASED RELIABILITY OF THE DESIGN. GUIDELINES SHOULD BE DEVELOPED FOR REQUIRING DYNAMIC ANALYSES FOR THOSE BUILDINGS WITH HIGH IMPORTANCE, OF UNCONVENTIONAL TYPE, WITH HIGHLY IRREGULAR SHAPES OR DISTRIBUTIONS OF RESISTANCE AND STIFFNESS, OR WITH FUNDAMENTAL NATURAL PERIODS IN EXCESS OF A LIMIT ON THE ORDER OF ONE SECOND.

One of a variety of equivalent static or dynamic methods of analysis may be used in the design of a particular building provided that its idealizations are dealt with consistently throughout the design process. Codes can credit more rigorous analytical procedures by reflecting the greater reliability of the design in the factor of safety required. A careful review for each type of building and material is needed to define conditions where dynamic analyses should be required.

RECOMMENDATION B24: DRIFT LIMITATIONS

LIMITATIONS ON STORY DRIFT SHOULD BE ESTABLISHED TO REDUCE TO ACCEPTABLE LEVELS THE HAZARDS TO PROPERTY DAMAGE AND STRUCTURAL COLLAPSE.

Structural response, which may be damaging to the structural and non-structural elements or may threaten collapse of the structure, may be controlled by drift limitations appropriate to the type of building, material, and level of response. Avoidance of human discomfort, which would also require consideration of dynamic properties in expression of drift limitations, is not considered an objective under disaster conditions. Drift limitations to avoid damage to building elements would consider elastic response and a load recurrence

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interval for a moderate earthquake or moderately extreme wind. Drift limitations to avoid collapse would account for inelastic behavior including available ductility and the destabilizing influence of vertical loads acting through lateral deflections (P-delta effect) for a loading recurrence interval corresponding to a major earthquake or extreme wind.

RECOMMENDATION B25: MANUALS OF PRACTICE

MANUALS OF PRACTICE FOR BUILDING DESIGN PROFESSIONALS SHOULD BE PREPARED TO DEMONSTRATE APPROACHES TO MEETING PERFORMANCE CRITERIA AND PRESCRIPTIVE REQUIREMENTS FOR WIDELY USED TYPES OF BUILDINGS AND MATERIALS. MANUALS OF ACCEPTABLE DETAILS AND PRACTICES SHOULD BE PREPARED FOR BUILDINGS WHICH DO NOT RECEIVE PROFESSIONAL ATTENTION IN DESIGN.

Rational consideration of dynamic hazards such as earthquakes and extreme winds requires building designers to work with concepts of dynamic response and ultimate range inelastic behavior which are presently familiar to only a minority of the profession. Manuals of practice will substantially aid designers and regulatory officials in applying improved building practices. Manuals of acceptable details and practices, for small buildings with low functional importance which may safely and economically be constructed without professional inputs, should demonstrate and explain the building qualities required for successful performance and define the limitations of their applicability. These manuals must include a commentary explaining the philosophy and goals used in their preparation to insure they do not become counterproductive "cookbooks."

RECOMMENDATION B26: CONTINUING EDUCATION

CONTINUING EDUCATION PROGRAMS SHOULD BE DEVELOPED AND IMPLEMENTED FOR DESIGN PROFESSIONALS, REGULATORY OFFICIALS, AND BUILDERS TO TRANSMIT THE KNOWLEDGE AND SKILLS REQUIRED TO IMPLEMENT IMPROVED BUILDING PRACTICES FOR HAZARD MITIGATION.

New concepts, information and techniques related to knowledge of extreme loads, dynamic and inelastic structural behavior, and reliability and performance-based design approaches can be transmitted effectively to design professionals through continuing education programs taught by knowledgeable design professionals and professional educators. In this way, the improved practices also will flow automatically into professional curricula. Comparable programs, using knowledgeable educators with practical experience from technology programs of junior colleges, can train draftsmen, inspectors and building tradesmen in improved practices for detailed design and construction.

Development of improved practices for survey and evaluation of existing buildings can provide effective mitigation of losses from future natural disasters and aid recovery from the ravages of disasters.

RECOMMENDATION B27: METHODOLOGIES FOR BUILDING SURVEYS

IMPROVED METHODOLOGIES SHOULD BE DEVELOPED FOR ASSESSING THE SAFETY OF EXISTING BUILDINGS.

ACCOUNT SHOULD BE GIVEN TO HAZARDS OF POTENTIAL EARTHQUAKES, EXTREME WINDS AND OTHER DYNAMIC HAZARDS AS WELL AS THE EFFECTS OF DAMAGES PRODUCED BY PREVIOUS EXTREME LOADS OR GRADUAL DETERIORATION.

There presently are no nationally recognized effective, systematic and economical procedures for assessing the hazards of existing buildings for future extreme loads. Present design codes are not directly indicative of the hazard represented by non-conforming existing buildings. Buildings which do not conform to codes may either still meet the performance level intended by the code or may be judged to represent acceptable risks when the very high costs of abating hazards in existing buildings is considered. Appropriate evaluation methods should be developed for systematic predisaster surveys of safety for long-term use, and for emergency surveys immediately following a disaster for safety in emergency use. Evaluation procedures should receive broad professional consensus. Criteria for acceptance or abatement of hazards should be capable of reflecting the responsible authorities' assessment of the social and economic consequences of action.

RECOMMENDATION B28: MANUAL FOR STRENGTHENING AND REPAIR

A MANUAL OF PRACTICE SHOULD BE DEVELOPED FOR THE STRENGTHENING OF EXISTING BUILDINGS AND REPAIR OF DAMAGED BUILDINGS. LABORATORY AND FIELD TESTS SHOULD BE CONDUCTED TO DOCUMENT THE EFFICACY OF THE PRACTICES.

The documentation and professional discussion of strengthening and repair procedures has been extremely limited. These procedures are usually expensive to carry out and inadequate measures may have severe consequencies. Thus benefits can arise from more effective procedures and reduced costs for strengthening or repair.

RECOMMENDATION B29: DATA GATHERING PROCEDURES

STANDARD DATA GATHERING PROCEDURES SHOULD BE DEVELOPED AND IMPLEMENTED FOR BOTH PRE-DISASTER AND POST-DISASTER SURVEYS.

The development of rational strategies for disaster mitigation and relief requires a nationwide inventory of disaster hazards. Efficient standard procedures for data acquisition are an essential step in this effort. Post-disaster damage surveys can be efficient and effective for the many organizations needing damage information only if survey procedures are developed in advance, personnel are trained in their use, and materials needed by survey teams are stockpiled.

The same technical measures applicable to building structures are involved in mitigation of losses of life, property, and function from failures of furnishings and services. However, major recommendations are separately stated here to bring them to the attention of the designers and authorities responsible for these systems.

RECOMMENDATION B30: FORCES ON EQUIPMENT

IMPROVED METHODS SHOULD BE DEVELOPED FOR DEFINING THE LATERAL, VERTICAL, AND TORSIONAL FORCES OR DISPLACEMENTS TO WHICH EQUIPMENT IS SUBJECTED IN EARTHQUAKES OR EXTREME WINDS.

When forces are defined, the level of resistance required for the desired reliability can be determined by the procedures used in structural design. Special attention to equipment forces or displacements is required because the mass and flexibility of an item of equipment can cause its response to differ from that of the portion of the building to which it is attached. Equivalent static forces or displacements can sometimes be used for conventional equipment in conventional buildings. Dynamic analysis methods will be needed in other situations as indicated in the review article by Sharpe, Kost, and Lord.

RECOMMENDATION B31: MANUALS FOR EQUIPMENT

MANUALS OF PRACTICE SHOULD BE DEVELOPED FOR SUPPORT, BRACING AND JOINTING DETAILS AFFECTING THE SAFETY OF ARCHITECTURAL, MECHANICAL, AND ELECTRICAL ELEMENTS AND FURNISHINGS OF BUILDINGS IN NATURAL DISASTERS. EDUCATIONAL PROGRAMS EMPLOYING THESE MANUALS SHOULD BE PRESENTED TO THE PROFESSIONS AND TRADES CONCERNED WITH THE DESIGN AND INSTALLATION OF THESE ELEMENTS.

The life hazards and direct and secondary costs of damage to nonstructural systems in natural disasters make these measures as important as the similar ones recommended for the structural system. Shake table tests should be required for details to assure improved performance.

RECOMMENDATION B32: CRITERIA FOR UTILITIES AND INDUSTRIES

CODES AND STANDARDS AUTHORITIES RESPONSIBLE FOR UTILITIES AND INDUSTRIAL FACILITIES SHOULD DEVELOP AND IMPLEMENT CRITERIA ASSURING THE PUBLIC SAFETY IN THE EVENT OF NATURAL DISASTER.

Building codes and standards often do not apply to utility and industrial facilities where failure could cause substantial public losses of life, property, and normal functions. Examples are water storage in dams and tanks, chemicals such as ammonia and chlorine, petroleum products, and electrical power transmission. Codes and standards should be developed to provide consistent public protection from earthquake and extreme wind failures of new facilities. Owners and regulators of existing facilities should review hazards from existing facilities and take appropriate action in abatement.

Actions by Researchers

Additional knowledge of the environments which threaten disaster is needed for more effective building practices. Continued research efforts can substantially improve capabilities to mitigate the hazards to buildings.

RECOMMENDATION B33: EARTHQUAKE RISK MAPS

SUBSTANTIAL SEISMOLOGICAL RESEARCH ACTIVITIES SHOULD BE FOCUSED ON SCIENTIFIC BASES FOR REGIONAL EARTHQUAKE RISK MAPS.

These maps should describe the motions on firm ground as a function of recurrence interval, in a manner which permits geologists and engineers to evaluate the hazards of failures of surficial soil deposits, buildings and other manmade facilities. Studies of the types of focal mechanisms, the statistical nature of earthquake occurrences, and the propagation of strong motions in rock will provide fundamental knowledge for use in risk mapping. Field and theoretical studies of active and potentially active faults related to the location, extent, type, mechanics and regional tectonic patterns should be expanded. Historical studies of seismicity in all areas of the U.S. and in seismologically similar areas abroad will increase the data base and, consequently, the reliability of risk assessments. The deployment of seismological instruments throughout the U.S. should be reviewed and augmented to assure adequate identification of the occurrence of small magnitude earthquakes (to aid in prediction of the recurrence interval of large earthquakes). The deployment of strong motion instruments should be reviewed and augmented to assure adequate collection of information with consideration of differences and uncertainties in anticipated ground motions in various regions.

RECOMMENDATION B34: SITE EFFECTS IN EARTHQUAKES

SUBSTANTIAL GEOLOGICAL RESEARCH ACTIVITIES SHOULD BE FOCUSED ON THE MECHANISMS BY WHICH SURFICIAL DEPOSITS AFFECT THE INTENSITY AND PHASING OF EARTHQUAKE GROUND MOTIONS, AS WELL AS ON THE PROBABILITIES OF SURFACE FAULTING, SUBSIDENCE, LIQUIFACTION, AND SLIDING.

Direct evidence of the effects of overburden on earthquake motions can be obtained by placement of two and three dimensional arrays of strong motion instruments in appropriate geologic and geographic areas. Material properties of the soil and rock profiles should be measured at all strong-motion accelerograph stations (unless this information is needed to select the siting of the station or would seriously delay utilization of the recorded data, it can be collected after the occurrence of a significant motion). A program of balanced installation costs and geotechnical investigation costs should be developed to increase the number and widen the geographical distribution of these stations.

RECOMMENDATION B35: INSTRUMENTED BUILDINGS

THE NETWORK OF INSTRUMENTED BUILDINGS SHOULD BE REVIEWED AND UPDATED TO ASSURE ACQUISITION OF MOST NEEDED DATA.

Gaps in the capability for collecting and evaluating data on the response of buildings to earthquakes can be minimized by tabulating building locations, types of buildings and foundation, surficial geology, soil conditions and detailed instrument locations for all buildings containing strong motion accelerographs. Installations of time-history and peak reading instruments should be considered for buildings exposed to strong winds. However, no building should be instrumented unless the construction drawings of the building can be included in the installation data file.

RECOMMENDATION B36: NEAR SURFACE METEOROLOGY

MICROMETEOROLOGICAL STUDIES SHOULD BE CONDUCTED TO DEFINE THE NEAR SURFACE CHARACTERISTICS OF WINDS IN HURRICANES, THUNDERSTORMS, EXTRA-TROPICAL STORMS AND TORNADOES.

Existing data on the temporal and spatial variations of wind speed with height and average wind velocity should be evaluated to define the additional information needed to predict wind loads on buildings and building elements. Research should be implemented to fill gaps in the knowledge.

RECOMMENDATION B37: CLIMATOLOGICAL ATLAS

CLIMATOLOGICAL DATA PERTINENT TO THE NEEDS OF THE BUILDING COMMUNITY SHOULD BE IDENTIFIED AND MADE AVAILABLE IN A CLIMATOLOGICAL ATLAS.

The atlas should account for localized effects of terrain features, principal wind directions, etc. Wind environments related to serviceability (comfort around buildings in normal weather, heating and cooling loads, etc.) may be covered as well as hazards to life and property. Data collection programs should be instituted to collect unavailable significant data.

Improved analytical methods are needed both as research tools to improve understanding of the behavior of buildings under extreme loads, and as aids to designers. The former class would be quite rigorous and complex, the latter may be simpler and more efficient than current procedures.

RECOMMENDATION B38: ULTIMATE DYNAMIC RESPONSE

ANALYTICAL PROCEDURES SHOULD BE DEVELOPED TO PREDICT IN DETAIL THE BEHAVIOR TO COLLAPSE OF BUILDINGS UNDER DYNAMIC LOADS.

Salient factors include the dynamic and stochastic nature of the loadings; three-dimensional interaction of lateral, vertical and torsional deformations in diaphragms, shear walls, frames and nonstructural elements; and repeated or cyclic inelastic deformations with degradation in stiffness and resistance. All these features are unlikely to be efficiently represented in any one computer program for all types of buildings. However, the parameter studies required for improvement of design practices for any type of building will require consideration of all or most of these effects.

RECOMMENDATION B39: SIMPLIFIED ANALYSES FOR DESIGN

SIMPLIFIED ANALYTICAL PROCEDURES SHOULD BE DEVELOPED FOR CODES AND GUIDES TO DESIGN.

The simplified procedures would be specific to limited classes of loadings, buildings and materials. The range of validity should be studied carefully and defined for users. These procedures should reflect the dynamic lateral, vertical, torsional and overturning motions, and the amount of damping and ductility available. Simplified procedures, rationally focused on salient characteristics of structural behavior, can provide more insight for design judgments than more rigorous complex and abstract methods.

The development of economical practices for buildings which will be safe under extreme loads requires understanding the structural behavior (the way forces and deformations distribute through the structure and the conditions and mechanisms by which it may fail). Understanding of behavior and confirmation of design concepts can come about only through experience. Although structural tests are expensive, experience is achieved at substantially lower social and economic cost if it occurs through deliberate testing rather than by failures of buildings in natural disasters.

RECOMMENDATION B40: LABORATORY AND FIELD EXPERIMENTS

LABORATORY AND FULL-SCALE FIELD EXPERIMENTS SHOULD BE CONDUCTED TO INVESTIGATE ULTIMATE STRUCTURAL BEHAVIOR UNDER LOADING CONDITIONS REPRESENTATIVE OF EARTHQUAKES, EXTREME WINDS AND SIMILAR HAZARDS. THESE TESTS SHOULD INVESTIGATE ANALYSIS, DESIGN, STRENGTHENING AND REPAIR PRACTICES AND DOCUMENT ABILITIES TO PREDICT OR ACHIEVE SPECIFIC LEVELS OF STRUCTURAL SAFETY.

A substantial part of the experimental effort would involve small specimens, for instance to study loading rate effects on unit strength properties, and static testing, for instance, to study effects of cyclic inelastic deformations on the resistance function. Also, there is great need for dynamic large-scale testing to confirm or to develop understanding of the dynamic response of buildings. Cooperative efforts should be explored with the Atomic Energy Commission to determine whether studies of structural behavior, soil-structure interaction, and soil amplification and attenuation effects can be conducted at the Nevada Test Site in conjunction with the underground nuclear test program. A national earthquake engineering experimentation facility should be established for testing large structures (about 100 ft. square in plan) to failure under simulated earthquake motions with three degrees of freedom.

RECOMMENDATION B41: FIELD AND WIND TUNNEL TESTS

LONG-TERM MEASUREMENTS SHOULD BE CONDUCTED ON ACTUAL BUILDINGS IN STRONG WINDS TO ESTABLISH DISTRIBUTIONS OF EXTREME LOADS, EFFECTS OF REPEATED LOADS, AND DAMPING PROPERTIES. COORDINATED WIND TUNNEL STUDIES SHOULD BE CONDUCTED TO DEVELOP MODELLING TECHNIQUES FOR TURBULENT FLOW CONDITIONS AND AEROELASTIC INTERACTIONS.

The field measurement program should emphasize minimal instrumentation of many structures, rather than elaborate and costly instrumentation of a few, to achieve strong response information at reasonable cost. Measured dynamic response would be statistically correlated with wind speed and pressure data and results of dynamic analyses. Wind tunnel modelling techniques would be critically reviewed through correlations with field measurements to define modelling procedures reliable for design.

RECOMMENDATION B42: ROCK AND SOIL MATERIALS

LABORATORY AND FIELD STUDIES OF THE BEHAVIOR OF ROCK AND SOIL MATERIALS UNDER LARGE DYNAMIC DEFORMATIONS SHOULD BE CONTINUED TO ASSIST IN

DETERMINATION OF THE RELATIONSHIP BETWEEN BEDROCK AND SURFACE MOTIONS IN EARTHQUAKES.

The non-linear behavior of soils and rocks causes the bedrock to surface relationships for strong motions to differ from those measured in the field for low intensity explosive or seismic excitations. The non-homogeneity of in-situ soils and rocks leads to substantial differences from the relationships that would be derived from dynamic laboratory tests of small samples. A thorough investigation is needed to develop and confirm effective and economical approaches to predicting effects of surficial deposits on earthquake ground motions.

Analytical, experimental, and field research studies are needed to provide knowledge for improved design criteria.

RECOMMENDATION B43: MECHANISMS OF FAILURE

LABORATORY STUDIES OF THE MECHANISMS FOR FAILURE OF STRUCTURAL ELEMENTS AND CONNECTIONS, UNDER CYCLIC OR REPEATED INELASTIC DEFORMATIONS REPRESENTATIVE OF THE EFFECTS OF SEVERE DYNAMIC LOADS, SHOULD BE CONDUCTED TO DEFINE STIFFNESS, STRENGTH, DAMPING, DUCTILITY, AND SERVICE LIFE.

The development of design provisions which assure intended performance of structural elements and connections is impossible without detailed understanding of their behavior. Current information consists only of a few pilot studies under cyclic inelastic deformations, some information on loading rate and strain history effects for coupons under simple states of stress, and substantial, but incomplete, information for structural elements and connections under monotonic loading.

RECOMMENDATION B44: DAMPING

INFORMATION ON DAMPING IN BUILDING STRUCTURES SHOULD BE ASSEMBLED FROM ALL AVAILABLE FOREIGN AND DOMESTIC SOURCES, ANALYZED, AND AN EXPERIMENTAL PROGRAM INITIATED TO FILL IMPORTANT GAPS.

Damping is a vital parameter governing structural response to dynamic loads in both serviceability and ultimate ranges. The amplitude of the motions under which damping was measured must be known to define whether the results pertain to service or ultimate behavior. The prior loading history of the structure in which the damping was measured must be known to determine effects of accumulated damage on damping.

RECOMMENDATION B45: STRENGTH AND DUCTILITY REQUIREMENTS

STOCHASTIC ANALYTICAL PARAMETER STUDIES SHOULD BE CONDUCTED TO INVESTIGATE THE REQUIREMENTS FOR DISTRIBUTION OF STRENGTH AND DUCTILITY IN PRINCIPAL TYPES OF STRUCTURES FOR EXTREME DYNAMIC LOADS SUCH AS EARTHQUAKES AND WINDS. SYNTHESES OF THESE STUDIES SHOULD BE PREPARED IN THE FORM OF PRACTICAL ANALYTICAL AND DESIGN APPROACHES FOR DETERMINING AND PROVIDING THE REQUIRED DISTRIBUTIONS OF STRENGTH AND DUCTILITY.

Research in analytical methods and structural behavior provides a capability to predict effects of earthquakes, winds, and other dynamic hazards. However, the ultimate objective is better buildings. Expert studies of the experimental evidence, with rigorous analytical studies used to enlarge the scope of parameters considered, will permit synthesis of practical design procedures and criteria which exploit effectively the new research knowledge.

RECOMMENDATION B46: NON-STRUCTURAL ELEMENTS

A MULTI-DISCIPLINARY PROGRAM OF ANALYTICAL, EXPERIMENTAL, AND DESIGN STUDIES SHOULD BE CONDUCTED TO ACQUIRE KNOWLEDGE AND DEVELOP STANDARDS FOR IMPROVING PRACTICES OF DESIGN FOR NON-STRUCTURAL BUILDING ELEMENTS.

Earthquake experiences have shown such extensive life hazards and property losses from failures of furnishings, mechanical systems and service systems that priorities are difficult to define. Functioning elevators are vital to evacuations of tall buildings prior to secondary hazards such as fire. Emergency services such as power, water and communications must be preserved. The resistance of ceilings, shelving, lighting fixtures and partitions must be improved to reduce life hazards and property losses.

ACKNOWLEDGMENTS

The success of the workshop was due to the enthusiastic participation of all attendees. However, special contributions by several individuals should be acknowledged.

Charles Thiel of the National Science Foundation and Ugo Morelli of the Office of Emergency Preparedness assisted in formulating the cooperative Federal program leading to the workshop. The Applied Technology Council of the Structural Engineers Association of California arranged for five of the workshop papers and their review by the Seismology Committee of SEAOC. The workshop program and arrangements were coordinated by H. S. Lew. Staff members from the National Bureau of Standards Boulder Laboratory assisted in conducting the workshop and supplying the support services and required facilities. Secretarial support for the workshop was provided by Sharon Weeks and Lynn Androsik. The complete workshop proceedings were typed with great care by Barbara Mullinix.

APPENDIX A

WORKSHOP PROGRAM

APPENDIX A

NBS/NSF NATIONAL WORKSHOP ON BUILDING PRACTICES FOR DISASTER MITIGATION

P R O G R A M

MONDAY, AUGUST 28, 1972

8:30 A.M. Registration

9:00 A.M. Welcoming Address and Introductory Remarks

-- H. S. Boyne, Chief
Quantum Electronics Division
NBS/Boulder

-- J. R. Wright, Director
Center for Building Technology, NBS

-- R. N. Wright
Deputy Director (Technical)
Center for Building Technology, NBS

-- C. Thiel, Program Manager
Division of Advanced Technical Applications, NSF

MORNING SESSION CHAIRMAN - R. N. Wright
Deputy Director (Technical)
Center for Building Technology, NBS

9:30 A.M. VALUES AND COSTS
H. Kunreuther, Economist
University of Pennsylvania

10:15 A.M. B R E A K

10:30 A.M. APPROACHES TO IMPLEMENTATION
P. Baseler, Codes Consultant
House Springs, Missouri

11:15 A.M. EARTHQUAKE HAZARDS FOR BUILDINGS
N. Donovan, Foundation Engineer
Dames and Moore

11:30 A.M. THE PROBLEM OF SEISMIC ZONING
S. T. Algermissen, Director
Seismic Research Group
National Oceanic and Atmospheric Administration

12:15 P.M. L U N C H

AFTERNOON SESSION CHAIRMAN - C. Culver
Disaster Research Coordinator
Office of Federal Building Technology
Center for Building Technology, NBS

1:30 P.M.	WIND HAZARDS FOR BUILDINGS J. Vellozzi, Associate and Civil Engineer J. Healey, Structural Engineer Ammann and Whitney
2:15 P.M.	ABNORMAL LOADING ON BUILDINGS AND PROGRESSIVE COLLAPSE N. Somes, Chief Structures Section Center for Building Technology, NBS
3:00 P.M.	B R E A K
3:15 P.M.	LAND USE PLANNING AND NATURAL DISASTER MITIGATION J. Wiggins, President W. Petak, Vice President M. McCoy, Research Associate D. Moran, Consultant J. H. Wiggins Company J. Slosson, Geologist James E. Slosson and Associates W. Monash, Planning Director Santa Cruz Company, California
4:00 P.M.	ARCHITECTURAL APPROACHES TO HAZARD MITIGATION E. Hillman, President Hillman, Biddison and Loevenguth A. Mann, Senior Vice President Daniel, Mann, Johnson and Mendenhall
4:45 P.M.	A D J O U R N M E N T
6:00 P.M.	R E C E P T I O N

TUESDAY, AUGUST 29, 1972

MORNING SESSION CHAIRMAN - S. Kramer, Chief
Office of Federal Building Technology
Center for Building Technology, NBS

9:00 A.M.	PROCEDURES AND CRITERIA FOR EARTHQUAKE RESISTANT DESIGN (PART I) C. Pinkham, President S. B. Barnes and Associates
9:15 A.M.	PROCEDURES AND CRITERIA FOR EARTHQUAKE RESISTANT DESIGN (PART II) N. Newmark, Head Department of Civil Engineering W. Hall, Professor, Civil Engineering University of Illinois

10:00 A.M.	<u>PROCEDURES AND CRITERIA FOR WIND RESISTANT DESIGN</u> J. Vellozzi, Associate and Civil Engineer J. Healey, Structural Engineer Ammann and Whitney
10:45 A.M.	B R E A K
11:00 A.M.	<u>CRITERIA FOR BUILDING SERVICES AND FURNISHINGS</u> J. Ayers, Principal T. Sun, Vice President Ayers, Cohen, and Hayakowa
11:45 A.M.	L U N C H

AFTERNOON SESSION CHAIRMAN - C. Thiel
Program Manager
Division of Advanced Technology Applications, NSF

1:15 P.M.	<u>BEHAVIOR OF STRUCTURAL ELEMENTS</u> B. Bresler, Professor University of California
2:00 P.M.	<u>BEHAVIOR OF STRUCTURAL SYSTEMS UNDER DYNAMIC LOADS</u> R. Sharpe, Executive Vice President - Operations G. Kost, Assistant Vice President John A. Blume and Associates J. Lord, Director of Systems Engineering Albert C. Martin and Associates
2:15 P.M.	B R E A K
3:00 P.M.	<u>SURVEY AND EVALUATION OF EXISTING BUILDINGS</u> F. McClure, Partner McClure and Messinger
3:45 P.M.	A D J O U R N M E N T
	E V E N I N G F R E E

<u>WEDNESDAY, AUGUST 30, 1972</u>

9:00 A.M.	Meetings of the Following Subcommittees:
and	SUBCOMMITTEE #1 - "Implementation of Earthquake Hazard Reduction Practice"
1:30 P.M.	
and	SUBCOMMITTEE #2 - "Formulation of Earthquake Hazard Reduction Practice"
7:30 P.M.	SUBCOMMITTEE #3 - "Implementation of Wind Hazard Reduction Practice"
	SUBCOMMITTEE #4 - "Formulation of Wind Reduction Practice"

THURSDAY, AUGUST 31, 1972

MEETINGS OF SUBCOMMITTEES # 1 - 4

Exchange Comments on First Draft of Recommendations Between Subcommittees and Rework Accordingly.

FRIDAY, SEPTEMBER 1, 1972

SESSION CHAIRMAN - S. Kramer, Chief
Office of Federal Building Technology
Center for Building Technology, NBS

9:00 A.M.	Group Discussion of Recommendations
1:00 P.M.	A D J O U R N M E N T

APPENDIX B

REVIEW ARTICLES

VALUES AND COSTS

by

Howard Kunreuther*

I. Introduction

During the past two years the United States has had more than its normal share of damage from the natural elements: an earthshaking event in San Fernando, a windblown affair in Lubbock, Texas and most recently a wall of water invading Rapid City and a stormy Agnes creating havoc in the Northeast. Not only have these recent disasters demonstrated that nature knows no boundaries but they have underlined the importance of conferences such as this one on disaster mitigation and damage prevention.

One of the lessons to be learned from these recent events is the failure of communities and individuals to adequately protect themselves against potential damage from disasters. In San Fernando a number of buildings including a hospital were poorly designed and did not withstand the force of the quake. Few individuals living in the area carried earthquake insurance even though the rates were relatively modest. For example, the premium rate for wood frame houses is twenty cents per $100 of coverage for straight earthquake insurance and fifteen cents per $100 if the risk is endorsed on a homeowners policy; both are subject to a mandatory five percent deductible. (Federal Insurance Administration [19], p. 14.) Even though Rapid City was one of two communities in South Dakota that qualified for the federal government's subsidized National Flood Insurance, few families had taken out policies. An official of St. Paul Fire and Marine Insurance Company commented that despite newspaper ads and other promotions we tried this year, we were only able to sell 300 policies covering $367,000 in two years [20].

Since few individuals are willing to protect themselves voluntarily against the consequences of natural disasters it may be necessary to develop formal guidelines for better building practices in the future. In making decisions on what type of structures should be built in hazard prone areas, detailed questions must be raised which I have categorized under four different headings:

1. Data on intensity and frequency of the hazard

   a. What data do we have on the probability of a particular type disaster affecting a particular area (e.g., an earthquake of intensity VIII in Bakersfield)?

   b. How sensitive are final decisions to changes in these frequencies? In other words, is accuracy of probabilistic estimates critical in choosing one type of structure over another?

2. Cost and damage information

   a. What are the differences in costs between types of construction?

   b. What damages are likely to result to a given structure from a disaster of specified intensity?

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c. How sensitive are final decisions to changes in these damage figures over time?

3. Factors difficult to quantify

a. How do you calculate the value of a human life and/or the cost of severe injury?

b. What differences exist between the private and social costs of human losses?

c. How sensitive are final decisions on building practices to changes in these values?

4. Who should bear the costs of disasters?

a. Individuals living in the hazard-prone areas

b. Federal government

c. Joint private and public responsibility.

Fortunately a number of excellent recent studies by engineers, geologists, seismologists and others have provided us with statistical estimates on expected damage from earthquakes and disasters caused by high winds.[1] McClure [11] has supplemented these methodological reports by providing detailed estimates of replacement costs of buildings damaged by the 1952 Bakersfield earthquake. By developing an approach for analyzing this information we can determine the acceptable level of risk which should be tolerated from a specific disaster. Not only will this acceptable risk level be a function of the accuracy of the statistical data, but it will also be critically dependent on the answer to the question: Who should bear the cost of natural disasters? As we shall see, the optimal type of structure in a given area may differ if future disasters are viewed from the eyes of a prospective homeowner in the area or an individual taxpayer in another part of the country.

To illustrate the critical elements in evaluating alternative building practices in hazard-prone areas, I will next consider a specific example which involves the choice of building a brick masonry structure or reinforced brick structure in the fictitious town (I hope) of Shakerville, Missouri. The example will be intentionally oversimplified to illustrate concepts and hence can be criticized from the point of view of accuracy. I look forward to learning more about the specifics on frequency data and costs from individuals at the workshop far more knowledgeable in this general area than I am.

II. Decision-Making in the Private Sector

A. Damage and Probability Estimates

Consider a family who is building for the first time in Shakerville or relocating in the area after having its home totally destroyed by a severe earthquake. The following steps would be required to specify the expected annual damage to a particular structure from earthquakes:

1. Determine the frequency of any earthquake of a given magnitude to the particular area in question. Techniques for doing this

[1] For an excellent discussion of a methodology for gathering earthquake damage statistics see Coast and Geodetic Survey [18].

have been detailed by Blume [2], Housner [5], and Coast and Geodetic Survey [18].

2. Compute the intensity of ground shaking over a given area for any specific quake. Housner [5] suggests that the distribution of intensity over a given region can be depicted by contour lines with the greatest intensity near the fault and diminishing intensity at an increasing distance from the fault.

3. Combining figures on the frequency it is then possible to compute the probability that a given site will experience ground motion which will be of a certain intensity.

4. For each particular structure it is possible to compute the expected damage if the particular structure receives ground shaking of at least a certain intensity. A detailed damage survey for Bakersfield has been undertaken by McClure [11] and has been elaborated in Coast and Geodetic Survey [18].

For simplicity assume that the family is considering building either a brick masonry structure or a reinforced brick home. Table 1 presents the following illustrative comparative data on costs and potential damage from an earthquake. As you can see, we have greatly simplified the problem by assuming that there would be no damage from any quake with intensity less than or equal to VI; the only two threatening events are quake of intensity VII and VIII with respective annual probabilities of occurrence of .02 and .01. Reinforced brick structures are safer than brick masonry buildings as shown by the expected annual damage figures in Table 1 but they cost slightly more to build. For example, a $50,000 brick masonry house would cost approximately $53,000 if the walls were made of reinforced brick.[1]

Table 1

EXPECTED ANNUAL PHYSICAL DAMAGE FROM AN EARTHQUAKE FOR A BRICK MASONRY AND REINFORCED BRICK STRUCTURE

Intensity of Earthquake (Modified Mercalli Scale)	Annual Probability of Occurrence	Expected Annual Damage to Structure j (percent of Original Cost)	
(i)	(p_i)	Brick Masonry	Reinforced Brick
VI or less	.97	0	0
VII	.02	10	0
VIII	.01	60	20

[1] This cost differential was obtained in the following manner. Reinforced brick masonry costs approximately 60 per cent more per square foot than brick masonry (i.e., $3.60 instead of $2.25). The building walls account for approximately 10 per cent of the total cost of a structure. Thus the walls of a $50,000 brick masonry house would cost $5,000. The identical reinforced brick structure would then cost approximately $3,000 more or $53,000. I would like to express my appreciation to Alan Yorkdale, Director of Engineering and Research at the Structural Clay Products Institute for providing me with these cost differentials. For a more detailed discussion of cost estimates for different type building walls, see [17].

B. Cost Comparisons for the Two Structures

A number of tangible and intangible cost elements will determine the homeowner's decision as to which house he will purchase. We will consider each of the factors in turn and then determine the sensitivity of this final decision to changes in his estimates.

B.1 Mortgage Terms

Undoubtedly the most tangible element with respect to the homeowner's final decision are the terms of his mortgage. Both the length of the mortgage, the rate of interest and the downpayment requirements will influence the homeowner's choice. Table 2 shows how the annual payments per $1,000 loan are affected by the interest rate and the length of the mortgage. An increase in the interest rate and/or a decrease in the length of the loan will require higher payments on the part of the borrower and will thus favor the purchase of the cheaper house. If all other factors remain the same, we would thus expect a homeowner to favor the brick masonry house over the reinforced brick home when money is tight and hence interest rates are higher. Thus a 30-year loan at an annual rate of 5 per cent will require annual payments of approximately $65 per $1,000 in contrast to $117 per $1,000 for a 20-year loan at 10 per cent.

Table 2

EFFECT OF INTEREST RATE AND LENGTH OF LOAN ON ANNUAL PAYMENTS PER $1,000 LOAN

Length of Loan (in years) / Annual Interest Rate (in %)	20	25	30
5	80.24	70.95	65.05
6	87.18	78.23	72.65
7	94.39	85.81	80.59
8	101.85	93.68	88.83
9	109.55	101.81	97.34
10	117.46	110.17	106.08

B.2 Expected Cost of Physical Damage

Who should bear the cost of damage to property from natural disasters? Currently there is a liberal system of disaster relief subsidized by the federal government where a combination of grants and low-interest loans are provided to individuals who suffer home or business losses. At the other extreme, we would require the homeowner to pay for his own losses by taking out a loan at the market rate of interest if he did not have sufficient insurance protection. The decision on the type of structure which should be erected and its location will be greatly influenced by who bears the cost after a disaster. We will illustrate differences which may result by contrasting a self-insurance policy where the individual must bear the losses himself with the current federally subsidized relief policy.

a. Self-Insurance Policy. To determine the expected annual cost of a self-insurance policy, it is necessary to specify both probabilistic and damage figures as we have done in Table 1. In more general terms, suppose there are m possible events (e.g., types of weather phenomena that can affect a given community) with each event i having an annual probability of occurrence, p_i. Suppose each event can cause damage D_i^j to a given type structure j in the community, where D_i^j may equal zero for a number of possible events. For our example, suppose event i is a quake of intensity VII so that $p_i=.02$. If the structure j is a $50,000 brick masonry house, then (from Table 1) D_i^j = $5,000 (i.e., 10 per cent of $50,000); if structure j is a $53,000 reinforced brick structure, then $D_i^j = 0$. The expected annual damage from a disaster for a specific structure j is given by

$$E(D^j) = \sum_{i=1}^{m} p_i D_i^j .$$

If the homeowner were to take out a loan at the market rate of interest whenever he received any damage to structure j then his expected annual costs in present value terms would be $E(Dj)$.

b. Current System of Disaster Relief. The cost of an earthquake to the private homeowner or businessman suffering damage will be quite different if the current system of federal relief prevails. A brief summary on the changing role of the federal government with respect to disaster relief should provide us with insight into how the present policy evolved.

At the turn of the century, people who were located in hazard-prone areas had to bear almost the entire risk of damage from natural disasters themselves. Relatively little insurance coverage was available and aid to the private sector was limited to voluntary charitable contributions. Since 1900, there have been several important developments which have reduced the losses which a person has had to bear himself, thus encouraging more people to locate in these areas than would otherwise have been the case. Red Cross contributions were the first formal source of relief to individuals suffering damage from a disaster. This organization, established by a federal statute in 1905 has had a long and distinguished history of private charity. Raising all of its funds through private donations, the Red Cross uses them to provide relief to victims suffering disaster losses.

Beginning in 1953, the federal government assumed a direct responsibility for financially aiding individuals hurt by a disaster. In that year the Small Business Act was passed, authorizing the SBA to offer low-interest loans to homeowners and businesses suffering injury from natural disasters. The general purpose of SBA disaster loans is "to restore a victim's home or business property as nearly as possible to its pre-disaster conditions." Before the Alaskan earthquake, the agency provided 3 per cent loans with a maximum repayment period of 20 years to cover the exact amount of physical damage. It was understood that the borrower would use the entire loan strictly for the purpose of rebuilding or repair. The severity of the damage in Alaska caused concern that unless the SBA liberalized its policy many individuals would not qualify for a disaster loan because of their inability to pay off their old mortgages and other debts and still make monthly payments to the SBA.

Perhaps the most significant revision of SBA policy was the authorization of loans for substantial debt retirement for any homeowner or business suffering losses from the earthquake. He was given funds not only to repair his damaged structure, but also to retire old debts (for example, outstanding mortgages, accounts payable) which may have had nothing to do with the disaster itself. Thus, instead of continuing to pay conventional 7 or 8 per cent rates on these outstanding claims, the borrower could now retire them at a subsidized 3 per cent rate. A further reduction in the size of a victim's monthly payment was achieved by permitting a 30-year amortization period instead of the normal 20-year maturity of SBA loans. If the property owner requested it, the agency would waive any principal and interest during the first year of the loan and on principal up to an additional four years. Thus, the victim's burden was minimized; in fact, in a number of cases, particularly for businesses, the borrower was financially sounder after the disaster than before the "catastrophe."

Although the SBA made it very clear that its actions in Alaska were taken to meet a special situation, there is clear evidence that the agency has not retreated to its more stringent policy. Using the Alaskan case as a precedent, a Congressional bill was passed at the end of June 1965, authorizing the SBA to permanently extend its maximum load period from 20 to 30 years. The Southeast Hurricane Disaster Relief Act of 1965 authorized the Small Business Administration to "forgive" a part of each loan up to maximum of $1,800, a provision which was not even permitted in Alaska.[1]

The Disaster Relief Act of 1970 increased the forgiveness amount to $2,500 and permitted the annual interest rate on loans to be set up to two per cent below the rate for 10-12 year government securities but never higher than 6 per cent per year. A recent Congressional Act (PL 92-385) inspired by the Rapid City floods and Hurricane Agnes has liberalized SBA policy even further. An owner of a home or business which receives damage from a disaster occurring between January 1, 1972 and July 1, 1973 can obtain a forgiveness grant of up to $5,000 and a 30 year loan at one per cent per year to cover the remaining losses.

To see the effect of the federally subsidized loans on private costs, suppose the average annual damage to a given structure from an earthquake is $1,000.[2] Assume that the owner can take advantage of a 30-year disaster loan at an interest rate below the market rate. Table 3 computes the present value of this loan for variations in both the SBA and market rates of interest. Naturally, if the SBA rate and market rate are identical the present value to the homeowner of a $1,000 loan will be $1,000. On the other hand, if the SBA rate is 1 per cent and the market rate is 7 per cent, the present value of the loan drops to $481.[3]

In general, as the market rate of interest increases and/or the SBA interest rate decreases, the present value of the loan also decreases. Hence even though the expected annual losses of a brick masonry building are considerably higher than an identical reinforced brick structure, the loss differentials between the two in present value terms is narrowed considerably by virtue of the SBA disaster relief policy.

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[1] For more details of the equity and efficiency of the SBA disaster loan policy, see Kunreuther [9], and Dacy and Kunreuther [4], Chapters 9 and 10.

[2] There would, of course, be long stretches of time where the building would not receive any damage; however, if a quake occurred, the damage might be $100,000. On the average, an annual loan of $1,000 would be required to repair the structure.

[3] The figures in Table 3 do not take into account the forgiveness feature of the SBA Policy and hence understate the benefits of PL 92-385 to the individual.

Table 3

PRESENT VALUE OF 30-YEAR $1,000 LOAN FOR COMBINATIONS OF SBA AND MARKET ANNUAL RATES OF INTEREST

SBA Interest Rate (in %) \ Market Interest Rate (in %)	5	6	7	8	9	10
1	596	533	481	436	398	365
2	686	615	554	503	459	421
3	784	702	633	574	524	481
4	889	796	718	651	594	545
5	1,000	895	807	732	668	613

B.3 Intangible Costs from a Disaster

Up until this point in the analysis, we have dealt with physical losses from a disaster. When one introduces intangible factors, such as the value of a human life, more difficult estimation problems arise. At the outset, we should distinguish between the private cost perceived by the individual and the public cost borne by society. If an individual is killed by a disaster then the costs to society include not only the expenses of training and educating this person, but also his potential earning stream discounted back to the present. Normally, an individual does not look backwards or project forward in this manner when making his decisions and so may implicitly value his life less (in dollar terms) than society would.[1] He may thus take actions which would be justified when using his own sets of values but would be considered unwise from the perspective of the general public.

Recently, attempts have been made to estimate the value of a human life.[2] By studying "loss of life" lawsuits one finds that jury awards have ranged anywhere from $50,000 to $500,000. These two extreme values could provide an upper and lower bound to the value of a human life. An alternative method would be to estimate this value by considering an individual's potential future earnings. Suppose a person's annual salary is estimated to be $10,000 for each of the next 25 years; then his discounted future earnings (at a 7 per cent annual interest rate) would be $116,636, which would be a proxy for the value of his life. Finally, an FAA study to compute the value of life saving in commercial air transport accidents estimated that the indirect and direct costs yielded a life value of $373,000 per average fatality.

The value of a human life is only part of the story. We must also consider the probability that one or more individuals will be in a building destroyed by a particular disaster. For example, the probability may be quite high that if a warehouse is damaged by an earthquake, any individual in the building at the time will be killed. But the percentage of time that

[1]For a more detailed discussion of these points, see Starr [16].

[2]For an interesting description of these studies, see Otway [14].

any one person is likely to be in the warehouse is quite small. In other words, we must also determine the occupancy factor associated with a particular structure. The occupancy factor has been defined by Blume [2] to be the ratio of the total person occupancy-hours per year to the theoretical number of hours per year that the building would be occupied if it was filled to capacity.[1] For example, if a residence houses a family of four (i.e., capacity is four) then the theoretical total number of hours per day that the house could be occupied would be 4(24) = 96 hours. In practice, suppose only 48 person-hours would be occupied on a typical day in the year. Then the annual occupancy factor will be .5. The number of person-years of exposure (N) for a family of four is simply (4)(.5) = 2. An increase in either the occupancy factor and/or capacity will increase the expected number of lives lost from a given disaster. For ease of illustration we will assume that all individuals in a house are killed if it collapses from a quake; otherwise there are no loss of lives. For each event i (i.e., a quake of a given intensity) there is some probability q_i^j that structure j will collapse. Since the probability of this quake occurring has been already specified to be p_i the expected annual number of lives lost for structure j is the simply

$$E(L^j) = N \sum_{i=1}^{m} p_i \, q_i^j$$

where N = number of person-years of exposure.

These data are assembled in Table 4 for the brick and reinforced brick homes when N = 2. The assumed probability of collapse (q_i^j) to either structure is relatively small even for an earthquake of intensity VIII. As expected, the reinforced brick home is considerably safer than a brick masonry structure.

Table 4

EXPECTED NUMBER OF LIVES LOST PER YEAR FROM AN EARTHQUAKE PER BRICK MASONRY AND REINFORCED BRICK STRUCTURE
(Based on Number of Person-Years of Exposure N = 2)

Intensity of Earthquake (Modified Mercalli Scale)	Annual Probability of Occurrence	Annual Probability that Structure Collapses Given Occurrence of Quake (q_i^j)		Expected Number of Lives Lost Per Year From Quake i ($N\, p_i\, q_i^j$)	
(i)	(p_i)	Brick Masonry	Reinforced Brick	Brick Masonry	Reinforced Brick
VI or less	.97	0	0	0	0
VII	.02	.01	0	.0004	0
VIII	.01	.02	.01	.0004	.0002

[1] Estimates of occupancy factors for particular buildings appear in Blume [2].

B.4 Evaluating the Two Structures

Table 5 summarizes the factors which will determine the type of structure to build and presents illustrative figures for brick masonry and reinforced brick homes. We have assumed that the homeowner is considering building either a $50,000 brick masonry home or the identical reinforced brick house for $53,000. A downpayment of $10,000 would be made for either house and the balance would be taken out as a 25-year mortgage at the market rate of interest (7 per cent per year). The expected annual physical damage figures are based on the data from Table 1 and the expected number of lives lost is taken from Table 4. Using these figures we can easily compute the expected annual total cost for each structure under a self-insurance policy and under the current disaster relief policy.

Table 5

FACTORS DETERMINING CHOICE OF STRUCTURE TO BUILD AND ILLUSTRATIVE FIGURES FOR BRICK MASONRY AND REINFORCED BRICK HOMES

Factor	Type of Structure: Brick Masonry	Type of Structure: Reinforced Brick
Cost of Structure	$50,000	$53,000
Downpayment	$10,000	$10,000
Outstanding Mortgage	40,000	43,000
(1) Length of Loan	25 years	25 years
(2) Annual Rate of Interest	7%	7%
Annual Mortgage Payments	$ 3,432[a]	$ 3,690[a]
Expected Annual Physical Damage [E(DJ)]	400[b]	106[b]
Present Value of Loan for $E(D^j)$:		
Self-Insurance Policy[c]	400	106
Federal Disaster Relief Policy[d]	192[e]	51[e]
Expected Annual Lives Lost $E(L^j)$	.0008[f]	.0002[f]

[a]Based on Table 2.

[b]Based on figures from Table 1.

[c]Loan discounted at market rate of interest.

[d]30-year loan at 1 per cent annual interest rate.

[e]Figures are based on Table 3 with 7 per cent market rate of interest.

[f]Based on figures from Table 4.

Based on present value analysis, Table 6 shows that the brick masonry structure will be chosen if the proposed disaster relief bill is enacted and the value per human life is slightly less than $200,000. If a self-insurance policy is followed, the reinforced brick house will be more desirable, no matter what value is placed on a human life.[1]

Table 6

COMPARISON OF EXPECTED ANNUAL COSTS FOR BRICK MASONRY AND REINFORCED BRICK STRUCTURES FOR DIFFERENT VALUES PER HUMAN LIFE
(Based on figures from Table 5)

	Type of Relief Policy			
Value Per Human Life	Self-Insurance		Current Disaster Relief	
	Brick Masonry	Reinforced Brick	Brick Masonry	Reinforced Brick
$100,000	$3,912	$3,816	$3,704	$3,761
200,000	3,992	3,836	3,784	3,781
500,000	4,232	3,896	4,024	3,841

How sensitive are these decisions to changes in cost estimates? We have already indicated that an increase in the interest rate (and consequently the discount rate) will have two effects: it will increase the differential in monthly payments between the brick and reinforced homes and it will decrease the expected annual physical damage differential between brick masonry and reinforced brick structures if disaster relief is federally subsidized. Both of these changes will make the brick masonry house a more attractive buy. On the other hand, an increase in the value of a human life will push the decision in favor of the reinforced brick home.

A graphical illustration of the type of sensitivity analysis which can be undertaken is depicted in Figure 1, based on data from Table 5, for both a self-insurance policy and the federal relief program. Suppose the expected annual physical damage differential of a brick masonry house over the identical reinforced brick home was $200; looking at Figure 1 it is clear that even if the value of a human life was as low as $96,333 the reinforced brick home would be preferred under a self-insurance policy. If a federally subsidized disaster relief policy was in effect then Figure 1 shows that the value of a human life would have to increase to $270,000 before the reinforced brick home would be preferred.

[1] It should be pointed out that we are making the implicit assumption in this analysis that the individual is risk neutral so his utility function is linear in money. If a person has some aversion to risk, then he will attempt to avoid large losses and may want to build the reinforced brick house even when present value analysis suggests that the brick masonry home is less expensive. If, on the other hand, individuals tend to underestimate the probability of extreme events, they may choose the brick house over the reinforced structure even when the "objective" figures suggest the opposite decision.

III. Decision-Making for Public Facilities

Theoretically it is possibly to analyze the costs and values of public facilities in the same way that we studied the private homeowner's decision. Now the relevant actors in the drama are the entire community (rather than an individual homeowner) and the taxpaying public. Because public facilities frequently affect the entire community and even the region, losses resulting from a disaster may be considerable. One way to compare the economic value of different public facilities is by introducing the concept of an importance factor. The importance factor should measure both the expected direct disaster losses to a given type structure in a particular locality as well as the indirect losses after a particular disaster if the building has lost its functional capability. The direct losses can be captured by determining expected annual physical damage as well as expected loss of life and/or injuries to people in the structure. The indirect losses reflect the critical functions each structure must perform during the immediate post-disaster period. To estimate this component one must consider alternative sources which may temporarily replace the damaged facility during the emergency period. For example, if there are a number of hospitals in a particular region then the importance factor associated with a particular one will be much lower than if it is the only facility serving the region. Similarly, regions which are centrally located would assign lower importance factors to structures than areas which are somewhat isolated. Table 7 presents a sample list of those public facilities which perform critical functions during the immediate aftermath of a disaster and have relatively high importance factors.

Table 7

PUBLIC FACILITIES WITH CRITICAL FUNCTIONS IN IMMEDIATE POST-DISASTER PERIOD

Facility	Critical Function
Fire Fighting Equipment	Mitigation of Physical Damage
Police Department Facilities	Maintaining Law and Order
Telephone Center	Communications
Hospitals	Treating Injured Victims
Sewage Treatment Plant	Waste Disposal
Water Treatment Plant	Providing Purified Water
Shelter Facilities	Temporary Housing
Food Supply Facilities	Feeding Residents
Roads and Highways	Transportation
Airports, Train Stations and Ports	Receiving Outside Food and Medical Supplies

There will also be long-run economic effects associated with disasters which should be included in the concept of an importance factor. The destruction of public facilities such as transportation and communication networks can severely affect the recovery process and the productive capability of a community and hence have indirect regional and national effects. By studying these long-run costs more closely it should be possible to develop a priority list for rebuilding public facilities. For example, it may not be feasible to reconstruct commercial buildings damaged by an earthquake until transportation facilities have been at least partially restored. The magnitude of outside aid and the speed with which it is forthcoming will play an important role in determining the long-run economic impact of natural hazards on the community. Despite the large amount of destruction caused by a disaster, recovery may be very rapid if capital in the form of low-interest loans and grants is immediately forthcoming. In fact, the disaster may actually turn out to be a blessing in disguise for the community. Aside from the positive short-run economic effects triggered by the reconstruction activity, there is an opportunity for improving damaged public and commercial facilities. More research is needed on specifying the long-run impact of natural hazards on a community and the positive opportunities for mitigating future losses from hazards.[1]

The federal government provides more generous relief to repairing damaged public facilities than to restoring damaged private property. In 1950 the first comprehensive Federal Disaster Act (PL 81-875) was passed which provided generous federal disaster assistance with respect to restoration of local facilities if the President declared the region "a major disaster area." In the ensuing years, a number of special bills have been passed following specific disasters,[2] culminating in the Disaster Relief Act of 1970 (PL 91-606). This act authorized up to 100 per cent reimbursement of the cost of permanent repair, restoration or replacement of disaster-damaged state and local public facilities, rather than just allowing only emergency repairs or temporary replacement. Federal loans are contingent upon compliance with applicable codes, specifications and standards. Following the San Fernando earthquake which damaged or destroyed several private medical care facilities, Congress amended PL 91-606 by authorizing similar grants to tax-exempt nongovernment medical care facilities (PL 92-909).

Consider a community decision with respect to location and type of construction of a proposed hospital. From the above remarks, it is clear that the community must balance the importance of this type of public facility following a disaster with the generous public relief forthcoming if the building is damaged or destroyed. The first factor argues for a very safe building in a nonrisky area; the second factor works in the opposite direction.

Rather than being forced to attach specific dollar estimates to the factors which are difficult to quantify, it is possible to undertake the same type of sensitivity analysis for public facilities that led to Figure 1. It is likely that for each factor there will be a wide range of dollar values that imply the same optimal decision with respect to location and choice of building material. As in the private sector, this range of dollar values will be a function of the type of disaster relief program in effect. Under the current system of disaster relief the federal government bears the entire costs of reconstruction and the expenses associated with emergency relief. Other things being equal, we would then expect a community to build more public structures in hazard prone areas than if the local residents were solely responsible for financing their own recovery.

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[1] Empirical evidence on the long-run recovery process following natural disasters is presented in Dacy and Kunreuther [4], Chapter 8.

[2] For a summary of disaster legislation, see OEP [12], Volume 1, pp. 167-173.

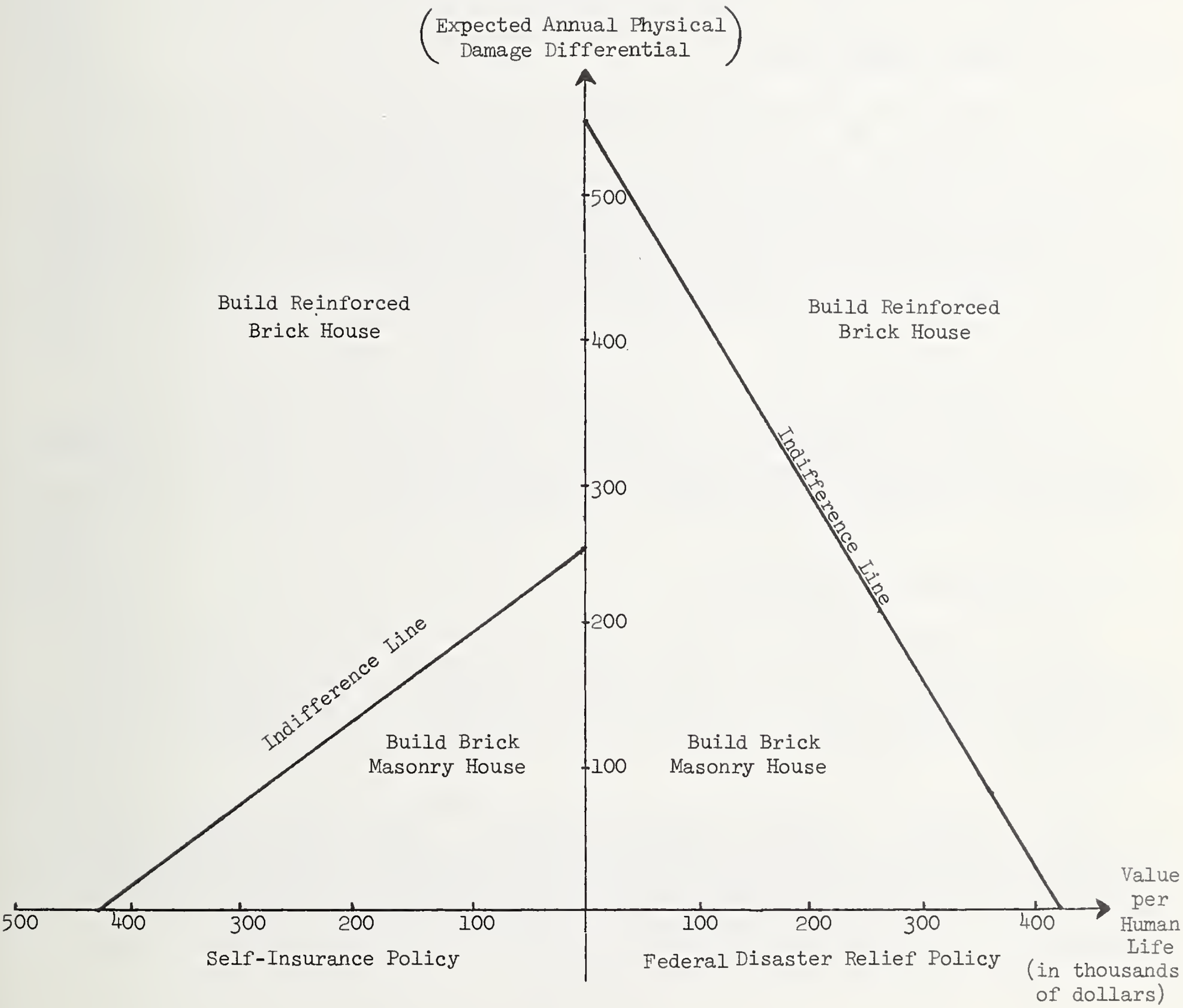

Figure 1. Effect of damage differential, value per human life and type of disaster relief policy on decision to build brick masonry or reinforced brick house.

IV. Alternative Cost Bearing Approaches

As we have already seen, the current federal relief policy treats disasters as if they were a public responsibility. In other words, every taxpayer in the U.S. bears a fraction of the costs of disaster-induced damage to sections of the country. As a result of this policy, individuals may find it in their own best interests to build less expensive but more disaster-prone structures in high risk areas. A look at the type of structures and their locations in California should convince us that this has actually happened. Let us examine five alternative cost-bearing approaches to disaster relief.

A. Total Federal Responsibility

If we accept the notion that disasters are "acts of God" which must be borne by all members of society then we should have a disaster relief policy whereby each homeowner who suffers a loss is given an outright grant to rebuild his house. Furthermore he should have the freedom to construct it in any part of the country he wishes without any restrictions. The New Zealand government's current attitude toward earthquake damage illustrates this point of view. Currently they have an earthquake and war damage insurance program where all property insured against fire is automatically protected against earthquake losses. The earthquake coverage is an extension of insurance against war damage which was developed during the latter part of World War II, at which time it was felt that the entire country should pay for any property damage by the enemy.[1] An Earthquake and War Damage Commission has since been established to determine government policy toward natural disasters. Although the Commission theoretically can require certain building standards before issuing insurance coverage, they have not actually done this nor have they established any criteria for safe buildings. Hence, an individual has the freedom to locate his home and business in any part of the country and will be protected against earthquake losses.

B. Self-Insurance by Homeowner

Another extreme approach is the self-insurance method where each individual pays for the entire cost of the disaster. We are only beginning to learn about people's perceptions of extreme events, but the evidence from geographers and psychologists suggest that individuals employ numerous mechanisms to reduce uncertainty about these events so as to avoid dealing with them.[2]

Unfortunately, the owner of a structure is normally not made aware of the risk of living in the area by the developers. Aside from considerations of social responsibility, there is no built-in incentive for the contractor, architect, or engineer to develop safer but more costly buildings in hazard-prone areas unless there are formal restrictions placed on them such as building codes. Although the builder or contractor who constructs a poorly designed house which is damaged by a disaster may be theoretically at fault on a charge of negligence or misrepresentation, it may be very difficult for the homeowner to win his case in court.[3] We are a long way from the days of

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[1] For discussion of the New Zealand program, see [18], Appendix A, pp. 86-87 and O'Riordan [13].

[2] For some specific illustrations of how individuals in hazard-prone areas perceive the risks of natural hazards, see Burton and Kates [3], Kates [8] and Slovic, Kunreuther and White [15].

[3] For an excellent detailed discussion of the risks and legal liability from earthquakes, see Hughes [6].

Hammurabi where the contractor was put to death if the house he built collapsed and caused the death of the homeowner. The victim today must rely on charity or federal aid or renege on his outstanding debts.

A current example is Wilkes Barre, Pennsylvania where thousands of homeless families without flood insurance are willing to declare bankruptcy rather than paying off the mortgage on their flooded homes. As a result, the President has asked Congress for an unprecedented $1.7 billion in relief funds to aid these victims. If the past is any guide to the future, then there is little chance that we will permit victims of disasters to suffer the consequences of their lack of foresight. Hence, reliance on self-insurance as an effective mechanism for preventing future damage is less than perfect. When the chips are down, we would most likely view relief to victims as a public responsibility.

C. Required Insurance Protection

An alternative method of protection against disasters would be through the mechanism of insurance.[1] Rather than providing federal relief following a disaster each individual would be required to have some form of insurance protection against possible future losses from disasters. The insurance requirement could be enforced directly through government fiat or indirectly by banks to protect their outstanding mortgages. In either case, the individual would be required to pay a premium based on the expected annual costs from a disaster.

If the premiums were based on risk then insurance would provide information on ways that individuals could protect themselves against a disaster. Using the figures from Table 5, reinforced brick structures would only have an annual premium of $2 per $1,000 (i.e., $106/$53,000) while brick masonry structures would be charged $8 per $1,000 (i.e., $400/$50,000). Individuals would then have precise information on the expected annual damage to each of these structures from natural hazards. After taking into account the difference in their personal estimate of intangible costs (e.g., loss of life) they could then determine whether the extra construction costs of reinforced brick were more than offset by the reduced insurance premiums.

There are practical difficulties associated with utilizing insurance as a mechanism for optimizing disaster preparedness. Not only would it be costly to develop premiums which would differentiate between types of structures and location, but the complexity of the rate schedule would be very confusing to the homeowner. There is also no easy way to make sure that the homeowner has met the standards upon which his premium is based. There would thus have to be a cost of checking reflected in the rate structure. If the cost of determining the rate structure, transmitting the information to the consumer and inspecting the structure were incorporated in the rates then the permium might be considerably higher than the actuarial figure. It might then unnecessarily discourage some individuals and businesses from locating in a particular area where it may be profitable for them to do so.

D. Land Use Restrictions and Building Codes

Given the transaction costs and imperfections associated with marketing insurance it may be possible to achieve similar results through land use restrictions and building codes.[2] In theory, the most efficient way would be to form a national commission of experts who could collect information on

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[1] For a critique of current disaster insurance programs, see OEP [12), Vol. 1, pp. 135-145.

[2] Specific land use and building code ordinances are discussed and critiqued in OEP [12], Vol. 1, pp. 125-133.

disaster, analyze it to determine the marginal benefits of different types of preventive activities and suggest a methodology for evaluating these alternatives. The commission could disseminate information on the potential costs of future disasters to homeowners and businesses while prescribing a set of building codes and zoning regulations. By centralizing the decision-making process, some of the inefficiencies of the market place could be eliminated. As we have seen, there is little incentive for the individual builders, homeowners or businesses to worry about future disaster problems so mechanisms for enforcing building codes and land-use restrictions must be developed.

The costs associated with enforcing building code restrictions and land-use regulations may be more than balanced by benefits in the form of reduced future disaster losses. In this sense these enforcement costs are a type of public investment. Arrow and Lind [1] have shown that it is appropriate to evaluate public investments (e.g., costs of enforcing alternative building codes) by simply comparing expected annual costs as we did in our illustrative example. It should thus be relatively straightforward to specify the appropriate building codes as a function of construction costs, expected physical damage and intangible costs. If one is not certain of the intangible costs, then a diagram such as Figure 1 can be utilized for sensitivity analysis. For example, it may be possible to show that a building code requiring a hospital to be of reinforced brick would be preferred over any other material if the value of a human life was more than $20,000. It would then only be necessary to specify whether a human life was worth more than $20,000. If the answer was "yes" then reinforced brick could be prescribed, otherwise, one of the other materials would be more desirable.

Once nationally accepted land-use planning and construction standards have been developed, initiative for enforcing and tailoring the regulations to specific characteristics of the area may be more effectively handled at the state or local level. An actual illustration of this point is the recent adoption of a specific and restrictive building code by the City of Long Beach to improve the ability of buildings to resist earthquake damage. The new code criteria were developed by equating involuntary earthquake risk with other voluntary risk situations, such as automobile accidents.[1]

E. A Suggested Program

Current federal policy suggests that the public feels some degree of responsibility toward helping victims of natural disasters. The increasing costs of these events also indicates that preventive action is justified from an economic standpoint. The challenge lies in developing a policy which strikes a balance between satisfying the objectives of the individual living in a hazard-prone area and the general public.

The Federal Flood Insurance program may serve as a prototype for developing a plan to mitigate future earthquake and windstorm losses.[2] Specifically some form of comprehensive disaster insurance could be made available to homes and businesses in a hazard-prone area but only after the community had taken positive steps toward reducing potential losses by enforcing adequate land use measures and building code regulations. The initiative could thus lie with the communities rather than with the federal government. In return, existing structures would be insured at a subsidized insurance rate while new buildings would be charged an actuarial rate. In

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[1] For a more detailed description of the recommendations and nature of the study undertaken for Long Beach, see Wiggins and Moran [21].

[2] For a detailed description of the provisions in the National Flood Insurance Act of 1968, see OEP [12], Vol. pp. 136-138. A complementary cost sharing model for structural flood protection programs is discussed in Loughlin [10].

essence, the federal government would help pay the costs of protecting individuals now residing in hazard-prone areas from future disaster losses while requiring that the communities make these areas safer places in which to live.

For such a plan to have any chance of success, the federal government would have to withdraw its liberal disaster assistance programs such as the SBA low-interest loans. As we have already seen, there would otherwise be no financial incentive for communities to develop land-use measures or building codes. The most effective way of achieving a favorable reaction by communities toward such a program would be for federal agencies, such as VA and FHA, and private lending institutions to require some form of comprehensive disaster insurance as a condition for mortgage. The owner would then want to follow better building practices with respect to his property simply as a way of reducing his insurance premium. For such a system or some variant of it to be successful, each of the concerned groups must recognize the need to jointly defend themselves against an unpredictable woman--Mother Nature.

V. Summary and Conclusions

The following conclusions emerge from this survey of values and costs of natural hazards.

1. Rather than attaching specific dollar estimates to factors which are difficult to quantify, some form of sensitivity analysis should be undertaken. There may be a wide range of dollar values implying the same optimal decision with respect to location and choice of building materials.

2. More specific data is needed to determine the costs associated with damage to public facilities. Community, regional and national considerations must be included in the analysis.

3. The federal disaster relief policy plays a critical role in private and public sector decisions with respect to location of structures and type of construction. Currently the federal government is bearing the lion's share of the costs of natural disasters.

4. A methodology for determining the appropriate site and types of building materials for a given structure should be written in a form that the potential user can understand.

5. Insurance supplemented by land-use restrictions and building codes appears to be an appropriate policy for shifting the cost burdens of disasters from the general taxpayer to individuals living in hazard-prone areas.

6. There must be more dialogue between the theoretician and the practitioner as to ways of mitigating losses from future disasters.

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ADDENDUM*

Undertaking Benefit-Cost Studies

Benefit-cost studies should provide valuable data inputs for developing a disaster mitigation program. It may well not be economically feasible to mitigate completely the effects of natural hazards, since the potential benefits may not justify the additional expenditures. It is necessary to have a solid base for deciding how much of our resources should be devoted to mitigating hazards as compared to other priorities, and for deciding which disaster mitigation methods are most effective. Important studies suggested by the discussions of the subcommittee are:

- Desirability of accelerated programs of replacing older buildings especially susceptible to collapse during natural disasters. An initial question might be: Are the costs of tearing down a building and replacing it by another balanced by sufficient reduction in expected losses from natural disasters. At a higher level, should the Federal government invest money into tearing down old unsafe structures or would it be more beneficial for them to allocate these funds to subsidizing new construction?

- Desirability of restricting construction near faults or in other hazardous zones. One question that must be answered is: Are the needs of the community best served by imposing (and possibly paying damages for securing) such land use restrictions?

Many factors must be considered when facing such questions, and systematic analyses based upon adequate data are essential. Benefit-cost analysis also will provide reliable data inputs upon which to base decisions with respect to building code provisions, land use plans, and insurance premiums or other loss indemnification programs.

Benefit-cost studies may be carried out on many different levels, and the elements in the objective function and the measures of effectiveness will vary accordingly. The paper by Kunreuther deals with an individual's decision concerning a greater level of earthquake resistance in the construction of a new home. Here the concern is primarily to minimize the individual's total annual dollar cost, and secondarily with the risk of loss of life in the individual's family. However, when a decision involves the public welfare, other costs--both tangible and intangible--must be considered. Hence, in developing a set of recommendations for a program of disaster mitigation, the objectives from the individual, community and national point of view must all be considered.

In carrying out benefit-cost studies for disaster mitigation, various viewpoints with respect to the role of the public and private sectors must be kept in mind. Due to political pressures following a disaster, the Federal government has borne, and probably will continue to bear, a substantial portion of the costs of damage to private facilities and all the costs to public sector facilities. On the other hand, some feel that the owner of a building should be encouraged to protect himself against losses from earthquakes through mechanisms such as insurance. The paper by Kunreuther

*This material was prepared by Howard Kunreuther and Robert Whitman during the workshop committee sessions.

illustrates the effect of differing policies upon an individual's or community's response to measures that might reduce the hazards from earthquake.

The need for such studies is not new. This need was clearly recognized and very well stated in recommendation A-7 of the Task Force on Earthquake Hazard Reduction (Office of Science and Technology, 1970). The difficulties in carrying out realistic studies are also well known and well documented (Perspective on Benefit-Risk Decision Making, Committee on Public Engineering Policy, National Academy of Engineering, Washington, D.C.). The following discussion suggests a general approach for benefit-cost studies as applied to the earthquake hazard problem, and identifies steps that must be taken to make it possible to accomplish meaningful studies in the near future.

Framework for Specific Studies

In very simple terms, benefit-cost studies for earthquake hazard mitigation might involve the following steps:

1. Identify and inventory the physical units (residences, commercial structures, public facilities, etc.) involved in the study.

2. Establish the probability that the physical units will be subjected to different intensities of ground shaking.

3. Establish the probability that the physical units will experience various levels of damage, as a function of the intensity of ground shaking.

4. Determine the many different tangible and intangible costs associated with the various levels of damage.

5. Establish the different tangible and intangible benefits derived from alternative strategies for disaster mitigation.

These various bits of information are then combined and displayed in a manner that shows the benefit-cost tradeoffs. The actual manipulation of the information is a straight forward procedure on today's computers; the problems lie in assembling valid input information and in interpreting and using the results. One point, which will be repeated later, is worth emphasizing here: Attempts to assign dollar values to loss of life or other factors which are difficult to quantify should be delayed until the very final phase of any study. In many cases, it will be possible to show that a particular decision will be optimistic for a wide range of dollar values assigned to any one of these factors.

Assembling Input

Obviously, the results of a benefit cost analysis can be no better than the input. For a major analysis, assembling the basic input information can be a major task. The steps involved in collecting input for earthquake hazard studies are illustrated by Steinbrugge et al (1969) and Whitman et al (1972). Certain types of input collection efforts are fundamental to all studies, and will be mentioned here.

1. Inventory of Physical Units: There is a need for an inventory and data on all buildings that are located in disaster hazardous urban areas of the United States. The magnitude of the work of gathering this data is so large that this information should first be gathered for buildings with high socio-economic values - emergency centers, hospitals, and other occupancies with higher disaster support responsibilities. The appropriate federal

agency should make a feasibility study of how to undertake such an inventory which would provide data on the type of occupancy, type of construction and other details of construction needed to make viable potential risk evaluations. The magnitude of such an inventory could exceed the National Fallout Shelter Survey Program and, therefore, a feasibility and pilot study is recommended as a first step.

2. Seismic Risk: Adequate data concerning the probability of ground shaking of various intensities is essential to all benefit-cost studies. Such data should be compiled for all parts of the United States. Approaches to assembling such information are described in the review articles by Algermissen and Donovan, and specific recommendations for implementation of these approaches appear elsewhere in this report. These recommendations should be implemented in two steps: an initial study based on available information using crude measures of the intensity of ground shaking, and a long-range program using more complete and more quantitative measures of ground shaking. Funding should be provided as necessary to complete the initial study by 1974.

3. Damage Probabilities: Nominally similar physical units will not necessarily experience the same damage for a given intensity of ground motion. For each type of physical unit, studies are necessary to express the probability of damage as a function of intensity of ground shaking. Once the physical unit(s) to be documented in the pilot inventory study have been selected, a concurrent study of damage probabilities should be undertaken. It is recommended that this concurrent study be accomplished via contracts with engineering organizations experienced in earthquake engineering design and in study of damage during past earthquakes.

4. Associated Costs: While the costs necessary to repair or replace a damaged building are quantifiable, there potentially are many other costs associated with the health, safety and welfare of individuals and with the resources of the community and nation. These associated costs are in part short-run and in part long-run. Some are tangible and some are intangible but all are difficult to quantify. Some may be expressed in dollars; others are not easily expressible in dollars, and, at least in early parts of the study, attempts to express them in dollars should be avoided.

It is necessary to distinguish between the private costs perceived by the individual and the public cost borne by society. If an individual loses his life from a disaster, then the costs to society include not only the expenses of training and educating this person, but also his potential earning stream discounted back to the present. Normally, an individual does not consider all these factors and hence may implicitly value his life less (in dollar terms) then society does. Benefit-cost analysis as viewed by an individual may suggest a different decision than when viewed from the point of view of the community or the nation.

Data on the importance of a particular structure immediately following a disaster must also be considered. The concept of an importance factor should measure both the expected direct disaster losses to a given type structure in a particular locality as well as the indirect losses after a particular disaster if the building has lost its functional capability.

Data on the long-run economic effects associated with disasters also are essential. For example, the destruction of public facilities such as transportation and communication networks can severely affect the recovery process and the productive capability of a community and hence have indirect regional and national effects. Reconstruction priorities may be determined if such data is made available.

It should be possible to develop associated cost data that will be applicable to a wide spectrum of benefit-cost analysis. This will entail considerable study and research, and the effort should be funded at a level sufficient to ensure rapid progress.

Application of Results

A benefit-cost study is a useful tool for legislative bodies and executives. It also is a technique whereby a complex problem may be reduced to terms comprehensible by the general public. The output of the analysis must be in a form that permits comparison of different alternatives while enabling the legislator, executive or layman to make his own judgments on the relative importance of the different benefits and costs. As an alternative to attaching specific dollar estimates to factors that are difficult to quantify, some form of sensitivity analysis should be used. In this way, it may be possible to show that one type of policy alternative will be preferred to another over a wide range of dollar estimates.

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APPROACHES TO IMPLEMENTATION

by

Paul E. Baseler

Introduction

The conclusion drawn from recent studies by the Office of Emergency Preparedness that "Land-use and construction regulations containing strong disaster mitigation features can in the long run alleviate losses caused by natural disasters" [1] appears justified by investigation of current practices. Variations in requirements in Federal standards, state and municipal building codes and in the several model codes, [2, 3, 4, 5] especially those providing for design for resistance to earthquakes, and in criteria for the application and enforcement practices for those requirements by local jurisdictions indicate need for greater accord than now exists.

It is not our purpose in this paper to explore the differences in the technical requirements for protection from natural disasters. We are concerned here with the manner in which these requirements are formulated, the basis for their application by Federal agencies and state and local governments, and with the extent and effectiveness of their enforcement, since this may result in differing degrees or levels of protection being provided in the various jurisdictions.

Scope of Review

The term "natural disasters" covers a wide range of subjects: wind in varying velocities; earthquakes; flooding as the result of rainfall; landslides, etc. Those conditions in which these forces are applied to buildings or specific elements of construction are usually covered in building codes. Land-use codes generally are used to cope with flooding and landslide conditions by prohibiting or limiting construction of buildings in areas subject to such conditions. This division of coverage has influenced this study. The typical approach to determining land-use regulations (planning and zoning laws) and construction regulations (building and building service systems codes) differs:

> Planning and land-use requirements are primarily influenced by local conditions. Suggested extent of coverage, recommended text for regulation of various conditions, and criteria for applying these are provided by state, regional, or professional organizations serving that field of local governments. But complete ordinances suitable for adoption in local jurisdictions by reference, perhaps with minor modifications, are not widely available. Consequently planning practices and zoning laws are primarily products of local governments, usually developed by professional consultants on the basis of conditions peculiar to the community.
>
> In contrast to this, building regulations are primarily technical requirements based on accepted design criteria or specific

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engineering formulae, established standards, and proven practices. Many of the standards are produced by trade associations in the building industry, and some of these are written in code language and presented to be copied in codes or incorporated by reference.

Federal agencies provide standards or requirements for their own buildings or for buildings built by others with their assistance. Similarly, state governments provide standards or regulations for their buildings and those for which they provide assistance. State agencies also establish regulations for a variety of buildings for specific uses; and there is an increasing trend toward the establishment of state building regulations which are effective state-wide.

Complete codes, referred to as "model codes," are published and maintained by established organizations. These are adopted by local governments, either by reference as published, or with modifications to adapt them to local conditions allegedly peculiar to the particular jurisdiction; or they may be used as the principal basis for locally prepared codes.

This basic difference also affects the systems for securing modifications in the regulations. Changes in land-use requirements must be worked out with each individual jurisdiction. For mitigation of loss from natural disasters, this may be simplified by development of model sections for zoning laws and recommended criteria for determining their appropriate application to local conditions.

Changes in building regulations, especially those involving engineering concepts, as in the case of regulations dealing with mitigation of loss from natural disasters, may be approached through trade associations, Federal agencies, state agencies or the organizations sponsoring the model codes. However, success in effecting changes in these standards or codes does not completely eliminate the need to promote comparable changes with individual local governments and other authorities having responsibility for building regulations.

Investigation of conditions for this paper covered the following specific matters:

... typical land-use control practices,

... philosophies and basic principles for development and maintenance of model building code requirements,

... practices of local governments for adopting or applying the model building codes,

... activities of other agencies and levels of government in developing code requirements,

... the development of standards by a wide range of interests and methods,

... local practices for plan examination, field inspection and enforcement of land-use and building regulations, and

... possible impediments to effective enforcement.

Obviously all of these subjects are more general than the limited scope of this workshop which is concerned with conditions dealing with natural disasters; but, except in certain subjects, we must consider the overall practices since they apply to the particular conditions with which this paper deals.

Wherever it has been possible to confine the investigation to practices relating to regulations for protection from natural disasters, as in the case of plan examination and field inspection practices, this has been noted in the text.

Information regarding land-use control was obtained from the American Society of Planning Officials, nationally known planning consultants, and selected communities.

Information regarding the development of building regulations was obtained from the four code-sponsoring organizations: Building Officials and Code Administrators International, International Conference of Building Officials, Southern Building Code Congress, and the American Insurance Association; and from the secretariat of the National Conference of States on Building Codes and Standards. Information about standards was obtained from published procedures of the respective organizations as applied to codes activities as hereafter noted.

Information about local practices and possible impediments to effective enforcement was obtained from published articles and by contacting local jurisdictions in Missouri, Michigan, Massachusetts, Texas, and California. The local jurisdictions contacted were selected to provide a range of experience from small medium, and large jurisdictions subject to various conditions of exposure to natural disasters.

The Existing System

Any study of land-use and building regulations must be based on a clear concept of their purpose, the legal authority and responsibility for their adoption and enforcement, and the basis and manner of their formulation.

The primary purpose of such regulations is to safeguard the rights of property owners, protect the life and limb of those who occupy or come in contact with buildings, and secure the common good of the community by minimizing the risk of disasters which would result in loss of physical property, indirect economic hardship, or depriving the public of vital services. It is now widely accepted that such regulations are necessary for the public interest, but the distinction between public interest, consumer interest, and individual interest is not always clearly drawn nor readily recognized.

The legal authority for these regulations is vested in state governments [6]. However, because these laws so fundamentally affect the future of a community, the personal safety of its citizens and the property rights of individuals, for many years it was considered desirable that they be enacted and enforced by the unit of government closest to the people directly affected by them, consistent with the ability of that unit of government to establish efficient enforcement organizations. On this basis most states in the past delegated much of their authority for land-use and building regulations to local governments, setting up the limitations on that authority in the statutes governing the various types of units of local government [7]. However, the growth of metropolitan areas beyond municipal boundaries and the increasing extent of production of industrialized construction marketed nationally have led to an increasing trend to greater emphasis on the fundamental authority of the state governments.

(a) Land-Use Regulations

Land-use regulations are a relatively modern concept of government authority when compared to building regulations which have been in force for many years. As we have already noted, the prevailing systems for developing land-use regulations and building regulations differ. Since environmental considerations must be the basis for land-use regulations, the prevailing conditions have substantial effect on their nature in any community.

The influence of these prevailing conditions is reflected in the master plan for the area. Standard principles for planning -- determining the most desirable distribution, confinement, or prohibition of use of land within a municipality or area -- have been developed. To the extent these are followed by planners, they may result in relative uniformity of regulation of land-use.

Similarly, model sections for establishment of specific requirements are available. These are frequently used, in various combinations, to produce zoning regulations for a municipality or area. As a consequence, there is a measure of uniformity in land-use regulations.

There is a growing trend toward greater flexibility in zoning requirements by use of special permits. This allows the governing body of a municipality to permit a broader range of use in a particular area or zone through consideration of certain kinds of specific use on the basis of conditions applying to the particular case. This practice may result in variations in adjoining or neighboring communities.

The regulations of use of land to guard against loss from natural disasters usually is covered in the master plan. Primarily it consists of curtailing or prohibiting building on land judged to be susceptible to the effects of natural disasters. Soil conditions, underlying geological conditions, danger of flooding, and similar conditions may be the basis for prohibiting buildings in a specific area. Such areas may be devoted to park or recreational use or left to provide open space.

(b) Building Regulations

Building regulations in one form or another, date back many thousands of years. They have been used for many different purposes, and have been both praised and blamed for all kinds of conditions. Building codes are not standards of good practice nor handbooks of construction. They are laws prescribing the minimum that is acceptable for the safety of those who occupy buildings and to minimize the hazard of one property to adjoining properties [8]. Efforts to use them as guides for design or textbooks for construction have resulted in confusion and misunderstanding.

Unlike land-use regulations, building codes are primarily technical requirements. The background for code requirements is an extensive system of design criteria, engineering formulae, and established standards combined with records that have been kept through the years of building fires and structural failures resulting from conditions of use and natural disasters [9].

Standards are, therefore, one of the essential bases for building regulations. But there is a fundamental distinction between "standards" which are voluntary, and "codes" which are mandatory. If this distinction is not carefully observed in the development of code requirements, confusion may result and uniformity of regulations with the consequent protection afforded the public, may be seriously impaired. This is especially important in regulations for mitigation of loss from natural disasters.

For many years each local government had to rely on its own resources to formulate the detail requirements of building codes. Lack of understanding of true purpose for such codes and the nature of their restriction on individual property rights to accomplish community benefit resulted in their being considered more of a necessary evil than important function of government. The resultant formulation of such laws by each community, more or less hastily as the need arose to cope with some specific problem; and the scattering of them through the municipal code, often under the jurisdiction of various enforcement agencies, resulted in intolerable conditions.

In addition to these local regulations some states established codes varying in scope from complete control over buildings, usually excepting dwellings and farm structures, to regulations governing buildings for particular uses. Like local activities, many of these were developed as the need arose, and their enforcement was spread among various agencies.

To cope with this situation, beginning in the early 1900's, various organizations, agencies, and individuals tried a number of different methods in a conscientious effort to establish a proper balance between the numerous interests, often conflicting, which must be served by building codes. These methods included, among others, the development of code requirements

... by segments of industry which produced full codes or specialized portions of codes to cover their respective interests;

... by Federal Government agencies which attempted to produce or sponsor development of standard building code requirements for specific purposes or subjects;

... by State Governments which established regulations where local governments had not done so, sometimes also applying in jurisdictions where codes were in force, resulting in overriding or duplication of local authority;

... by organized industry-supported standards activities which attempted to apply the principles used for developing standards to produce standard building code requirements;

... and finally, by the model code system which provides opportunity for open debate of controversial proposals before volunteer committees which recommend approval, modification or denial of specific wording as submitted.

Various approaches for full consideration of viewpoints to arrive at a consensus or decision on controversial provisions were tried in these activities. One of the basic problems encountered was that of securing public agreement between the highly competitive segments of the industry. It was a solution to this problem through the model codes system that resulted in the progress which has been made in the development of codes.

Some of the former activities continue as an important part of this system. Industry standards (some of which are referred to as codes or code requirements) are developed by committees of researchers, professionals and experts representing the industry and potential users. Strong efforts are made to achieve consensus among the industry and users.

Federal standards and state codes may be drafted by the staff of the agency or by consultants. Drafts are published for review and comments by all affected organizations, interests, or individuals. The resolutions if conflicts may be accomplished through open forums, discussion with agency staff, or by legal processes.

The Model Codes System

Under the model codes system four codes have been generally accepted. These are frequently referred to as model codes in the sense that they are considered ideals as contrasted to the dictionary definition of "model" as a "pattern to be copied, or for imitation or emulation." They are intended for adoption by local governments by reference "without prejudice or local amendment except as necessary to adapt the code to the administrative organization of the community" [10] or to justifiable local peculiarities.

Three of these model codes are sponsored by organizations representing the officials who administer such codes in the various jurisdictions, with provision for participation by the building industry. Originally these codes were developed primarily to serve the needs of specific areas of the country and to some extent they still reflect influence by typical conditions of those respective regions. In those requirements affecting the use of materials and engineering design there is a continuing trend toward standardization among these codes.

The fourth model code is sponsored by an organization representing a segment of the insurance industry.

While there is still some difference in the technical requirements of these four model codes, one of the major differences is in the manner of applying certain of the requirements relating to natural disasters. Of the four nationally recognized model codes, two make earthquake requirements applicable to all buildings, with certain exceptions, in variable degree according to the recorded intensity of experience, in all communities where the code is adopted. Another establishes "loss of life or damage of buildings resulting from earthquakes," [11] as shown by "local experience or the records of the U.S. Coast and Geodetic Survey" [11] (which is now The National Oceanographic and Atmospheric Administration) as the basis for application of earthquake requirements. The fourth model code does not include earthquake requirements. To some extent similar differences exist in requirements for wind loads. These requirements may be further modified by local governments in adopting one of the model codes. The result is the possibility of appreciable variation in the level of protection afforded the public against the hazards from natural disasters.

To correct these conditions enforcement officials in state and local governments, and Federal agencies must be provided with criteria for applying disaster resistant requirements in their respective jurisdictions, and with rational bases for use of these criteria in an optimum manner recognizing the concept of risk and an uncertainty which is attached to the knowledge on any specific subject [12].

It is obvious, therefore, that there is some exigency for the establishment of uniform criteria for the application of regulations abating loss of life, property, and vital services in all areas on the basis of applicable contributing conditions. But regulations should only set the minimum requirements and must be sufficiently flexible to permit use of any materials or methods which have been proven to accomplish the prescribed level of protection. To maintain the proper relationship between code requirements and standards there is need for a critical look at the substance of the codes and the prevailing enthusiasm, especially in regard to engineering design, for incorporating methods and formulae or limiting factors in the text of codes. There is danger of this inhibiting the use of improved standards as they are developed.

The establishment of regulations, in itself, will not necessarily result in the use of advanced technology. The design and construction professions must be alerted to the advantages of advanced methods so that they will use them in the development of plans for buildings and structures. Design procedures and engineering formulae to aid and encourage the professions in the use of the most up-to-date techniques are properly the function of the professional and standards organizations. It is the responsibility of the professional practitioners to make use of such standards to produce the best possible solution to their client's problem, with full consideration to all contributing conditions and within the scope of applicable requirements for safety. The building official is responsible to check the professional's designs to determine that they are within the scope of the regulations.

The procedures of the three building officials organizations for maintaining their respective codes up-to-date with progress in the building industry are similar. The emphasis placed on these procedures by the several organizations varies; and the impact of the updating process on local regulations also varies.

Proposed changes come from a number of different sources: from local officials, usually members of the organization; from committees of the organization; from consultants usually representing some specific interests; and from the highly competitive segments of the construction industry.

These procedures provide for submission of proposed changes in the specific wording of provisions in the respective codes to the sponsoring organizations. These are processed and published by the organization's staff and are referred to committees established to study specific portions or subjects of the code. The committee system varies and this difference may have some effect on the final scope of the regulations.

After publication of proposed changes, such organization holds open hearings presided over by committees, where proponents and opponents of each change may present data and oral arguments in support of or opposition to the proposed text and its ostensible effect on their respective interests. Based on the information presented at the hearing and their collective judgment in closed session, the committees make recommendations to the membership of the organization for final disposition of the proposed changes. In two of the organizations the final decisions are made by vote of the public official members present in an open session of the organization's annual meeting; and other submits the committee recommendations to its entire public official membership for vote by letter ballot. After the membership has voted on the proposed changes, those which are approved are published as supplements to the codes. Complete new editions of the codes, incorporating all changes approved subsequent to the previous edition, are published periodically. Two of the organizations issue new editions every three years; and other issues a new edition every five years [13].

The procedures of the fourth organization for determining code requirements and proposed changes are primarily staff functions. Meetings may be called for discussion of proposals but final decisions are made by members of the organization's staff. The staff is guided by the policies and principle interests of the membership of the organization.

The ultimate effectiveness of this system would be attained if every local jurisdiction adopted one of these model codes "without prejudice or local amendment except as necessary to adapt the code to the administrative organization of the community [14];" and updated the code as it applies in its jurisdiction by prompt adoption of changes approved by the sponsoring organization. Unfortunately, this is more the exception than the rule.

While these codes may be adopted as published by some smaller communities, they are seldom adopted without amendment of some sections other than those related primarily to administration. In larger jurisdictions it is more likely that the model code will be used as a model for a completely independent code prepared by a local committee or firm of consultants. In moderate sized jurisdictions, well-meaning members of local code committees often persist in amending the model code as published so as to retain many of the requirements of the old local code with which they are familiar. They fail to realize that the model code was not written by an individual or group of individuals in an ivy covered tower, but represents a sincere effort by many organizations, agencies, and individuals to establish minimum regulations necessary for safety and maintaining equitable consideration of the many, sometimes conflicting and highly competitive interests involved [15].

It may be argued that these local amendments are necessary to adapt the model code to local conditions, but too often they are influenced by the personal experiences and individual prejudices of members of the local committee, or result from pressure by local industries or other interests.

On the other hand, where climatic, geographic, geological, or other natural conditions have a bearing on the application of standard or model requirements -- such as frost line depth, valleys or natural corridors subject to peculiar wind velocities, earthquake prone areas, etc. -- local amendments of model codes, or some other device such as an official rule or executive order, is necessary to fix the specific application of the model requirements in the individual jurisdiction. Such amendments should be limited, so far as practicable, to fixing the specific conditions to be guarded against; such as minimum depth of footings for protection from frost upheaval, minimum wind velocity or other load factor to be used in design for protection against the hazard involved. Except for this difference, the substance of the technical requirements in the model codes, which are based on standards or engineering formulae, should not be changed.

Although it is contended that there is no substantial difference in the technical requirements of the four model building codes, they all differ in format, in the manner and extent of use of standards, and in certain primary classifications which affect the application of many of the other code requirements. Under certain combinations of conditions these differences could result in appreciable variance in the protection provided the public by the different model codes. These variances may be further aggravated by the freedom of local governments to modify a model code in adopting it.

Code Enforcement Practices

In most jurisdictions land-use regulations and building codes are enforced by the local officer designated as building official. In examining plans for use of land or proposed buildings the official must first determine that the proposed use is in accord with the master plan of the community and in conformance with the zoning regulations intended to implement that master plan. If it is not, there is little reason for him to spend the time and effort to examine the details of construction until the question of use has been settled.

Few local departments charged with enforcement of such regulations are adequately staffed to fully discharge their responsibility effectively. In smaller jurisdictions individuals so engaged may be qualified primarily by practical experience; while in the larger governmental units both education, training and experience are usually prerequisites for employment.

Fortunately, the one time prevailing propensity for filling such positions with political appointees as a reward for support, without serious consideration of the appointee's qualifications, has largely disappeared. The establishment of recommendations for organization of departments, job descriptions, and personnel qualifications [16], has been a substantial factor in accomplishing this.

Increasing emphasis on education of incumbent building officials, promoted by the professional organizations representing those officials, has done much to improve enforcement practices and quality. But there is room for additional improvement in this activity, especially in reference to sometimes little understood requirements such as those referring to natural disasters based on infrequent exposure.

There appears to be a range in the effectiveness of enforcement between jurisdictions in highly populated areas and those where population is more scattered. This appears to be more the result of exigency than indifference or inefficiency. As a result, some regulations may be enforced more energetically than others.

Such investigation of enforcement practices as was possible in the time available in preparing this paper reveals that most officials check the basic information used by the designer of a building -- such as bearing value of soil, strength of framing elements, live and dead loads, wind load, etc. In the larger jurisdictions, where the staff usually includes professional engineers, design procedures may be spot checked. In smaller jurisdictions, the accuracy of design calculations may be often accepted on the integrity and legal professional responsibility of the designing engineer or architect.

In jurisdictions where the code specifically applies earthquake load requirements, the designs will be checked to determine that these requirements have been observed. In jurisdictions where the code does not specifically apply earthquake load requirements, especially in areas in a zero or number one hazard zone, it appears to be a generally accepted philosophy that wind load requirements provide sufficient protection against earthquake hazard. In areas where there is hazard of greater intensity, failure to require protection against possible damage may be serious.

In the investigation of the application and enforcement of earthquake requirements the practice in several communities in southeastern Missouri was checked. This area is rated as a number three hazard zone. In one community it was found that there were no building codes in force. In spite of the fact that in the past two years this community has experienced noticeable quakes it is claimed there has been no serious damage to buildings. In the town proper there appeared to be no buildings over two stories in height except the old stone courthouse. Outside the immediate town, however, a new industrial park complex is being developed in which there are several tall buildings and stacks. No detailed information was available concerning the design of these buildings. In two other communities checked in this area, both having formal code enforcement, one does not apply earthquake requirements and the other does.

Field practices for inspection of buildings varies in almost direct proportion to the plan examination practices. This appears to be strongly related to the training and experience of the personnel. In the concentrated portions of a large metropolitan area, field inspection is more comprehensive than it is in small jurisdictions on the fringes of these areas or removed therefrom. In one county investigated, which is not yet heavily populated but is included in a large metropolitan area and in the direct line of growth in that area, it was found that little attention is paid to the structural design of buildings and field inspection is cursory, even though building codes have been in force in the jurisdiction for well over five years.

The effectiveness of inspection and enforcement practices appears to be influenced largely by internal conditions such as adequacy and quality of staff. The overlapping of authority between agencies and the several levels of government is more of an harassment to owners and designers than an impediment to effective enforcement. For example, an architect designing a nursing home must examine applicable requirements of Federal, state, county, and local governments, both for their effect on construction and for health, safety and sanitary conditions. He must determine the most restrictive of these and design the facility to meet them. Often this may necessitate time-consuming direct contact with a number of different agencies at each level of government in an effort to resolve conflicting requirements.

Cumbersome procedures for prosecution of offenders -- crowded court dockets, continued delays, suspended sentences -- often seriously impair the effectiveness of enforcement of codes. Local influences may have some adverse effect, especially where there is opposition to regulations or when appeals to local boards or commissions are involved.

Enigmas in the Existing Codes System

No criticism of the present codes system in the United States can detract from the impressive progress that has been made under its influence. It is built upon the doctrine of free enterprise which is the essence of the American Economy. The evolution of the present codes methods is a fascinating and inspiring study. It is hard to imagine the chaos that would exist, with the modern materials and advanced techniques now available for building, if we had to depend on each local government developing its own regulations instead of the development of model codes or other broad based regulation systems through procedures which provide open forums for discussion of opposing views.

In spite of this, building codes and related regulations continue to be blamed for a wide variety of ills. It appears evident, therefore, that good as the present system is, there must be some conditions or methods which could be improved. One of the main problems in the past has been a certain measure of complacency with the present system. Those involved seem to feel that they have a satisfactory working arrangement with each other and they are skeptical of any efforts to change it.

Some of the other conditions under the present model codes system which may be worthy of study are:

... It depends heavily on voluntary committees of men whose principal responsibility is to the local communities they serve. While these men are dedicated to their respective organizations' activity, much of their organization work must be done on their own time and must be shared with other responsibilities also having claims on that time.

... It does not use available industry and professional expertise to the best advantage. Instead, it incites and encourages the ruthlessly competitive nature of the several segments of the industry to seek preferential consideration for their respective vested interests.

... In the limited time available, the volunteer committeemen can do little more than review the selected competitive information presented to them by industry interests instead of probing into the technical data on which sound regulations should be based. The result too often is a compromise between those competitive interests based on the scope of personal experience of selected individuals comprising the committees.

... It lacks specific policies fixing the proper scope for codes, definition of terms relating to code work, and establishment of a proper relationship between standards and codes. The incorporation of design data and engineering formulae into the codes as requirements, instead of confining regulations to performance requirements and acceptance criteria, leads to variances which may significantly affect the protection provided to the public.

... It lacks meaningful communication with related governmental agencies to encourage acceptance of the model regulations and prompt use of available improvements in code requirements.

Many of the individuals who serve on committees or vote on code requirements cannot or do not follow the organizations' recommendations in their own jurisdictions.

... It lacks clear definition of the respective proper roles for the various levels of government -- federal, state, county, local -- resulting in overlapping of jurisdiction and superimposing of regulations.

The code change cycle varies among the several organizations from nine to twelve months from final date for submission of proposed changes to date of official action. Time for publication of supplements varies from two to four months. Since the time for submission of proposed changes varies from January to August, and the time for final action varies from September of one year to June of the following year, it could require approximately eighteen months to process a specific change through all of the model codes change cycles, assuming that the proposed change is processed within the cycle in which it is submitted. Complicated or controversial changes may be held over from one code change cycle to another by any one of the organizations, thereby substantially increasing the time for total acceptance.

In 1971 one of the model code organizations modified its procedures to place consideration of proposed changes in its codes in alternate years, and banning consideration of changes every fifth year when new editions of its codes are scheduled. Under this schedule a lapse of three years may occur in consideration of changes for that code.

There are other conditions in the present codes system which might be improved with study so that it would better serve the public interest without bias or prejudice. The existence of these conditions in the present system is not an indication of indifference, but rather is the result of the heavy demand on the participating agencies to cope with the accelerated advancement of new technology, although the situation is aggravated by jealousies between the various interests involved. There is need for the establishment of an unbiased coordination of the existing efforts, in a manner commanding the respect of all concerned, to use the vast resources and expertise available in the best possible manner.

The nature of the governmental structure in the country, the basic principles of the economic system, and the wide variety of conditions and interests which are affected by land-use and building regulations are not conducive to voluntary cooperation. The resultant rivalry between the several interests, between levels of government, and among government agencies has long been one of the major problems in code activities. There has not yet been established a meaningful effort toward coordination of the activities and programs of the many interests which presently are endeavoring to achieve unity, each in its own way. Fortunately, very recent activities, discussed later in this paper, show recognition of this and offer prospects for some improvement.

Problems in Effecting Improvements

As has already been pointed out, improvement of building regulations relating to mitigation of loss from natural disasters will not, in itself, necessarily result in the use of advanced technology. However, the regulations must be such that improved technology which has been proven to accomplish the prescribed level of protection, may be used.

The first step in effecting changes in code requirements covering natural disasters, if needed, is the development of technical data from which the basic principles for code provisions can be established. From these basic principles, performance criteria fixing the acceptable levels

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of safety should be developed. In the case of safety from the hazards of natural disasters the application of requirements may vary from area to area depending upon the magnitude of the hazard in each area.

Coincident with this code activity, and perhaps preceding it if the basic principles involve engineering design criteria or formulae, the standards organizations responsible for development of standards must be contacted and the necessary criteria and formulae established. Promotion of the use of these standards may be accomplished through the organizations representing the design professions.

With performance criteria established, acceptable levels or risk fixed, and engineering design criteria or formulae developed, performance requirements can be prepared for protection against the specific hazards. The development of these and of modifications in state or local codes, or in the model codes, is governed by the established procedures of the government, agency or code-sponsoring organization. If the requirements affect any of the competitive segments of the industry, opposition or counter proposals may be expected.

Changes in a state code may become effective immediately upon adoption or at a date fixed in their adoption. After changes have been approved by a model code organization, there remains the need for securing acceptance of them by local jurisdictions which are operating under that code. While the sponsoring organization promotes the use of approved changes, the most effective method for securing meaningful use of them is by direct contact with local governments. There are also those individual jurisdictions which have not specifically adopted or used one of the model codes. Each of these must be contacted, usually with specific changes in their individual code.

Present Efforts and Recommendations

Since many of the conditions contributing to the hazards from natural disasters extend beyond the arbitrary boundaries of local jurisdictions, there is need for voluntary cooperative action or control by area-wide regulation. This is also desirable where the development or expansion of one community may affect natural conditions so as to cause flooding or in some other manner be detrimental to other communities in the area or communities some miles distant. Conditions extending beyond the borders of one community into the area of an adjoining community should be similarly controlled in both jurisdictions.

It is unrealistic to expect this to be accomplished by unilateral action of the affected communities. Some states have recognized this and have taken steps to set up land-use guideline regulations. In other areas metropolitan planning agencies have been established to coordinate land-use requirements in all of the communities in the area. The same principle has not been widely applied to building regulations.

Recognizing this, and the need for encouraging the use of new materials and advanced technology in building construction, the National Bureau of Standards some years ago assisted in the establishment of the National Conference of States on Building Codes and Standards. This was done in response to requests from several state governments which began to recognize their basic responsibility for such activities.

In the reports of the committees of this organization, and official actions and resolutions approved at its recent meeting [17] the National Conference of States on Building Codes and Standards proposed activities intended to increase unity in building regulations, pledged its cooperation to this end, and called on others to help. Some of these proposals were:

... greater cooperation among Federal agencies developing, promulgating or enforcing rules and regulations affecting building construction so as to eliminate the present rivalry, duplication of effort and superposing of regulations;

... greater cooperation among state agencies having authority and responsibility for regulation of buildings so that there may be meaningful reciprocity between the states in the acceptance of products and systems manufactured and marketed on a national basis;

... greater unity in building regulations by the adoption of one of the four nationally recognized model building codes, without change, by state agencies which have the authority to adopt mandatory or optional state building codes.

In support of these resolutions the National Conference of States on Building Codes and Standards has activated committees on

... Standards and Evaluation, dealing with standardization of testing, evaluating, and continued quality control of building products;

... Education and Qualifications, dealing with building regulation enforcement personnel;

... Management and Regulatory Procedures, dealing with the organization for building code enforcement; and

... Reciprocity, dealing with the mutual acceptance of both enforcement personnel and approved products among the states.

These activities recognize the responsibility of state governments for regulations providing for safety to the public in the use of land and the construction, use and maintenance of buildings. They acknowledge the value of the model codes system and the advancement toward uniformity which has been achieved through it. They are directed to bringing about unity in the preparation, maintenance, and enforcement of land-use and building regulations, administrative techniques and personnel qualifications at the local level through the exercise of state authority.

The present emphasis on the authority of local governments for building regulations was based on conditions when it seemed that home rule could better serve the people through the cities because they were widely separated and the economy was largely local. But the country has outgrown these conditions and the bulk of the population is now concentrated in metropolitan areas. This change necessitates readjustment in the codes activities of the country.

The model code organizations anticipated this many years ago by forming regional associations which produced model building codes suited primarily to the needs of the regions they served [19]. These are the major elements of the present model codes system.

These organizations have recognized the need for closer coordination of their respective activities to cope with the changing conditions in the country. In the past few years they have voluntarily increased cooperative efforts among themselves. Through these efforts they have jointly:

... produced a model One and Two Family Dwelling Code,

... developed a proposed model Residential Rehabilitation Standard,

... begun work on a similar standard for Mobile Homes,

... reorganized the Joint Committee on Building Codes and strengthened its activity through the Model Codes Standardization Council,

... cooperated in the development of model standards for evaluating industrialized building systems,

... and, most recently, have revived former cooperative efforts by forming a Council of American Building Officials to provide a unified voice on matters of national importance.

Under the Model Codes Standardization Council work has progressed on proposed standardization of [18]

... The Format for Building Codes

... Definitions Used in Building Codes

... Classification of Occupancy

... Types of Construction

Within the past few months there has been a major breakthrough in cooperation by a number of groups. This involved the consolidation of independent efforts by the Model Code Groups, the National Conference of States on Building Codes and Standards, the National Association of Building Manufacturers, the Department of Housing and Urban Development, and the Council of State Governments to produce a Model Manufactured Building Act. This has been published by the NCSBCS, and when adopted by state agencies having authority and responsibility to implement state standards, will result in acceptance of manufactured buildings on a nationwide basis regardless of where they are manufactured [20].

The activities of the National Conference of States on Building Codes and Standards offers promise of progress in the standardization of code requirements. If the state authorities make use of the model building codes as recommended, considerable progress can be made in a short time. But the state activity does not fully cope with the problem of large metropolitan areas which overlap state boundary lines. There is a growing recognition of the importance of involving Regional Councils of Government in land-use planning and coordination of building regulations [21].

In addition to the activities of local and state governments which have been described, a number of Federal Government agencies have rules, regulations, codes, or other activities which affect the design and construction of buildings. Some of these are:

Department of Housing and Urban Development -- FHA

Minimum Property Requirements
Operation Breakthrough Guide Criteria

Department of Health, Education and Welfare

Health Care Facilities
Educational Facilities

General Services Administration

Federal Specifications
Guidelines and Requirements for Federal Buildings

Department of Labor

Occupational and Health Safety Act

These Federal activites generally focus on standards for buildings constructed by Federal agencies for their own use or constructed for other use with Federal assistance. Controversies have arisen concerning these requirements in areas where they are at variance with local codes, which may be more or less restrictive than the Federal standards. Although in some cases the application of local regulations has been insisted upon by local authorities, the threat to withhold Federal support for the building activity has been effective in encouraging use of the Federal standards.

These controversies have aroused concern that Federal standards will be imposed upon local codes covering building that is not Federally assisted, and there have been efforts by some Federal agencies to impose such standards, indeed to impose language which is not documented by widely accepted standards. This activity is of extremely dubious constitutionality.

The cumbersome, time consuming necessity of having to prepare specific code requirements for several model codes; promote their adoption by several organizations, and examine their application in local jurisdictions at the state level and in the rules and regulations of numerous Federal agencies; and the need for the repeated pitting of one interest against the other, is wasteful of valuable expertise and human resources. It results in substantial delay in effecting modernization of regulations and providing the public with adequate protection.

It is obvious, therefore, that the present model code system can and should be improved. Experience has shown that any effort to accomplish this must be undertaken so far as practicable within the framework of the system. It can be done without drastically affecting the several organizations or interests involved in the present system, although their respective roles and particular activities may have to be adjusted. The principal aim should be a coordination of those activities which now are a duplication of efforts, providing opportunity for greater emphasis on other activities.

The shift of basic effort from preparation of precise code requirements by competitive industry interests for four model codes, local codes and state and Federal agencies, to the development of universally accepted basic principles for such regulations will no doubt reveal a need for specific basic research for the development of standards. Some of these standards may differ from the type of standard with which we are accustomed to dealing. Material standards, engineering standards, test standards, and safe practice standards are well known. These have been developed under well established standards procedures maintained by recognized organizations. But in addition to these standards, the development of basic principles for code requirements requires

... standards of safety -- minimum levels of safety, in clearly defined terms, desirable in buildings to protect the public against the hazards involved;

... evaluation criteria -- the basis for determining the performance of a material, product, or assembly by analysis according to accepted engineering design formulae or by test; and

... performance criteria -- the basis for establishing desirable levels of performance for materials, products, systems, assemblies, or elements of buildings.

The need for standards of safety is especially important in establishing the basic principles for code requirements dealing with natural disasters.

The magnitude of hazard differs from area to area in proportion to measurable severity of exposure. This necessitates the gathering of pertinent data; assembling it in usable form; and establishing definitions for words and terms to be used so as to facilitate communication in regard thereto.

The first step in applying these principles of improvement to the standards and model codes systems should be the establishment of a council independent of the various levels of government and the organizations currently involved in the state and model code system; but having their full respect, confidence, cooperation and participation. This council should sponsor the coordination of all codes activity affecting the construction of buildings and their maintenance and use. It should provide liaison with research and standards activities, and with Federal, state and local government representatives.

This council should be made up of individuals from all interests concerned with building codes and the development and use of new products or construction systems or techniques; selected from the several participating interests on the basis of their qualifications and apparent ability to perform constructively and without bias in the activities to the council.

The purposes of the council should be to pull together existing data, and sponsor development of new data where necessary --

... To recommend a national policy concerning the building codes system clearly stating: The purpose of codes; the codes to be considered; the proper role for various levels of government; the responsibility of governmental agencies; the desirable administrative organization and personnel qualifications for code enforcement; and the basis for determining code requirements.

... To develop a positive program for implementing that policy, providing for: review of the existing codes activities and problems therewith to determine how they may be coordinated and improved; standardization of administrative techniques and personnel; prompt dissemination of information to all concerned; and public relations and public education activities.

In the development and implementation of this program the substantial accomplishments by the code-sponsoring organizations, and the responsibilities of the several levels of government must be recognized. The object must be to enhance these, coordinating them to minimize duplication of effort, and expanding them where necessary to make use of the resources of manpower and expertise to the best possible advantage.

Summary

It is concluded from this study that there is need for greater uniformity in the regulations presently in force for protection against loss from natural disasters. There is special need for more precise criteria for applying such regulations in individual jurisdictions; particularly with reference to the hazards from earthquakes. To accomplish this, a rational system or formula is needed for establishing a balance of risk since absolute protection may be impracticable.

The evolution of the development of land-use and building regulations in the United States has produced a codes system that is responsive to national needs. Although the codes activity has been blamed for all sorts of ills besetting the construction industry and the public, careful examination of these has shown that they are more the result of isolated incidents than gaping faults in the present system. The organizations which have been active in the development of this system are deserving of admiration and must be given substantial credit for their accomplishments.

The envolvement of the elements of this system on a regional basis, has resulted in certain conditions that require considerable duplication of effort. This has also resulted in some variations which make compliance with the codes cumbersome and sometimes unnecessarily costly under the present shift of emphasis to national production of materials and products, including major components of, or complete buildings.

There has been a growing awareness of this by all concerned and in response to it the model code organizations have voluntarily increased cooperative efforts among themselves. They have jointly issued a model code for residential construction; they have collaborated on the development of a proposed residential rehabilitation standard; they are working on a cooperative effort for evaluating industrialized building systems. For years they have worked together in the Joint Committee on Building Codes, now the Model Codes Standardization Council, comparing, studying, and attempting to resolve some of the fundamental differences between their codes. But these activities have been established as individual elements of the program as the need for them has arisen, rather than as part of a well planned, long range program for improvement of the present codes system. In effect, they duplicate one of the major problems, which is compounding of effort.

State authorities also have recognized the need for greater responsibility on their part and have formed the National Conference of States on Building Codes and Standards. This organization has made an effort to develop a meaningful working relationship with the sponsors of the model codes and with those agencies or organizations which develop standards.

In recent months these cooperative efforts have resulted in a substantial breakthrough producing a Model Manufactured Building Act. This development should result in the acceptance of sound new building techniques on a nationwide basis.

Although there was Federal agency participation in this, there appears to be less effort at the Federal level to coordinate the activities of the many agencies involved. Perhaps one Federal Government agency should be designated as the control agency for all Federal Government rules, regulations, codes, standards, etc. To a large extent this has been done with Federal Specifications, why can it not be done with codes and standards?

Within the legislative branch of the Federal activity there is also need for much improvement. A Legislative Clearinghouse through which all bills may be checked to eliminate multiplicity, may be the answer.

Above all, there is need for an unbiased, critical examination of the present codes system to determine what improvements can be made in it and how these can be best accomplished without discrediting or destroying the impressive progress which has already been made. The need for this is significantly evident in requirements regarding mitigation of loss from natural disasters over which we have no finite control. Such an activity, to be successful, will require overcoming the complacency with the present system, and the foregoing of individual interests in a truly cooperative spirit to produce results that will genuinely serve the public interest and provide the safety to which all are entitled without imposing unnecessary restraint or unnecessary expense.

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EARTHQUAKE HAZARDS FOR BUILDINGS

by

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I. Introduction

When the ground is shaken by an earthquake, a building resting on the ground must respond to the adjacent ground motion. Damage produced by earthquakes has provided some insight into how buildings respond. This information has been greatly augmented in recent years by theoretical and experimental studies of structural response to random vibratory loadings similar to those produced by earthquake motions.

The ground motion to which the building responds can be considered from two different aspects. These are, the general geographic region in which the building exists, and the close-in environment where the building can be influenced by the local conditions. This review of the earthquake loading on buildings considers the effects of these two different aspects and how they are related. The areas where significant knowledge exists are described and illustrated by examples of their use in design practice. Areas where information is deficient are also outlined. Some areas where a research effort could efficiently provide useful information are described.

II. Regional Effects

Seismic Environments

Consideration of the regional aspects of earthquakes includes the disciplines of geophysics, seismology, geology, and engineering. The more frequent occurrence of earthquakes in some areas than others is readily apparent from any review of seismological data. The instrumental epicenters of events with Richter magnitudes greater than about 4 between 1961 and 1967 were compiled and plotted by Barazangi and Dorman (1969). The data have since been extended through 1969, and a map has been published by the Environmental Science Services Administration (N.O.A.A.) of the Department of Commerce. This map has been reproduced in Figure 1.

The emerging field of plate tectonics has provided considerable insight to aid the understanding of the major global sources of earthquakes. Plate tectonics and the associated relationships of sea floor spreading and continental drift are beyond the scope of this review. A reference by Bullard (1969) is suggested as a general introduction for the interested reader. Concentration of epicenters around some of the major continental blocks and along the mid-ocean ridges can be readily seen in Figure 1. The large concentration of events on the west coast of the contiguous states and the southern coast of Alaska can be seen in Figure 1. The heavy concentration of events in the area south of Anchorage, Alaska, includes the main event and aftershock sequence of the March 27, 1964 Prince William Sound earthquake.

A linear relationship between large and small magnitude earthquakes, when the number of earthquakes are plotted logarithmically, was demonstrated by

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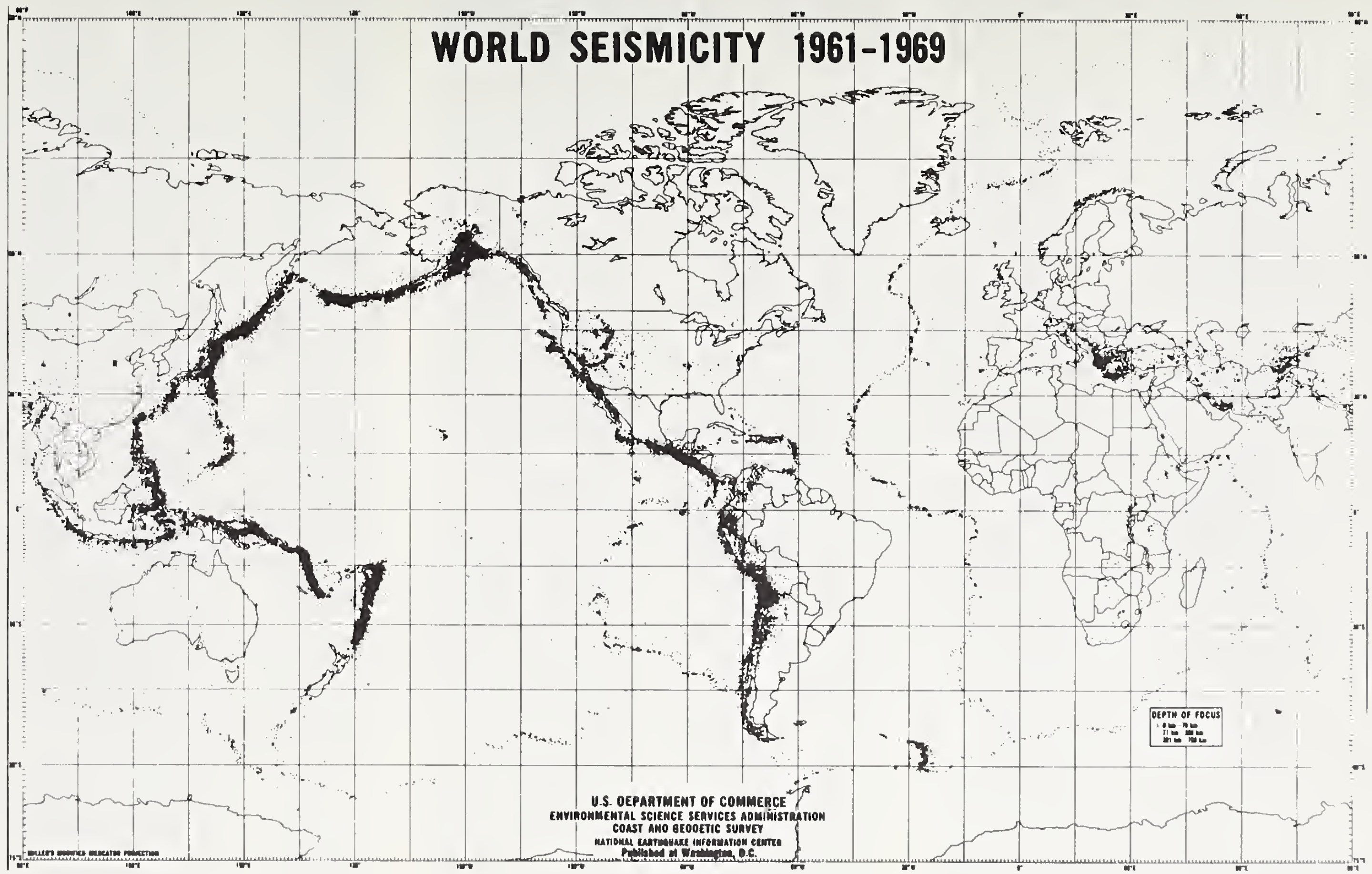

FIGURE I

WORLD SEISMICITY 1961 - 1969

Gutenberg and Richter (1949). A recent compilation by Evernden (1970) has shown that the relationship is valid for seismic data obtained from all seismic regions. A representative plot of this type is shown in Figure 2 for data obtained over a 163-year period within 100 kilometers of the downtown San Francisco area. Where historical data are available, records of small earthquakes can be of considerable value in estimating the level of the seismological hazard.

The risk of exposure to earthquakes is greater in regions where earthquakes occur most frequently. This simple premise is complicated by two separate factors. The first of these factors is the known occurrence of catastrophic earthquakes in other portions of the United States, such as the 1811-1812 New Madrid earthquakes in Missouri and the 1886 Charleston, South Carolina earthquake, where the overall seismicity is lower.

The second factor is more complex. Unlike floods or wind storms which either take time to develop or give indications of their approach, earthquakes occur without warning. The lack of warning makes a severe earthquake a more frightening event than it might otherwise be. The relative infrequency of major earthquakes and their occurrence without warning when combined with the lack of knowledge of the mechanisms which cause them have delayed the approach to seismic design on the same logical basis using the probable risk of exposure as is applied to other variable phenomena such as floods.

The concept of dual earthquake levels for design has evolved over the last decade. This was developed initially for nuclear power plant design requirements. The upper level represents a large earthquake with a low probability of occurrence. Should this earthquake occur, non-critical parts of the plant may be damaged, but the plant would be able to close down without any potential radiation hazard. The lower level represents a smaller earthquake with a higher probability of occurrence. The plant would be expected to remain operational during and following this smaller earthquake (Newmark and Hall, 1969).

The seismic design criteria for nuclear plants are much more stringent than those applied to other structures, and their direct use for other structures is not recommended. Nuclear power plants, where consequences of an accident are greater, are designed much more conservatively than buildings. This difference can be illustrated by considering the roughly estimated levels of probability of failure in a modern building. This probability is of the order of 10^{-6} (Ang and Ellingwood, 1971). For nuclear power plants, the unpublished probability has been estimated to be 10^{-9}, one thousand times less than the number for buildings.

The dual earthquake concept when applied to buildings represents a more complex interaction of probability, design conditions, and risk levels. However, there are within the present state-of-the-art differences of opinion relating to the choice of the level of earthquake motion for design. For this review paper, the two design level earthquakes are simply designated, Level I and Level II.

The Level I earthquake represents an event with a low probability of occurrence or a long return period. Such an event has only a low probability of occurring during the life of the proposed structure. Should such an event occur, it is anticipated that the building would respond with excursions of the structural frame into the yield range. The structural behavior should be such as to minimize the potential loss of life from collapse or failure of non-structural elements. Damage to the building could be extensive enough to require major repairs.

The Level II earthquake represents an event which has a moderate probability of occurring during the lifetime of the structure. The response of the structure to the Level II motion should result in only minimal damage to non-structural elements. Although not required, it is prudent to assume that the structure will respond within the elastic range. The economic relationship between the possible costs of minimizing non-structural damage and the possible costs of repairs of this damage and the effects upon the public welfare should be balanced for the Level II earthquake.

Procedures have been developed where the probable level of exposure to earthquakes can be evaluated from the seismic history of the region. Interpretation of the seismic history must be made with recognition of the associated geologic and tectonic information. The computed exposure can be expressed in terms of probable levels of seismic motion which are in turn used to estimate response levels of a building and the possible consequences of the response. Comparative consideration of the costs of repairing possible damage, and costs of preventive design, and the effects upon the public community, allow the risk analysis to include an economic evaluation.

In this paper, a suggested procedure is described in some detail with alternative methods described where possible. It is recognized that the full procedure described cannot be economically applied to the design of smaller buildings. Some suggested improvements and modification to the lateral force design code are suggested. It should be stressed that the use of present codes as the sole basis for design of a major building is not in accordance with the current-state-of-the-art. However, observance of a design code is usually a minimum standard required by regulatory agencies.

The science of instrumental seismology dates from the late 19th Century. The first recording seismographs in the United States were installed at the University of California and the observatory on Mt. Hamilton in 1887. The history of large earthquakes is much longer. Some Chinese records have been compiled representing 3,000 years. In the United States, historical records range from about 70 years for Alaska to 400 years for areas along the Atlantic coast. Early seismic histories often must be compiled from old newspapers, books and other records. The early records are quite subjective in nature but provide valuable extensions of more recent instrumental records of both large and small events. Compilations of old records have been made and published for some portions of the United States. An excellent example of such a compilation is the documentation of earthquakes on the Pacific Coast of the United States between 1769 and 1928 by Townley and Allen (1939).

When combined with detailed geologic data, the probable earthquake source mechanisms can often be identified. Where not identifiable, it is often adequate to assume that the source may be randomly located within the region.

Risk Evaluation

Seismic risk should be expressed in terms of return periods such as is done for winds and floods. The basic seismological information obtained from the seismic geology and the available seismic record should be converted into terms suitable for use by the engineer. Magnitude information or even Modified Mercalli intensity values can be expressed as return periods or probability curves rather than as ill-defined terms such as "probable maximum" or "lifetime earthquake." For a well-balanced economic design it is necessary to know how quickly the risk of occurrence decreases as the intensity of ground motion increases. It has been found empirically by several investigators (Nordquist, 1945; Milne and Davenport, 1965; Dick, 1965) that the distribution of maximum magnitude or intensity in a region can be represented by an extreme value distribution of the Gumbel (1958) type. Cornell (1968) showed that the double exponential distribution, which is widely used in engineering studies of extreme events, could be derived from accepted seismological relationships.

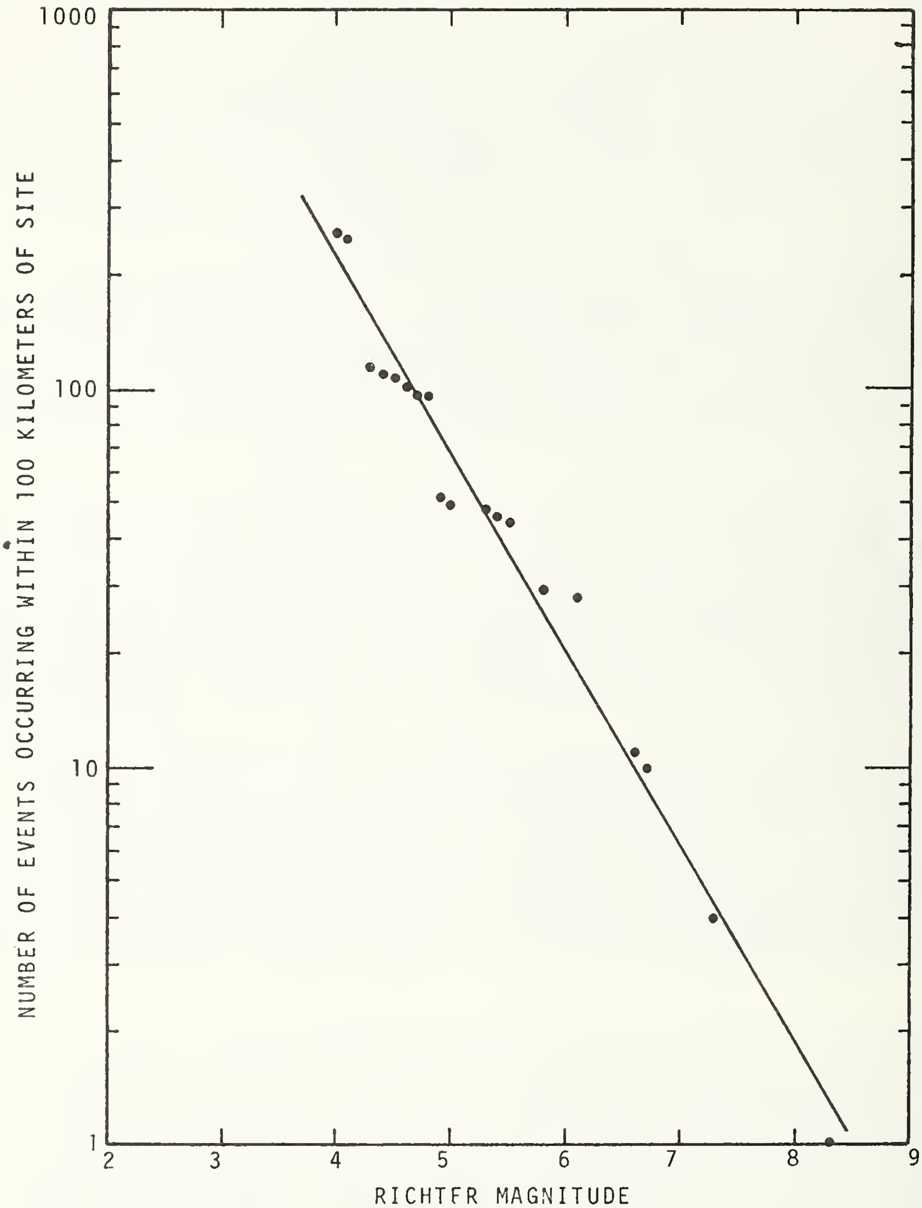

FIGURE 2 REGIONAL SEISMICITY 163 YEAR RECORD

(MAGNITUDES GREATER THAN 4.0 WITHIN 100 KILOMETERS OF SAN FRANCISCO)

The simple extreme relationship was extended to a continuous range of intensities of motion and return periods over regional areas (Milne and Davenport, 1965, 1969; Lacer, 1965). Correlation with the known seismic geology was difficult with these procedures. A somewhat different approach giving direct relationships between various assumed sources of earthquakes was produced by Cornell (1968). This procedure has since been extended to consider restrictions such as finite limiting magnitudes on different sources (Cornell & Vanmarcke, 1969; Cornell, 1971). The Cornell procedure and the procedure developed by Milne and Davenport for the seismic zoning of Canada have both been used by the writer on the San Francisco example. Where the Cornell procedure is restricted to assumed areal sources only, and the same acceleration attentuation law is used in both procedures, the results are comparable. In addition to giving similar results to the method developed by Milne and Davenport in regional cases, the Cornell procedure is distinctly superior in areas where the fault locations are well defined.

The San Francisco epicentral data were plotted as shown in Figure 3. For a seismic risk evaluation, the events are more easily considered as originating from distinct sources. A simple four source model was chosen. The four sources were three fault zones, the San Andreas, Hayward and Calaveras, and an areal source to the north where the fault locations are less distinct. Details of the source modelling away from the immediate vicinity of the site of interest do not greatly affect the risk estimates. The events located within the bounds shown on Figure 3 were assigned to the sources shown. It was also assumed that the maximum magnitude of possible events from all sources except the San Andreas Fault would not exceed 7.0. On the San Andreas Fault, events with magnitudes as large as 8.5 might occur.

Cornell and Vanmarcke (1969) showed that the risk estimates were very sensitive to the slope b in the Gutenberg and Richter equation ($\log n = a-bm$; where n is number of recorded events and m is the magnitude) and to the attenuation equation values. Evernden (1970) assembled a worldwide set of data relating magnitude and numbers of events. Where data are complete, the value of the coefficient b usually has a value between 0.65 and 1.4. The data plotted in Figure 2 have a b coefficient of only 0.53. It is believed that the smaller value for b in this example is caused by the incomplete recording of the smaller events in the almost 100-year portion of the record prior to instrumental seismology. As the historical data were used to assign numbers of events to each source mechanism, the coefficient computed from the same record was used in the numerical procedures.

Motion Attenuation

Attenuation of acceleration has been studies in several different ways. Cloud (1963) and Cloud and Perez (1971) have suggested an envelope relationship between distance from the zone of energy release and the maximum acceleration independent of the magnitude of earthquake. Other studies have included the magnitude of earthquake as a parameter (Gutenberg and Richter, 1959; Esteva and Rosenblueth, 1963; Blume, 1965; Kanai, 1965; Milne and Davenport, 1970; Esteva 1970; Schnabel and Seed, 1972).

The 1971 San Fernando earthquake provided a valuable contribution to earthquake engineering, as more than 100 strong-motion instruments recorded accelerations from this one event. The large data set from the one event allowed a study of attenuation free from the additional effects of source mechanism, event magnitude and regional tectonic variations. The author has also accumulated strong-motion acceleration data from the United States, Japan and world earthquake lists into a data set of more than 500 instrumental acceleration values. These data are shown in Figure 4. Curves in Figure 4 represent the least squares fit between acceleration and distance where the distance is modified by a constant value of 25 kilometers (Esteva, 1970). The data were sorted and ranked after the method of Gumbel (1958) to obtain

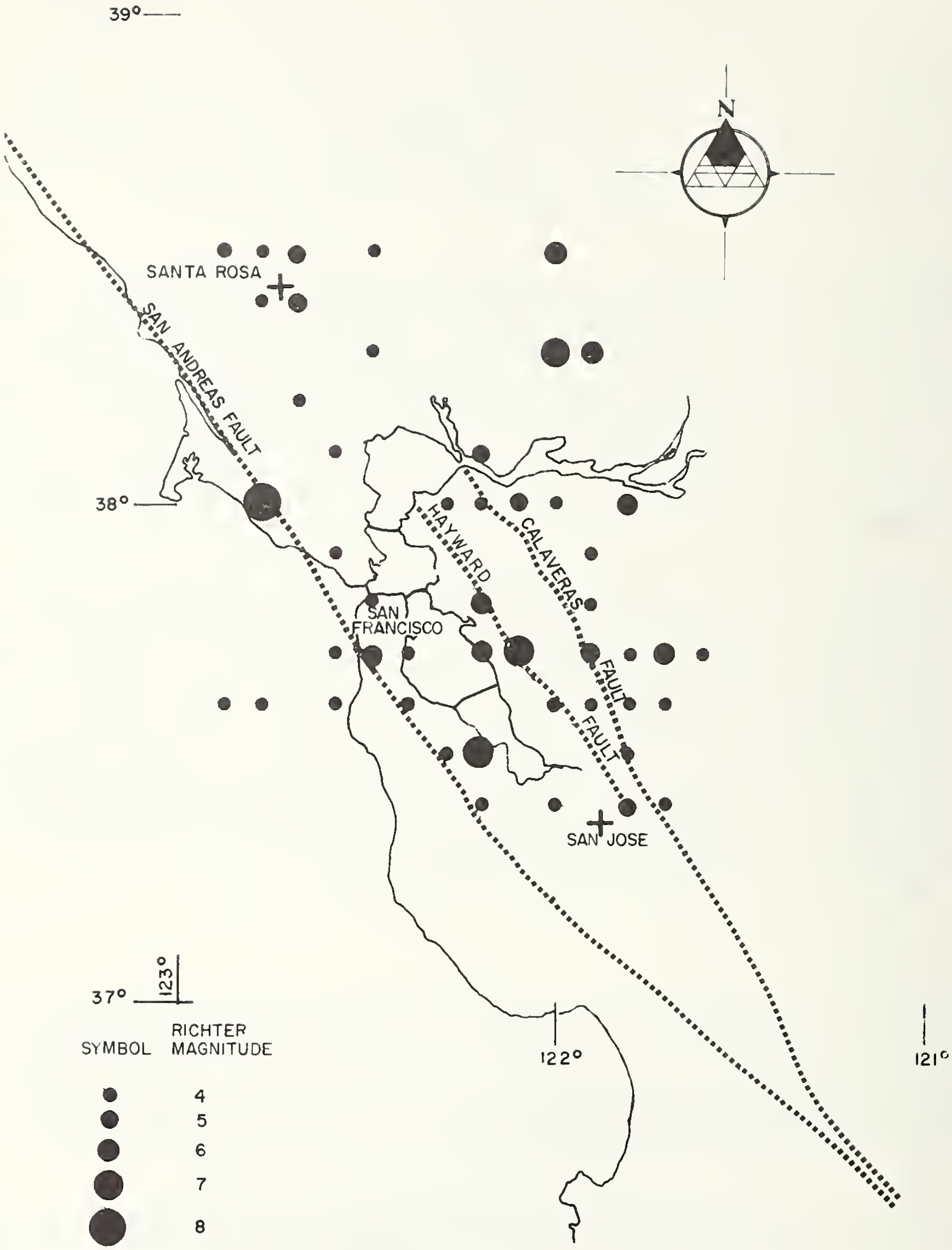

FIGURE 3

EPICENTER MAP

(HISTORICAL INSTRUMENTAL OR ESTIMATED EPICENTRAL LOCATIONS WITHIN 100 KILOMETERS OF SAN FRANCISCO)

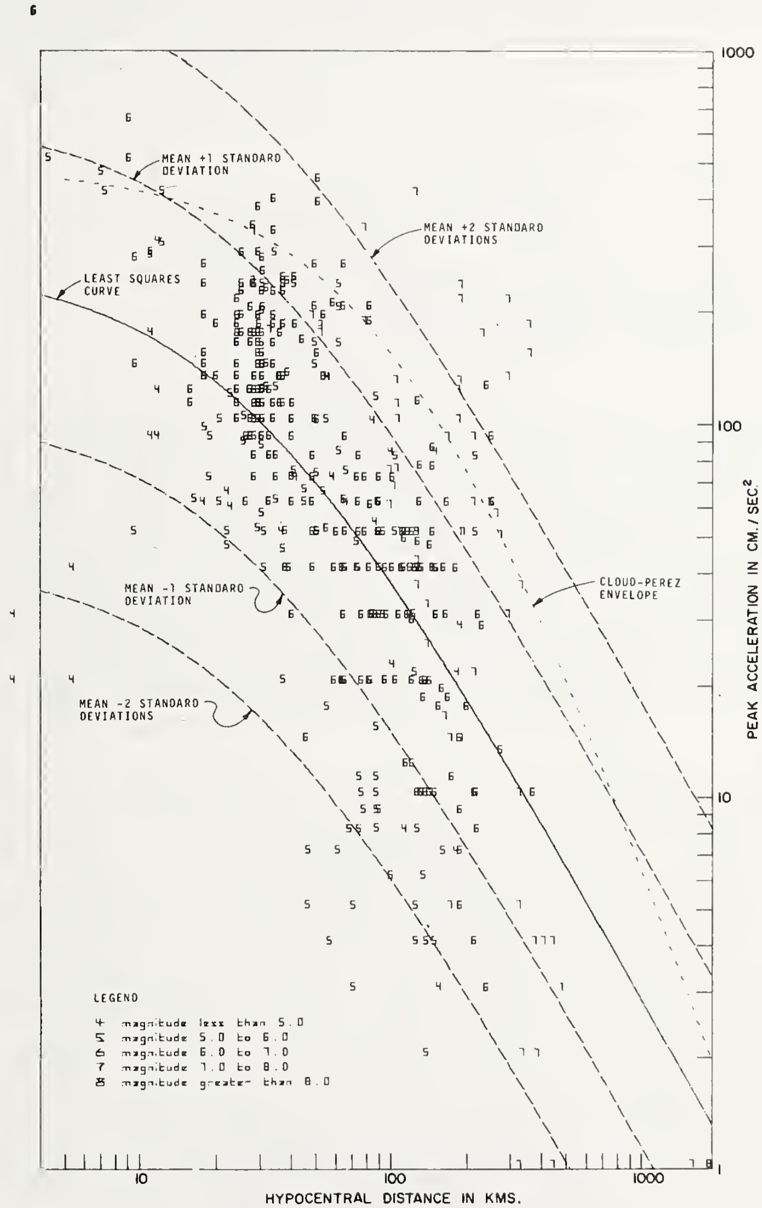

FIGURE 4

PEAK GROUND ACCELERATION IN TERMS OF DISTANCE FROM CAUSATIVE FAULT

WORLDWIDE SET OF 515 STRONG MOTION RECORDS WITHOUT NORMALIZATION OF MAGNITUDE

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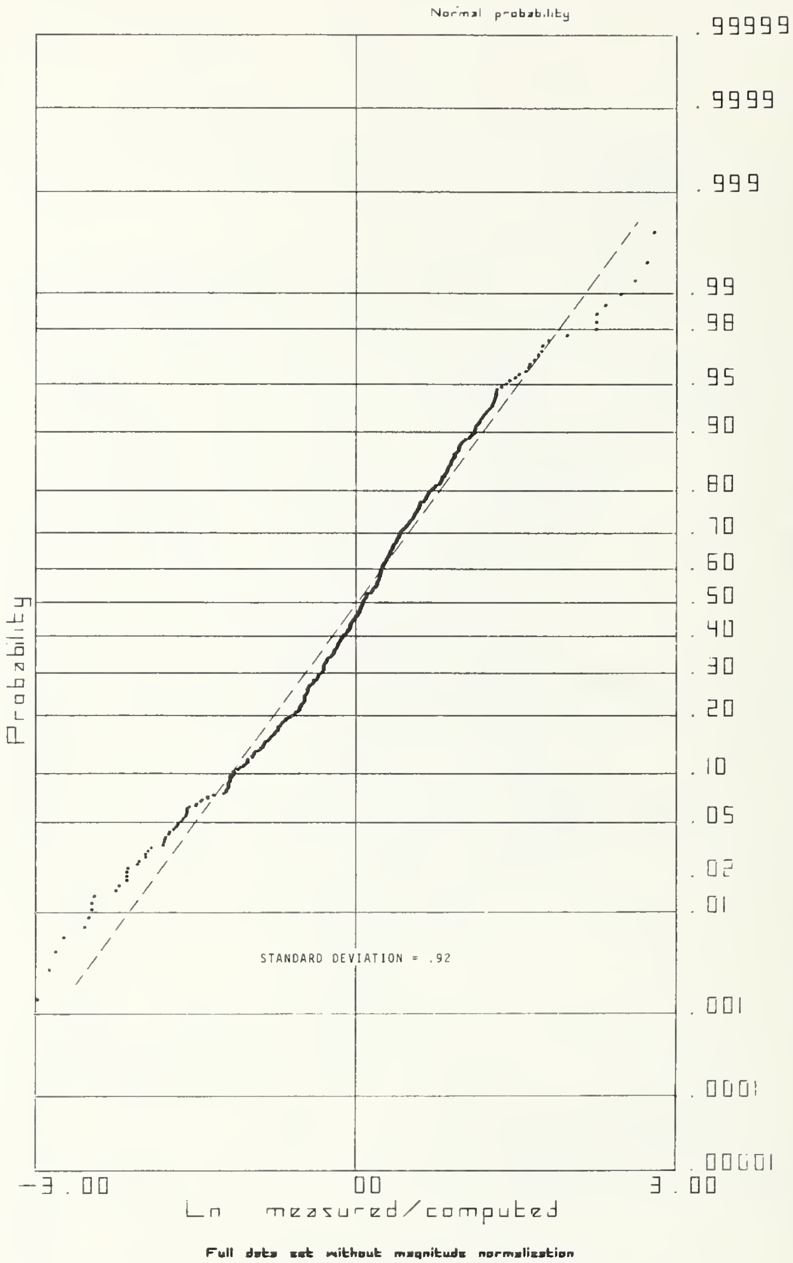


FIGURE 5
DISTRIBUTION OF
MEASURED / COMPUTED PEAK GROUND ACCELERATIONS

WORLDWIDE SET OF 515 STRONG MOTION RECORDS
WITHOUT NORMALIZATION OF MAGNITUDE

the standard deviation of the data set. Dashed lines represent the one and two standard deviation limits about the mean curve. As the data shown in Figure 4 have not been normalized in any way with respect to magnitude, the envelope curve developed by Cloud and Perez (1971) is shown as a lightly dotted line. The standard deviation of this data set is 0.92. Normalization of the data with respect to magnitude in the form first suggested by Esteva and Rosenblueth (1963) slightly reduces the standard deviation of the data to 0.84. A plot of the ranked data in Figure 5 to normal probability scaling shows good linearity suggesting that a normal distribution is a good representation of the variation of the data. The resulting attenuation equation for surface acceleration maxima obtained by the author using least squares procedures for the three variables is:

$$y = 1320\, e^{0.58m}\, (R + 25)^{-1.52}$$

where y = the peak acceleration in cm/sec^2

m = the Richter magnitude

R = the hypocentral distance, the distance to the causative fault or the distance to the center of energy release, if known, in kilometers.

The small reduction in the standard deviation when the magnitude was normalized in the data set is not considered as an indication that acceleration attentuation and magnitude are not related. While it is believed by many writers (Housner, 1965; Bolt, 1972) that maximum acceleration values close to the causative fault are independent of magnitude, the attenuation away from the fault is dependent on magnitude and distance. The assumption that magnitude and distance can be considered as independent variables is believed to be the reason for the small reduction in the standard deviation. Theoretically, the exponential coefficient relating acceleration values and distance, without consideration of material damping and dispersion, should vary between 2 for small magnitude earthquakes and 1 for very large magnitude earthquakes. The dependence of the distance attenuation coefficient on magnitude was also considered by Lastrico (1970).

Acceleration attenuation curves obtained by other investigators are summarized in Table 1 and compared graphically for a shallow focal depth magnitude 6.5 earthquake such as the 1971 San Fernando earthquake in Figure 6. All the attenuation relationships summarized in Table 1 are based principally on California data with some records from other locations. The applicability of the relationships developed outside of the western portion of the United States and Canada has not been demonstrated. There is evidence that the attenuation of motion in eastern North America is much lower. This has not been discussed extensively in the literature. Milne and Davenport (1969) in their seismic regionalization studies of Canada prepared attenuation relationships for eastern North America from a study of isoseismal maps. Figure 7, taken from the Milne and Davenport study, illustrates this much slower attenuation. Recent Japanese strong motion data assembled by the author also appear to attenuate more slowly. It is not known at this stage whether the apparent slow attenuation of the Japanese acceleration is a regional phenomenon or a consequence of the site conditions at the location of the recording instrument. Denham and Small (1971) reported results of a small statistical study of acceleration attenuation based on data obtained from repeated triggering of one strong-motion instrument located in New Guinea. Data from this instrument, which was located on 50 meters of soft sedimentary rock, gave an exponential coefficient for the distance term of 1.1, similar to that obtained from the western North American data. Studies of attenuation effects in different seismic areas should be undertaken.

Acceleration maxima are only one parameter of an earthquake and do not

TABLE 1

ATTENUATION EQUATIONS

Data Source	Equation	Reference
1. San Fernando Earthquake February 9, 1971	$y = 186206\ R^{-1.83}$	--
2. California Earthquakes	$y = \dfrac{981\ y_o}{1 + \left(\frac{R'}{h}\right)^2}$ where $\log y_o = -(\bar{b}+3) + 0.81m - 0.027m^2$, $\bar{b}$ is a site factor	Blume (1965)
3. California Earthquakes	Graphical Presentation	Housner (1965)
4. California & Japanese Earthquakes	$y = \dfrac{5}{\sqrt{T_G}}\ 10^{0.61m - P \log R + Q}$ where $P = 1.66 + \dfrac{3.60}{R}$, $Q = 0.167 - \dfrac{1.83}{R}$, T_G = fundamental period of site	Kanai (1966)
5. Cloud (1963)	$y = \dfrac{6.77\ e^{1.64m}}{1.1e^{1.1m} + R^2}$	Milne & Davenport (1969)
6. Cloud (1963) Housner (1962)	$y = 1230\ e^{0.8m}\ (R+25)^{-2}$	Esteva (1970)
7. U.S.C. & G. S.	$\log_{10} y = 6.5 - 2 \log_{10} (R'+80)$	Cloud & Perez (1971)
8. 11 Selected Records	Graphical Presentation	Schnabel & Seed (1972)
9. 303 Instrumental Values	$y = 1300\ e^{0.67m}(R+25)^{-1.6}$	--
10. Western U. S. Records	$y = 18.9\ e^{0.8m}\ (R^2+400)^{-1}$	--

y is cm/sec^2
R is kilometers (distance to causative fault)
R' is miles (epicentral distance)
h is miles (focal depth)
m is magnitude

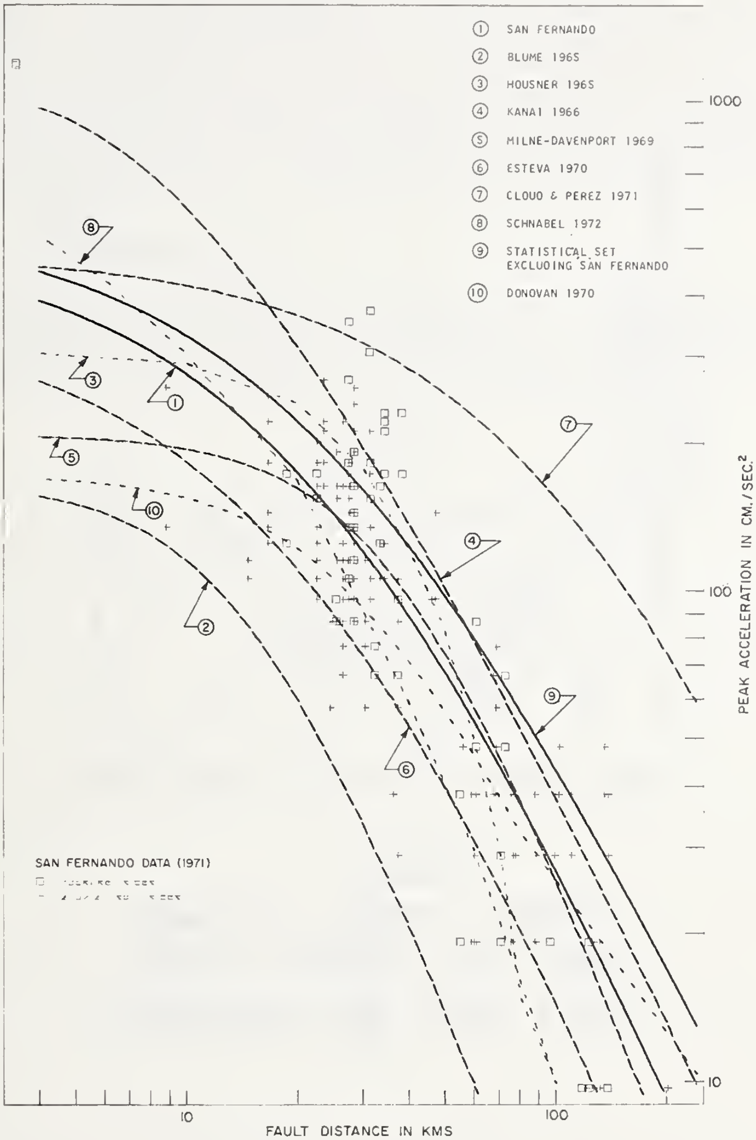


FIGURE 6

ATTENUATION EQUATIONS FOR MAGNITUDE 6.5 COMPARED TO DATA FROM STRONG MOTION STATIONS RECORDING SAN FERNANDO EARTHQUAKE

FEBRUARY 9, 1971

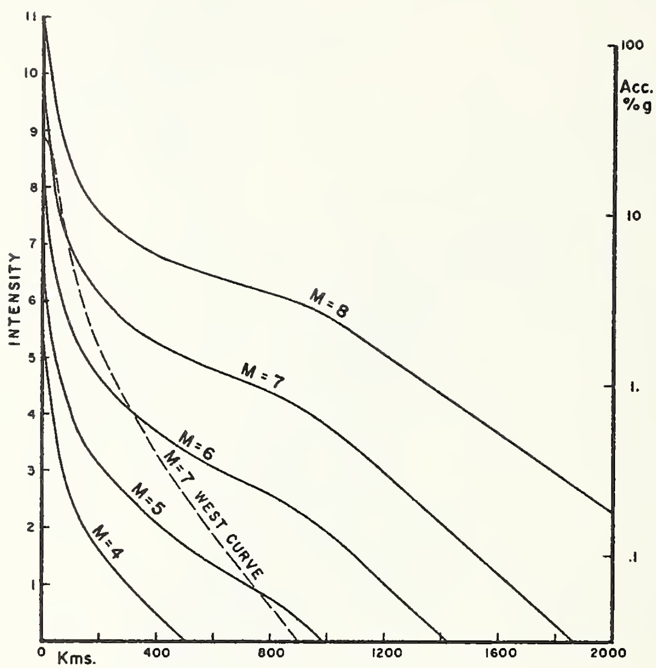


FIGURE 7
INTENSITY VERSUS DISTANCE
GRAPH FOR EASTERN CANADA.
(AFTER MILNE AND DAVENPORT)

alone express a measure of damage potential. Other parameters are probably of more significance. The most importance of these additional parameters are duration, frequency distribution of the seismic motion, the relative intensity of motion in terms of the peaks (such a factor could be the ratio of the peak value to the root mean square value), the maximum ground velocity and the maximum displacement. All these parameters must be considered when characterizing an earthquake. The characteristics of these parameters are considered later. Provided the perspective between these characteristics and acceleration is maintained, peak acceleration can be considered a reasonable parameter for describing the risk for seismic zoning (Milne and Davenport, 1969). Regional maps prepared by Milne and Davenport, relating acceleration levels and return periods have been incorporated in the Canadian seismic design code.

Return Periods and Probabilities

The risk procedures developed by Cornell is readily suitable for coding on a digital computer. When this risk procedure is applied to the sources which are modelled in Figure 3 with the attenuation relationship of Equation 1, the return periods listed below in Table 2 are obtained.

Table 2

RETURN PERIODS FOR PEAK ACCELERATIONS

Acceleration g	Return Period (Years) Surface Motion Competent Soil	Rock Motion
0.05	4	8
0.10	20	30
0.15	50	60
0.20	100	100
0.25	250	200
0.30	450	300
0.40	2,000	700

The second set of figures representing rock accelerations are based on a different attenuation equation prepared by this writer and currently being revised. Seed et al, (1970 and Schnabel and Seed, 1972) have also considered attenuation in rock.

The return periods in Table 2 can be best understood for the risk decisions when they are expressed as probabilities. If the events are considered to be unrelated and random in time, then a Poisson distribution may be used to express the probability of an event occurring during the proposed lifetime of the building. With the long return periods predicted for large acceleration levels there is considerable additional uncertainty. The Poisson distribution is unable to represent this. A Bayesian procedure outlined by Benjamin (1968) allows this uncertainty to be conservatively included. Where the return period is shorter than the period of the historical record, the uncertainty is reduced and the Bayesian probabilities become asymptotic to the Poisson values. The probabilities of the various acceleration levels being exceeded during a 50 year building lifetime at the surface of firm ground, and on rock, based on the return periods in Table 2 and the Bayesian procedure are illustrated in Figure 8. The low probability of a single event with a large acceleration maximum occurring and the high probability of several events with low acceleration occurring are readily apparent.

From the probability values shown in Figure 8, the acceleration levels for the Level I and Level II design earthquakes can be selected. The risk decisions are therefore made with uncertainties represented at a later stage in the analytical process. This gives a clearer understanding of risk than the subjective procedure where a source mechanism and magnitude is chosen without knowledge of the risk level. The latter procedure has had acceptance because of a reticence to accept statistical procedures. Also, once the basic decisions are made, the remainder of the analysis proceeds deterministically. Illustrative examples of the subjective and probabilistic approaches can be provided from other areas of engineering. Only recently has the probabilistic approach been considered extensively. The procedures are not yet widely taught at engineering schools within the United States.

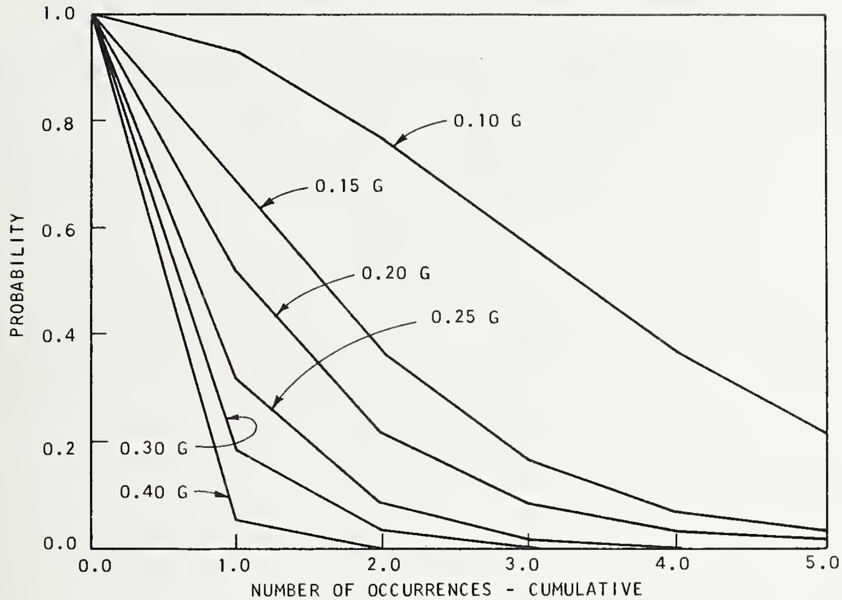
The choice of the levels of acceleration for design requires a trade-off between the cost of higher resistance and higher risks of economic loss. Structures are made earthquake-resistant rather than earthquake-proof, so sound design will suggest some possible economic loss. The Level II earthquake for which design will be essentially elastic will usually be an event with a probability of about 0.6 of occurring at least once during the economic lifetime of the structure. The Level I earthquake will have a probability of between 0.1 and 0.2. On this basis then the anticipated peak accelerations for the Level I and Level II earthquakes would be 0.40g and 0.15g for the example conditions. The frequency content of the motion at the rock level and the ground surface may be considerably modified by the soil profile. Observational data obtained from the 1971 San Fernando earthquake and shown on Figure 6 suggest that the rock acceleration maximum is not greatly different from the soil surface maximum acceleration at motion levels of engineering significance. The general trend of the data suggest that attenuation by the soil profile occurs at accelerations greater than 0.1g and amplification occurs at lower acceleration levels.

Two approaches can be taken once the design acceleration levels are chosen. Smoothed response spectra (Housner, 1961; Newmark and Hall, 1969) have been developed by averaging response spectra computed from accelerogram records. These represent motions for rock or stiff profiles and can be used for design where the local geologic conditions are appropriate. Modern design techniques use either a response spectra or an acceleration time history. Recently efforts have been made to produce artificial time histories which have smooth response spectra. As real earthquakes do not have smooth response spectra, it is suggested that where acceleration time histories are required for structural design or where the soil profile may be expected to modify the site response other techniques be used.

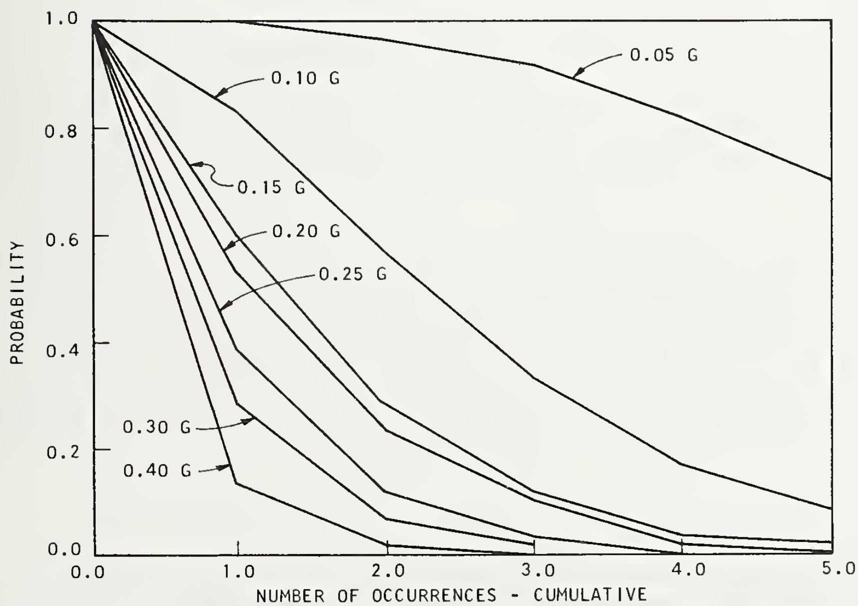
III. Local Effects

Site Geology and Soil Profile

The effect of the local geology and site soils on seismic response (Kanai, 1952; Duke and Leeds, 1962) was first studied as a wave propagation problem using reflection and refraction based on Snell's laws. Computational difficulties in the analyses led to development of other methods. A lumped-mass model representation of the soil profile as a vertical shear beam



A) BASED ON RETURN PERIODS USING EQUATION 1.



B) BASED ON RETURN PERIODS USING ROCK ATTENUATION EQUATION

FIGURE 8

ESTIMATED PROBABILITIES FOR 50-YEAR PROJECT LIFE

(Penzien et al, 1964) was first used to analyse site response of soft soils. The shear beam model and wave propagation procedures have both been used to show the variation of site response on different soil profiles (Herrera and Rosenblueth, 1965; Seed and Idriss, 1968). The development of an extremely efficient algorithm for machine computation of Fourier spectra by Cooley and Tukey (1965) assisted development of the wave propagation analysis. The wave propagation technique appears to be the most efficient procedure for computing the response of a soil profile to vertically ascending shear waves.

Analysis of the site response as a function of only vertically ascending waves can be shown to be an over-simplification of a complex response problem. The computed response spectra for horizontal components at a single instrument location should have similar characteristic shapes with variations reflecting the random nature of the basic rock motion. For deep soft profiles, the assumption of vertical wave propagation gives excellent results. There are appreciable and seemingly inexplicable differences in other types of soil profiles. Vertical acceleration cannot be produced by a vertically ascending shear wave. If the vertical motion is principally due to vertically ascending P waves, the ratio of vertical acceleration to horizontal acceleration should increase with distance due to the higher velocity of the P wave relative to the S wave and the consequent lower damping. A study of the 1971 San Fernando acceleration data showed no evidence of this. Trifunac (1969) concluded that a predominant part of the near field ground shaking may be composed of surface waves associated with energy propagating horizontal through near surface rock and soil layers acting as a wave guide. The analytical data presented by Trifunac (1970) provides an excellent explanation for the non-stationary time dependence of the intensity and frequency, but does not conclusively show that the predominant part of the ground shaking is due to surface wave propagation. Rayleigh and Love wave propagation in layered media have also been studied analytically (Lysmer, 1970; Lysmer & Drake, 1971). These studies demonstrated that selective amplification and reduction of specific frequencies is possible across areas such as large alluvial valleys where the geologic profile is locally horizontal. The use of a vertical shear wave model is recommended by this author subject to the realization of its limitations. Allowance for the inadequacies of the method by incorporation of some variational procedures in any deterministic response analysis is highly recommended.

Analyses using the shear wave model can be made in either the time or frequency domains. Using Fourier techniques the analyses are most readily performed in the frequency domain. Complex Fourier spectra of the output motion can be obtained by multiplication of the Fourier spectra of the input motion and the profile transfer function. The development of damped response spectra within the frequency domain is extremely difficult. Therefore site response analyses using acceleration time histories must be used even where it is intended that structural design be based solely upon the response spectra. To eliminate the effects of the randomness of motion within any individual acceleration time history it is advisable to repeat the response evaluation with several time histories and obtain an average response spectra. This has the additional advantage of allowing an estimate of the probable variations within any one time history when structural analysis is performed within the time domain.

Input Motion

The acceleration time history used for input to the base of the soil profile must be selected on the basis of the available knowledge of the parameters of earthquake motion. These parameters include the duration of the earthquake, the frequency distribution of the motion with time and the variation of frequency content with time. Because the collection of recorded strong motion accelerations is not large, most of these parameters are not well understood.

There is no accepted definition of the effective duration of an earthquake for engineering purposes at the present time. For example, Housner (1961) referred to the duration of strong shaking while Cloud and Perez (1971) used the time during which the accelerations exceeded certain values. An approach to computing the effective duration of shaking was proposed by Husid, Medina and Rios (1969). In their approach the digitized acceleration values are squared and the sum of the values is accumulated. A plot of the accumulated value against time has a shape resembling a cumulative distribution curve and provides a direct graphical representation of the duration effects. Two examples of these summations, for Pacoima Dam, 1971, and El Centro, 1940, are illustrated in Figure 9. A numerical definition of effective duration could be taken from the summation by considering the time period containing some percent, say 90 per cent, of the total cumulative acceleration squared. Duration values obtained from recent earthquakes, including Japanese earthquakes, were added to the durations listed by Housner (1970). These are shown in Figure 10.

There is not sufficient knowledge at this time of the variation of frequency content of the motion during an earthquake to include this in development of earthquakes for structural design. Some studies of the 1971 San Fernando earthquake aftershocks have been made with variations of frequency included. The variation of intensity with time is somewhat better understood. A simple subdivision of an acceleration time history into three parts is usually adequate. They are, a short rise time, a stationary mid-portion and a final section in which the decay of motion is approximately exponential. Many research workers have made attempts to generate earthquakes using white noise, stationary Gaussian processes, or non-stationary processes (for example, Jennings et al., 1968; Ruiz and Penzien, 1971). Some workers have used recorded motions to construct rock motion accelerograms (Seed and Idriss, 1969). This is a somewhat contradictory procedure as a historic motion which has probably been modified by the profile upon which the recording instrument was located is used as part of the input to another unrelated soil profile.

It is recommended that several design earthquakes be developed at each level. A numerical technique using a random number generator and allowing choice of the appropriate motion parameters is believed to be the most suitable procedure. A sample motion for a Level II earthquake with an estimated maximum acceleration of 0.15g representative of a magnitude 7 earthquake approximately 50 kilometers form the closest point on the fault is shown in Figure 11. This was generated using a modification of the procedure developed by Ruiz and Penzien.

Following a site investigation the soil profile can be modelled using measured or estimated soil properties (Seed and Idriss, 1970). The analytical techniques used to compute the soil response were developed for linear elastic materials. The elastic properties of soil are nonlinear and very strain-dependent. A linear approximation is made by using equivalent linear modulus and damping ratios representing some average strain (Seed and Idriss, 1969; Dobry, Whitman and Roessett, 1971). The surface motion can then be computed using either a lumped mass procedure or wave-propagation theory. An output time history using the wave-propagation theory is shown in Figure 11. The output time history using the wave propagation theory and incorporating uncertainties in the measured soil properties in the soil profile transfer function is shown in Figure 12. Figures 11 and 12 also show the response spectra of a single input and output motion for several damping levels, the mean output response spectrum obtained using a set of generated input motions, and one standard deviation bounds about the mean spectrum. The output acceleration time histories or the output response spectra can be used directly by the structural engineer for design of the building. For most buildings the effect of soil-structure interaction modifying the ground response need not be considered. Soil-structure interaction becomes a significant factor only for very rigid shear wall buildings or buried structures such as a nuclear reactor.

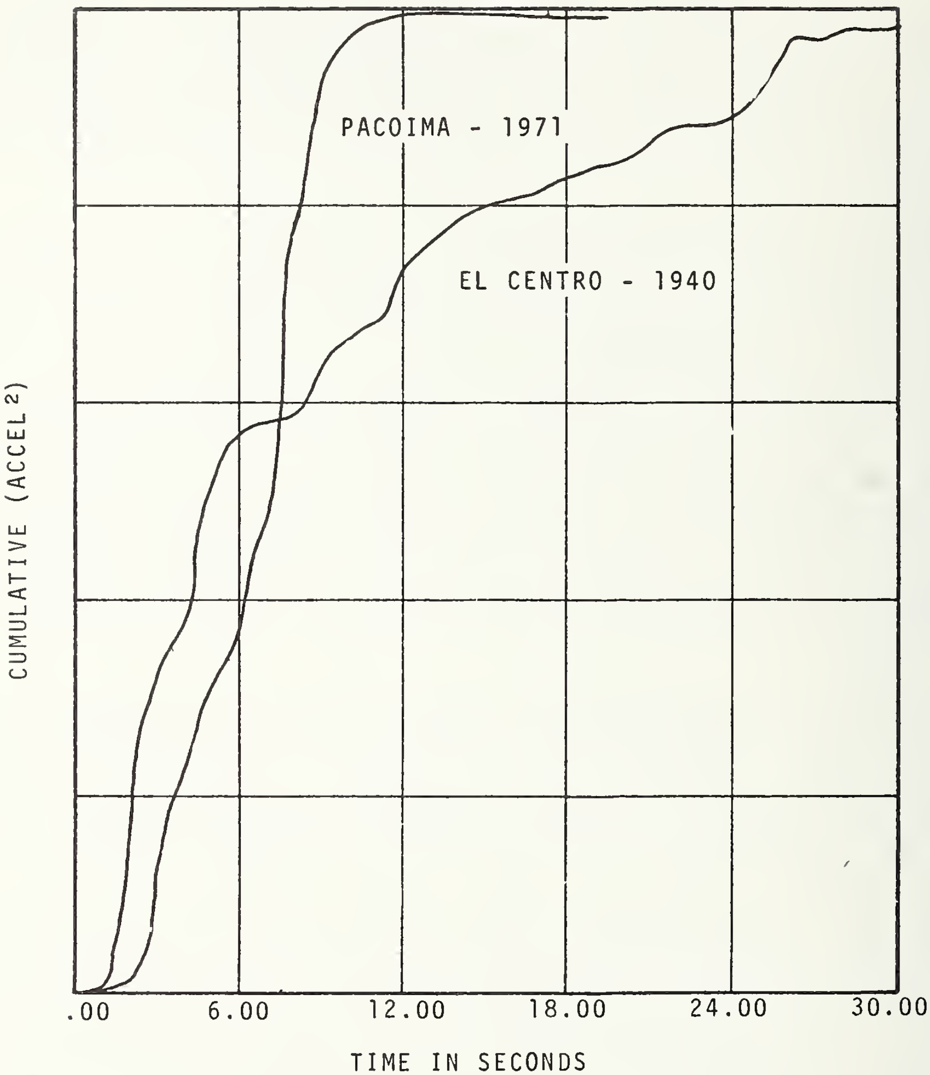


FIGURE 9

CUMULATIVE SQUARED ACCELERATION RELATIONSHIP WITH TIME

(AFTER HUSID, ET AL, 1969)

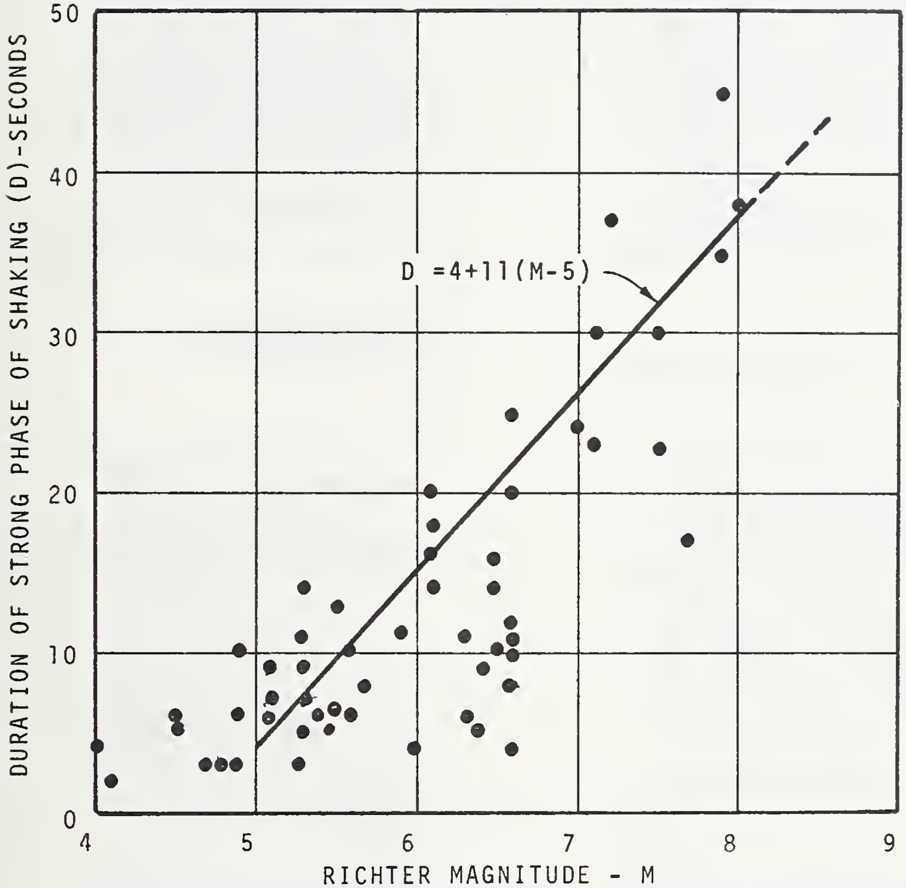
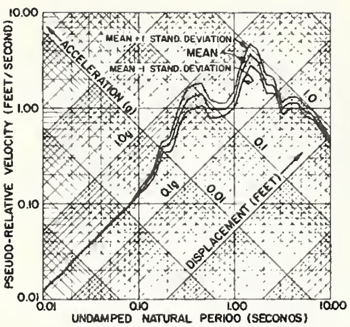


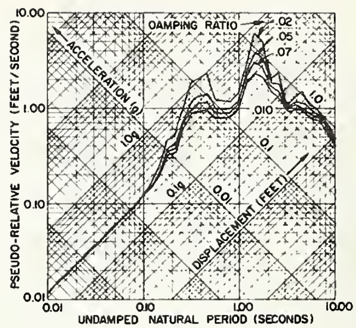
FIGURE 10

RELATIONSHIP BETWEEN MAGNITUDE & DURATION OF STRONG PHASE OF SHAKING

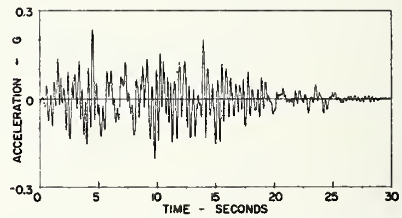


RESPONSE SPECTRA AVERAGE

(SHOWING STATISTICAL SPECTRAL VARIATION ANTICIPATED BETWEEN INDIVIDUAL TIME HISTORIES IN THE ENSEMBLE)



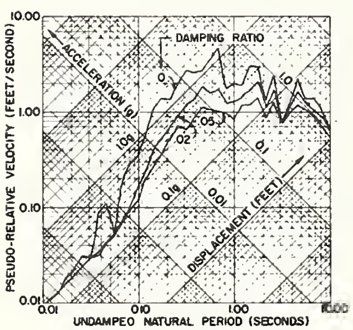
RESPONSE SPECTRA



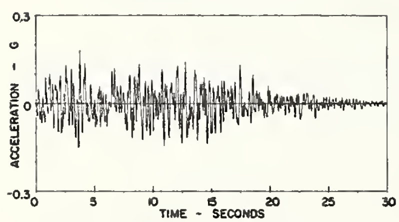
ACCELERATION TIME HISTORY

SURFACE MOTION OUTPUT

(USING INPUT MOTION SHOWN)



RESPONSE SPECTRA



ACCELERATION TIME HISTORY

SOIL	LAYER THICKNESS IN FEET	TOTAL UNIT WEIGHT PCF	SHEAR MODULUS KSF	DAMPING IN PERCENT
LOOSE SAND	15	130	403	15.8
DENSE SAND	20	130	585	14.0
VERY DENSE SAND	41	132	735	12.5
VERY STIFF CLAY	16	116	643	13.2
VERY DENSE SAND	16	130	813	11.6
VERY STIFF CLAY	40	112	402	18.5
VERY DENSE SAND	45	130	966	9.8
VERY STIFF CLAY	15	130	530	14.0

BEDROCK

FUNDAMENTAL PERIOD = 1.5 SECONDS

INPUT MOTION AT BASE OF PROFILE

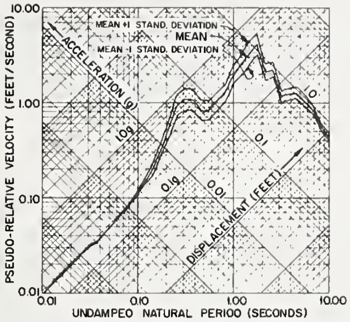
(FROM SET OF GENERATED ACCELERATION TIME HISTORIES)

IDEALIZED SOIL PROFILE

FIGURE 11

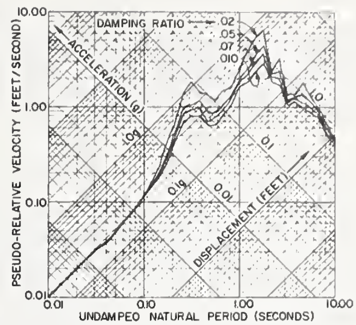
INPUT AND OUTPUT MOTIONS TO SOIL PROFILE

FIXED PROPERTIES

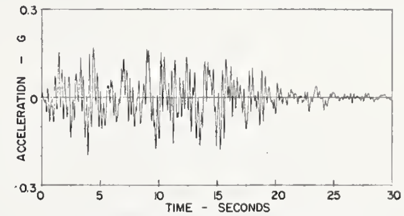


RESPONSE SPECTRA AVERAGE

(SHOWING STATISTICAL SPECTRAL VARIATION ANTICIPATED BETWEEN INDIVIDUAL TIME HISTORIES IN THE ENSEMBLE)



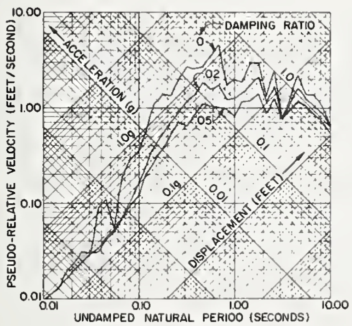
RESPONSE SPECTRA



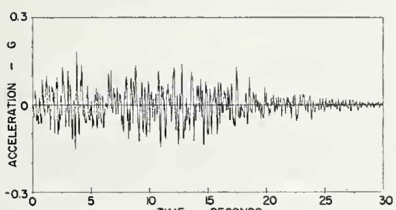
ACCELERATION TIME HISTORY

SURFACE MOTION OUTPUT

(USING INPUT MOTION SHOWN)



RESPONSE SPECTRA



ACCELERATION TIME HISTORY

INPUT MOTION AT BASE OF PROFILE

(FROM SET OF GENERATED ACCELERATION TIME HISTORIES)

SOIL	LAYER THICKNESS IN FEET	TOTAL UNIT WEIGHT PCF	SHEAR MODULUS KSF	DAMPING IN PERCENT
LOOSE SAND	15	130	403	15.8
DENSE SAND	20	130	585	14.0
VERY DENSE SAND	41	132	735	12.5
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VERY DENSE SAND	45	130	966	9.8
VERY STIFF CLAY	15	130	530	14.0

BEDROCK

MEAN SOIL PROPERTIES VARIED TO GIVE A RANGE OF FUNDAMENTAL PERIOD BETWEEN 1.25 AND 1.75 SECONDS

IDEALIZED SOIL PROFILE

FIGURE 12

INPUT AND OUTPUT MOTIONS TO SOIL PROFILE

VARIABLE PROPERTIES OVER CONTROLLED RANGE

IV. Structural Design Codes

Present seismic design codes specify lateral forces to be used in a static design procedure. The physical behavior of a structure during an earthquake is considered only in general terms and risk potential is not considered at all. The current consensus of many structural engineers is that the present seismic codes in use in the United States represent only a minimum guideline standard and require considerable revision. As the seismic codes have developed over a period of time from simple concepts several important factors have not been included. This has led to some conflict in the understanding of the philosophy upon which the code is based.

Seismic design forces are but one type of loading which a building must resist. A rational design code would be expected to be based on probabilistic concepts and incorporate an approach reflecting systems analysis. The purpose of this paper is to review the present state-of-the-art and to suggest modest changes which may improve the understanding of structural design parameters within the framework of the present codes. Dynamic design procedures, whether based on modal superposition using response spectra or direct use of acceleration time histories consider many of the factors affecting the response behavior of buildings, which have not been included in a seismic design code. Site characteristics and the response of higher vibration modes are examples.

Seismic design codes of several countries (Argentina, Canada, Chile, Cuba, France, Greece, India, Japan, Mexico and Turkey) include some modification of seismic coefficients in an attempt to allow for the variation of response of different soil types. This foreign acceptance of site response response effects is demonstrated by Tezcan (1971). The Uniform Building Code, the Recommended Lateral Force Requirements of the Structural Engineers Association of California, and the proposed new seismic code for the City and County of Los Angeles, contain no recognition of the variation of response on different types of soil. The need for incorporation of some recognition of site effects is becoming increasingly important as more and larger buildings are being constructed on sites with foundation conditions very different from those envisioned by the original developers of the codes.

Changes in the present code should be implemented to recognize the larger lateral forces that can be produced for some combinations of soil conditions and building properties. To be effective the approach must be simple to use, while representing rationally the complex aspects of site response. The suggested procedure presented below is no more complicated than the formulation of the other portions of the Uniform Building Code provision for the base shear coefficient.

It is recommended that the equation for the base shear coefficient be modified by the insertion of an additional term S representing the interrelationed effects of the soil and the structure. The equation would therefore become V = ZKSCW with all the present coefficients remaining unchanged.

The factor S should be the product of two separate factors. These are:

1) A factor independent of period, which is dependent only on the average soil properties in a zone of influence below the foundation approximately equal to one half of the building width. This factor which is designated S_0 is designed to allow for the possible motion modification with increasing softness of the surface soil and the possible influence of static settlements upon the reserve strength of the structure. The factor S_0 may be determined from the following table which uses the same subdivisions as Table 29-B in the 1970 edition of the Uniform Building Code.

Type of Soil	Maximum Allowable Pressure (psf)	S_o	Minimum T_s Seconds
Good - hard clay, medium dense to dense sands and gravels, rock of all types	8000	1.0	0.4
Average - medium stiff clay, compact sand or gravel	2500	1.25	1.0
Poor - loose sand or gravel, soft clay	1500	1.5	1.0

2) A factor dependent upon period which recognizes that larger lateral forces may occur when the periods of the ground motion and the building are close to each other. This requires estimates of T, the fundamental period of the building, and T_s, the predominant period of the underlying soil. The soil coefficient may then be obtained by use of the following relationship:

$$S = S_o \left(1.25 - 0.25 \cos \frac{T}{T_s} \pi\right) \quad \text{for } 0 \le \frac{T}{T_s} \le 2$$

$$S = S_o \quad \text{for } \frac{T}{T_s} > 2$$

Where the predominant ground period has not been established by a detailed site study and the proposed building height does not exceed 150 feet the minimum values for T_s in the table above may be used. The building period T should be the estimated period of the completed structure including any soil structure interaction effects.

The 1967 edition of the Chilean Earthquake code modifies the seismic coefficient C to include the relationship between the ground and structure periods. A replacement of the existing factor C was also recommended by Seed, et al, (1970). The existing C parameter is probably low, especially in the period range less than 1 second and some separate upward adjustment may also be necessary. Direct replacement of the C parameter should be done with recognition that the present relationship is based upon response spectra studies which show that the energy distribution within an earthquake decreases with increasing period. In Figure 13 the maximum and minimum bounds of the product of the proposed soil factor and the C coefficient are shown for a ground period of 1 second. The present C coefficient and the Chilean coefficient are shown as dashed lines on the figure. The trapezoidal shape for the C coefficient suggested by Seed et al, which is shown as a dotted line has been normalized downward to a maximum of 0.1. All of the code modifications representing site response effects require either the measurement or estimation of the predominant period of the building site. Within the present state-of-the-art, it is believed that this ground period can be estimated with about the same accuracy as the fundamental period of a completed building structure.

V. Conclusions and Recommendations

The types of structures being built today are frequently larger and use different construction techniques and materials than structures which have survived earthquakes and served as guides for the developers of seismic design codes. Structures are now being built on a wide variety of sites. Code forces envisage that the major response of a structure will be in the

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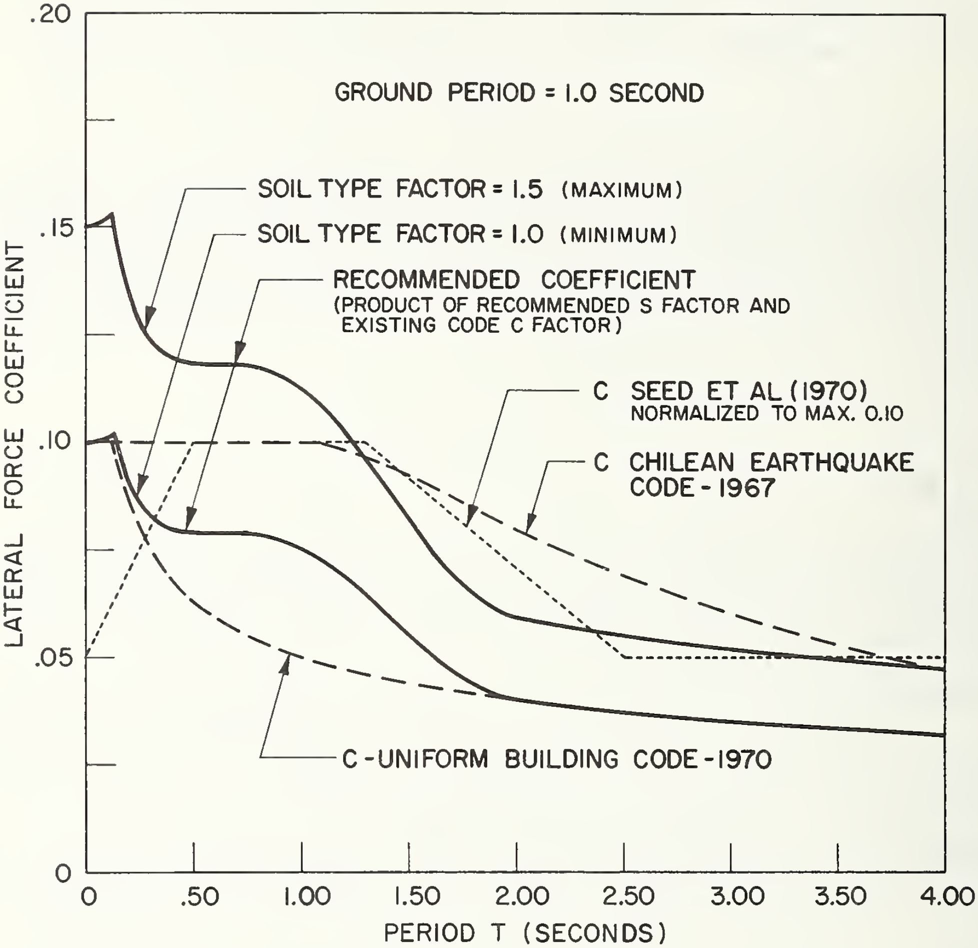

FIGURE 13

SUGGESTED LATERAL FORCE COEFFICIENT
TO INCLUDE EFFECTS OF
SOIL PROFILE AND SOIL-STRUCTURE INTERACTION

first mode of vibration. This is not always so.

Listed below are four separate areas where it is believed that a concerted group effort aided by some research could produce improvements in the engineering field quite rapidly. These are:

1) Levels of motion to be used for design. This is possibly an interdisciplinary communication problem. Single motion pulses may produce large seismological observations that are not of practical engineering significance. A rational series of design basis motion parameters should be developed. These may use, but should not be confined to, parameters widely used at present.

2) The treatment of seismic risk as a relative factor based on probability of occurrence. This may require more emphasis in studying regional tectonics to allow more objective interpretation of historical seismicity.

3) The recognition that site conditions be considered in modifications of the seismic code. Some first step such as suggested herein to include site effects should be undertaken as soon as possible.

4) A direct corollary of item 3 is the need to include multiple-mode response in seismic design codes. The combination of high rise buildings on developed sites frequently produces predicted response patterns that cannot be represented by existing design codes. It must be realized that a design code cannot adequately cover unusual or complex structures. In such circumstances it is suggested that building officials should establish advisory boards who would review and approve the necessary analytical techniques.

There is a wide gap between the practice of enlightened structural engineers and the practice presented in code procedures. Although it is impossible to prescribe a substitute for sound engineering judgment it is still believed that steps toward bridging this chasm could be a major achievement of this workshop.

VI. Acknowledgements

The soil structure response factor suggested for code use has resulted from extensive discussions with Professors Joseph Penzien of the University of California at Berkeley and Robert V. Whitman of Massachusetts Institute of Technology. The assistance of the writer's colleagues, Jogeshwar P. Singh and Julio E. Valera in the preparation of this review is gratefully acknowledged.

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THE PROBLEM OF SEISMIC ZONING

by

S. T. Algermissen[1]

Seismic zoning maps for the United States are a relatively recent development in seismological and engineering research. The first zoning man of the entire United States was prepared by Franklin P. Ulrich [1] and publisted in 1948. John R. Freeman [2], however, refers to a number of maps that were prepared for various areas and cities in the early 1900's. These maps considered some particular aspect of the earthquake hazard problem. Freeman noted that the chief engineer of the Spring Valley Water Company sketched a map of San Francisco "long before the great earthquake of 1906" which considered the possibility of breakage of distribution pipes within the city during an earthquake. Freeman also includes in his book a map of "probable relative stability of ground" in Boston during an earthquake. In general, zoning of foreign countries with earthquake problems, such as Japan and the Soviet Union, was begun at an earlier date than in the United States. Medvedev [3] provides a summary of early zoning methods.

The purpose of this paper is to outline the factors I believe to be of most importance in seismic zoning and to discuss the difficulties in estimating these factors and how these difficulties affect the final zoning map.

A Zoning Map Concept

First, it seems desirable to consider what form a zoning map or maps should assume. The effects of earthquakes can be divided into three categories: (1) those effects which are the result of a certain intensity of ground shaking; (2) those effects resulting from faulting associated with the mechanism of the earthquake; and (3) those effects resulting from the generation of a seismic sea wave or tsunami. All geological effects such as landslides, liquifaction, slumping, etc., occur because of some particular physical condition of the materials involved, but they all are triggered or activated by a particular level of ground shaking. Thus, it seems desirable, at least in theory, to prepare zoning maps by displaying the expected maximum intensity of shaking in a specified time interval together with the areas where surface faulting might be expected with an estimate of the return periods of the faulting.

Estimates of geological risks such as landsliding, etc., can be made at a particular site or in a particular region based upon the maximum intensity of shaking (taken from the zoning map) together with an evaluation of available geologic and engineering maps depicting the hazard or through geological and engineering investigation of the particular site in question. Since the intensity of shaking displayed on the zone map would necessarily have to be given with reference to a specific type of surface material, corrections would have to be made at each site for the effect of the particular material actually beneath the site.

It is probable that no single measure of intensity of shaking will be optimal for the estimation of all possible geologic hazards at a site or

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optimal for use in the design of earthquake resistant structures of all types. Thus, it is likely that a number of zone maps will ultimately have to be prepared, each describing intensity of shaking in a way most appropriate for the use it is intended.

Most zoning maps in the past have been based upon a mapping of Modified Mercalli (or some other) intensity. Clearly it is desirable to map a quantity that can be more easily interpreted in terms of engineering design than intensity, but there is considerable question as to how this is best to be accomplished. It is suggested, at least as an initial map, that ground particle velocity be used as the parameter to be mapped since there is appreciable evidence that this quantity is significant for design considerations for a fairly wide range of types of structures.

What are the principle elements involved in the construction of a zoning map and what problems are encountered in estimating the significance of these elements in zoning? The elements involved can be roughly divided into estimates of seismicity and fault breakage, the nature of the seismic source, seismic wave attenuation, and site response.

Estimates of Seismicity and Fault Breakage

Seismicity is commonly presented in the form of hypocenter maps together with empirical relations of the forms:

$$\log N = a - bM \qquad (1)$$

where N is the number of earthquakes of magnitude M or greater and a and b are constants to be determined for the region in question. A conversion from Io (maximum intensity) to M must be made for the older earthquakes for which instrumental magnitudes are not available so that effective use may be made of the total historical record of earthquakes, thus improving the statistical sample and estimates of seismicity. The fit of the data to equation (1) generally improves as the size of the earthquake sample increases, but may be very poor for small areas of moderate seismicity. Attempts have been made to represent the occurrence of earthquakes as a Poisson process but this assumption has been seriously questioned [4, 5].

It is not known whether equation (1) is valid for very small M because the networks of seismographs are inadequate to detect and locate small shocks. It is commonly not understood, for example, that earthquakes as large as M=4 can occur in some parts of the United States at the present time which are <u>not</u> located [6]. For some large M, equation (1) does not hold since there is an upper magnitude limit for shocks in each area which depends on the properties of the crust and upper mantle. It is, therefore, inadvisable to estimate the recurrence of large earthquakes by extrapolation of M from (1) beyond the range of M actually observed. Geological investigations of the extent of faulting observed to have occurred in recent geological time may be very valuable in estimating the largest shock likely to occur in a region in the future.

It is a common opinion that not much new information can be obtained from further study of the historical record of earthquakes. This is a misconception easily demonstrated by examination of a number of recent studies such as the one reported by Nuttli [7] for the Mississippi Valley. In fact, great improvement in earthquake statistics would likely result from a very thorough study of the historical records. Other improvements could result from expansion of the domestic network of seismograph stations and through additional studies of microearthquake activity.

Reasonable estimates of probable length of faulting can be made on certain faults, notably the San Andreas in California, but much additional work needs to be done. Trenching across suspected active fault zones and age dating techniques promise to greatly expand the amount of information currently available concerning active earthquake areas.

In summary, the statistical sample of earthquakes available throughout the United States varies considerably in completeness. It should be used in conjunction with geological evidence of recent faulting and geodetic data whenever possible to establish the best possible recurrence rates for earthquakes.

The Seismic Source

The nature of the seismic source or mechanism introduces two principle complications into the zoning problem: (1) some earthquakes appear to be multiple shocks while others are a single release of energy; and (2) earthquake energy is rarely released at a point source but is released in faulting which may extend horizontally and vertically for considerable distance. Multiple event sequences have been postulated for the 1940 El Centro earthquake [8] and for the 1964 Alaska earthquake [9]. A number of questions remain to be answered. Are all earthquakes above a certain magnitude threshold multiple sequences or do multiple sequences only occur in certain regions? If multiple sequences occur only in certain regions, which regions produce these types of earthquakes? Multiple sequences tend to significantly prolong the intensity of strong shaking. Since only a very small number of earthquakes have been investigated for multiple events, it is uncertain whether earthquakes for which strong motion data exist are multiple or single events. Obviously this greatly complicates the analysis of strong motion data. If data from accelerograms recorded in an area where multiple sequence earthquakes predominate are used for design purposes in other areas where the earthquakes tend to be single shocks, considerable errors in estimating the duration of shaking will occur.

The direction and length of faulting during an earthquake greatly affects the radiation of seismic waves from the earthquake. Figure 1 shows the general distribution of maximum horizontal acceleration during the San Fernando earthquake of 1971. While the contours in Figure 1 are only approximate, they do show a pronounced northwest-southeast acceleration pattern which coincides with the strike of the preferred fault plane for the earthquake [10]. The faulting of the San Fernando earthquake had a significant component of dip slip motion. For earthquakes in which the faulting is purely strike slip, the elongation of the acceleration pattern might be even more pronounced. Clearly, earthquake mechanisms need to be taken into account in seismic zoning. Unfortunately, surface faulting is not common for earthquakes in the United States outside of California and Nevada and mechanism studies based on instrumental data have been constructed for only a relatively small number of events. Mechanism solutions could be computed for a significant number of additional earthquakes in many areas using seismic data already available and through calculation of composite mechanism solutions for small earthquakes using data from portable stations.

Attenuation

One of the most important factors in seismic zoning is the effect of attenuation of seismic waves away from the source. At the present time, the problem of estimating attenuation in the United States can easily be separated into two parts. Sufficient strong motion data are probably available in California to reasonably estimate attenuation for zoning purposes. For the balance of the country almost no acceleration data exist.

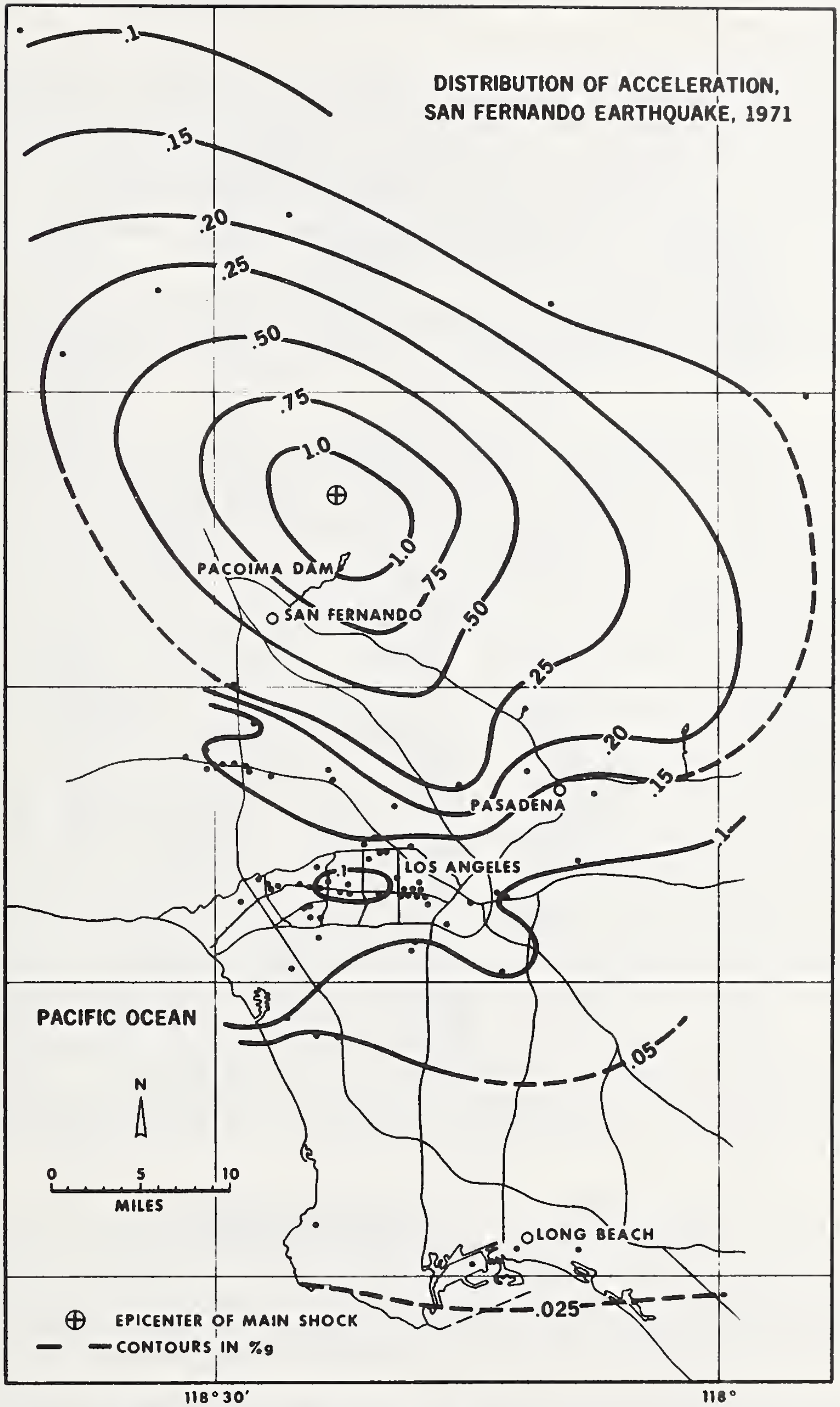

FIGURE 1. APPROXIMATE DISTRIBUTION OF MAXIMUM HORIZONTAL ACCELERATION DURING THE 1971 SAN FERNANDO EARTHQUAKE

First, consider the attenuation data available in California. Figure 2 shows a sample of the available data selected to illustrate some of the current problems. Accelerations recorded during the 1952 Kern County, the 1966 Parkfield, and the 1971 San Fernando earthquakes have been corrected to bedrock acceleration using a method developed by Campbell [11]. They are plotted in Figure 2.

The correction method developed by Campbell is only an approximate one. He divides all possible sites into four general categories: crystalline rock (bedrock); sedimentary rock; shallow alluvium (20-60 feet); and deep alluvium (60 feet or greater). Accelerations recorded on deep alluvium, shallow alluvium, and sedimentary rocks must be multiplied by .53, .64 and .80 respectively to correct them to the acceleration on bedrock. The data from the 1957 San Francisco earthquake, which has been corrected to bedrock, was taken from a paper by Idriss and Seed [12].

The 1966 Parkfield and the 1957 San Francisco earthquakes are nearly the same magnitude and as can be seen from Figure 2, the recorded accelerations compare quite favorably after applying the approximate correction for site response.

During the San Fernando earthquake, accelerations were recorded at a number of bedrock sites and these are plotted in Figure 2 together with accelerations recorded at other types of sites which have been corrected to bedrock. A number of conclusions can be reached from the data in Figure 2: 1. the accelerations, despite some scatter, show a rather regular increase with increasing earthquake magnitude. There is uncertainty because of lack of data as to the maximum values of acceleration that might occur near the causative fault and there is some disagreement among various workers as to what the maximum accelerations might be. 2. The scatter of the data increases with increasing magnitude. This is an expected result since the factors of fault length and departure from a spherical radiation pattern become more pronounced as magnitude increases. Scatter should be decreased by introduction of a correction for the focal mechansim. 3. The scatter of data recorded at "bedrock sites" is considerable and of the same order of magnitude as for accelerations recorded on other types of materials. This suggests that more site information, particularly P and S wave velocities together with a more sophisticated technique for correction to bedrock are needed to further reduce the scatter of the data. 4. Attenuation relationships need to be developed, taking into consideration all of the data currently available. Corrections for site geology and the mechanism of the earthquake should be considered in developing the attenuation relationships. The attenuation formula of Estava [13] plotted in Figure 2 for earthquakes of magnitudes 5.5 and 6.5 does not fit the data very well. A number of other published attenuation relationships were also tried with limited success. The expression for attenuation currently in the literature all seem to be based on rather limited data; however, a very recent study by Schnabel and Seed [14] appears to be a comprehensive study of attenuation using the California data.

Until strong motion ground data become available for other parts of the United States, attenuation of strong ground motion outside California will have to be worked out using principally Modified Mercalli intensity data. One approach is to correlate intensity data available for historical earthquakes with intensity and attenuation data available for recent earthquakes. Attenuation curves have recently been developed by Nuttli [7, 15] for 3 to 10 sec. Rayleigh and 1-sec. Lg waves in the eastern United States.

Relationships for the attenuation of surface waves in the United States can be used to develop a general attenuation curve because at distances greater than about 60-80 km. surface waves are the predominant type of ground motion. Very close to the causative fault, the maximum ground motion is caused by P and S-body waves. The <u>maximum</u> ground motion very near the

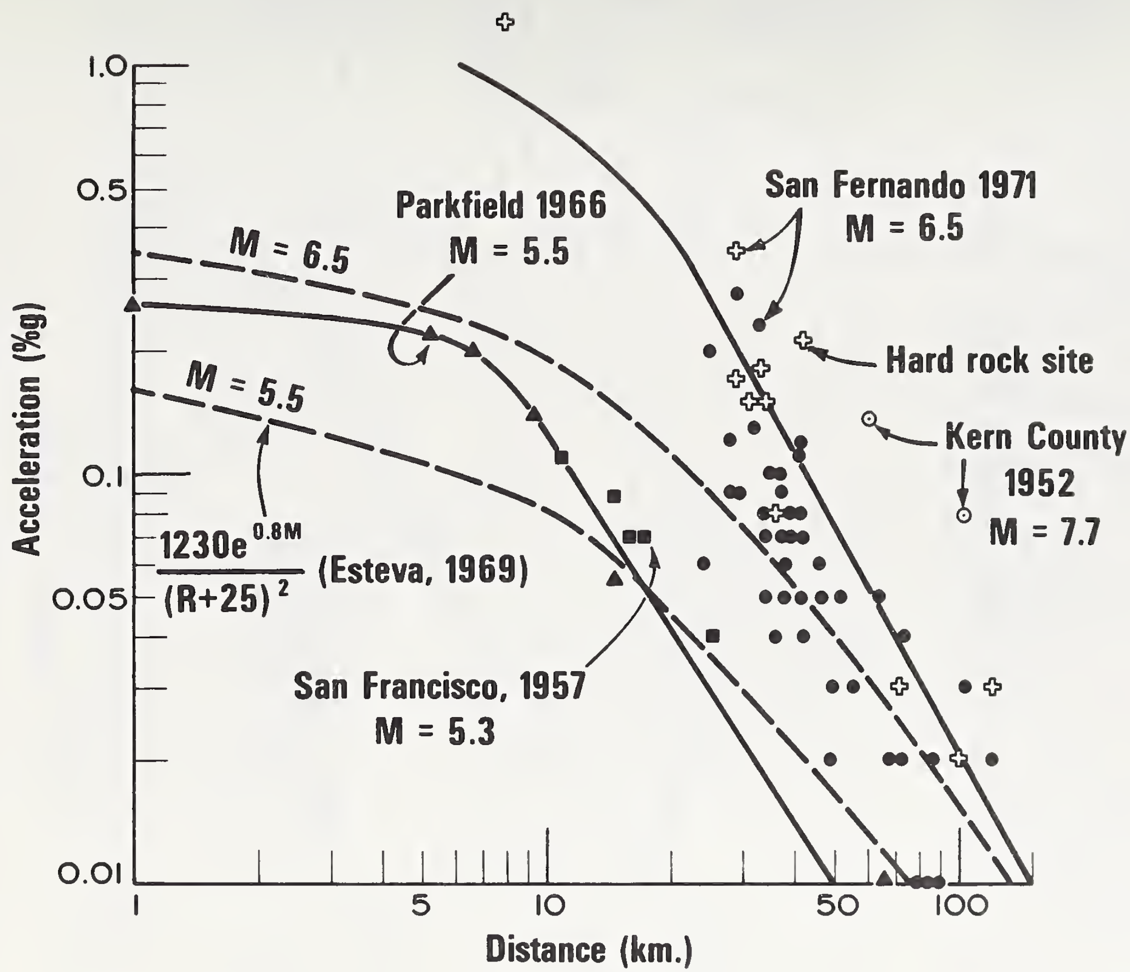

FIGURE 2. SOME EXAMPLES OF ATTENUATION OF HORIZONTAL ACCELERATION WITH DISTANCES

causative fault will be approximately the same as that recorded by strong motion accelerographs in California. With these constraints (at close distances and at distances greater than about 60-80 km.) and by use of Modified Mercalli intensity data (converted to particle velocity, which appears to be fairly closely related to intensity [16, 17]) satisfactory attenuation curves can probably be worked out for most areas. Some estimate of the duration of shaking at moderation distances, greater than about 60 km., from the causative fault can be made by a consideration of surface wave dispersion. Theoretical attenuation curves for California, Nevada, and the Basin and Range area cannot be used for the rest of the United States. Figure 3 shows the relative areas shaken at Modified Mercalli intensity VIII for earthquakes with maximum M.M. intensities of VIII through XI in California-Nevada compared with the eastern United States. The data for California-Nevada are taken from Algermissen, et al [18] and that for the eastern United States from Brazee [19]. These data show that the area shaken at intensity VIII in the eastern United States varies from a factor of about 4 times that in California-Nevada (for Io = VIII) to as much as 40 times (for Io = XI).

The much larger areas shaken in the eastern United States compared to the western United States have very significant implications with regard to seismic zoning. Consider the data in Table 1. In a 100 year period, based on the historical record, approximately 138 earthquakes with maximum intensities of VII and 40 earthquakes with maximum intensities of VIII have occurred in the Zone 3 area of California (using the map prepared by Algermissen [20] shown in Figure 4). In the same period, six earthquakes with maximum intensities of VII and two with maximum intensities of VIII have occurred in the Zone 3 area centered in southeast Missouri. The ratios of the number of maximum intensity VIII and VII earthquakes in California in 100 years to the number of the same size earthquake in southeast Missouri in 100 years are 20 and 23 to 1 respectively (not shown in Table 1). The ratio of the areas shaken in the two areas are 4.5 and 6.4 to 1. If we consider only a portion of California (in Zone 3) equal in size to the Zone 3 area in southeast Missouri and reduce the number of earthquakes that occur in the portion of California in proportion of the area, we obtain the results shown in Table 2. The ratios of the intensity VIII and VII areas in California and in southeast Missouri for a 100 year period are reduced to 1.0 and 1.5 respectively.

The conclusion is that if the two areas are compared on the basis of the total shaken area for a specific time interval the risk is much more nearly the same than if the number of earthquakes of a particular size are compared. A similar argument can be advanced for other Zone 3 area in the eastern United States with more or less success depending upon the reliability of the historical data. This will be developed further in a paper presently being prepared for publication. Not too much significance should be placed on the specific ratios presented in Tables 1 and 2 since the ratios of shaken areas vary considerably with Io as shown in part in Figure 3. The significant point is that the zones shown on the map shown in Figure 4 do represent equal risk in a general way when the time period considered is of the order of 100 years or more and the areas shaken are compared rather than the number of earthquakes that occur in each area.

Site Response

At the present time there are insufficient data available to classify sites according to the physical properties and thicknesses of the materials and relate these properties and thicknesses to the expected intensity of shaking for a range of bedrock intensities. In addition, there is disagreement regarding the behavior of site materials when subjected to strong shaking [21]. It is doubtful if the problem of site response can be solved satisfactorily until a number of additional recordings of earthquake strong motion are obtained at a number of sites (whose properties are well known).

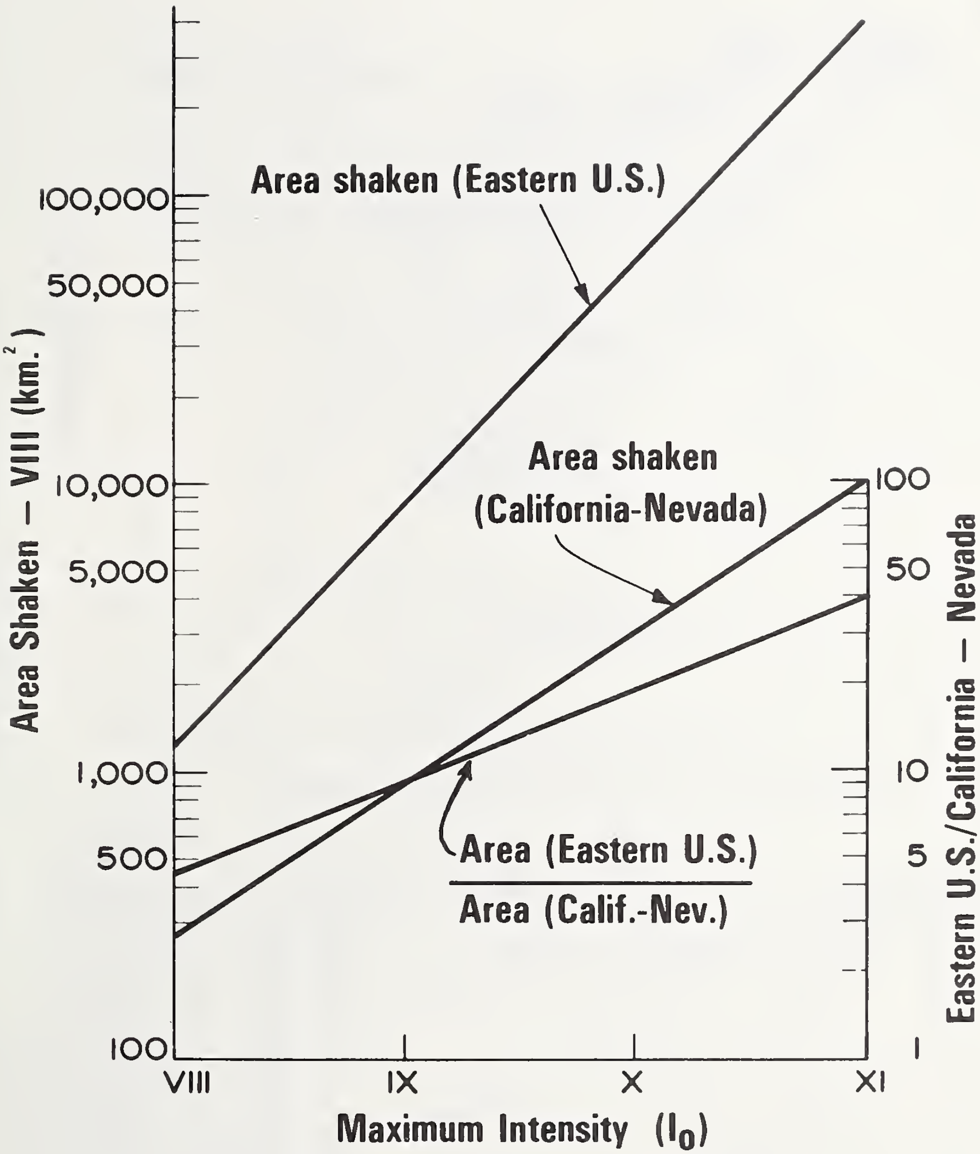

FIGURE 3. AREA SHAKEN AT MODIFIED MERCALLI VIII FOR EARTHQUAKES WITH MAXIMUM INTENSITIES OF VIII, IX, X AND XI IN: (a) THE EASTERN UNITED STATES; AND, (b) CALIFORNIA-NEVADA. THE RATIOS OF THE AREAS SHAKEN IS ALSO SHOWN.

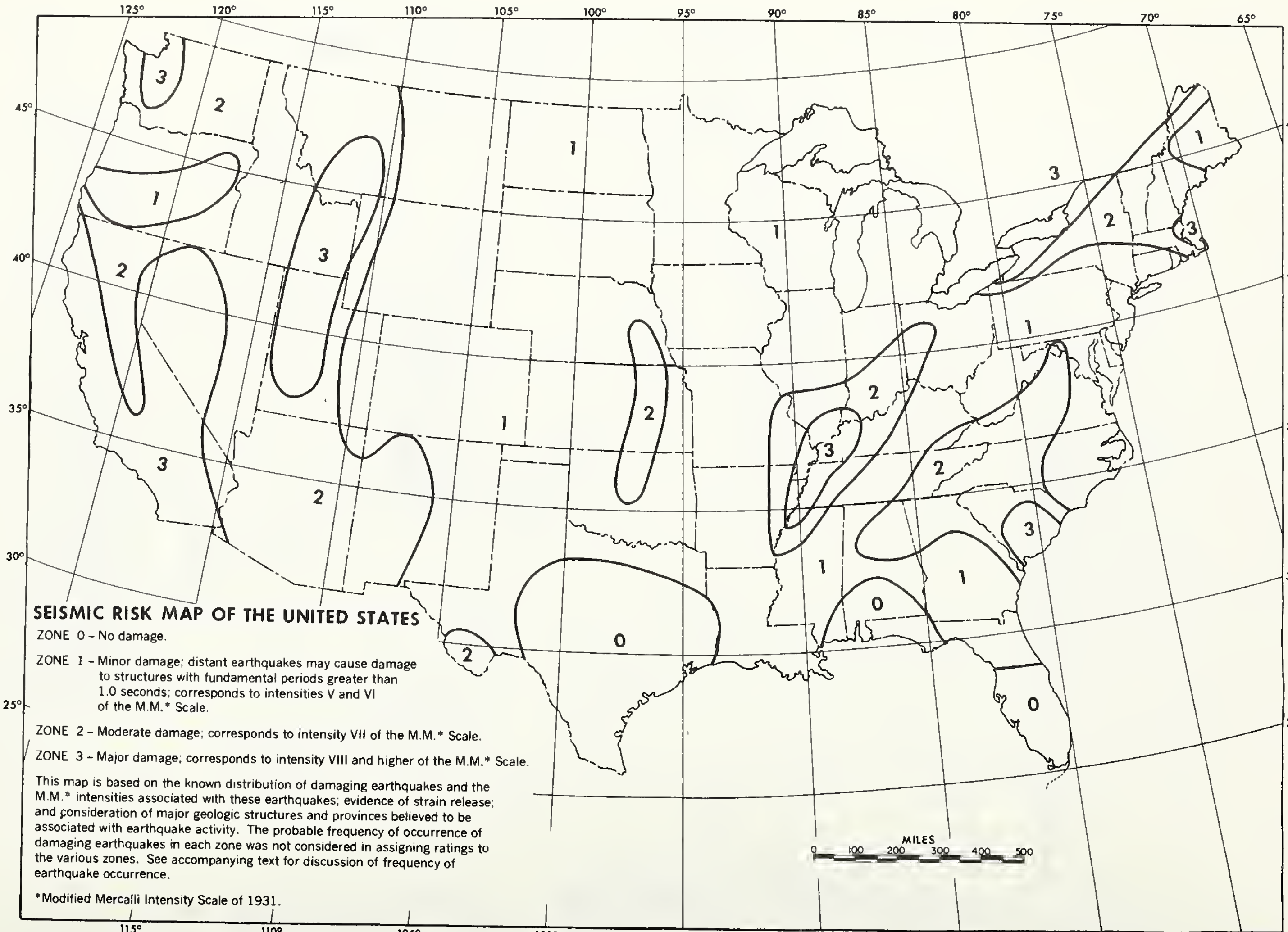

FIGURE 4. SEISMIC RISK MAP OF THE UNITED STATES (ALGERMISSEN, [20])

Table 1

COMPARISON OF AREA SHAKEN IN CALIFORNIA (ZONE 3)

AND SOUTHEAST MISSOURI (ZONE 3) IN 100 YEARS

Area	Earthquakes/100 yrs. Io VII	Earthquakes/100 yrs. Io VIII	Area Shaken (km^2) I VII	Area Shaken (km^2) I VIII	Ratio Area (Calif.) / Area (S. E. Mo.) VII	Ratio Area (Calif.) / Area (S. E. Mo.) VIII
California (Zone 3)	138	40	48300	10800		
					6.4	4.5
S. E. Missouri	6	2	7500	2400		

Table 2

COMPARISON OF AREAS SHAKEN IN A PORTION OF

CALIFORNIA AND SOUTHEAST MISSOURI (ZONE 3) IN 100 YEARS

Area	Earthquakes/100 yrs. Io VII	Earthquakes/100 yrs. Io VIII	Area Shaken (km^2) Io VII	Area Shaken (km^2) Io VIII	Ratio Area (Portion of Calif.) / Area (S. E. Mo.) VII	Ratio Area (Portion of Calif.) / Area (S. E. Mo.) VIII
California Portion of California equivalent in area to Zone 3 S. E. Missouri)	32	9	11200	2430		
					1.5	1.0
S. E. Missouri (Zone 3)	6	2	7500	2400		

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Recordings will have to be obtained at sites over a considerable range of site excitation to satisfactorily resolve the site response problem.

A Suggested Technique for Zoning

The elements involved in a proposed technique for zoning are illustrated in Figure 5. Figure 5A shows a hypothetical source area together with historical earthquakes and known faults. Source areas are chosen such that: (a) they enclose an area of discrete seismicity and insofar as is known, related tectonic elements. Figure 5B illustrates the log N vs. M relationship for the source area shown in Figure 5A. The first step is to decide upon a distribution of earthquakes in space in the source area based upon the log N, M curve. This can be accomplished in a number of ways, two of which are described here. If the relationship between historical seismicity and known recent faulting and tectonic activity is not well known, the earthquake activity in the future (based on the log N, M curved derived from historical data) can be assumed to be equally likely anywhere in the source area in the future. If the source is divided into n smaller divisions and the number of earthquakes in the magnitude range ΔM is $N_{\Delta M}$, then the number of earthquakes likely to occur in the magnitude range ΔM in each small division or block of the source area is:

$$\frac{N_{\Delta M}}{n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

If some particular part of the source area is believed for some geophysical or geological reason to be more active than other parts of the source area, the seismic activity in certain of the small divisions of the source can be increased (weighted) and the activity in other parts of the source decreased. In this case the slope of the log N, M curve, (b) would remain constant for the entire source region but the intercept (a) would vary over the area. The only restriction is that the total number of earthquakes in the range M_1 to M_2 in the source area cannot exceed the number predicted in the range of M_1 to M_2 from the log N, M curve for the source area.

Once the distribution of earthquakes likely to occur in each small division of the source is decided upon, the effect at each site due to the occurrence of earthquakes in each small division of the source can be computed using an attenuation curve such as those shown in Figure 5C. In practice, the distribution of acceleration (velocity or some other parameter) would be computed for a large number of sites on a grid pattern and the source would also be included in the grid pattern.

From the distribution of acceleration (or some other quantity) at each site (Figure 5D) it is possible to determine directly the number of times a particular intensity of shaking is likely to occur in a given period of years at a given site.

The system has a number of advantages, some of which are outlined below:

1. The technique allows for incorporating geologic information concerning earthquake occurrence into the source. For example, suppose that an active fault is known in the source area but that historical seismic activity has been largely concentrated at one end of the fault. Future seismic activity can be postulated as occurring at (a) the same end of the fault as the historical activity; (b) concentrated at some other position on the fault, or (c) distributed equally or in some other manner along the fault. The choice would depend upon the amount of geophysical and geological information available

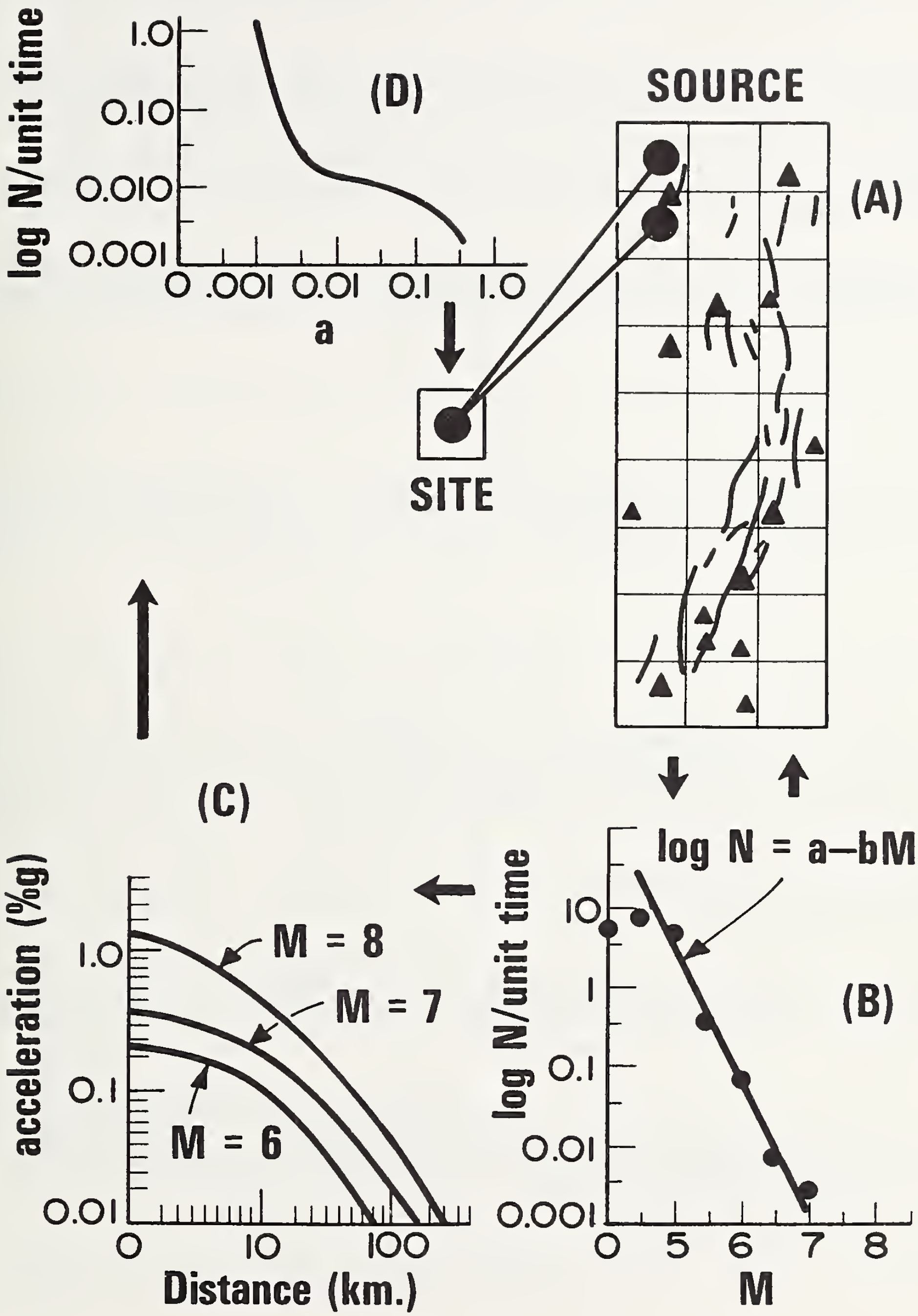

FIGURE 5. ELEMENTS INVOLVED IN PROPOSED ZONING METHOD

at the time the map is made and the best current hypothesis of the nature of earthquake occurrence.

2. The technique does not require the assumption of a specific distribution or process such as the Poisson process.

3. Programmed for a digital computer, variation of the parameters involved becomes an easy matter. Variation of parameters is considered important because it is not obvious at the onset which parameters will most influence the distribution of shaking at a particular site. For example, the effect of changes in the log N, M curve for the source area can easily be evaluated.

4. The technique allows for the construction of many source areas, depending upon the amount of geological and seismological data available.

Summary

This paper has discussed some of the problems in preparing suitable zoning maps and has suggested a technique for the construction of zoning maps which provide flexibility in the use of seismic and geologic data together with a quantitative evaluation of the geophysical risk.

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WIND HAZARDS FOR BUILDINGS

by

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and

John J. Healey**

Introduction

This paper presents a review of the current methodology and criteria for determining wind loadings on buildings. The objectives of the paper are to:

(1) evaluate practices in current use and identify the best current practices;

(2) identify priorities for developing improved practices; and

(3) recommend high priority research activities to advance the state of the art and fill gaps in knowledge.

In the following pages this review is presented in the following order of topics:

(1) Meteorology and Climatology

(2) Drag Loading and Gust-induced Response

(3) Aerodynamics and Aeroelasticity

(4) Wind Tunnel and Full-scale Investigations

(5) Recommended Research

(6) Bibliography

While it is not the purpose of this paper to assemble and present detailed wind loading coefficients and other data which are currently being used in design calculations, this information may be obtained from the references.

Meteorology and Climatology

For the purpose of this discussion, it is convenient to divide the lower atmosphere into three regions: (1) the surface layer up to about 100 feet; (2) the "tower layer" up to about 500 feet; and (3) the planetary boundary layer up to about 2000 feet.

The wind profiles in the surface layer are quite well understood over homogeneous terrain. For engineering purposes, the terrain is considered homogeneous if the upwind fetch is of uniform roughness for a distance of about 20 times the height in question. Over uniform terrain the variation of

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wind speed with height can be completely described by the roughness length of the terrain, the friction velocity or drag coefficient and a quantity related to the vertical distribution of temperature, such as the Richardson Number. The actual equations, [1]* however, though well understood, are very complex and the current practice is to describe the variation of wind speed with height in the surface layer by a power law equation of the form,

$$V/V_1 = (Z/Z_1)^P$$

where V and V_1 are the wind speeds at heights Z and Z_1 and P is an exponent which depends chiefly upon the roughness of the terrain and to a lesser extent upon temperature and wind speed. This exponent can be estimated quite well from meteorological theory. For example, in strong winds the effects of temperature and wind speed on P are negligible and P is equal to the reciprocal of Ln (Z/Z_0) where Z is the geometric mean height over which the power law is to be applied and Z_0 is the roughness length which is typically a few inches over flat open country, 1-2 feet over built-up suburban areas and 10 feet or greater in downtown areas. The exponent tends to be smaller in unstable air where the temperature decreases rapidly with height compared to the exponent for stable, strong wind conditions.

It has been observed that wind speeds over the surface layer are greater than those obtained by extrapolating vertically the power law formula for the surface layer. It is known that the Coriolis force begins to influence the wind profile above the surface layer and the general form of the equations describing this phenomenon has been determined [1, 2]. However, the equations contain several coefficients which are not well known.

Above the tower layer the wind speed will continue to increase gradually with height up to the gradient height; that is, the height at which the effect of surface roughness becomes insignificant. Average gradient heights are known to lie in the range of about 900 feet over flat open country to about 1500 feet over downtown areas. Above the gradient height the wind speed can be assumed independent of height, particularly for strong wind conditions. Also, above the tower layer the wind turns clockwise in the northern hemisphere by about one degree for every 100 feet of elevation.

Although, strictly speaking, the above power law formula is appropriate only in the surface layer, the best current estimates of the variation of mean wind speed up to the gradient height are based on this simple law. On the basis of reported surveys of the variation of wind speed with height [3], typical values of the exponent, P, in current use are 1/7, 1/4.5 and 1/3 for open country, suburban areas and city centers, respectively; the corresponding gradient heights are about 900 feet, 1200 feet and 1500 feet.

To account for the effect of terrain roughness on the variation of wind speed with height, the current practice is to make use of the wind speed at the gradient height. Unlike airports where routine measurements of basic wind speeds are taken for statistical analysis, the centers of large cities and suburban areas have greater surface friction. Hence, mean wind speeds and velocity pressures in downtown areas will be less than for open country at the same height. To adjust for this phenomenon, use is made of the fact that the wind speed at the gradient height is independent of surface roughness. The power law formula becomes [4],

$$V_Z = 1.63 V_{30} \, (Z/Z_g)^P$$

where V_Z is the mean wind speed at height Z, V_{30} is a reference or design wind speed at a reference height of 30 feet at a nearby airport station and P and Z_g are the appropriate power-law exponent and gradient height, respectively, for the terrain in question.

*Numbers in brackets refer to the similarly numbered references at the end of this paper.

In the recommendations of the ASCE Task Committee on Wind Forces [5], the variation of wind speed with height is based on conditions in only open level country but a distinction is made between coastal areas and inland areas. For inland areas the 1/7 power law speed is recommended by the ASCE Task Committee. For coastal areas, however, the task committee recommended a power law exponent which varies with wind speed. While this recommendation is based on observations in coastal areas, it is inconsistent with the current belief that the variation of wind speed with height is dependent chiefly upon terrain roughness rather than on wind speed or proximity to large bodies of water, and additional measurements are required to resolve this difference.

Also, there is considerable doubt as to whether simple power law equations are valid under all conditions for estimating the variation of mean wind speed with height in large cities.

Current codes of practice on wind loading such as ANSI [6] do not permit reductions in velocity pressures due to direct shielding afforded by adjacent buildings or structures or by terrain features. However, in the new ANSI Standard the cumulative effect of upwind structures and/or terrain features on the transition in the wind profile and the attendant reduction in pressures is permitted as described above.

For the centers of cities direct shielding can result in markedly reduced loads for certain wind directions. On the other hand, buffeting in the wake of upwind structures plus channeling of the flow between adjacent buildings can increase the loads. Current codes including the new ANSI Standard require that such increases in loading be taken into account, but the magnitudes of such increases are known only approximately.

Although not explicit, it is not the intention of the new ANSI Standard to disallow the use of shielding effects obtained on the basis of wind tunnel tests in which the surrounding structures and terrain features are carefully modeled provided that any increases in pressure or suction as a result of such obstructions are considered in design.

Current American practice makes use of the latest National Weather Service data on extreme fastest-mile wind speeds [7] as a basis for establishing design wind loads. These speeds are measured by recording the time it takes for a mile of wind to pass a fixed point with an anemometer which makes an electrical contact with the passage of each mile of wind. The measurements are obtained at airports and at other open country locations where the exponent in the power law for wind speed with height is about 1/7. Although observations may be made at different elevations, they are adjusted to the standard 30-foot elevation prior to analysis by means of the 1/7 power law [7].

Since the extreme wind speeds of each year govern the annual maximum wind load on a structure, only extreme speeds, corrected for instrument errors, when necessary, are considered in determining design values. The annual extreme series for each of 138 Weather Service Stations were fitted by Thom [7] with a Fisher-Tippett Type II extreme value distribution to establish design values associated with various probabilities or risks of being exceeded in any given year. The associated mean recurrence intervals are the reciprocals of these probabilities and each gives the average time interval in years between the occurrence of all extreme winds exceeding the design value.

In order to perform statistical analysis of the dynamic response of structures to atmospheric turbulence, it is necessary to obtain improved estimates of wind gust spectra and co-spectra. While a limited amount of such spectra is available, many of the values are still controversial, particularly with regard to thermal gradients. It appears that wind speed time series at high speeds exhibit departures from stationarity which may be critical in the analysis of the data. In most applications, wind fluctuations are assumed to be Gaussian which is approximately the case for strong winds. However, much of the available data on atmospheric turbulence is for relatively low wind speeds. Indications are [1, 8] that the character of turbulence may be quite different at wind speeds affecting the design of structures and much more data is needed at such speeds.

Since mean wind speeds are known to be lower in the centers of large cities compared to speeds in flat open country, accurate information on the transition in the wind speed profile between smooth and rough exposures is required in order to transfer meteorological data for open exposures to suburban areas and city centers, and to determine the upwind fetch required for this transition to take place. Only one form of heterogeneous terrain has been studied to data--a change of surface roughness along a line at right angles to the air flow. The results are considered reliable for strong winds but, unfortunately, such simple geometry is rare. In particular, an abrupt change in surface roughness produces an internal boundary layer which causes a kink in the wind profile. The kink occurs at a height of about one tenth of the distance to the roughness change. "Regional" or micro wind maps are required in locations where the transition in the wind speed can have a major effect on relative construction costs. These maps should give the annual probability of exceedance on wind speed as a function of wind direction and should be continually updated as the terrain becomes more heavily built up.

Drag Loading and Gust-Induced Response

Important problems associated with establishing the wind loading and dynamic gust response of buildings or structures in the direction of the wind include: (1) the effects of shielding and channeling plus topographical features on the free stream wind; (2) the assessment and distinction between high local pressures on the cladding versus the lower overall pressures for design of the main frame; (3) the effects of nearby structures on the flow pattern; and (4) the effect of the building or structure dynamic response characteristics.

A major innovation in recently adopted building codes and standards on wind loading (e.g. the National Building Code of Canada (1970 Edition) and the ANSI Standard) is the concept of varying load requirements both with the dynamic properties of the building or structure and with the roughness of the terrain. With this newly adopted practice the calculation of wind forces involves the use of three numerical factors: (1) an exposure factor which is intended to account for the boundary layer flow over the topography or terrain at the site; (2) a gust response factor which is intended to translate the relatively complex wind-gust induced dynamic response phenomenon into an equivalent peak static load; and (3) a pressure coefficient for the various surfaces and for local tributary areas which relates the pressures and suctions on the various surfaces or portions of a structure to the dynamic pressure of the design wind speed. The formula for computing the effective velocity pressure for design purposes is thus written,

$$q_Z = K_Z G q_{30}$$

where q_Z is the effective velocity pressure at height Z above the ground, K_Z is the exposure factor which depends upon the roughness of the terrain and height above ground, that is, upon the free stream boundary layer, G is the gust response factor which depends upon the dynamic response of the building or structure to turbulence and q_{30} is the design velocity pressure at the reference height (usually 30 feet). The velocity pressure is the computed force per unit area of surface perpendicular to the wind, representing the kinetic energy per unit volume of moving air. It may be expressed by the product $K_Z \times q_{30}$, which is influenced only by geographical location, air density, height above ground, and exposure. The effective velocity pressure, on the other hand, takes into account the additional effect of turbulence on the dynamic response of the building or structure and is affected by the gust response factor G.

Although power spectral techniques have proved to be powerful tools for analyzing the dynamic response of structures to random wind loads and for formulating gust response factors for inclusion in building codes, standards and handbooks, the application of these techniques in the estimation of gust response factors has resulted in major differences between the results obtained among several investigators [4, 9, 10]. The major reason for these differences is the area over which the gust loading is correlated for the structure as a whole. For example, Vellozzi and Cohen [4] correlate the gust pressures over the entire surface of a building or structure and arrive at somewhat lower gust response factors compared to the results of Davenport [10] and Vickery [9], who correlate the flow only over the windward face. A resolution of these differences requires a better understanding of the mechanism relating free-field wind fluctuations to the fluctuating loads on the windward and leeward faces of a structure. Although wind tunnel tests in boundary layer wind tunnels [11, 12, 13, 14, 15, 16] have provided significant insight into this problem, the correlation between such tests and full-scale measurements is relatively poor and it is recommended that a greater emphasis be placed on full-scale programs to resolve this deficiency.

The general form of the equation for the gust response factor used in current wind loading practice is given by,

$$G = 1 + g\ (\sigma/p)$$

where σ/p is the ratio of the standard deviation of the structure dynamic response to the mean or average response, and g is a peak factor which relates the probable peak dynamic response to the root mean square of the response. It has been shown theoretically that the peak factor can vary over the range of about 2.5 to 4.0 depending upon the natural frequency and damping ratio for the building or structure. The peak factor will increase as the natural frequency increases because the number of cycles of vibration in a given period will increase and the chances will be greater that a given response level will be exceeded. Consistent with the aerospace industry, an accepted practice is to use the usual peak factor of 3.0, regardless of frequency. This corresponds to a probability of 0.9973 that the peak value will not be exceeded.

The pending ANSI Standard makes a distinction between gust response factors for the design of the main frame of buildings with those for the design of parts and portions such as glass, curtain walls and cladding. This is because average effective gust pressures tend to increase as the exposed tributory area decreases. As a consequence, the gust response factors in current use for parts and portions of structures are somewhat greater than the gust response factors for the building or structure as a whole.

For ordinary buildings or structures which are usually defined as being less than 200 feet in height and having a ratio of height to least width less than 4, the current practice is to use simplified or average gust response factors which account only for the intensity of the free stream turbulence plus the size of the building or structure on the correlation of the gust pressures. However, the structures which are wind sensitive (i.e., greater than 200 feet in height) and for tall slender structures such as skyscrapers, towers, stacks and masts having dynamic properties which tend to make them wind sensitive, a detailed gust loading analysis, in conjunction with wind tunnel tests in boundary wind tunnels, is generally required. Since the dynamic response of tall flexible structures to wind loading is critically dependent upon damping, the accepted practice is to use conservative damping levels of about one percent for steel structures and two percent for concrete structures, unless properly substantiated test data are available for establishing higher (or lower) values. For cantilevered structures such as tall buildings and free-standing towers, it has been observed [17] that the deflected shape of the structure is the same for both the mean wind loading and for the dynamic gust loading, or stated differently, that the wind loading does not excite higher modes of vibration. This phenomenon leads to a single gust response factor for the building or structure as a whole even though the "intensity" of the gusts in the free stream decreases with height. However, for structures such as guyed towers and cooling towers, the wind has been observed to excite higher modes of vibration and a single gust response factor would not suffice in these situations.

In the case of very large structures, it has been observed [4], at least theoretically, that the total effective velocity pressure for design purposes can be less than the fastest mile pressure, which is currently used as a basis for computing design loads. In other words, a gust response factor which accounts for the turbulence over the fastest mile of wind can be less than unity for very large structures because: (1) the fastest mile of wind may not be of sufficient spatial extent to permit the ultimate pressures to fully develop on a large structure and (2) the fluctuations in wind speed over the fastest mile of wind are so irregular and uncorrelated that they are ineffective in producing an increase in the gross loading for large structures. Nevertheless, current practice does not permit gust response factors which are less than unity. The effective velocity pressure for a gust response factor of unity is equal to the fastest mile pressure.

Although the gust response factor increases as the roughness of the terrain increases, the exposure factor, K_Z, and hence the fastest mile windspeed, decreases with an increase in roughness. The decrease in fastest mile pressure is generally greater than the increase in the gust response factor and the net result is that the total wind loading is less for a suburban or downtown area than for flat open country by a factor of about 2 to 3. Neglecting wake effects and vorticies shed from nearby, upwind structures, it follows that the gust-induced vibrations for city structures will generally be less than for similar structures situated in flat, unobstructed exposures. However, the percentage of the total deflection, stress, etc. due to gust loading will usually be greatest for city structures.

An important advantage in treating the wind loading in terms of an exposure factor and a gust response factor in arriving at an effective velocity pressure is that it aids the designer in understanding the dynamic nature of wind loads and the application of turbulent, boundary layer flows to real structures. Although the gust response factors and exposure factors in current use are based on the best information available, a continual measurement program of gust effects and wind speed profiles, particularly for built-up exposures, will be required for a period into the future in order to continually improve our assessment of gust loads and effects.

The pressure coefficients used in current practice are based largely upon wind tunnel tests conducted under conditions of smooth, uniform flow [18, 19, 20, 21] and their validity in extremely turbulent, boundary layer flows has not yet been completely established. There is evidence [22, 23] to indicate that suction pressure on leeward walls in urban areas are reduced by the effects of large scale turbulence. However, the allowances which can be made for these effects will require additional research on both model and full-scale structures.

Whereas the practice in the old ASA Standard on wind loading for computing gross loads on buildings was to use a single drag or shape coefficient, the current ANSI practice is to use consistent external and internal pressure and suction coefficients and to compute the net wind loading by taking the algebraic difference between the external and internal pressures. That is,

$$P = q_Z(C_p - C_{pi})$$

where P is the net wind loading, q_Z is the effective velocity pressure, Cp is an external pressure coefficient and Cpi is an internal pressure coefficient. Internal pressures will not affect the gross drag loads which tend to produce sliding and over-turning, but can affect the design of the main frame and walls and must be considered. Consistent pressures and/or suctions for both the windward and leeward walls plus the roof must be applied simultaneously in determining the maximum stresses.

With the exception of stack-like structures and signs, it is currently recognized that drag coefficients for gross wind effects are not adequate for the complete design of buildings and other enclosed structures. Rather, it is necessary to have accurate information regarding average and peak pressures and suctions for each of the various surfaces of buildings in order to perform a rational design. A general drag coefficient, although adequate for the study of gross effects, gives no indication of the proportionate wind load on each surface, especially for the side walls. Prior to the new ANSI Standard, the wind pressure used in the design of the main framing of buildings was generally assumed to act on only the windward wall.

The intensity of local suction pressures on the exterior surfaces of buildings is known [14, 18, 22] to exceed the mean dynamic pressure of the approaching wind by factors as high as 5. These high pressures are caused by one or more of the following mechanisms: (1) action of gusts arriving at the structure in the free stream wind; (2) buffeting in the wake from an upwind structure; (3) channeling of the flow between adjacent buildings; (4) reattachment on the structure of flows which have been separated because of sharp edges, corners, or other architectural features; (5) local vortex generation by exterior architectural features.

The new ANSI Standard presents local pressure coefficients which are intended to account for these phenomena. The values are based on measurements in both wind tunnels and in full-scale investigations. The new coefficients, in certain cases, are greater than those specified in the earlier edition of the standard by factors as high as 3 and in other current codes of practice; however, there have been sufficient structural failures due to wind forces (e.g. during the recent hurricanes, Camille and Celia) related to inadequate anchorage of individual panels and roof elements, including glass breakage, to justify the increased magnitude of the new coefficients.

Relatively little quantitative information exists regarding the effect of building openings, building volume and layout of internal partitions on the internal pressure changes within the building during strong wind conditions and a great deal of research is needed in this area. The effect of building openings and building volume on the net loading is significant during the passage of a tornado. In fact, an effective means of alleviating damage due to severe pressure gradients during tornadoes is the addition of venting systems (openings or blow-out panels) to provide a direct release of the internal pressure during the low pressure phase.

The significant loads to which a structure is subjected to during the passage of a tornado include the following: direct pressure on the windward face, outward pressure on the leeward face due to pressure deficit, outward pressure on the sides, roof and leeward face resulting from the low pressure in the vortex, and forces due to missiles striking the structure.

Aerodynamics and Aeroelasticity

The chief aerodynamic and aeroelastic problems in the present context center about the mechanical and fluid instabilities associated with the flow around bluff bodies, since the ground-based structures in question are usually not streamlined or designed primarily for aerodynamic lift or efficiency. Flow-induced mechanical oscillations and flow instabilities are the focal points of interest here. Associated with flow-induced mechanical oscillations are the numerous galloping and flutter phenomena, whereas flow instabilities are associated with the velocity and pressure fluctuations occurring in vortex shedding, flow through screens, edge flow instabilities, etc. While the above phenomena are understood reasonably well for smooth uniform free-stream flow, the phenomena for turbulent, boundary layer flow occurring in nature are not well understood.

Since the flow around bluff buildings and structures is subsonic and essentially incompressible, the influence of the wake conditions upon the gross wind forces is great and of paramount importance towards an understanding of the flow phenomena. The effect of the Reynolds number upon wake flow and wake-body interaction is not well explored so that the ranges of validity of the results of wind-tunnel tests become an important consideration. There is a need for Reynolds number effects to be evaluated in terms of both the local and gross flow patterns around bluff bodies in order to give physical insight into flow instabilities.

While the study of the flow over a circular cylinder is still a formidable problem, the study of flow over various bluff bodies of sharp-edged cross section may well prove to be of equal, if not greater, importance towards as improved understanding of structural wind problems.

Because of the relative analytical intractability to date of flow around bluff bodies, suitable analytical models of instability for such bodies are virtually non-existent. However, the use of potential flow models of bluff bodies in the presence of wake vorticies [23] appears to be of some promise, and, generally, the judicious use of computer models of the flow can provide insight into certain instability phenomena although this process is time consuming and relatively costly. Consequently, experimental research will probably remain as the area of interest for a period into the future. It will be of importance in this area for the experimental aerodynamicist to observe and record key flow characteristics which provide a broad understanding of the phenomena. The need also exists for identifying and classifying those flow phenomena (such as separation phenomena) which are present in the various classes of instability so that an ordered picture can be established of the aeroelastic phenomena of bluff bodies. Recent research [24, 25] has been concerned with identifying the non-linear phenomena of galloping and vortex-induced oscillations of bluff bodies and it seems promising that sound dynamical models of these oscillations, with the unsteady aerodynamic forces as empirical inputs, can be established theoretically.

Those classes of instability which can occur for streamlined structures (such as flutter in coupled torsion and translation modes) presently require somewhat less attention than the bluff-body problem because of the strong emphasis already given to them by aeronautical engineers. They are nevertheless of fundamental importance as a background towards an improved understanding of bluff-body aerodynamics.

The effect of upstream turbulence upon the flow over bluff bodies is of fundamental concern. To simulate this phenomena, it becomes necessary to establish experimental methods for adequate representation of the turbulent boundary layer flow occurring in natural winds. It has been observed that turbulence in the free stream can delay the onset of certain aeroelastic instabilities such as flutter and in certain cases can practically eliminate them due to turbulence-induced changes in the flow over the body or due to reduced correlation between random flow effects at separated points on the body.

Continued development of representative test and simulation methods, so that similarity to full scale in enhanced, is necessary with regard to forecasting the performance of complex structures in natural wind environments. Such methods are presently the last effective resort in particular cases where existing analytical methods cannot follow the complexities of the actual full-scale environment.

Wind Tunnel and Full-Scale Investigations

Because of the complexity of turbulent boundary layer winds in the atmosphere, it is unlikely that quantitative solutions of the wind loading and dynamic response phenomena for real structures can be obtained without the aid of wind tunnel tests. Moreover, for proposed structures of taller and lighter construction and having higher strength materials and lower overall safety factors compared to "traditional" designs, modeling in a properly simulated wind environment produced in the laboratory may be the most economical if not the only way to obtain the required information.

The scaling criteria required to achieve dynamic similitude between model and full scale are well known [24]; however, these criteria are extremely difficult to satisfy, even when special purpose wind tunnels are used. The best practice is to adhere to the following criteria, whenever possible, in wind tunnel investigations: (1) Full scale characteristics of turbulent boundary layer flow including mean wind speed profile, distributions of scale and intensity of turbulence, and thermal gradients should be simulated; (2) Adjacent structures and terrain features which influence the flow should be modeled; (3) Where dynamic response of the building or structure is important, suitably scaled aeroelastic models should be used.

In general, boundary layer type wind tunnels, such as those at the University of Western Ontario and at Colorado State University, in which the floor of the tunnel is roughened artifically to simulate natural terrain, are highly desirable for simulating natural winds. However, other methods of generating turbulence have been found to be successful including the use of graduated grids and screens, air jets, shutters and vortex generators [14].

There is need to establish correlations between models and full-scale measurements, particularly with regard to the difference in Reynolds number. All too often, however, the results of such tests are considered proprietary and are not generally available to the public.

Although the Reynolds number for the model scale is typically only 1/300 to 1/500 of full scale values, the results for sharp-edged models are considered reliable because of the fixed position of the flow separation. This is not the case, however, for rounded structures such as stacks and cooling towers, and more full-scale studies should be conducted on such structures.

The Commonwealth Advisory Aeronautical Research Committee has proposed a round robin of tests using a standard prismatic model to aid researchers in evaluating results obtained from different wind tunnels. This model was scaled from an actual building on which surface pressures have been measured so that the different wind tunnel test results can also be correlated with the full-scale measurements. Such a program affords an excellent opportunity to fully evaluate all aspects of the flow simulation including effects of Reynolds number and wind tunnel blockage, and would be a worthwhile undertaking in the United States.

In addition to the need to simulate the characteristics of turbulent, boundary layer winds, tests to determine the dynamic response of structures to gusty winds necessitate that suitably scaled aeroelastic models be used. A major limitation is the availability of modeling materials with the required physical properties. In the case of tall slender structures where the dynamic response due to wind gusts is predominately in the first mode of vibration, one approach [14] has been to use a rigid model which is permitted to rotate about its base on a gimbal. The gimbal, in turn, makes use of springs and electro-magnetic dampers to simulate the modal stiffness and damping of the full scale structure. The assumption here is that the full-scale structure deflects in essentially a straight line which is approximately the case only for framed buildings with shear wall cores. The errors that result when the predominant mode of vibration is not a straight line should be explored. Since the modal damping is not generally known for unique structures, a range of probable damping coefficients is covered in the test program to identify worst-case conditions.

While the use of spring mounted rigid body models appears to be satifactory for many structures, this technique is not appropriate when the critical mode of vibration is not known a priori or when the response occurs in several modes of vibration. Shell structures such as cooling towers and membrane type roof structures, plus suspension bridges, are examples in which particular attention must be paid to aeroelastic modeling in order to obtain meaningful results. For suspension bridges, it has been found that the results from tests on aeroelastic models of the full bridge always present a more favorable picture than tests on spring-supported section models [13].

Full-scale investigations of structures is an area in which considerable effort has been expended within the past four years. For these investigations various types of instrumentation and measurement techniques are being developed. Hot film anemometers and Doppler radars are presently being used in atmospheric studies but they are still in a state of development. Concurrent with this development there is a need to continually improve pressure transducers and to develop more reliable laser devices and tilt-meters to record slowly varying drifts of buildings. The problem of obtaining adequate field data on mean velocity profiles, particularly in urban areas cannot be overemphasized. Much useful information on wind characteristics already exists and should be compiled. To accomplish this, a task committee (perhaps an ASCE Committee) should be established to assemble information on wind characteristics as it becomes available and to coordinate new research programs.

Because there is an increasing interest in the total performance of structures there appears to be little doubt that Monte Carlo methods will eventually be applied to the wind loading problem in structural design. For this purpose magnetic tapes of detailed observations will be needed, especially under critical conditions. In addition, meteorologists devote most of their efforts at present to real time or forecasting problems which are of little value in the planning and design of future structures. More effort should be devoted to the non-real time or climatological problems with special emphasis on the internal characteristics of storms.

With regard to full-scale investigations of wind forces on buildings and structures, it is recommended that companion wind tunnel tests be carried out whenever possible. This recommendation is emphasized for rounded structures where the effect of differences between model and full scale Reynolds numbers is presently unknown.

While the primary motivation of full-scale investigations has been the correlation with wind tunnel tests, information necessary to establish distributions of extreme loads should also be obtained from these investigations. Such studies should establish as a minimum the relationship between the wind speed fluctuations in the free flow and the pressure fluctuations on the structure. Supplementary measurements of the dynamic response should be correlated with the wind speed and pressure data using spectral techniques.

In addition to the data obtained from elaborate instrumentation set-ups on specific structures, there is a need to obtain long-term field data involving minimal instrumentation of several structures within a given region.

Recommended Research

As a final summary, research activities which should be given priority are listed below:

1. - Long term measurements should be taken on full-scale structures for the purpose of establishing distributions of extreme loads, effects of repeated loads and damping ratios.

2. - An urgent effort should be made to establish correlations between model and full-scale measurements.

3. - A task committee should be formed to compile existing wind data relevant to structural design and to coordinate future wind research programs.

4. - A standard aeroelastic model of a full-scale building should be tested in various wind tunnels to afford an evaluation of the different methods of simulating turbulent boundary layer winds. The results should also be compared with measurements on the prototype.

5. - Meteorologists should be encouraged to devote more effort to the non-real time or climatological problems with special emphasis on the internal characteristics of storms.

6. - A meteorological survey report should be prepared which summarizes the existing meteorological and climatological data which is relevant to structural design.

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LAND USE PLANNING AND NATURAL DISASTER MITIGATION

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I. Introduction

The land use planning process consists of a series of surveys, analyses and policy decisions starting at the general level and ending with specific regulations regarding the construction, use and function of a building or structure on a specific piece of ground. As such, land use planning encompasses the full range of policy-planning-implementation actions of any governmental unit engaged in disaster mitigation.

Land use planning must be interdisciplinary in nature. Although the major early contributions came from engineering, architecture and the law, the disciplines of public administration, political science, economics, geography and sociology have added new information and understanding. Modern planning must take into account these new perspectives.

Thus, land use planning must be reviewed as a key component of governmental planning. As a key component, land use planning will be concerned with the totality of disaster mitigation, and not merely the physical location of buildings. There must be concern for the older and already developed urban areas as well as the open and as yet undeveloped rural areas.

A plan can be only a guide to implementation of a program designed to serve the public good. The impact of disasters, such as earthquake or tornado, can only be diminished if the programs and regulations set forth in the plan are implemented. To be effective in disaster mitigation, land use planning must successfully integrate the policy process with the implementation program. Only through this successful integration will the public good be served.

Generally, implementation occurs through the enforcement of zoning regulations, subdivision ordinances and building codes. It is at this point

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in the planning process that the best of plans may fail. Failure, as such, is usually manifested in a deviation from the regulation, and is frequently granted because of political pressure. This pressure can usually be shown to result from economic concern or from a lack of full comprehension of the risks involved.

Thus, a major difficulty in the area of land use planning for disaster mitigation at any governmental level is the significance, largely in economic terms, of the impact of plans and regulations, and the resultant argument that the "state-of-the-art" does not warrant such rigid controls. It should be apparent then that effective policy can only be developed when data and information are made available by the technical community in such a manner that all parties (political, administrative and citizen) can comprehend the impact and risk of various decisions.

Finally, a fundamental question to be addressed in disaster mitigation is the relative role of the federal, state and local levels of government in establishing policy which effectively guides urban development. Generally, it is only when on level of government fails to respond adequately to the public need that another level of government enters into the process. This has been fully demonstrated in the area of environmental legislation.

There are indications that review and intervention by higher levels of government may be forthcoming for local land use planning agencies concerned with disaster mitigation.

This paper attempts to address each of the above areas. The paper is divided into seven sections, including the Introduction and a list of recommendations for research.

Following the Introduction, there is a discussion of the function of land use planning in disaster mitigation. The third section presents background on the responsibilities of the levels of both organization and government in the planning function. Section four discusses various natural disasters that can be mitigated through effective planning. Current practices in land use planning which may be used in disaster mitigation are presented in section five. Finally, an approach to seismic impact assessment for general planning purposes is presented in section six.

II. Function of Land Use Planning in Disaster Mitigation

Natural disasters may be "acts of God," but unnecessary losses in life, property, and social disruption often result from improper land use in disaster-prone areas. Within federal, state and local levels attention is now focusing on providing a wider and more effective range of methods to mitigate the effects of these disasters. Land use planning and regulation at the state and local level to guide private and public uses of land in hazard-prone areas can be one of the important management tools for avoidance of major losses.

There are, however, some formidable difficulties in making this tool operationally effective in the application to hazard mitigation. As background to consideration of the problems and the promise of land use planning as an approach to disaster mitigations, a discussion of the development and function of planning in government is provided below.

Land Use Planning: Overview

"Since planning is directed primarily toward the end of rational and systematic decision-making by government, the concerns and scope of planning should be as broad as the concerns of the government it serves, the activities the government engages in, and the decisions the government must make. The scope of planning must also relate to administrative devices available, and the tools at hand to carry out proposals." [1]

Planning, as a separate and identifiable function of government, began in the cities of this country early in the century. Early city planning was almost totally concerned with the physical aspects of the city. "Land, people, buildings - throughout its history city planning has been expressed mostly in these terms. Of the three, the land has given planning its most distinctive cast. At the core of any planning program are land use studies." [2]

As the cities grew, the complexity of their problems grew ahead of them. Cities, which once were clearly delineated from the country that surrounded them, became instead centers of vast urbanizing areas. Satellite cities, suburbs, exurbs - slurbs; the metropolis. An ever diminishing portion of effective city planning could consider only the area within the core city boundaries. City planning became urban planning to reflect a broader scope. Broadening further, interstate and intrastate regional planning has received the support of the Federal government as the need becomes apparent to consider urban problems within the context of their areas of influence. And state planning, once limited to providing an enabling statute for city planning at the local level, is beginning to show evidence of assuming responsibility for planning problems properly considered at that level.

Accompanying the wider spatial concepts in planning has been the realization that physical planning cannot be separate from the other subsystems of the metropolis. As Daniel Patrick Moynihan has said, "the trouble with urban problems is that everything is related to everything". In response to this understanding, urban planning has become an interdisciplinary science. Its techniques and perspective are derived from a variety of disciplines. Although the major early contributions came from engineering, architecture and the law, the disciplines of public administration, political science, economics, geography and sociology have added new information and understanding. The role and function of modern urban planners must take into account these new perspectives.

Land use planning must, therefore, be viewed as a component of governmental planning. It remains a key component, the target or visible manifestation of most of local and regional governmental planning, as enabled by the states. As a component of the government planning function, land use planning is primarily concerned with the physical location of buildings and/or facilities and the use of land in a geographical area. The land use planning process consists of a series of surveys, analyses and policy decisions which should begin at the general level and develop toward specific regulations regarding the construction, use and function of a building or structure on a specific piece of ground.

Land use planning can be considered as sequential, involving three major activities:

1. Development of general plans.

2. Adoption and administration of zoning regulations, subdivision regulations, and building and grading regulations, for implementation of the general plans.

3. A constant feed-back process in which existing regulations are regularly reviewed to insure that they continue to serve community objectives in the light of changing conditions, new knowledge or evolving perceptions of the public interest.

A. General/Comprehensive Plan

"The General Plan is the official statement of a municipal legislative body which sets forth its major policies concerning desirable future physical development" [3] according to T. J. Kent, Jr., a leading authority on the General Plan concept. (To avoid confusion, it should be noted the "General Plan", "Comprehensive Plan" and "Master Plan" are used interchangeably and often in combination in the literature.)

The general plan is not a law. It is a long range plan, often for 20 or 30 years hence, and thus cannot be interpreted specifically. As an official document it is supported by more detailed, shorter range planning documents which are law; the zoning ordinance, subdivision regulations, and grading and building codes.

The planning process which moves from the general policy decisions of the Comprehensive Plan to the specifics of ordinances and codes should be used to encompass hazard mitigation into any program of implementation. Stuart Chapin describes the process thus: "Urban land use policies are - a series of guides to consistent and rational public and private decisions in the use and development of urban land. They are maxims to guide land development in principle. We may conceive of the formulation of policies as proceeding from the general to the particular, with each level of policy-making supplying the foundation for subsequent, more detailed policy determinations". [4]

Communities are products of human efforts and, as a result are no better or worse than the sequences of decisions that shaped them. If planning is "the process of making rational decisions about future actions directed toward the attainment of community goals", the planning process must rest upon research. That is, data and insight gained by analysis must be used in the development of general policy judgments.

It must be understood that plans are not made in a normless vacuum and that they are not without normative impact. The problems of physical development and social welfare are related in very complex ways. Specifically, in the area of disaster mitigation, the protection of society from disasters (social welfare) is a legitimate goal for proper and effective physical development regulated through policies of the general plan and specific ordinances which support it. Therefore, a part of any comprehensive plan must be "an examination of all existing codes, ordinances and administrative policies dealing with land use, control and regulation" [5] to ensure that they effectively support the goals an/ objectives of the general policy decisions.

B. Land Use Regulations

Land use regulations such as zoning and subdivision control are the major instruments of implementation for the general policies of the Master Plan. The regulations involve legislative and administrative action at the state and local levels. The power of government to regulate land use on behalf of the public health, safety, and general welfare is considered a sovereign power of the state, which it may delegate to cities and counties. This power is guaranteed under the Tenth Amendment to the United States Constitution.

The power of the states to regulate in the best interests of its citizens is broad but not unlimited. Individual rights are protected by both the state and federal constitutions, including the right to hold and use private property against arbitrary, unjust or overzealous legislative action.

State constitutions authorize state legislatures to enact general laws. The legislatures commonly delegate a portion of this legislative power to local municipal units by enacting "enabling" statutes which authorize municipal legislative bodies to enact local regulations. State and local legislative bodies also establish administrative agencies to administer and implement legislative programs.

Administrative agencies usually have no inherent powers except for those delegated to them by enabling legislation. An attempt to exercise local power without express authorization and careful compliance with the procedures of the state law is generally considered invalid. Land use controls such as zoning, subdivision regulation, and building codes have been enacted primarily at the local level. Procedures specified in state enabling acts, authorizing adoption of these local regulations such as requirements for notice and hearing, prior planning, voting, publication, and other matters, are mandatory and must be carefully observed.

Public facilities, those administered by local, state or federal agencies, are not subject to land use regulation. The recommendation that they be made subject to these regulations is common but has been ignored so far, with a few specific exceptions.

Land Use Planning for Disaster Mitigation

Land use regulation to reduce the effects of natural hazard shares the same sources of authority and is subject to the same constraints as all other land use regulation. Its restrictions, placed upon private uses of land in disaster-prone areas, are developed through the legislative process, are established in the public interest, and are intended to allocate lands to their most appropriate uses. Most common among existing regulations for disaster prone areas are the restrictions placed on land use in flood hazard areas, often referred to as flood plain zoning.

Regulation of land uses in disaster-prone areas, like all other land use regulation, must meet four general criteria in order to be effective and valid:

1) Local regulations must be in accord with state statutes authorizing such regulations.

2) Regulations must serve valid police power objectives.

3) Regulations must demonstrably aid in the accomplishment of the stated land use and general plan objectives.

4) Regulations must not be discriminatory in content, in application, or enforcement.

These criteria must be observed in each phase of a valid regulatory program. A regulatory program of this type may generally involve:

1) Enactment by the state legislature of a statute authorizing local units and/or a state agency to plan and regulate private uses in disaster-prone areas. This may be done as part of a broader land use planning and regulatory effort or more specifically for some type of disaster common to the area.

2) Selection of protection policies by local agencies for management of disaster-prone areas, probably as part of a broader land use planning effort.

3) Drafting of local regulations required to implement the desired

protection policies.

4) Adoption of regulations.

5) Administration of regulations.

6) Enforcement.

In considering the range of regulatory objectives, it is important to note that zoning and subdivision controls are utilized to promote the general social, economic, and political well-being of a community through implementation of a plan for community growth. Each of the control sets is usually enacted to achieve a wide range of goals and those related to disaster would be "preventative" in nature.

Building codes, in contrast, are "protective" or "corrective" regulations and are used to regulate construction through controls over materials, design, and building size. They are not necessarily implemented in relation to the general plan.

A. Zoning

Zoning as a method of land use control has been used throughout the country by local units of government to guide private development of land in a manner consistent with the public health, safety and general welfare. It is somewhat surprising, however, that land use controls which have been in wide-spread use have rarely contained control provisions to guide land uses in hazard or disaster-prone areas. The literature, for example, has recommended for many years the use of zoning and other land use controls to compel adjustment of land uses to the flood hazard, but, of the total communities affected, few have established effective flood plain controls [6].

Zoning involves the division of a geographical governmental unit into districts. Within these districts, zoning typically regulates: (1) The dimensions of buildings and other structures; (2) The area of a lot which may be occupied and the size of required open spaces; (3) The density of population; (4) The use of buildings and land for trade, industry, residence or other purposes [7].

One major characteristic of zoning is that the regulations can differ from district to district in terms of the items regulated, as well as zoning definitions applied.

The division of a government unit into districts should be based upon the general land use plan which was developed to guide the growth of the community. Many state enabling acts require that zoning regulations be consistent with the general plan.

Typically, a zoning ordinance will consist of two parts:

(1) A written test which details the regulations applying to each district, including appropriate administrative provisions.

(2) A zoning map which specifies the boundaries of districts for various uses.

Thus, zoning can be used to regulate WHAT the land and buildings may be used for, WHERE specific uses may be conducted and, HOW uses are to be performed.

In this regard, zoning can be used to establish effective restrictions regarding land use in certain disaster-prone areas. For example, zoning can prohibit the building of certain structures across an earth fault or prohibit structures from being built in floodways.

B. Subdivision Regulations

Subdivision regulations are enforced through ordinances which prohibit the filing or recording of a plat showing a division of a tract of land into smaller parcels, usually building lots, without prior approval of a municipal planning agency. All states have delegated the power to regulate subdivision of land to at least a portion of their municipalities. Although the enabling laws vary widely, most have their common beginnings in the early subdivision acts drafted by Bettman and Bassett [8].

Subdivision controls appear to be nearly as useful as, or in some instances, even more useful than, zoning in regulating hazard areas. First, they may be used to discourage land speculation and "victimization" of buyers of lands by requiring that hazard information be shown on the plat or by prohibiting subdivision of certain unsuitable lands. Second, they appear to be more flexible than zoning in requiring substantial improvement of building sites. Third, the regulations, if properly authorized in the enabling legislation, might be used to require reservation or dedication as open spaces of certain high risk disaster-prone areas.

The provisions of subdivision regulations for disaster-prone areas may typically require (1) that the location of all known earth faults show on the plat; (2) that the location of various other geologic hazards be shown on the plat; (3) that floodways be shown on the plat; (4) prohibition of encroachment upon active faults or floodway areas; (5) the placement of streets and public utilities so as to minimize the effects of disasters.

A typical provision in a subdivision ordinance may require that no plat will be approved for development in areas subject to probable severe disasters unless reasonable provisions or improvements are made. In this regard, subdivision regulations commonly authorize the attachment of conditions to the approval of any plat. These conditions must reasonably promote the general welfare.

C. Building Codes

Building codes are regulations enacted under police powers to guide the design and construction of buildings or structures. The codes attempt to protect occupants, neighbors, and passers-by from structures which may jeopardize their health, safety or welfare.

The content and scope of building codes vary widely from state to state and municipality to municipality. In some areas, codes deal not only with the physical framework of the buildings, but also with plumbing, electricity, elevators, and other mechanical functions. In addition, codes can be concerned not only with new buildings, but also the repair and/or modification of existing structures. Standards for building occupancy may also be included in building codes.

Building codes generally set minimum standards for construction methods and materials. They do not regulate the location of developments, but are concerned with how development is carried out. Building codes are one of the most significant means to reduce the effects of natural disasters when they include performance specifications which: (1) restrict use of materials which are vulnerable to the type of disaster common to the area; (2) require structural design consistent with the expected ground shocks or flood velocities; (3) require consideration of the concept of risk as a key element in the selection of design criteria; (4) require the performance of special engineering or geologic studies of the site based upon the type of hazard exposure (e.g., seismic refraction surveys to determine fault locations).

Building codes must be reasonably based on the principles of general welfare. The codes must provide standards to guide administrators in issuing or refusing building permits. Ordinances may take either a detailed *specifications approach* or a *performance standards* approach. Rather than restict construction to narrowly defined methods and materials, performance standards define a performance objective and allow the builder flexibility in his choice of materials and methods to meet the objective. (Performance standards may also be found in many zoning ordinances.)

Building codes therefore appear to have considerable potential in reducing disaster losses. Courts appear to have been generally receptive to these regulations. However, building codes appear to have several limitations. First, they cannot be used alone to control land use, but must be used in conjunction with zoning or encroachment. Second, building codes which include disaster mitigation requirements will require more detailed information on the specific hazards to be addressed; information which is often lacking.

D. Land Use Ordinances and Building Codes Interaction

An understanding of the nature of the hazard problem is necessary to comprehend the social, technical, administrative, political, legal and economic issues in regulation of disaster-prone areas. Only with this complete understanding can regulations play an important and effective role in reducing disaster losses.

Within the general area of regulations most governmental units are faced with a serious problem of coordination, uniformity and changes in policy making, thus making the role of land use planning in disaster mitigation a difficult one to define. Some of the problems in this area are supported by a study by the University of California, Irvine, entitled "Building Housing Zoning Codes and Their Enforcement" which concluded that within the County of Orange, California and its incorporated cities there exists:

1. "A lack of uniformity in zoning codes and their enforcement, which leads to confusion, misunderstanding, and quite often costly operations among governmental agencies, builders and developers.

2. "A relatively uniform set of building codes, but a non-uniform method of code interpretation, which tends to reduce the value of code uniformity." [9]

Special provisions within the administrative framework of existing zoning, subdivision regulations, and building code programs should be made for the unique features of any hazard provisions in the form of more comprehensive and interrelated ordinances. Provision for seismologic, geologic and engineering expertise seems essential to determine earthquake hazards, locate faults, evaluate the effects of specific uses, and impose specific conditions for development. Coordination of the various government structures is vitally important in order to effectively use the existing agencies and link enabling legislation with authority for enforcement.

There does not now appear to be much planned interaction between the land use ordinances and building codes. Further, under municipal corporations law there seems to be a distinction between building codes and zoning. Specifically, a building code applies to the construction, maintenance and repair of buildings; whereas hazard zoning may restrict methods of construction and repair of buildings, but only in accordance with the community general or comprehensive plan pertaining to land use.

Further, building codes generally apply to the whole governmental area, and there is no distinction between districts as there is in the zoning ordinance. This is not to say, however, that building codes could not be developed for districts, but only to point out that, to the best of the authors' knowledge, no such regulations have been adopted.

It is in this area that the general state of the art needs to be upgraded. It is possible to improve our codes by applying already existing knowledge in a manner which all parties involved can comprehend. To make no provisions in the codes, because we cannot accurately predict a disastrous event, is to make *no* decision and to reject the logic of probabilities and the knowledge gained from science.

Codes cannot be perfect and accurate for *all* situations. Thus, they must: (1) reflect the concept of risk and uncertainty; (2) be dynamic in allowing for amendment resulting from new knowledge and improved understanding; (3) be rationally interrelated and tied to a plan which considers probable forms of natural disasters among its elements; (4) be based on a logic which the legislator, administrator, and citizen can fully comprehend; thus, allowing for effective participation in the decision-making process.

III. Planning Responsibilities in Disaster Mitigation

"Planning, it is said, leads to the formulation of plans; implementation is concerned with carrying them out. The intervening critical step is a decision." [10]

PLANNING LEADERSHIP IN GOVERNMENTAL ORGANIZATIONS

In any discussion of planning, the location of the decision making function within governmental structures becomes a matter of concern. It is easy to arrive at the conclusion that the most efficient way to plan -- both in arriving at decisions regarding final and acceptable plans and the techniques for implementation -- is within a governmental structure in which decision-making is highly centralized. Such an arrangement makes it fairly certain that any plans developed will be accepted and will be implemented as developed. However, in a system subject to the changing views of a single leader, or in the event of a complete change of leadership, the understanding of the plans developed and the continuity of implementation will probably be totally interrupted. Thus, a long-range planning process will be difficult if not impossible to develop.

In almost total contrast to the potential effectiveness of planning within a centralized leadership system, is the extreme difficulty of being able to develop acceptable plans in a governmental structure where there is extremely diffused leadership. This may occur in many situations as a result of the current phenomenon of citizen or community participation.

Since planning activities must often respond quickly and efficiently to issues and problems that arise unexpectedly, the diffused leadership or decision making mechanism appears not to be as effective a method of dealing with hazard or disaster situations as might be desired.

Recognizing the problems which exist between the extremely centralized planning function and that which is decentralized and diffused, it becomes necessary to look for a focal point for planning in a democratic governmental organization. The location of that function within local government has occasioned a debate within the planning profession that is almost as old as the profession itself. The earliest concept of city planning in this country held that it should be an independent activity within government, apart and aloof from the bias and influence of politics. The city planner served the selected citizens of the planning commission as a "technical" expert. The planning commission was in turn, advisory to the city council. The resulting ineffectiveness of the planning function under these conditions was inevitable, and was brought to general notice in a classic text, *The Planning Function in Urban Government,* written by Robert A. Walker [11].

Since that time it has been argued that the professional planner and his staff should serve directly under the chief executive, the position Walker favored. Others, most notably T. J. Kent, Jr.; believe that the "city planning process should be designed to involve directly and continuously the city council and the city planning commission, and not just the city planning staff and the chief executive." [12]

The best location for the planning function in a democratic governmental organization would seem to be that which is directly responsible to the chief executive function. Since, in an organization of this type, the executive is ostensibly responsible and responsive to the legislative structure, the planning function thereby becomes directly related to that legislative activity. It may sound trite to repeat the phrase that one should not have responsibility without authority, but an executive operation cannot make effective use of a planning function unless that executive also has the authority to implement the plans which have been made. This must take place within the overall *budgetary* and *policy* direction of the legislative organization.

It is often said that the executive is, in fact, the responsible planning function within a governmental structure. In any organization of any size, however, the executive will need professional and technical support in order to fulfill that planning function. If that function is too decentralized from the executive activity, it may suffer and lose its effectiveness. *Experience at all levels of government has shown often with tragic results, that if the general planning function is placed into sub-departmental status under functional line departmental activities, there will never be a comprehensive nature to that planning activity.*

Range of Planning Functions in Government

A. General Physical Planning

Planning functions can range very widely. Some are so limited that most professionals would not even call them planning. They are simply a response to the pressures that are placed on governmental agencies, (e.g., responding to development pressures with subdivision regulation and zoning ordinance variances, conditional use and appeals).

B. General and Functional Physical Planning and Planning Administration

From these specific regulations in ascending order of importance, the next level of the planning function is the development of general planning for physical development within the appropriate geographic area. This includes the preparation of reasonably long-term projections and plans for the area covered as well as the necessary implementation and administrative mechanisms.

At this level, a large number of planning activities remain within the operational departments of each government organization. The degree of sophistication and competence of planning in these departments in relation to the central planning function will depend on the authority of the governmental unit considered and its understanding of the general planning process. Also important is a willingness to subordinate objectives to the higher level goals and objectives of general planning.

C. Comprehensive-General Planning

Next is the beginning of a comprehensive planning function, which would pull together in one agency the various activities in the next level of sophistication beyond the physical development process. These will deal with the physical environment, the social environment, and the economic structure of the community being planned.

D. Comprehensive-General and Functional Planning

This comprehensive level can be further structured and refined when all of the various aspects of comprehensive planning are brought to bear, not only at the comprehensive level, but also at the functional planning level. This may be best effected through a restructuring of the organization into one comprehensive planning organization.

E. Total Planning - Including Budget Preparation

American governmental opinion may be ready for a rationalization of planning organizations which would pull together all the various comprehensive and also functional activities. As Chapin has said, "there has been a growing awareness of the importance of integrating planning more directly with policy formulation in the political process." [13]

Such a comprehensive planning organization could plan budgets in a form appropriate for adoption by the appropriate legislative body. Specific items can be designed and detailed to the point where the legislative body can evaluate the proposals in light of the policy implications, and finally approve the general policy and the budget for implementation.

All activites which take place prior to budgeting would be handled by the comprehensive planning organization whereas all following activities would be handled by various levels of the organization concerned primarily with implementation. This means that the planning agency develops the general plan to be followed by more specific sub-area plans. Within and subservient to these, would be developed specific locational plans for parks, schools, sewage facilities, libraries and circulation facilities.

Disaster Mitigation Within the Range of Planning Functions

Reviewing this range of planning functions, it would appear that land use planning for disaster mitigation would be easiest to apply at the lower levels. For example, planning for hazards could occur first as an element of the Comprehensive-General Plan (as is now required in the case of earthquakes by the State of California). It would result in actions in other departments such as Building and Safety, thus, implementation becoming dependent on the individual agency evaluations of the need and feasibility for such planning. Planning for natural hazards at the lowest level, would then be administered through regulations appearing in codes and ordinances.

Such an approach to inclusion of hazard considerations within land use planning could result in fractionated, uneven and ineffective administration of whatever programs emerge. It should be apparent from previous discussion that whatever policies are used to address the problems of hazards by government, they must have been considered and agreed upon at the highest, most comprehensive level of planning and policy formulation. Only at the highest level can concepts such as risk be viewed within a framework capable of integrating planning for hazards with the social, economic and physical components of the community. Without this integration, a rational approach to the decisions needed to formulate politically feasible hazard mitigation standards cannot be made.

At the highest level, however, an evaluative mechanism must be established as part of the over-all planning process to enable a balanced analysis of various alternatives, i.e., location, risk, cost and timing. On that basis, recommendations can be prepared leading to adoption of standards and allocation of responsibilities required for incorporation of hazard mitigation planning into land use planning. This will enable the legislative body, in light of the comprehensive and balanced presentation made, to rationally determine the allocation of various resources and costs and to assign responsibilities for implementation of the approved policies and programs to the appropriate agencies.

Such a planning approach would make it possible for the executive or the legislative body to analyze the multitude of options, within the acceptable levels of risk, the acceptable levels of costs and restrictions, and the generally limited resources available. It would also permit the balancing of these elements within a policy framework recognizing political implications. At that point, the various political implications could be placed within a context of a rational evaluation mechanism. If those recommendations were to be overruled, it could be done on the basis of some rational and factual information, rather than on the basis of intuition or emotion. Also, a rational risk mechanism could be built into the evaluation of the various alternatives, so that legislative or executive decision-makers would have a rational and comprehensible mechanism within which to make and to support their risk decisions.

There is, however, a risk in this system which should be pointed up. That is, due to the difficulty in comprehending the various ramifications and interrelationships of the total process, the legislative or executive function may abdicate its decision making authority to the various professional administrators operating at lower levels of responsibility. The result of this situation is the bypassing of the political process; allowing the opportunity for the professional administrators to make political decisions.

An effective legislative or executive function can, and must, take the responsibility to make comprehensive decisions which relate various costs, resources, needs and constraints to each other. This can be done providing a comprehensible and comprehensive plan is used as the basis for judgments.

Delegation of Planning Function to Governmental Levels

Definable levels of government can be identified as follows: international, national, inter-state region, state, intra-state region, county, city, district and neighborhood.

It is obvious that planning takes place in varying degrees at all levels within governmental structures. It is equally obvious that almost everyone would agree that different governmental levels have varying and, hopefully, supplementary functions. Most analyses of regional decision making have run into difficulty when specific assignments had to be made as to the responsibilities of specific agencies. Because of the complication of these intergovernmental relationships, a set of criteria has been developed to aid in determining the responsible planning level [14]. A set of ten planning-related actions were specified which, when implemented as a system, make up the total process. These ten actions are:

1. Inventory - survey of existing conditions, e.g., physical, social, and economic.

2. Analysis - review of implications and interpretations, based on the data developed in 1.

3. Policy setting - formulation of policy priorities, strategies, specific areas of concern, etc.

4. Plan making - applying the results coming from 1, 2 and 3 - to the specific locational or functional area of study.

5. Standard setting - devising specific criteria, levels of conformity and physical standards.

6. Acquisition - where necessary, make provisions for funds to accomplish ends, noting the sources of the funds.

7. Regulation - application of public methods, techniques, ordinances and laws, to achieve policy.

8. Development - improvement or construction of facility.

9. Monitoring - ensuring that standards are maintained or modified as experience indicates.

10. Coordination - ensuring that all elements of the programs are pursued in an organized and unified manner and at the appropriate level by various agencies.

The following is a series of criteria by which each of the above actions can be measured in order to find the appropriate governmental level to which responsibility should be assigned.

1. Responsibilities for *inventory* must be undertaken by the levels of government that will: be the units that will utilize the data; have within their jurisdiction the information to be surveyed or inventoried and have data accessible or available to it; have familiarity with the data to be collected; have the facilities and manpower to collect the data, and collect data efficiently.

2. The levels of government that undertake responsibilities for *analysis* must: be technically equipped to handle data (including manpower and machinery, and the possibility of computerization, data-banking, long-term storage, and retrieval); serve as a clearinghouse for all similar data; have expertise and competence in dealing with such data; understand all aspects of the problem so that analysis reflects both explicit and implicit needs; provide the greatest efficiency in conducting the analysis.

3. The levels of government that undertake the responsibility for *policy-setting* must: have the clearest perspective on the issue; have some potential for implementing policies; have access to basic data and analyses; be accessible to the public; be free of conflicts of interest; be sufficiently flexible to alter policies quickly, as needed; and be able to relate proposed policy and its long-term impact.

4. *Plan-making* must be the responsibility of the governmental level that is: able to define the plan to make it clearly correspond to basic policies but remain workable; has a technically competent staff to develop the plan within a reasonable time; has familiarity with the subject of the plan being developed; and has access to basic data collected.

5. The *setting of standards* must be undertaken by the level of government that will: include inputs from units affected, or in some way, impacted by the standards being developed; represent no special interest and be free of conflicts-of-interest; have access to the

specialized skills and equipment necessary for detailed standard-setting, and have flexibility to respond to new information and data to change standards rapidly to reflect that information.

6. The levels of government that undertake the responsibility for *acquisition* must; be the one having the financial capability and revenue base; be able to achieve economies of scale in the funding of the activity; have the legal authority or options to finance such activities (e.g., assessment districts); be responding to its own policies in regard to the population being served.

7. The government level responsible for *regulation* must: be the one that is able to deal with the problem most directly; geographically covers the entire areas for which regulation is necessary; be able to provide optimum coverage of the issue (not geographic, but for the full range of issues involved); have the competence to draw up documents and ordinances to best and appropriately describe the action and activities desired; be directly related to the issue of concern; be large enough to ensure efficient enforcement of the regulation; and be accessible and controlled by the service population.

8. The governmental level that undertakes the responsibility for *development* must: be geographically and technically familiar with the project; be one that ultimately has a role in the ongoing operations functions of the development; have some element of the project occur within its jurisdiction; be able to ensure efficiency and effectiveness in development; and be able to alter or affect existing regulations or sub-policies to satisfy or accommodate requirements for development.

9. *Monitoring* must be undertaken by that level which has the authority to deny or cease activities that do not conform with standards; be able to maintain monitoring on an ongoing basis; be able to provide optimum area coverage (efficiency of time, material, geography); include monitoring as part of another agency so that monitoring is aligned with policy and not just standards; and have the manpower and expertise to conduct the monitoring operations.

10. Finally, the levels of government that undertake the responsibility for *coordination* must: have resources and competence to allow for coordination, and be capable of ensuring coordination by other levels of government.

Based upon these criteria a "functional-responsibility matrix" can be developed which would identify the various levels of governmental responsibility on one axis and the activities or responsibilities which must be assumed on the other. This would then permit the rational matching of the responsibility and function with the most appropriate governmental agency or level in practice. This is most difficult, however, due to the fact that responsibility is very diffused between governmental levels.

Regional Government and Planning for Disaster Mitigation

Land use planning has, by tradition, been almost totally a matter of local concern. There is ample evidence gathering, however, that higher levels of government will be increasingly involved in these matters.

President Nixon has cited the need for national land use planning policies [15] and several bills are pending related to this need, but no specific action has yet resulted.

State level assumption of land use planning responsibility and authority is, also, increasingly common. "State governments see themselves as strategically situated to plan and manage the environment, particularly in the matters of land use and waste management. Local governments are too close to the economic and political pressures that create environmental problems to be sufficiently detached", [16] is a common viewpoint in the current trend. Land use planning related to natural hazards can be seen as a sector of environmental management and thus, subject to the same considerations for governmental controls.

In the *Quiet Revolution in Land Use Control*, Bosselman and Callies describe a number of planning programs in which states as well as interstate and intrastate regional commissions have successfully addressed environmental problems which could not have been encompassed by local jurisdictions. "The innovations wrought by the 'quiet revolution' are not, by and large, the results of battles between local governments and states from which the states eventually emerge victorious", [17] but are instead a cooperative venture in intergovernmental coordination.

This is the large view. Examined more closely, local governments, like any other organization, do not welcome incursions into their realms of authority or a continuing erosion of their powers.

Local planning agencies can often show that the reason they have not been effective in solving environmental problems is that the state has not granted them the necessary authority. Local land use plans often note natural hazards, but the implementation powers necessary to correlate land use regulation to hazard are lacking so no such provisions can appear. For instance, the Los Angeles City Planning Departments' preliminary General Plan for the Santa Monica Mountain Area, written in 1962, shows a full treatment of earthquake fault lines, geologically unstable areas, fire prone areas, steep slopes, and other hazards to development. Yet, in spite of these admonitions, pressures for permitting greater residential densities and more intensive land uses have been unremitting.

California's approach to new environmental planning concerns, which will probably include land use planning related to natural hazards, has been two-pronged. One trend has been toward planning and management at the regional scale. The Water Quality Control Boards and the San Francisco Bay Conservation and Development Commission are two examples often cited as outstanding successes. Although as will be shown, Councils of Government, such as ABAG (Association of Bay Area Governments) more often serve as a reminder that the simple act of forming a regional planning group does not insure efficient or effective problem solution.

We believe that planning, as a means of mitigating the effects of a disastrous event should be undertaken on a regional basis.

However, the history of regional concerns, especially as related to planning, has been beset by many difficulties. There is no question that many people are concerned about the philosophical implications of the development of regional approaches to solutions. If, however, governmental agencies had been able to develop satisfactory regional coordinative systems, these concerns may have been overcome. In the development of regional programs and activities, professionals have looked for the perfect and all-encompassing solution and, not being able to find that perfect solution, have settled on extremely limited single purpose implementation mechanisms. As an example of the confusion which results, consider the following:

> "Who makes the environmental development decisions in the San Francisco Bay Area?
>
> "The pattern of government there includes one state government. There are, in addition, nine counties, 110 city

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governments and 604 special districts. Of the latter, 294 are governed by elected boards or commissions, while 310 are governed by one or another of the county boards of supervisors. In addition, there are 213 elected school boards, thirteen appointed local housing authorities and sixteen appointed redevelopment or renewal agencies. This is a total of 966 decision-making bodies.

"That is not the whole story, however. The legislature apparently has felt that the people of the Bay Area cannot completely govern themselves this way so it has created, or permitted to be created, several other agencies: There is a state-created and appointed San Francisco Port Authority, a San Francisco-Oakland Toll Bridge Authority and, most recently created, a Bay Area Hospital Planning Committee. Also, the San Francisco Bay Conservation and Development Commission is charged with licensing permits to fill the shores of San Francisco Bay while, at the same time, it is required to prepare a comprehensive plan for the bay's development. There is more: In its wisdom, the 1965 legislature directed the state Water Quality Control Board to prepare a comprehensive plan for disposing of all liquid wastes in the entire drainage basin of the Central Valley and the San Francisco Bay.

"On top of this, there is the legislatively created Bay Area Transportation Study, which is to plan comprehensively, in accordance with the Highway Act of 1962, for highways (not really transportation) in the Bay Area.

"Sitting above and aloft is the Association of Bay Area Governments (ABAG) organized under the Joint Exercise of Powers Act. Its charter gives it the responsibility for comprehensive planning for the area. Although ABAG has taken on the overall planning job, it is really a voluntary debating society from which any of its members may resign at any time. It acts, on the one hand, like the United Nations, a collection of sovereign states. On the other hand, when there is a need for more aid, these local governments run to the legislature to be bailed out.

"There are four super-regional special districts, that operate their own fiefdoms, each in its own way creating the Bay Area environment without reference to anyone or anything else. The East Bay Municipal Utilities District lays the water and sewer lines over most of the eastern part of the bay. It is governed by an elected board. And I will guarantee that 99 per cent of the electorate in the communities which it serves do not know the name of one member of that board.

"The Golden Gate Bridge and Highway District is a device created to build and operate one bridge: the Golden Gate. It is not part of the Toll Bridge Authority.

"The Bay Area Rapid Transit District is going to spend close to a billion dollars on transit. It is separate, too, and not related to highways or regional transportation planning.

"Finally, there is the Regional Air Pollution Control District. This group, made up of locally elected municipal and county officials, has done a fine job, which gives one hope that broader aspects of the police power can be exercised on a regional basis through this same kind of politically representative and responsible body.

"All in all, it is estimated that there are over 4,500 officials, 3,500 of them elected, who make the decisions that affect the development of the environment in the San Francisco Bay Area." [18]

A region should be dealing with such issues as general planning, transportation, air and water pollution, air and waste generally, the supply and dissipation of both, disaster mitigation, and many other non-physical concerns such as health services, law enforcement services, etc. In the past, as we have seen, it has been the practice to establish specific regional combinations of cities and counties for each of the particular functional needs. In only a very few situations have the boundaries of various regional agencies been coterminus. Because of the variety of these special designations, it has been difficult to interrelate the various planning activities being undertaken. It is therefore, almost impossible to establish any kind of a comprehensive planning framework within which all these various activities are undertaken.

An additional major difficulty concerning a rational alignment of regional activities lies in the fact that local governments in the undertaking of various functional activities are depending very heavily on varying and often very confusing funding sources from both the federal and state levels. These funding agencies also require various regional review bodies, each with varying government bodies totally unrelated to each other.

It is strongly recommended that a general and probably national policy be adopted and that a set of regional alignments be established. These should be applicable for all activities which require a regional concern. Even though these regional alignments may not be perfectly suited to any of the specific functional requirements, they should be designed in such a manner that they will meet a substantial number of the most common overlapping concerns. All state and all federal agencies with their funding or other assistance programs should be required to work through the same regional alignments, and thus the current confusion, contradiction and competition - all of which tends to produce nothing but a questionable use of public resources - may be avoided.

Such a solution will not satisfy the administrative perfectionist, but should allow for a more effective management of public resources.

With specific reference to disaster mitigation, a public agency should be able to respond immediately, and should not be forced to screen a handbook of resource organizations to find the appropriate regional, state or federal agencies to be addressed for immediate assistance.

Considering the possibilities of this approach to environmental management, (and particularly land use planning related to natural hazards) a more effective and efficient model for governmental administration must be formed. Broad statutes enabling implementation powers for land use standards related to natural hazards must be provided by the states. Objectives for regional and local planning related to hazards would be defined in the parameters of authority set forth. Regional commissions, responsible to the state, would analyze hazard impacts and degrees of local area vulnerability to specific natural phenomena. Data, guidance, and technical assistance would be offered to local agencies in formulating and implementing General Plan elements by the regional governments. Local plans, codes and policies would be reviewed by regional agencies and minimal standards enforced, beyond which local option would prevail. Coordination of local policies would be achieved at the regional level, and regional coordination at the state level. Appeals from local action would follow the same hierarchy. Data, policies and programs would be scaled to the area needs of the three levels.

True comprehensiveness could only be achieved in this model if the regions were held constant, that is, occupied the same areas for all functions rather than being separately delineated according to the problems addressed. If this were true, land use planning related to hazards could be rationally coordinated with all land use planning policies and related by consistent programs of social and economic concern.

This is the bare bones of a rational model. Political feasibility, however, is rarely measured in terms of rationality. The resistance of local governments to further administrative constraints, the resistance of the public to acceptance of the costs of a third level of government, the resistance of private enterprise to another level of police, permit, and license authority, are but a few of the obstacles in the path to the rational model.

IV. Natural Disasters That Can Be Mitigated Through Land Use Planning

Earthquake

Seismic safety through land use regulation implies a consideration of the various hazards associated with tectonic earthquakes in terms of density and location of new structures and facilities, the rationale and criteria utilized in site and structural design, an evaluation of existing structures and facilities, and a review of the efficacy of existing codes and ordinances. These criteria must be based upon a consideration of the various modes of failure that can occur during an earthquake in both open and developed areas.

A. Ground Shaking

The majority of damage to man made works from earthquake is caused by earth vibration as differentiated from localized damage due to permanent ground movements, water forces and flooding. However, there have been cases of significant losses as a result of the latter which will be discussed later.

Structures can be designed to survive severe earth vibration with little damage but it is not usually considered economically feasible to design structures to withstand permanent ground movements or tsunamis. Damage due to earth vibration is a function of the overall earthquake resistance of the structure and the intensity and duration of ground vibration.

In a severe and prolonged earthquake, the area of intense ground shaking (MM IX and above) will likely cover an area in excess of 1000 square miles. Within this area, there will be severe structural damage to modern buildings. Variations in damage within this same area can be traced to differences in intensity of surface vibrations as affected by soil conditions, epicentral distance, azimuth from source to site as well as direction and extent of faulting (energy release). Also the soil structure interaction will cause differences in response of various types of structures. The variations must be provided for in building codes and zoning regulations. Codes and regulations, can be operationally improved by zoning risk maps which (a) show the earthquake response characteristics of particular sites within mapped area, and (b) require additional in-depth investigations of sites located in mapped areas, and/or vulnerability of existing structures located on the sites.

The severity of shaking that can be *reasonably expected within the* *lifetime of a structure* is a function of the seismicity of the area on which it is located as well as the areas relationship to major fault locations and directional trends. In Southern California, for example, the equivalent statistical risks for shaking (excluding site conditions effects, based on historical data) vary by as much as 50 percent within a distance of forty miles. The risk from this parameter can also be treated by zoning regulations which are interfaced with building codes.

On a regional basis, the Uniform Building Code (the principal U. S. Code that treats earthquake) utilizes an earthquake risk map which shows large zones of equal seismic probability. The code forces for aseismic design vary for the different zones as follows:

Zone	Portion of Force
0	None
1	1/4
2	1/2
3	1

Present U. S. Codes do not provide for differences in soil-structure interaction. In order to adopt this factor, it would be necessary to have much more information including the dynamic response characteristics of soils and seismic velocity profiles of adjacent areas. Presumably, if this information were available, risk maps could be made to guide local agencies in the foundation of codes which would include soil-structure interaction as a function of seismic safety for structures.

A code which recognized these variables would be complex and very few local building departments, even in Southern California, much less in the midwestern and Eastern parts of the U.S., are currently manned to administer such a code.

All facilities need <u>not</u> be designed to the same level of safety. Important and critical structures such as those housing medical, emergency and utility services should be required to have a higher level of safety, or accept a lower level of risk, than ordinary commercial and industrial building. Thus, different interpretations of risks maps should be provided in the codes for the design and location of these structures.

B. Permanent Ground Movements

Earthquake damage to man made structures caused by permanent ground movements (landslides, lurching, liquefaction and fault break) can be effectively treated by land use planning. In order to accomplish this, it is necessary to define and locate potentially hazardous areas, *rate them as to risk relative to ground shaking*, and then reduce the additional hazard of permanent ground movements by proper engineering. Prohibition of certain types of construction based on preassigned risk allowances, may be necessary in some areas, subject to some of the hazards considered below where engineering is unfeasible.

B.1 Landslides and Lurching

Scientifically prepared and properly administered grading codes and land use regulations have essentially prevented landslides in the City of Los Angeles. There are no known landslides on sites prepared after 1963 when the current grading code was adopted in the City of Los Angeles. Published data [19] clearly indicates that scientifically prepared and properly administered grading codes are effective in preventing loss of life and property. It is presumed that these or upgraded codes will mitigate the effects of earthquake caused landslides.

Chapter 70 of the Uniform Building Code is a minimal code which, if properly administered, can be extremely effective. It requires that engineering measures be applied in certain areas to control landslides and settlement. In addition, code provisions cover fill compaction and foundation design requirements which are intended to minimize damage due to settlement.

However, even with these regulations, there were many instances of settlement and landslide damage in the San Fernando earthquake of February 9, 1971. Fills settled along freeways and roads, landslides and settlement occurred in the vicinity of the Jensen Filtration plant, Sylmar Converter Station and L. A. County Juvenile Facility [20].

It must be borne in mind that codes alone are of little value, unless there are adequately educated and trained personnel available to staff the enforcing agencies. This staff must include qualified and possibly licensed personnel in the following disciplines: (1) Architecture or Land Planning, (2) Engineering Geology, (3) Soil Engineering, and (4) Civil Engineering. Above all, quality control is an essential ingredient.

B.2 Liquefaction

In a major earthquake which affects coastal areas, there are usually severe problems associated with saturated natural and man made fills. A phenomenon known as liquefaction occurs where the earth vibration causes saturated silty-sandy soils to subside rapidly and flow laterally.

Land use control measures for this failure mode must (a) limit the use of the subject area for the construction of certain critical facilities, (b) propose better engineering design practices for structures and soils and/or, (c) accept certain risks for special facilities (like harbor facilities) which could be damaged severely, but which also must function daily for the economic survival of an area.

B.3 Faulting

Damage as a result of surface fault rupture has been common in California earthquakes. When fault ruptures extend to the surface and through man made structures, severe damage usually results. It is generally not considered economically feasible to design structures to resist these movements. Therefore, potentially dangerous faults should be located and avoided, if the risk level warrants such action.

It is doubtful that the present "state of the art" will enable us to *accurately* locate all of the potentially dangerous areas, except on a very broad basis. For instance, the San Fernando-Sylmar area in 1971, the White Wolf fault in 1952, or the 1940 El Centro fault were not recognized as potentially dangerous areas from the standpoint of surface fault rupture. It is possible to map the more well-known faults such as the San Andreas, San Jacinto, Hayward and other fault systems. However, the task of mapping potentially hazardous surface areas for the majority of other faults throughout the country would be tremendous.

The problem of mapping is complicated by the fact that some active faults such as the Newport-Inglewood are covered by several thousand feet of alluvium and even if a surface break occurred, due to movement on this fault, it would be very difficult to predict where it would intersect the surface. Then the only way to positively eliminate the hazard would be to prohibit development in a wide area on the surface over the fault zone. Such a plan would probably not be economically sound. Perhaps it would be better to accept the risk in cases like this provided a quantitative risk of *Fault Activity Relative to Ground Shaking Risk* can reasonably be identified.

Fault breakage is critical in the case of public utility systems, such as water, gas, and electricity. In this area, systems planning with respect to potential surface fault breakage is essential. Where these "City Lifelines" cross active faults there must be provisions for prevention or rapid repair of damage.

Public utility systems must be planned so that, if portions are damaged at active fault crossings, there will be alternative systems and storage resources available for immediate restoration of service. There has been some limited activity in systems planning for catastrophe but there is a real need for a more significant effort in contingency planning in this area.

The problem of land use planning in relation to active fault locations in an area such as California is very difficult. There are so many potentially active faults that it is impossible to prohibit construction in these areas. Appropriate fault activity risk, tied with planning and design restrictions, must be accepted or tolerated. In other areas of the U. S., fault mapping is virtually nonexistent. Unless this mapping is accomplished, land use planning relative to active faults would be meaningless.

C. Water Forces and Flooding

We can see how various modes of failure are interrelated (e.g., a landslide can trigger a flood by causing a dam to overflow a breech).

C.1 Tsunami

A tsunami is a rare type of sea wave that is produced by (a) large submarine, near or distant, earthquakes, (b) earthquakes whose epicenters are near a coastal area, (c) large submarine landslides, or (d) volcanic eruption.

The first recorded Tsunami dates back to 1400 BC, when it destroyed the town of Amnisos, Crete. Subsequently, the Greek town of Helice perished under the effects of a tsunami. More recent examples of other damage due to tsunamis are:

> April 1, 1946: Hawaiian Islands - damage on the five main islands; listed as Hawaii's worst natural disaster with 159 dead and an estimated loss of $25,000,000.
>
> May 22, 1960: Chile - a violent earthquake in Chile propagated tsunami waves that traveled across the Pacific Ocean killing people as far away as Japan. The loss in Chile alone, was 4,000 dead and an estimated loss of $400,000,000.
>
> March 28, 1964: Alaska - "Good Friday" earthquake generated tsunami waves that destroyed much of Valdez, Alaska and extended death and destruction as far away as Crescent City, California.

In 1948, two years after the 1946 Hawaiian tsunami loss, the "Seismic Sea-wave Warning System (SSWWS) was initiated by the U.S. Coast and Geodetic Survey. The first major tsunami after initiation was in 1952, with a giant tsunami generated near Kamchatka, Alaska. The waves caused $800,000 of damage, but no loss of life. Since then, the system has been expanded to most of the Pacific nations. The Chilean earthquake-induced tsunami in 1960 was the most destructive since the initiation of the SSWWS system. The wave left thousands dead in Chile. Hundreds and possibly thousands of lives were saved by the SSWWS system in other areas.

Another precaution can be found in the telephone directory for the Island of Hawaii. Five pages are devoted to explaining the tsunami warning system (SSWWS), showing graphically the areas that may be inundated and must be evacuated. Many of the newer buildings are designed to be tsunami resistant. For example, one hotel in Hilo and one on Oahu are designed with essentially breakaway walls at ground floor level in combination with tsunami resistant columns to allow the waves to pass through the lower floors. Based on present knowledge, it should be possible to identify and map potentially hazardous areas considering the threat of tsunami. These hazardous areas would undoubtedly include most of the existing heavily industrialized and commercialized waterfront areas. The imposition of building restrictions on existing structures would be expensive and difficult. The tsunami hazard must however be considered for new construction and for very important structures such as nuclear and fossil fuel generating stations.

C.2 Flood Due to Dam Failures

There has never been a failure in the U.S. of a modern compacted earth fill or concrete dam due to earthquake. Those earth dams which have suffered damage, were the older dams constructed with hydraulically placed fill. One obvious way to eliminate the hazard of earthquake induced dam failure would therefore be to identify, remove or strengthen all old hazardous dams. Such a program is underway in California.

Land use planning would be effective in reducing the hazard by evacuating, or prohibiting building in areas below hazardous dams. However, it would seem that the preferable solution would be to remove or correct the hazardous dams.

Wind

The hazard of damage and loss of life due to strong winds has some parallels to the earthquake hazard. Both concern lateral and uplift loadings for which structures are not normally designed. Experience has shown that structures which are especially designed to be earthquake resistant are also highly wind resistant. Both hazards have been evaluated on a regional basis by providing "risk" or loading maps in building codes.

The Uniform Building Code provides a map of the U.S. which shows resistant wind pressures and locations of localized winds such as Santa Ana, Chinook, Columbia River Gorge and Wasatch Mountains. The choice of code wind loadings is largely left up to local jurisdictions. Frequently, local choices do not agree with wind loading maps, nor do they recognize localized wind conditions.

A. Hurricanes

One of the major wind hazards is the tropical hurricane which is generated in the low latitudes (8°-15°) and often travels northerly into the southern states (some from the Pacific Ocean may extend infrequently into Southern California). The average annual loss from hurricanes is estimated to be $440,000,000.

B. Regional Winds

Regional wind conditions are created by cold, strong, regional winds passing over mountain ranges, descending on the lee side, and encountering a warm low pressure area with subsequent high velocity winds. These winds are often referred to as Chinook, or Santa Ana winds. They may reach velocities up to 90 miles per hour and can create havoc. They must be considered in land use planning.

The Santa Ana wind, typical to Southern California, is very strong at the mouths of certain major canyons. Improperly placed and/or designed structures in these areas can be subject to wind damage. Planned low intensity use or such areas, placement of wind breaks or proper structural and site design can compensate for this natural hazard.

The Chinook winds of the regions of Colorado and Wyoming are similar, but often of broader scale and definitely should affect land use planning. This is particularly true when considering the location of schools, air fields, highway overpasses and recreational lakes.

C. Tornado

Tornado winds are of extremely high velocity and low pressures. It has not been considered feasible to design structures to survive these winds. Damage due to tornadoes is severe over a relatively small area and general areas of occurrence can be predicted, but specific spots where they will strike cannot be predicted.

D. Storm Surge

Damage related to storm surge may occur wherever strong winds can affect large expanses of water. This includes portions of the eastern seaboard which are subject to hurricane winds or strong surge activity related to storms in the North Atlantic, the Gulf Coast which is battered by the greatest number of hurricanes, the West Coast which is subject to local strong wind and surf action, and the Northern California-Oregon coastal area which receives the full force of the prevailing westerly winds and their wind-generated waves and surf.

Examples can be listed where valuable property has been severely damaged by storm surges. In recent years the Ventura County Coastline of Southern California has undergone serious erosion and loss of valuable coastal lands until groins (sea walls perpendicular to the coastline) were constructed to retard the rate of erosion. The loss of sand from surge action along a few miles of Ventura Beach averaged 90,000 cubic yards prior to the remedial measures, for example. In addition to the loss of sand, there was an accompanying loss of structures and an impairment of highway facilities.

It should also be pointed out that in our complex society and environment, there are other factors contributing to storm surge erosion. These other factors are:

a) Drought, which reduces the production of sediment through erosion and the accompanying transporting of sand from the hinterland to the beach.

b) Construction of flood control facilities which tend to reduce erosion and transport power of the streams and allow settlement of sediment in reservoirs.

There is a need for awareness of the effects of wave erosion to beaches and wave-cut cliffs, especially in areas where population expansion has increased both land use and land values. New concepts are currently being planned in these coastal zones subject to strong storm action. These include: offshore roadways or causeways; offshore oil drilling platforms; offshore nuclear power plant sites and possibly floating sites; below sea-level aqueducts and other similar facilities.

With regard to land use planning, one recommendation of ASCE states, "In special locations such as on promontaries, on mountains, in gorges or for other unusual reasons, where records or experience indicate that the basic velocities are inadequate, higher basic velocities may be used at the discretion of the engineer." [21]

Wind forces due to hurricanes are generally accounted for in the wind pressure maps provided by ASCE. However, there is a need for more modern wind codes which recognize the necessary variables such as height, shape factors, gust factors, uplift forces, negative pressures and variation of forces on different building components. The hazard of hurricanes must be handled by engineering and building codes rather than zoning.

The existence of local high velocity winds is well known. However, the hazard of these winds is not generally recognized by incorporation of increased wind forces in local codes. For example, Santa Ana wind velocities in excess of 100 miles per hour are measured *every year* near the Fontana-Riverside area in California. However, building codes in these areas do not require increased wind design forces.

Land use planning could be effective in reducing the hazard caused by *local high winds* by mapping these areas and requiring higher design forces in the high risk areas by restricting the types of structures which can be erected.

Volcanic Hazards

The island of Hawaii, which is the youngest of the Hawaiian Islands, has the only active volcanoes, although Mount Rainier and a few other areas in the United States may become active. The Hawaiian islands are built over a 1600 mile fissure or fault zone that is directly related to sea floor spreading, faulting, and volcanic activity. Urban development(subdivisions) is taking place currently on the Puna Coast some 20 miles Southeast of Hilo, Hawaii. This coastal area is beautiful and possesses some of the most spectacular views that can be found on the Hawaiian Islands. However, major flows from the Puna Rift (or fault system), which is located on the westerly flank of the active volcano Kilauea, spread over these areas in 1955 and 1960. In fact, *the majority of these new developments are located on the 1955 flows.*

Technically, there is a very good possibility that the Puna Rift will be active again - probably in the very near future. To replant papaya, orchids, and other productive vegetation on a recent flow is one thing, but to place housing developments in such areas is less than prudent.

The island of Hawaii is a young island, still growing, and one which will witness much volcanic and earthquake activity in the future. The two active volcanoes, Kilauea and Mauna Loa, have been very active in recent and historical times and should remain so into the future. These portions of the island of Hawaii should be utilized for national parks, recreation, and agriculture and not for housing and/or commercial structures which will undoubtedly be destroyed in the near future.

Other Unusual Hazards

Other hazards can result from unexpected explosions due to industrial activities. Both airborne pressure waves and projectiles can result from this source of hazard.

On the other hand, quarry blasting, road blasting and other sources of blasting set up ground vibrations which are alleged to cause damage. Only in very rare instances have blasts of this nature actually caused any damage, however, the nuisance has caused much concern, especially in the court rooms.

Zoning could be accomplished around certain quarries or regulations could be developed for the use of fixed charge sizes (or a combination of both). However, zoning or regulations should be established primarily on the basis of nuisance limits, since damage from such sources is not generally expected. If regulations are developed based on nuisance limits, court ruling and public opinion surveys should be thoroughly documented and taken into consideration rather than technical arguments.

Earthquakes can be triggered by changing local earth pressures such as the filling of a large reservoir (example, Koyna Dam in India and the Konya earthquake of December 11, 1967). Removal of fluids from an oil field or repressuring an oil field may also trigger small to moderate earthquakes. Examples of repressuring would be the quakes in the vicinity of the Rangely Field in Colorado and the Newport-Inglewood zone in California with some quakes in 1941, 1944, and 1949 being attributed to withdrawal. During the mid-1960's Denver, Colorado witnessed a series of earthquakes that have been attributed by many to be directly related to the placement of fluid wastes under pressure into relatively deep rock materials at the Rocky Mountain Arsenal near Denver.

These latter examples generally can not be considered in zoning, but should be regulated.

V. Current Land Use Planning Practices Applicable to Disaster Mitigation

As noted previously, there appears to have been little regard for natural hazards in land use planning ordinances and statutes. Certain trends are now evident, however, which may indicate imminent change in this field. Some of these trends will be reviewed here.

Environmental Land Use Planning: A Methodology for Disaster Mitigation Planning

The recent upsurge of interest in environmental-ecological problems has introduced a spate of federal and state legislation addressed to these concerns, accompanied by a growing number of landmark court decisions interpreting the new laws. Mainly concerned with protecting nature from man, environmental legislation is of interest here because of its converse application, protecting man from nature.

"Man has chosen some of the worst places to live with respect to his own life and health. He has selected flood plains, aquifier recharge areas, fault zones, unstable soils, and other equally poor environments." [22]

In order to correct man's errors, environmental planning has come into existence. Under the Environmental Quality Act of 1969, the U.S. Government requires that environmental impact statements be prepared for all projects requiring federal funding support. Recommendations for regulation of land use can be developed from these statements.

One common methodology of environmental land use planning is well-suited for use in hazard studies. Typically, a set of maps is developed from area data showing, individually, areas sensitive to important natural forces. Thus, for a given region, the maps may show areas of steep slope, areas of subsidence, locations of delicate and/or irreplaceable flora or fauna, or whatever physical systems are matters of concern to the planners. Overlaid, the maps will show a priority of environmental protection needs within the mapped region. For consideration of specific locations, such as development sites, matrices are developed from the maps and the data, and used to evaluate the site in relation to its scores on all the environmental factors to be considered.

This same approach can be used to consider matters of natural hazards to man. In land use planning related to earthquakes, for instance, known earthquake faults, geological conditions which exacerbate earthquake damages, and other components of earthquake damage can be shown on a seismic risk map together with existing structures or land use patterns. The map can be divided into grids scaled to the appropriate area of study, whether state, regional or local. Matrices accompanying the maps can evaluate more specific locations in relation to the degrees of hazard shown by the maps. And, as with environmental planning, recommendations for *degrees* of regulation correlated with *degrees* of hazard can be developed from and supported by the evidence shown on the maps.

The advantage of this methodology is that it can be presented in such a manner that it is easily understood by the layman. Thus, public understanding and participation are possible in considering appropriate risk levels, measures for damage prevention, density standards and regulatory provisions.

A more detailed approach to performing an impact analysis is presented in a document entitled, *A Procedure for Evaluating Environmental Impact*, prepared as a Geological Survey Circular 645, U.S. Geological Survey [23]. An application of this methodology to seismic impact assessment is presented in a following section of this paper.

Flood Plain Zoning: Lessons to be Learned

Flood plain zoning appears among the earliest land use planning efforts devoted directly to improving public safety from natural hazards, and can be considered a pioneering effort in environmental planning for hazard mitigation.

As a concept, flood plain zoning should not be confused with flood control measures. Flood control pertains to a system of engineering devices and structures to contain or divert flood waters; dyking, dredging, damming, channelizing, etc. The flood plain zone approaches the flood problem preventively, seeking to minimize damages at the site and downstream by keeping the flood plain relatively free of structures, fill and other detriments to its sponge-like capacities to store flood waters.

Zoning utilizes the police power for its authority. As was noted earlier, "The characteristic feature of zoning that distinguishes it from other police-power controls is that the regulations differ from district to district. For this reason it can be used to set special standards for land uses in flood hazard areas . . . The important aspect of zoning is that it can be used in riverine or coastal areas to regulate what uses may be located in flood hazard areas, where specific uses may be permitted, and how uses are to be constructed or carried out." [24] The usefulness of the zoning power in planning hazard amelioration is not, of course, restricted to floods. It could be equally well applied to areas vulnerable to extreme winds, earthquake activity, tsunamis, dam break, or any of the other natural hazards to be considered.

Case law resulting from environmental planning is still fairly spotted, too few cases and too many conflicting decisions to yet provide a basis for generalization. Flood plain zoning has, in contrast, a respectable body of case law to which reference can be made for parallel judgments on other hazard zoning.

It is important to remember that all zoning stems from nuisance law and *must show that it has its police power to protect the health, safety, and welfare of the public.* The general area of conflict in zoning regulation, as in all environmental management, lies between the individual property owner's right to enjoy the benefits of his property as he wishes, and the need for regulation of the uses of private property to safeguard the community as a whole. The courts require that, in the exercise of the police power, the regulations be *reasonable.* It must be shown that the restrictions upon the individual are necessary and effective in serving a proper public purpose, and that the benefits accruing to the community are great enough to warrant the degree of restrictions on the individual.

Planning which seeks to regulate land use to mitigate damages to the community from natural hazards enjoys a great legal advantage in that it addresses directly the charge of its responsibility for public health, safety and welfare. Even this advantage will be overcome, however, if the test of reasonableness is not met. For example, flood plain zone regulations which deprive a property owner of almost all remunerative uses of his land in marginal floodway areas with low probabilities of flooding *have been overturned by the courts.* The lesson for all hazard-related planning, thus, is *to match severity of the regulation to the severity of the risk.*

If it can be shown that regulation by police power is being used where condemnation and compensation for land taken for a public purpose is more appropriate, the courts will uphold the property owner. In flood plain zoning cases the suspicion that regulation was being used to protect land which a municipality wanted for water storage, parks, or wild life protection has led the court to rule against the flood plain zone provision. In formulating land use regulation to ameliorate natural hazards, therefore, the purposes of the regulation must be clearly limited to protection of public health and safety. Other public values such as recreation, open space or aesthetics must be considered apart from hazard zoning lest the authority of the police power be jeopardized.

There has been some speculation that the sophistication and detail of the data needed to regulate land use in hazard areas would make the cost of regulation unfeasible. Data collection and analysis is always expensive to some degree, and requirements on local government for new ranks of data, analysed and applied by new methodologies to new programs will surely give any local finance director pause. For this reason there is a strong case to be made for state and federal assistance to enable local government to fulfill the obligations imposed by higher authority.

Nonetheless, in the history of flood plain litigation, the demands of the courts for data supportive of regulatory decisions has been moderate and would not, of itself, constitute an obstruction to flood plain or other hazard zoning. The requirements have incorporated historical data to support flood high water level probabilities for the five year flood, etc.; benchmarks placed at reasonable intervals to establish elevations; a well defined map; a clear statement of prohibited and permitted uses as well as performance standards for the uses; and some flexibility of administration provided for nonconforming use, exceptional circumstance and the like. Similar stipulations have been customary in most land use zoning.

Flood Plain Zoning and Environmental Planning: Overview

As noted earlier, environmental planning and planning for land use as applied to natural disaster mitigation share many of the same concepts and concerns. It is understandable that planners seeking more comprehensive, more rational, and more interrelated bases for their plans, should favor an amalgamation of the two areas of concern. Kusler and Lee are no exception when they "have repeatedly urged that flood plain regulations be part of broader planning and regulatory efforts. Often it is desirable to preserve flood plain areas and wetlands to serve a range of goals . . . These lands are often rich in wildlife and other environmental values." [25]

Some caution is advisable here. If protection of the public from effects of natural disasters is the goal to be pursued, then practical, if short range, considerations dictate that current enablement measures in which there is legal precedence and established criteria should be sought. "The constitutionality of very stringent regulations which prevent all structural development to protect broad environmental values is still unsettled." [26]

Environmental planning and resulting legislation has been much criticized for its single focus ecological determinism at the expense of social and economic values. Richard L. Meier, in a much-cited article, states: "A more serious error in the positions taken by aggressive advocates of 'the environment as it should be' is their insensitivity to social justice, a position that will do their cause great harm over the long run." [27] Hazard amelioration planning, although in its infancy, has shown an excellent beginning in avoiding this pitfall.

Flood protection zoning, both coastal and riverine, has been built on a solid base of equity, reasonable probability and comprehendable risk factors correlated with appropriate protective requirements. The excellence of the grading codes achieved by the City of Los Angeles were only attained when the hazards were evident and public pressure supported the necessary regulations. And, a final example, the pioneering study for earthquake safety in the City of Long Beach [28], has provided a technical framework to improve rational decision-making in the public sector. It formulates "a code that is *not* handed down by edict of an elite technical community" [29] but provides instead "a means by which representatives of the lay citizenry can establish or modify code limits recommended." [30]

Planning of any kind, to be effective, must achieve popular and political understanding and support. Laws and ordinances which attempt to force the populace to do what technical experts believe to be good for them will be difficult to enforce and will not long endure [31]. Costs and benefits, those who pay and those who enjoy, must always be considered in the conflict and tradeoff process of the political arena. A continuing sensitivity to these vital truisms will insure that future land use planning related to natural hazards can be effectively implemented.

Examples of Current Regulations Which Consider Hazard

The following is a brief discussion of examples of current ordinances or proposed ordinances which require consideration of a natural hazard. The technology needed to effectively cope with the hazard problem is available today and can be implemented as evidenced in current codes. Although the ordinances presented in this discussion are for California governments, they serve as examples of what can be done in the general area of land use planning for disaster mitigation.

A. Seismic Safety Element - Section 65302.1 of the State of California Government Code

The State of California has recently amended the Government Code, relating to land use planning. This amendment requires that a seismic safety element be included in the general plan of general law cities.

Specifically, the law states:

> "The general Plan shall consist of a statement of development policies and shall include a diagram or diagrams and text setting forth objectives, principles, standards and plan proposals. The plan shall include . . . A seismic safety element consisting of an identification and appraisal of seismic hazards such as susceptibility to surface ruptures from faulting, to ground shaking, to ground failures, or to effects of seismically induced waves such as tsunamis and seiches." [32]

In support of the code, the California Council on Intergovernmental Relations has prepared and submitted to all of the City and County Administrators interim guidelines. The guidelines state that the basic objective of the revised State Code is

> "to reduce loss of life, injuries, damage to property, and economic and social dislocations resulting from future earthquakes." [33]

B. Long Beach Existing Buildings Rehabilitation Ordinance No. C-4950

This ordinance has several unique features which are important to the improvement of planning for hazards. They are presented as an example of one attempt to rationally interrelate zoning and building codes; and to show how the citizen and the elected representative, along with the career civil servant, are interrelated and all considered part of the policy-planning-implementation system for hazard mitigation.

1) It couples land use planning with a building design ordinance by requiring different designs in different zoned areas of the City.

2) It required the City Council to make a decision on risk (in this case death risk) which makes any future council accountable for its decision.

3) It prescribes performance standards for repairing or rehabilitating structures.

4) It provides for owner options of structure life and human exposure.

5) It provides a legal means for demolition of a structure at the end of the selected structure life. This requirement is stated in the title to the property.

6) Imperfections are recognized, specifically by requesting funds to provide for soil dynamics investigation to upgrade the risk maps.

7) It provides for a uniform rating system by the Building Official within a priority structure.

8) It allows for new construction materials or designs affecting the damping properties, to be reviewed for purposes of adjusting code values, thus lowering costs.

9) It provides a measure for dynamically analyzing structures and sites. Thus, requiring building official personnel to be upgraded in their profession.

C. <u>County of Los Angeles Interim Seismic Design Consideration for County Buildings - Chapter 2-B-.07</u>

This policy incorporates similar principles to those considered in the Long Beach Ordinance, specifically importance of vital structures, cost risk, life risk, and land use. In addition Geologic-Seismologic investigations are to be obtained for all projects involving structures.

D. <u>Senate Bill 479, (1971) to be amended by SB689 (1972), California State Legislature</u>

This bill would prohibit the construction of a school over the trace of an active fault or below an area of slide, or in any other location where the geological characteristics are such that the construction effort required to make the site safe for occupancy is economically unfeasible. This is one of the first legislative attempts to prohibit construction near or on an "active fault."

E. <u>Federal Legislation</u>

Federal guidelines are currently being developed but in a very fragmented manner. HUD, FHA, AEC, EPA, etc., are all developing guidelines but apparently without coordination with each other and/or local legislative jurisdictions. In many instances, the vocabularies differ and the meanings of key words such as "active fault" have different connotations. These discrepancies should be eliminated and the regulations or ordinances refined and integrated.

<u>Summary</u>

As explained in the foregoing discussion, there are land use planning practices which consist of a series of surveys, analyses and policy decisions starting at the general level and ending with specific regulations regarding land use. As such, land use planning can and must encompass the full range of policy-planning-implementation actions of any governmental unit engaged in disaster mitigation.

As evidenced by environmental legislation and flood plain zoning experience, as well as current hazard regulations, the "state of the art" is sufficient to develop plans, programs and controls which effectively cope with the problem of natural disasters. These plans, programs and controls must, however, be prepared by engineers and planners in coordination with professionally <u>qualified</u> governmental personnel; and presented in a manner which both the <u>political</u> and citizen groups can understand and which can economically and effectively provide safety from natural hazards. Only then will it be possible to provide the necessary enforcement required for disaster mitigation.

<u>VI. An Approach to Seismic Impact Assessment</u> [34]

The following discussion presents a methodology for preparing a seismic safety element for a General Plan similar to that utilized in the preparation of environmental impact statements. This same methodology can be applied to the other natural hazards, flood, wind, landslides, etc.

<u>Preparation of A Seismic Impact Assessment</u>

Seismic safety requires a consideration of the hazards associated with earthquake in terms of the location of new structures and facilities, the rationale and criteria utilized in design, an evaluation of existing structures and facilities, and the efficacy of existing codes and ordinances. The following basic modes of failure that can occur during an earthquake in both open and developed areas are to be considered.

a. Ground Shaking (addressed primarily by building code development and enforcement)

- failure of structures due to shaking
- foundation failure beneath structures due to soil bearing failure
- differential settlement of structures due to soil compaction

b. Permanent Ground Movement (addressed primarily by planning and zoning code enforcement)

- landslide
- fault break through ground surface
- liquefaction of subsoil
- general land subsidence or lurching beneath structure

c. Water Forces and Flooding (addressed primarily by planning and zoning code enforcement)

- failure due to tsunami or seiche
- flood due to dam break

To properly consider these failures and to balance the risks between the failure modes, a specific sequence of acts must be accomplished which will result in the development of a seismic element from which to make planning policy decisions.

A. Statement of Objectives

As a first step each jurisdiction must set forth its objectives regarding the planning element related to seismic hazard. The objective statements should define the boundaries of the area to be evaluated and the degree of sophistication to be incorporated into the evaluation. A plan of action should be developed and the technologic possibilities of achieving the objectives analyzed.

B. Preparation of an Emergency Plan

Since earthquakes occur without warning an emergency plan should be of the highest priority in the preparation of a seismic element. The plan should contain preparedness measures, as well as a response and recovery plan. If prepared immediately, the plan will not have the benefit of the completed hazard study, but it can become the basis of an improved emergency plan which should be developed upon completion of the seismic element. Early preparation of an emergency plan will serve to mitigate the effects of a disaster, even though formal programs have not been fully implemented.

C. Examination of Engineering Geologic and Soil Characteristics

Some State agencies, agencies of the Federal Government, and private firms are actively developing seismic and geologic data necessary to prepare a seismic element. This data (including surface and subsurface maps) must be collected and examined in order to establish the geologic environment and the related soils mechanics and dynamic properties of the subject area.

D. Development of Seismic Intensity Risk Maps

The results of hazards analyses are generally presented in the form of *risk maps*, which portray the type and degree of hazard represented in a given geographic location. Frequently, however, risk maps are prepared for large regional areas which may not take into consideration local characteristics and local hazards. Further, they are frequently prepared for undeveloped areas on the tacit assumption that the developed areas are not exposed to the same risks as undeveloped areas. Micro-regional seismic risk mapping is therefore required in order to develop a rational element for the general plan of any local jurisdiction.

The risk maps are developed from the performance of a seismic risk analysis. The results of this analysis, utilizing statistical techniques, the geologic and soils characteristics, and the past earthquake history are shown on appropriately scaled maps divided by grids. Matrices accompanying the risk maps show earthquake intensity *probabilities* for each map grid or cell. The decision matrices identify the probability of failure for each of the nine failure modes in each cell. Since the earthquake hazard is directly related to the numbers of people and structural types exposed, each decision matrix must relate the risk of earthquake induced failure to structural life, building importance or occupancy and economic factors.

E. Coordination of Risk Levels

At this point, the seismic impact analysis is performed based upon the risk of failure in each mapped cell for the selected alternative modes of failure considered important.

Failure of structures due to ground shaking and foundation failure due to soil compaction or soil bearing failure are generally associated with building codes. As such, a seismic impact analysis must include an assessment of the potential hazard for both existing and new structures based upon an evaluation of the building code and the earthquake resistant capacity of existing structures.

Structural failures due to fault rupture, landslide, liquefaction of soil, land subsidence and lurching, tidal waves and dam break are generally associated with zoning ordinances. In this area the impact analysis should include an assessment of the current zoning regulations and planning policies as related to the general physical land use elements; specifically, housing, transportation and circulation networks and community/public facilities.

In its combined form the risk analysis can be graphically illustrated as a three dimensional matrix (Figure 1). Each cell from the Community Seismic Risk Matrix may contain several alternative probabilities of earthquake intensity and risk levels which must be correlated with all of the various structural types located in the cell area.

Based upon this impact analysis it is possible to assign risk levels for loss of life and property in each cell area. Since the hazard in each area depends upon the various failure modes considered, as well as the selection of risk levels, all are to be shown on hazard or risk maps. This will present a comparative hazard for each area.

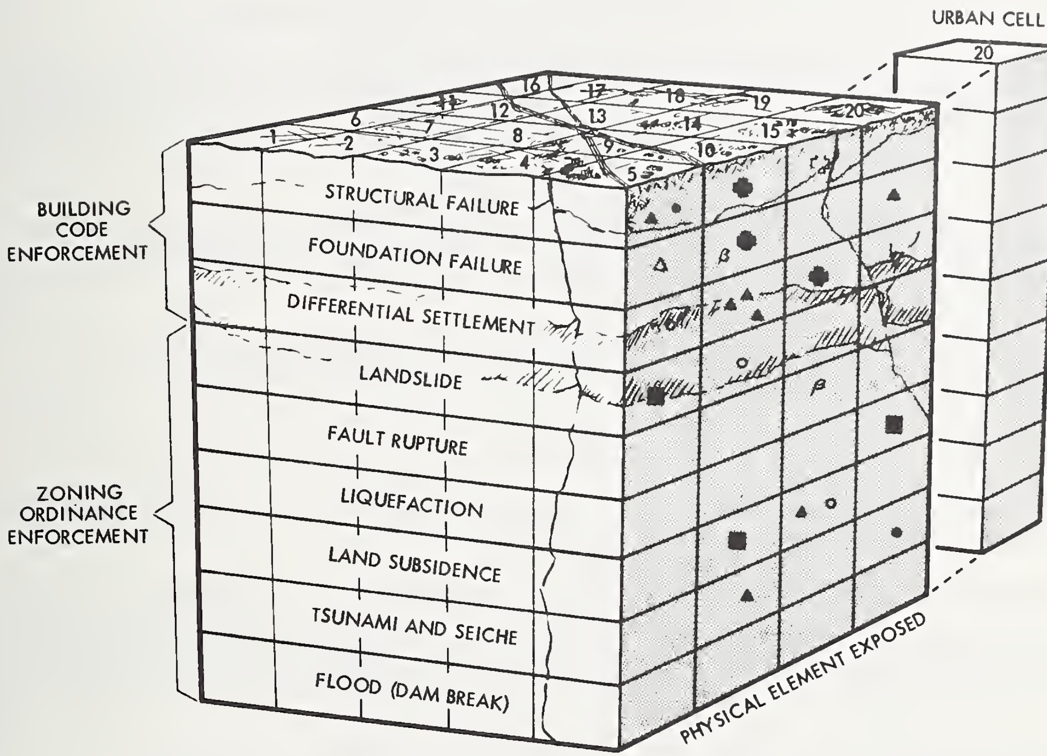

FIGURE 1 COMMUNITY SEISMIC RISK MATRIX

The seismic impact analysis can be made with the aid of a seismic impact matrix. The matrix would include on one axis the various modes of failure; and alternative risk decisions based upon time and probable death risk or probable economic loss. The other axis of the matrix would identify the various physical elements and structures which would be affected in the event of a failure. Old, new and proposed structures are to be considered. This system provides a format for a comprehensive review to remind the investigators of the variety of interactions that might be involded. It also aids the planners in identifying alternatives which could lessen the impact of an earthquake. A typical impact matrix is presented in Figure 2.

F. Preparation of the Seismic Safety Element of the General Plan

The actual seismic safety element should include the risk maps which establish limits on development and redevelopment based upon the probability of seismically induced failures. In addition, the risk maps should identify the risk exposure for those already developed areas within the Government unit.

The test of that section of the general plan containing the seismic safety element should include an assessment of the impact analysis and a discussion of the individual critical areas. The discussion should cover the following points:

1) A description of the proposed action including information and technical data adequate to permit careful assessment of the impact.

2) Alternatives to the proposed action.

3) A discussion of the social and economic impact of each proposed action.

4) The relationship between short term and long term programs.

Based upon the risk and impact analysis, appropriate program alternatives designed to mitigate the disaster effect of future earthquakes can be incorporated in local planning policy. A typical set of programs may include:

1) A program for improving the emergency plan based upon an understanding of the risks, gained from the results of the completed seismic impact analysis.

2) Development of a means of equitably rehabilitating or eliminating high risk hazards associated with existing developments and establishment of various capital improvement projects aimed at eliminating high risk situations.

3) Development and implementation of inspection and evaluation programs for evaluation of old structures.

4) Implementation of appropriate building code and zoning ordinance revisions and the revision of other associated regulations.

5) Performance of more detailed seismic, soils, and geologic investigations of problem areas where required in order to gain confidence and minimize hazard.

6) Implementation of appropriate programs for protection of existing critical facilities against the earthquake hazard.

7) The design of a seismic monitoring instrumentation program to

INSTRUCTIONS

1. Examine RISK DECISION MATRIX for each failure mode and identify RISK FACTORS considered <u>important</u>.

2. For each of the <u>important</u> RISK FACTORS identify the PHYSICAL ELEMENTS EXPOSED and place a slash through the appropriate box.

3. In the upper left hand corner of each box with a slash, place a number from 1 to 10 which indicates the <u>magnitude</u> of the possible impact in terms of deaths; 10 represents the greatest magnitude and 1, the least. In the lower right hand corner of the box place a number based on the same scale indicating the possible <u>magnitude</u> of the impact in terms of economic loss.

4. The text which accompanies the matrix should provide a discussion of the significant risk areas.

	I. HIGH SEISMIC RISK									II. MEDIUM SEISMIC RISK									III. LOW SEISMIC RISK								
	A. GROUND SHAKING			B. PERMANENT GROUND MOVEMENT				C. WATER FORCES		A. GROUND SHAKING			B. PERMANENT GROUND MOVEMENT				C. WATER FORCES		A. GROUND SHAKING			B. PERMANENT GROUND MOVEMENT				C. WATER FORCES	
	1. Main Structure Failure	2. Foundation Failure (Soil Bearing)	3. Differential Settlement of Structure	1. Landslide	2. Fault Break	3. Liquefaction	4. Subsidence	1. Dam Break	2. Tsunami or Seiche	1. Main Structure Failure	2. Foundation Failure (Soil Bearing)	3. Differential Settlement of Structure	1. Landslide	2. Fault Break	3. Liquefaction	4. Subsidence	1. Dam Break	2. Tsunami or Seiche	1. Main Structure Failure	2. Foundation Failure (Soil Bearing)	3. Differential Settlement of Structure	1. Landslide	2. Fault Break	3. Liquefaction	4. Subsidence	1. Dam Break	2. Tsunami or Seiche
PROBABLE LIFE LOSS RISK FACTORS	X_{11}	X_{12}	X_{13}	X_{14}	X_{15}	X_{16}	X_{17}	X_{18}	X_{19}	X_{21}	X_{22}	X_{23}	X_{24}	X_{25}	X_{26}	X_{27}	X_{28}	X_{29}	X_{31}	X_{32}	X_{33}	X_{34}	X_{35}	X_{36}	X_{37}	X_{38}	X_{39}
PROBABLE ECONOMIC LOSS RISK FACTORS	Y_{11}	Y_{12}	Y_{13}	Y_{14}	Y_{15}	Y_{16}	Y_{17}	Y_{18}	Y_{19}	Y_{21}	Y_{22}	Y_{23}	Y_{24}	Y_{25}	Y_{26}	Y_{27}	Y_{28}	Y_{29}	Y_{31}	Y_{32}	Y_{33}	Y_{34}	Y_{35}	Y_{36}	Y_{37}	Y_{38}	Y_{39}
PHYSICAL ELEMENTS EXPOSED — 1. HOUSING: a. SINGLE FAMILY																											
b. MULTI-UNIT - 2-6 STORIES																											
c. MULTI-UNIT - 7+ STORIES																											
d. MOBILE PARKS																											
e. OTHER																											
2. COMMERCIAL: a. RETAIL - SINGLE STORY																											
b. RETAIL - 2-6 STORIES																											
c. RETAIL - 7+ STORIES																											
d. OFFICE - SINGLE STORY																											
e. OFFICE - 2-6 STORIES																											
f. OFFICE - 7+ STORIES																											
g. HOTEL/MOTEL - SINGLE STORY																											
h. HOTEL/MOTEL - 2-6 STORIES																											
i. HOTEL/MOTEL - 7+ STORIES																											
j. THEATRES																											
3. INDUSTRIAL: a. LIGHT MANUFACTURING - ONE FLOOR																											
b. LIGHT MANUFACTURING - MULTI-FLOOR																											
c. HEAVY MANUFACTURING																											
d. CHEMICAL PROCESSING																											
e. OIL & GAS REFINERIES																											
f. CHEMICAL STORAGE																											
g. OIL & GAS STORAGE																											
h. WAREHOUSES																											
i. GAS & OIL TRANSPORT LINES																											
j. EXPLOSIVE MANUFACTURER																											
4. TRANSPORTATION NETWORK: a. MAJOR ARTERIAL HIGHWAYS																											
b. MAJOR ARTERIAL BRIDGES/OVERPASSES																											
c. SECONDARY ROADS																											
d. BRIDGES - SECONDARY ROADS																											
e. RAIL RIGHT OF WAYS																											
f. RAIL BRIDGES																											
g. RAIL TERMINAL FACILITIES																											
h. AIRPORT FACILITIES																											
i. AIRPORT EMERGENCY FACILITIES																											
j. AIRPORT RUNWAYS																											
5. PUBLIC/COMMUNITY FACILITIES: a. HOSPITALS																											
b. SCHOOLS																											
c. LIBRARIES																											
d. AUDITORIUMS																											
e. FIRE FACILITIES																											
f. POLICE FACILITIES																											
g. JAILS																											
h. EMERGENCY COMMUNICATION CENTERS																											
i. CITY OFFICE BUILDINGS																											
j. PUBLIC WORKS FACILITIES																											
k. POWER GENERATION FACILITIES																											
l. TELEPHONE SWITCHING FACILITIES																											
m. SEWAGE PROCESSING FACILITIES																											
n. GAS STORAGE TANKS																											
o. WATER STORAGE RESERVOIRS																											
p. DAMS																											
q. UTILITY TRANSPORT SYSTEMS																											
r. PUBLIC PARKS																											
s. OPEN SPACE																											
6. OTHER: a. BUILDING CODES																											
b. ZONING ORDINANCES																											
c.																											
d.																											
e.																											

RISK DECISION MATRIX
PROBABLE LIFE LOSS
MAIN STRUCTURE FAILURE

HIGH RISK X_{11}		EXPOSURE FACTORS (eg., 1 death/million people per year)					
		1	2	3	4	5	6
EXPOSURE YEARS	5						
	10			RISK FACTORS			
	20						
	40						
	100						

FIGURE 2 INFORMATION MATRIX FOR SEISMIC IMPACT ASSESSMENT

provide data for the continual upgrading of knowledge necessary to more accurately assess risk.

8) Development of a program for updating the various parts of the seismic element as more and new data become available.

Summary

The current state of the art will permit the preparation of a seismic element utilizing the procedure outlined above. It is understood that our knowledge of seismic effects is neither complete nor perfect; and that uncertainties do exist. However, objective analysis which incorporates all available data and knowledge should provide a better and more rational foundation for a decision; even though the decision may remain one of *muddling through*.

VII. Conclusions and Recommendations

It may be concluded from the foregoing discussion that the process of achieving an effective system of planning for disaster mitigation is one that is frought with many difficulties. It is in this area that the general state-of-the-art needs to be upgraded. It is possible to improve our codes by applying existing knowledge in a manner which all parties involved can comprehend. To make no provisions in the codes, because we cannot accurately predict a disastrous event, is to make no decision and thus, reject the logic of probabilities and the knowledge gained from science.

Regulations cannot be perfect and accurate for all situations. Thus, they must: (1) reflect the concept of risk and uncertainty; (2) be dynamic in allowing for improvement through increased understanding; (3) be rationally interrelated and tied to a plan which considers various disasters as key elements; (4) be based on a logic which the legislator, citizen, and executive (administrator) can fully comprehend, and thus allow for effective participation in the decision making process.

In order then, to make effective progress toward improving our condition, it is necessary that new information and knowledge be utilized. The following is a list of recommended actions for Federal, state and local levels of government.

Federal Government

1. Develop criteria for establishing levels of risk, compile and maintain guidelines of good standard practice for engineers and planners to achieve various levels of risk, and make building owners aware that current practice involves risk to the owner.

2. Provide a clearinghouse for dissemination of engineering design criteria of regulatory measures and administrative processes to assure implementation at the Federal and local levels.

3. Provide coordination of planning processes to assure reasonable consistency among states. Specifically, prepare a planner's "handbook" for disaster mitigation which formulates elements within the comprehensive plan to include:

   (a) Guidelines and procedural steps for preparing disaster elements, such as California's Seismic Safety Element.

(b) Regulatory instruments and procedures for implementation of controls, such as "Earthquake Hazard Regulations for Rehabilitation of Existing Structures Within the City," (Subdivision 80, Long Beach Municipal Code).

4. Coordinate disaster mitigation research and disaster investigations.

5. Function as data bank and actively disseminate information to those concerned. Specifically,

   (a) The Federal Government should undertake a comprehensive program of disaster risk mapping appropriate to the needs of local planning and regulatory bodies, the design and building professions, and industry practitioners.

   (b) In so doing, it should make a feasibility study of how to undertake an inventory of buildings that would provide data on the type of occupancy, type of construction and other details of construction needed to make viable potential risk evaluations.

   (c) Concurrently, there should be a study to establish the damage probability of the buildings for various kinds of hazard: Engineering organizations with experience in earthquake resistant design should be utilized for this study.

   (d) Data concerning the probability of different levels of hazard occurrence must be compiled and made available through data banks. The form and detail of the necessary data are described in other recommendations appearing in this report. It is essential that initial efforts using simple (and possibly crude) measures of hazard be completed before more detailed compilations are undertaken.

   (e) Data must be developed concerning the many less immediate and often intangible costs of disasters: Loss of productivity, loss of tax base, psychological impacts on individuals and community, etc. Attempts to express all such costs in terms of dollars should be avoided; for some studies, for example, the choice of the best alternatives may be insensitive to any dollar value of human life.

6. Employ financial grant and loan insuring programs to encourage enactment of effective land use planning and building codes and their strict enforcement. Specifically, the Federal Deposit Insurance Corporation, the Federal Home Loan Bank Board, The Federal Housing Administration, and the Veterans Administration should request that the amount of money loaned or guaranteed be insured against Natural Disasters. Furthermore, natural disaster hazards posed by old and poorly-maintained buildings should be one of the important factors considered in Federally financed urban renewal programs.

7. Immediately commence with programs to correct deficiencies in buildings occupied by the Federal Government in order to assure the survival of vital government facilities in times of natural disaster.

8. Incorporate hazard mitigation considerations in Government building and leasing operations comensurate with local risks, including certification of Federal construction projects on an individual basis.

9. The Federal Disaster Assistance Program should be reformulated so as to provide incentives for disaster mitigation efforts prior to a disaster and to remove incentives for ignoring the risks associated with a structure and site. Specifically, the Federal Disaster Assistance Program should recognize the need to consider post disaster rehabilitation and replacements to be designed and constructed with a view towards

mitigating future losses.

10. The Internal Revenue Code should be reviewed to identify features that would provide equitable tax relief to individuals incurring losses in natural disasters and taxing procedures for insurance companies writing policies on infrequent disasters.

State and Local Government

1. Establish minimum rules, regulations and enforcements to provide for the safety of the public and building occupants considering the hazards of natural disasters.

2. The legislative body adopting a building code should create by statute or ordinance a "Review (or Appeals) Board" to hear appeals from the decisions of Building Officials and requests for variances. The legislation creating the board should define its functions and require that most, if not all, members must be licensed professionals.

3. States should implement criteria for organization, staffing, procedures, qualification, and certification of personnel for regulatory bodies.

4. States in cooperation with the appropriate federal agencies should set minimum standards with respect to land use planning, building codes and emergency planning to protect vital emergency services during and after the occurrence of natural disasters.

5. States should require that local land use planning and public regulatory measures be coordinated to assure that adequate disaster mitigation provisions are incorporated into comprehensive plans. In this regard, local land use plans should include a section or element dealing with natural disasters and other hazards faced by local jurisdictions including definition of acceptable risk levels.

   Also, states should provide for the establishment of a coordinating mechanism to assure that the requirements of locally-conceived plans, codes and ordinances are consistent with accepted area-wide standards as related to natural disasters.

6. Regulations for land use and building construction should include provisions for evaluation of the safety of existing buildings against all natural hazards.

7. Locally developed land use plans should be coordinated with area-wide and state land use planning, to assure that maximum effective use is made of limited resources.

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34. This Section is a revised version of a PAS Memo entitled, "Procedure for Developing a Seismic Safety Element for the General Plan," by William J. Petak, soon to be published by the American Society of Planning Officials.

ARCHITECTURAL APPROACHES TO HAZARD MITIGATION

by

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and

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I. Introduction

This review paper presents an evaluation of current practice as it relates to Architectural Approaches to Hazard Mitigation. The subject matter endeavors to treat the topic as stated in the program announcement for the forthcoming Workshop sponsored by the National Bureau of Standards. The scope of this paper is:

> "Building forms, spatial distributions, and interior designs can be adapted to natural disaster hazards as part of the design to provide a functional environment in harmony with the natural environment. The review should evaluate paradigms, procedures, and criteria for synthesis of external and internal forms which provide effective function and benign interactions with earthquakes, extreme winds, explosives and similar causes of disasters. Methods should be reviewed for encouraging architectural approaches to disaster mitigation at the programming and schematic design stages."

Although the program is intended to be of national scope, the principal subject presented relates to building hazards caused basically by earthquakes and, for this reason, most of the information revolves about the current situation in the State of California where the occurrence of earthquakes and earthquake-resistant design are more prevalent.

The design of structures for wind forces can be quite complicated and current codes list wind forces which usually vary with the height of the building. Designing a structure for the largest code force is the simplest method; but, when designing larger structures, the determination of wind forces by modeling or by a micro-climatologist may result in considerable construction economy. The shapes of the building could be so arranged that the least surface area would be normal to the "prevailing wind" direction. Sheltered conditions may be a factor. (Paper 4 will review "Procedures and Criteria for Wind Resistant Design.")

Explosion hazards are rather unusual situations and are somewhat undefinable but, in general, good earthquake resistant design and attention to accepted fire and panic practices would provide a substantial deterrent to such damage. An approach to hazard mitigation in blast design would be to provide a structure that would not fail progressively due to a local failure.

Because of the regional aspect of earthquakes, most of the references to current practice now appearing in the Building Codes and research documents are of California origin.

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This paper will be presented in three main parts under the following headings:

- Current Architectural Practices and Policies with Respect to the Design of the Structural Portions of a Building.
- Current Architectural Practice as it relates to Design of the Non-Structural Interior Elements of a Building.
- Summary of Recommendations to Further Implement Building Hazard Mitigation.

In the Appendix of this paper, references to various Codes, Rulings, and research data by California engineering societies and manufacturers' manuals are listed.

II. Current Architectural Practices and Policies with Respect To the Structural Design of a Building

Architect-Engineer Consultant Relationship

On large, major structures it has been the current general policy of Architects to engage a Consulting Structural Engineer to design and prepare the structural plans for the project. In some areas, if warranted by a steady volume of commissions, large architectural firms have established in-house engineering capability to perform this service.

A majority of Architects engage structural services on the more modest projects and even on small structures. Many projects, however, are developed with a minimum of structural assistance, perhaps only partial service. This is most unfortunate because the Architect-Engineer relationship is possibly the most important parameter in achieving good structural design as a means of promulgating hazard mitigation.

Virtually every state in the nation, through registration laws, requires structures to be designed by Architects or Civil Engineers and the State of California further restricts design to Architects or Structural Engineers (Civil Engineer plus Structural Certificate) for all public school buildings.

By custom, and justifiably, the Architect is generally engaged as the prime professional, as he is basically trained and qualified in planning and esthetics and is legally permitted to design all facets of a building project. In seismically active regions, the role of the Architect becomes more complex. Inflation and the resultant demand for economy in construction compounds his design problem. Very often his client may not accept the esthetics of an "earthquake-safe" design, nor permit spatial interruption that might be required to provide a high level of lateral load resistance.

Except for unusual structures, static vertical load design is not mysterious or complex, but can become increasingly complex when lateral force design must be recognized, and more particularly so when the project is located in known or suspected seismic regions. The need for a close relationship between the Architect and Structural Engineer is generally well recognized by these professionals in California.

A vivid historical picture of earthquake damage and loss of life dates back to the San Francisco earthquake and fire of 1906.

Of necessity, accelerated interest in seismic design followed the Long Beach earthquake in 1933, and for the first time, legislation was enacted to require recognition of this serious building hazard. Apparently the lesson of proper structural design was not learned until 1933. Countless hours of time and research by engineering society committees and academic circles has brought us to the current "State of the Art."

Much of this "art" and current code provisions were generated by the Structural Engineers Association of California which, in 1959, prepared the document "Recommended Lateral Force Requirements and Commentary." Many revisions have been made, the last being Appendix F, which includes 1969, 1970 and 1971 revisions. This "blue book" has formed the basis for seismic design since 1960 and has strongly influenced seismic design throughout the world.

In 1971 a new document, "Report of the Ad Hoc Committee on Direction Study - Seismic" was published in the Annual Proceedings of the Structural Engineers Association of California. Included in this document is the subject "Risks Not Considered by Code," which points up two major areas of design process, namely, siting of the project and basic architectural concepts [7].

The complexity of current seismic design virtually demands that the Architect and the Consulting Engineer have an extremely close relation, particularly in the initial design or schematic phase of the project. Currently this relation is not always as close as it might be, nor does it occur as early in the project planning as it should. All too often the architectural design concept is crystallized before all structural problems are known and consequently the structural design becomes a compromise involving esthetics, economy and good seismic-resistant design. This situation seems more prevalent in the small to medium sized projects where, for some reason, the importance of the Architect-Engineer relation is discounted. The same consulting fee can generally cover the complete service when the Engineer becomes involved at the outset of the project.

Geology and Soil Stability Consideration in Site Selection

The selection of a building site by an Owner can have a great impact on the design and ultimate cost of a project. Unfortunately, the selection of urban area sites and, to a substantial degree, sites in outlying districts of metropolitan population centers is not very flexible. Where a choice is available, preliminary geologic studies are occasionally made in addition to the customary soil stability and capacity investigation in order to select the most favorable site. It is essential therefore that the Architect and Engineer be involved at the outset of the project to aid in site selection, but currently this early relation with the Owner seldom occurs.

Where site selection is inflexible, current practice is to adjust the design within economically feasible limits to provide reasonable safety against geologic hazards.

Soils geology is a relatively new profession, at least in regard to its application to building design. It will take some time for data banks to accumulate enough information for general use.

In general, building regulatory agencies have no provisions requiring geologic studies of a building site. The State of California Education Code does require that geologic investigation be made prior to the acquisition of a school site and for additions to an existing school facility where no investigation has been previously made [1].

An ordinance adopted by Los Angeles County in 1970 prohibits the construction of a building directly over a known fault trace, or within a certain distance. There are evidently many fault traces not located and require trenching to uncover, if possible [2]. This is a step in the right direction but does not solve all the problems that should be considered in earthquake-resistant design. The County Code also includes a broad provision relating to flood hazard and earth fills along with reference to geologic hazards, but this is aimed primarily at subdivision developments. Alameda County (San Francisco area) has a similar ordinance. Article 5 of the California State Planning Law has similar provisions for the same purpose. On July 14, 1972, the Governor's Earthquake Council elaborated on this law by preparing a document entitled "Suggested Interior Guidelines for the Seismic Safety Element in General Plans" [2].

In March 1972, Los Angeles County commissioned Woodward-McNeill & Associates to prepare a comprehensive study on "Geologic and Seismic Consideration for Land-Use Planning." This study will produce information on two essential items, namely: (a) Land-Use Maps, which will delineate areas where various earthquake-resistant features should be provided for building; and (b) Safety Guidelines (Code Provisions) for the location, design and construction of structures in the County [3].

Currently, subsurface soils investigations which are made on a large percentage of projects include some geologic data, but in general the "State of Geologic Art" is such that many potential geologic problems cannot be accurately predicted. The San Fernando earthquake of 1971 in Southern California where considerable damage caused by earth faulting and surface rupture occurred, demonstrated that much of this ground failure would have been difficult to predict.

<u>Structural Framing Systems v.s. Architectural Design</u>

Subject matter presented relates primarily to structural system design for lateral forces. Structures should be designed for vertical loads with proper consideration given to the effect of lateral loads. The structure should then be analyzed for lateral loads (wind or seismic) and connections and members modified to accommodate the combined loading. Consideration must be given to the drift of the building to minimize non-structural damage.

There are two basic design systems to provide lateral force resistance in a structure: (1) shear wall, or braced frame systems, and (2) moment frame systems. These systems may be used in combination if proper consideration is given to their interaction. Codes recognize the differences in these systems and assign lower seismic loading to the moment-resisting frame.

The architectural planning problem is generated in endeavoring to provide open floor areas uninterrupted by permanent structural walls or extremely large columns. Also, the esthetic design of the exterior facade may demand a high degree of fenestration. These two criteria generally set the pattern for the structural system. The Architect and Engineer should work to develop a symmetrical framing system that will meet architectural requirements.

Current design practice generally utilizes moment frames exclusively in structures more than 15 stories high. These can be of structural steel or ductile concrete frame design. Frame structures obviously provide a maximum latitude in architectural design.

Structures under 15 stories high and particularly those in the 2 to 10 story range create the broadest spectrum of shear wall v.s. frame design as shear wall structures in lower rise buildings are generally more economical provided the architectural planning can accommodate them. It is in this range that current knowledge and code provisions are multi-faceted. It is also in this area where the ingenuity of the Architect is taxed to the utmost in developing effective tenant space and producing an esthetically acceptable building. If minimal interior and/or exterior wall elements only are available, current design criteria provide variable lateral load factors when frame action is also included. The architectural design then must be balanced with economy and good seismic design which translates into employing the maximum amount of wall consistent with function and appearance.

Code lateral force factors now can vary as much as 50 per cent, when building height and amount of walls vary. Unfortunately these theoretical reductions do reduce the reserve strength in a structure and current Code factors do not take into account irregular floor plans and the mixture of different building materials. An example would be an "L" shaped building of variable height, using steel, masonry, concrete and wood throughout the structure. Here, the absence of horizontal symmetry, vertical regularity coupled with the marriage of different materials creates serious architectural design problems with respect to inherent safety. Reserve strength available in older structures has been constantly reduced by development of lightweight building materials, both structural and non-structural and the use of lightweight and relatively flexible exterior architectural facades. Hazard mitigation is inherently improved when architectural planning accepts the assistance of good engineering judgment used within the framework of existing Building Code provisions.

Due to the current gap between scientific data and its practical application to building design, reliable knowledge of actual lateral loading and resultant building response is still limited. Therefore until more capability is generated, the Architect of necessity must give serious consideration in spatial and esthetic planning to simplify lateral force problems. This is an enormous order, considering the fact that he is in competition for commissions awarded many times on the basis of exotic designs and severe economy demanded by clients who do not completely comprehend (except for the first few days following a disaster) the serious need for recognition of hazard mitigation.

Selection of Exterior Facades

Building perimeter enclosure walls can be of the following types:

- Concrete walls, piers and spandrels poured monolithically with the frame.
- Pre-cast concrete panels of architectural concrete with or without bonded stone veneer.
- Masonry filler type walls.
- Stone or ceramic veneer applied on concrete or masonry walls.
- Metal and glass curtain wall panels.
- Wood or metal studs and plaster.

The type of exterior facade selected is based on the architectural design and structural requirements, with economy being an important factor. The integration of the selected facade with the total structure should provide for an infrequent, but full, fury of nature. Monolothic concrete enclosure elements generally act also as lateral force resisting elements. Precast attached units may also be used to resist lateral forces, but generally in low rise structures. In these cases, the structural design requirements may have a substantial impact upon the architectural design. More architectural freedom is permitted when the other non-lateral force resisting types are selected.

Current practice and code provisions establish criteria for lateral force design and connections. These current methods to date have performed rather satisfactorily with virtually little history of failure in recent earthquakes. In addition to design for wind (15 to 30 psf) or seismic loading (.2W) normal to the wall, specific requirements are established in codes for connections. Connections to the frame shall resist twice the weight of the panel. Connections and joints shall allow a relative movement between stories of not less than two times the story drift or 1/4 inch, whichever is greater. Connections must have sufficient ductility and rotational capacity to preclude fracture of concrete or brittle failure at welds. Design shall also allow for story drift in the plane of the panels. Obviously, these design provisions place an extra burden upon the Architect in that his design must accommodate structural features.

Equivalent design requirements are used for "curtain wall" panels and general practice follows standards set forth in good detail in the "Metal Curtain Wall Manual" published by the National Association of Architectural Metal Manufacturers.

Attachment of concrete, masonry and ceramic veneers is specified in detail in Chapter 30 of the Uniform Building Code, and also in corresponding chapters in the major California city codes as well as the California State Code (Title 21) which is somewhat more stringent. The systems currently in use have performed satisfactorily. Adhesive methods have experienced some minor problems.

Current glass and glazing criteria set forth in Chapter 54 of the Uniform Building Code and corresponding chapters in local codes appear to provide adequate safety against lateral force loading, although some breakage has been experienced in stronger earthquakes. Investigation of these failures usually reveals improper installation. However, some control over glass pane clearances for distortion in the plan of the window wall should be established. In regions of high wind velocities, more attention must be given to strength and particularly deflection characteristics of glass panels normal to their surface. The elimination of potential glass breakage is probably the most important item in architectural hazard mitigation.

Current Research by Various Agencies

With respect to earthquake hazard mitigation, the Seismology Committee of the Structural Engineers Association of California, functioning in close relation with the Seismology Committees of the four local area Structural Associations, has exerted the greatest influence on earthquake-resistant design. Engineering Departments of the major colleges in California provide substantial input as many faculty members serve on these committees. Similarly, staff members of research departments of the various building code enforcement agencies perform a similar function. Record of accomplishment has been very good, despite the fact that practically all service is provided without remuneration.

Occasionally some industry research is generated but usually occurs when a proprietary item of a manufacturer needs acceptance by a building code agency. This research is generally performed by a commissioned consulting engineering firm.

The Applied Technology Council, a non-profit corporation sponsored by the Structural Engineers Association of California has been established to provide the primary function of implementing current technological research development into active building design practice. The Council will operate with a limited paid staff who will define needed projects and award commissions on a priority basis to qualified agencies. Other professional design societies will probably be contacted and invited to join or provide input.

III. Current Architectural Design Practices and Policies

With Respect to Interior Non-Structural Elements in a Building

Architect-Engineer Consultant Relationship

Interior elements of a building such as ceilings, light fixtures and partitions installed by historically conventional methods offer no special hazard except in regions subjected to earthquakes. In California, relatively little engineering attention has been given to this segment of construction until recent years. It has become necessary that lateral force design attention be given to these elements in order to eliminate hazard to occupants and to minimize property damage. As part of their engineering service, many consultants assist architectural clients by reviewing details of these "non-structural" items.

Current Design Criteria for Ceilings, Fixtures and Partitions

Except for free floating "cloud" type elements, suspended metal lath and plaster ceilings on a steel channel system create a minimum hazard because of their inherent stiffness and ability to transfer load to perimeter structural elements, provided the ceiling system is properly wire tied to the structure. Integrated ceilings consisting of light metal T-bars, drop-in acoustic panels and light fixtures require greater structural attention. California State and some local Building Codes have established minimum lateral bracing criteria for ceilings of this type. In general, these Codes require that the ceiling construction be designed to resist or successfully transfer a load 1 pound per square foot minimum, or 20% of the total ceiling weight to structural elements plus lateral loads from partitions that are not independently braced. Additional independent lateral and vertical support is required for light fixtures equal to 100% of the fixture weight. The Ceilings and Interior Systems Contractors Association has prepared recommended standards which present in more detail the requirements of current building codes. This standard pattern will probably be adopted [4, 5, 6].

Light fixtures other than those set in integrated ceilings, such as pendant type are required to have ball joints permitting large arc free swing or have tube hangers of sufficient strength to safely resist bending stresses. However, free swinging fixtures require special attention, so as not to allow the fixture nor its stem to hit adjacent portions of the building during an earthquake thereby resulting in destruction of the fixture and hazardous falling debris.

No specific design provision appear in building codes for lateral support of interior partitions except that, in general, partitions should be braced at the top to resist a lateral load of 20% of the tributary weight of the partition. This load can be transferred into the ceiling if designed to receive partition loads or braced to the structural framing above.

In the 1971 San Fernando earthquake, plaster ceilings and walls in primary exit stair towers were damaged due to drift in several high-rise structures. The falling of small portions of plaster is somewhat hazardous to occupants during evacuation, but the more serious consequence can be panic in the minds of the layman, who would assume structural failure was occurring. Stair towers are often the stiffest elements in a building and yet on many occasions experience great damage. Stair towers are absolutely vital for evacuation and for access of rescue teams. The towers must be designed to remain intact.

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Mechanical and Electrical Equipment and Elevators

This subject is mentioned here only to point out that it is a very major item of "non-structural" hazard in a building. This subject is presented in a separate Paper which, no doubt, will include the importance of piping, air conditioning equipment and ductwork support and anchorage. Elevators received considerable damage in the San Fernando earthquake and considerable attention is needed to mitigate this hazard.

Studies are now underway by manufacturers and Code agencies to correct deficiencies which were experienced in the earthquake. Counterweight security and guide rail alignment are particularly important factors.

IV. Summary of Recommendations to Further Implement Building Hazard Mitigation

Successful design of a project, specifically with respect to hazard mitigation virtually demands a close working relation between the Architect and the Consultant. Architect and Engineer associations through joint action should encourage closer working relations particularly in the early stages of project planning.

Starting with the architectural student, schools of architecture should be encouraged to develop courses in all types of major and even minor hazard mitigation, even though design and planning capability is very important.

By expanded public relations the Architect and Engineer should inform the public of inherent hazards to buildings in seismically active regions and of the importance of accepting less spectacular design concepts for projects in these localities. In addition to safe major building design, professionals should develop a program of general education of the public on minor but disturbing earthquake hazards in the home or office. Anchor tall objects to the wall and improve closet and cupboard door hardware to prevent the falling out onto the floor of glassware and medical supplies that very often result in foot hazards or injury.

As more scientific research is successfully translated into practical application, the Engineer-Consultant should produce techniques that will develop better and perhaps more economical designs for exterior facades of buildings. Development of special materials and proprietary items of manufacturers should be encouraged by the design professionals. More intensive programs of fenestration design for earthquake and, more importantly, wind resistance is needed.

Greater emphasis must be placed on the importance of geologic study of building sites as it relates to ground motion to supplement the more conventional soils investigation which is now aimed primarily at foundation design. The Architect and Engineer must develop a closer relation with the Geologist so that they will better understand the needs of the design professional.

Architect and Engineer code committee activities at present are mainly confined to reviewing proposed building ordinances and are usually defensive in nature. Instead, Architect and Engineer associations should work diligently to find methods of financing, including public funding, for applied research and technology related to all phases of building hazard mitigation. These, and continuing studies should be carried out as expeditiously as possible so that building regulatory agencies, who are responsible for building code changes, can be assisted and influenced by the design profession. This important activity is necessary to minimize the "panic factor" that results from public and political pressure on building departments to take drastic action immediately following a natural disaster. These quick, violent moves tend to produce ultra-conservative requirements that have a great impact on building construction cost.

Although residential buildings receive little professional structural design attention, they represent a large segment of construction dollar volume and house a large segment of the population. The San Fernando earthquake created extensive damage to residences. Better arbitrary standards of design in building codes should be provided, paying particular attention to split-level homes, need for shear elements in the main building and garages, and design and installation of large glazed wall openings.

In conclusion, the Architect as the prime design professional, along with his Consulting Engineers, must face building design and hazard mitigation in perhaps a two-fold attitude. First, buildings must be designed to protect all occupants from injury or loss of life. This philosophy is obviously mandatory and fundamental. Second, design approaches must include a favorable balance between property or physical damage to a structure with respect to initial cost. Even remote structural damage may be tolerated in extreme disasters if occupant hazard is nil. The profession should promote the increase in awareness by the public investor or owner of this situation.

V. References

1. Section 15002 of California State Education Code. Revised July 11, 1972 by Senate Bill 689.

2. Los Angeles County Ordinance 10037. Los Angeles County Code Sections 308 & 309. City of Hayward, California, Code 306. California State Code, Article 5 (65300).

3. Proposal of Woodward-McNeill to Los Angeles County for Geologic and Seismic Considerations for Land-Use Planning.

4. Design and Construction of Ceiling Suspension Systems. Office of Architecture & Construction, State of California.

5. Rule of General Application RGA 3-67 Department of Building and Safety, City of Los Angeles.

6. Recommended Standards for Seismic Restraint of Direct Hung Suspended Ceiling Assemblies, Prepared by Ceilings & Interior Systems Contractors Association.

7. Ad Hoc Committee on Direction Study--Seismic, Structural Engineers Association of California.

National Bureau of Standards Building Science Series 46, Building Practices for Disaster Mitigation, Proceedings of a Workshop Sponsored by the National Science Foundation and the National Bureau of Standards, August 28-September 1, 1972, Boulder, Colorado (Issued February 1973).

PROCEDURES AND CRITERIA FOR EARTHQUAKE RESISTANT DESIGN

by

C. W. Pinkham*

1. Introduction

Purpose of Paper

This paper intends to (a) assemble the current status of procedures and criteria for earthquake-resistant design of buildings, (b) indicate the direction the procedures and criteria should take to improve the reduction of earthquake hazards to buildings, and (c) establish priorities for future research.

History of Structural Design Criteria

The development of codes governing the design of earthquake resistant buildings in the United States is synonymous with the development of such codes in California, even though experience of severely damaging earthquakes extends over many areas of the Nation. For example, the three earthquakes centered near New Madrid, Missouri, in 1811 and 1812; the earthquake of 1886 centered near Charleston, South Carolina; and, the earthquake centered in Prince William Sound, Alaska in 1964.

In the Commentary of their publication "Recommended Lateral Force Requirements and Commentary," the Structural Engineers Association of California (SEAOC) [1] provided a short history of the development of earthquake codes in California. It is reprinted here so that current provisions may be viewed in their historical context:

> "San Francisco was rebuilt after the earthquake and fire of 1906 under a code which provided 30 psf wind force to effect both wind and earthquake resistance. In the years that followed, leading structural engineers employed the concept of lateral earthquake forces proportional to masses, but this simple Newtonian concept did not find its way into codes until 1927 in the Uniform Building Code and more extensively in 1933, following the Long Beach Earthquake.
>
> The real impetus for extensive earthquake studies and research resulted from the damaging effects of the 1925 Santa Barbara Earthquake. At that time, the U.S. Coast and Geodetic Survey was directed to make studies in the field of seismology. The work of U.S.C. & G.S. in this field is continuing and is recognized worldwide. Shortly before the Long Beach Earthquake of 1933, the first strong-motion seismograph was developed. Although the range of the instrument was exceeded, the essential acceleration record of that earthquake was obtained.
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> In 1927, the Uniform Building Code contained a section on earthquake provisions in the appendix with the following preamble:
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> > "The following provisions are suggested for inclusion in the Code of cities located within an area subject to

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earthquake shocks. The design of buildings for earthquake shocks is a moot question but the following provisions will provide adequate additional strength when applied in the design of buildings or structures."

In 1928, the California State Chamber of Commerce recognized the need for a building code which was "dedicated to the safeguarding of buildings against earthquake disaster." Studies under this sponsorship included work by many of the leading structural engineers of the State and resulted in a report which, while not adopted, formed the foundation of codes that followed.

In March, 1933, the Long Beach earthquake destroyed many buildings in that area of the State including many public school buildings. If the shocks had occurred during school hours, the loss of life among the school children would have been appalling. Realizing that much of the loss and damage could be avoided if the buildings had been properly constructed, the State Legislature adopted the Field Act that assigned to the Division of Architecture of the State Department of Public Works, the authority and responsibility, under the police power of the State, to pass upon and approve or reject plans and specifications and to supervise the construction of all public school buildings. Appendix A of the Rules and Regulations adopted by the Division of Architecture required masonry buildings without frames to be designed for a lateral force of 10% of dead load plus a portion of live load. Other buildings were to be designed for 2% to 5%. The lower coefficients were related to higher allowable foundation loads. In 1937 the requirements were revised to make the coefficients 6% to 10% for buildings 3 stories or less in height, or buildings without a moment-resistant frame. Buildings more than 3 stories in height with a complete moment-resisting frame had coefficients of 2% to 6% provided the frame could resist 2% of the load. Again, the range of values were related to the allowable soil pressures. In 1941, the coefficients used were 6% to 10% depending only upon the type of foundation materials. Since 1953, the coefficient used is based upon the equation (60 over N+4.5) where N is the number of stories above the story under consideration. The present requirements are given in Title 24 California Administrative Code.

In 1933, the Riley Act adopted by the California State Legislature required all buildings except certain type dwellings and farm buildings, to be designed for a lateral force 2% of the vertical design load. In 1953, this requirement was revised to require 3% for buildings less than 40 feet in height and 2% for those over 40 feet in height.

In 1933, the Los Angeles Building Ordinance required a coefficient of 8% of dead load plus half live load. This was also required in the Uniform Building Code of 1935 on soils good for 2000 psf or more in areas of highest seismicity (Zone 3), with double this value for weaker soils.

In 1943, the City of Los Angeles recognized indirectly the influence of flexibility on the earthquake design coefficients, making the coefficient a function of the number of stories in the building by the formula

$$C = \frac{60}{N + 4.5}$$

where C = coefficient % of dead load
N = number of stories above the story under consideration. The maximum number of stories was thirteen.

In 1959, when the height limit was removed from the Los Angeles Code, this formula was modified to read

$$C = \frac{4.6S}{N+0.9(S-8)}$$ where S = total number of stories in the building, except S = 13 buildings of 13 stories and less.

It wasn't until 1947 that San Francisco had anything more stringent than the Riley Act in its Code. A table of variable coefficients was adopted, with maximum value for one-story buildings of 8% and minimum value for 30 stories of 3.7%, with variations for soil conditions. These were applied to design vertical loads.

This prompted the formation in 1948 of a Joint Committee on Lateral Forces of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California. This committee, after several years' study recommended a code in which the coefficients were related to the estimated or calculated fundamental period of the structure.

$$C = \frac{K}{T}$$ where K = 0.015 for buildings
= 0.025 for other structures
T = period in seconds

For buildings: Cmax = 0.06 Cmin = 0.02
For other structures: Cmax = 0.10 Cmin = 0.03

These coefficients usually were applied to dead load plus 25 per cent live load, with 50 per cent live load for storage or warehouse areas.

San Francisco, in 1956, adopted a variation of the recommendations of the Joint Committee:

$$C = \frac{K}{T}$$ where K = 0.02 for buildings
= 0.035 for other structures
T = period in seconds

For buildings: Cmax = 0.075 Cmin = 0.035
For other structures: Cmax = 0.10 Cmin = 0.04 "

In 1957, SEAOC formed a Statewide committee to develop a seismic code that would be acceptable to structural engineers throughout the State. The committee's final report in 1959 has become the basis for current codes most frequently used by governing agencies. The base shear of the structure was determined by the formula V = KCW. The term Z was added by the International Conference of Building Officials (ICBO) to the Uniform Building Code (UBC) to account for seismic zoning. The factor C was specified as equal to $\frac{0.05}{\sqrt[3]{T}}$ with a maximum value of 0.10. K was introduced to account for the fact that, historically, certain types of building systems had shown better performance than others. For buildings, the value of K varied from 0.67 to 1.33. Specific requirements and limitations were also specified for individual portions of buildings and for use of different construction materials.

From 1960 to the present, most of the changes to the SEAOC Code have been directed toward specific requirements necessary to accomplish the degree of ductility deemed appropriate for earthquake-resistant design.

Codes in many foreign nations have formed in general the provisions of the SEAOC Code. Others have developed independently using different concepts.

Comprehensive listings of world seismic codes have been compiled [2, 3].

II. Status of Design Criteria and Procedures

Production of Earthquake-Resistant Buildings

Structural design is only one element of the processes necessary to produce a building with adequate earthquake resistance. Each element is vital; neglect of any one may produce a building with insufficient resistance. The total effort should emcompass these general categories of elements:

- Knowledge of site seismicity hazards. (See review articles by Donovan and Wiggins et. al.)
- Allocation of required space and architectural requirements in shaping the building. (See review article by Hillman and Mann)
- Compatibility of architectural layout and structural concept. (See review article by Hillman and Mann)
- Choice of structural options.
- Structural design procedures and criteria. (Principal emphasis of this paper, see also review articles by Donovan, Bresler, and Sharpe et al.)
- Compatibility of structural design with non-structural elements. (See review articles by Hillman and Mann, Ayers and Sun)
- Structural contract documents.
- Selection and organization of the construction team.
- Quality control, supervision and inspection.
- Maintenance after construction

The degree to which each element can be achieved is dependent, in part, on the importance of the building to the owner or to society, and on the consequences of failure. Importance of the building will be reflected in the amount of money available to perform each of the elements. For most buildings, funds needed to perform all currently feasible design procedures are not available, and approximations in analysis and the sound application of good judgment by the designer are substituted as alternatives to a comparatively expensive, rigorous analysis. For some buildings, such as nuclear reactors, each of the elements listed must be performed with the best possible proficiency.

Knowledge of Site Seismicity Hazards

The review articles by Donovan and Wiggins et. al. cover this element in detail. It should be kept in mind that the need for detailed knowledge of site seismicity may not be required for all building types. In many cases, design decisions can be made adequately from a cursory examination of site seismicity. Broad area zoning with appropriate modifications of design criteria is still a valid design approach.

Space Allocation, Architectural-Structural Compatibility and Choice of Structural Options

The early conceptual-design decisions on space allocation and architectural layout are especially important, particularly as they affect the choice of structural systems. These elements are covered in detail by Hillman and Mann; they are mentioned only to emphasize their importance in the total effort. When the structural engineer studies various options for the structural resisting system in a proposed or established architectural layout, cost is usually the determining factor. It has been demonstrated that in instances where seismic resistance is of primary concern, particularly if inelastic deformations are anticipated, the choice of a structural system that is only slightly more expensive will provide a much higher degree of seismic resistance and a substantially reduced damage potential [4, 5]. The design engineer frequently finds it difficult to justify the added cost, especially when the governing building code does not consider these potential benefits in its deterministic format, a format in which loads and member-resistances are specified - usually on the basis of past experience and judgment. Current building codes, in an oversimplified manner, attempt to get better structural systems by assigning different modifying factors to the force function based primarily on the observed behavior of each system-type during earthquake motions [1].

Structural Design Procedures and Criteria

This paper will discuss structural design procedures and criteria more completely. The influence of design criteria in the total effort of producing earthquake-resistant buildings, however, should be kept in mind. It is no more important than the other elements.

Structural Compatibility with Non-Structural Elements

The importance of the behavior of a building's non-structural elements with respect to life, safety, and cost varies with the type of building and with the function of the non-structural element. Provisions should be made, for example, that elevators will protect the life of cab occupants, that exitways will provide for evacuation of the building after major shaking and, certainly, that facades will not collapse. Most economic losses in earthquakes have involved non-structural elements in and on buildings. In most cases, the design of the structural frame can assist in providing seismic resistance for non-structural elements. (Detailed discussion in the review articles by Hillman and Mann, Ayers and Sun)

Structural Contract Documents

Proper presentation of the design concept on structural drawings and in specifications is an important element of earthquake-resistant design. Unclear or confusing items in the contract documents lead to errors and mistakes in the contractor's performance and thus a building with reduced earthquake resistance. Drawings should be as complete and easy to read and follow as possible. A complex specification calling for unusual materials or unusual performance by the contractor may result in poorer construction than would a less rigid specification permitting the contractor to act in his customary manner.

Selection and Organization of the Construction Team

The assembly of an organized construction team is an equally important element. This implies cooperation among the general contractor, his subcontractors, the architect or prime designer and his consultants. With a diligent contractor, quick solution of problems will improve the flow of work. It has often been noted that if a major item on a job goes awry, many subsequent errors occur.

Quality Control, Supervision and Inspection

The effort by the construction team to ensure that the work conforms to the design concept and specifications is an essential element. Most important are the quality control measures set up by the contractor and his subs. The duties and limitations of all segments of the design team (the architect usually has the major role in defining these assignments) should be clearly understood by all parties. Periodic observation by the design team and/or close inspection by assigned inspectors or building officials should in no way detract from the builder's quality control measures and should serve only as an independent check to verify the results of the contractor's efforts [6]. The effectiveness of compliance with the plans, specifications and building regulations has been historically demonstrated. Also, in general, the degree of compliance has varied with the effectiveness of the independent checks on the contractor's quality control.

Maintenance After Construction

Though it may seem obvious, the necessity of maintaining the integrity of the structural system of a building after completion is frequently overlooked. Removal of a brace member, inappropriate placement of a new opening in a shear wall, and other alterations to a building are frequently in the inspection of buildings damaged by earthquakes. Such alterations, when made without proper engineering considerations, can seriously impair the building's seismic resistance.

To Summarize: The subject of this paper, structural design procedures and criteria, is essential, but successful achievement of all the elements outlined is necessary to produce a building capable of adequate earthquake resistance.

Trends in Specifying Design Criteria in Building Codes

Code Design Philosophy

Basic building design philosophy states or implies that little structural damage should occur during moderate intensity earthquake ground motion but that some damage could occur to other elements in the building. It is understood that for very high intensity earthquake ground motion some structural damage could occur but that the possibility of structural collapse should be minimal. Since the San Fernando, California, Earthquake in 1971, the further requirement has been proposed that buildings housing emergency services critically needed by the public for post-disaster functions be designed, detailed, and constructed to remain usable and operable after very intense ground shaking [7].

Design Factors of Safety

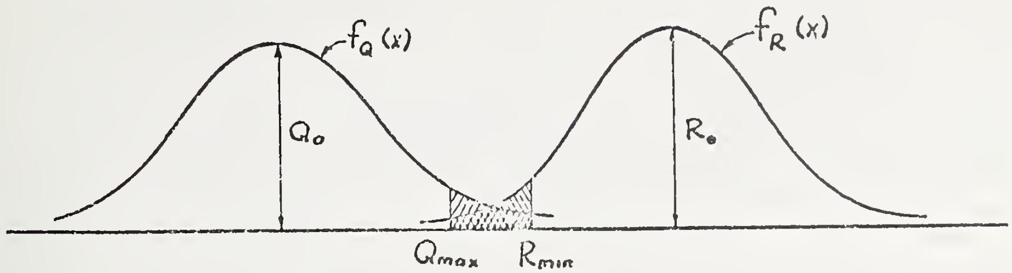
Current design practices uses deterministic analyses - analyses based on an evaluation of the effects of specified loads (some maximum, some minimum, some mean) on the structural system as compared to a set of specified working capacities. The relationship between the specified load effect and the specified resistance contains a factor of safety, which encompasses provisions for the many uncertainties that are seldom isolated in the design process. This design procedure was prevalent during the development of seismic provisions when criteria for seismic design were adapted from experience using what has become known as "working stress design methods." One of the uncertainties contained within the factor of safety was the reliability of predicting not only the load effects but also the resistance capabilities. With increasing refinement in determining both sides of the design equation - particularly since the advent of the computer - the actual factor of safety, in general, has been reduced. This has resulted from "precise" stress analyses on the one hand and more accurate determination of maximum stress capacities on the other, but with no change in the specified design seismic forces, which were never intended to relate directly to observed maximum ground motions.

Originally, seismic design criteria were merely design procedures used to develop a somewhat consistent sizing of elements. Because the design process did not acknowledge some of the inherent seismic resistance, except in an implicit manner, seismic resistance depended not so much on the size of elements but on the detail and arrangement of the component parts and their fastenings. Some of this implicit design included items as described by John Blume in his discussion of the Reserve Energy Technique [8]. Much of the earthquake resistance provided in buildings has not been due to the solution of the design equation (force-less than-or-equal to-resisting stress) but due rather to the details of design which were not covered by specific criteria. This is still true today if the design model does not match the actual building. The taller, fairly regular buildings can be mathematically modeled with a reasonable degree of accuracy; most buildings, however, have resisting systems for which realistic models are very difficult to develop. In addition, the placing of non-structural elements in the building often has a profound effect on modifying the assumed model.

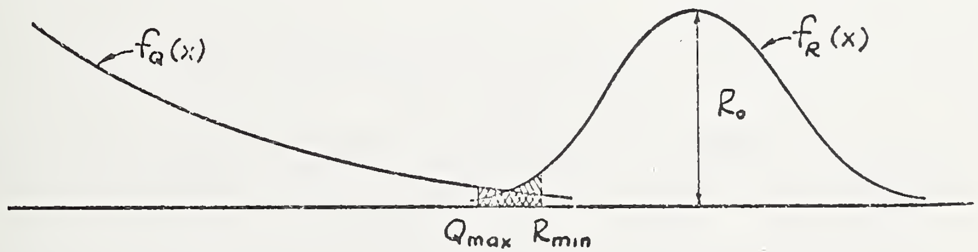
Current code-specified forces, anticipating working stress design procedures, have not been specifically related to the response generated by an earthquake. The specified forces were determined as the envelope of the response of a uniform, multi-degree of freedom (MDOF) system. The code forces were intended to provide elements reasonably sized, consistent with past practices, to resist earthquakes, and to require the designer to create the details necessary to provide adequate ductility for the building.

Concurrent with the development of the present code-prescribed minimum design forces, the codification of load factor design principles in "ultimate strength design" (USD) in concrete, and in plastic design in steel were being presented. In some instances, particularly in concrete columns, the "factor of safety" was effectively and substantially reduced. The concept of load factor design requires that a factored specified load effect (UQ) shall be determined to be equal or less than the minimum specified elastically determined resistance ($\emptyset R$). For use in this paper it is assumed that Q and R are specified deterministic load and resistance factors, and that U and $\emptyset$ are factors to modify Q and R to give the maximum load and the minimum resistance effects. This concept is the first instance of a commonly used system in which "factors of safety" are specified. Thus, the specified factor of safety would be $\frac{U}{\emptyset}$. The trend which this concept portends is important in better understanding the ultimate behavior of materials and in designing for earthquake resistance. Due to the change from working stress, however, a close look should be given to the $UQ<\emptyset R$ relationship so that the specified loads will give the desired "factor of safety."

The safety factor is a much-studied concept, particularly as it applies to the usual dead and live loads. Simplified probability-based structural codes have been proposed [9 through 17]. These could be extended to include the concepts of elastic and inelastic response of buildings to earthquakes and it is felt that this should be the direction in which the development of new design criteria should advance. Extensive development work and education of users is needed, however, before general acceptance of the proposals can be expected. The concepts of probability are useful in describing some aspects of the earthquake design problem. For vertical loading, the following curves can be drawn [9]:



These curves represent probability density functions. They are indicators of the probabilities of occurrence at various levels "x" of load, $f_Q(x)$, and of resistance $f_R(x)$. The area under each curve is equal to unity or 100%. The $f_Q(x)$ curve represents approximately the distribution for dead or live loads. Usually, $f_R(x)$ represents the ultimate strength of the resisting element. A safety factor has been assigned in several ways. One termed "central safety factor," is defined (R_o over Q_o). If the specified loads are considered as a maximum Qmax (usually dead loads are mean loads) and the specified resistance of the material is considered a minimum, the specified factors of safety would be (Rmin over Qmax). In the case of concrete USD, this latter ratio theoretically is 1, if Qmax = UQ_o and Rmin = $\emptyset R_o$. In the case of earthquake loading, the curves are more like the following:



Assuming the design equation of Qmax < Rmin is still desired relationship, it becomes important to choose the level of Qmax to be used in design. The "central safety factor (R_o over Q_o) has no meaning in this case. The problem of assigning design criteria and procedures can be stated as the selection of the appropriate Qmax to be associated with Rmin. The following discussion summarizes some of the items of consideration in the selection of Qmax and Rmin for earthquake design.

Ground Motion

Many studies have dealt with the problem of determining the anticipated ground motion at a site [18, 19, 20, 21]. The subject is discussed in detail in review articles by Donovan, Algermissen, and Wiggins, et. al. Both the probability of damage (risk) and the probability of survival have been used as parameters for design decision [22].

Response Spectra and Modal Analysis

A detailed description of dynamic analyses of buildings is given in review articles by Newmark and Hall, Sharpe et. al.

A response spectrum is a plot of the maximum responses of different oscillators with varying fundamental periods to a given input motion [23]. The plot can relate acceleration, velocity, or deflection response to the fundamental period. Response spectra can be developed for many types of vibrators. The one most frequently used is an elastic single degree of freedom (SDOF) vibrator with varying amounts of viscous (proportional to velocity) damping [20, 24, 25]. Similar spectra can be developed for vibrators responding inelastically, for vibrators having other types of damping and for those having multi-degrees of freedom. The inelastic response most frequently used assumes that members of the frame respond elastically (stress proportional to strain, that is) to a specified stress level beyond which strain increases without further increase (or decrease) of stress. The MDOF system could be one having a constant mass-to-stiffness ratio, one deforming primarily in flexure, one deforming primarily in shear, or any specific relationship desired.

SDOF elastic spectra have been most frequently proposed for use in design. Smoothed spectra are drawn which either envelope the individual spectrum determined from a number of earthquakes, or a spectrum is chosen which would give the same response to the average MDOF system. The information necessary to make an accurate choice of the appropriate damping factor for the building system is not available, particularly if the effects of interaction between the structure and the foundation media (both rocking and translational interaction) are assumed to be an equivalent amount of damping. As a design procedure, however, an accurate prediction is not always required. The analysis using the SDOF elastic spectra is accomplished by performing the following operations:

- Determine the elastic mathematical model that best represents the building to be built.
- Determine the period (frequency) and the mode shape of the first few modes of vibration of the assumed model.
- Determine the modal participation factors which are factors describing the portion of the total mass of the building which is associated with each mode.
- Determine the base shear and the distribution of shears throughout the structure for each mode using the specified SDOF spectral value with an appropriate choice of damping ratio.
- Combine the responses of the various modes by a suitable means. Combinations most frequently used are the square root of the sum of the squares or the straight addition of modal effects.

The complex portions of analyses of this type involve the assumption of the appropriate model and the determination of the mode shapes, although approximations can be made. The relevancy of the results, however, is only as good as the validity of the initial assumptions. The placing of non-structural elements often will modify the assumed model and thus completely alter the building response. Added elements may require multiple analyses if the effect on the mathematical model is severe enough.

One difficulty presented by an analysis using spectra is that reference to an actual earthquake event is usually a maximum elastic response. The time or the number of high amplitude cycles is not specified. Thus, for a given period, the same spectral value could be describing a single cycle of motion of a small earthquake or the maximum of many large-amplitude cycles of a large earthquake. If all response were fully elastic, the number of cycles would have no significance. Considering the complex structures used for buildings, however, it is doubtful that true elastic behavior occurs even though this assumption is made for the design model. The number of cycles of significantly high accelerations would make a difference in the extent of damage a building would undergo should inelastic deformations occur.

As a design method, the analysis by spectral modal analysis can be considered a static load approach as time (an indicator of duration of load and motion) is not a variable. This is so even though the analysis provides the same maximum response in each mode as would a time-dependent dynamic analysis. The MDOF spectra currently used as the basis of design are usually considered as static forces applied to the building. It has converted the dynamic problem to a solution which uses the same procedures as for other loading conditions. The dynamic nature of the problem, however, should never be overlooked by the designer.

It should be remembered that MDOF response spectra, similar to those in current codes, are based on the assumption of uniform mass-to-stiffness ratios. Designers who follow the strict wording of the "Code" tend to overlook this. For a building with non-uniform ratios at various levels, the base shear and distribution of shear should be modified to an MDOF spectrum suitable to the particular building.

Time-History Response Analysis

The full dynamic analysis can be accomplished by use of all or a portion of the motion record of an actual or assumed earthquake [20, 25]. The reliability of predicting the response of a structure is still dependent on the choice of the appropriate mathematical model, including the damping value. The earthquake motions in suitably small increments of time are used and the sequential responses of the assumed model to these motions are determined based on the equations of motion. Approximate or assumed inelastic as well as elastic behavior can be modeled. Usually the ground motion used in the analysis is independently determined at the base of the structure. The building model may or may not account for the properties of the supporting foundation material. By use of finite element analysis, the motion at the base rock can be assigned and the vibrator model composed of the supporting soils and the structure can be used. This technique can also be used in three dimensional analysis. The cost of analysis mounts, however, as additional variables are added to the equations of motion. Fully elastic response is usually used in this type of analysis and the model is somewhat simplified to reduce the cost of analysis. The relevancy of the results depends on how well the analytical model describes the actual building. The choice of the time-history record to be used can be that of an actual record, a predetermined or modified "standard" earthquake, or an earthquake generated by use of filtered "white noise" techniques to produce a certain result on a standard vibrator or one to match a chosen SDOF spectrum. Design criteria should not be devised so that the use of the time-history dynamic approach is denied, particularly for those buildings with longer fundamental periods. Due to the uncertainties of model- and earthquake-selection, however, a time-history analysis does not necessarily provide the engineer with a better design than that obtained using other methods. The choice of the appropriate model for shorter period buildings is at the present time very difficult.

Consideration of Inelastic Response in Design

The cost of including specific inelastic motions in response calculations is usually too great relative to the value of the information obtained. In order to simplify the problem, it can be assumed that inelastic response follows a bilinear path. One such method is commonly referred to as the elasto-plastic load-deformation curve from which plastic design concepts could be applied. Even the cost of performing a bilinear analysis usually is not warranted except for some occupancies. Another approach has been to perform a fully elastic response analysis with the assumption that maximum deformations will be the same as with elasto-plastic response, an assumption that has been shown in some cases to produce incorrect results [26]. This approach does not necessarily reveal local instability effects but at least provides the engineer with probable locations in the structure which should receive closer scrutiny and gives him valuable information for making design decisions.

In most design situations, inelastic analyses involving time-history responses have not been performed. It is assumed that the ductile behavior of the framing system will adequately support inelastic response. This ductile response is accomplished by providing adequately sized and spaced reinforcement in reinforced concrete construction and by appropriate selection of type, shape, and connection in structural steel construction. In smaller buildings, the use of properly detailed wood framing generally has exhibited adequate ductile response, provided that proper consideration has been given to connections and control of construction quality. Combinations of construction methods can also be used and minimum code provisions have been developed for these items. Additional collateral material that can absorb energy as it deforms also provides inelastic response capacity. This material should be detailed so that it does not become a hazard. Lacking "ductile" capability, the elements should be designed to be able to respond essentially in an elastic manner to design earthquakes.

Limiting Conditions (Limit States) of Design Procedures

In designing for seismic resistance, two limiting conditions (limit states) should be considered. A lower limit is the design elastic response of the structural model (elastic limit, $Qmax < Rmin$). A maximum limit is the response to an earthquake which is determined to have the maximum intensity of ground motion critical to the particular structure under consideration. For this latter case, Rmin in the equation $Qmax < Rmin$ is not necessarily the ultimate strength. In geographical areas where it is determined that the potential of high earthquake motion warrants consideration, these two limits are usually assumed separately. Where the potential for high earthquake motion is determined to be not great enough for consideration, one limit may suffice. Methods have been established wherein the probability of damage (risk) or the probability of survival have been associated with the limit of elastic response. Where the maximum limit is equal to or closely associated with the elastic limit, the establishment of the relative risks involved can easily be a parameter that can be introduced into the design equation. The effectiveness of this approach depends on the accuracy of the prediction of potential earthquake motions. This is covered in detail in review articles by Donovan and Algermissen. As this date, however, the state-of-the-art of prediction of ground motion, particularly near the fault trace, has not produced procedures which have gained general design acceptance.

Another condition to be considered in design is a limitation on deformations. This limit could be based on any loading condition to serve as a guide to minimize pounding between units, as a check on building stability, as protection of non-structural elements, and for maintaining serviceability of the building. In order to provide the most visible indication of behavior, the "maximum limit" ground motion should be used. Lesser ground motion with appropriate limits, however, could be used if by so doing the analysis is simplified.

Arbitrary Design Criteria

The use of performance criteria for seismic design would greatly simplify the code language. The application of such performance criteria in practice, however, could result in great misuse of materials due to the current lack of basic understanding of the resistance of building components to dynamic loading, particularly response to the higher frequency vibrations found in earthquake motions and inelastic response. Also, chances would be greater for misinterpretations by the design profession, the regulating profession, the regulating agencies, and the building contractors. As a result, some arbitrary limitations have been placed on the use of some materials [1]. In general, these limitations have been to restrict the use of systems to those areas in which either experience or tests have demonstrated their applicability. (See review article by Bresler for current status of testing.) Relaxation of these arbitrary limits should be allowed only as additional substantiating information becomes available.

Considerations for Future Code Criteria

Minimizing Complexity of Design Criteria and Procedures

Future design criteria must be of such a nature that there will be uniform interpretation and application of the intent by design engineers and the regulating agencies. Extremely complex formulation will lead to varied interpretation and varied application. Complex criteria also tend to generate a greater incidence of error in analyses, which offsets many of the benefits from such criteria. Any rules (as separate from methods of analysis chosen by the design engineer) should be easily understood by the practicing engineer. Such rules should not invalidate the use of more complex design procedures if deemed appropriate by the design engineer.

Use of Past Experience

Reviewing the overall earthquake experience, it can be said that designs based on current design procedures have produced buildings with a high level of seismic resistance for a comparatively low level of cost. Most of the damage in recent earthquakes has resulted from shortcomings in building layout, design execution, detailing, or construction quality control. Some of the experience gained by observations of the behavior of buildings, however, must be considered when establishing new criteria. A few instances of damage have indicated that the design criteria used were insufficient. Some of this evidence of insufficient design criteria:

1. All major earthquakes have indicated the poor performance of unreinforced masonry. The problem of the eradication of existing unreinforced buildings in earthquake-prone areas has the greatest potential impact on society. We either eliminate the potential hazard at tremendous cost or we accept the risk of a great loss of life. The most effective way to reduce the total hazard problem in earthquakes is to solve the problem of old buildings. See review article by McClure. Examples: 1971 Sylmar VA Hospital; 1933 Compton, Long Beach; 1952 Kern County.

2. Recent earthquakes have indicated that the axial stresses in columns within a structure resulting from overturning moments are determined by the actual stiffnesses and capacities of the building rather than by the specified design forces acting on an assumed resisting system. This was indicated particularly in the Caracas Earthquake of 1967 [27].

3. The need to provide ductility and adequate shear capacity to concrete columns was demonstrated by the San Fernando Earthquake of 1971 [1, 28, 29, 30].

4. Also, the San Fernando Earthquake demonstrated the need to have a new philosophy of design with respect to structures required for post-earthquake recovery.

5. The need for a comprehensive review of design levels as related to buildings with medium fundamental periods (say 0.25 to 1.5 sec.) was indicated by the Santa Rosa and San Fernando earthquakes [28, 29, 30, 31].

Soil Factors in Design Criteria

Most U.S. building codes do not contain specific soil factors in design criteria. The differences in the elastic response of structures (on varying soil structures) has been recognized for some time, but a specific way to provide for soil effects in a code has not gained general acceptance [21, 32]. This is particularly true when considering the inelastic response of buildings. Where earthquakes of moderate magnitude are associated with many faults in an area, the determination of the potential shaking and the effects of soil on response becomes very uncertain. Reasonable modifications of the design equation to account for the change in potential shaking with varying soil conditions should be considered.

III. Status of Applied Research

Reconciliation of Data from Earthquake Records

During the 1971 San Fernando Earthquake, more than 200 usable accelerograph records were obtained. Many of these were obtained in taller buildings because the Los Angeles City Building Code requires three instruments in high-rise buildings - one at the roof, one at the lowest level, and one at an intermediate level. Most of the instrumented buildings had a response to the earthquake shaking which was primarily elastic. Two, at least, indicated definite structural inelastic response. Analyses have been made on a number of these instrumented buildings [23, 27, 28, 29]. The general conclusion from these studies is that buildings having long fundamental periods (say 1.0 sec. or more) can be reasonably modeled for determining response to earthquake motions. The amount of damping required for reconcilation varied, but a reasonable value could be selected for use in design. Whether damping values should be itemized in a building code is a matter for consideration.

Research on Materials of Construction

A detailed report on the status of the resistance of materials of construction is given in the review article by Bresler. A few items of overall interest to the development of design procedures will be discussed here.

Most testing of materials for earthquake resistance has used either static loading or loading at a very slow rate. Also, most testing has been done without reversal of loading. The tests do give a reasonable representation of the behavior of materials in the case of taller buildings, however, where the fundamental period is long (as mentioned before, on the order of 1.0 or more). For shorter period buildings, the relationship between static testing and dynamic response may be quite different. For instance, in blast design, the effect has been accounted for by an increase in the compressive capability of concrete. The effect of fairly fast loading on the buckling characteristics of steel shapes is not well defined nor has the effect on shearing strength of concrete been adequately demonstrated.

Until further research provides data necessary to answer the questions regarding short-period loading, reasonable assumptions must be made based on the actual experience of past earthquakes and on the results of the available static tests.

Each type of construction material responds to earthquake shaking differently. This is particularly true when inelastic response is considered. Thus, for concrete the problem of "non-ductile" compressive and shear failures is of major concern. Local, lateral, torsional, or shear buckling is the special concern on the inelastic behavior of structural steel elements. The special concern in cold-rolled sheet steel diaphragms is the development of adequate fasteners of individual panel units and the development of diaphragm chords and struts. Similarly, for wood diaphragms, the principle concern is the development of struts and chords and, as indicated by the 1971 San Fernando Earthquake, the development of adequate lateral support of adjoining walls. The special concern for masonry and pre-cast construction is the development of adequate connection between individual elements. Joinery is of concern for each material but the development of adequate joint details is distinct for each material. The properties and details of construction using each material require that separate studies be made to determine the special requirements to establish design criteria. Until this is done, consistent use of limit strength procedures cannot be performed by the design engineer.

Research on the Effect of Foundation Materials

In the past, some research has been done on reconciling the effect of foundation materials on building response. Much more needs to be done. The data from the San Fernando Earthquake should be analyzed to determine if current theories are applicable. There is some indication they are not.

Research on the Design Earthquake

It is now possible to collect data currently available to arrive at design criteria applicable to buildings with longer fundamental periods. The development should be done using probabilistic concepts which could be modified to deterministic terms for code use. Additional basic research is required before such an approach can be undertaken on buildings having short periods, and until such research is completed, criteria for these buildings should be based on current experience in the damage surveys of previous earthquakes. The interface between these two approaches should be such that no large discontinuity exists between the products of the two design approaches. This task is one that should be started as soon as possible so that currently available expertise can be made useful.

IV. Research Priorities

Current Goals and Priorities in Applied Research

In the past, the criteria for earthquake design in the United States have been established primarily by volunteer committee effort by members of the four associations comprising the Structural Engineers Association of California. The task of assembling the criteria into legal code language and presenting it in a form suitable for practical application by design engineers has become too great for volunteer effort alone, especially considering the speed of change being asked by political and administrative bodies. A new approach has been launched, therefore, with the intent of bringing together the expertise of the practicing engineer and the data from the specialists on seismicity, risk and dynamics. The new approach is through an SEAOC-sponsored corporation, Applied Technology Council, to organize research and assemble data for new design criteria under the guidance and supervision of practicing engineers so that the criteria will be workable and practical.

A broad outline for this work is contained in a position paper prepared by the Seismology Committee of SEAOC [36]. This outline provides the background for a list of items to be followed in developing revised seismic design provisions to reflect current knowledge and experience. This work should be the first priority for the tasks ahead.

1. The attainable goals of the seismic code should be clearly defined in the areas of:

   - Structural damage.
   - Non-structural damage.
   - Post-earthquake functions.
   - Human risk.

   Comment: Current commentaries on seismic codes do not specifically outline the various damage levels expected to be obtained by application of the provisions even though the basic design philosophy has been expressed. After arriving at the decisions on later items in this list, these specific areas of concern should be delineated so that all concerned can properly assess the risks involved.

2. Basic provisions of the seismic code should contain:

   - Equivalent static forces for most structures.
   - Dynamic and inelastic analyses for others which would be
     a. Required in certain cases.
     b. Optional for all structures.
   - Simple factors for low buildings.

   Comment: The first and third items are the design approaches currently in use in seismic codes. The experience to date has indicated that when designing structures which are applicable, the concepts need not change. The equivalent static force approach (MDOF spectra) currently in use for uniform buildings is recommended with some possible modification in quantities. For one- and two-story buildings constructed with a reasonable amount of inherent damping, the criteria should remain as a specified per cent of gravity. This could vary, however, depending on site location and conditions and on importance factors.

   The new added item is codification of the requirements for dynamic analyses. Specific inelastic analyses should be required only on structures for which the probability of failure is to be reduced to a minimum. The types of structures to be included in this category have been the subject of frequent discussions. Usually the only types considered are those housing nuclear reactors. Emergency communication centers might also be included. However, consideration for inelastic behavior should be provided in all classes of structures.

3. Seismic codes should include provisions for determining:

   a. Basic realistic levels of ground motion to represent a design earthquake at a site of average exposure having no unusual soil conditions.
   b. Necessary levels of structural systems damping, ductility and stability to survive this ground motion adequately including techniques for handling the P-delta problem.

   Comment: The choice of the appropriate levels to assign to both sides of the design equation $UQ \leqslant \emptyset R$ and the manner of presentation should be made with consideration for all elements of design, including the time-dependent problems of overturning and building stability. Specific numbers for the levels will follow once the basic concept of design is chosen based on experience to date.

4. Tolerable levels of cracking for brittle materials should be established.

   Comment: This level should be reviewed so that criteria for drift limitations may be formulated and so that a better understanding of cost risks may be established in the code.

5. Factors for determination of equivalent static force design should be established.

   a. Determine C factor to apply to a structure of minimal ductility for the realistic response to ground motions outlined in 3.a.
   b. Establish K reduction factors based on the items in 3.b.
   c. Establish S factors for site soil effects.
   d. Establish importance factors I based on human risk and post-earthquake use of emergency facilities (deformation or force factor).
   e. Provide for modification of the zone factor (Z) for force function and details when data on site exposure become available. Review zone factors for areas other than California.

   Comment: The specific factors relating to the equivalent static forces currently in use should be reviewed. There is some indication that in the medium period range (0.25 to 1.5) the relative values of C are not consistent with those in the short and long period ranges. An examination of the currently used force modifying factor K should be undertaken to see if the definitions for the various categories should be changed and to determine if current values are appropriate. The data are available to make a decision on whether or not soil factors should be included in the code or whether they should be left to individual determination for each site. The inclusion of importance factors at this time should be restricted to buildings housing facilities necessary for post-earthquake use and to certain classes of buildings in which probability of failure should be minimized. The current zone factors for broad areas of varying seismicity should be reviewed (i.e. 1, 0.5, 0.25) particularly as they relate to the choice of details of construction.

6. Realistic deterministic ultimate design stresses and load and resistance factors should be established for all materials consistent with the design factors above. Uniformly consistent load factors should be established for various loading conditions which are common for all materials. Variability factors should be established for different materials, combinations of materials and types of construction. Arbitrary limits and details necessary to accomplish these factors also should be established.

   Comment: See Comment on 3.

7. Conventional one-story light frame requirements should be redrafted consistent with requirements for designed structures.

   Comment: The design criteria for conventional one-story light frame construction has been specified by standard minimum details rather than by stress design. This method should be retained but a review of the commonly used arbitrary construction details should be undertaken to make the provisions consistent with the level of safety chosen for other construction.

8. The classes of structures requiring dynamic analyses should be established.

Comment: The choice of those structures to be named in the code to have a specified kind of dynamic analysis should be carefully considered so that the cost of analysis will not be out of line with the risks involved. Currently under consideration by the ICBO is a provision as follows:

> "Buildings or structures which have highly irregular shapes, large differences in lateral resistance or stiffness between different stories or other unusual structural features affecting seismic response shall be designed for forces which their dynamic properties induce."

Also, consideration should be given to requiring modal analysis for buildings having fundamental periods in excess of 1.0 to 1.5 seconds.

9. Specific criteria should be established to be used for dynamic analyses and designs consistent with the criteria used in determining the equivalent static force criteria above but with some reduction in conservatism so as to encourage dynamic design. Both service level and inelastic criteria should be established.

Comment: See Comment on 3.

10. Drift limitations should be established consistent with the realistic response to strong earthquakes. Consider tolerable limits of non-structural materials as well as structural systems, materials and stability. Establish building separation limitations.

Comment: Compatibility with architectural, mechanical, and electrical systems should be considered for establishing limits.

11. Where required, criteria should be established to cover the vertical acceleration problem.

Comment: Existing data on the concurrent behavior of vertical and lateral motions should be reviewed. Provisions should be incorporated in the design code to account for this behavior. This can be done by appropriately modified load factors. Basic study should continue on this subject.

12. Shear wall - frame interaction provisions should be established.

Comment: Special shear wall - frame interaction provisions are required so that the difference in ductility between the two systems may be accounted for in elastic analyses.

The remaining items are independent subjects but are necessary for completing a working code.

13. Criteria should be established for the repair of earthquake-damaged buildings.

Comment: Even though very little information is currently available on the efficacy of repair procedures for buildings damaged by earthquakes, specific provisions for repair should be included so that recovery in a damaged area can be facilitated.

14. Criteria should be established for rehabilitation and upgrading of existing buildings.

    Comment: As mentioned in Section 1 of this paper, the practical solution for a corrective program would go further than any other provision toward reducing the earthquake hazard relative to loss of life.

15. Seismic design requirements for mechanical and electrical equipment should be established.

    Comment: The requirements for these items should not only be the assignment of a lateral force of x per cent of gravity; for some pieces of equipment a more sophisticated analysis will be required. Assignment of the responsibility for design should also be determined.

16. Minimum requirements for quality control should be established.

    Comment: An outline for effective quality control should be given. The varying abilities of communities to perform the checks of the contractor's quality control should be a factor in the level of design just as the ability of a community to combat fires is considered in insurance rates.

17. A commentary should be written and it should be complete and instructive for those not experienced in seismic design.

    Comment: The use of a building code is dependent on a common understanding of the intent of the code provisions. A detailed commentary explaining the intent of the various provisions is necessary.

The foregoing list should remain flexible for revision as the work proceeds.

V. Basic Research Priorities

While the state-of-the-art criteria are being developed, basic research which could contribute to a better understanding of seismic motions and structural response should proceed. Research should be designed to produce information on the following items:

1. Shear walls of all materials to determine

    a. Capacity and ductility, particularly in resisting high frequency motions.
    b. Tolerable strain and/or cracking.

2. Ultimate column capacities and limitations for all materials.

3. Frame connection capacities and limitations for all materials.

4. The characteristics of elastic and inelastic frame instability to earthquake motions.

5. Inelastic capability of braced systems.

6. The capacity and limitations of connection details other than for frame members (all materials)

    a. Tying together of building elements.
    b. Requirements for details on drawings.

7. Damping and energy absorbing characteristics of various materials and assemblies.

8. Efficacy of modeling techniques for various framing systems and materials at various fundamental frequencies.

9. Vertical acceleration effects on buildings and structures.

10. Amount of reserve energy capacity in buildings of various categories.

11. The effect of three dimensional seismic wave motions on large building configurations.

12. Soil effects on ground motion at site.

13. Soil-structure interaction.

14. Soil damping and strength characteristics consistent with the design equation.

15. An extension of the nework of strong motion accelerographs should be implemented. Smaller buildings than those that are currently covered by laws such as exist in Los Angeles City should be instrumented. Servicing of the instruments by the National Oceanic and Atmospheric Administration should be continued. Uniform criteria for instrument location should be developed so that information can be more easily interrelated. Means should be established for rapid data reduction and analysis of earthquake records when they become available.

16. Development of probability-based code provisions.

17. Determination of feasible procedures for upgrading existing hazardous buildings.

18. Implementation of the construction of a large, three-component earthquake simulator [37].

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PROCEDURES AND CRITERIA FOR EARTHQUAKE RESISTANT DESIGN

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I. Introduction

When a building or other structure is subjected to earthquake motion, its base or support moves with the ground. Since this motion is relatively rapid, it causes stresses and deformations throughout the structure. If we neglect temporarily the interaction between the base of the structure and its foundation, when the structure is quite rigid, its motion is nearly the same as that of the ground and the dynamic forces acting on it are very nearly equal to those associated with the ground accelerations applied to the structure as a rigid body. If the structure is quite flexible, large relative motions or strains can be induced in the structure because of the differential movements between the supports and the masses of the structure. In order to survive the earthquake motions the structure must be either strong enough or ductile enough to resist the forces generated by these deformations; the combination of the required strength and ductility is a function of the stiffness or flexibility of the structure.

Seismic effects on a structure, component or element, depend not only on the earthquake motion but also on the properties of the structure, component or element itself. Among these properties, the most important are the energy absorption within it or at interfaces between the item under consideration and its support, either due to damping or inelastic behavior, its period of vibration, and its strength or resistance.

It is the purpose of this paper to describe the general nature of the principles upon which earthquake resistant design is based, and to consider the development of design procedures for the design of structures, facilities or components.

Examples of structures that did not have sufficient strength and ductility to resist the earthquakes to which they were subjected are well known. Failures occurred in the columns and frames of buildings in Caracas, for example, when inadequate strength and energy absorbing capacity were available for the earthquake of 1967. Other failures in earthquakes are clearly due to lack of adequate support details, or lack of adequate continuity between individual elements.

Emphasis is placed herein on design as contrasted with analysis, and essentially on preliminary design or the selection of the general outline, type of framing, and first estimate of requirements. This choice of emphasis is made because methods suitable for such purposes generally can assure adequate performance and serve as a check on designs made by more sophisticated methods.

The general concepts presented in this paper have been adapted from those given in References [1], [2], [3], and [4].

The design of a structure, as either a complete system in itself, or as part of the system of which the structure is only a component, can be a highly complex matter involving a number of input data of various types and a host of special requirements. Once the structure has been dimensioned, that is, laid out in plan and the size and strength of its various elements selected, then the analysis of the structure for given conditions of loading and foundation motion can be made by relatively well understood methods, even though the analysis can be a tedious and lengthy one for a complex system. However, unless the designer uses a so-called "direct design" procedure, he is faced with a problem of the preliminary selection of the structural layout, framing, element strength, and the like, before he has a structure which he can analyze. Even with direct design procedures, for important structures he will want to have some handy approximation that can be used for his preliminary studies.

The steps which the designer must take are generally as follows:

(1) Select the earthquake hazard.

(2) Select the safety factor, or the allowable limits of deformation, or the allowable probability of damage or failure. This may depend on step (3).

(3) Select the type or layout of the structure, and estimate its dynamic (and static) parameters. These include a) dynamic resistance, b) natural frequency or period of vibration, c) damping or energy absorption, d) ductility that can be counted on before failure. These may be assigned in a direct-design procedure, or are subject to successive revision in more traditional procedures.

(4) Verify the adequacy of the structure selected, and make any necessary changes in strength or other parameters, or in the complete layout or plan.

(5) Repeat steps (3) and (4) until a satisfactory design is achieved.

(6) Make a more accurate analysis of the final design, and make further changes that may be necessary. If these are not minor, steps (3) to (6) may need repeating. In some cases revisions in steps (1) and (2) may be desirable. In other cases an upper bound direct procedure may be used involving essentially only steps (1), (2), and (3). Most so-called "static design codes" are intended to be of this type.

II. Earthquake Hazard

Earthquakes are relatively rare occurrences, but in many regions of the world one can count on a high probability of at least a small earthquake occurring once in the lifetime of a building. However, the stronger or more intense the earthquake the smaller is the probability of its occurrence. An earthquake that has a relatively high probability of occurrence is appropriately considered as a loading for which the design must provide in such a way that the cost of the minor repairs required is not excessive. Major strengthening of a structure to resist intense forces is expensive, and the cost of such design provisions must be weighed against the possible cost of repairs in order to design whether the additional design strength or ductility is economically justified.

It is generally agreed that structural collapse of such a nature that it might endanger a great many lives should be prevented by the design, even for the maximum credible earthquake. But it would be unreasonable and uneconomical to provide for resistance to an extreme earthquake with the same factor of safety or margin of safety that one normally uses. The selection of the factor of safety for the maximum credible earthquake is in part dependent on the nature and importance of the structure, and on the consequences should the structure fail.

Unfortunately the earthquake hazard for which designs should be made is subject to a high degree of uncertainty. In only a few areas of the country are there relatively long periods of observation of strong earthquake motions. By correlating the available strong motion records with the more common records available from the sensitive recording instruments used by seismologists, and by use of qualitative reports of the effects of earthquakes where motion records are not available, some measure can be obtained of the maximum intensities which have occurred in various geological regions, and predictions can be made of those which might occur in the future. In other regions of the country, where records are scarcer, estimates of a similar nature can be inferred but are much more uncertain.

However, the maximum historical earthquake determined by such a procedure is not a proper measure of the possible intensity of an earthquake which might occur in the future nor of the earthquake for which the design should provide. At some sites, maximum or extreme earthquakes might never have occurred in the past; it is almost certain that they did not occur within the period of recorded history.

In order to specify adequately the earthquake intensity for either the historical or the extreme earthquake, one must do more than determine the possible or probable acceleration of the ground. The character of the earthquake motions must also be described in a way that is representative of the geologic conditions, taking into account the local soil conditions including overburden depths and characteristics, presence of water, depth to basement rock, and the like. A better measure of the free-field earthquake motions is a description which includes not only the maximum ground acceleration, but also the maximum ground velocity and the maximum ground displacement, with some measure of the number of pulses or the duration of the strong motions that should be considered. All of these quantities are dependent on the geologic and soil characteristics, and are in part dependent on the soil-structure interaction of the structure supported on the soil or rock.

Earthquakes in different parts of the world on different types of foundations and rock or soil strata have greatly different characteristics. The Niigata Earthquake of 1964 was characterized by the phenomenon of liquefaction of the soil and foundation failures causing tilt and overturning of a number of buildings that otherwise would not have been badly damaged. The earthquakes in Mexico City are affected by the natural frequency of the huge bowl of soft jellylike soil which underlies most of the central part of the city, and which emphasizes and amplifies motions in the range of periods of vibration of 2 to 2.5 sec. and diminishes those of very low period, of less than 0.5 sec.

The low buildings, the old churches and the cathedral in Mexico, many of which have been in existence several hundred years during which time Mexico has been subjected to serious shaking from earthquakes, generally have not been severely damaged by earthquakes, but modern tall buildings were seriously damaged, especially in the earthquake of 1957. Nevertheless, one building, construction of which was completed during the early 1950's, the Latino Americana Tower, survived this earthquake and, subsequently, the several slightly less intense earthquakes of the past decade with no damage, not even cracking of window panes. This is due to the fact that the expected nature of the earthquake motions was taken into account in its design. The building is of interest because it was the first building which was designed in accordance with modern analytical methods that was actually subjected to an earthquake approaching the intensity of the earthquake for which it was designed.

With the increased knowledge of the characteristics of earthquakes from the records obtained of strong motions in earthquakes in various parts of the world, we now have the basis for a much more detailed description of the type and intensity of earthquake motions that should be considered for design of structures of various types in various regions, taking into account the geologic conditions as well as the foundation materials under the buildings.

III. General Design Concepts

General Principles

The designer's freedom of choice in selecting methods of resisting earthquake motions is restricted by the necessity that he comply with the architectural form selected for the building. If the form follows the function, the constraints are generally minimal. However, it is not necessarily true that an efficient earthquake resisting capability can be put into any arbitrary form of envelope for the structure. The designer must, therefore, have latitude in his selection of the resisting elements of the structure. He may choose a flexural framework; or a structure having resistance primarily in the outer walls as a monocoque assembly; or a structure strengthened by shear walls or by bracing; or a structure with a resisting core from which the lower parts of the building are hung; or various modifications and combinations of these. The methods of achieving strength and ductility in these various forms are necessarily different and the design criteria have to take this into account.

The permissible level of response of a structure, element or component, must be associated with a loading criteria. The response criteria should properly be dependent on the type of structure, the relative cost of repairs for minor damage, and the hazard in terms of possible loss of life should the item fail or reach extreme deformation limits. The seismic resistance of an element is a function primarily of its yield strength, its natural frequency of vibration, its damping and energy absorption in the elastic range, and its ductility and energy absorption capacity in the range before unacceptable damage occurs.

Dynamic Resistance

Detailed descriptions of the response of simple elastic systems, or more complex structure and elements, subjected to dynamic loading and especially to seismic loading, are given in References [2], [3], and [4]. In general, it can be shown that the response of a simple damped oscillator to a dynamic motion of its base can be represented graphically in a simple fashion by a logarithmic plot as shown in Figure 1. In this figure, there are shown on the one plot, using four logarithmic scales, the following three quantities:

D = Maximum relative displacement between the mass of the oscillator and its base

V = Maximum pseudo relative velocity = ωD

A = Maximum pseudo acceleration of the mass of the oscillator = ${}^2\omega D$

In these relations, ω is the circular natural frequency of the oscillator.

The effective maximum ground motions for the earthquake disturbance for which Fig. 1 is drawn are maximum ground displacement d_m = 10 in., maximum ground velocity v_m = 15 in. per sec., maximum ground acceleration a_m = 0.3g, where g is the acceleration of gravity. The curve shown is a smooth curve rather than the actual jagged curve that one obtains from a precise calculation. The symbols 1, 2 and 3 on the curve represent oscillators, item 1 having a frequency of 20 cps, item 2 of 2.5 cps, and item 3 of 0.25 cps. It can be seen that for item 1 the maximum relative displacement is extremely small, but for item 3 it is quite large. On the other hand, the pseudo acceleration for item 3 is relatively small compared with that for item 2. The pseudo relative velocities for items 2 and 3 are substantially larger

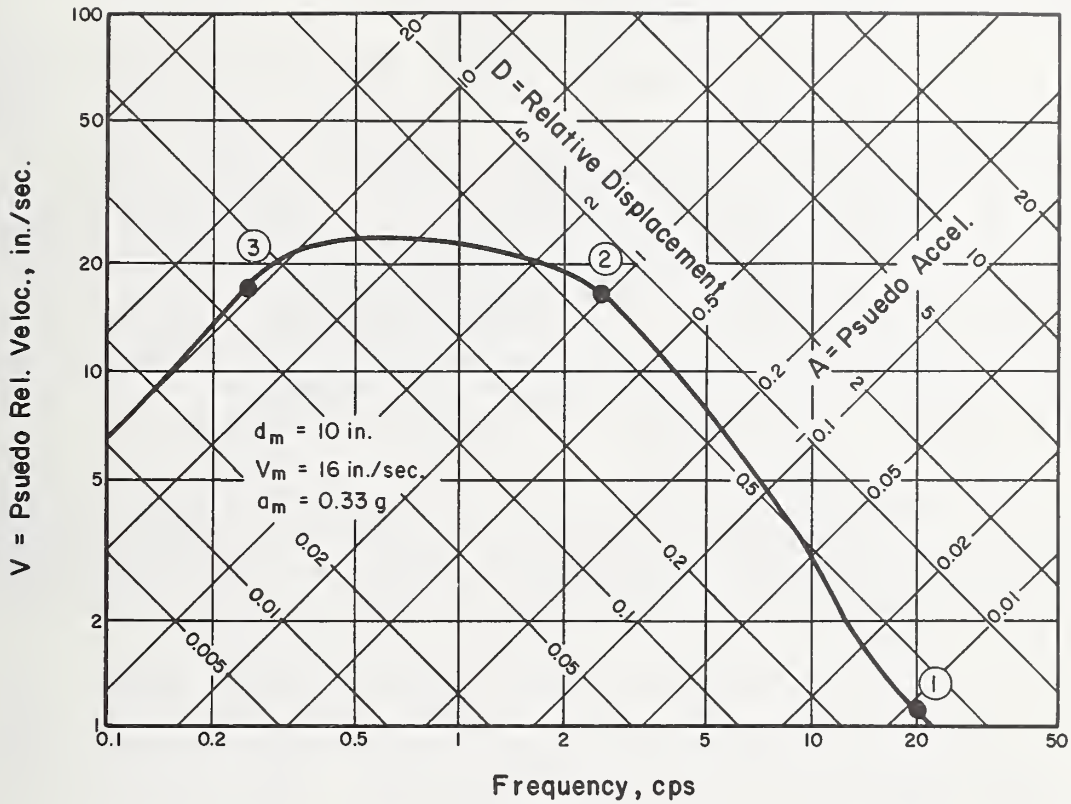

FIG. 1 RESPONSE SPECTRUM FOR TYPICAL EARTHQUAKE

than that for item 1.

The advantage of using the tripartite logarithmic plot, with frequency plotted also logarithmically, is that one curve can be drawn to represent the three quantities D, V and A. The pseudo relative velocity is nearly the same as the maximum relative velocity for higher frequencies, but differs substantially for very low frequencies. It is, however, a measure of the energy absorbed in the spring. The maximum energy in the spring, neglecting that involved in the damper of the oscillator, is $MV^2/2$, where M is the mass of the oscillator.

The pseudo acceleration is practically the same as the maximum acceleration, and the quantity MA is precisely the maximum force in the spring. Therefore, the pseudo acceleration is exactly the same as the maximum acceleration when there is no damping.

In the discussion and figures which follow, the terms "velocity" will be used for V and "acceleration" for A without the explanatory words maximum, pseudo, relative or absolute.

There are many strong motion earthquake records available. One that has been used for a number of years is that for the El Centro earthquake of May 18, 1940. The response spectra computed for the earthquake for several different amounts of damping are shown in Figure 2. The oscillatory nature of the response spectra, especially for low amounts of damping, is typical of the nature of response spectra for earthquake motions in general. A replot of Fig. 2 is given in Fig. 3 in a dimensionless form where the scales are given in terms of the maximum ground motion components. In this figure, the ground displacement is given by the symbol y, and the subscript m designates a maximum value. Dots over the y indicate differentiation with respect to time.

It can be seen from Fig. 3 that for relatively low frequencies, below something of the order of about 0.05 cps, the maximum displacement response D is practically equal to the maximum ground displacement. For intermediate frequencies, however, greater than about 0.1 cps, up to about 0.3 cps, there is an amplified displacement response, with amplification factors running up to about three or more for low values of the damping factor β.

For high frequencies, over about 20 to 30 cps or so, the maximum acceleration is practically equal to the maximum ground acceleration. However, for frequencies below about 6 cps, ranging down to about 2 cps, there is nearly a constant amplification of acceleration, with the higher amplification corresponding to the lower values of damping. In the intermediate range between about 0.3 to 2 cps, there is nearly a constant velocity response, with an amplification over the maximum ground velocity. The amplifications also are greater for the smaller values of the damping factor.

The results shown in Fig. 3 are typical for other inputs, either for other earthquake motions or for simple types of dynamic motion in general. The data from which Fig. 3 was drawn, as well as other similar figures, are taken from Reference [2].

Natural Frequency

The dynamic response of a structure is a function, among other things, of its natural frequencies of vibration in its various modes. Natural frequencies can be computed from the mass and stiffness distributions of the structure but such calculations involve an idealization of the structure for the purpose of the analysis. The influence of nonstructural components on natural frequencies can be of particular importance. Also the natural frequencies may be affected to a large degree by the foundation-structure

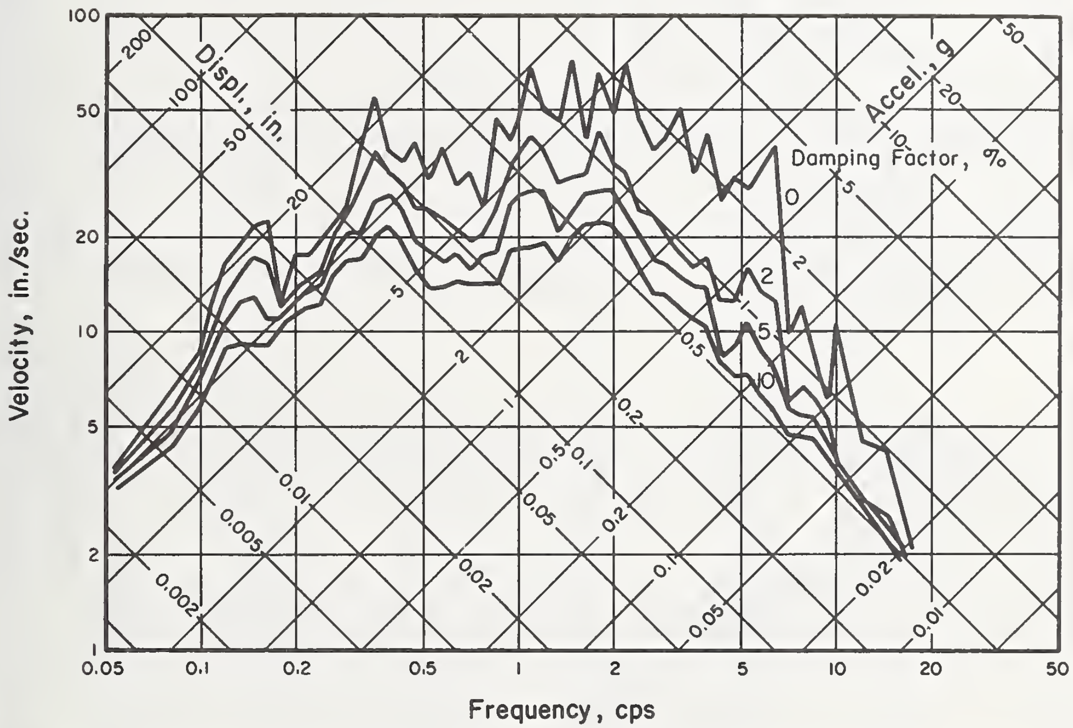

FIG. 2 RESPONSE SPECTRA, EL CENTRO EARTHQUAKE, MAY 18, 1940, NORTH-SOUTH DIRECTION

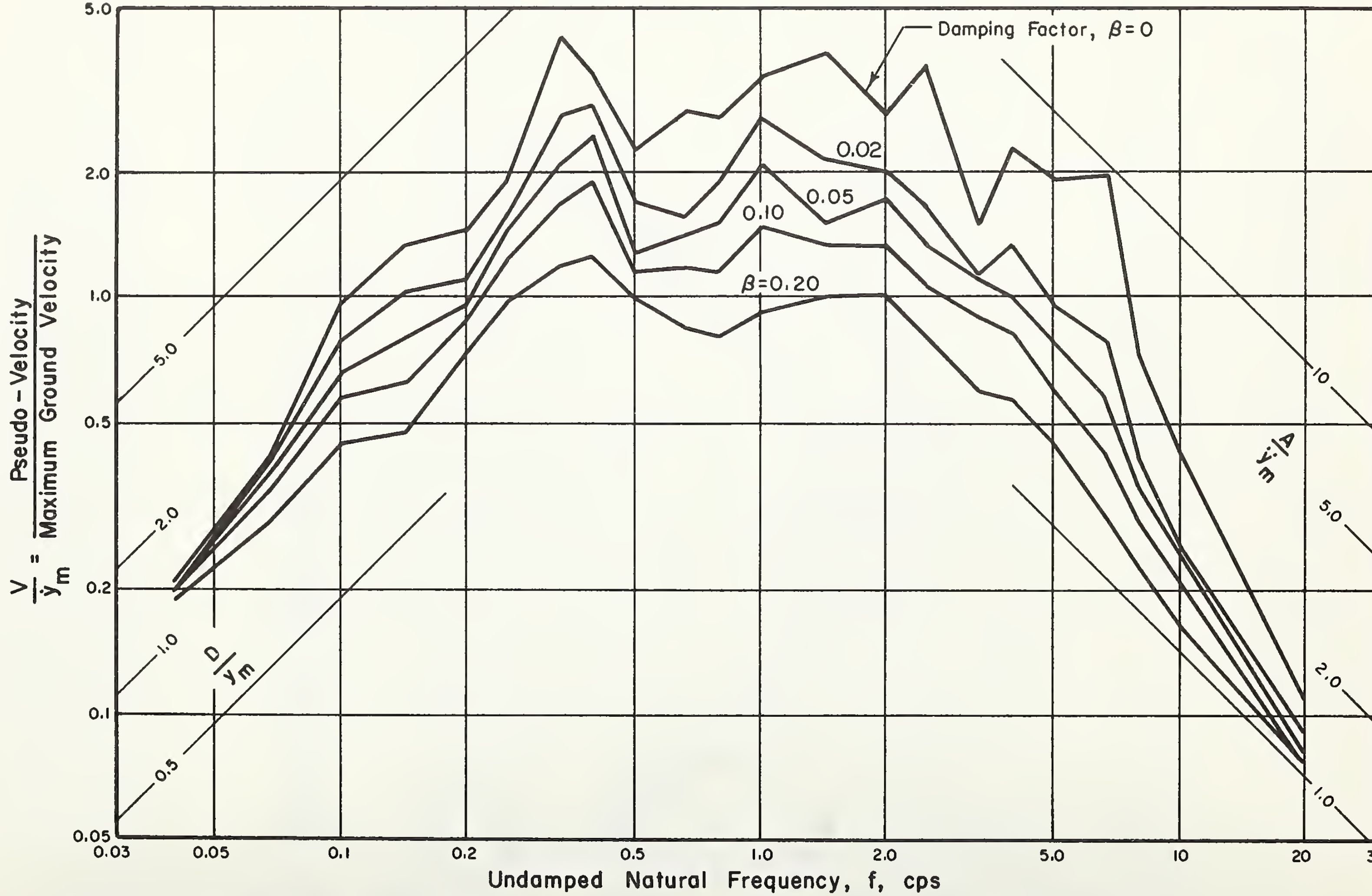

FIG. 3 RESPONSE SPECTRA, ELASTIC SYSTEMS, EL CENTRO 1940 EARTHQUAKE

interaction.

Design specifications which involve natural frequencies have the disadvantage that the structure must be designed, at least in a preliminary way, before the frequencies can be determined, or else the frequencies must be estimated from factors involving judgment and overall dimensions. Hence, such methods may involve relatively large errors in the response or else the method of design must be one of continuing approximations and revisions.

Damping

Energy absorption in a structure arises in various ways including damping or energy absorption of various types within the structure itself, friction or viscous damping, or other types of damping in the structure as well as in the parts of the structure interfering with each other or moving against one another. These can all be generally approximated by use of a damping coefficient. The damping is a function of the intensity of motion and of the stress levels induced within the structural components, and it is highly dependent on the makeup of the structure and the energy absorption mechanisms within it and at its interfaces with the foundation or with other structures. The importance of damping is indicated by the fact that the dynamic response of a structure in an earthquake may be affected to as great a degree by damping as by almost any other parameter. This is especially true in those instances when long sustained nearly harmonic motions are involved. It is because of this reason that the greatest difficulties are found with design specifications other than of the performance type in which the design forces do not properly reflect the differences in damping associated with different materials, different types of framing, and different levels of allowable deformation and stress.

Inelastic Behavior and Ductility

Let us now consider the situation in which the simple oscillator has a spring which can deform inelastically during the response. The simple resistance-displacement relationship for the spring is shown by the light line in Fig. 4, where the yield point is indicated, with a curved relationship showing a rise to a maximum resistance and then a decay to a point of maximum useful limit or failure at a displacement u_m. An equivalent elasto-plastic resistance curve is shown by the heavy line in the figure, rising on a straight line to a point where the yield displacement is u_y and the resistance r_y, and then extending without appreciable increase in resistance to the maximum displacement u_m. The effective resistance curve is drawn so as to have the same area between the origin and u_y as the actual curve, and again the same area to the maximum displacement point. The ductility factor μ is defined as the ratio between the maximum permissible or useful displacement to the yield displacement, for the effective curve.

It is convenient to use an elasto-plastic resistance-displacement relation because one can draw response spectra for such a relation in generally the same way as the spectra were drawn for elastic conditions in Fig. 2 and 3. In Fig. 5 there are shown acceleration spectra for elasto-plastic systems having 2% of critical damping for the El Centro 1940 earthquake. Here, the symbol D_y represents the elastic component of the response displacement, but is not the total displacement. Hence, the curves also give the elastic component of maximum displacement as well as the maximum acceleration, A, but they do not give the proper value of maximum velocity. This is designated by the use of the symbol V' for the pseudo velocity drawn in the figure. The figure is drawn for ductility factors ranging from 1 to 10. It is typical of other acceleration spectra for elasto-plastic systems, as indicated by the acceleration spectra shown in Fig. 6 for the step displacement pulse sketched in the figure.

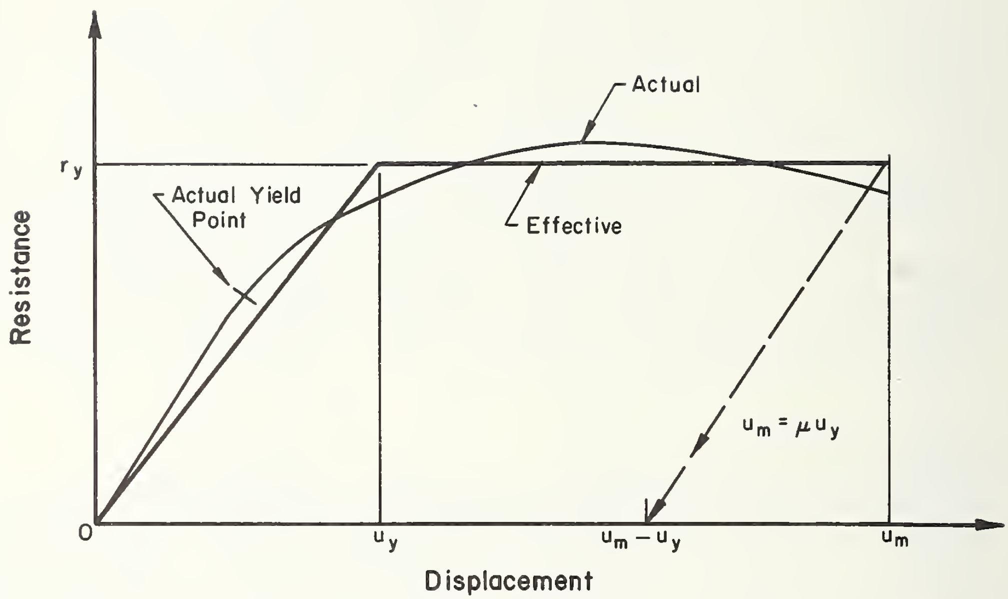

FIG. 4 RESISTANCE — DISPLACEMENT RELATIONSHIP

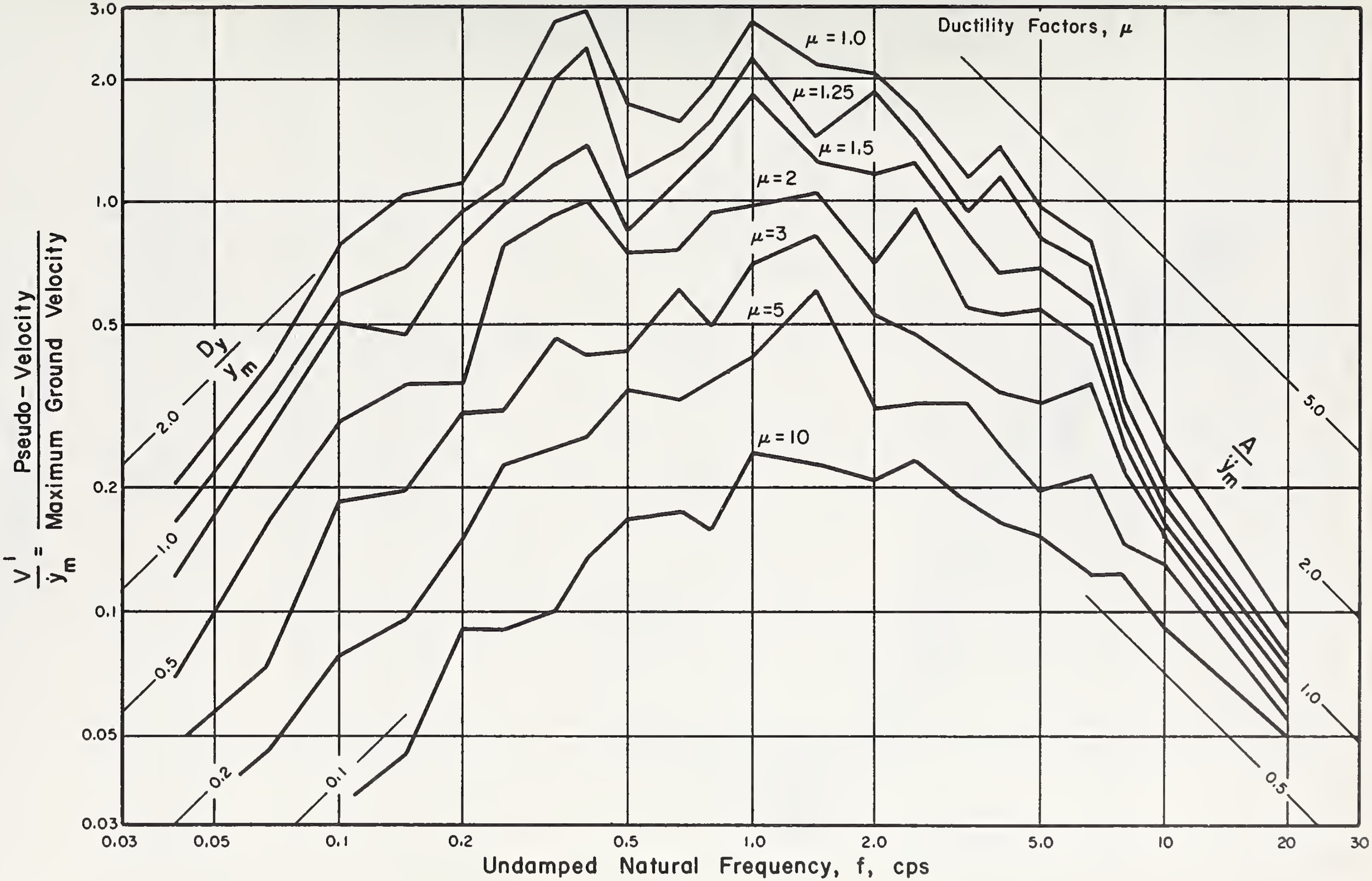

FIG. 5 ACCELERATION SPECTRA FOR ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

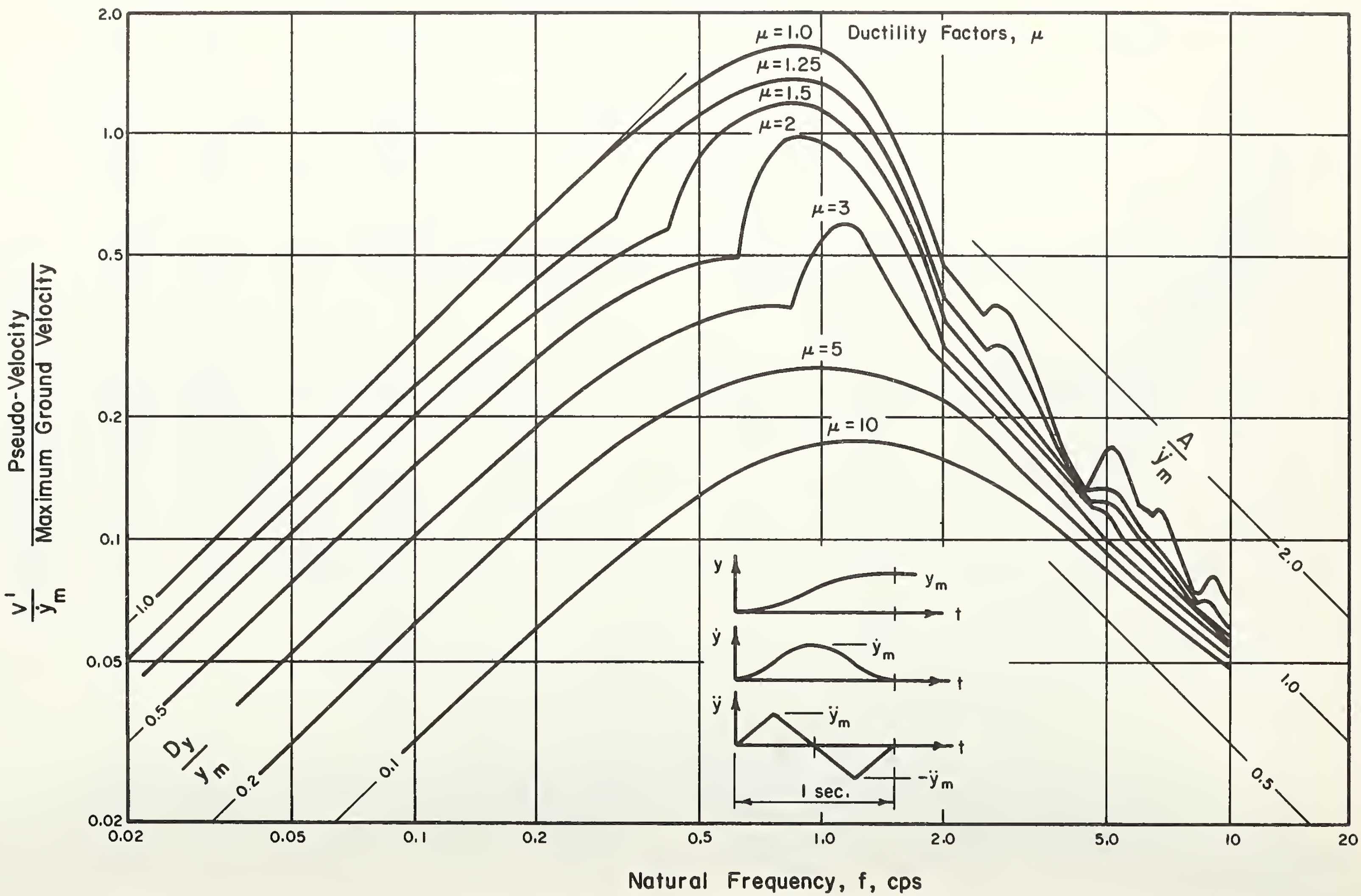

FIG. 6 ACCELERATION SPECTRA FOR UNDAMPED, ELASTOPLASTIC SYSTEMS, STEP DISPLACEMENT

Figure 6 is drawn for a step displacement pulse corresponding to the two triangular pulses of acceleration shown, where the total length of time required to reach the maximum ground or base displacement is 1 second. The frequency scale shown in Fig. 6 will be changed for any other length of time, t, to reach the maximum displacement by dividing the frequencies f by t. In other words, for a step displacement pulse that takes 0.2 sec., the abscissa for a frequency of 1 cps would be changed to 5 cps, and that for 3 cps in the figure would be changed to 15 cps, etc. The general nature of the similarity between Figs. 5 and 6 is important.

One can also draw a response spectrum for total displacement, as shown in Fig. 7. This is drawn for the same conditions as Fig. 5, and is obtained from Fig. 5 by multiplying each curve's ordinates by the value of ductility factor μ shown on that curve. It can be seen that the maximum total displacement is virtually the same for all ductility factors, actually perhaps decreasing even slightly for the larger ductility factors in the low frequency region, for frequencies below about 2 cps. Moreover, it appears from Fig. 5 that the maximum acceleration is very nearly the same for frequencies greater than about 20 to 30 cps for all ductility factors. In between, there is a transition. These remarks are applicable to the spectra for other earthquakes also. One can generalize about them in the following way for general nonlinear relations between resistance and displacement, for single degree of freedom structures.

For low frequencies, corresponding to something of the order of about 0.3 cps as an upper limit, displacements are preserved. As a matter of fact, the inelastic systems have perhaps even a smaller displacement than elastic systems. For frequencies between about 0.3 to about 2 cps, the displacements are very nearly the same for all ductility factors. For frequencies between about 2 up to about 6 cps, the best relationship appears to be to equate the energy in the various curves, or to say that energy is preserved, with a corresponding relationship between deflections and accelerations or forces. There is a transition region between 6 and 20 to 30 cps, depending on the damping ratio. Above 20 to 30 cps, the force or acceleration is nearly the same for all ductility ratios.

Structure-Foundation Interaction

Earthquake motions are transmitted through the ground to the foundation of a structure and then to the structure itself. The interaction between the foundation components of the structure and the earth upon which it rests are of particular importance in defining the nature of the forces and motions transmitted to the structure. Energy absorption can take place at the interfaces between the structure and the foundation, and between the foundation and the supporting medium. Under certain conditions amplifications of motion may even occur. The interaction between the foundation medium and the foundation structure can be particularly complicated when the building is set into the soil or rock rather than resting upon it.

Design specifications either of the cookbook type, the intermediate type, or the type solely concerned with environmental and performance criteria fall short of their requirements if they do not consider the interaction between the structure and its supports, and especially the type of supports, whether it is pile or caisson foundations, isolated footings, or a mat, or some combination of these.

Nonstructural Components

In buildings, particularly, it is necessary to make a distinction between those components which are essential parts of the structure in its resistance to loads and deformations, and nonstructural components which are those parts needed to perform the proper function of the structure but which

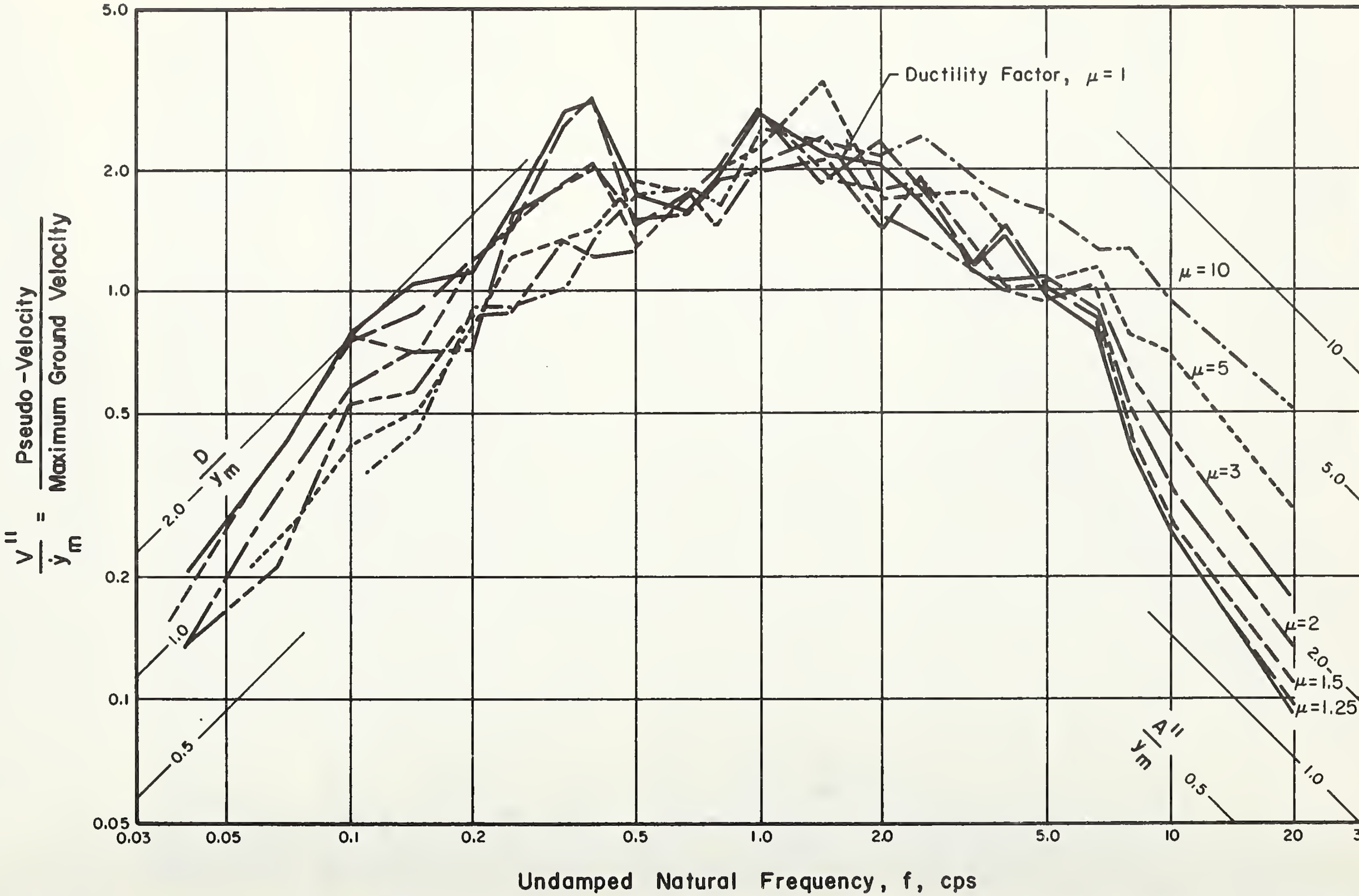

FIG. 7 TOTAL DISPLACEMENT SPECTRA, ELASTOPLASTIC SYSTEMS, TWO PERCENT CRITICAL DAMPING, EL CENTRO 1940 EARTHQUAKE

are not added primarily for resistance to lateral forces. Partitions in a building may be structural or nonstructural depending upon whether they are designed to act as part of the load-carrying framing. However, whatever the designer's intent may be, all the elements of the structure, whether functional or otherwise, have an effect on the behavior of the building under dynamic excitation, and must be considered in terms of dynamic response, strength, and the damage which may be caused by exceeding allowable stress or deformation limits. Even nonfunctional ornamentation on a building that can be dislodged because of lateral motion can cause hazard to life as well as property.

IV. Design Procedures

General Approach

The designer has considerable freedom of choice, in general, as to the type of resisting structure he will use in the design. He may choose a flexible, energy-absorbing structure which can comply with the ground motion readily, or he can use a rigid structure to limit the relative deformation within the structure itself. In one case the strains in the structure are determined primarily by the maximum transient ground displacement or velocity, and in the other they are determined primarily by the maximum transient ground acceleration.

If the structure is in an intermediate range of stiffness, then its energy absorbing capacity is of the greatest importance, which involves both its strength and ductility in some balanced manner. Under these conditions, one may reduce the strength and increase the ductility, or increase the strength and reduce the ductility, in both cases arriving at a satisfactory design. Although the designer must be careful in the determination of these balances, and must look into the strength and ductility of elements or components as well as those of the completed assemblage, he can make up for deficiency in one by an overdesign in the other, in many instances. Constraining the designer to use always highly ductile elements may be unreasonably restrictive since it appears possible to design a structure with as much margin to resist failure by making it less ductile but stronger, in an appropriate fashion. It appears unwise to establish design criteria solely on preconceived notions as to either strength or ductility without considering the combination of both of these that is required for adequate performance.

Theoretically and to a considerable extent practically, it is possible to use any material in almost any fashion one chooses to use it, by providing the proper combination of strength and ductility associated with the particular structural configuration and dimensions, thereby to insure that the completed structure will be able to perform adequately under the appropriate loading or motion environmental conditions.

It has already been noted that in many structures it is desirable, and in fact quite proper and reasonable, for the structure to go well into the inelastic range of behavior, especially for the maximum or extreme environmental seismic conditions. Different types of framing and different materials pose a variety of problems for an adequate specification of performance involving deformations and stresses beyond yield. This has been taken into account in existing codes in various ways, usually by specifying the relative intensity of loading to be considered for different types of framing. Each material must be studied from the point of view of its particular characteristics of strength and ductility, when fabricated into structural members or elements, or when connected together to form a structure. The performance criteria must be prepared in such a way as to avoid unusual handicaps to any one type of framing or material, or to give unusual advantages to any other type.

Selection of Parameters

In the light of the preceding discussion, we can now develop a basis for design of structures, elements or components, where these are subjected directly to the ground or base motion for which we have maximum values of displacement, velocity and acceleration. We first proceed with selection of values of damping. Table 1 is reproduced with some changes from References [5] and [6], and gives the percentage of critical damping for various types and conditions of structures or elements, as a function of stress level. It represents the best information available at the present time, but certainly involves a great deal of judgment and interpretation.

The damping in structural elements and components and in supports and foundations of the equipment is a function of the intensity of motion and of the stress levels introduced within the structural component or structure, as well as being highly dependent on the makeup of the structure and the energy absorption mechanisms within it. For example, a structure with riveted or bolted joints that can undergo relative motion during deformation will absorb a great deal of energy in friction in these joints. A reinforced concrete beam that is cracked, where the elements on the two sides of the crack can move relative to one another with the absorption of energy at the faying surface, will also absorb considerable energy. On the other hand, a homogeneous solid structure or a welded steel structure has relatively small amounts of lost energy because of play in the joints, and a concrete beam before cracking has a relatively small amount of energy losses except those within the material itself. Hence, the degree of damping depends on the framing and makeup of the structure or elements, and on the material used and the stress level within the material for the degree of excitation which it experiences in the shaking motion. For low stress levels and for homogeneous structures, steel or reinforced concrete below cracking levels, the damping may be no greater than in the range of one-half to one percent. For stresses at the level of working stresses or at about half the level of yield point values, the damping may range from about 2 percent for welded steel structures, for well reinforced concrete structures with only small amounts of cracking or for prestressed concrete structures, to 3 percent to 5 percent for ordinary reinforced concrete structures with considerable cracking, and possibly above 5 percent for riveted or bolted connections, or for wood structures with nailed joints and the like. At or near yield point values of stress, the damping may be in the range of about 5 percent for steel structures and prestressed concrete structures that have not completely lost their prestress, ranging to 7 to 10 percent for ordinary reinforced concrete, and as high as 10 to 15 percent for structures with play in the joints, or for masonry structures.

The fundamental frequency of vibration, or its reciprocal, the fundamental period, is best estimated by a simple calculation by use of standard methods of analysis such as are described in Reference [3]. For buildings simple rules, also given in [3], are often used to approximate the fundamental frequency, but are generally not reliable for unusual types of framing or for extremely heavy or extremely light construction.

The ductility factors for various types of construction are more difficult to characterize. They depend on the use of the building, the hazard involved in its failure, the material used, and the framing or layout of the structure, and above all on the method of construction and the details of fabrication of joints and connections. A discussion of these topics is given in Reference [3] also.

Design Spectrum - Elastic

In either analysis or design for earthquake resistance it is convenient to use the concept of the response spectrum. A response spectrum developed to give design coefficients is called a "Design Spectrum".

Table 1

RECOMMENDED DAMPING VALUES

Stress Level	Type and Condition of Structure	Percentage of Critical Damping
Working stress, no more than about 1/2 yield point	a. Vital piping	0.5 to 1.0
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	a. Vital piping	2
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5
	c. Prestressed concrete with no prestress left	7
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

In general, for any given area or site, estimates might be made of the maximum ground acceleration, maximum ground velocity, and maximum ground displacement. The lines representing these values can be drawn on the tripartite logarithmic chart of which Fig. 8 is an example. The lines showing the ground motion maxima in Fig. 8 are drawn for a maximum ground acceleration of 1.0g, velocity of 48 in/sec., and displacement of 36 in. These data represent motions more intense than those generally considered for any postulated design earthquake hazard. They are, however, approximately in correct proportion for a number of areas of the world, where earthquakes occur either on firm ground, soft rock, or competent sediments of various kinds. For relatively soft sediments, the velocities and displacements might require increases above the values corresponding to the given acceleration as scaled from Fig. 8. However, it is not likely that maximum ground velocities in excess of 4 to 5 ft per second are obtainable under any circumstances.

Amplification factors for the various ranges in the response spectrum were considered in References [5] and [6]. The values determined therein for a number of earthquakes, with some smoothing and reduction of peaks to present a reasonably consistent probability of failure (of the order of about 10 percent or less), are given in Table 2. The amplification factors given in that table are used in connection with Fig. 8, as explained below.

For each of the amounts of damping shown in Fig. 8 or tabulated in Table 2, the amplified displacements are shown on the left, the amplified velocities at the top, and the amplified accelerations in that part of the right-hand side of the figure for which the lines are parallel to the maximum ground acceleration line, but lie above it. We shall identify these portions of the line as the amplified displacement region, the amplified velocity region, and the amplified acceleration region, respectively.

At a frequency of about 6 cps, the amplified acceleration region line intersects a line sloping down toward the maximum ground acceleration value, and intersecting that line at various frequencies, depending on the damping. The intersection is at a frequency of about 30 cps for 2% damping, and the other lines are parallel to the line for 2% damping. These lines are designated as the acceleration transition region of the spectra. Finally, beyond the intersection with the maximum ground acceleration line, the response spectrum continues with the maximum ground acceleration value for higher frequencies.

The spectra so determined can be used as design spectra for elastic responses. The spectra are completely described when the maximum ground motion values are given for the three components of ground motion, and the damping is known. When only the maximum ground acceleration is given, the values used for maximum ground velocity and displacement are taken as proportional to those in the figure, or as scaled by the same scale factor relative to the maximum ground acceleration compared with 1 g.

The amplification factors given in Table 2 and shown in Fig. 8 are still under study, but it is not expected that major revisions in them will be required.

Design Spectrum - Inelastic

To use the design spectrum to approximate inelastic behavior, the following suggestions are made. In the amplified displacement region of the spectra, the left-hand side, and in the amplified velocity region, at the top, the spectrum remains unchanged for total displacement, and is divided by the ductility factor to obtain yield displacement or acceleration. The upper right-hand portion sloping down at 45°, or the amplified acceleration region of the spectrum, is relocated for an elasto-plastic resistance curve, or for any other resistance curve for actual structural materials, by choosing

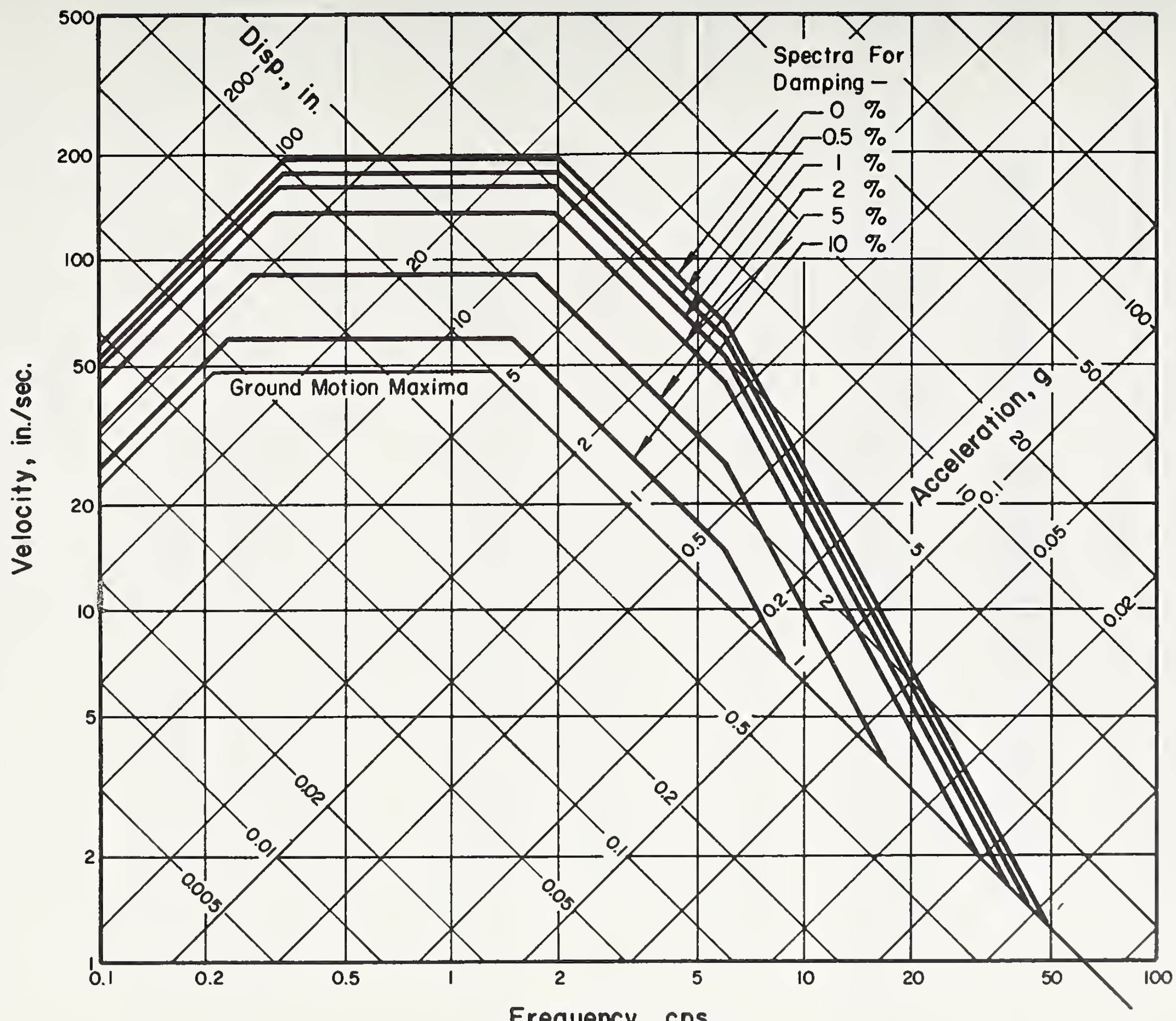

FIG. 8 BASIC DESIGN SPECTRA NORMALIZED TO 1.0 g

Table 2

RELATIVE VALUES OF SPECTRUM AMPLIFICATION FACTORS

Percent of Critical Damping	Amplification Factor For Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
5	1.4	1.9	2.6
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

it at a level which corresponds to the same energy absorption for the elasto-plastic curve as for an elastic curve shown for the same period of vibration. The extreme right-hand portion of the spectrum, where the response is governed by the maximum ground acceleration, remains at the same acceleration level as for the elastic case, and therefore at a corresponding increased total displacement level. The frequencies at the corners are kept at the same values as in the elastic spectrum. The acceleration transition region of the response spectrum is now drawn also as a straight line transition from the newly located amplified acceleration line and the ground acceleration line, using the same frequency points of intersection as in the elastic response spectrum.

In all cases the "inelastic maximum acceleration" spectrum and the "inelastic maximum displacement" spectrum differ by the factor μ at the same frequencies. The design spectrum so obtained is shown in Fig. 9.

An earlier procedure for the definition of inelastic response spectra for design was presented in Reference [2]. In that presentation, the displacement bound, the velocity bound, and the acceleration bound were determined, respectively, by keeping the displacement constant, the energy constant, and the force in the spring constant, and drawing the corresponding maximum response displacement limits.

The revised procedure presented in this report is shown in Fig. 9 for 2% damping, for an elasto-plastic system with a ductility factor of 5. Both the maximum displacement and maximum acceleration bounds are shown, for comparison with the elastic response spectrum.

The solid line $DVAA_o$ shows the elastic response spectrum. The heavy circles at the intersections of the various branches show the frequencies which remain constant in the construction of the inelastic design spectrum.

The dashed line $D'V'A'A_o$ shows the inelastic acceleration, and the lines $DVA''A''_o$ shows the inelastic displacement. These two differ by a constant factor $\mu = 5$ for the construction shown, but A and A' differ by the factor $\sqrt{2\mu - 1} = 3$, since this is the factor that corresponds to constant energy, as indicated in Reference [2].

Of course, the elasto-plastic or other inelastic response spectra can be used only as an approximation for multi-degree-of-freedom systems.

In the development of a design spectrum one may choose to use an "effective" value of maximum ground acceleration rather than an actual value, particularly in cases where the higher spikes of acceleration are associated with very short durations and correspond to velocity changes much smaller than the maximum ground velocity, or where the duration of the earthquake motion is extremely short and the influence on failure or inelastic behavior is thereby lessened.

Vertical and Horizontal Excitation

Since the ground moves in all three directions in an earthquake, and even tilts and rotates, consideration of the combined effects of all these motions must be included in the design of important structures. When the responses in the various directions may be considered to be uncoupled, then consideration can be given separately to the various components of base motion, and individual response spectra can be determined for each component or direction of transient base displacement. Calculations have been made for the elastic response spectra in all directions for a number of earthquakes. The complete results for the three components of motion for these are not yet available, but the trends are summarized below.

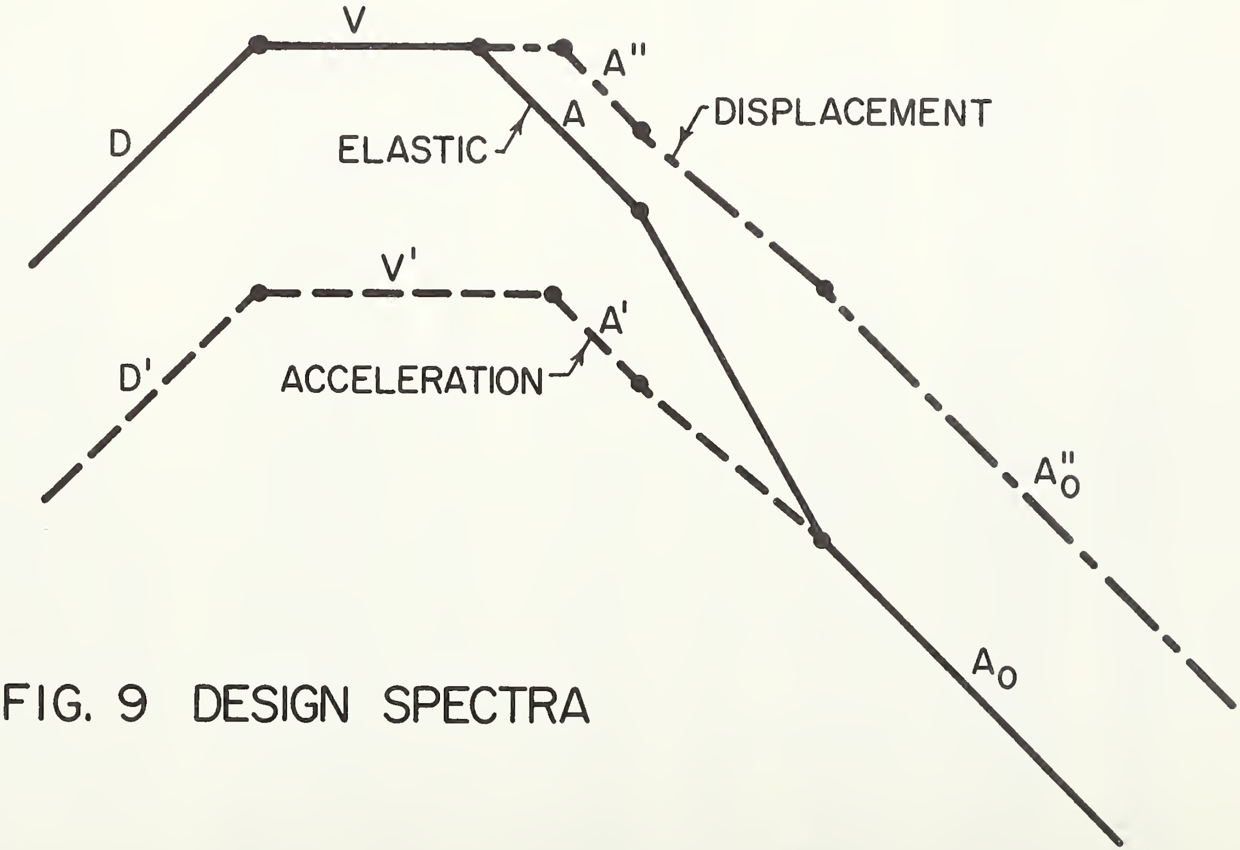

FIG. 9 DESIGN SPECTRA

There are several interesting features of the response spectra. For example, the frequencies at which spikes and valleys occur are generally not the same for the different directions of any earthquake nor for the same directions at the same site for different earthquakes. The responses for the two horizontal directions show cross-overs and significant differences in some ranges of frequency. The vertical response is often equal to or greater than the maximum horizontal response in the high frequency region, but is somewhat to a great deal less in the intermediate and low frequency regions.

It is suggested that until further information becomes available the following design criteria be used:

(a) The design spectrum for vertical response be considered equal to that for horizontal response for frequencies in the amplified acceleration range or higher frequencies. In other words, the acceleration bounds are the same for both horizontal and vertical response.

(b) The design spectrum for vertical response be considered equal to two-thirds that for horizontal response for frequencies in the amplified velocity or displacement ranges.

Combined Effects of Earthquake Motions

Since the responses for motions in the various directions (horizontal and vertical) may not occur at the same time, it is considered reasonable to combine the effects of the several components of motion in a probabilistic manner, by taking the maximum stress, deflection, or other specific response as the square root of the sums of the squares of the corresponding responses to the individual components of motion.

The effects of transient tipping, tilting, and rotation of the ground during an earthquake have not been studied extensively. An elementary treatment of some aspects of these movements has been given in Section 7.7 of Reference [3], and the effects of rotation of the ground about a vertical axis on the accidental torsion in symmetrical buildings, for example, is given in Section 15.6 of the same reference.

When the responses of the structure or component are coupled, the analysis becomes much more complex and a three-dimensional (or at least two-dimensional) response analysis must be considered. However, data regarding the simultaneous input motions must be used in such an analysis, and little guidance is available on this topic.

The motions due to an earthquake occur in both horizontal and vertical directions in a complex manner. It is necessary to consider the interactions between the responses in the various directions, and especially important to consider the interaction between the vertical and the maximum horizontal response. Vertical loads, and eccentricities of the vertical loads caused by horizontal displacements, must often be taken into account with especially heavy structures that carry large masses at or near the points which may deflect a great deal. Some of the resisting capacity for horizontal motions may be used up by the secondary effects of the eccentricities of the gravity loads.

Quite often, the vertical motions may produce vertical stresses in the structure or element that exceed by a large amount those stresses due to the inertial forces corresponding to the vertical acceleration multiplied by the mass of the element. This is true when the frequencies of vibration in the vertical direction of the element or component are in the range where major amplification of response can occur.

Special Considerations and Quality Control

A number of points are often overlooked in the design of structures or components to resist dynamic motions. A summary of some of the more important factors, but by no means a complete listing of all of them, is contained herein.

One of the factors that is commonly overlooked is the matter of relative motions between the parts or elements of a system having supports at different points, because the support motions may not occur simultaneously. Hence, there may be transient relative motions which produce strains in the structure, in addition to the strains produced by the dynamic effects of the overall motion. This is especially important in piping, electric wiring, or other elements connecting parts of a facility.

Finally, there are a group of items which do not lend themselves readily to analytical consideration. These concern the details and material properties of the element or component, and the inspection and control of quality in the construction procedure. The details of connections of the structure to its support or foundations, as well as the various elements or items within the structure or component, are of major importance. Failures often occur at the connections and joints because of inadequacy of these to carry the forces to which they are subjected under dynamic conditions. Inadequacy in properties of the materials can often be encountered, leading to brittle fracture where sufficient energy cannot be absorbed even though such energy absorption may have been counted on in the design.

In order to insure that the intent of the designer is achieved, control of construction procedures and appropriate inspection practices are necessary. It is important that the practical aspects of seismic design be emphasized and that both designers and constructors be fully aware of their importance.

V. Desirable Features of Design Codes

Relation Between Analysis and Design

When the configuration of a structure is fixed by architectural or other requirements, the designer has a restricted choice in the development of the strength and ductility required to insure adequate seismic behavior. It is not always possible to say that some design layouts are better than others for dynamic resistance, although it is fairly clear that different choices of framing can lead to vastly different requirements of strength and ductility. For example, a framed structure is generally less stiff and usually lower in frequency than a shear wall structure with nearly solid walls providing lateral resistance. Hence the design forces may be smaller for the framed structure than for the shear wall structure, although the required ductility may be larger. Methods of design for the dynamic loadings arising from earthquakes are in general simpler and better understood for structures for which there is a great deal of experience. However, unless methods are developed and specifications are devised to take account of new structural types or of new imaginative architectural designs, such designs will be placed at a disadvantage relative to more standard designs because of the necessity for providing greater margins of safety for those designs for which experience is unavailable.

The methods of analysis, and also the details of the design specifications, have implications on the cost and the performance capability of the design. If the specifications are unduly conservative the design will not only be unduly conservative, but may also be forced into a type that is stronger and less ductile than is desirable. It is difficult to avoid differences in the degree of conservatism among different types of structures, and in some cases it is undesirable to do so. Some materials by their nature, including their variability or lack of adequate control of properties, may require a greater factor of safety than other materials the properties of which are more accurately determinable and controllable. The margin between incipient failure and complete collapse may differ for different materials and may therefore involve a difference in the factor of safety required in the design.

Basic Function of Design Codes

The designer, as well as anyone else who has a responsibility for the final structure, has to have some general method of knowing that gross errors have been avoided, and must have some basis of comparison to insure that the design is adequate in an overall sense. It is the purpose of building codes and specifications to perform these functions. However, it is not yet established that building codes can do this kind of job without introducing constraints and controls that may be a severe handicap on the development of new design concepts and procedures. Where building codes are used to insure, by rule-of-thumb methods, that a design is adequate, they embody the result of experience and judgment and must therefore deal implicitly, if not explicitly, with particular structural types and configurations.

The most desirable type of design code or specification is one which puts the least restrictions on the initiative, imagination and innovation of the designer. Such a code might involve only criteria for: (1) the loading or environment; and (2) the level of response, the stresses and deformation, or the performance of the structure under the specified loading or environmental conditions. Such an approach need not, and preferably should not, indicate how the designer is to reach his objective, provided he can show that he has achieved a structural capability to resist the specified environmental conditions. This approach is generally the one that is now used for the design of nuclear reactor power stations. Experience over the past several years in approaching seismic design criteria in this way has indicated a number of problems, but has also been reasonably successful in avoiding constraints due solely to the specifications themselves, although there have been constraints based on the environmental conditions and the stress and deformation levels allowed.

Seismic Response Criteria

The permissible level of response of a structure must, of course, be associated with the loading criteria. One cannot be specified independently of the other. This implies, for example, that different response criteria are to be associated with the probable earthquake or the historical earthquake from those used for the maximum credible or extreme earthquake. Moreover, for either of these, the response criteria should properly be a function of the type of structure, the relative cost of repairs for minor damage, and the hazard in terms of possible loss of life should the structure or any of its elements fail. Hence, the response criteria could be greatly different for individual homes than for multistory buildings housing hundreds or even thousands of people, and certainly different even from these for high dams above large centers of population or for nuclear reactor power stations.

It appears reasonable to establish such criteria in terms of the consequences of failure, and in relative terms associated with yield points or buckling loads of similar dynamic limit loads for the particular material or structural elements used. The aim of the criteria should be consistent with the basic seismic design philosophy stated earlier. Appropriate performance criteria may well be stated most rationally in terms of probabilities of failure or collapse associated with various levels of the probability of the hazard considered.

Performance Criteria

It is essential that the response levels or maximum stresses and deformations be limited, for structures, components, and details such as joints and connections, in order to insure adequate strength and ductility of a structure as well as of its various component parts. However, it is desirable, in the development of the basis for a performance criterion, that the designer's approach not be too greatly constrained. For example, it may be unwise to prescribe limits for both strength and ductility in such a way that the balance between the two cannot be adjusted to take account of new material properties or new structural types as they are developed. A trade-off between ductility and strength should be available in the methods that are permitted, so as to achieve economy without the sacrifice of safety. But whether one is interested in achieving strength or ductility, or both, the materials have to be used in an appropriate fashion, and adequate methods of inspection and control of construction are needed to insure that their use is proper.

Methods of Analysis

A variety of methods of analysis are now available, ranging from simple upper-bound static coefficient methods to modal response combinations, including time history analyses either in the elastic or inelastic range, and extending to probabilistic methods or methods involving consideration of random vibrations. It should be possible to use any of these that are sufficiently justified by either general acceptance or by demonstrated mathematical and physical validity, with requiring that any one particular method be used. Of course, it is desirable always to allow the use of simple methods that are adequately conservative. Properly stated criteria, used with appropriate methods of approximate analysis and design, may make it unnecessary in many cases to perform detailed and costly analyses, particularly in those instances where the simpler approximate methods insure adequate margins of safety.

To push these concepts to the limit involves generally the idea of approximate methods that give reasonably accurate results or, at least, results that are consistent with those obtained from more precise analyses. Depending on the degree and extent of the approximation, it may be necessary, from the point of view of achieving a reasonable degree of precision, to have special modifications of a general procedure for various unusual structural classifications. Methods may be used that do not require knowledge of the period of vibration of the structure, for example, or the methods may involve an approximate determination of the natural period. Approximate damping factor determinations may be involved as well.

For the complete development of this type of approach it may be desirable to explore the possibility of a hierarchy of methods ranging from the crudest approximation, for very simple structures and structural elements, to more accurate approximate methods, for structures of intermediate complexity, and to relatively precise and accurate calculations, for extremely complex structures.

Special Structures

Although many of the problems associated with the design of special structures such as nuclear reactors, high dams, schools and hospitals, are similar to those involved in other more ordinary types of structures, there are some implications of failure of these special structures that require special consideration in selecting margins of safety and the development of design procedures and criteria governing them.

Many structures or parts of structures and many items of equipment can be severely damaged without any implication of loss of life or even of major property damage in an earthquake. For such items and structures it is unnecessary and certainly uneconomical to provide great margins of safety for unlikely earthquakes. The margin of safety of the provision made for an earthquake of a reasonable degree of probability of occurrence within the lifetime of a structure need only be great enough to offset the cost of repairs or reconstruction.

However, structures whose loss of function might cause hazard to life, or structures which are important to prevent damage to major services, have to be designed on an entirely different basis. For such structures, an earthquake even of relatively low probability of occurrence, but one that possibly could occur, should not cause collapse or damage of such a nature that endangers the health or life of large numbers of the population.

Hence, for such structures, much greater accuracy in procedures and assumptions is required, and the type of design specification must be more carefully framed and more clearly stated to give an assuredly adequate margin of safety against failure even for unusual types of structure and framing.

The type of design specification used for major nuclear reactor power plants has emphasized loading or environmental criteria, and performance criteria in terms of stress and deformation levels of response. The experience that has been gained with criteria of these types indicates that benefits are possible for other types of special structures with similar kinds of design specifications. The advantages are in the encouragement of the designer to explore various types of structures, to consider the use of a variety of materials, and to look for economical ways of achieving the desired levels of safety and performance.

Detailed prescriptions of methods and procedures were necessary when the majority of practicing engineers had neither the sophistication nor the computational aids to take account of the more accurate methods of analysis and design that are now available. However, this situation has changed and is continuing to change at a rapid rate. With the increase in numbers of more highly trained engineers, and with a greater store of knowledge available, together with more efficient ways of using that knowledge which have become possible with the general availability of and accessibility to high speed digital computers, it appears that it is now possible to make a major change in our methods of specifying or codifying seismic design.

General Comments and Conclusions

The field of earthquake engineering is relatively new, not much more than three decades old, and advances in knowledge are progressing at a rapid rate, not only because of a greater emphasis on analytical and experimental work in the laboratory, but also because of the availability of more definitive information on earthquake motions, the accumulation of strong-motion records, and recent accelerated expenditures on research. It is important that the discoveries from observations, and studies from the research laboratory and the analyst, find their way into practice as soon as possible. But this is difficult because engineers are traditionally unwilling to take chances on things that they have not proved out. Too little attention has been given to methods of demonstrating adequate performance capability for major structures and structural elements. Although this is not necessarily a part of our consideration in design specifications, it might well be desirable to look for ways of determining the capability of completed structures or for ways of proving the performance capability in the design stage by appropriate methods, so as to encourage the development of more economical designs and methods.

In order to insure that the intent of the designer is achieved, control of construction procedures and appropriate inspection practices are necessary. This point cannot be overemphasized. It is difficult to synthesize and distill the collective experience and judgment of the engineering profession into a set of rules, especially when they cannot be put into a mathematical formulation. This is a difficulty, however, not only with performance and environmental criteria but also with more standard types of design specifications of the current and past eras. Nevertheless, it is important that the practical aspects of seismic design be emphasized and that engineers and constructors be fully aware of their importance.

It is the intent of this discussion to focus attention on the aims and objectives of seismic design in such a way as to encourage the development of methods and practices suitable for structural design of the future; methods that will permit more freedom and latitude to the architectural and engineering innovator.

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PROCEDURES AND CRITERIA FOR WIND RESISTANT DESIGN

by

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and

John J. Healey**

Introduction

This paper is intended to review the current status of wind resistant design, to identify areas where improvements in design methodology and building code provisions are required and to make recommendations regarding the acquisition of basic data, research activities, and the development of analytical and experimental procedures.

Total losses in terms of property damage and loss of life resulting from severe wind storms will tend to rise in the future as population growth increases the probability of a storm striking a developed area. In addition, the need for increased economy in terms of site usage and building materials adds to the problem by providing more slender structures subject to more severe operating conditions. It is felt, however, that economical structures with more consistent overall safety and greater resistance to severe wind conditions can result from a coordinated effort to develop more sophisticated analysis and design procedures, to accumulate necessary data, and to enact upgraded building code provisions.

The major sections of this review are as follows:

(1) Severe Storms

(2) Public Safety and Protection

(3) Structural Analysis and Design

(4) Stochastic Methods

(5) Recommendations

(6) References

References provide supporting material and background information related to the four major topics in the paper.

Severe Storms

The natural phenomena which produce severe wind conditions of potential danger of life and property are hurricanes, thunderstorms, tornadoes and extratropical storms. Knowledge of the origin, characteristics and duration of these wind sources is therefore essential to a program of disaster mitigation. At present, detailed climatological and structural loading data for

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severe storms is quite limited, particularly for tornadoes. As the population grows and built-up areas spread, storm damage and loss of life will naturally rise due to the increased probability of structures being subjected to severe loads. The following sections review the data currently available on severe storms, discuss the requirements for basis data, design criteria and code provisions related to wind-resistant building design, and recommend research activities.

On the basis of duration, severe storms can be divided into two fairly distinct groups: relatively long-term storms (hurricanes and extratropical storms) and short term disturbances (tornadoes and thunderstorms).

Hurricanes (tropical cyclones) which affect North America originate in the middle latitudes of the Western Atlantic Ocean and tend to travel northward along the southeastern and eastern regions of the United States before they diminish in intensity. The path route of these intense storms (75-150 mph) has an important effect upon the character of the storm. In addition, factors such as the intake of moisture and cold air, together with path route, determine if the storm will gain or lose intensity, or perhaps develop into an extratropical cyclone. In general, these latter storms are disturbances of great intensity covering areas of perhaps hundreds of miles in diameter.

Due to their long duration and somewhat predictable behavior, a good deal of data has been gathered regarding maximum wind speeds and pressure distributions of hurricanes and extratropical storms. In addition, quite a bit is known about the conditions which spawn hurricanes and the effect of path upon storm characteristics and duration. In fact, computer models of hurricanes have been developed for forecasting purposes and for use in conjunction with "seeding" studies to investigate the possibility of reducing a hurricane's intensity and/or altering its path [1]*.

Efforts in the area of strong wind measurements and basic studies of these long duration storms to provide understanding of their formation and possible alteration are important and should be encouraged. The most pressing need, however, is in the area of hurricane path prediction to increase the amount of warning time provided to communities in the path of the storm.

The most devastating winds in this country are those resulting from tornadoes; yet comparatively little information on tornadoes is available, especially in terms of measured data. Tornadoes are intense, fairly localized storms composed of a translating vortex with high winds and a center of very low pressure. A large number of these storms occur each year in the United States and the amount of damage rivals the total losses caused by earthquakes [2].

The other type of short-duration storm, the thunderstorm, is also a source of appreciable short-term violent wind. In contrast to tornadoes, however, the maximum wind data from this source are included in the wind records for a particular site, although little is known about gust action during thunderstorms.

Due to their random occurrence, localized extent and short duration, data collection for tornadoes is extremely difficult [3]. Most of the information on tornadoes is related to occurrence and gross characteristics such as width and length of path and resulting damage. Thom conducted a statistical analysis of records for the period 1953-62 and established estimates of annual tornado probability for one-degree square regions within the United States, estimates of the probability of a tornado striking a point, and frequency distributions of tornado path width and length [2]. Tornado

* Numbers in brackets refer to corresponding items in the References.

occurrence within a given region varies considerably in number and severity from season to season and from year to year. Tornado occurrence studies should be continued and more detailed estimates obtained [4]. Various studies are underway to study the basic mechanisms at work within a tornado and to describe the characteristics of engineering significance [5]. Research is being conducted on analytical and experimental tornado models and field measurements (waterspouts) to investigate tornado formation, changes during the lifetime and the mechanism for decay. This work may result in increased warning time to tornado-threatened areas and, possibly, some means of altering tornadoes after their formation.

The objectives of field observations of tornadoes include measurements of translation velocity, space and time gradients of pressure and velocity, both horizontal and vertical. Direct measurement of tornado characteristics with present forecasting systems is virtually impossible because there is insufficient time to place the required instruments directly in the path of the storm. As a result, wind speed and pressure differential measurements can only be obtained through indirect measurements [6].

The significant loads to which a structure is subjected during the passage of a tornado include the following: direct pressure on the windward face, outward pressure on the leeward face due to pressure deficit, outward pressure on the sides, roof and leeward face resulting from the low pressure in the vortex, and forces due to missiles striking the structure. The initial negative pressure from the vortex and the re-application of positive pressure when the ambient air pressure returns to atmospheric are actually applied as significant dynamic loads. The severity of this dynamic effect is directly related to the translational speed of the storm. In addition, short duration gusts and pressure drops occur, apparently at random, as the major loads are being applied. From the standpoint of design, data is required on the maximum rotational velocity, the translational velocity and the pressure drop. The significance of the vertical wind velocity in the tornado core should be investigated as well as the possibility of wind-induced oscillations during the passage of the storm. Bounds should be established on impact loading due to missiles striking the structure.

Existing data on tornado wind velocities and pressure is inconclusive and displays considerable scatter. Aside from the translational velocity which can be measured fairly accurately, indirect measurements must be relied upon for tornado loading data. Some of these indirect measurements are investigation and analysis of damaged structures, observation of ground marks, scaling from motion pictures taken of the funnel, observation of the shape of the funnel and missile penetration data. The situation is further complicated by the height effect within the vortex and the variation of severity within a given storm. In addition, the transition from an indirect measurement to an estimated velocity or pressure usually involves some assumptions which render the results less reliable. At present, the most reliable indirect measurement for both velocity and pressure information is the analysis of the motion of flying debris in the funnel as shown in motion pictures taken during the storm.

The storm's translational velocity influences two important design factors as follows: the maximum wind velocity is the vector sum of the rotational and the translational velocity; in addition, the pressure gradient is a function of the speed of storm passage. Estimates of the translational velocity vary considerably from storm to storm and along the path of a given storm. A range from 5 to 65 mph has been observed with the average velocity about 45 mph.

A rather wide range of values for the maximum rotational velocity is found from both measurements and expert estimates. Estimates as high as the speed of sound (600-700 mph) and indirect measurements of up to 485 mph have been made. In addition, the speed of gusts has been estimated to be as high as 100 mph. The variation from storm to storm and within a given storm is considerable. It appears that a reasonable estimate for the maximum speed in an average tornado would be between 200 and 300 mph. The Atomic Energy Commission loading requirements for the licensing of nuclear facilities list a wind velocity of 300 mph [7].

There is even less data on the pressure drops resulting from tornadoes. Estimates as high as 2000 psf have been made and unofficial measurements as high as 400 psf have been obtained. It follows that little data is available on the pressure gradients as well. The AEC requirement is a gradient of 3 psi (432 psf) in 3 seconds.

It has generally been considered impractical to design for the severe loading conditions of wind pressure and pressure differentials experienced during the passage of a tornado. As a result, building code provisions which directly address the tornado problem are non-existent. In view of the current efforts to upgrade building construction procedures and practices to accomplish significant reductions in property losses and human casualties, this situation should and undoubtedly will be rectified. The development of special design regulations for earthquake construction should be paralleled by a similar effort for construction in tornado-prone areas.

An assessment of present building construction indicates that significant improvements in tornado resistance can be achieved without disproportionate cost increases by research in the areas of data acquisition, code formulation and improved building practices. In fact, it appears that reinforced concrete structures and steel frames designed in accordance with recognized standards (e.g. ACI and AISC regulations) can already be classified as tornado-resistant construction. While the basic frame is capable of surviving a storm, the exterior cladding, glass, etc. would suffer severe damage and improved design criteria are needed in this area. However, more basic modifications will be necessary in masonry and wood-frame construction to insure that these structures work together as a unit. The performance of reinforced concrete and rigid frame construction are proof positive of the improved performance which will result. Due to the large number of residences of wood-frame construction, improvements in this area will have a great effect on the total losses incurred due to tornadoes. Areas where new requirements are needed include the following: anchoring of roof to walls; anchoring of the structure to the foundation, connection details in general; the addition of increased lateral stiffness by the use of interior walls as shear walls, additional bracing, etc. An effective means of alleviating damage due to severe pressure gradients is the addition of venting systems to provide a direct release of the internal pressure during the low pressure phase [8]. The development of venting system criteria requires realistic data on pressure gradients.

One of the factors which argued against tornado-resistant design in the past was the high values of maximum velocity and pressure drops commonly quoted for tornadoes in general. Undoubtedly, tornadoes with extraordinary wind velocities and pressure drops do occur but, like any other natural source of loading, tornado severity is a statistical quantity. Once again the need is apparent for reliable data so that design criteria can be established in terms of mean recurrence intervals for pressure drops, velocities and missile impacts for particular regions.

Risk analysis should play a significant role in this area. The anticipated benefits from tornado-resistant design must be weighed against the added costs. The list of benefits should include, in addition to the reductions in annual losses in property damage and human lives, other less dramatic effects such as reduced insurance rates, increased resale value and the intangible value associated with increased public confidence and security.

Careful study will be necessary to set proper levels of tornado resistance. It is unlikely, for example, that any but the most special structures (e.g. nuclear reactors and public shelters) will be designed to withstand the direct hit of a major tornado considering the minute probability of such a storm striking a given point.

A coordinated effort is recommended to gather necessary data, develop design procedures and construction practices and implement the new requirements in practice. It would also seem feasible to institute interim measures for damage reduction prior to completion of the studies necessary for a more complete understanding and treatment of the problem.

Specific recommendations for this section on severe storms are summarized below:

(1) The forecasting of hurricane paths must be improved.

(2) Modification effects of seeding hurricanes by both field experiments and computer studies should be established.

(3) The feasibility of a sensor system for gathering data on actual tornadoes should be investigated.

(4) Existing data on severe storms should be collated and assembled in a single document relevant to structural design problems.

(5) An interdisciplinary field team should be formed to assess wind damage for useful data regarding severe storms.

Public Safety and Protection

This discussion is concerned with practical measures to (a) obtain knowledge through instrumentation studies and field inspection of wind storm damage, and (b) effect increased protection to life and property by the recommendation of improved procedures and their implementation through building codes and specifications. Overall, it is recommended that a central agency, perhaps a national disaster agency, be authorized to provide coordination in the area of wind data acquisition and standardization, post-storm damage surveys, and formulation of building code revisions. Other important functions would include monitoring of related research projects, dissemination of information, educational activities associated with new codes and procedures, and cooperation with other national and international groups.

Instrumentation

A coordinated program of instrumentation for wind effects should be established to eventually replace the present piece-meal efforts. Steps should be taken to obtain maximum return from the funds available, i.e. results should be of benefit to all interested parties--statisticians, researchers, designers and building code groups. The data gathering process should be generally upgraded [9]. Consistent standards should be established for basic wind measurements, e.g. a uniform averaging period should be used for all official observations. Economic, efficient and, preferably, portable instrumentation packages should be developed and discrepancies due to instrument differences eliminated. Measurements should be taken in a form suitable for automated data analysis and the number of available instruments should be greatly increased. Selected buildings should be instrumented with measuring devices that are automatically turned on at the onset of a severe storm. Finally, public cooperation must be encouraged in the area of tornado observations. In particular, programs should be established for the purchase of valuable photographs and motion pictures of tornadoes.

Basic wind measurement studies should include the continuation of climatological programs, and the gathering of information of severe storm characteristics and occurrences. Specific programs to determine wind profiles and turbulence spectra in cities, especially under storm conditions, should be initiated without delay.

A good deal of valuable information must also be obtained on wind-structure interaction effects and basic structural characteristics such as internal damping levels. It would be valuable to establish data on significant dynamic effects such as vortex shedding, aeolian vibrations, galloping, and interference effects for various standard sites. Systematic field tests combined with wind-tunnel investigations should establish valuable design guidelines applicable to a large number of common situations. Also, the emerging stochastic design method requires that rather sophisticated statistical data be available on the wind environment. Close correlation between wind tunnel results and on-site behavior will be necessary in order to develop this data.

Assessment of Wind Damage

One of the most valuable sources of data is the inspection of damaged areas immediately following a severe storm. The existing program for sending disaster survey teams should be continued and strengthened. Maximum benefits will result from the ready availability of survey teams composed of experts from a number of technical and non-technical (e.g. economists, lawyers) disciplines. The cooperation of local officials and the public is essential for these efforts to be valuable.

Post-storm inspection reports should include a description of the storm itself (path, intensity), and the type and degree of damage to specific structures [10, 11]. Where possible, general observations regarding the response of various classes of buildings and types of construction, should be made. In some cases detailed analyses may be justified in order to determine wind forces from inspection results or for the study of the dynamic response of a particular structure. In addition, recommendations on improved building practices, based on the observed damage, should be made. Conclusions regarding the actual performance of wind-resistant construction together with economic analyses of resulting loss reductions, should provide valuable feedback to risk analysis studies and to the financial and insurance aspects of the wind problem.

Building Codes and Specifications

In order to accomplish reductions in losses from severe storms, significant results and recommendations from field, laboratory and theoretical investigations must be incorporated into building codes and specifications. It is urgently recommended that the major codes of practice be updated continually to refer to the most current sources of wind loading information [12, 13]. In addition, guidelines and background information covering all aspects of wind resistant design, from risk analysis through occupant comfort and construction practices, should be compiled for convenient usage.

Basic code modifications, such as inclusion of reliable stochastic design methods and upgraded climatological data, will no doubt require time for implementation. However, there are a number of practical steps which can be taken immediately to improve code provisions for design against wind forces. Among these are: (1) stringent requirements for connection details to insure that the structure and its parts are sufficiently anchored; (2) provisions for increased lateral stiffness for structures which previously were primarily designed for vertical loads only; (3) requirements for local suctions for cladding design to reduce costly and dangerous glass breakage;

and (4) provisions for the temporary bracing of new structures throughout each phase of their erection.

Adequate provisions for control of construction operations and for modifications of buildings by occupants are required in building codes to ensure that such operations or modifications do not create a wind hazard.

Protection from Wind and Storms

Improvements in the area of storm forecasting and path prediction are urgently required to provide increased warning time to residents of threatened areas. In addition, it may be possible as a result of basic research to modify or divert storms before they strike heavily populated areas.

Other areas where basic meteorological data and predictions should be fully utilized are that of regional planning and local zoning. Effective steps must be taken, on both the regional and local levels, to prevent private interests from improperly developing areas whose geographical location, topographic features or proximity to large bodies of water, make them particularly susceptible to wind-induced damage.

Finally, special studies should be made in other areas where improvements in storm protection are possible. For example, the use of ground level portions of buildings for storm shelters should be investigated for cases where a storm cellar or other shelter is not available. Post-storm surveys often reveal a small room, closet or bathroom left intact while the remainder of the structure was destroyed. Also, special attention should be paid to the wind resistance of communication structures, such as telephone poles, which play an important role in the maintenance of public services under severe storm conditions.

The above recommendations may be summarized as follows:

(1) A comprehensive program for development and installation of instrumentation for measuring the loadings and response of selected types of structures, and for recording meteorological data should be accelerated.

(2) Post-storm survey programs should be continued and strengthened, followed by comprehensive reporting, including a description and analysis of the physical damage and economic analyses for future planning and design.

(3) Codes and specifications should be updated to reflect the best information available on wind loads and effects. Design guides and criteria to supplement codes should be prepared.

(4) Continuing efforts should be expended to improve the forecasting of severe storms and their areas of traverse. Better warning systems are urgently needed.

(5) Further attention should be given to development of control, operations and management, prior to and during severe storms and to development of recovery plans and stockpiling of supplies for recovery operations in the event of natural disasters.

(6) The basis for insurance protection against wind damage should be broadened and liability considerations should be delineated.

(7) A national data center should be established to provide a central base for obtaining information on meteorological storm data, storm damage and economic data, codes and design guides, regional planning

and other critical items pertinent to providing protection against natural disasters.

(8) A comprehensive program of education should be developed for practicing engineers, architects, code authorities as well as for students, to present continually updated knowledge for design and planning against severe storms.

Structural Analysis and Design

The objective of any design approach to wind loading must be the prediction of those modes of structural performance which in one way or another impair the intended usage or serviceability of the structure. For buildings, the major factors governing the design against wind are as follows:

Failure due to:

(a) Yielding with excessive permanent deformations.

(b) Fatigue.

(c) Instability of the structural frame or widespread damage of the facade or cladding; i.e., the "skin".

Unserviceability due to:

(a) Excessive deflections causing cracking of walls and ceilings, plus degradation of the structural skin and mechanical systems.

(b) Breakage of glass.

(c) Excessive sway accelerations resulting in discomfort of occupants.

(d) Discomfort to pedestrians in areas surrounding the building.

It is obvious that the return period in years or mean recurrence interval on wind speed that would bring about failure should be extremely large and much greater than the recurrence interval which renders the structure unserviceable.

While the past practice has been to use recurrence intervals of the order of 50 years for the overall strength design of buildings against wind loading--based more on historical factors in the development of wind loading than on rational consideration of the loads that the structure should sustain--the current practice is to use a design recurrence interval which accounts for the intended operational usage and life of the structure as well as the degree of wind sensitivity and risk of human life. In the new ANSI Standard [12], a 50-year mean recurrence interval is required for the design of all permanent structures except those that, in the judgement of the engineer or authority having jurisdiction, present a high degree of wind sensitivity and an unusually high degree of hazard to life and property in case of failure. In the latter case, a 100-year mean recurrence interval is required. For structures having no human occupants or where there is negligible risk to human life in case of failure, a 25-year mean recurrence interval is permitted.

Obviously, many structures have no human occupants, but are wind sensitive. In such cases, the new Standard is not explicit and the choice of a design recurrence interval must be based upon judgement. Considerations of safety (or risk) require that the design wind speed must not be underestimated while it would be uneconomical to grossly overestimate this speed.

Wind sensitive buildings are defined in the new Standard as being taller than 200 feet and having a ratio of height to least width greater than 4.

There are no provisions for tornadoes in the new Standard.

It should be noted that the mean recurrence interval on wind speed for failure of a building or structure is much greater than the design recurrence interval when conventional factors of safety are employed. In addition, because complex buildings consist of a considerable number of effectively independent components, the overall probability of failure is greatly reduced when the probabilities of failure of the individual members are combined. Even in simple structures, combination (convolution) of the probability of substandard structural members will increase the mean recurrence interval for failure greatly.

The above may appear to be incongruous because, on the one hand, structures are designed with adequate factors of safety above that required to cover variations in material and workmanship, while, on the other hand, failures are not uncommon. The answers appear to lie, at least partly, not only in the differences between the design wind loads and the actual wind loads to which structures are subjected, but also in possible "weak" or brittle points in structures which prevent them from developing their potential ductility. Examples are inadequate anchors for the roofs and curtain walls of buildings plus tie-downs for mobile homes. Normally, the strengthening of potential weak points can be accomplished at a relatively small cost and is tantamount to a great increase in overall factor of safety.

At present, design recurrence intervals on wind speed are obtained from one Code or Standard (such as ANSI) while factors of safety and load factors for structural design are obtained from other codes (such as AISC and ACI). It must not be overlooked, however, that wind loads and structural strength are both probabilistic and a rational design can be achieved only by considering the two in combination. In view of the current trend towards taller and lighter structures utilizing high strength materials and overall lower safety factors, it would be unconservative to rely solely on favorable unevaluated experience with structures of the past in establishing wind design criteria for future structures. Even the experience for modern wind sensitive structures is very limited since the loading and design criteria have been in a state of flux over the past 20 years. There is an urgent need, therefore, for better cooperation among different code writing bodies in order to establish realistic wind design criteria for combined loading and to deduce more nearly predictable probabilities of failure.

In the new ANSI Standard for minimum design loads for buildings and other structures, the procedure for calculating wind loads has been reformulated to facilitate the application of new and more sophisticated wind load design criteria. The new provisions do not, on the average, result in different wind loads than have been used in the past, but rather permit the design of buildings and structures for wind to be performed on a more realistic and rational basis, consistent with the building dynamic characteristics, location and intended operational usage and life. Major innovations are, (1) varying load requirements both with the dynamic properties of the building and with the roughness of the terrain, and (2) separate load requirements for the main frame and for exposed elements such as curtain walls and glass.

The new Standard includes new distributions of extreme fastest mile wind speeds in the United States based on National Weather Service data. The new data results from the extension of the annual extreme wind speed data series to the longer records available at airports. Maps are given for 25-, 50- and 100-year recurrence intervals so that a suitable wind speed can be selected for design as described above.

Whereas the old Standard specified the 1/7 power law variation of wind velocity with height for all terrain, without distinction, the new Standard presents different wind profiles and different gust response factors and ground wind speeds for three different exposure conditions.

In applying the new Standard the calculation of wind loads involves: (a) the choice of a basic or reference wind speed with a suitably large mean recurrence interval, and (b) the use of three numerical factors, an exposure factor depending upon the roughness of the terrain, a gust response factor depending upon both the roughness of the terrain and the dynamic response characteristics of the building, and pressure coefficients which relate the external and internal pressures and/or suctions on the various surfaces to the effective dynamic pressure of the turbulent wind. The pressure coefficients for local design of cladding and curtain walls have been updated to reflect the current experience regarding numerous structural failures due to wind forces related to inadequate anchorage of individual panels and roof elements.

The statistical approach to wind loading seems to be the most promising and realistic method for formulating design loads for wind resistant design, and the new ANSI Standard is a step in this direction. Although the annual extreme fastest mile wind speed is used as a basis for determining loads, the gust response factors in the Standard are based on stochastic analyses of the wind speed fluctuations over 60 minute averaging periods, which provide the necessary constancy and stability for treating wind loading on a statistical basis. All factors were then referenced to fastest mile averaging periods in order to maintain consistency with the accepted American practice of using fastest mile wind speed data in design calculations.

While the new ANSI provisions on wind loading are expected to result in designs for broad classes of buildings and other structures of more uniform safety and of greater economy, specialized buildings will normally require the use of wind tunnel tests and detailed climatological investigations to assure that the design criteria is both safe and realistic.

For buildings having dynamic properties which tend to make them wind sensitive, the design procedure should include a dynamic analysis to determine the response of the building to wind gusts. This dynamic analysis should determine as a minimum the fundamental frequency of the building. A knowledge of the fundamental frequency, together with the wind gust spectra of the basic wind, permits the design for wind to be carried out on a dynamic basis using gust response factors [15]. The gust response factor is analogous to the seismic coefficient or "g" factor used in earthquake design procedures.

In some areas of the country (e.g. southern California) the code wind forces exceed the code earthquake forces. The local codes specify static wind forces to be used in the design of the building and also specify minimum equivalent static earthquake forces. However, the large investment in a tall building and the cost of repairing earthquake damage dictates more thorough dynamic analysis of the response of the structure to earthquake ground motions of realistic intensity. Because of this design procedure the strength and form of the structural frame is determined by the earthquake induced deformations rather than by the code prescribed wind forces. It is suggested that if similar dynamic design procedures were used to determine the response of buildings to wind forces of realistic intensity, then the design of the building would be more closely tailored to the actual loading and the net result would be a design of more uniform safety and thus of greater economy.

The significant problems encountered in wind resistant design are as follows: (1) overturning forces; (2) drift and distortions; (3) curtain wall, cladding and glass design; (4) anchorage of small structures including light frame buildings; and (5) serviceability requirements.

Overturning forces are not only of considerable concern in regard to safety, but are very significant in their effect on building drift. Reliable prediction of these forces is essential to providing building economy and performance for wind. The prediction and control of drift is critical to estimating dynamic response and damage control for buildings subjected to severe winds. The effects of interfloor drift are dependent upon the structural materials and construction techniques used, and therefore research into the establishment of a design criteria for drift must keep pace with the development of new materials and/or methods. Static wind design procedures are not satisfactory for predicting permissible horizontal movements or drifts of buildings. Drift stability calculations, particularly for a tall structure, should consider all contributory factors including the effects of column axial deformations, beam and column bending, possible local yielding or foundation settlement and the secondary movements induced by the interaction of gravity loads and lateral sidesway (P-Δ effect).

Although failures of such items as window panes may not lead to collapse of the structural frames, it is important that the design procedure include a rational procedure for design of curtain walls and cladding [14, 16, 19] design. Damage or failure of these items under severe winds can be catastropic in order of magnitude in terms of dollars and possibly human lives. As part of the design procedure for buildings against wind, the statistical nature of structural resistance must be taken into consideration since it is well known that yield strength of structural steel, fracture strength of window panes, strength of reinforced concrete, resistance of structural joints, fatigue resistance of most structural elements, etc. exhibit considerable statistical dispersion.

In addition to strength and stability requirements, certain serviceability conditions have to be considered in designing for wind loads. These requirements are intended to ensure the satisfactory performance of the structure under service conditions. The most significant serviceability criteria relate to:

(1) Lateral deflection or drift of the building, particularly as it affects the stability and cracking of members.

(2) Relative vertical movements between columns, particularly as these affect the cracking of members and partitions.

(3) Motion of the building as it affects occupancy comfort.

The degree of control of structural cracking under lateral load depends upon the type of loading. For wind loading the aim should be to keep the cracking within acceptable limits whereas for severe earthquake loading, buildings may be expected to develop plastic hinges in the beams and at column faces.

Stochastic Methods

Stochastic vs. Traditional Design Procedures

Present design practices based on nominal or theoretical values of resistance and load leave the actual safety completely unspecified. The designer has no knowledge of the structure's reliability at any stage from partial failure through total collapse. Consequently, efforts are underway to establish probability-based design procedures [20] to eventually supplant present techniques. In stochastic design both loads and resistance are considered realistically, i.e. as random processes. A formulation results in which quantities are specified in statistical terms, data and testing requirements are more severe, and more sophisticated procedures are employed. The result, however, is a rational approach from which meaningful statements can be made at all levels of structural performance [21]. Moreover, the results can be stated in terms appropriate to risk analysis, i.e. the design decision process which considers structural performance and economic factors concurrently.

Assessment of Structural Integrity in the Wind Environment

Various techniques are available for evaluating the response of a structure to a random wind loading. Input to the problem includes the dynamic characteristics of the structure and the statistical description of the wind environment [22, 23]. Required wind data include the wind speed spectra, i.e. the frequency-wise distribution of wind speed and the structure of the gustiness specified in terms of the following: the power spectra distribution of fluctuations in the wind; the correlation between velocities at different points in the flow; coherence and the probability distribution of velocity fluctuations. Since data from standard weather bureau sites would rarely be sufficiently representative of the actual site condition to justify its use in a sophisticated analysis, meteorological studies of the site are required. Wind tunnel tests on an accurate topographic model in a boundary layer wind tunnel are the most efficient means of gathering the required information. Ultimately this wind load data should be generated for a variety of standard sites including the center of a large city. In this way, wind tunnel tests would be performed only in cases where the site peculiarities or the structure itself dictated that a special climatological study should be made. The vital role that wind tunnel tests play in this entire development emphasizes the need for a program to improve and validate wind tunnel results by comparison studies with full scale tests for all aspects of wind tunnel testing.

The response of the structure to the site environment can then be evaluated either experimentally or analytically. It should be noted that random variability in the structure itself must be considered in the analysis. With respect to wind tunnel testing, it is not feasible to include random building effects in the test program. Random material properties could conceivably be treated in an analytical development along with the random load but would add considerable complexity. As an alternative it may be possible to adjust the random structure response to reflect inherent structural variability by considering the results obtained by others on the effect of material property variation on overall structural response.

Boundary layer wind tunnel tests on an accurate aeroelastic building model can provide the necessary data on structural response to wind and in the process account for the sequential effect of gust application, the spatial extent of gusts and the three-dimensional aerodynamic interaction between the structure and the actual wind environment.

Analytical procedures for evaluating random response are presently available for a relatively small number of special cases. More sophisticated statistical dynamics approaches including correlation methods are available for treating more complex cases. These techniques should be developed to consider the elastic response of three-dimensional bluff bodies to random excitation. An independent approach by Monte Carlo procedures can treat the statistical variation in both loading and material properties in one analysis by digital simulation.

Full scale tests should be performed to verify the assumptions made in the wind tunnel and analytical approaches and to develop confidence in the ability to define the random structural response.

The random response results can be related to design criteria in various ways. For example, the maximum drift deflection anticipated over a given mean recurrence interval can be established, or, in terms of damage prediction, the expected number of oscillations with maximum deflection greater than a limiting value, can be found. Similar statements can be made regarding the possibility of severe damage or collapse, floor vibration, glass breakage, cumulative fatigue damage per year, etc.

Finally, based on the amount of rather complex wind tunnel or analytical analysis necessary to obtain the results at this stage, it appears that this treatment would only be justified for a fairly significant structure. As experience and expertise develops, simpler techniques and appropriate design guidelines should be established.

Risk Analysis

The next step in this design process is the establishment of the acceptable level of risk for each design criteria, i.e. actually the probability of unserviceability or the probability of failure. A great deal of judgment is necessary to establish proper levels for the risk criteria since they should represent a balance between structural safety and overall cost considering both the economic and social consequences of failure. For example, it would seem that a general criterion could be established that the mean recurrence interval corresponding to the collapse load must be longer than the life of the structure. However, this rule does not necessarily hold in the case of a structure in the path of a major tornado since economics may dictate a higher risk level for the great majority of structures.

A great deal of effort must be expended before risk criteria can be established with confidence. Detailed data must be made available, e.g. the following: statistics on the strength of glass are needed to arrive at meaningful cladding design criteria; data on human perception and tolerance for motion are required in order to set limits on tolerable floor vibrations and building drifts.

Consideration should be given to the use of Bayesian concepts in this work. This approach has proven to be valuable in other areas by providing a logical framework for decision making considering cost optimization and proper utilization of past experience and subjective data.

Establishment of Specifications on Wind Design in Statistical Terms

The incorporation of statistical concepts into design codes is an active area of which wind loading provisions are only one part. Some of the requirements for this new code formulation are:

The essential features of a probabilistic code should be maintained but technical difficulties should be minimized.

Safety levels or risk criteria must be established and guidelines provided so that this legal obligation is not placed upon the designer.

Safety levels must be high enough to protect the public but not so strict that intolerable construction cost increases result.

The code should cover all common structures and be sufficiently flexible so that no restrictions are placed on special structures or more detailed studies.

Codes must be readily amendable for updating as new data is acquired and new practices evolve.

Regarding wind data, detailed information should be included for a

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variety of typical site conditions.

It goes without saying that a good deal of work remains to be accomplished in order to bring this new design formulation into active use.

Summary of Major Recommendations

As a final summary it is recommended that the following tasks be given priority:

1. A central agency should be authorized to coordinate the wide range of required activities in an overall disaster mitigation program.

2. Basic studies on the origin and behavior of severe storms and effective measurement programs to obtain reliable data on tornado loads should be encouraged. The economic feasibility of a sensor system for gathering data on actual tornadoes should be investigated.

3. The modification effects of seeding hurricanes by a combined effort of field experiments and computer simulations should be determined.

4. Design criteria and building practices for tornado resistant construction should be developed for future implementation.

5. Major codes of practice should be updated continually to refer to the most current sources of wind loading information.

6. Current knowledge regarding hurricanes and tornadoes should be compiled into a single document relevant to structural design against natural disasters. Design guides and criteria to supplement building codes and specifications should be prepared.

7. The development of stochastic design procedures should be encouraged, i.e. the necessary analytical and experimental techniques and probability-based design provisions including risk criteria.

8. Statistical data on loading and response characteristics of buildings should be collected for implementation of the stochastic design approach.

9. A comprehensive program of education should be developed for architects, engineers, builders, students, and code officials, to present continually updated knowledge for planning and design against natural disasters.

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CRITERIA FOR BUILDING SERVICES AND FURNISHINGS

by

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and

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A. Introduction

A serious need exists to develop improved building practices to minimize disaster induced failures to building services, such as elevators, lighting, air conditioning, plumbing and fire protection systems, and furnishings, such as suspended ceilings, partitions and storage racks. This is especially true in buildings that must be safe and useable after a major emergency, in order to preserve the order and safety of the general public. This discussion will be limited to earthquake resistive design, because seismic induced forces on these nonstructural elements very likely exceed those caused by hurricanes, tornadoes, extreme winds and blast or explosion effects.

In the past, earthquake resistant design has been concerned primarily with the structural integrity of buildings, and very little attention has been given to the performance of the nonstructural elements. This has been based on a design philosophy that the structural frame will be deflected by the seismic forces and some inexpensive "cosmetic" damage to nonstructural elements will occur. Until recently [Reference 1], however, the extent of this damage and its effect on the safety of the building occupants was never fully evaluated. One of the most significant lessons learned from the moderate San Fernando earthquake, where most buildings suffered no structural failures, was the magnitude and cost of the nonstructural damage [References 2, 3 & 4 refer to the problem and more detail is presented in Reference 5. Damage cost statistics on high rise buildings in the Los Angeles metropolitan area are currently being collected].

The development of corrective measures to reduce this type of damage is restricted somewhat by our lack of knowledge regarding the behavior of buildings during earthquakes, but most of the problem stems from traditional divisions of responsibilities between design professionals, and a construction industry that does not require careful detailing of nonstructural elements. The structural engineer usually limits his attention to the design of the structural frame and leaves the detailing of the nonstructural elements to others on the design team. This generally results in no analysis at all, because the architect and the other engineers responsible for this design work, either ignore the problem or lack any real knowledge of its importance or how to handle it. Many of the installation details of nonstructural elements are deliberately omitted from drawings, because of long standing trade practices that have left many of these decisions to product manufacturers and installers. To overcome these problems the structural engineer must lead the design team to see that all nonstructural elements are fully analyzed and then detailed or carefully described in the specifications. The entire design team must then vigorously defend these details from contractor proposed alternates that do not meet the design intent, and then demand that they be properly executed in the field.

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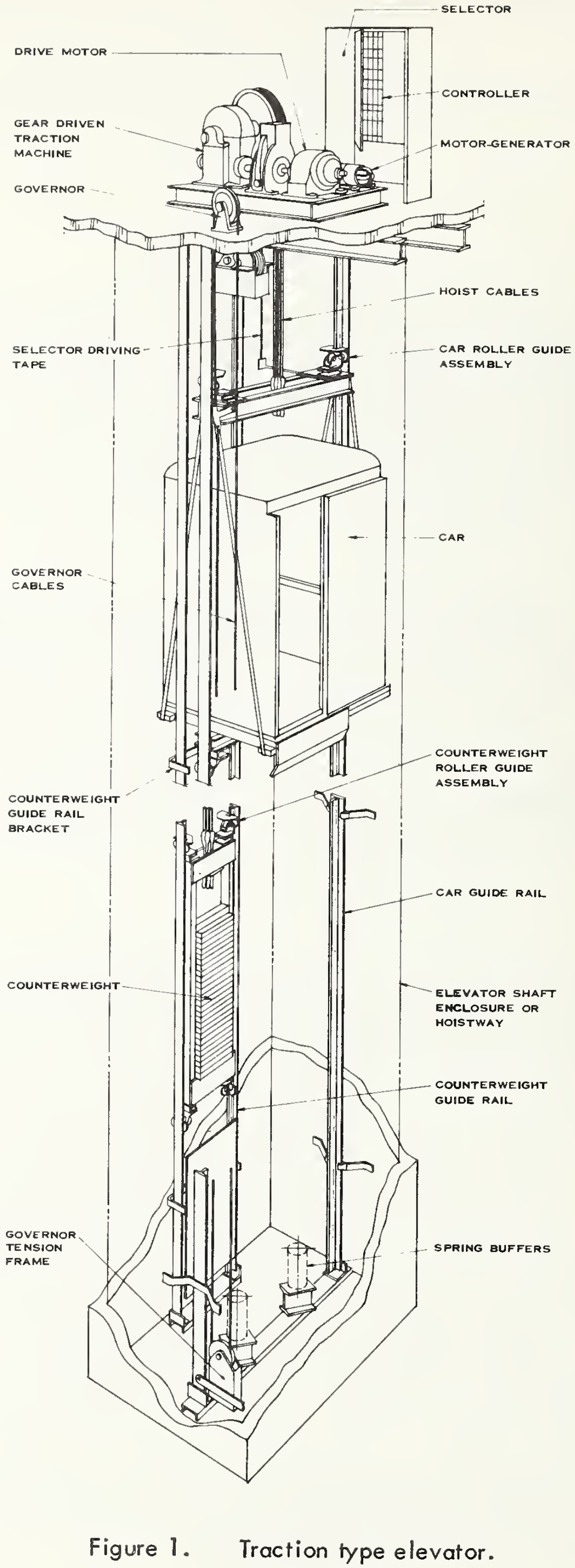

Figure 1. Traction type elevator.

Most building owners, and unfortunately their architects and engineers, consider building code minimum requirements as adequate protection against earthquake damage, and they will not increase their capital costs to improve occupant safety or reduce future repair costs. This firm belief in the infallibility of building codes is usually badly shaken after each earthquake. But memories are short and the magnitude of repair costs and other post earthquake difficulties with buildings are not made public, so owners usually resist added costs for earthquake resistive features that are not spelled out in a code. Even though we recognize the serious problems in writing codes and their practical enforcement difficulties, we recommend stronger legislative controls over the work of architects and engineers, methods of plan checking and approval, and the procedures for construction supervision and inspection.

In the following discussion we have limited ourselves to elevators, mechanical systems, ceilings, light fixtures and storage racks. In each category, the subject matter has been further subdivided for clarification and we have used a few drawings to illustrate and support the text. The suggested corrective measures do not include minimum recommended design forces. These data are included in various proposed code revisions and in References 6 and 7.

B. Elevators

Elevators in buildings are primarily the traction type illustrated in Figure 1, because hydraulic elevators are limited by speed and design to low buildings. Traction type elevators suffer most of the damage during earthquakes while hydraulic elevators, dumbwaiters and escalators survive with only minor damage. The damage summary on approximately 2,000 elevators compiled by the State of California Division of Industrial Safety after the San Fernando earthquake, see Table 1, provides an excellent indication of the vulnerability of elevator systems to earthquake forces. Note that 674 counterweights were thrown out of their rails and 109 of these struck cars moving in the opposite direction. Similar damage to elevators has been reported in other earthquakes [1].

Most cities in our country do not have an elevator code or even elevator inspectors in their building departments. They depend on manufacturers adherence to the requirements of the American Safety Code for Elevators, Dumbwaiters, Escalators, Moving Walks, and to the integrity of the elevators installers to protect the life and safety of building occupants. Some of the larger cities do have elevator codes and inspectors, but these codes do not include requirements for earthquake resistant design.

The following comments relate to components of traction type elevators.

1. Counterweights and Guide Rails:

During the initial shocks of an earthquake, the heavy (7,000 pound average) counterweights bend or break their roller guides and deform their guide rails. They become derailed and as the earthquake progresses the free swinging counterweights inflict additional damage on guide rails, brackets, spreader beams and cars. In many instances, the loosened counterweights have hit the roof of cars moving at high speeds in the opposite direction.

A relatively simple corrective measure is to strengthen the guide rails and install safety shoes on the counterweights similar to that shown on Figure 2. Under normal operation, the safety shoe would ride free of the rails, but during an earthquake the roller spring would give way to allow the shoe to contact the rail and carry the lateral forces to the guide rail. The

Table 1

ELEVATOR DAMAGE DUE TO SAN FERNANDO EARTHQUAKE

Item	Description	Numbers
Buffers	Stinger damaged, guides damaged	2
Cables, compensating	Out of grooves or damaged	100
Cables, governor	Hang up, cut	20
Cables, hoisting	Damaged, jumped deflect sheave (sets)	7
Cables, traveling	Hung up or broken	7
Cars	Out of guides, out of alignment	18
Cars	Steady rest loose	102
Controllers	Moved or damaged	5
Counterweights	Out of guide rails	674
Counterweights	Out of guide rails, damaged cars	109
Electrical power problems	Various causes	4
Electrical	Conduit loose	5
Generators moved	Some damaged armatures	174
Generators	Burned out	5
Guide rails, car	Limits not operable, doors not working	7
Guide rails, counterweights	Out alignment, bent or broken	49
Guide rails, counterweights	Brackets broken or damaged	174
Hoistway doors	Off tracks, gibs out of sill, draffing	22
Hoistway doors	Glass damaged	2
Hoistway walls	Bowed and hitting car	2
Hoistway walls	Severe cracks, loose plaster and holes	50+
Hydraulic	Leaks in casting, plunger rubbing	8
Interlocks and car gate contact	Loose, broken	19
Leveling units	Damaged	12
Machine room	Floor plates broken, buckled	1
Miscellaneous	Annunciators off, brake rods out, lights, broken selector tapes, fuse blown, door operator	83
Motor, hoisting	Burned out, out of alignment, slipped rings	13
Pits flooded	Broken pipes, etc.	7
Pump units	Moved, reset leaks	1
Roller guides, counterweights	Broken or loose	286
Safeties set		22
Selector	Turned over	1
Sheaves, drive	Broken or cracked	3
Sheaves, counterweights	Broke loose, weight in pit, moved	2
Shoes, guide	Broken	9
Victaulic fitting	Replaced	1
Dumbwaiters: Cables	Off drum	1
Counterweights	Out of rails	3
Escalators: Conduit	Pulled out	1
Governor chain	Off	1
Switches	Skirt switches tripped, skirts spread	5
Truss bolts	Sheared off	5

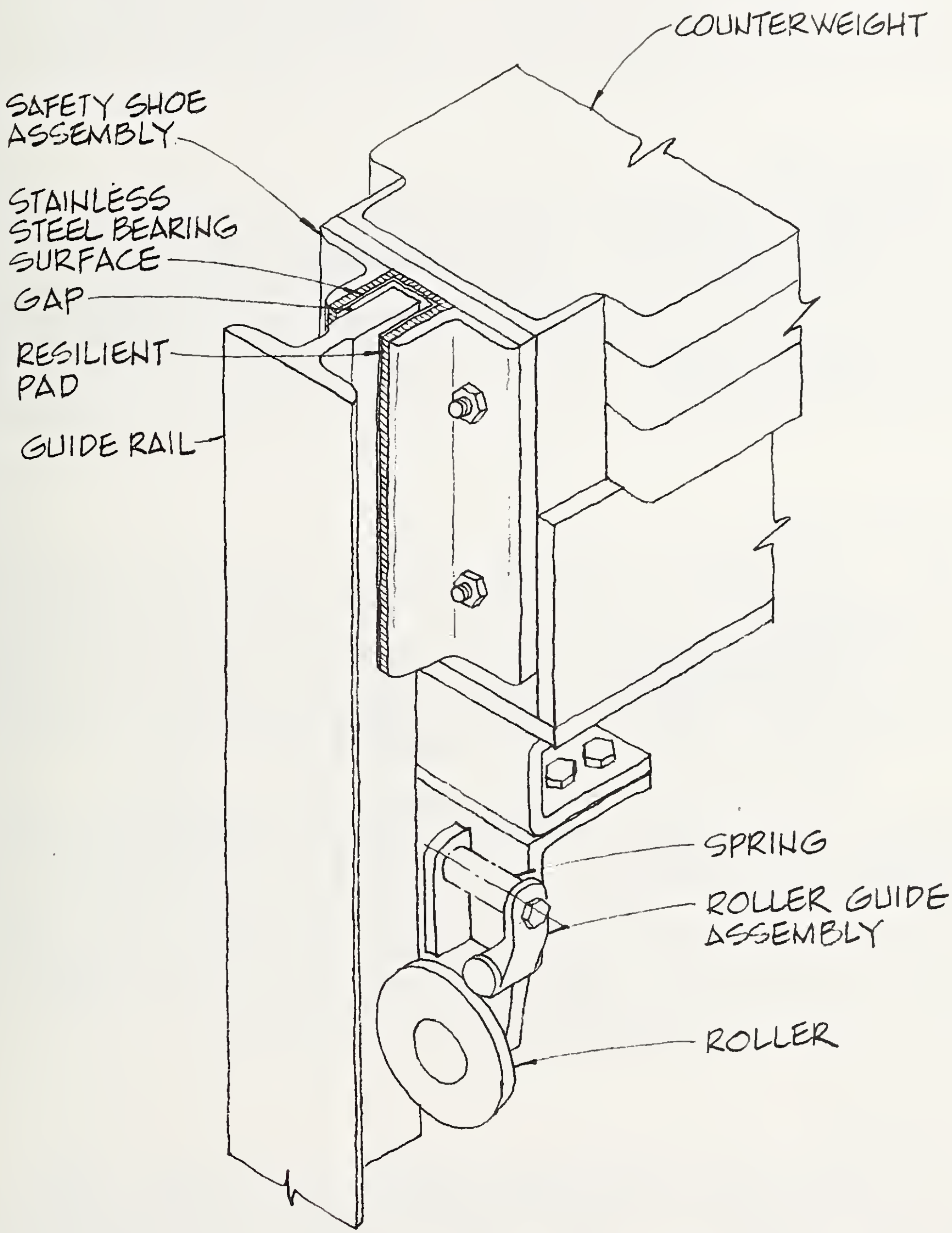

Figure 2. Proposed counterweight safety shoe.

guide rails should be strengthened by using heavier than the typical 8 pound per foot rolled section. Brackets should be strengthened by the use of gusset plates or ties similar to that shown in Figure 3, and placed at more frequent intervals.

Seismic switches to shutdown the elevator during an earthquake and then lower the cars to the nearest floor should be considered. Manual override switches are needed for slow speed operation by rescue and maintenance personnel after the earthquake. A more direct solution to the counterweight problem could be a safety switch mounted on the counterweight guide rail assembly to shut down the elevator when the counterweight is derailed. Signal transmission using wireless methods should be considered to avoid additional wires in the hoistway.

2. Cars and Guide Rails:

Other than damage from derailed counterweights, guide rails for cars sustain very little earthquake damage primarily because they are heavier than the counterweights guide rails. They are designed to withstand eccentric live loads in the car during loading and unloading plus braking forces from the car safeties, and they are fastened directly without brackets to the structural frame either to the walls of the elevator hoistway or to heavy spreader beams. It is advisable to strengthen the car guide rails on long spans by installing spacers between the back-to-back rails at midpoints between the spreader beams, as shown in Figure 4, to increase the rigidity of both rails.

3. Cables and Hoistways:

The hoist, compensating, and governor cables are tossed around in the hoistway during an earthquake. They become twisted and snarled, jump out of their sheaves or guides, and inflict secondary damage when they catch on limit switches and other exposed equipment. Corrective measures that should be considered include sheave guards to contain cables, and guards on all rail brackets, ends of landing sills, door-driving mechanisms and other exposed equipment. Manufacturers should be urged to develop new designs that minimize the use of loose traveling cables in hoistways.

Hoistway construction varies from reinforced concrete used as a structural shear wall, to gypsum board or lath and plaster to form the required fire rated vertical shaft enclosure. The deflection of the building frame during an earthquake, with some help from free swinging counterweights, cracks and breaks up the shaft walls. Debris falling onto the tops of cars inflict secondary damage, and the movements around the elevator door openings leads to jammed and misaligned doors. Suggested corrective measures include the design of more rigid structural frames around hoistways, and door frames that can accommodate the predicted maximum interstory movements. A defensive solution to falling debris is a screen mounted on the top of cars.

4. Motor Generators:

Motor generators must be mounted on vibration isolation mounts to inhibit noise transmission to the occupied areas of the building. During an earthquake these isolators fall apart, and the generators are tossed around the equipment room tearing loose their electrical connections and inflicting secondary damage on other equipment. A simple and direct corrective measure is to use molded neoprene isolation mounts, and bolt them to the legs of the generator and to the floor of the equipment room.

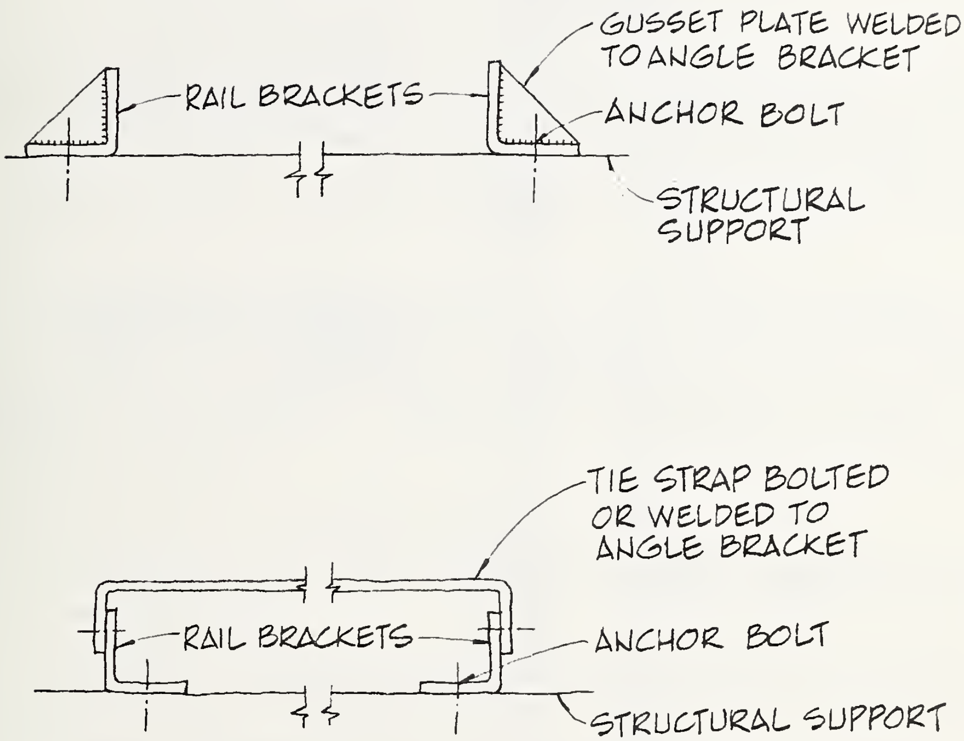

Figure 3. Proposed counterweight brackets.

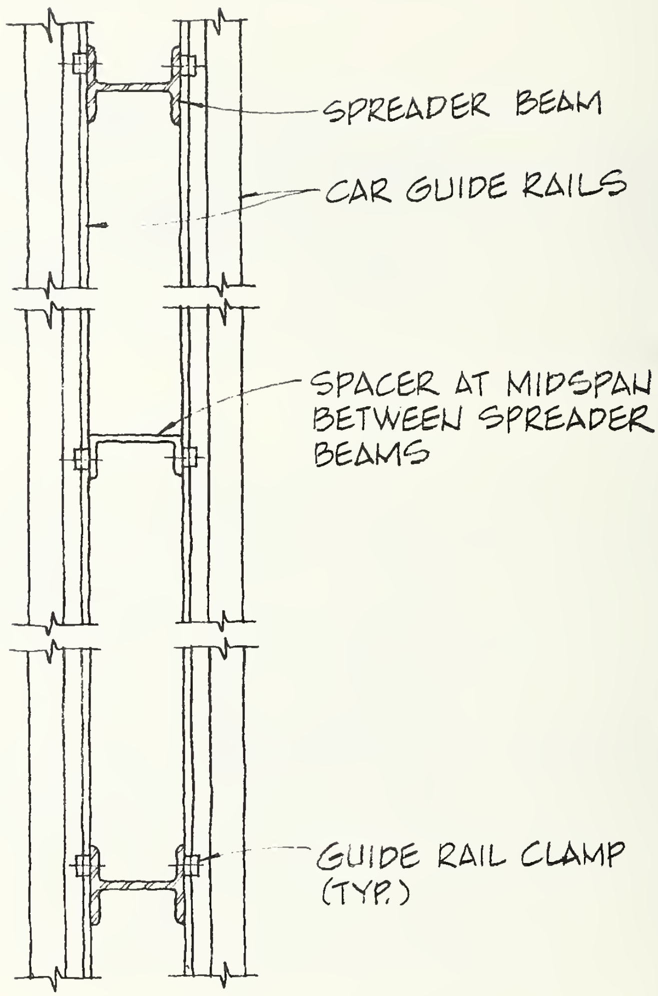

Figure 4. Proposed spacer between back-to-back car guide rails.

5. Drive Motors and Traction Machines:

The motors and drives are usually mounted on steel bases and bolted to the floor, to assure good alignment with the cables, and vibration isolation mounts are not required because they operate at slow speeds. The reports of burned out motors, misalignments and slipped rings after earthquakes relate to the internal design of the unit, and suggests a need for manufacturing seismic standards.

6. Selector and Controller Panels:

These panels sustained very little damage during earthquakes if they are bolted to the floor and provided with sway braces at the top. All electrical components within the panels should be properly secured and hinged panels provided with positive locking latches.

Most of the hydraulic elevators survive earthquakes with very little damage because they are essentially hydraulic jacks consisting of a fixed cylinder and a movable plunger that is attached to the elevator car. Damage is usually limited to casings out of plumb or with leaks due to ground movements, oil pumps and tanks that shifted causing oil spillage, and minor pipe leaks. This type of damage is easily avoided by properly securing equipment to floors and/or walls and using splash proof oil tanks.

Dumbwaiters survive earthquakes with only minor damage, usually derailed counterweights. Escalators also perform well because they are essentially two welded steel trusses that span between floors to house the tracks, drives and motors. Damage is limited primarily to sheared truss bolts at floors and some secondary damage to skirts, switches and governor chains. It should be noted that this form of vertical transportation, unlike the elevators, even though damaged by the earthquake, remain in place and serve as an emergency exit for building occupants.

Other more generalized recommendations include:

1. At least one elevator plus the ventilation, communication and lighting systems on all elevators in a building, connected to emergency power.

2. Establish preplanned emergency procedures for checking and restoring damaged equipment.

3. Educate building managers and occupants regarding emergency procedures for checking and restoring damaged equipment.

4. Educate building managers and occupants regarding emergency procedures.

5. After each earthquake require detailed damage reports from owners and elevator repairmen.

C. Mechanical Systems

Mechanical systems--boilers and flues, pumps and piping, tanks, fans and ducts, compressors and chillers, cooling towers, etc.--generally survive earthquakes with only minor damage. Heavy tanks will shift or fall over if their supports fail, and heavy unanchored equipment will move; but most of the damage to mechanical systems in modern buildings is caused by the use of vibration isolation devices. Additional research is needed to satisfactorily

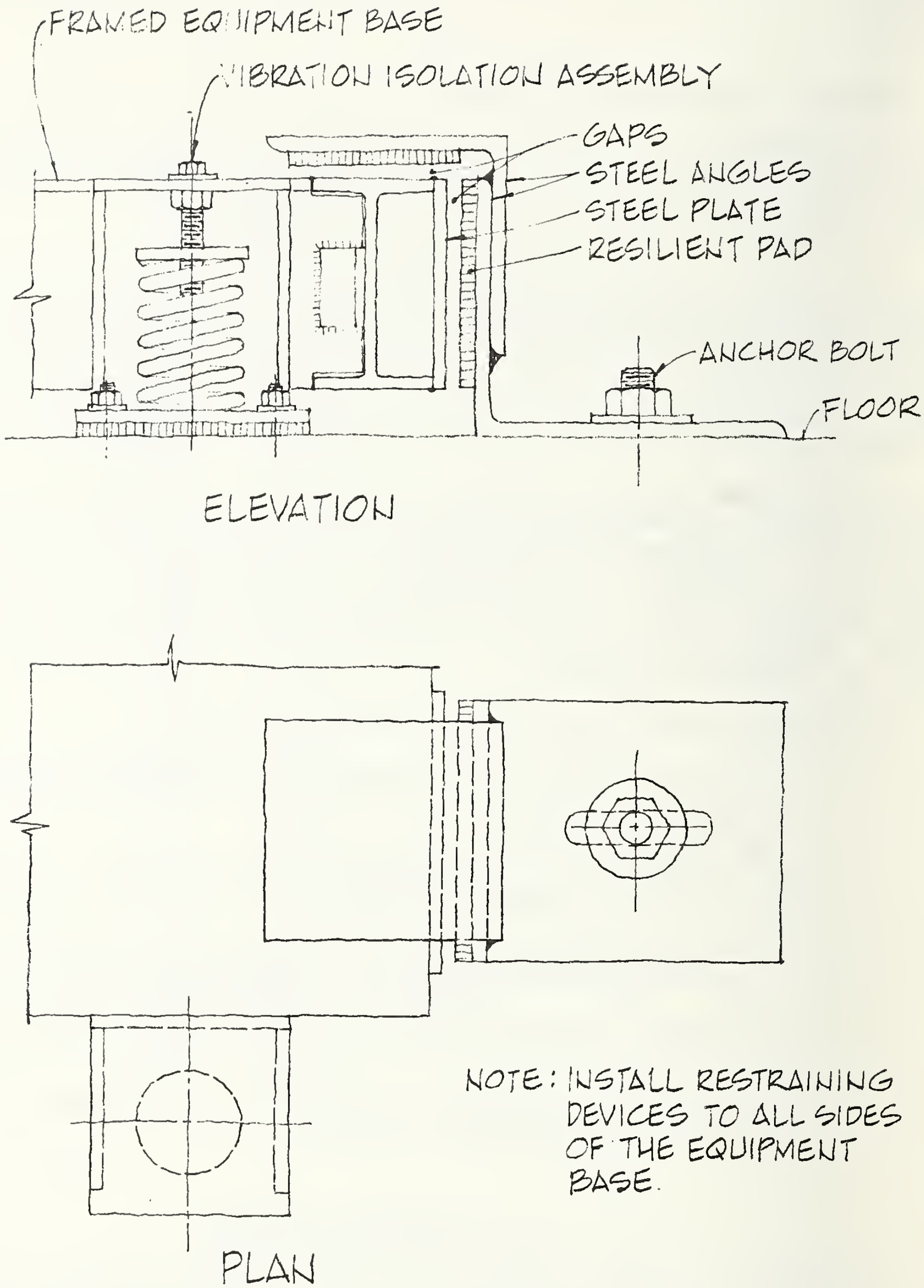

Figure 5. Vibration isolation mount with field erected lateral & vertical restraints.

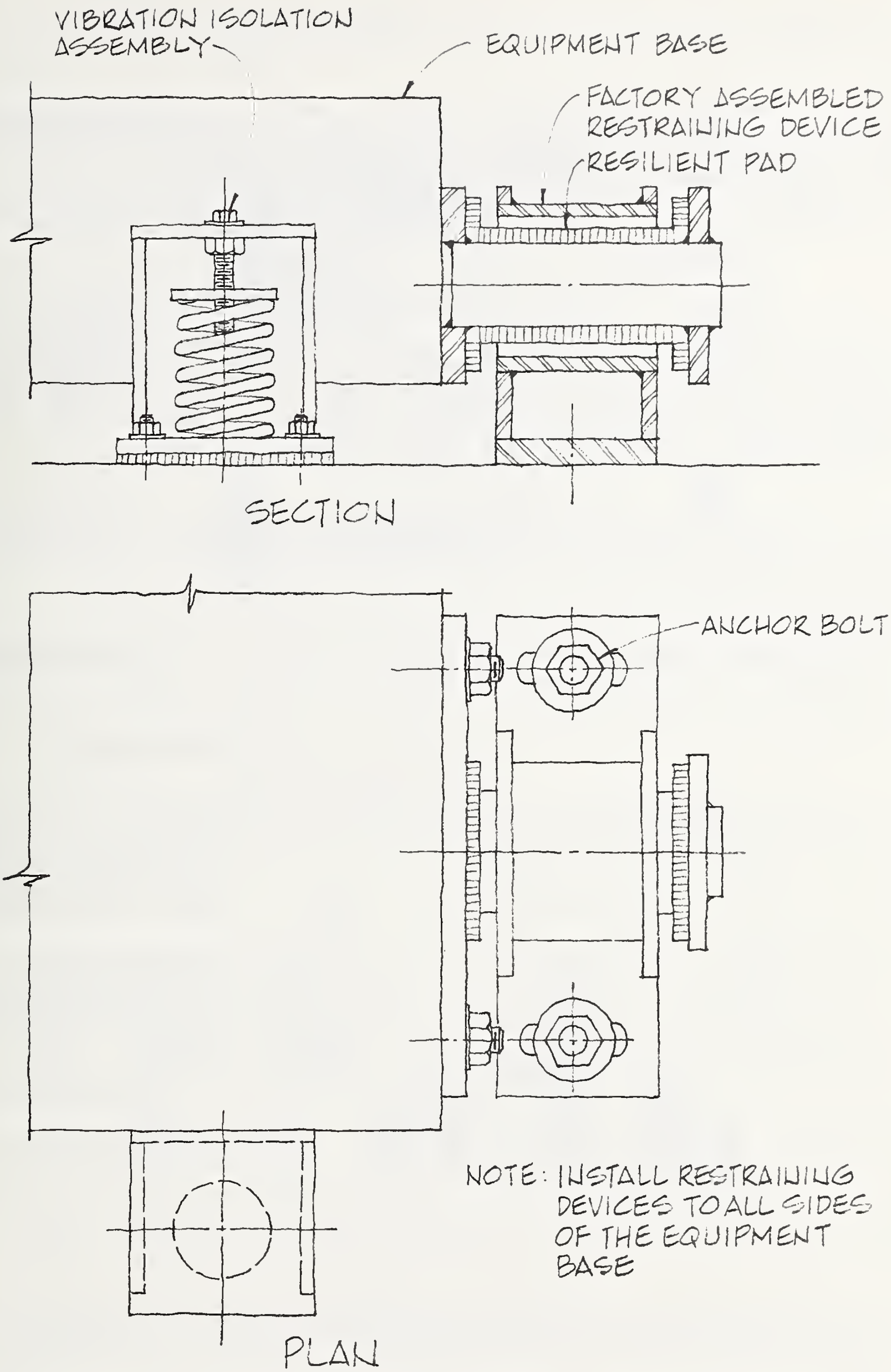

Figure 6. Vibration isolation mount with factory assembled lateral & vertical restraints.

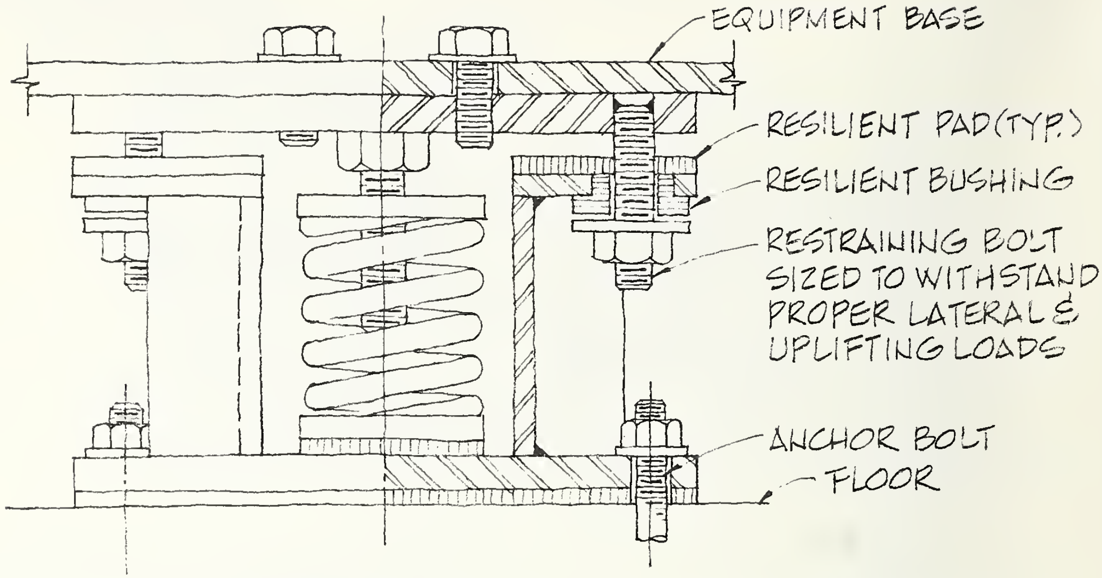

Figure 7a. Vibration isolation mount with built in lateral & vertical restraining capability.

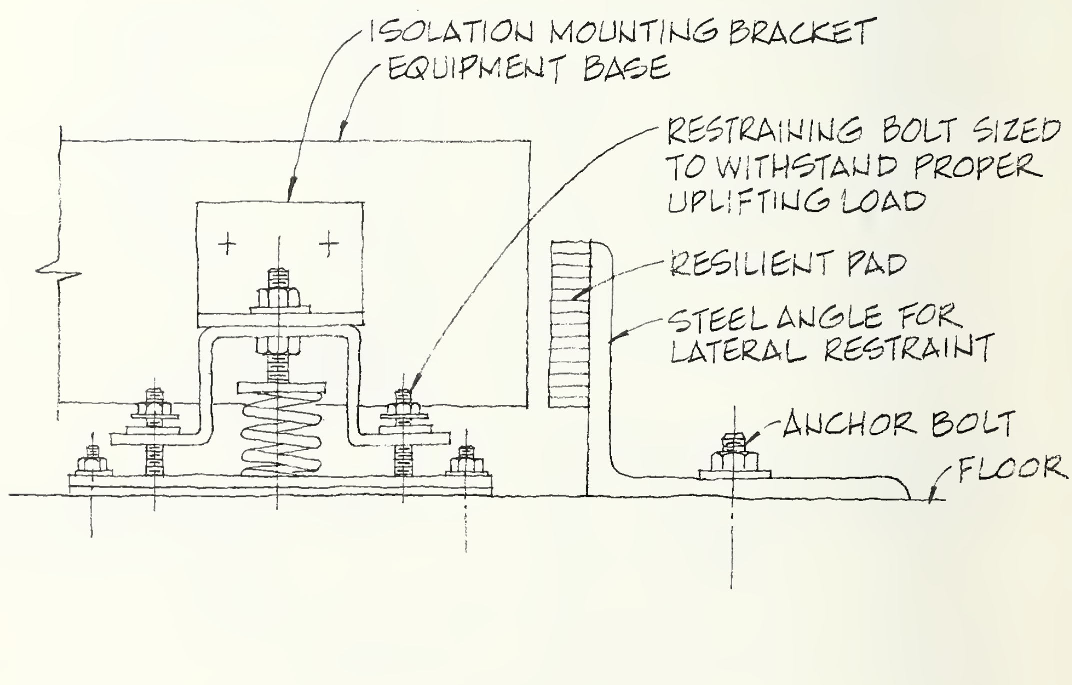

Figure 7b. Vibration isolation mount with vertical only restraining capability with lateral restraint added.

resolve the fundamental conflict between the requirement to isolate vibrating equipment and piping from the structure for acoustical control, and the requirement to tie down equipment and brace piping to minimize damage from earthquakes.

Except for the long standing bracing criteria to protect fire sprinkler piping [8] and some code requirements for anchoring of tanks, very little thought has been given to earthquake resistant design of mechanical systems. In the following discussions of proposed corrective measures, all anchors, bolts, braces, rods, supports, etc., must be designed to resist maximum anticipated lateral and uplifting loads.

1. <u>Equipment with Vibration Isolation</u>:

All types of vibration isolation devices--from the simplest rubber pads and molded neoprene devices to the large single and multispring high deflection units, and even air bags--were field tested during the San Fernando earthquake [5]. Generally, isolators that are not bolted to the equipment and to the floor are easily damaged during an earthquake. Molded neoprene isolators sustain less damage than single or multiple spring isolators.

Large diameter spring isolators supporting heavy equipment on upper floors of tall buildings are easily damaged during an earthquake when the equipment and its mounting base move with respect to the floor slab. When the isolators fail, the equipment drops to the floor and connecting piping and electrical service connections are damaged. Similar secondary damage to connecting services occurs when suspended equipment with vibration isolation hangers are subjected to differential movements with respect to the supporting structural member. Vibration isolators lower the natural frequency of the equipment assembly. Resonant response with large amplitude violent movements is possible when the predominant period of the structure approaches that of the equipment assembly.

To provide earthquake resistant design for equipment with vibration isolation and not interfere with the degree of isolation required for acoustical control the following corrective measures should be considered.

a. Bolt floor mounted vibration isolation devices to the equipment base and to the structural slab.

b. Provide lateral and vertical restraining devices around the base of vibration isolated floor mounted equipment to restrict the displacement of the equipment, and provide resilient material on the contact surfaces of the straining devices to retard impact loads. Acceptable devices of this type are shown in Figures 5 and 6, and other devices with varying amount of built-in restaining capability are shown in Figure 7.

c. Floor mounted equipment isolated by resilient pads or neoprene isolators and bolted to the equipment and to the structural slab, do not require lateral and vertical restraining devices.

d. Install vibration isolation hangers for suspended equipment tight against the supporting structural member.

e. Provide cross bracing between hanger rods on all four sides of suspended light weight equipment, similar to that shown in Figure 8a.

f. Provide a structural restraining frame around suspended heavy equipment, similar to that shown in Figure 8b.

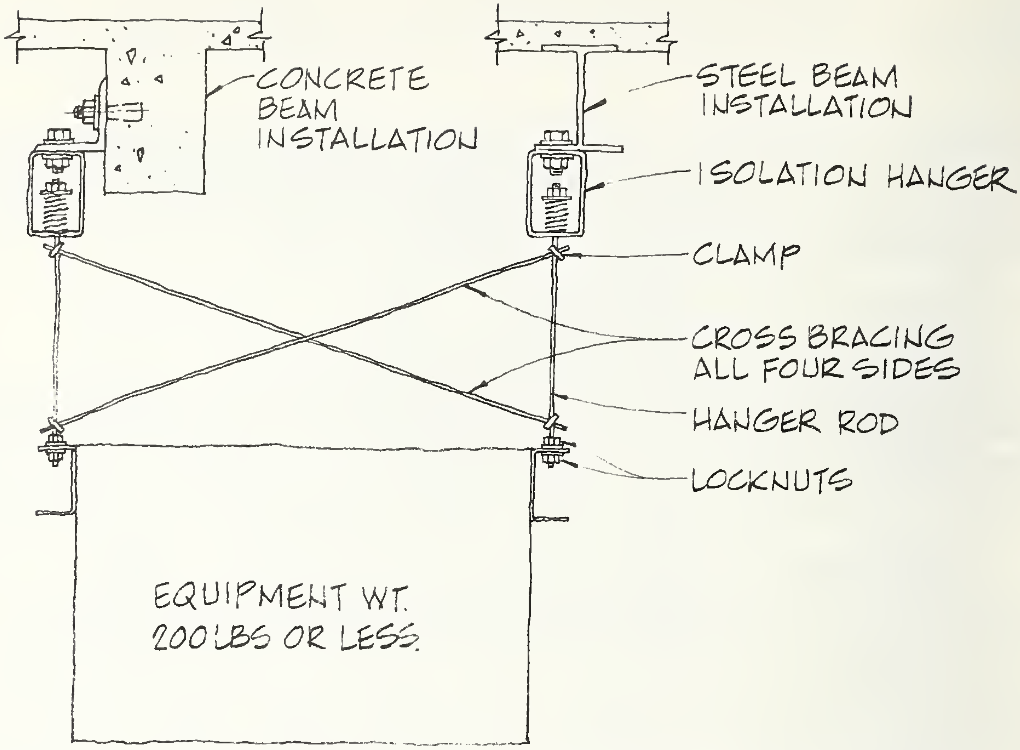

Figure 8a. Bracing for light weight suspended equipment with vibration isolation hangers.

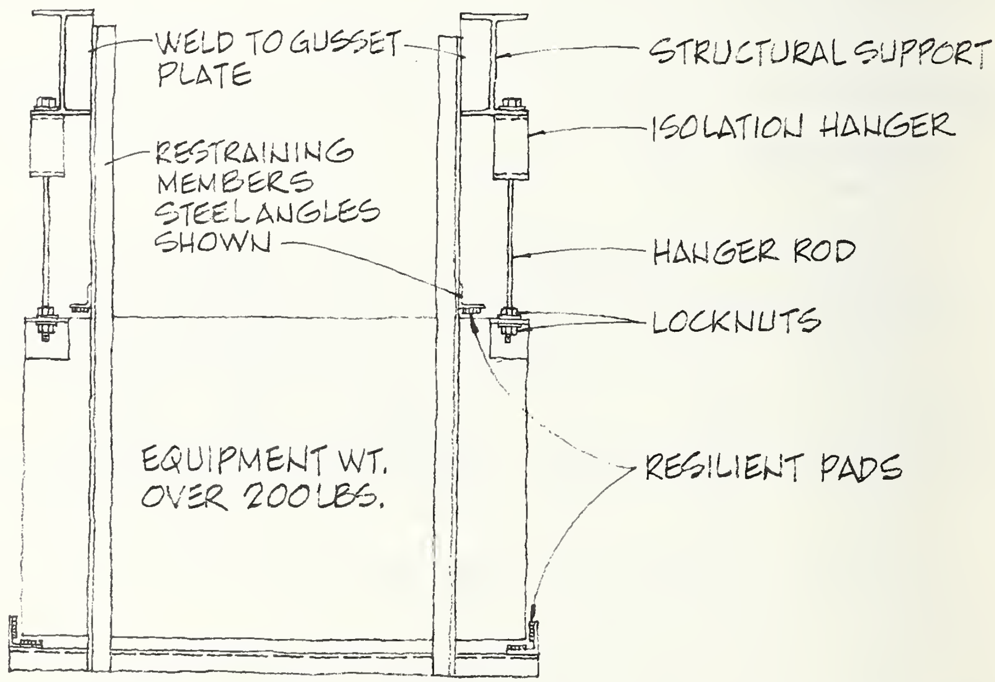

Figure 8b. Restraining frame for heavy suspended equipment with vibration isolation hangers.

2. Equipment Without Vibration Isolation:

Equipment without vibration isolation mounts that are well anchored to supporting structural members sustain little or no damage during earthquakes; while equipment without anchors will shift and move, often inflicting secondary damage on connected piping and electrical service connections. The movement of equipment during an earthquake often induces internal damage. Tanks containing liquids are extremely susceptible to damage during earthquakes. Unanchored tanks tip over, legs and support brackets collapse, they shift, and sometimes they slip out of supporting straps. These movements usually cause secondary damage to connected piping.

To avoid this type of damage the following corrective measures should be considered.

a. Bolt all floor mounted equipment and tanks to the structural slab.

b. Do not use threaded pipe for tank or equipment legs.

c. Strap all horizontal tanks to their saddles, weld lugs to tank at support points to prevent horizontal movement, and bolt saddles to structural slab.

d. Wherever possible, install ceiling mounted expansion tanks tight against the supporting structural member.

e. Any frame supporting elevated tanks or equipment must be provided with adequate bracing and anchors to the structural slab and walls.

f. Brace all suspended equipment on all four sides, similar to that shown in Figure 9.

3. Piping Systems:

Piping systems generally survive earthquakes with only minor damage. Excessive pipe movements and differential deflections between piping and connected equipment or structural elements, often induces failures at fittings. Screwed fittings are more vulnerable to earthquake damage than welded or brazed fittings. Pipes are damaged when subjected to excessive movements at seismic joints, or where a main is free to move and small branch lines are clamped to a structural element. Fire sprinkler systems are seldom damaged because they are provided with earthquake bracing.

Earthquake resistant design of piping systems should consider the following:

a. Provide sway bracing in both longitudinal and transverse directions on all pipes 2-1/2 inch and larger following the recommendations of Reference 8.

b. Do not use branch lines to support or brace large piping.

c. Where pipes are suspended by vibration isolation hangers, provide sway bracing similar to that shown in Figure 10.

d. Provide flexible joints where pipes pass through building seismic or expansion joints, or where rigidly supported pipes connect to equipment with vibration isolation.

e. Do not fasten one rigid piping system to two dissimilar parts of the building that may move differently during an earthquake; for example, a wall and a roof.

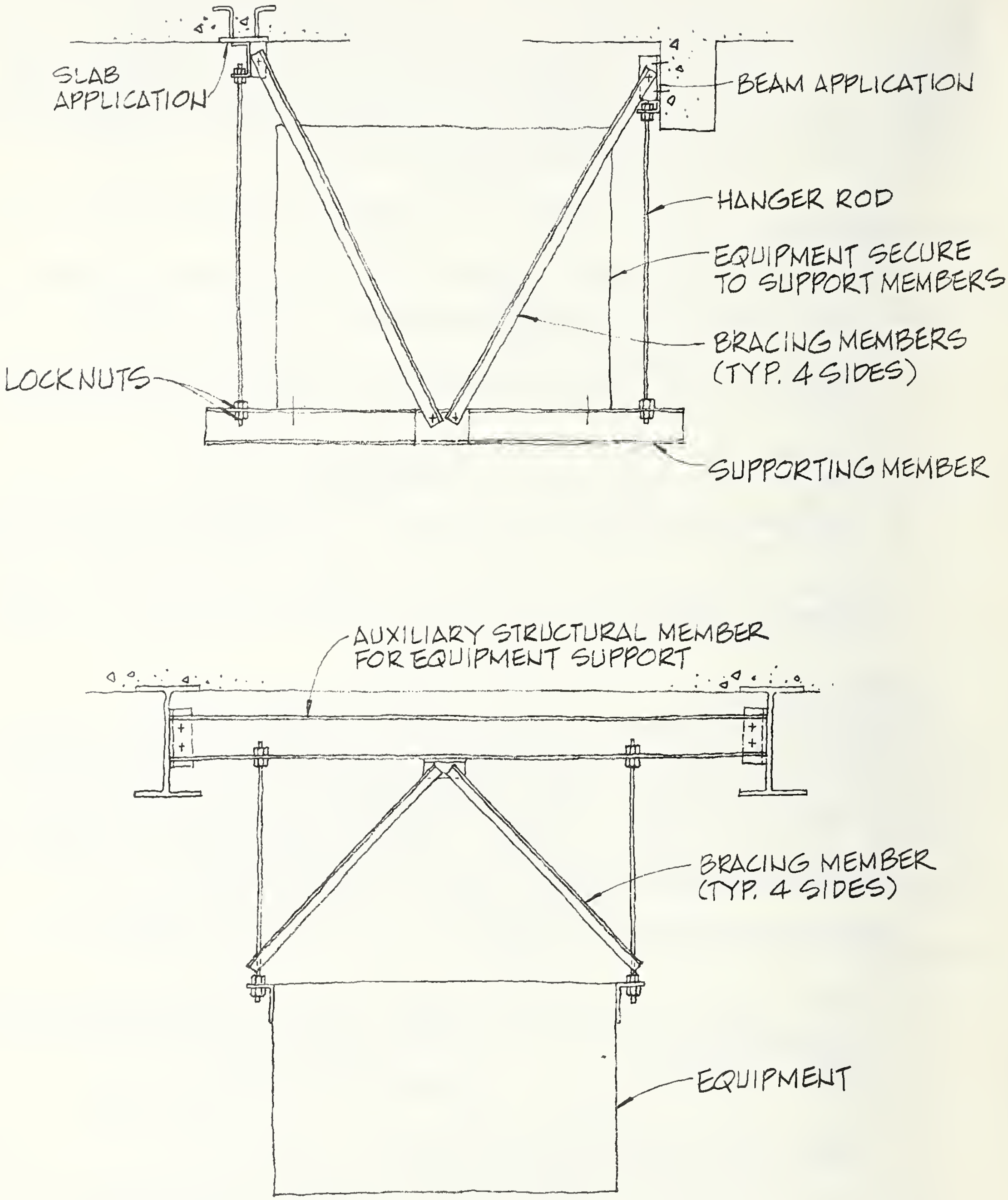

Figure 9. Bracing for suspended equipment.

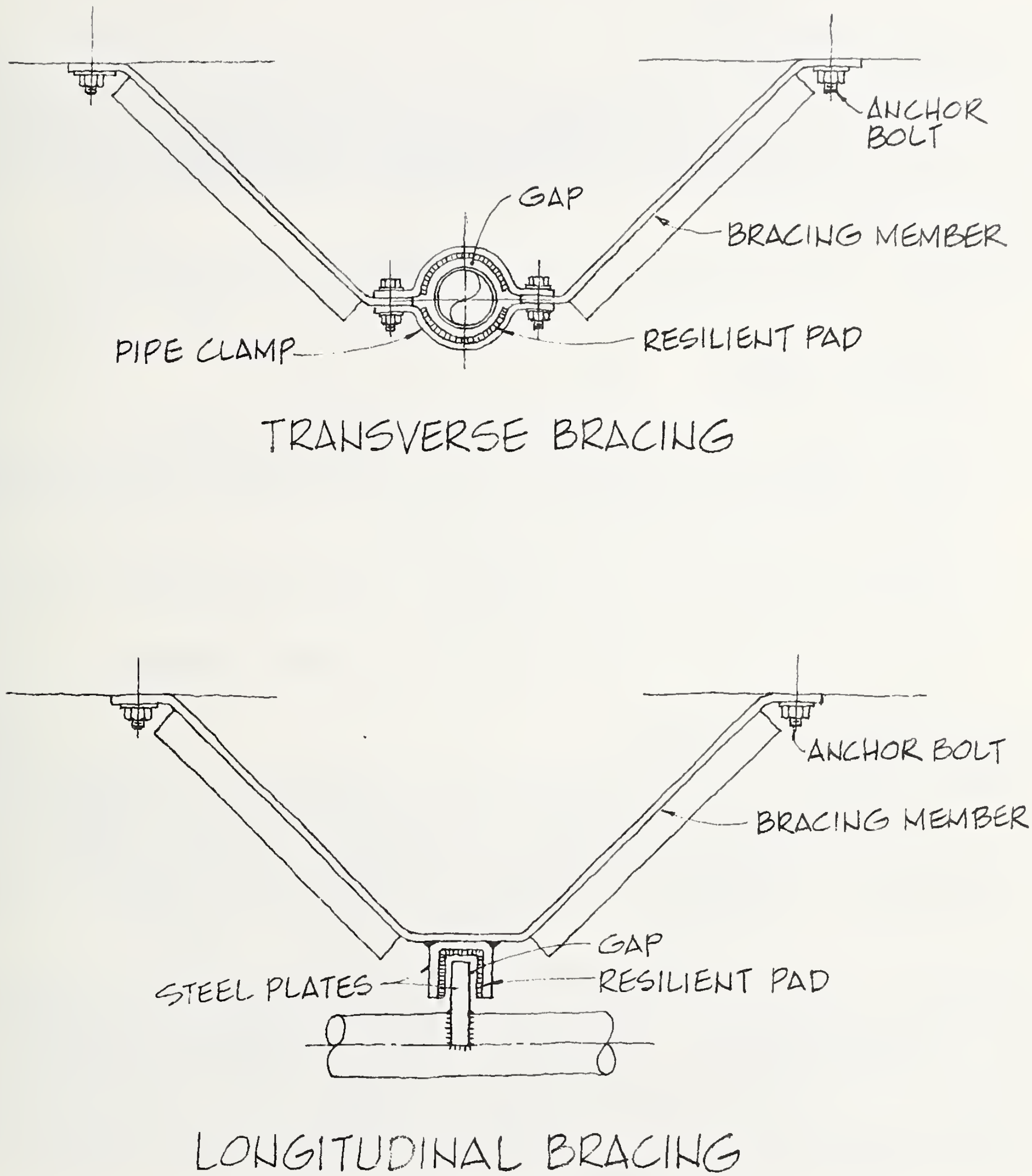

Figure 10. Bracing for piping suspended by vibration isolation hangers.

f. Wherever possible, support the weight of the vertical pipe risers at a point or points above the center of gravity of the riser, and provide lateral guides at regular intervals.

g. Provide large enough pipe sleeves through walls or floors to allow for anticipated differential movements.

4. Air Distribution System:

Ductwork, diffusers, grilles, etc., survive earthquakes with only minor damage. When long runs of large ducts are set in motion by the earthquake, joints are loosened and flexible connectors at fans are torn. In some instances the ducts have pounded ceiling suspension wires inducing secondary damage to ceilings. Large diffusers have dropped from ceilings where they lack proper support.

Earthquake resistant design of air distribution systems should consider the following:

a. Support horizontal ducts as close as possible to the supporting structural member.

b. Provide sway bracing similar to that shown in Figure 11.

c. Secure ceiling diffusers and registers to ductwork with sheetmetal screw. Diffusers connected to the flexible ducts must have positive ties to the ceiling suspension system.

d. Side wall registers and grilles must have positive ties to the ductwork and/or to the wall opening.

e. Secure vertical duct risers to shafts or floor openings.

5. Plumbing, Flues and Chimneys:

Plumbing systems survive earthquakes with practically no damage. Water heaters, that are essentially vertical water storage tanks with weak legs, fall over, inflicting secondary damage on the pipe connections and flue vents. Corrective measures for this type of damage are same as that for the floor mounted tanks. Some water closets seals are loosened during an earthquake and plumbing fixtures are occasionally chipped or cracked.

Heavy masonry stacks and old unreinforced chimneys are classical victims of earthquakes. The Uniform Building Code and all local codes in the major cities along the west coast require that chimneys and smokestacks must be designed to withstand 0.2g. Evidence after the San Fernando earthquake suggests that the 0.2g requirement may have to be strengthened. The lightweight double wall vents and boiler stacks, where properly installed and guyed survive the earthquake with very little damage. The only damage occurs at the vent connectors to boilers, furnaces or domestic water heaters, where the movement of the appliance damaged the vent connector.

D. Light Fixtures

The various types of light fixtures behave differently under seismic conditions depending upon their inherent design and their connections to ceilings. Suspended fixtures that are free to twist and rack are severely damaged when failures occur in supporting stems or chains and at their ceiling support points. Surface mounted fixtures sustain very little damage if

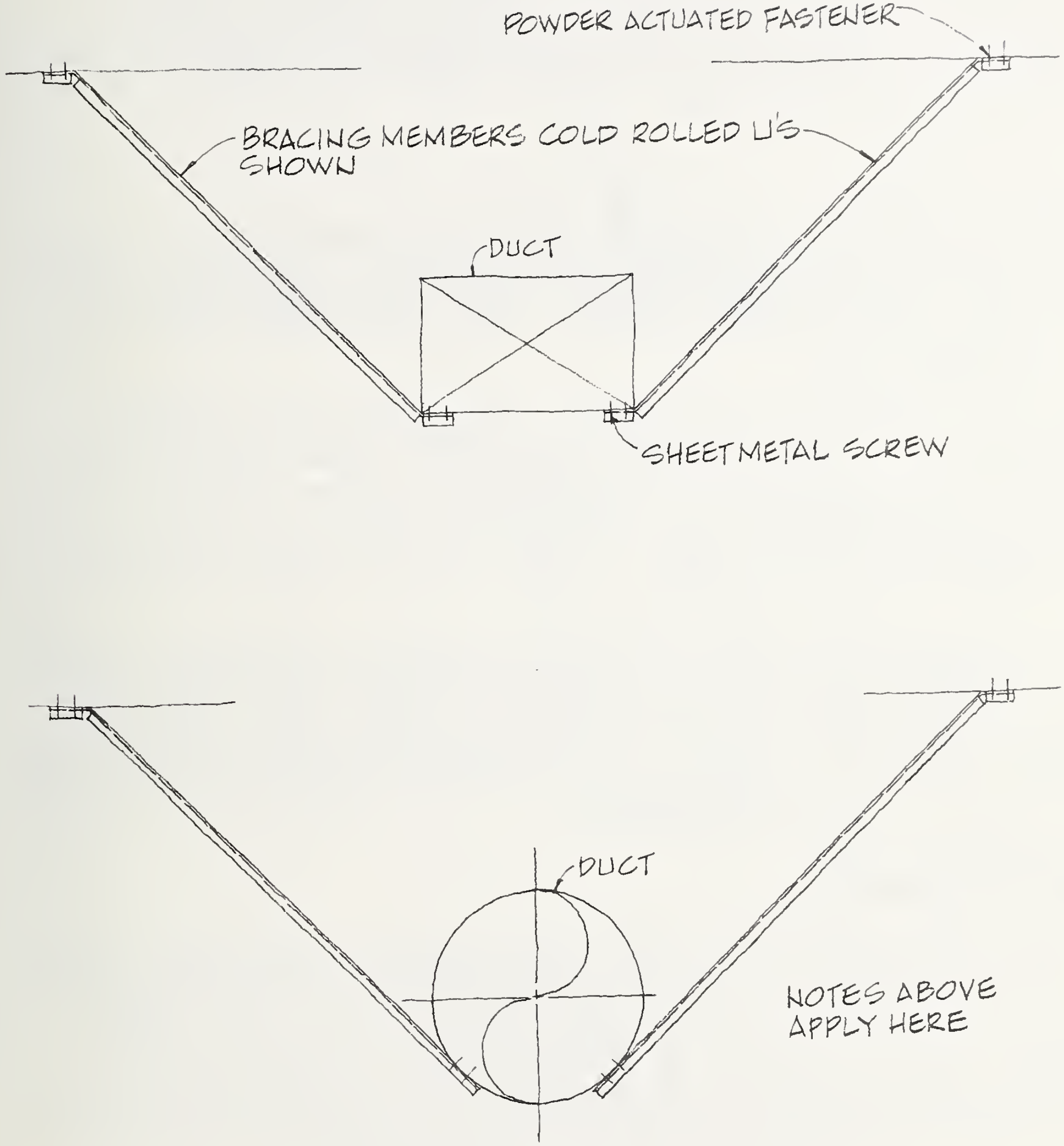

Figure 11. Bracing for long duct runs.

properly installed, and recessed fixtures are damaged when they are not securely fastened to the suspended ceilings.

The only codes that refer to earthquake resistant design of light fixtures and ceilings are two "rules of application" issued by the City of Los Angeles Department of Building and Safety, and portions of Title 24 of the California Administrative Code, that apply to primary and secondary schools. The Title 24 requirements specify that all light fixtures, ceilings and supports must be designed to withstand 0.2g lateral force [11, 12]. The City of Los Angeles requirements describe shake table tests for pendant fixtures [10], and discuss light fixtures installed in suspended ceilings but do not specify design lateral forces [9].

1. Recessed Fixtures:

Recessed fluorescent fixtures are damaged during earthquakes because they are supported on ceiling systems without positive attachments as shown in Figure 12. The fixtures pound on the surrounding ceiling elements and slide or jump off their supports and fall. Most of the damage occurs in exposed tee bar ceilings where the light weight ceiling elements are easily moved or deformed.

Corrective measures include properly supported and braced suspended ceilings as described herein, and the use of commercially available attachment accessories. The fixtures should be supported by and secured directly to the main runners of the ceiling support system as shown in Figure 13. They should not be supported by furring or cross runners, or nailing bars in the case of gypsum board ceilings. Where the locations of the main runners are not compatible with the lighting pattern, auxiliary support members of equal strength should be provided.

All recessed fixtures should also be provided with independent secondary supports attached to the fixture housing and the building structures as required in Reference 12. These supports are a minimum of two 12 guage wires placed on diagonal corners of each fluorescent fixture, and each wire must be capable of supporting four times the weight of the fixure. Although this requirement appears to be redundant after the fixture is secured to a ceiling support system, it is desirable in the event of ceiling failure or improperly installed fixture to ceiling attachments.

2. Surface Mounted Fixtures:

Surface mounted fixtures are generally undamaged by earthquakes because their installation method requires that they be securely tied to the ceiling system. Some damage has occurred when support clamps of the type shown in Figure 14a for fluorescent fixtures opened under twisting action, and tee bar clips of the type shown in Figure 14b loosened. These devices are unacceptable because they depend on friction and good workmanship to hold the fixture in place. Preferred installations are direct attachments to the building structure; suspended ceiling installations using positive locking devices similar to that shown in Figure 15 are acceptable.

3. Pendant Fixtures:

All types of pendant fixtures are inherently susceptible to damage during earthquakes. Their supports fail at ceiling connections, in swivel joints, at fixture housings, and in supporting stems or chains. The pendant fixtures also cause damage to suspended ceilings as they rack and twist during an earthquake.

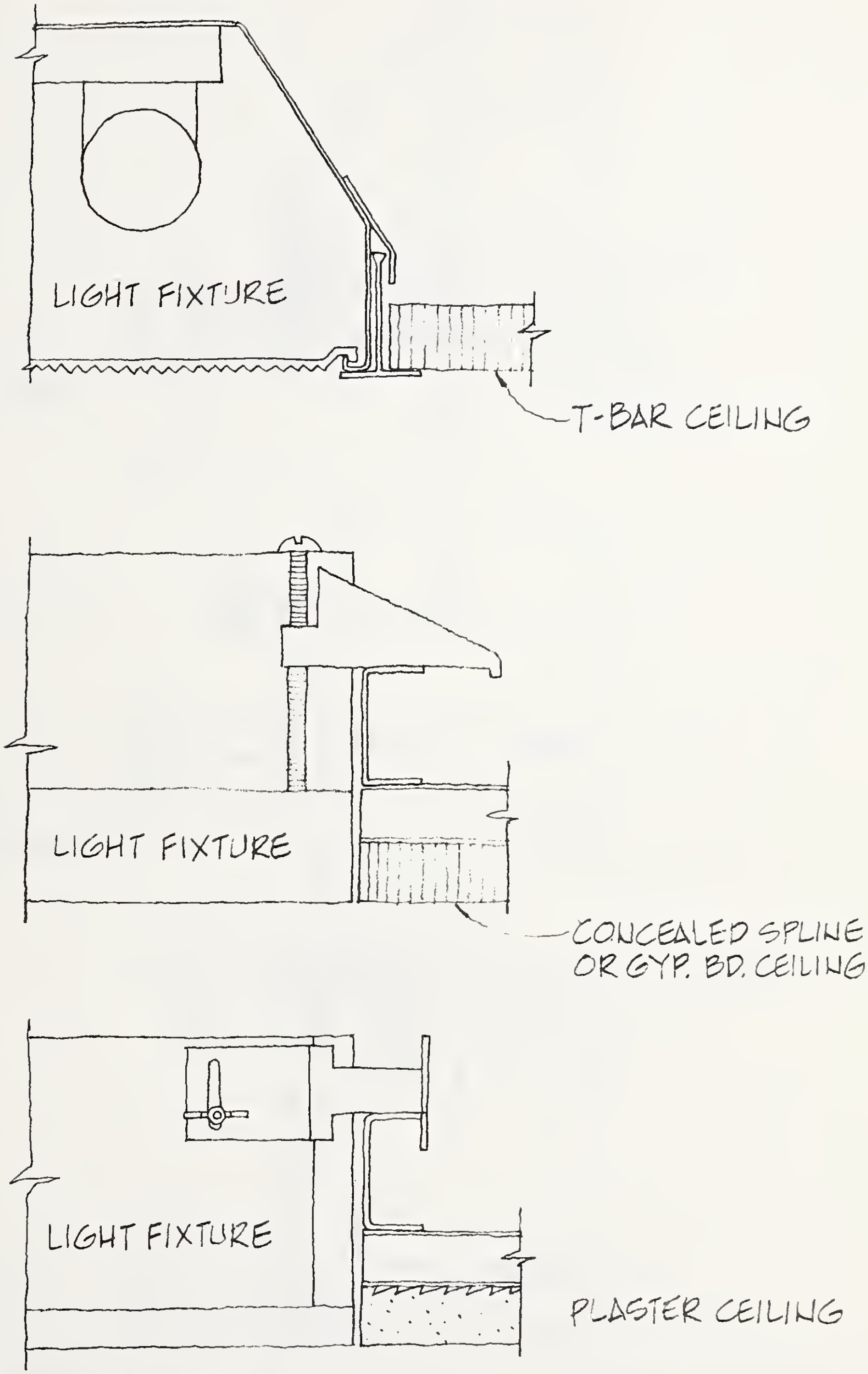

Figure 12. Unacceptable recessed light fixtures without positive attachments to ceiling.

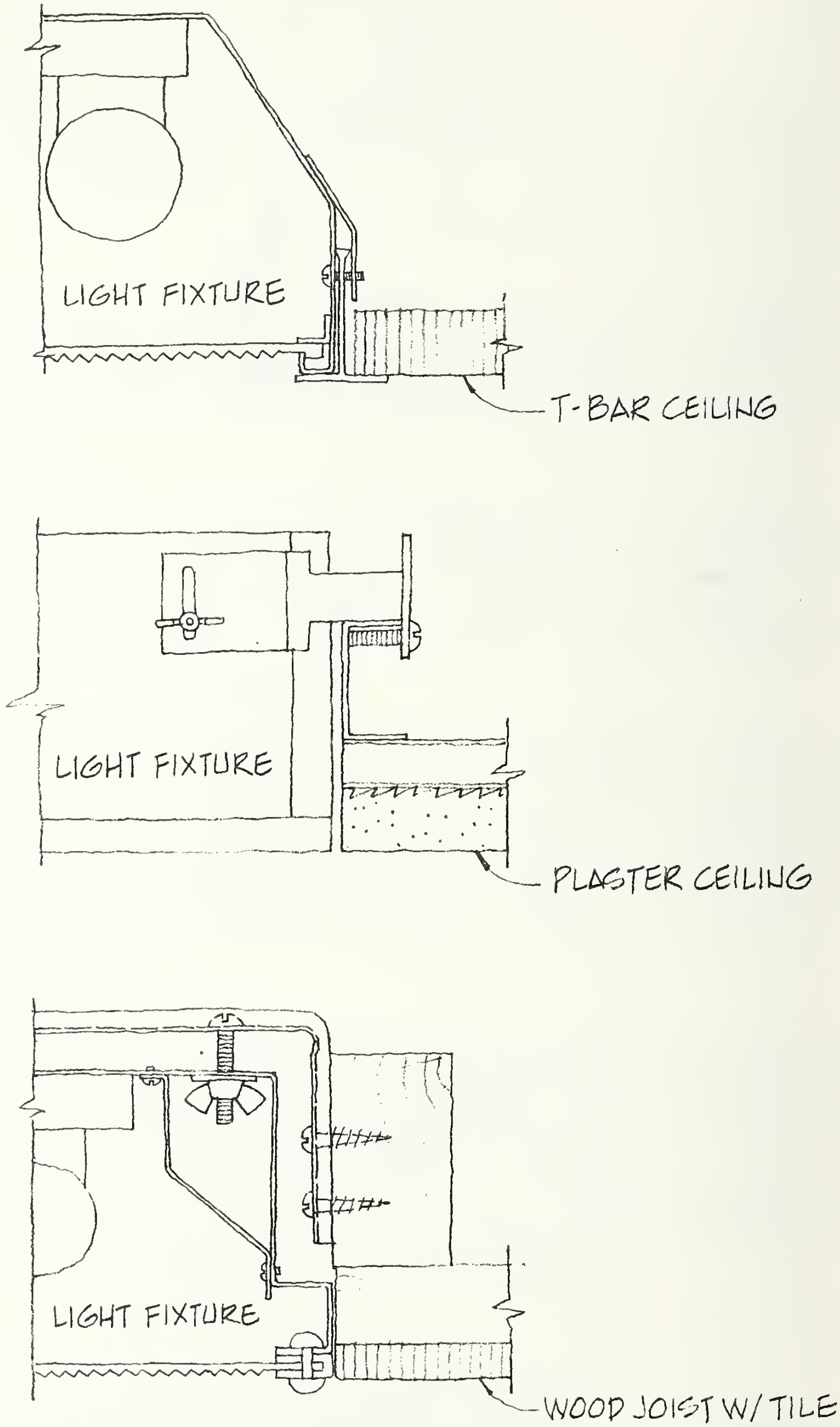

Figure 13. Acceptable recessed light fixtures with proposed positive attachments to ceiling.

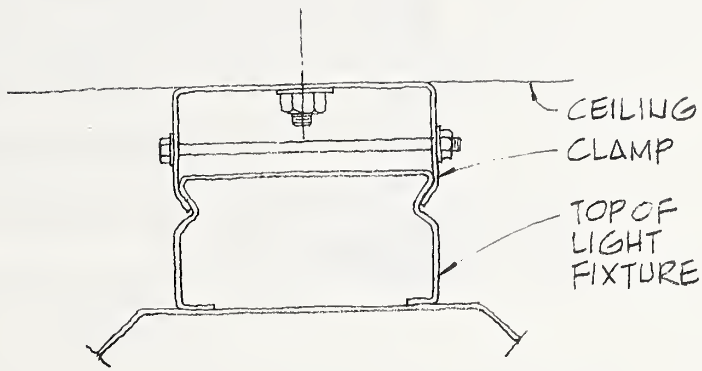

Figure 14a. Unacceptable surface mounted fixture clamp.

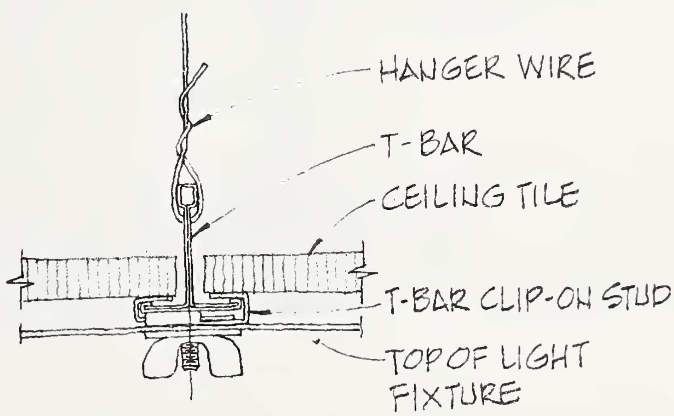

Figure 14b. Unacceptable surface mounted fixture tee bar clip.

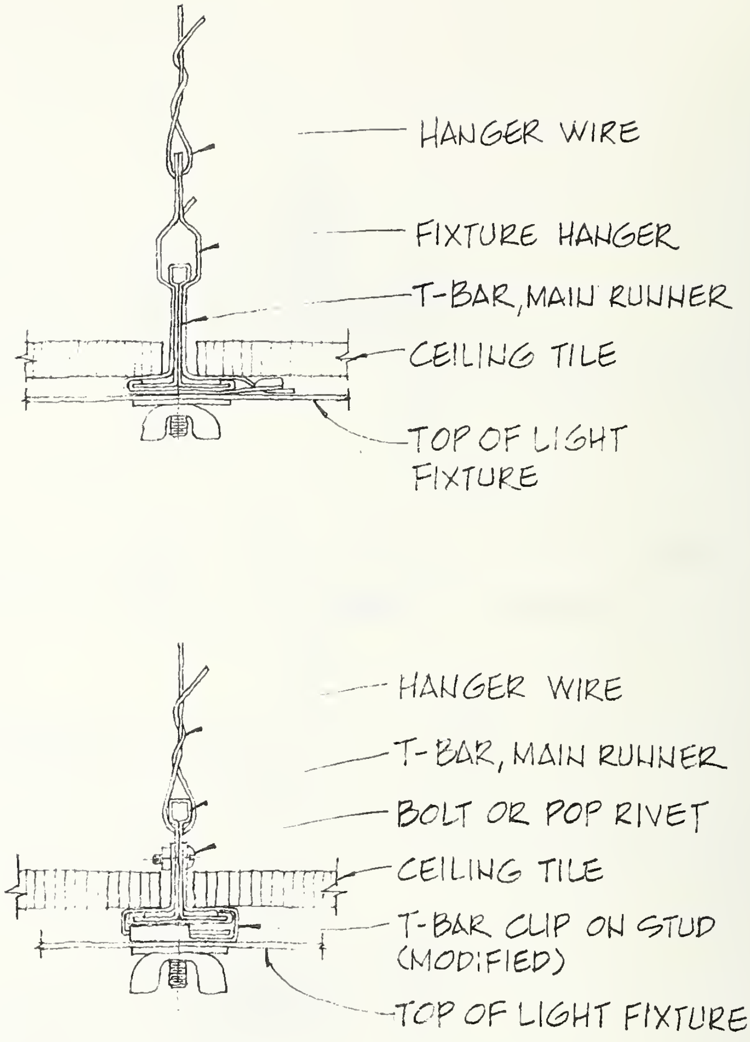

Figure 15. Acceptable surface mounted fixture tee bar clips with positive locking devices.

Contrary to popular opinion, the ball joints in pendant fluorescent fixtures that were originally developed as an alignment device, do not limit earthquake damage. It now appears that the conventional supporting assemblies --pendant hung from free swinging ball sockets--are set into resonance by the predominant frequencies of the earthquake [5].

After the 1952 Kern County, California earthquake caused pendant fixtures to fall in several tall buildings in the Los Angeles area, shake table tests [9] were required on all fixtures installed in the City of Los Angeles. This pioneering effort successfully established a minimum standard for the manufacture of light fixures, but it also introduced a false security that if the fixture passed the test it is "earthquake proof". The San Fernando earthquake provided the necessary field test to indicate the deficiencies in this laboratory simulation. For example, in only five schools approximately 700 "approved" pendant fixtures were damaged and had to be replaced.

The test procedure is deficient in the following areas:

a. Test is limited to a single one cycle per second frequency; earthquakes generate a spectrum of frequencies.

b. Test is limited to horizontal movements; earthquakes generate horizontal and vertical movements.

c. Test is set at 0.2g maximum acceleration; measured accelerations in the moderate San Fernando earthquake exceeded this limit.

d. Test sample is limited to 8 ft. (two 4 ft. fixtures); common installations exceed 20 ft.

Following the San Fernando earthquake the Los Angeles Unified School District installed safety cables through every stem of existing and replaced pendant fixtures. The upper end of the steel cable was roped to the ceiling support system and the lower end was tied to a large washer inside of the fixture canopy. The safety cable will prevent the fixture from falling, but the fixtures will still be damaged in future earthquakes.

We have concluded that there are no safe commercially available pendant light fixtures and where fixtures must be located below high ceilings they should be the surface type secured to a supporting grid system that meets the support and bracing requirements of suspended ceilings. Manufacturers of lighting fixtures should be urged to develop new pendant fixture assemblies that can withstand horizontal and vertical earthquake forces and incorporate the concept of dampened flexibility in stem materials and/or at swivel joints and connections.

For example, light fixture manufacturers could consider:

a. Stem wall thickness of not less than that of a rigid conduit (0.109 inches).

b. Ball joints that are restrained from upward movements. One suggestion is to install a spring in the cup on top of the ball designed to press against the socket at the bottom of the mounting plate as shown in Figure 16.

c. Some form of damping should be built into the pendant assembly. The most effective form of damping, of course, is the viscous type, but its application in this case would require additional research. The suggested spring would produce some frictional damping at the joint, but shake table tests would be required to establish its effectiveness.

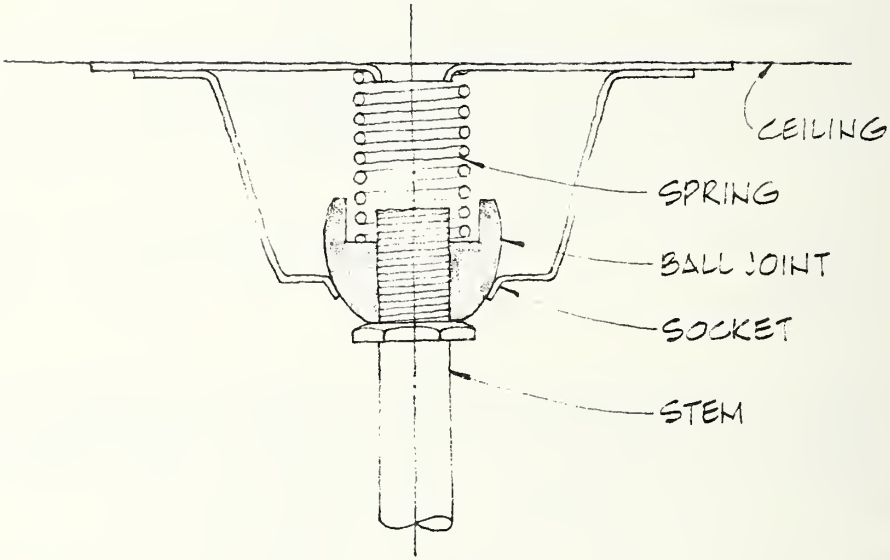

Figure 16. Suggested spring loaded ball joint for pendant fixtures.

d. Damping could also provided in the stem by the use of material similar to flexible conduit but with sufficient tensile strength to support the fixtures. The exterior of the stem could be coated with a plastic material to improve its appearance.

Industrial pendant fixtures with long steel rod stems and/or chains are often damaged during earthquakes when they strike each other and adjacent objects. Failures occur at outlet box connections and when support chains break or jump out of open support hooks. Corrective measures include the use of solid link chains and safety latch hooks.

Luminous ceilings are essentially a combination of surface mounted lamps on one ceiling plane, and a suspended ceiling with plastic diffusers or egg crate grilles on a lower plane. Lightweight incandescent recessed or surface mounted light fixtures are seldom damaged during earthquakes. Detachable fixture accessories such as louvers, diffusers and lenses that are loosened and fall during earthquakes should be provided with locking catches or screws and safety chains or cables.

E. Ceiling Systems

Suspended ceiling systems are damaged during earthquakes because they are free to swing on their suspension wires and batter against adjacent partitions or walls. They are also subjected to the damaging pounding forces from light fixtures, ceiling diffusers, sprinkler heads, etc. Systems that are flexible at the ceiling plane (exposed tee bar, concealed spline, and luminous) sustain greater damage than systems with greater rigidity (metal lath and plaster, gypsum board with glue-on tiles). Wood joist with wood hangers and nail or glue-on tiles sustain the least damage because the system is inherently rigid at the ceiling plane and the wood hangers provide good lateral and vertical bracing.

The only codes that refer to earthquake resistant design of ceiling construction have been cited herein under the Light Fixture portion of this paper [9, 11 and 12]. None of the codes call for designs to resist vertical uplifting loads.

1. The Exposed Tee Bar, Luminous and Concealed Spline:

Exposed tee bar and luminous ceilings are easily damaged during earthquakes, because the system lacks rigidity and supports to inhibit lateral and vertical movements. The ceiling tiles and recessed light fixtures in the case of tee bar ceilings, are dropped into the ceiling grids to complete the ceiling installation. The entire system and light fixtures are set in motion during an earthquake, and the differential movements at perimeter walls cause the tees to deform or pull away from their wall supports and drop out the light fixtures.

Concealed spline installations are inherently more stable than exposed tee bar and sustain less damage during an earthquake. The tiles are tightly keyed together by splines so that entire ceiling plane is more rigid and acts as a unit. Damage usually occurs at perimeter walls where supports are bent and slit tile are torn enough to fall.

The suspended ceilings also sustain other types of earthquake induced damage. Pendant light fixtures that are clamped onto exposed tees or tied to ceiling outlet boxes exert serious twisting and bending forces on the system. Recessed light fixtures that are not tied to the ceiling, jump or slide off of their supports and batter the surrounding ceiling elements. Fire sprinkler piping that is supported from and laterally braced to the supporting structure does not move with the ceiling system during an earthquake and sprinkler heads tear ceiling tiles. Similar but less punishing damage is inflicted by the sheet metal ducts and ceiling diffusers.

Suggested corrective measures should include all of the requirements of Reference 12 with the following additions or clarifications:

a. Ceiling grid should be braced at regular intervals against lateral and vertical movements. Some suggested bracing methods are shown in Figure 17.

b. Do not fasten ceiling system to the surrounding walls or partitions. Where possible, use soffit to return the ceiling system to the supporting slab as illustrated in Figure 18a. Where the ceiling must join a wall or partition, provide an angle wall trim wide enough to allow for differential movements as shown in Figure 18b. Main and cross runners should have hangers at the perimeter so that wall trims do not support the ceiling.

c. Cross runners should be fastened to the main runners, using lacking clips of the type shown in Figure 19, or similar devices designed to prevent cross tees from pulling or twisting out of the main runners.

d. Ductwork and piping systems in the ceiling space should be braced against lateral and vertical movements.

e. Avoid the use of pendant light fixtures.

2. <u>Gypsum Board with Glue-On Tiles, and Lath and Plaster</u>

These two types of ceiling systems are considerably heavier and more rigid than the tee bar or concealed spline systems and they usually survive earthquakes without significant damage. Typical failures in the gypsum board ceilings are dropped or loosened tiles at perimeter walls. Indications of damage due to vertical uplifting forces were noted during the San Fernando earthquake. Loosened hanger wires allowed a ceiling to sag because expansion shields and powder actuated fasteners were pulled from the supporting slab; and in another installation, nails were pulled out of nailing bars and nail heads were pulled through the gypsum board.

Plaster ceilings usually are undamaged except at perimeter walls where they are subjected to differential movements. In large irregular shaped ceilings where furring channels are wired together and not rigidly connected, plaster cracking occurs because of flexible corner supports. Where long rows of recessed light fixtures and linear air diffusers divide large plaster ceilings the differential movements of the two ceiling sections will allow the lights and diffusers to lose their supports and fall.

Suggested corrective measures include all of the requirements of Reference 12 with the following additions or clarifications.

a. Ceiling system should be braced at regular intervals against vertical movements and provided with lateral braces at the perimeter.

b. Gypsum board ceiling should be reinforced at nail points by the use of steel nailing strips.

c. Furring channel joints in irregular shaped ceilings should be made up using rivits, bolts or welds. Corners should be braced so that they do not pivot.

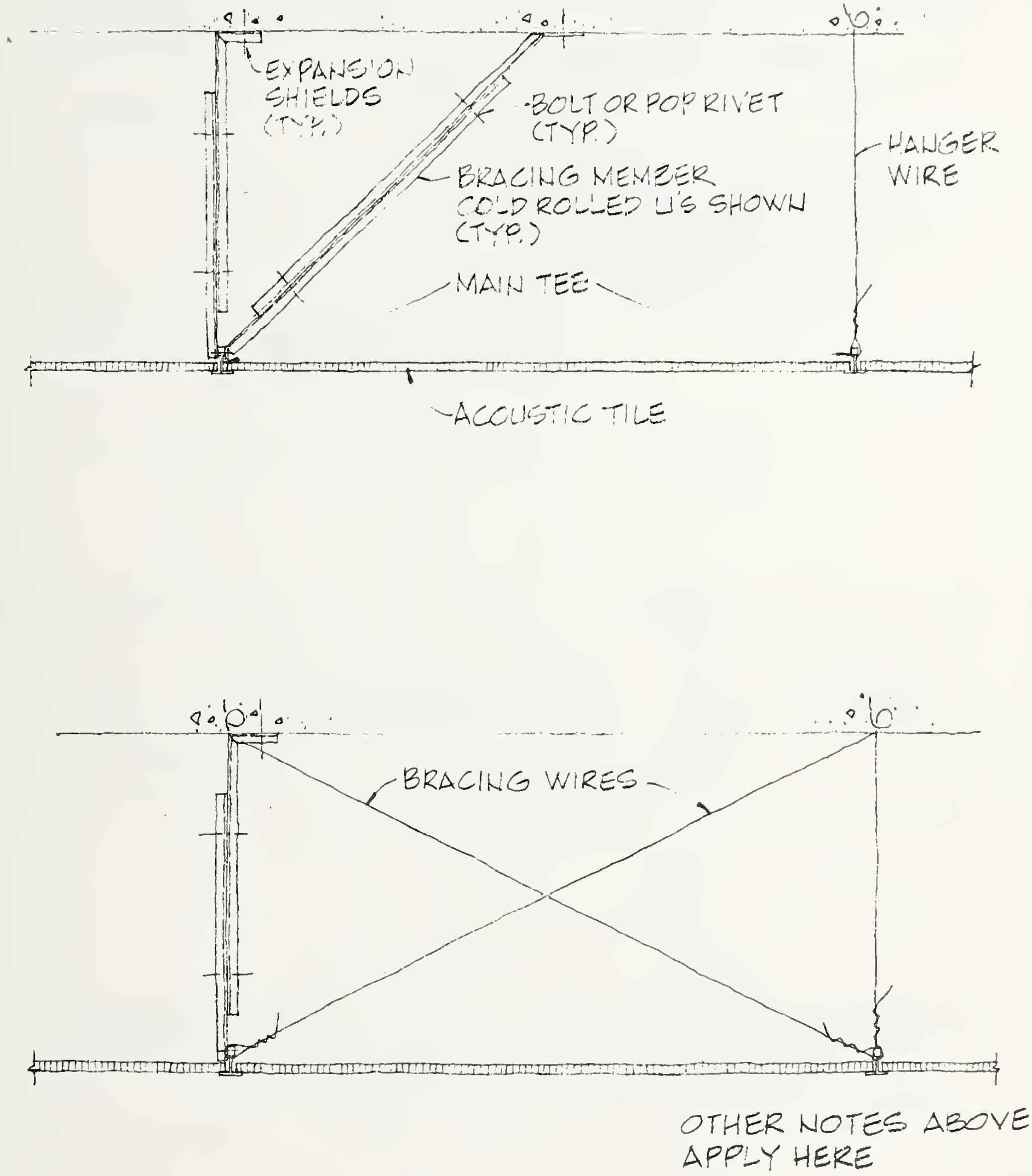

Figure 17. Proposed methods for providing lateral & vertical bracing of suspende ceilings.

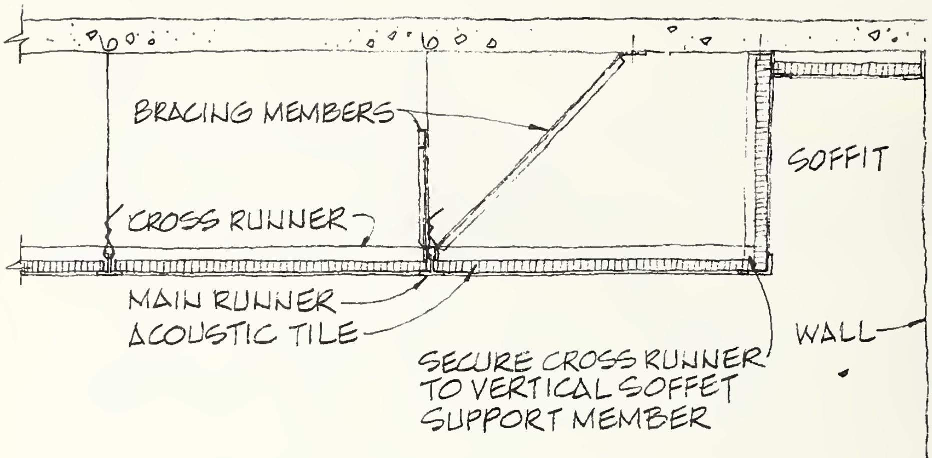

Figure 18a. Use of soffit to separate ceiling from wall.

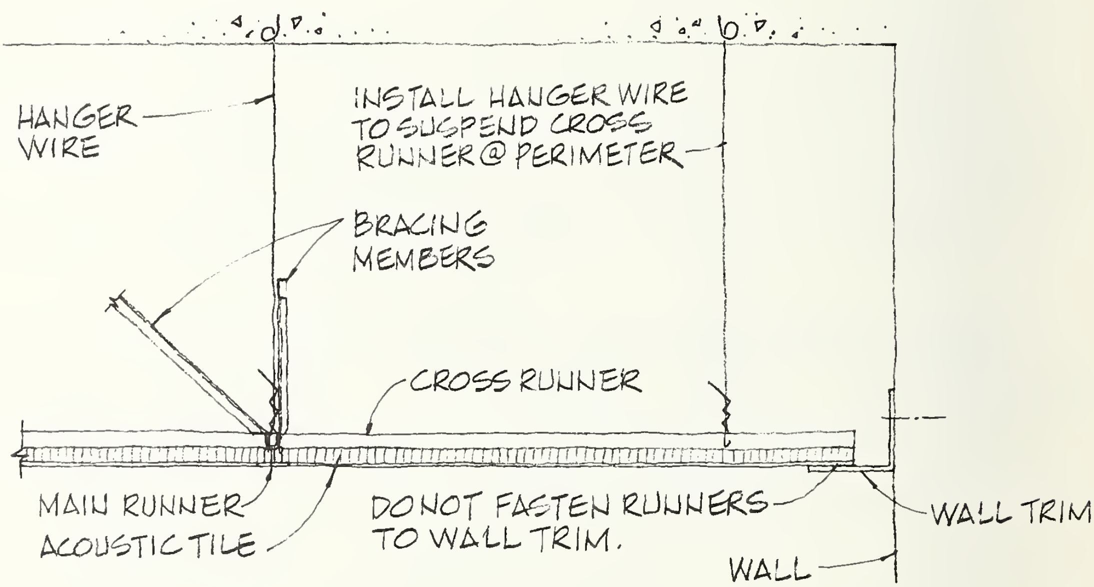

Figure 18b. Use of angle trim to allow for movement at wall.

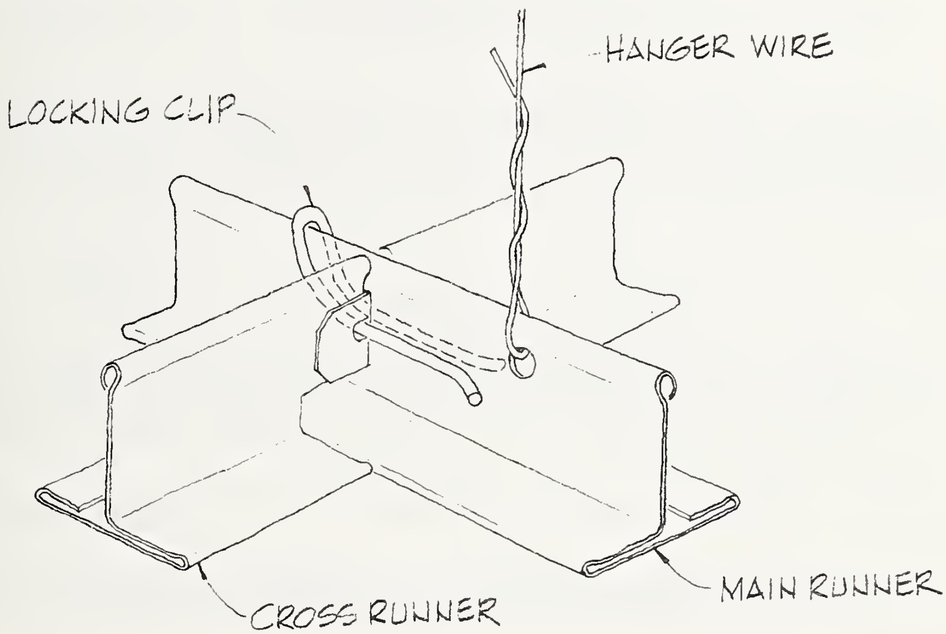

Figure 19. Locking clip to fasten cross runner to main runner.

d. Large ceiling areas separated by rows of linear diffusers or light fixtures should be held together with rigid ties and the diffusers and light fixtures should be secured to the ceiling system.

3. Wood Joists with Nail or Glue-On Tiles:

Wood joist with nail or glue-on tiles is the most earthquake resistant of all ceiling types. The ceiling plane, made of wood striping nailed to the 2 x 4 joists, is very stable and the wood hangers, nailed to the wood joists and the structural members above, provide vertical and horizontal bracing. Where wood hangers are substituted for hanger wires, the ceiling should be braced vertically and horizontally in the same manner suggested for the plaster ceilings.

F. Storage Racks

It is surprising that more people are not injured during earthquakes when storage racks dump their contents, totally collapse or fall over. Some racks collapse because they lack internal strength, and others because they are not anchored to floors and/or externally braced to a structural member. In some instances parallel rows of racks with strong ties at their tops, survive earthquakes with no damage. Free standing unanchored rows of tall storage racks or cabinets usually fall over like a row of dominoes.

The only code that refers to earthquake resistant design of storage racks is Title 24 of the California Administrative Code [11]. It requires that all case work in schools to be anchored to the floor and/or laterally braced at the top to resist 0.2g applied in any direction. These requirements must be strengthened and extended to all types of buildings, with special design consideration for important racks carrying emergency power batteries, communication equipment and vital records.

Earthquake resistive design for storage racks should consider the following:

1. Racks must be designed with lateral bracing and anchor bolts that can withstand anticipated lateral and uplifting loads.

2. Rows of racks should have rigid ties installed at the top of the racks to brace and stabilize the entire installation.

3. Racks placed along walls should be anchored to the wall to avoid battering between the wall and the rack.

4. Develop educational programs that suggest face bars on high shelves.

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BEHAVIOR OF STRUCTURAL ELEMENTS

A REVIEW

by

B. Bresler*

A. Introduction

This review is intended as a brief summary of the current knowledge regarding behavior of structural steel and concrete elements under seismic and similar extreme conditions, such as hurricanes, tornadoes, etc. In considering the special structural design requirements for buildings under extreme natural hazards, particular attention is placed on the earthquake resistant design. This is done for several reasons: (1) earthquakes present an extreme natural hazard, as exhibited by the Alaskan earthquake, 1964, and San Fernando earthquake, 1971; (2) structural design requirements for assuring safety against hurricanes and similar effects are also similar to the requirements for earthquake resistant design; and (3) most of recent research has been directed toward increased knowledge of structural behavior under seismic conditions.

Current practices in design and construction and development of improved practices from documented research are discussed and some suggestions for further high-priority research activities are included.

B. General

B.1 Loading Characteristics

Most structural design criteria (including design loads, methods of analysis, evaluation of stresses, deflections, and load-carrying capacities, as well as specifications and codes) have been developed assuming that the applied loads were static (or quasi-static) and that loads which might produce distress (or failure) would increase monotonically and remain proportional to each other throughout. In general, separate loads may change independently, may be repetitive, and can be described more properly as "variable repeated loads."

Under normal service load conditions structures behave essentially elastically. Buildings designed for such conditions, excluding earthquakes, hurricanes, and similar extreme conditions, are not likely to experience adverse effects of variable repeated loadings, particularly when the ratio of live load or normal wind load effect to dead load is one-half or less. It has been generally concluded that under such loading conditions design based on monotonically increasing loads is satisfactory.

Two traditional design approaches are based on this concept. The working stress method of design, based on service loads and allowable stresses, is most commonly used in design of steel buildings. The "ultimate strength" method of design, based on "ultimate" loads (prescribed multiples of service loads) and "ultimate capacity" (calculated strength) of structural elements is now widely used in design of concrete structures.

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A more comprehensive and logical design approach, now gaining wide acceptance in Europe and U.S.A., is the "limit state approach." This permits recognition of the different critical conditions of loading and of structural response as independent "limit states" and therefore may be more adaptable to design for extreme loadings.

A basic characteristic of severe earthquake and extreme wind loadings is the cyclic repetition of high intensity reversible loads. Under such conditions design for monotonically increasing loads to failure is not sufficient to ensure limited damage and safety.

Two principal differences are introduced by earthquake induced loadings:

(a) Time-History of Loading. Because of the dynamic nature of this loading, it is usually characterized by repeated load reversals and excursions into the inelastic range. The consequence of such cyclic loading is the possibility of deterioration in load carrying capacity, (low-cycle fatigue), even when the number of such repeated overloads is small.

For rigid structures, the period may be as low as 0.1 sec., and the number of inelastic reversals may be as great as 100. For flexible structures, the period may be as high as 10 sec., and the number of inelastic reversals would be small, perhaps only 2 or 3 during the earthquake.

Any deterioration due to alternating load reversals may cause changes in the stiffness, as well as strength, of the structural elements. Because the overall dynamic behavior depends on the stiffnesses of the various elements in the structure, knowledge of the time-dependent stiffness of the elements becomes an important consideration in the analysis and design of the structure.

(b) Energy Absorption and Dissipation. Seismic as well as hurricane loadings are usually associated with a release of large amounts of energy causing randomly varying ground motion or movements of large masses of air at high speeds. The energy so released must be dissipated in various ways--by ground, by man-made structures, by wave-action in large bodies of water, etc. The amount of energy dissipated in the structures by inelastic action is often preferable to the very high intensity of stress (or force) which would be generated in the elastically behaving structures exposed to large transient inputs of energy. Therefore, knowledge of inelastic behavior of the structures from the point of view of energy absorption and dissipation is essential.

B.2 Special Requirements

In the presence of inelastic deformations the load-displacement relationships are described in terms of ductility. Ductility factor μ is generally defined as the ratio of total deformation to elastic deformation at yield, Fig. B.1.

The fundamental concept of ductility is based on the material behavior under essentially static loading, and for an elasto-plastic material it is a valid measure of energy absorption capacity. This concept of material ductility (or strain ductility) has been extended with reference to curvature ductility or extensional ductility--in which case the effects of cross section shape and size are combined with material characteristics, and even more generally, the concept of ductility has been used with reference to overall displacements--in which case the overall geometry of the structure and the loads upon it are involved in the evaluation of ductility.

Based on the assumption that the building will undergo significant inelastic deformations during a severe earthquake and in this manner will be capable of dissipating appreciable amount of energy without failure--the usual design criteria specify approximate methods for calculating seismic forces which rely on the ductility of the structure.

Use of ductility factor as an index of satisfactory performance under cyclic reversed loading is not reliable. First, ductility does not adequately describe behavior of structural elements under cyclic loading, particularly when the element exhibits a decrease in stiffness or capacity to resist loads with increasing number of cycles. This is often the case for concrete structures. Second, ductility does not adequately describe energy dissipation capacity of the structure; the ideal elasto-plastic behavior, for which the concept of ductility was originally established, often overestimates energy dissipation capacity. Finally, it may be difficult to relate ductility to damage, particularly cumulative damage under cyclic loading.

In the presence of cyclic loading the load displacement relationships are described by hysteresis loops. In the simplest case, say an ideal elasto-plastic material, the <u>hysteresis</u> loop for a constant plastic deformation in each half-cycle becomes a parallelogram, Fig. B.2. While elasto-plastic model is a good representation of the behavior of steel under monotonic loading, it is not valid for cyclic reversed loading. Tests have demonstrated that inelastic behavior upon load reversal is strongly non-linear and is particularly sensitive to the previous inelastic straining. This nonlinear characteristic of steel is usually referred to as "Bauschinger effect", Fig. B.3

Two types of hysteresis loops have been observed in tests of structural elements under fully reversed repeated loads. The first is characterized by a stable set of hysteresis loops, in which the loop does not change significantly with successive cycles of loading, Fig. B.4. The second, is characterized by "degradation" or changes in stiffness, and energy dissipation with successive cycles of loading, Fig. B.5. Various mathematical models have been proposed to represent behavior of structural elements having either the first or the second mode of hysteretic behavior, Fig. B.6-B.8.

A convenient parameter in evaluating <u>energy dissipation</u> is plasticity ratio π_d, defined as the ratio of residual plastic deformation in any given cycle to the elastic deformation at yield, Fig. B.1.

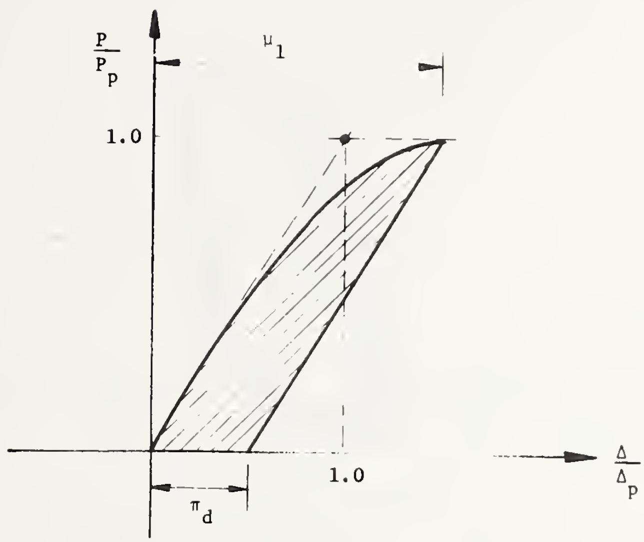
The area under the hysteresis loop in a given half-cycle (dissipated energy) is clearly related to π_d for that half-cycle. In reporting experimental data values of μ and π_d for each half-cycle would provide a valuable basis for generalizing test results, Fig. B.1. Energy dissipation may also be useful in defining cumulative damage.

For an ideal alternating cyclic loading the hysteresis loop becomes stable and values of maximim displacement Δ_j and corresponding loads P_j become constant, Fig. B.4. When either the loading varies or when material is sensitive to prior history the loops for consecutive cycles will vary. Even with constant values of Δ there may be a change in the corresponding resistance, usually a reduction in resistance due to some cumulative damage with cycling, Fig. B.5.

Such reduction in resistance can be expressed as a ratio λ_j which is the ratio of resistance P_j to the initial value P_i, assuming cyclic variation with constant maximum deformation Δ for each cycle.

B.3 Types of Elements and Loading

The types of elements considered in this review are identified by considering the most common types of structural systems in buildings. These are: braced space frames, unbraced moment-resisting space frames, and box systems. In an unbraced moment-resisting space frame the lateral force



Δ_p - elastic deformation at yield load P_p

π_d - plasticity ratio = $\dfrac{\text{residual deformation}}{\Delta_p}$

Fig. B.1 Ductility Factor and Plasticity Ratio

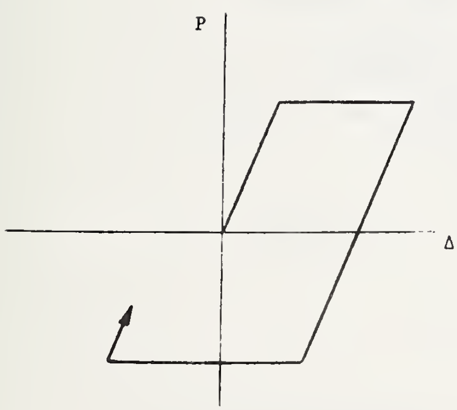


Fig. B.2 Ideal Elasto-Plasto Model of Load Reversal

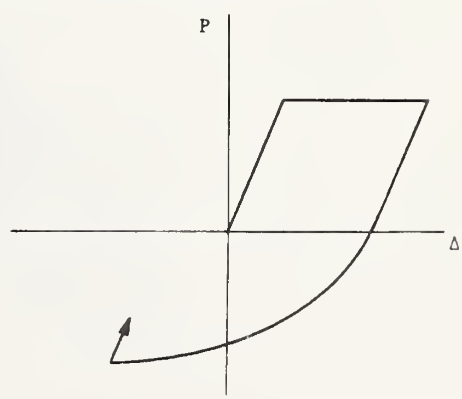


Fig. B.3 Elasto-Plastic Model With Bauschinger Effect

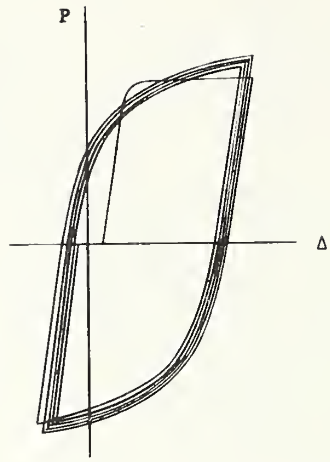


Fig. B.4 - Ideal Cyclic Loading - Stable Hysteresis Loops.

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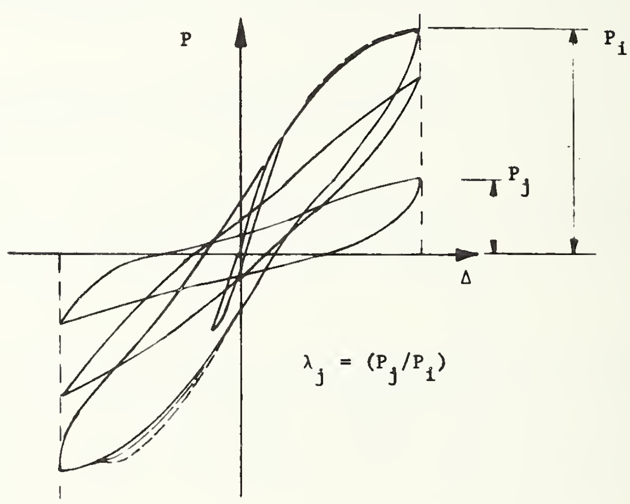
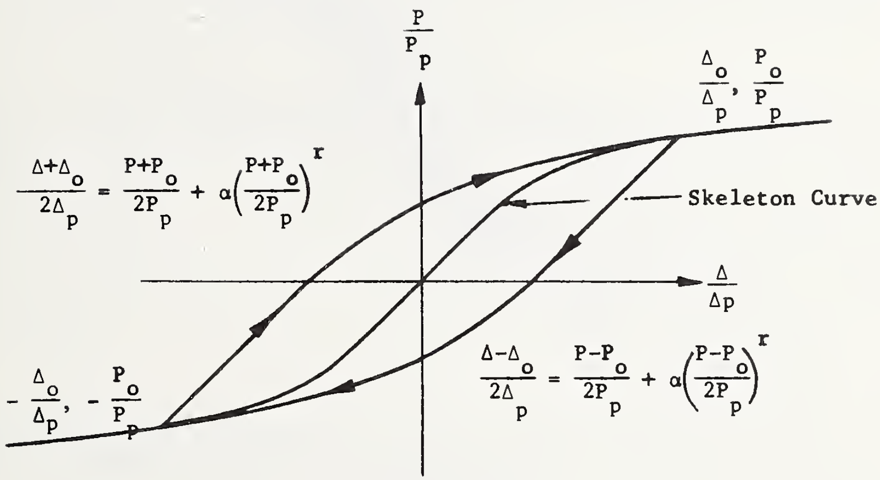


Fig. B.5 Hysteresis Loops for Reinforced Concrete Column Subjected to Shear Reversal. P - shear, Δ - lateral displacement (from Ref. 12)



<u>Skeleton Curve</u>:

$$\frac{\Delta}{\Delta_p} = \frac{P}{P_p} + \alpha\left(\frac{P}{P_p}\right)^r$$

Fig. B.6 Skeleton Curve (Ramberg-Osgood) and Corresponding Hysteresis Loop (after Masing)

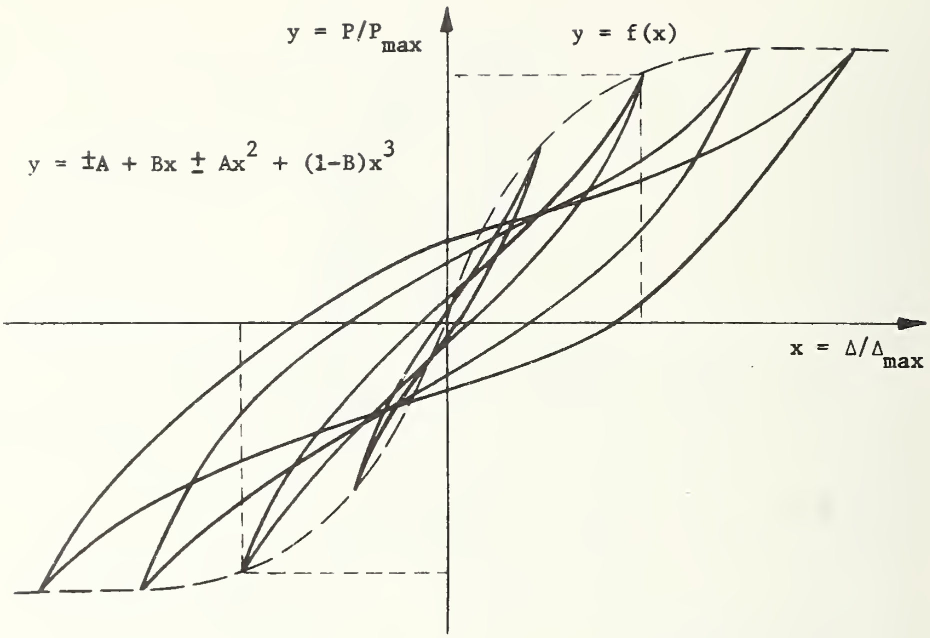


Fig. B.7 Cubic Skeleton Curve and Hysteresis Loops
(from Ref. 73)

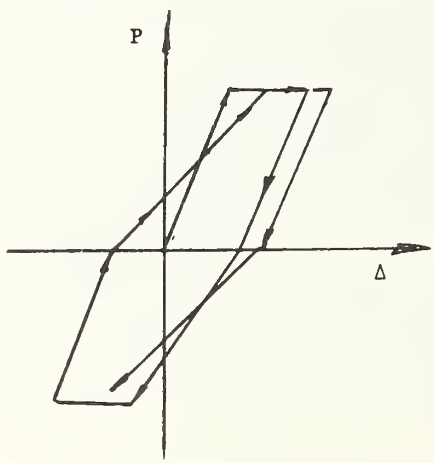


Fig. B.8 Degrading Stiffness - Hysteresis Loops
(from Ref. 82)

resisting system consists of beams, columns, and joints. The joint--which is the entire assemblage of members and connections at the intersection of the members--is a particularly important element. In a braced space frame the lateral force resisting system may consist of frames with X-bracing, frames with shear walls, or a structural core (box or tube). A "box" system is a structure in which the walls carry both lateral and gravity forces.

In evaluating the type of loading on a given element it is important to define the loading history. Because peak loadings are particularly important - it is necessary to define the maxima and minima values which indicate the range of variation in loading and possible magnitude of reversal. For example, in a multistory building, the gravity effect may be dominant in the upper stories, where the lateral force per story is small, and relatively less important in the lower stories. For these reasons moment and shear reversals in the beams are unlikely in the upper stories. For a realistic evaluation of loading - envelopes of axial and shear forces and bending moments should be constructed indicating the extreme values produced by combinations of gravity and lateral forces.

For interior columns of moment resisting frames gravity loads produce essentially axial compression in the columns, as the bending moments due to unbalanced loads or unequal adjacent spans are relatively small. Lateral forces are likely to produce full reversals of shear and moment; for this reason loading conditions on columns are often more critical, from one point of view of cyclic behavior. Loading conditions for interior joints in moment resisting frames generate reversals in column and beam sections adjacent to the joint, and cause high shears in the joint region. Some of the detailed mechanisms in resisting these forces will be discussed in greater detail in subsequent sections.

<u>In moment resisting frames</u> ductility in lateral displacement is developed through localized inelastic deformations which occur at certain critical regions of the structures, Fig. B.9. Usually these critical regions are located at the points of maximum internal forces or moments. Typically these critical regions are located in the beams (or girders) of the frame, or in the connections of the beams to the columns. This is not necessarily the case--but follows from the recognition of the fact that extensive yielding in the columns, particularly under combined bending and compression, may quickly lead to frame instability and collapse. Therefore, the usual design choice is to prevent formation of plastic hinges in the columns, and to allow them to form only in the beams or girders.

Localization of critical regions in which large inelastic deformations can take place suggests that a large portion of the structure may be free of such inelastic deformation and these "uncritical" portions may be designed in a more-or-less conventional way. There is some danger in this premise. Clearly in order to develop the full potential of the critical regions--a redistribution of forces (moments) may occur resulting in high tension, compression, or shear forces wehre they do not occur normally. Capacity to carry the redistribution forces must be provided in order to prevent premature failure and to develop fully the potential local ductility. In providing the capacity to carry these redistribution forces the possibility of cyclic load reversal must be considered.

Typical critical regions which may develop inelastic deformations under the action of seismic loadings are shown in Fig. B.9. They are:

1. End of beam and connection between beam and column where large negative moments can develop; likely location for plastic hinge; partial shear and moment reversal possible.

2. Midspan of beam where maximum positive moment may result in a plastic hinge; shear reversal possible in this region; also

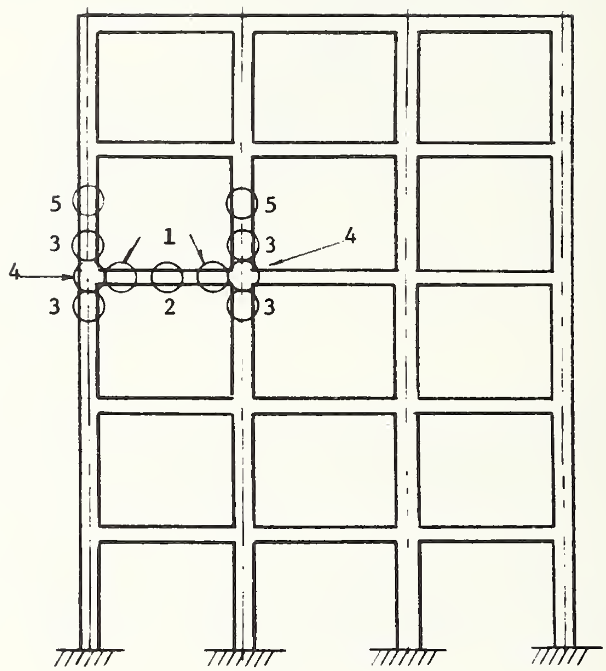


Fig. B.9 Critical Areas in Ductile Moment Resisting Frames (from Ref. 31) partial moment reversal is possible.

3. Ends of columns where maximum moments develop due to lateral forces; while normally plastic hinges should not be allowed to develop in these regions, values of maximum column moments closely approaching plastic moment capacity can be expected; these moments are likely to undergo full reversal; high shears can be developed (based on opposite sign moments at the column ends) - and these shears can undergo full reversals.

4. "Joint" regions (for steel frames may be referred to as panel zone) - undergo very high local shears; mechanisms of inelastic behavior in steel and in reinforced concrete are different - but in both cases may be very important; full reversal of shear is likely; local deformations may be a significant part of rotation at joint - increasing lateral displacements of frame.

5. Midheight of columns where bending moments are low, but full reversal of shear is likely; this is more important in reinforced concrete frame design than in steel design.

B.4 Material Characteristics

Behavior of structural elements under variable dynamic loading conditions can be predicted if material behavior is properly taken into account.

Under dynamic loading higher values of yield stress have been observed in structural steel and higher values of compression strength have been observed in concrete. Typically, yielding of steel under dynamic loading conditions may occur at stresses 10 to 15% higher than under slow monotonic loading conditions while values of modulus of elasticity of steel are essentially unaffected by the rate of loading. There is some question as to whether the steel under cyclic loading achieves the higher yield strength in other than the initial yield cycle.

Under monotonic loading and for large inelastic deformations strain-hardening of steel may significantly influence strength and deformation of the structural element. Under normal (relatively slow rate) cyclic reversed loading structural steel and reinforcing steel exhibit a "work-softening" characteristic, known as "Bauschinger effect," Fig. B.3. This greatly reduces the effective (tangent) modulus of elasticity at stress levels approaching yield strength after the first reversal. This reduction in modulus may result in local buckling at substantially lower stresses than yield strength, particularly in cases where full reversal occurs. Fig. B.10 shows results of a typical cyclic test on a specimen of reinforcing steel. After first reversal the stress-strain becomes nonlinear almost immediately, certainly at a much lower compression stress level than the initial yielding in tension. In subsequent cycles all loading curves in either tension or compression are highly non-linear, while the unloading diagrams are essentially linear. Effect of aging can be seen in the figure: portion ABCDE - was obtained at initial test time; EFG - was obtained 20 days later; GHJ was obtained 73 days after initial test.

In many structural elements the stresses are complex: in most steel structural elements the stresses are biaxial, and locally triaxial states of stress often exist. Similarly in concrete, the stresses are biaxial or triaxial. Effect of state of stress on yielding and failure under repeated reversed loads is not well established. For this reason, i.e., because of lack of information, the criteria for yielding and failure developed for monotonic loading, are used for cyclic reversed loading as well. Thus the Hencky-Mises yield criterion for steel is usually used.

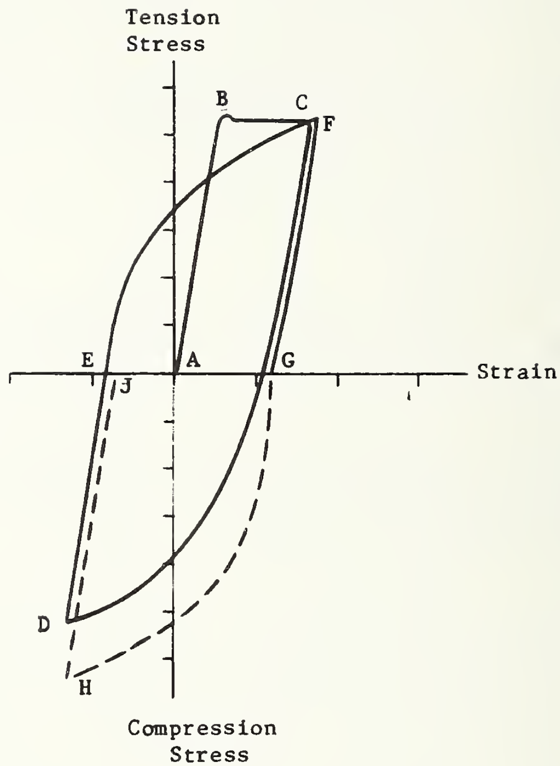


Fig. B.10 Typical Stress-Strain Hysteresis Loops for Reinforcing Steel Under Complete Reversal Cyclic Loading (from Ref. 58)

It is well recognized that steel may fail in a relatively brittle manner when subjected to biaxial or triaxial states of stress, when at low temperatures, and particularly when these conditions exist in conjunction with some metallurgical defects, welding defects, or local imperfections. These conditions are more likely to exist when welded steel elements using plates of large thickness are used in a building. Behavior of such elements under repeated reversed loads requires further investigation.

Compression strength of concrete under dynamic loading conditions may be 15 to 20% higher than under normal test conditions and values of modulus of elasticity of concrete under dynamic conditions are about 5 to 10% higher. No one criterion of failure under combined stresses has been widely accepted for concrete. The octahedral shear criterion seems to fit the available data the best. For the special case of cyclindrical biaxial compression the following empirical equation

$$f_{cu} = f_{co} + 4f_r$$

is widely used. In this equation f_{cu} is the longitudinal compression strength, f_r is the radial compression stress, and f_{co} is the uniaxial compression strength when $f_r = 0$.

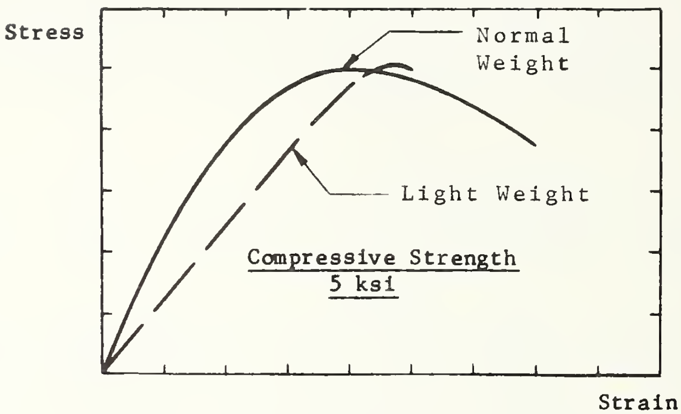
With repeated loading at fairly high stress levels, concrete (and masonry) develop microcracks which reduce the stiffness of these materials. Studies by Gerstle et. al., on plain concrete in compression have shown [27] that after one or two cycles the stiffness of the concrete indicated by the slope of the reloading stress-strain curve diminishes greatly. On the other hand, cycling with partial unloading in each cycle does not produce a significant reduction in stiffness.

Figure B.11 shows test results comparing stress-strain diagrams of hard-rock and lightweight aggregate concretes, both having a nominal compressive strength of 5 ksi [39]. Initially, the normal-weight concrete has greater stiffness than the lightweight and it peaks (reaches the compressive strength level) at a slightly lower strain. For the particular concretes tested, the lightweight crushed suddenly at the maximum stress level, while the normal-weight concrete failed gradually showing a typical descending branch. In a similar test of two concretes - lightweight and normal-weight, having compressive strength of only 3 ksi, the lightweight concrete failed more gradually than the normal-weight one.

The sudden (brittle) failure of the lightweight aggregate concrete is due to the fact that for this concrete (and aggregate) failure occurs through the aggregate when its strength is reached. Reduction in ultimate strain capacity of such lightweight aggregate concretes limit the use of high strength steel reinforcement and the ductility of reinforced concrete members. Additional information is needed for evaluating behavior of lightweight aggregate concretes under different loading conditions, with different types of aggregate, and with different arrangement of reinforcement. In the meantime, it seems that use of special concretes with ultimate strain capacity ε_u below 0.003 should be subjected to specific limitations with respect to strength and ductility, particularly where seismic loading must be considered.

B.5 Limited Damage

While the primary function of a building code is to ensure public safety, the designer is also interested in minimizing damage (clearly a building without any damage will protect its occupants as well!). As discussed in another Task Report in this Workshop, the code provisions recognize the technical and economic difficulties of designing structures which could survive any earthquake altogether without damage, and have accepted the principle that some damage may occur in moderate and major earthquakes. It



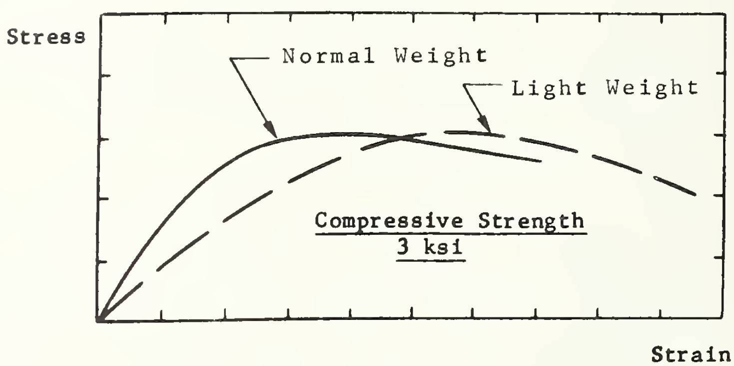


Fig. B.11 - Test Results for Lightweight and Normalweight Aggregate Concretes
(from Ref. 39)

is desirable to limit damage in moderate earthquakes to non-structural elements, and in major earthquakes some structural as well as non-structural damage is acceptable, provided the structure does not collapse. Furthermore, it is desired to limit the damage so that with appropriate repairs both serviceability (functioning under normal service conditions) and structural integrity (ability to withstand a subsequent major earthquake) can be restored.

There is some question as to the extent that the present codes provide for repairable damage. For example, reliance on amount of ductility which significantly reduces the seismic design forces, necessitates acceptance of substantial local damage due to large inelastic deformations. Depending on geometry of the frame, on the relative stiffnesses of the elements and on the mode of the local inelastic deformation, the effective local ductility (say for curvature in a frame girder) may be 3-5 times that of the ductility desired for lateral displacement or drift. If the latter is in the range of 4-6, than the former (curvature ductility) may have to be in the range of 10-30, with the consequence that such deformation may produce severe local structural damage.

Development of quantitative methods for evaluating the extent of damage and its repairability (i.e. restoration of both serviceability and integrity) is now only in planning stages. In some cases the repair is only "cosmetic" -- and there is very little reliable information on the adequacy of the various repair techniques.

Certain types of damage in steel framed structures can be repaired relatively simply by removing the damaged parts and replacing them by suitable steel elements welded into place. However, welding in place may introduce new imperfections and residual stresses. For these reasons special care must be exercised in quality control of such repair.

Reinforced concrete damage may be repaired in a variety of ways. Epoxy-injection method of repair may restore original load carrying capacity, or original stiffness. However, preliminary tests indicate that hairline cracks in concrete and damage due to slip between steel reinforcement and concrete are not effectively repaired by epoxy injection. This tends to reduce energy absorption capacity of repaired specimens. Also, depending on the properties of the polymer materials used - a brittle failure may occur upon dynamic reloading of the repaired element.

Damage and Repair. It is desirable to develop analytical methods which would result not only in the indications of the ductility factors required for meeting public safety objectives of the code, but would also output indications of the type and severity of local damage.

It is also necessary to obtain basic information on repairability (restoration of both serviceability and integrity) of a given level of damage. Developing such information would require an extensive series of tests. Such a program should include development of acceptable measures of nature and extent of damage, techniques of repair, quality control of repair, and its reliability.

The final decision as to the course to be followed - whether to repair or to demolish a structure - will be based on economic considerations. Some optimization techniques which take into account the cost of repair as well as cost of replacement are available.

C. Reinforced Concrete

C.1 General

Design of concrete structures for seismic loading has been receiving special attention for many years. Based on observations of damage in numerous earthquakes, both in U.S. and abroad, it has been suggested that concrete structures are particularly vulnerable to earthquakes. On the other hand, many concrete structures withstood severe earthquakes without significant damage. This suggests that there is nothing intrinsic to concrete structures per se to make them more vulnerable to earthquakes. Properly designed structures will perform well - whatever the material, although concrete may be less "forgiving" or less "tolerant" of improper design. For this reason, consideration of earthquake resistant design of concrete structures requires special attention.

The primary requirement in earthquake-resistant design is capacity to develop inelastic deformations (ductility) and to dissipate some of the energy input without severe damage. In concrete structures this can be achieved if brittle type failures such as crushing, local buckling, shear, loss of bond and anchorage are prevented, and "degradation" (reduction) in residual stiffness and strength are minimized. Degradation in stiffness - and loss of bond or anchorage - may be appreciably reduced by "confinement" or "basketing" of concrete by special transverse reinforcement.

Strength and ductility of reinforced concrete elements under monotonic axial and bending loads can be evaluated with a good degree of reliability using well established methods. Special problems associated with ductility, confinement of concrete, cyclic loading and reversal, buckling of steel reinforcement, shear capacity, bond and anchorage, are discussed in the following sections.

C.2 Ductility

Ductility is evaluated from the load-deformation relationship, Fig. B.1. Under axial compression this requires an examination of the axial load vs. shortening or elongation curve; in flexure - it requires an examination of moment-curvature relationship. While behavior under cyclic loading is of prime interest for earthquake-resistant and wind-resistant design, the general load-deformation relationships under monotonic loading gives an indication of both strength and ductility under cyclic loading. In reinforced concrete, cyclic loading tends to reduce the stiffness of the element, while strength and ductility of properly designed elements need not be adversely affected.

Ductility of reinforced concrete under monotonic loading has been demonstrated for pure axial compression and for flexure. Clearly ductility of a composite, such as reinforced concrete, requires ductile behavior of both or at least one component. Ductility of most reinforcing steels in tension is not subject to any conditions; ductility of the steel bars in compression is achieved when local buckling is prevented. In reinforced concrete this requires fairly close spacing of lateral reinforcement (ties or spirals).

Plain concrete in tension behaves essentially in a brittle manner - i.e. it cracks (fractures) at a relatively low tensile stress. There is some evidence that extensibility of concrete is enhanced by proper reinforcement, but as yet this behavior of concrete has not been generally mobilized in a practical way.

For plain (unreinforced) concrete in compression true ductility can not be achieved, because once the maximum compression stress f'_c is reached at a strain ε_0, the concrete can not support this level of stress with increasing deformation. The typical descending branch of the compressive stress-strain curve for concrete precludes true ductility. However, as long as the reduction in stress is gradual, i.e. the slope ($df/d\varepsilon$) of the descending branch is reasonably flat, and as long as the ultimate crushing strain ε_c for concrete is much greater than ε_0, some approximation of ductility is possible, particularly when concrete is combined with reinforcing steel.

Ductility in concrete structures is obtained when yielding of all reinforcement--tensile, compressive or lateral--can be developed prior to occurrence of crushing or shear failure in concrete. Flexural ductility is primarily influenced by amounts of reinforcement in tension and in compression p and p'; by yield strength of steel f_y, and by compressive strength of concrete f_c'. The effect of these parameters can be conveniently described in terms of a generalized reinforcement index $q^* = [(p-p')\ f_y/f_c']$. Ductility increases as q^* decreases. Thus increasing p' and f_c' tends to increase ductility; increasing p and f_y tends to decrease ductility. To ensure ductile behavior of a flexural member under monotonic loading it is usually sufficient to limit the reinforcement index q^* to about 0.75 q_b^*, where:

$$q_b^* = 0.85\ [(E_s\varepsilon_c)/E_s\varepsilon_c + f_y)] = 0.72\ [(87)/(87 + f_y)]$$

is based on assuming that yielding in tension steel and crushing of concrete at a compression strain 0.003 occur simultaneously ("balanced condition"). When plastic hinges develop in girders and when the redistribution of moments is used in design, only 0.5 of the reinforcement ratio producing balanced conditions is allowed in order to prevent premature crushing. For seismic loading conditions the 0.5 "balanced" ratio limitation is also specified in the ACI Code. When p approximately equals p' (for example in beams where seismic effects are much greater than gravity effects, so that practically full reversal of moments can be expected at the crtical section) - the limitation is satisfied whatever the value of q. However, it may be desirable to limit the amount of tension reinforcement so that $(p\ f_y/f_c') < q_b^*$ in order to prevent or to limit crushing.

In general, as f_c' increases the value of q^* decreases and leads to an increase in ductility. This conclusion must be accepted with reservations--as some concretes tend to behave in a more brittle manner as strength increases [39]. Brittleness of some concretes may be associated with aggregate weakness and therefore a general conclusion on improvement of ductility with high strength concrete does not seem appropriate.

Typical reinforced concrete elements with conventional ties fail in a rather brittle manner when subjected to axial compression. The mechanism of failure is due to crushing of concrete cover and exposing the longitudinal steel, which then buckles and is followed by failure of the central portion of concrete. To achieve a more ductile mode of failure Considere [77] demonstrated the effectiveness of spirally reinforced concrete columns for resisting axial compression. He discussed the increased capacity and ductility of confined concrete and practical details of construction to achieve effective confinement. Brandtzaeg, Richart, and others [83] obtained additional evidence on effectiveness of spirally reinforced concrete columns under axial compression. Recent tests and experience have demonstrated excellent performance of spirally reinforced concrete columns; however, their effectiveness under reversed cyclic lateral loads has not been fully evaluated [12].

The strength of a reinforced concrete element under compression and bending is usually described by a P-M interaction diagram, such as shown in Fig. C.1.

Axial compression also influences moment-curvature relationship. Because axial compression postpones initiation of flexural cracking, the initial slope of M-ϕ curve remains steep up to a larger value of moment. Also increase in axial compression up to $P = P_b$ increases moment capacity from M_o to M_b.

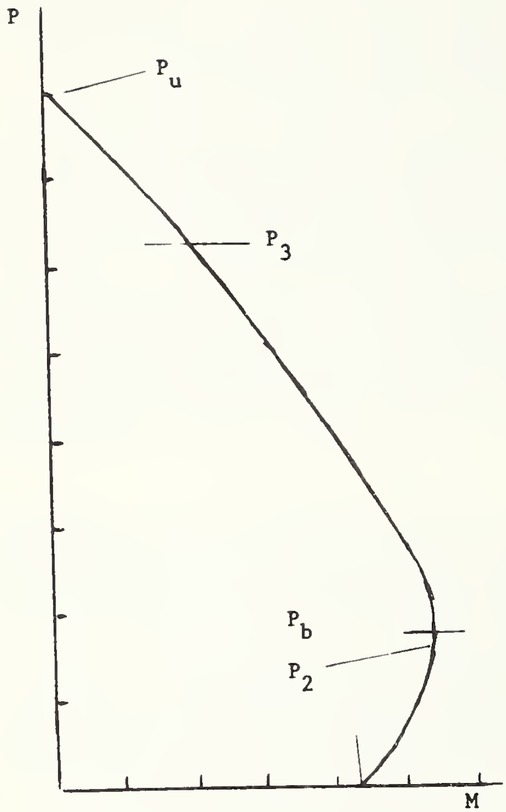


Fig. C.1 P-M Interaction Curve

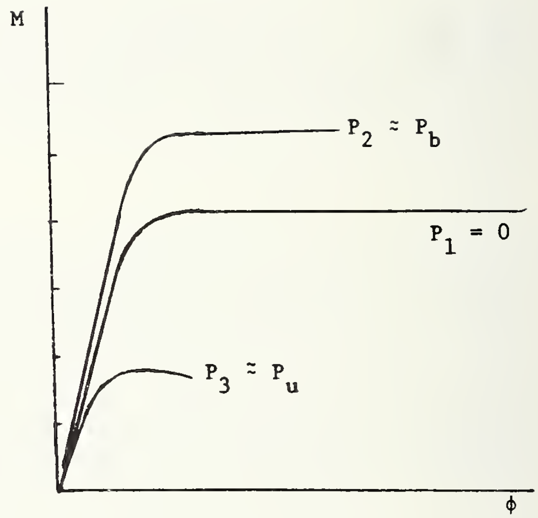


Fig. C.2 Effect of Axial Compression on M-ϕ Curves

However, even in this range $0<P<P_b$, the influence of axial load is to produce crushing at lower curvature than for pure bending. With further increase in P, crushing occurs at lower curvature values, and the effect is to produce a more brittle mode of failure, that is to reduce the ductility which can be attained under pure flexure, Fig. C.2.

C.3 Confinement

When concrete is confined laterally (hydrostatically - by a metal jacket, or by a closely spaced circular spiral) it can withstand large compressive strain as well as large compressive stress, and can be characterized as a "ductile" material. However, most recent studies of concrete confined by hoops and rectangular "spiral" indicate that these are not fully effective [2]. Typical results are shown in Fig. C.3. If confinement comes into play before concrete reaches its maximum stress, then the stress increment Δf_c may be substantial, but as the hoops reach their yield strength they cannot provide additional confinement, and the stress-strain curve follows a descending branch similar to that of plain concrete. In this case, ductility of confined concrete is not reliable. When confinement is "delayed", its contribution maintains the ductility - but the strength increment is small. The "delay" in confinement can only be achieved when the concrete has a low value of Poisson ratio initially, and its increase starts at a high level of compression stress, as close as possible to compressive strength f_c'.

Confining effectiveness of lateral reinforcement depends on its shape. As shown in Fig. C.4 the effectively confined zone ("core") is much smaller for columns with square single hoops than it is for a spiral column. Assuming full confinement (i.e., additional strength of concrete due to confinement is taken as 4x confining pressure) the required spacing s of spirals or hoops is

$$s = 2\pi(f_y/f_c')(\frac{d_h}{D_c}) \frac{d_h}{[(A_g/A_c)-1]}$$

where d_h - is the diameter of spiral or hoop wire, D_c - is core diameter, A_g - gross area, and A_c - effective core area. Typical required spacing s for hoops and spirals is shown in Table C.4.

Table C.4 HOOP AND SPIRAL REQUIREMENTS BASED ON CONFINEMENT OF CONCRETE

Column size	18 x 18 in. square
Concrete cover	1.5 in.
Spiral steel	f_y = 60 ksi
Tie steel	f_y = 40 ksi

Concrete Strength		3 ksi		6 ksi	
Hoops	Size	No. 2	No. 3	No. 2	No. 3
	Spacing, in.	3/8	7/8	1/4	3/8
Spirals	Size	No. 2	No. 3	No. 2	No. 3
	Pitch, in.	3/4	1 3/8	3/8	3/4

It can be seen from the table that when the strength of concrete is high and yield strength of hoop steel is kept to grade 40, the required spacing

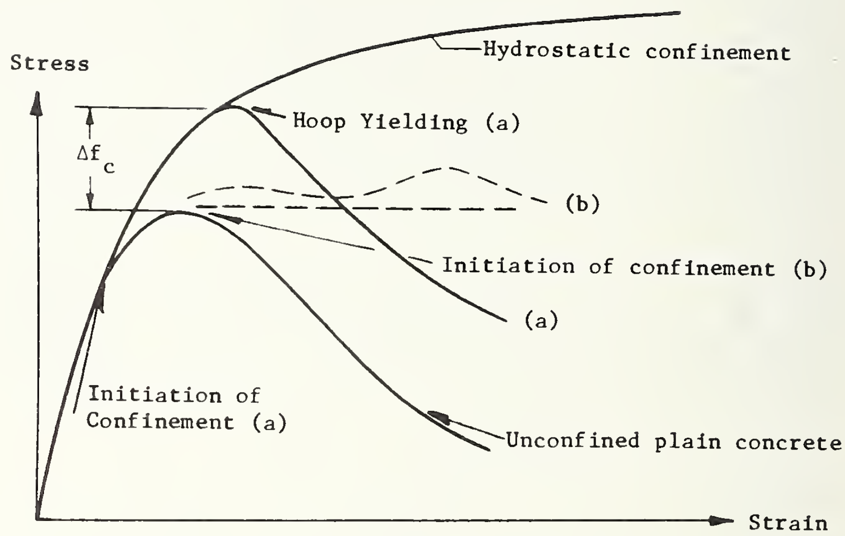


Fig. C.3 Effect of Different Methods of Confinement on Stress-Strain Characteristics of Concrete in Compression.

(from Ref. 2)

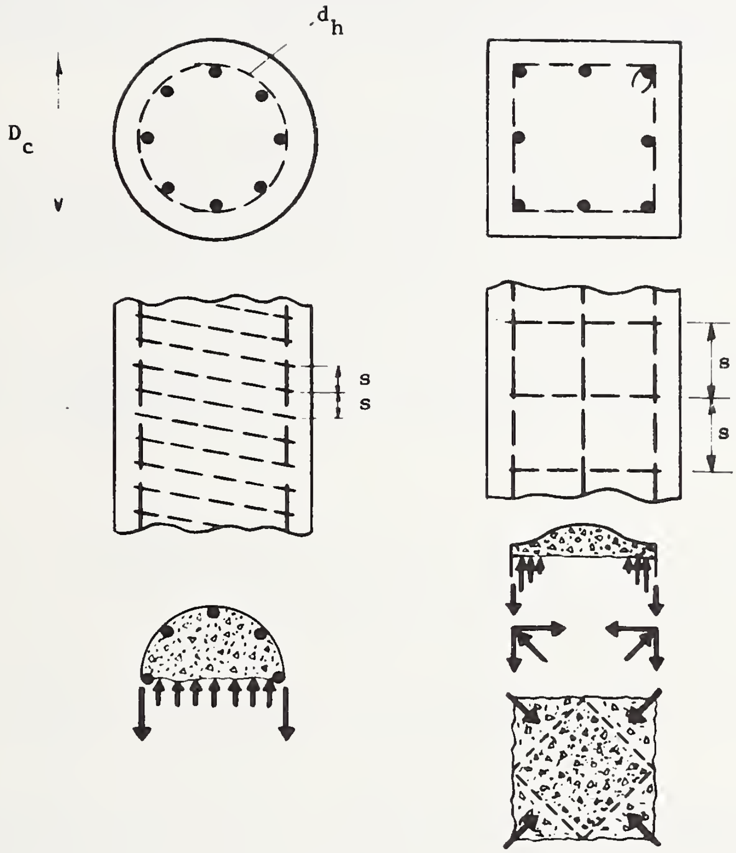


Fig. C.4 Effect of Hoop Shape on Confinement may equal hoop wire diameter! Clearly, hoops can not be spaced so closely, which simply means that adequate confinement can not be developed by a single hoop.

Present (1971) SEAOC design recommendations provide for use of multiple ties assigning them an effectiveness somewhat less (about 20% less) than that of a spiral. This assumption has not been substantiated by tests. The SEAOC provision allows for a maximum hoop spacing of 4 in., and it is not clear that an effective confinement can be provided with such spacing even with multiple hoops.

Two other factors tend to limit effectiveness of hoops. The intensity of the confining stress depends on the hoop tension σ_h which is reacted locally by <u>bearing stress</u> σ_{br} on the concrete core. For <u>circular hoops</u>, Fig. C.5 this stress σ_{br} can be expressed in terms of σ_h:

$$\sigma_{br} = (2\sigma_h \pi d_h^2 / 4d_h . D_c) = 1.57\ \sigma_h(d_h/D_c)$$

Depending on concrete characteristics and the ratio of hoop wire diameter d_h to core diameter D_c this stress may limit hoop effectiveness. For example, if the bearing stress is limited to 3 ksi (as for some lightweight aggregate concretes) and for D_c = 12 in., d_h = 0.375, the limiting hoop stress will be

$$\sigma_h = \sigma_{br}(D_c/1.57d_h) = 61.2 \text{ ksi}$$

In this case the effectiveness of high-strength hoop wire cannot be fully realized. The following table shows maximum values of σ_h for 3/8 in. and 1/2 in. diameter steel hoops for two types of concrete having bearing strengths σ_{br} of 3 and 5 ksi, and three columns sizes having core diameters of 10, 15 and 20 inches.

Table C.2 HOOP STRESS LIMIT VS. BEARING STRESS

(Hoop Stresses Shown in Ksi.)

σ_{br}	3 Ksi		5 Ksi	
d_h-in / D_c-in	3/8	1/2	3/8	1/2
10	51	38	85	63.5
15	76.5	57	127	95
20	102	76	170	120

It can be seen that for concretes which cannot withstand a high local bearing without crushing, effective hoop stress intensity may be limited to 40-70 ksi, particularly for concrete columns with relatively small core diameters.

Hoops prevent bulging of the core locally--but depending on the spacing of the hoops some bulging may take place in the interval between the hoops. This action produces some local transverse cracking in the zone near the hoop which reduces the effective core area. The cracking is shown in Fig. C.5.

This cracking depends on hoop spacing and concrete characteristics (particularly its tensile strength) and causes a reduction in the core area which may reduce the effectiveness of confinement and, in extreme cases, may greatly reduce the ductility of the column. The depth of the crack c may be related to hoop spacing, so that

$$c = \alpha s$$

and the diameter D_e of the effective core area is then

$$D_e = D_c - 2c = D_c(1-2\kappa)$$

where κ is $(\alpha s/D_c)$. The area of the effective core is A_e and its ratio to the normal core area A_c are:

$$A_e = (\pi D_e^2/4) = (\pi D_c^2/4)(1-2\kappa)^2 = \beta A_c$$

$$\beta = (A_e/A_c) = (D_e/D_c)^2 = 1 - 4\kappa(1-\kappa)$$

It is clear that κ cannot exceed 1/2, which correponds to the stage when transverse cracks have propagated through the entire core, i.e., the column no longer is in compression. The core area reduction factor β calculated for a given value of κ is shown in Table C.3.

Table C.3 CORE AREA REDUCTION FACTOR β VS. CRACK DEPTH FACTOR

$\kappa = \alpha s/D_c$	0.01	0.03	0.06	0.10	0.15	0.20	0.25	0.30
$\beta = A_e/A_c$	0.96	0.88	0.78	0.64	0.41	0.36	0.25	0.16

It can be seen that peripheral cracks extending only 3% of the diameter into the core reduce the effective core area by 12%.

In the absence of effective "confinement", advantages may be derived from multiple rectangular hoops as a consequence of "binding" effect (sometimes referred to in European literature as "stitching through" effect, or in American literature as "basketing" effect). Also use the multiple rectangular hoop (ties) provides necessary restraint of longitudinal reinforcement against local buckling of reinforcing steel and in this way plays an important role in developing full strength of compression reinforcement.

C.4 Cyclic Loading and Reversal

Studies on singly-and doubly-reinforced concrete beams under cyclic loading without reversal [28, 29] have shown that there is no reduction in strength with cycling, Fig. C.6, when amount of tension reinforcement is below $(pf_y/f'_c) = 0.25$, and when adequate shear reinforcement and hoops (stirrups) to prevent local buckling of compression steel after crushing of concrete are provided.

Studies [31] on beams under cyclic loading with reversals indicate that while properly detailed beams can develop their full strength (as determined for static loading) there is a reduction in initial stiffness at successive

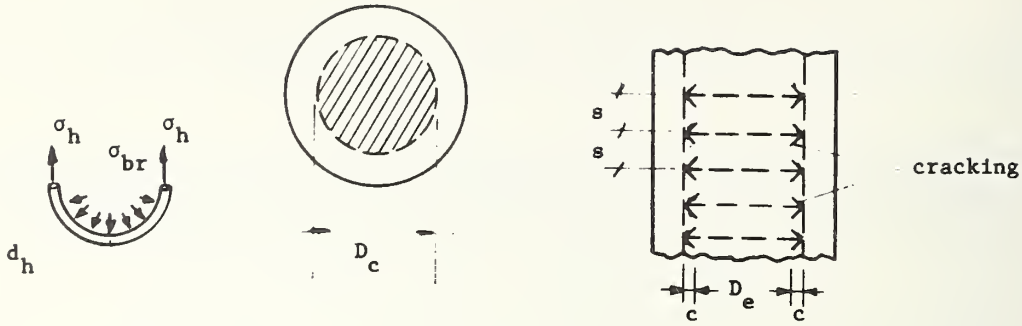


Fig. C.5 Confinement Using Circular Hoops or Spirals

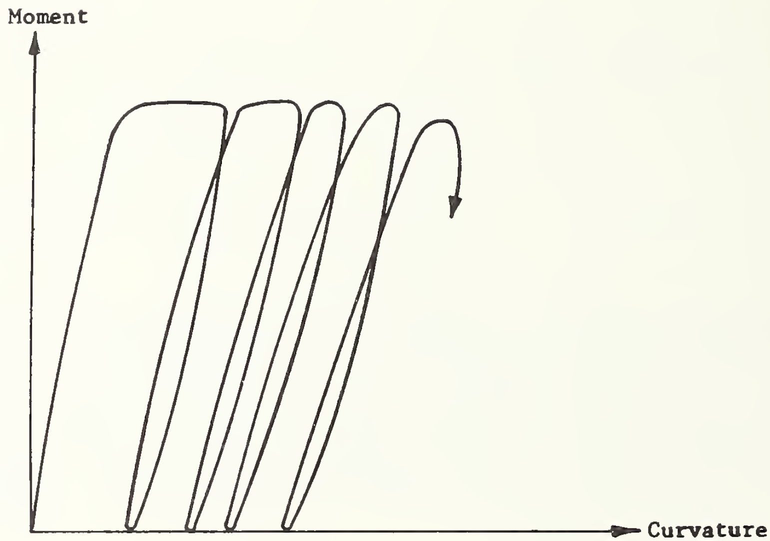


Fig. C.6 Moment Curvature Relationship of Reinforced Concrete Beams Under Cyclic Loading

(from Ref. 29)

cycles in which load or deformation is increased beyond that in any previous cycle. The energy absorption and energy dissipation capacities are also sensitive to the loading history. While elasto-plastic idealization may overestimate energy dissipation capacity by as much as 100%, the capacity for energy dissipation may be substantial--increasing with the amount of inelastic deformation (compare energy dissipation in cycle 20 with that in cycle 32 as shown in Fig. C.7).

The factors which influence the shape of the hysteresis loop for a rectangular doubly-reinforced concrete beam are the following: (a) inelastic deformation of steel reinforcement - taking into account alternating plasticity, (b) extent of cracking in concrete - taking into account incremental nature of crack width, (c) effectiveness (or deterioration) of bond between reinforcing steel and concrete - including the possibility of slip (or partial loss of anchorage) at some distance from section being considered, and (d) presence of shear (diagonal) cracking or shear deformation.

The process of local deformation with large load (or inelastic strain) reversals, Fig. C.8 has been described in reference 31.

In a doubly-reinforced concrete member loaded well into the inelastic range the major flexural crack denoted as C_1^t in Fig. C.8.1, will not close completely on unloading, Fig. C.8.2. The degree of opening will depend on how far the tensile steel was strained into the plastic range during first loading. If the tensile steel was strained well beyond the initial yielding, a crack C_2^b may originate on the bottom side during unloading.

If the member is then loaded in the opposite direction, the critical section which has already been cracked will offer considerably less resistance to rotation than during the first loading. This decrease in resistance may be caused by the fact that two faces of the former crack C_2^t are not in perfect contact. The crack at the top may close or not, depending on the peak value of the reversed load P_3 relative to P_1 on the amount of top and bottom reinforcing steel, and on other factors. Because the concrete in the two faces has undergone a process of disruption, a reduction in the stiffness of the critical region should occur even if the crack closes.

If the load P_3 in the reversed direction reaches the same peak value as P_1, the width of the crack C_3^b would be larger than C_1^t -- observed under P_1. It is evident that if the member is now unloaded, the critical cross-section will be cracked throughout C_4^t - C_4^b and the width of the crack will depend mainly upon the amount of yielding of the steel, the effectiveness of composite action (bond) between steel and concrete and, to a lesser extent, on the degree of the concrete disruption.

At the start of a new cycle of alternating load, the original doubly-reinforced concrete section will behave as steel cross section represented by the tensile and compressive steel reinforcement. If the reinforcing steel exhibits pronounced Bauschinger effect, this will result in the reduction of stiffness of the critical region. Furthermore, at this stage, the presence of shear will tend to displace the faces on either side of the crack relative to one another, Fig. C.8.5. The tendency is resisted by the "dowel action" of the main reinforcement and will cause the steel bars to be pressed against the concrete and consequently may lead to longitudinal splitting of the concrete. The degree of damage introduced by this effect of shear will depend

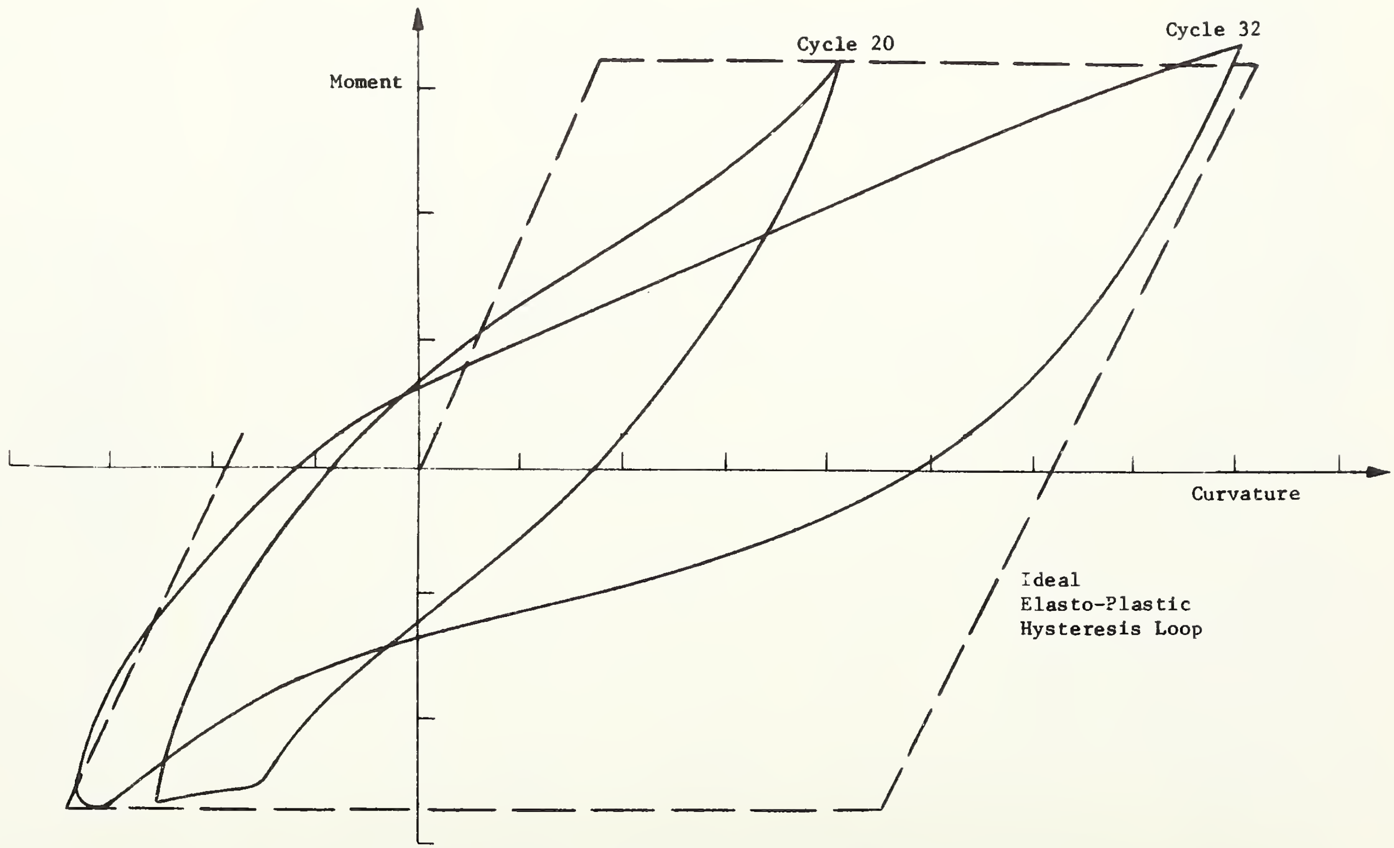


Fig. C.7. Effect of Loading History on Energy Dissipation Capacity
(from Ref. 31)

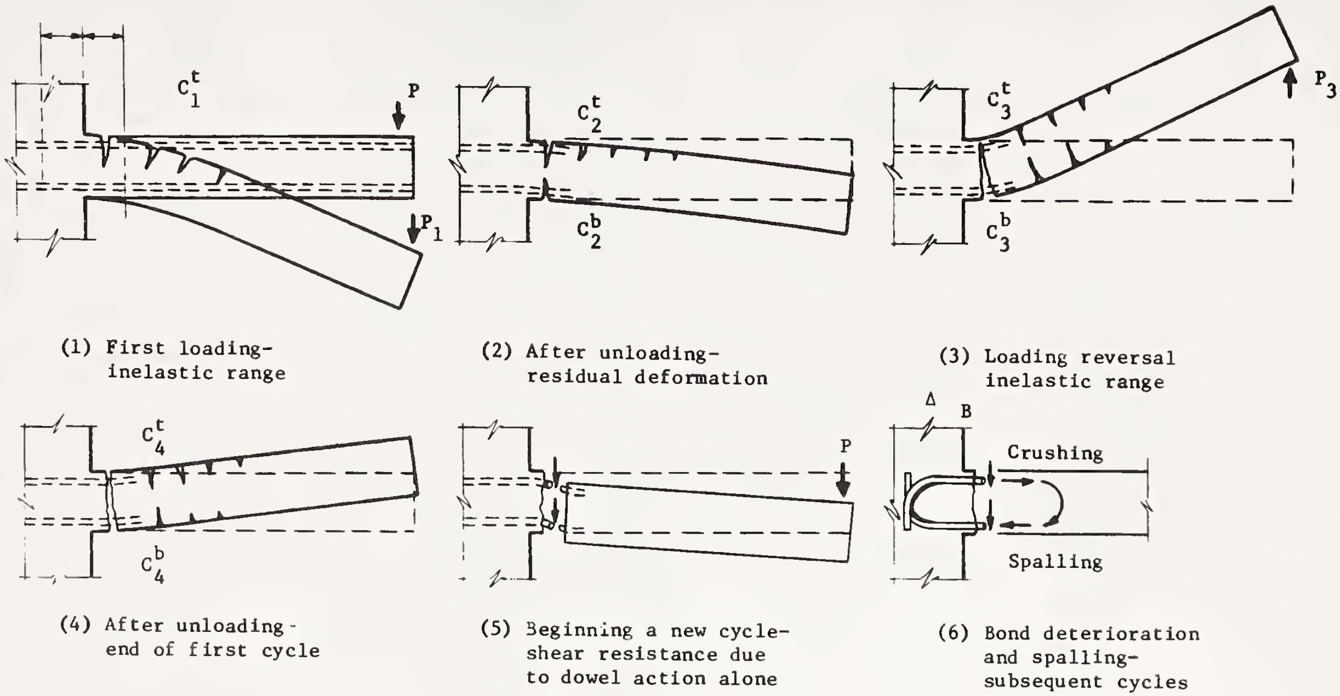


Fig. C.8 Effect of load reversal on reinforced concrete element
(from Ref. 31, 69)

on tie spacing, but it is most likely that it will affect the bond and consequently the overall stiffness of the member.

Deterioration of bond which may be increased by the shear in sections cracked throughout may cause local failure at any point of discontinuity in the main reinforcement and particularly in the beam-to-exterior column joint. For example, in the joint shown in Fig. C.8.6, deterioration of bond along length BA due to alternating stress in the steel and the effect of any shear force acting in section C^t - C^b may lead to the development of high radial stresses at A and a bearing failure may occur. This may lead to spalling of the concrete beneath the bar, particularly if the amount of concrete cover is inadequate. This type of failure has been observed in several investigations and in field inspections of earthquake damage.

The behavior of the critical region at the girder-column joint may be affected both by the shear and also by the details of anchoring the girder reinforcement. In particular, steel detailing in a monolithic connection between the exterior column and the girder may contribute to stiffness deterioration.

Precise evaluation of the stiffness is not possible at present because of the absence of adequate basic experimental information. However, it is clear that once a sufficient number of cycles occur, bond between cracks may completely disappear and, therefore the initial stiffness (at M = 0) will be provided by the steel reinforcing alone, where concrete acts just as a filler and consequently any stiffness deterioration from then on will be due to more pronounced Bauschinger effect as the inelastic deformation increases. Thus, the stiffness degradation of inelastic regions subjected to pure bending reverslas is a consequence mainly of bond deterioration and Bauschinger effect.

While repeated reversal loadings have detrimental effects on the initial stiffness of reinforced concrete members -- due to the yielding of steel and bond deterioration between cracks --, they may have beneficial effects on the ductility of such members. In the case of a reinforced concrete region subjected to monotonically increasing bending moment, sudden unloading may occur due to crushing of the outer concrete cover and buckling of compression steel, as illustrated in Fig. C.9. However, under repeated reversed loading, as a result of the gradual opening of cracks due to the yielding of steel, the sudden (brittle) failure of concrete is delayed because the concrete is not strained in compression until the cracks have completely closed, i.e., the section is converted from conventional reinforced concrete, in which ductility is controlled by the semi-brittle concrete, to essentially a steel section in which the compression steel is restrained against buckling by the concrete cover and therefore provides high ductility, Fig. C.10.

C.5 Buckling of Reinforcing Steel

In reinforced concrete elements (beams and columns) there are a number of reasons for spalling of concrete cover: crushing at a high compression strain, concentration of local stresses and strains at transverse steel hoops, high shear and bond stresses (splitting preceding spalling), etc. Compression of reinforcing steel, which is not confined by the concrete or properly restrained (braced) by lateral ties (hoops or stirrups), will lead to local buckling, Fig. C.11. This problem was treated in Ref. 76 for static monotonic loading. It was suggested that spacing of ties s (stirrups) can be derived from the condition that the critical inelastic buckling stress shall not be less than the yield strength of reinforcing steel. If D is the diameter of the longitudinal reinforcement then

$$(s/D) = B(E_t/F_y)^{1/2}$$

where E_t is tangent modulus at yield strength based on non-linear stress-

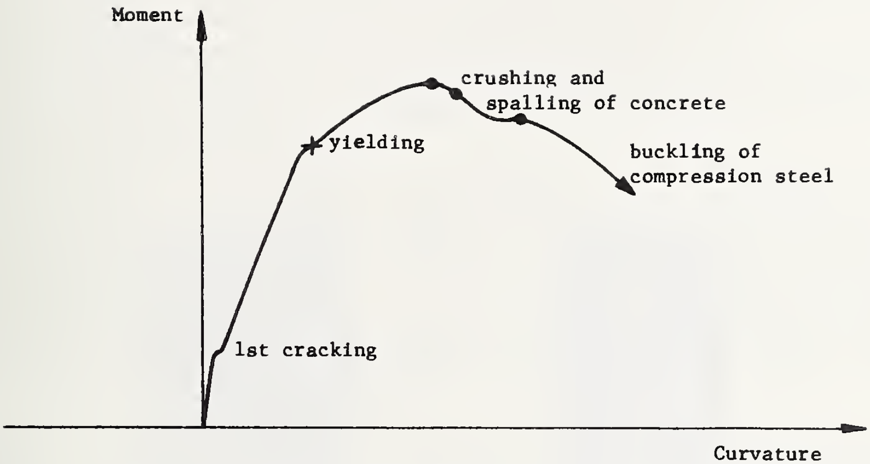


Fig. C.9 Reinforced Concrete Beam Under Monotonic Loading.
(from Ref. 31)

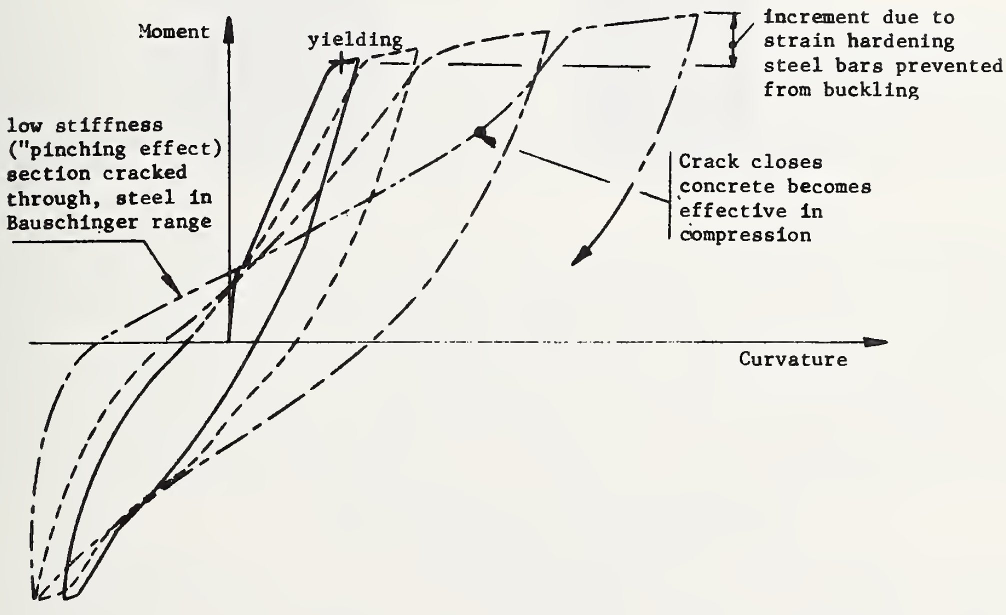


Fig. C.10 Reinforced Concrete Beam Under Cyclic Reversed Loading
(from Ref. 31)

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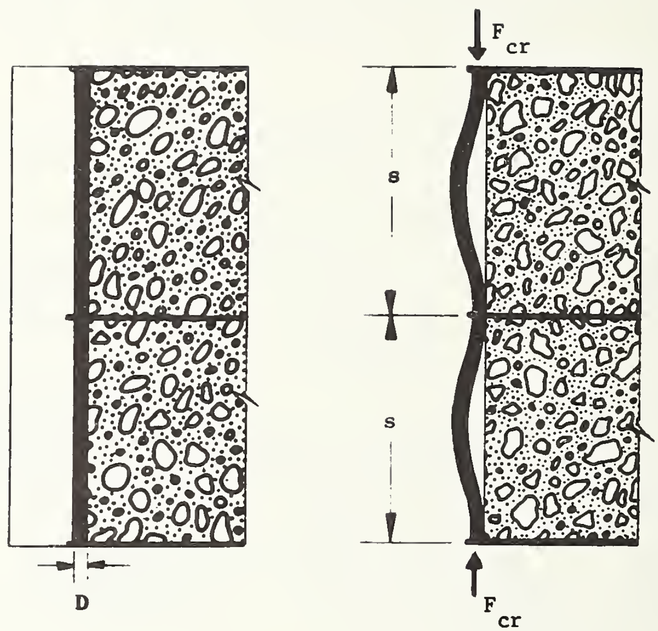


$$F_{cr} = C\pi^2 E_t (r/s)^2 = F_y$$

$$s = \pi\sqrt{C}(E_t/F_y)^{\frac{1}{2}}(D/4)$$

Fig. C.11 Buckling of Reinforcing Steel Exposed After Spalling of Concrete Cover

(from Ref. 76)

strain diagram (elasto-plastic idealization is not sufficient for this representation), and $B = 0.5\pi C^{1/2} = 2.2$. For typical values of F_y and E_t = 10,000 ksi values of s vary from 16D to 12D, the closer spacing corresponding to F_y = 85 ksi.

If the compression steel reinforcement is expected to withstand stresses (strains) in the strain hardening range, then the values of E_t should be chosen accordingly. Estimating E_t = 1,600 ksi, the spacing of stirrups (ties) should not exceed 6D (or even 5D for higher values of F_y).

Under reversed loading a value of E_t = 2,000 ksi may be appropriate for reinforcing steels having F_y = 50 ksi. The tie spacing s under these conditions would vary from 7D to 5D.

It should be noted that the 1971 reversion of the SEAOC recommendations calls for a maximum spacing s not greater than 8D. This requirement is clearly justified for cases when reversal is present. For cases of monotonic loading the appropriate value of s depends on the extent of inelastic deformations of reinforcing steel bars. If it extends into strain hardening range, then the value of s should not exceed 5 to 6D, as shown above. However, where no reversal occurs and where the inelastic deformation in the elements is also limited - then the tie spacing may be increased by about 50%.

In providing lateral restraint against buckling three factors are of importance: (a) size of hoop (or ties), (b) effective restraint of the longitudinal bar (absence of any freedom of movement because of the flexibility or gaps between ties and main bars), (c) adequate anchorage of hoop ends (prevention of "opening up" of hoop or tie).

The size of the hoop is important as it affects its strength and stiffness. An analytical evaluation of required tie stiffness [76] showed that when restraint was provided by extensional stiffness (ties bend around main bar) rather than flexural stiffness, the required tie size was quite small. For such shapes the size should be selected to prevent premature local crushing in concrete or fracture in the tie restraining the lateral deformation of concrete.

To provide effective restraint the ties should be bent around the bar (limiting the included angle to somewhere between 90-135°), should be in contact with the bar, and attached to it by tie wire. While under monotonic loading a gap between the bar and the tie may not be important, as it is likely to be filled by concrete, under cyclic reversal of loading any concrete between the tie and the bar will be destroyed. With successive reversals of tension and compression in the main steel reinforcement and the surrounding concrete the gap will allow lateral movement of the bar, thus aggravating initiation of buckling. Reasonable tolerances of tie placement with respect to main bars, which will minimize possible buckling of main bars require further study.

To prevent "opening-up" of hoops through anchorage failures, it is advisable to bend each end through a 135° angle with a suitable straight extension (about 10 bar diameters) in a region of concrete which is preferrably subjected to compression. Hairpin ties or caps may not be fully reliable and their effectiveness should be investigated experimentally.

C.6 Shear

High intensity shearing stresses in reinforced concrete elements produce inclined ("diagonal tension") cracks which, if allowed to propagate may cause failure by one or more of the following modes: (1) stress concentration at the tip of the crack in the compression zone and subsequent crushing, and (2) dowel action of steel reinforcement producing secondary longitudinal splitting along the bars.

The first mode may lead to buckling of compression steel after crushing; the second - leads to loss of bond, and may result in failure due to loss of anchorage. Even in the absence of failure - loss of bond results in substantial reduction in stiffness.

Shear failures can be prevented by arresting the propagation of inclined cracking by adequate transverse reinforcement (ties or stirrups) which should limit diagonal cracking and prevent crushing or splitting due to shear. Also the transverse reinforcement should be sufficient to resist a portion or all of the shear on the section.

The magnitude of the shearing force V for which the element must be designed should be at least equal to or greater than either:

$$V_u = (V_D + V_E)$$

or

$$V_u = [(M_{p1} + M_{p2})/h] + 0.5\ wh$$

where V_D and V_E are shear forces due to factored dead and earthquake loads respectively, M_{p1} and M_{p2} are plastic hinge moments at the ends of an element or its segment, h is the distance between the plastic hinges, and w is the intensity of uniformly distributed load on the element.

The total shear capacity of a reinforced concrete element usually has three components:

$$V = V_c + V_{sw} + V_{sl}$$

where V_c = concrete shear resistance (if any),

V_{sw} = transverse (web) steel shear resistance developed by tension in stirrups (ties),

V_{sl} = longitudinal steel shear resistance developed by dowel action of main bars.

The concrete component V_c is determined by the shear capacity of the uncracked section, with some contribution from the aggregate interlock shear resistance (frictional resistance). Normally, the shear strength of the uncracked portion of concrete and the aggregate interlock are enhanced by compression up to a limit stress of about 0.8 f'_c.

For reinforced concrete elements under monotonic loading the value of V_c may be taken as:

$$V_c = \phi(1 + 0.0005\ N_u/A_g)(2\sqrt{f'_c})(bd)$$

where N_u - design axial load occurring simultaneously with shear V_u, positive for compression, negative for tension, and A_g is gross area of the section.

The shear resisted by the transverse steel reinforcement may be approximated by the conventional assumptions of equal participation of all stirrups (ties) in a projected length of a crack inclined 45° to the axis, in which case the number of stirrups resisting the shear is (d/s). Then

$$V_{sw} = f_{sv} \cdot A_v \cdot (d/s)$$

Under monotonic loading, when failure occurs at a section critical in shear, and all the effective stirrups are assumed to yield:

$$V_{sw} = f_y \cdot A_v \cdot (d/s)$$

The shear resisted by the longitudinal steel is developed by dowel action of steel bars. Effectiveness of such action depends greatly on preventing splitting along longitudinal reinforcement (i.e., on spacing of stirrups or ties). Usually contribution of longitudinal steel shear resistance is neglected (i.e., $V_{sl} = 0$). This is a conservative procedure - which is particularly desirable because the estimate of V_{sw} based on $f_{sv} = f_y$ may be somewhat unconservative.

In the absence of any reversal of shear and moment but with cyclic inelastic deformations the shear capacity of concrete may be reduced: diagonal cracks may propagate further into compression zone, local bond may deteriorate, diagonal cracks may become wider and not close after the disturbance. Thus, the contribution of concrete to shear resistance may be reduced to about 1/2 that normally assumed as effective.

Under reversed loading diagonal cracks develop in both directions forming typical X-shaped cracks. In this region the shear must be carried by the stirrups with some contribution by the dowel action. The magnitude of shear carried by the dowel action is highly indeterminate, as it is only possible if longitudinal splitting along the main bars is prevented by closely spaced stirrups. No shear whatever can be transmitted by the concrete. Even "aggregate interlock" is unreliable as it is destroyed by loosening and fracturing of the aggregate with cyclic reversals of loading and "working back and forth" along the diagonal cracks. Therefore, in beams where full reversal takes place, assuming V_c and V_{sl} are zero, the stirrups must be proportional for the entire shear, i.e., $V_c = 0$, and

$$A_v = \frac{V.s}{f_y d} \qquad \text{or} \qquad s = \frac{A_v f_y d}{V}$$

Effects of detailing special transverse reinforcement in the area of reinforcing bar cut-offs and splices on behavior under cyclic repeated loads have not been studied extensively. Therefore, provisions in ACI 1971 Code Sections 12.1.6.1, 12.1.6.2, 12.1.6.3, and 7.12.6 are to be considered minimal.

C.7 Combined Axial Compression, Bending, and Shear in Columns

In ductile moment-resistant reinforced concrete frames the columns are subjected to axial loads (primarily compression due to gravity) and to reversals of shears and moments due to lateral loads (seismic, wind, etc.). Under reversed shear and moment loading, with relatively constant compression, the main effects to guard against are reduction in shear capacity (due to diagonal cracking) and buckling of compression steel (after spalling of cover). The reduction in shear and buckling of compression steel are more critical in the presence of reversal.

Two possible cases are shown in Fig. C.12. It can be seen that the values of maximum moment are about the same for the two cases. However, initial loading conditions are quite different, and while almost complete

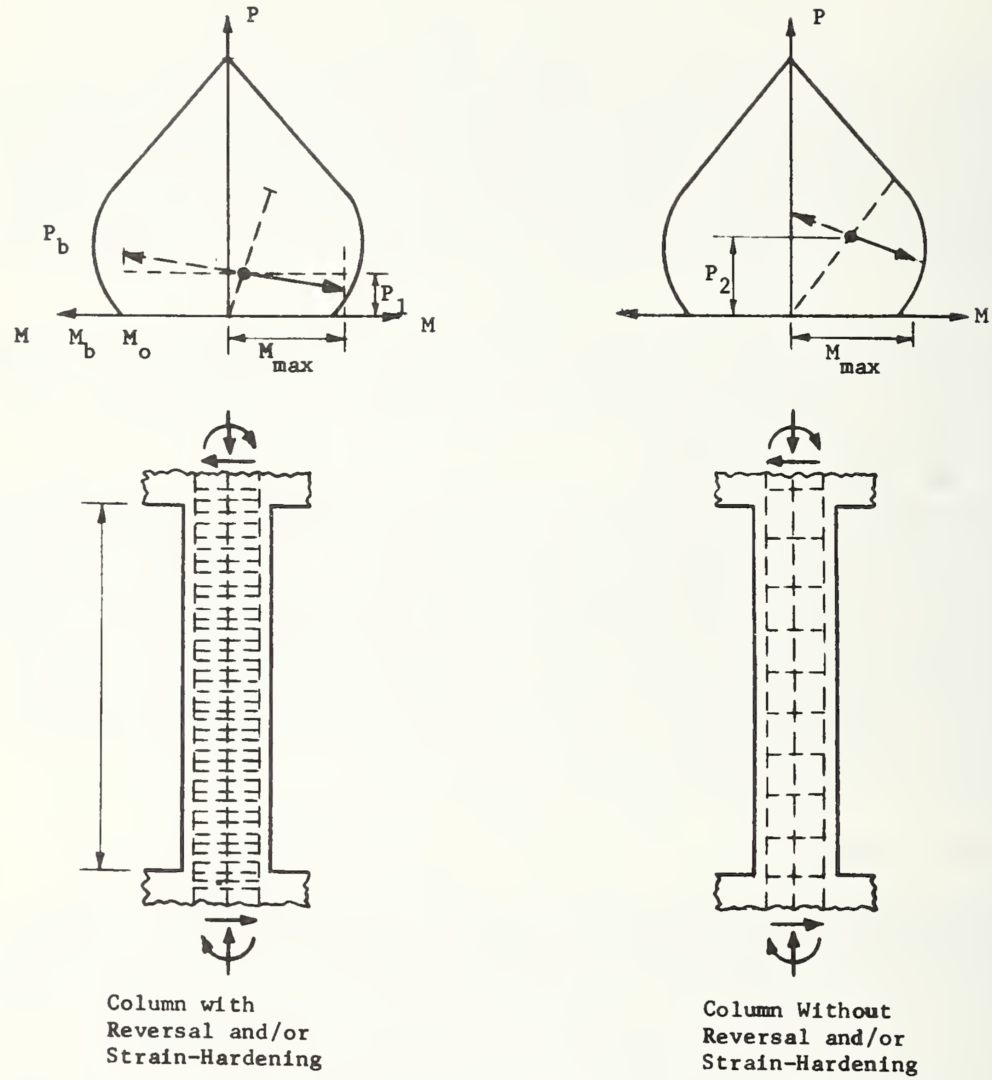


Fig. C.12 Effect of Reversal on Spacing of Hoops or Ties.

moment reversal occurs in one case, there is no reversal in the second case. It seems reasonable to expect that in the case with reversal and/or with strain hardening the spacing of ties should be selected more conservatively than in the case without reversal and/or strain hardening.

Resistance of reinforced concrete columns to repeated alternating shear and bending, simulating seismic loading in columns of moment resisting frames has been studied experimentally by T. Hisada and his associates [12].

Dimensions of specimens, arrangement of reinforcement, and test set-up are shown in Fig. C.13. Reduction in the resistance of the columns at a specified deflection with alternating loading cycles can be expressed by a "reduction factor" λ which for any given cycle is defined as follows:

$$\lambda_i = \frac{P_i}{P_o}$$

The reduction factors for the nine specimens are shown in Fig. C.14. It can be seen that for spirally reinforced columns--reduction factor reaches a limit value of about 0.70 at about 30 cycles. Most of the reduction in the resistance takes place in the first 10 cycles. The limit reduction, is not sensitive to the amount of transverse reinforcement. For columns with ties restraining each longitudinal bar by a 90° bend, reduction factor at 20 cycles varies from 0.62 to 0.84 and is sensitive to the amount of transverse reinforcement. It is interesting to note that such tied columns exhibited a lesser reduction in resistance than spirally reinforced ones. This may be due partly to a different arrangement of longitudinal bars, and partly to a more effective restraint of longitudinal bars provided by a 90° bend in the tie as compared to a circular hoop of a spiral.

Columns with rectangular hoops, which restrain corner bars only,performed poorly, exhibiting the greatest reduction in resistance. After 25 to 30 cycles the resistance was reduced to about 25-30%.

The results of these tests clearly indicate that properly designed tied columns perform quite well under combined axial compression, and reversals of shear and bending. Furthermore, these tests suggest that in some cases SEAOC requirements for lateral reinforcement, based on the concept of "confinement" may be too conservative.

Several special problems may arise in column design which usually do not arise with regard to beams. One of these is the skew loading (not in plane of symmetry). The plane of the lateral forces may vary in direction during the cyclic variations in load and it has been suggested that the capacity of columns may be sensitive to the variations in the loading plane as well as to the variations in the load magnitude.

Another problem is the variety of column shapes, e.g. L- or U- shaped column sections. This problem is related to the skew loading - and there is no experimental evidence on behavior of such irregularly shaped columns under cyclic reversals of shear and moment.

C.8 Connections in Moment Resistant Frames

One of the critical areas in the design of ductile moment-resistant frames is the joint between columns and beams, Fig. C.15. Under gravity loading the end moments in the adjacent beams tend to cancel each other, and only the moment unbalance, which must be resisted by the columns, induces shear in the joint. Under seismic conditions the end moments in the adjacent beams act in the same sense, and the columns must resist the sum of beam moments. The transformation of beam moments into column moments generates large shears in the joint, and repeated reversals of moments (shears) may

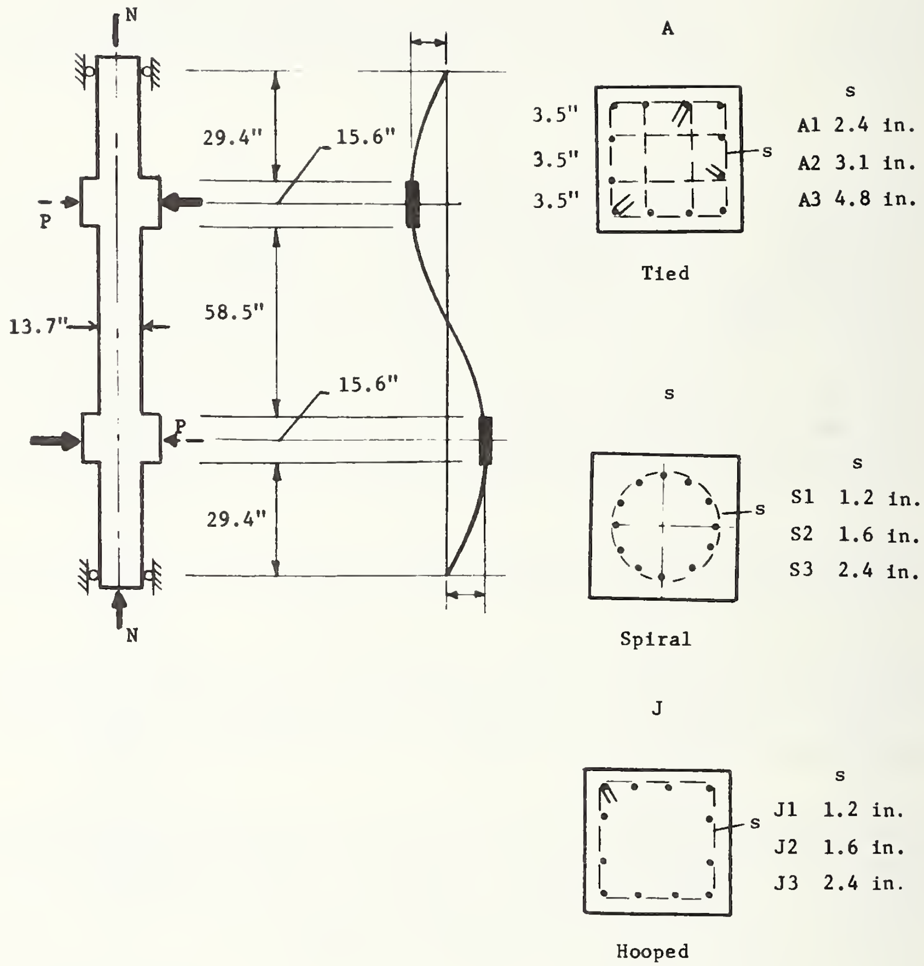
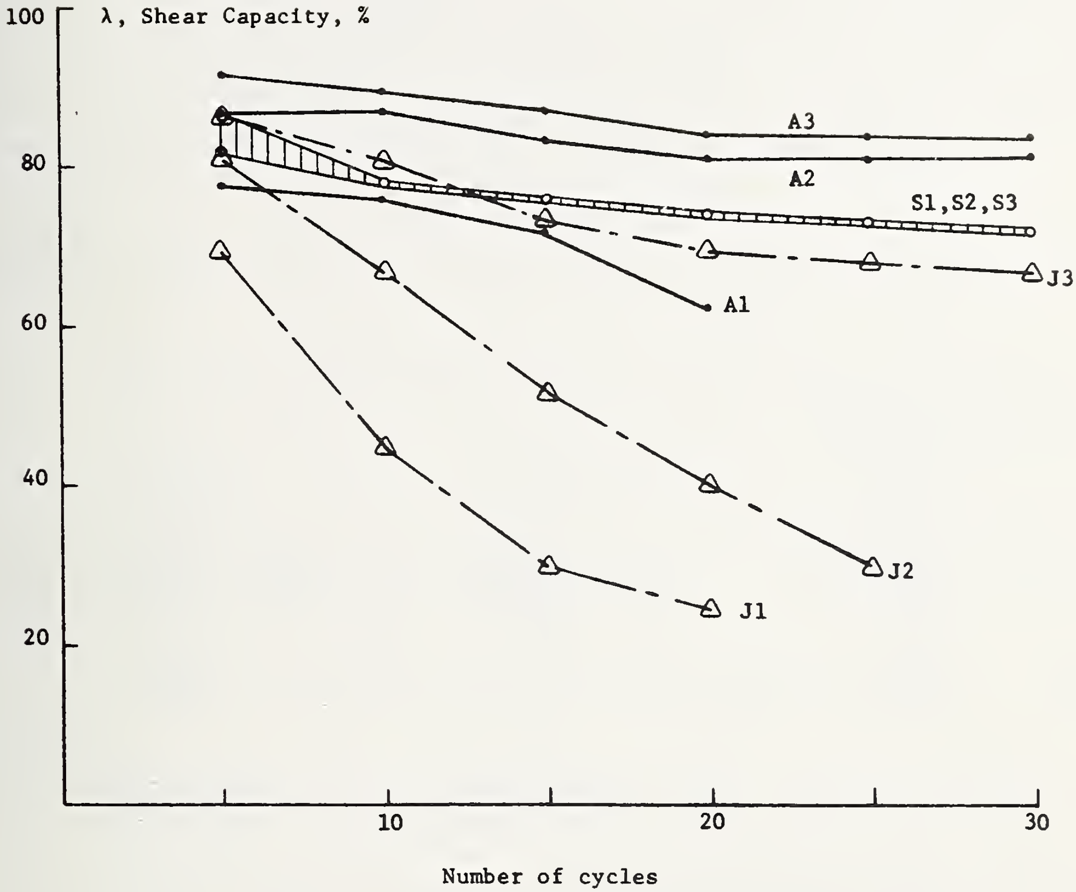
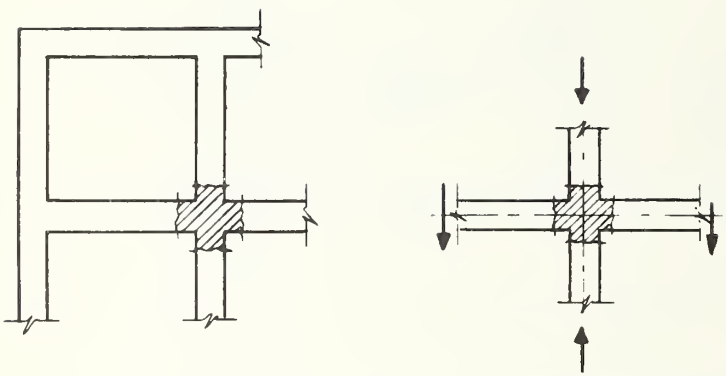


Fig. C.13 Dimensions of Specimens and Test Set Up (from Ref. 12)

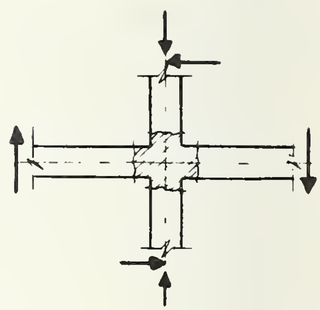


Note - Transverse reinforcement significantly below that required by SEAOC Recommendations.

Fig. C.14 Reduction in Shear Capacity at a Specified Deflection (from Ref. 12)



Gravity Loads



Lateral Loads

Fig. C.15 Gravity and Lateral Loads on a Joint produce extensive cracking and loss of bond between concrete and reinforcing steel.

The first loading cycle, Fig. C.16, produces cracks in beams and columns and yielding of reinforcing steel in these regions leads to fairly wide cracks. Upon reversal the first cracks close and cracking and yielding develops on the opposite face.

The stress distributions on the faces of the joint region and the freebody of the top bar are shown in Fig. C.17. It may be assumed that the tension stress is equal to yield strength f_y, and that the compression stress f_s' is less than f_y. Then while the concrete around the bar is still intact, bond stresses are developed along the full length. The distribution of the bond stresses depends partly on the state of stress of concrete surrounding the bar - where concrete is in biaxial compression, the resistance to slip is greater and the bond stress intensity is greater. Where concrete is in tension, resistance to slip is diminished and the bond is reduced.

With cycling, there is likely to be considerable deterioration of bond or slip resistance. Based on experimental results [26, 69] the following mechanism of bond deterioration under repeated loads was suggested:

1. The basic mechanism of bond deterioration is a failure in the concrete "boundary layer" adjacent to the steel-concrete interface. This failure occurs when the high local stresses reach critical values, and inelastic deformation and/or local fracture take place. Due to the nature-of the failure surface and to the interlocking of the deformed bar lugs with with surrounding concrete, shearing stresses below the critical value may be transmitted by friction and by wedging action. Some slippage of the steel relative to concrete takes place and is accompanied by inelastic deformation and local crushing at the steel-concrete interface.

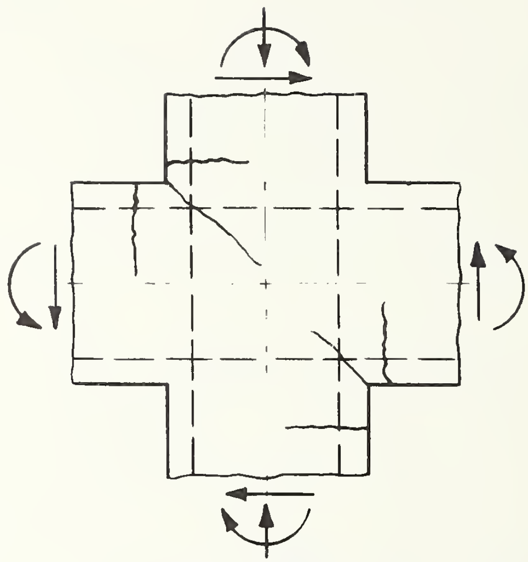
2. With repeated cycles of loading and unloading, provided the tension in the steel does not exceed the previous maximum, there may be some further disruption in the boundary layer.

3. The effectiveness of bond between concrete and steel depends on the given level of tension stress in the steel and on the magnitude of the previous maximum tension. The greater the magnitude of the previous maximum tension, the greater is the disruption of the boundary layer, and the lesser is the effectiveness of the bond at lower levels of tension. When the previous maximum tension is considerably greater than the working stress level, even a small number of cycles of this high tension reduces the bond effectiveness at the working stress level.

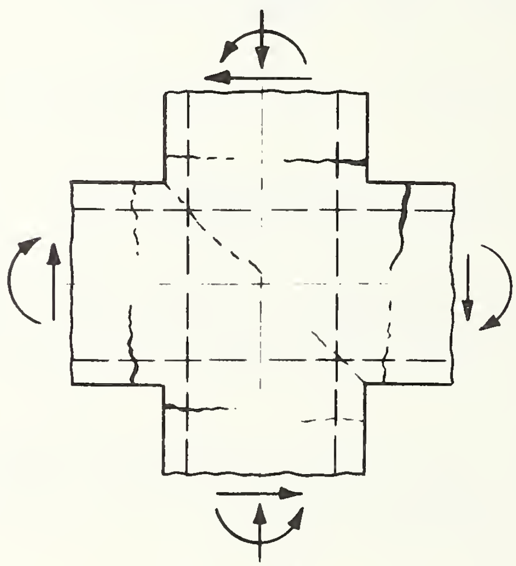
With repeated cycles of reversals in the inelastic range, longitudinal splitting takes place in the joint further reducing bond effectiveness and allowing considerable slip (and hence rotation) in the joint. Finally, resistance to slip is generated only in the compression zone, where "bond" is effected only by friction and wedging.

After several extreme strain reversals the joint can act only as a mechanism with flexible joints. This behavior can greatly increase the lateral displacements, and thus increase the lateral ductility. On the other hand local damage will be greatly increased and possibility of frame and member instability enhanced. Furthermore, the joint moment resistance will be decreased, and the collapse mechanism will be altered, reducing the load carrying capacity of the frame as a whole.

The deterioration of the joint effectiveness described above is based on test results with planar frame subassemblies. It has been shown that for



First Loading



Reverse Loading

Fig. C.16 Cracking on First Loading and Reversal

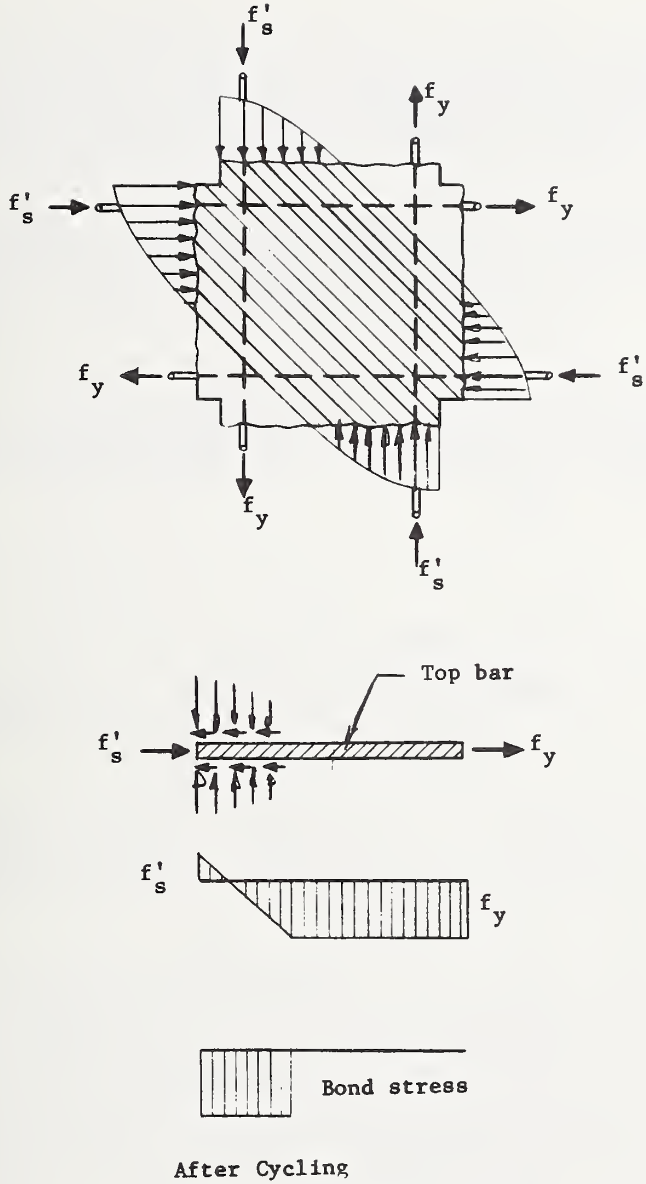


Fig. C.17 Stresses on Joint Faces and Bond three-dimensional frame joints - presence of transverse beam stubs considerably improves the performance of the joint, [56]. Presence of a floor slab should further improve joint behavior, unless the transverse beam elements and the slab are also subjected to reversals of inelastic deformations (loading).

C.9 Shear Walls

The ability of reinforced concrete walls to resist large shear forces with small deformations makes them particularly effective structural elements in earthquake resistant design. It must be recognized however that their rigidity while limiting deformation (drift), also changes the response of the structure: it reduces the period, increases the shear forces generated, and increases the number of load reversals. Also, unless the walls are properly designed, they may fail in a rather brittle mode - greatly reducing their potential energy dissipation capacity. Such failures are also difficult to repair fully, i.e., to restore the structural integrity which would ensure their effectiveness in a subsequent earthquake.

A number of difficulties arise in developing proper design criteria for reinforced concrete walls. In this review the discussion will be limited to shear walls, which can be characterized by the fact that the wall panels act primarily as shear elements in a frame which resists bending moments primarily by axial forces. This implies that shear walls constitute panels (in-fill) with frame members on all four edges.

The desired behavior of the system is to have a ductile frame sufficiently strong and properly reinforced so as to restrain excessive distortion of the wall after initial cracking, and to produce a large number of small, well distributed, cracks in the wall panel. In this way maximum energy dissipation would be achieved, and at the same time overall lateral deformation would be relatively small, and the frame would undergo minimum damage. The optimum shear wall design is to produce a combination of a ductile frame with a ductile wall panel. This effect has been achieved in some cases by such devices as a "slit shear wall" designed by Muto [79].

It has been suggested that shear walls will crack initially when they reach a shearing strain of 0.2 to 0.3 x 10^{-3} radians, and usually the shear cracks originate at a stress substantially higher than that for reinforced concrete beams (two to three times as great).

The shear capacity of a reinforced framed wall depends on the amount of reinforcement and on the frame contribution. Because the ultimate objective of a shear wall is to allow formation of a large number of diagonal cracks, it may be appropriate, from the strength point of view, to disregard the contribution of the concrete to shear strength of the wall panel. The shear capacity would be defined by the amount of reinforcement and by the shear capacity of the frame surrounding the panel. This criterion might provide a basis for frame design - frame should carry the excess shear (design total shear less that carried by wall reinforcement). Of course other requirements for proportioning frame members (compression, flexure, ties for preventing local buckling, etc.) must be taken into account.

The shear wall system described above must be differentiated from a "deep beam wall", which does not have an adequate frame to resist bending moments and to constrain the wall in such a way that it would generate a large number of small cracks. In such "beam wall" systems, often one major crack forms suddenly and greatly reduces the capacity of the wall to resist lateral forces. A "beam wall" or "frameless wall" must be designed to resist the lateral shear without cracking.

In design of shear walls the following questions must be answered: (1)

what is the load (or stress) that causes first cracking in the wall, (2) what is the capacity of the wall (acting with the frame) to resist shear, (3) what are the proper criteria for design of a suitable ductile frame to act with the shear wall, (4) what are the proper criteria for detailing wall and frame reinforcement and dimensions, and (5) what are the proper provisions for design of shear walls with openings.

A major problem in design of shear walls is the effect of openings. The number and size of openings must not be too great; otherwise the behavior of the wall will not be realized, for it can become merely a frame with some deep spandrels and short columns. Assuming that the openings are relatively small (with respect to the size of the wall panel) the usual criteria in the U.S. Codes is to provide a specified amount of corner and edge reinforcing bars.

It is suggested here that a greatly superior system of reinforcing around wall opening would be obtained by creating an effective frame of reinforcing steel around the opening as shown in Fig. C.18. The basis of this design would be to strengthen the subpanels A, B, and C surrounding the opening in such a way that the wall with the opening would have the same rigidity and capacity as a wall without the opening.

At the present time there is no sufficient information on which rational design criteria could be based. Therefore, additional studies - both analytical and experimental, are required. Particularly important would be studies of interaction between frame and panel elements in shear walls, failure mechanisms, capacity, ductility, and energy dissipation - particularly under cyclic reversed loadings.

D. Prestressed Concrete

D.1 Prestressed and Partially Prestressed Elements

Prestressed concrete structures in localities where severe earthquakes and winds are anticipated have been used only to a limited extent, although recently use of such structures has been increasing. Published experimental studies of behavior of prestressed concrete elements under cyclic reversed loadings also have been limited. In a review of these studies Bertero [90] observed that prestressed concrete framed structures can resist moderate earthquakes without structural damage, and can withstand severe earthquakes - although in this case, structural damage may occur with consequent difficulty of repair restoring structural integrity of a fully prestressed system.

Some of the difficulties experienced in prestressed concrete structures in seismic regions is related to connection details of precast elements. In many cases the prestressed elements themselves behaved well. Test of such elements under monotonic loading revealed amply ductile behavior (large energy absorption), and largely elastic recovery (little energy dissipation, low damping). Partially prestressed concrete members (containing some conventional reinforcing steel) have greater capability for energy dissipation.

In the absence of sufficient data, no recommendations can be made as to revised design procedures of prestressed concrete systems. In carrying out such designs however, considerable attention must be devoted to consequences of local crushing or cracking in critical regions of the elements and connections, caused by inelastic cycling caused by extreme loads.

Additional research is needed to determine behavior of realistic models (subassemblies) of prestressed and partially prestressed concrete systems under cyclic reversal of loads in the inelastic range.

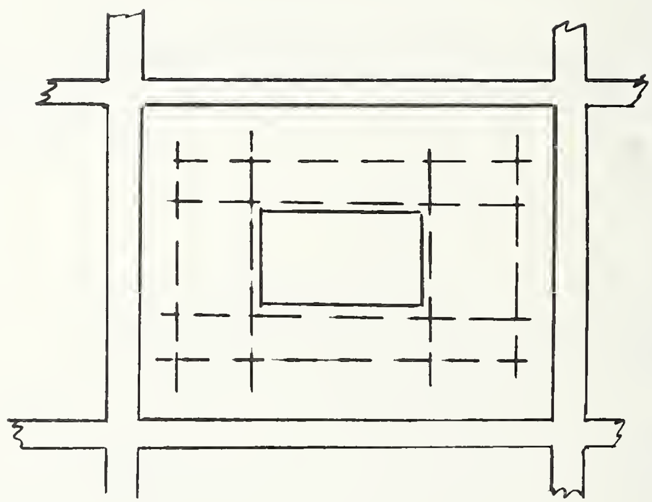


Fig. C.18 Reinforcement of Wall Openings

E. Structural Steel

E.1 General

The viewpoint that steel structures are less vulnerable to severe earthquakes than masonry or concrete structures because of the natural ductility of the material has some merit. However, observations of damage in severe earthquakes can not fully support this point of view, first - because some steel structures have undergone damage in severe earthquakes, and second - because many severe earthquakes occurred in areas where only a few steel structures existed, thus biasing (statistically) occurrence of damage in structures of different materials. At best, considering the natural ductility of steel, one can conclude that steel structures tolerate design or construction inadequacies more than other types of structures.

Basically, the requirements in earthquake resistant design of steel structures is the same as for all structures - i.e. ductility and energy dissipation capacity without excessive cumulative damage. Any conditions that tend to counter ductility, energy dissipation and endurance of load reversals without damage must be closely examined. For steel structures these are: frame deformation (P-Δ) effects, local buckling, and problems associated with connections (welded or bolted).

While modern design criteria for concrete structures are primarily based on strength and overloads, current design criteria for steel structures are primarily based on allowable stresses and service (working) loads.

Section 1.5.6 of AISC Specification for the Design of Structural Steel for Buildings states that under wind and seismic conditions allowable stresses may be increased one-third above the values under normal service conditions. Depending on the level of (D+L) load stresses, this criterion may lead to a wide range in the reserve capacity for overloads. The inconsistencies in reserve capacities are best illustrated by simple examples suggested by Professor T. Y. Lin (private communication).

Consider a ductile steel element for which the strength may be characterized by plastic capacity P_y (also yield capacity). The allowable load under normal service conditions is $P_a = 0.6\ P_y$, and under combined (D+L+E) conditions it is $P_a' = 1.33\ P_a = 0.8\ P_y$. Should the design earthquake intensity be underestimated, it is expected that the reserve capacity $P_r = (P_y - 0.8\ P_y) = 0.2\ P_y$ will accommodate the increase in earthquake intensity λE. The factor λ is the ratio of reserve capacity P_r to earthquake effect P_E, i.e. $\lambda = (P_r/P_E)$. This factor will vary with the levels of (D+L) and (E) stresses.

If (D+L) condition produces a design load P_1 equal to $0.6\ P_y$, and if design earthquake effect due to E is $P_{E1} = (P_a'-P_1) = 0.2\ P_y$ (gravity load controls design, earthquake effect is small), then $\lambda = (P_r/P_E) = (0.2\ P_y/0.2\ P_y) = 1$, or a 100% increase in earthquake effect can be accomodated. This condition may be typical of girders, particularly in upper stories, where earthquake effect is small compared to gravity. On the other hand, if (D+L) condition produces a design load $P_2 = 0.2\ P_y$, and if design earthquake effect $P_{E2} = (P_a' - P_2) = 0.6\ P_y$ (i.e. gravity effect is small and earthquake effect controls the design), then $\lambda = (P_r/P_E) = (0.2\ P_y) = 0.33$, or only a 33% increase in earthquake effect can be accomodated. This condition may be typical of columns in a ductile moment resistant frame, particularly in lower stories.

It can be seen from the above that a fixed increase in allowable stress (say 1/3) can not give a uniform λ reserve capacity for earthquake loadings for all elements in the structure.

Another inconsistency appears to exist in the AISC provisions, which stems from possible differences in the interpretation of "low cycle fatigue" effects. AISC Specification, Section 1.7.1, states that "the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design." On the other hand under provisions of Sec. 1.7.2 for some types of welded connections, e.g. fillet welds and transverse partial penetration groove welds, subjected to a "large range of stresses, but for cases of repeated loading of less than 20,000 cycles" (see AISC Commentary on Sec. 1.7) the values permitted are considerably below those based on the 1/3 increase in allowable stresses specified in Sec. 1.5.6. The discrepancy between Sec. 1.5.6 and Sec. 1.7.2 allowable stresses is illustrated in Table E.1.

Table E.1 ALLOWABLE STRESSES IN FILLET AND PARTIAL PENETRATION TRANSVERSE WELDS

Stresses in Ksi			
Service Load (Monotonic) Sec. 1.5.3	1.33 x Service Load Sec. 1.5.6	Low-Cycle Fatigue Sec. 1.7.2 R = 0*	R = -1*
18	24	22.5 (1) 25.0 (2)	11.25 (1) 12.5 (2)
21	28	22.5 (1) 25.0 (2)	11.25 (1) 12.5 (2)

*R is the stress reversal ratio.

(1) Partial penetration groove welds.

(2) Fillet Welds.

In the light of this inconsistency and because of possible distress due to low cycle fatigue in elements subjected to stress reversals into inelastic range, and also taking into account uncertainties regarding the intensities of extreme loading conditions, the 33% increase in allowable values seems unconservative for the local critical sections were full stress reversals takes place under earthquake loading.

E.2 Beam-Column Behavior

In the usual structural analysis and design calculations, the deflections are assumed to be so small that they have a negligible effect on forces and moments in the various structural elements. Thus, the so-called "first-order theory" is developed neglecting the deformations of the structure. In some cases, particularly, in slender members involving compression combined with bending, the effects of deflection may become non-linear with load increase, and must be taken into account. This is the basis for the so-called "second-order theory". The effect of the center deflection Δ in a beam subjected to end moments M_o and axial compression P can be seen from the expression for center bending moment M:

$$M = M_o + P\Delta$$

The increase in bending moment at the center, and the corresponding increase in moments along the beam-column, cause an increase in end rotation θ, so that a moment-rotation curve M_0 vs θ appears as shown in Fig. E.1. The hysteresis loop observed in cyclic repeated loading is also shown in Fig. E.1. The importance of the so-called $P\Delta$ effect depends on the type of analysis employed: if second-order theory is used and values of P, M and Δ are properly calculated, then the shape of the load-deformation curve (or hysteresis loop) will be based on the basic properties of the section (or element) - developing full plastic moment capacity. However, when first-order theory is used to define $M = M_0$, then an apparent reduction in capacity and in energy dissipation is obtained.

The P-Δ effect may be important from the point of view of overall frame stability and of design for appropriate maximum moment. However, it shall not be treated here in detail as it is not a characteristic of the structural element, but rather a question of proper evaluation of forces and moments in a structural system (selection of 1st order or 2nd order analysis).

E.3 Local Buckling

The principal effect of local buckling during cyclic loading (within limited number of cycles examined) is to reduce stiffness; the reduction in load is small. Tests on cantilever beams [92] have shown that after a small number of reversals with large inelastic deformations the overall stiffness of the beam may be reduced, primarily due to local buckling. For example a cantilever steel beam (W18 50), Fig. E.2, which initially develops a deflection of about 0.25 in. at the free end when subjected to a concentrated load of 25 kips, after about 12 cycles of reversed loading developed a deflection of about 0.5 in. under the same load of 25 kips. This reduction in stiffness occurred gradually, during 7-8 cycles of loading in the range of ±50 to 70 kips. Even though reduction of stiffness is significant, the capacity of the beam was not affected. Examination of hystersis loops for the tests reported to date indicates some progressive variation in the stiffness of the members.

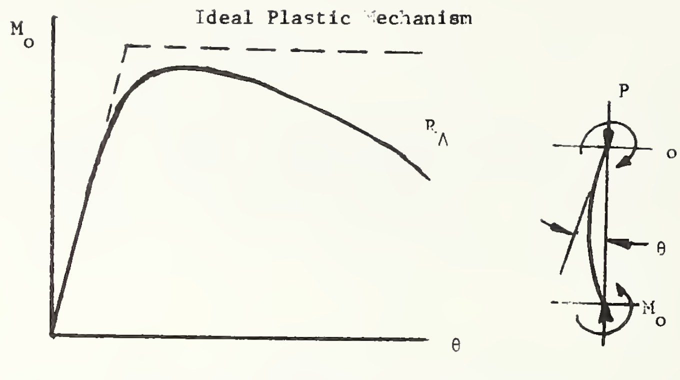
Also, tests have shown that at high inelastic strains even when no local buckling is exhibited during the initial loading - after a small number of inelastic reversals local buckling may develop. Such local buckling tends to develop large secondary inelastic deformations and with sufficiently large number of cycles it may lead to low-cycle fatigue fractures.

For elements subjected to reversal of plastic deformation width-thickness ratios for steel shapes at the critical regions for load reversal may have to be chosen more conservatively than for static loading. Damage observed in the Cordova Building during the Alaska earthquake in 1964 supported these findings.

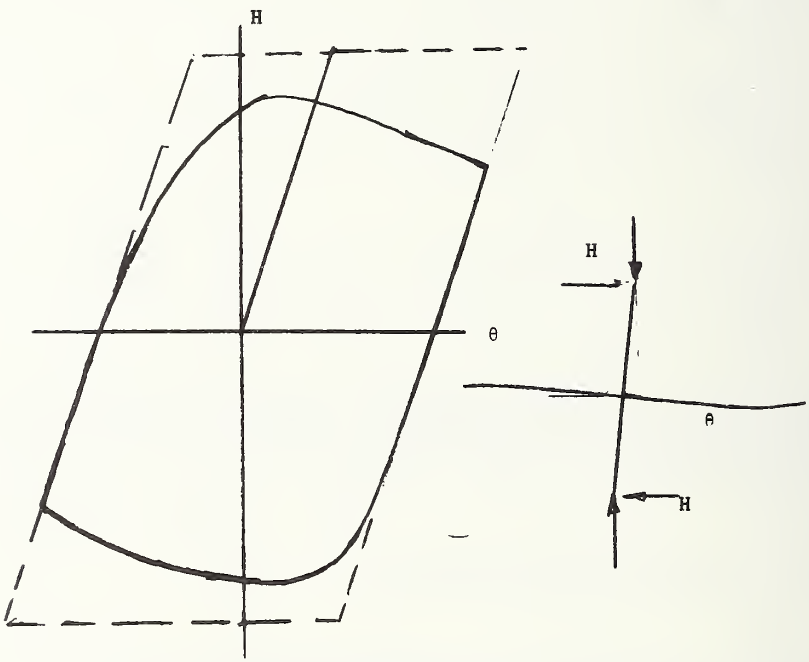
Therefore, a 25% reduction in width-thickness ratio specified in AISC Sec. 2.7 is recommended under conditions of full reversal of plastic deformations. This leads to the following:

Table E.2 REDUCED WIDTH-THICKNESS RATIOS (b_f/t_f) - CYCLIC REVERSED LOADING

Yield Strength F_y, ksi	Sec. 2.7 Plastic design-monotonic loading	Proposed Earthquake design-cyclic load reversal
36	17	13
50	14	10.5
60	12.6	9.5



Moment Rotation Curve for Beam Columns
(After Ref. 93)



Load vs Joint Rotation Hysteresis Hoop
(After Ref. 9)

Fig. E.1 Beam-Column Effect

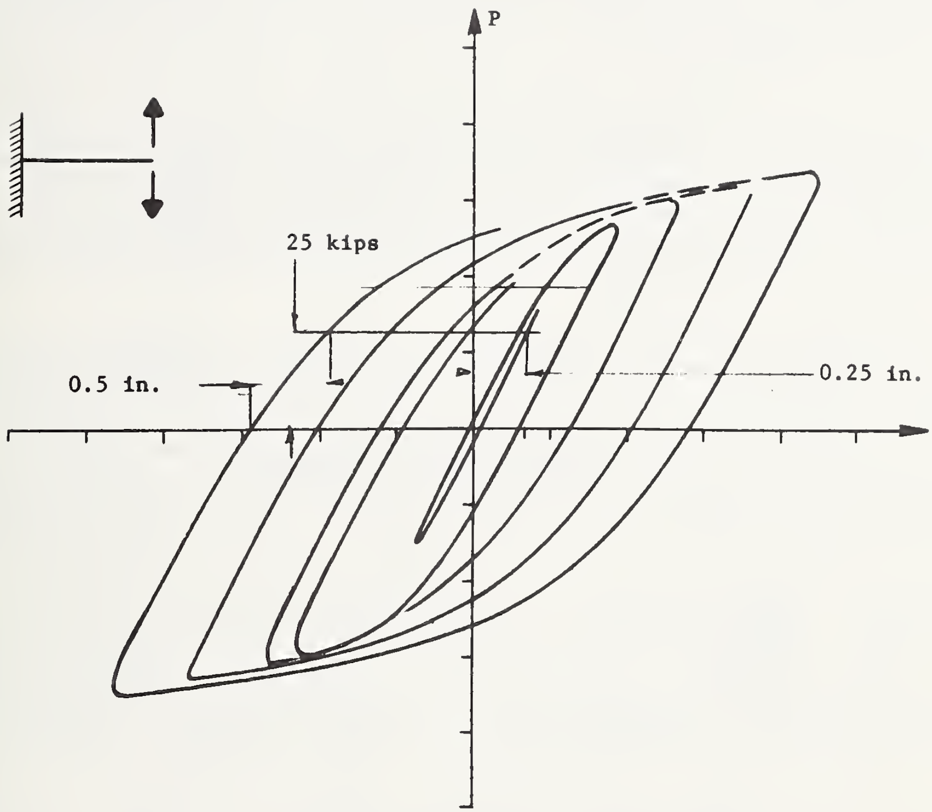


Fig. E. 2 Load Deflection Hysteresis Loops

E.4 Structural Steel Shapes and Connections

For tension-compression cycling, with relatively small plastic deformations (without strain hardening) and with "compact" shapes which do not exhibit local buckling at a low number of reversed cycles the steel elements develop stable hysteresis loops similar to those shown in Fig. B.4. For bending, within the conditions imposed above (no strain hardening and no local buckling), the same stable and ductile behavior is observed.

A limited number of tests on structural steel connections under cyclic reversed loadings have been reported [8, 10]. These include tests on all-bolted connections, all-welded connections, and on flange-welded and web-bolted connections, beam flanges framed into column flange--producing column bending in the strong direction. A few tests were conducted with beams framing into the column web.

Test results indicated that from the standpoint of cyclic behavior (energy dissipation and endurance) the welded connections performed best. The hysteresis curves remained stable, and even with cyclic recurrence of local buckling and some reduction in stiffness--the capacity of the connection was maintained. Generally failure occurred by cracking the flange near the weld after a number of cycles of buckling. Connections using a moment flange plate did not differ appreciably from the connections with flanges welded directly to the column.

While properly designed and welded connections have demonstrated excellent performance under reversed cyclic loading, defects either in design or in welding quality can significantly reduce the capacity of connections subjected to cyclic reversed loads. To achieve consistent high performance of welded connections requires proper design and execution (welding sequence, etc.) of all welds and limitation of plate thicknesses in welded joints in accordance with weldability of different steels. The potential problem of earthquake-induced stress cracking in welded connections, particularly in elements built-up using thick plates, requires further investigation.

Hysteresis loops for moment resistant beam-to-column connections made with A325 bolts show characteristic effects of slippage, Fig. E.3. It can be seen that some slippage was observed in the first half-cycle, and that with repeated excursions into inelastic range - slipping was occurring at a lower load. Following the slipping stage in each cycle the bolts came into bearing contact with the holes (punched 1/16 in. oversize), and the overall deformation at a maximum load was approximately the same as that in the first cycle.

However cyclic loading and recurring slipping resulted in decreasing friction and reduced amount of energy dissipation.

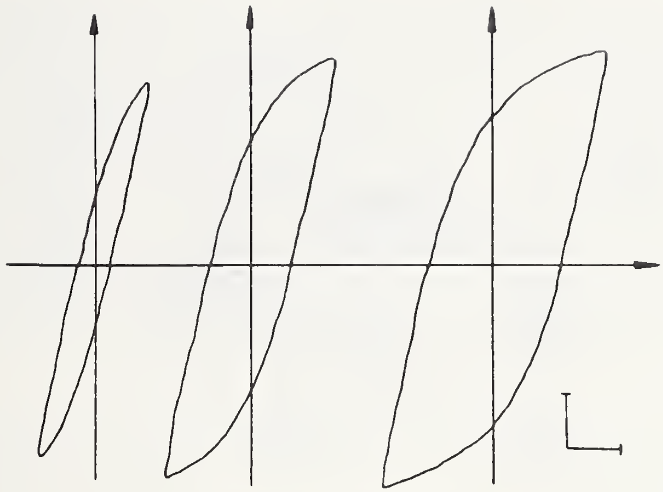
Excessive distortion and local buckling in the panel region of a frame joint, Fig. E.4, can substantially increase the rotations at the joint and translational displacements of joints [9, 14]. To prevent excessive shear distortion, doubler plates or diagonal stiffeners may be required.

Until substantiated or proved unwarranted by appropriate tests, the shear stress caused by cyclic inelastic reversals of (D+L+E) loads in the panel region surrounded by stiffeners and flanges on all four edges shall not exceed 0.3 F_y (a 25% reduction in the normally allowed shear stress).

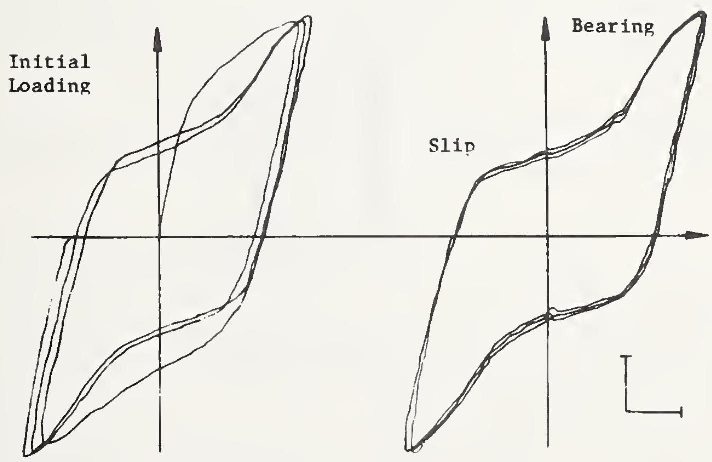
F. Concrete-Steel Composite

F.1 Encased Construction - Beams and Columns

Encased concrete-steel construction is defined as one in which the steel

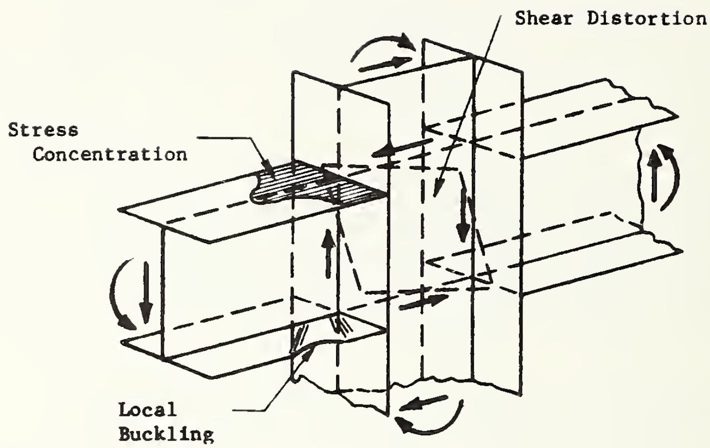


Welded Connections-Stable Loops

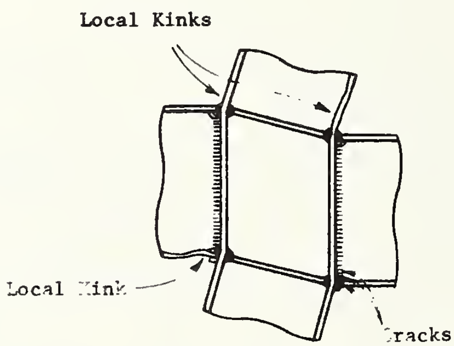


Bolted Connections

Fig. E.3 Hysteresis Loop-Welded and Bolted Connections



(a) Lateral Loading



(b) Panel Distortion

Fig. E.4 Problems Introduced at Beam-to-Column Connection.

element is totally surrounded by concrete at least 2 inches thick and in which adequate steel mesh (or cage) surrounds the steel beam. In accordance with current specifications, the behavior of such encased beams is considered to be fully composite, relying on the natural bond between steel and concrete. The present AISC Code provides for this type of construction for beams but not for columns. The design of such beams is based on allowable stress criteria, with some special provisions for the case when the steel beam acts independently of encasement.

The ACI Code specifically excludes the encased steel flexural members from its scope, stating that such designs are covered by AISC. On the other hand, in the absence of AISC provisions for design of encased columns, the ACI Code contains provisions for composite compression members. Actually, it provides for composite action under combined compression and bending of columns - using ultimate strength design criteria. Also some provisions for the special case of concrete-filled steel tubes is included in the ACI Code.

The problem of composite action of beam-column subassembly, i.e. the behavior of a joint in an encased construction system is not covered by either code and so handicaps the designer in proper detailing of a joint. Furthermore, the inconsistency of AISC working stress design provisions for beams vs. the ACI ultimate strength design provision for columns further handicaps practical use of this type of construction.

The advantages claimed for the encased steel construction are: (1) high fire-resistance rating, as the frame requires no special fire proofing; (2) efficient utilization of concrete in compression, where its use is particularly economical; (3) increased stiffness of the composite system; (4) confinement of structural steel by surrounding concrete and reinforcement, preventing local buckling which controls width-thickness ratios which can be used; and (5) large reserve shear and flexure strength under cyclic load reversals.

Studies of encased steel construction under cyclic reversed loading are very few, and most of these have been carried out in Japan, where this type of construction has been widely used. The types of some test specimens used in these studies are shown in Fig. F.1.

Although the effectiveness and the economy of encased steel construction vs. other alternative systems is yet to be evaluated, it can not be done in the absence of consistent and reasonably complete design criteria.

F.2 Composite Concrete-Steel Construction

Composite construction defines a structural system in which interaction of a concrete slab with a steel beam is accomplished by means of mechanical devices (shear connectors). Usually, concrete slab becomes the compression flange of the composite beam, and the steel section resists primarily the tensile stresses. This basic concept clearly provides little or no advantage when full reversal of bending moments controls the design. In the absence of reversal - some advantage may be gained from composite construction, provided the shear transfer mechanism can survive the cyclic inelastic loading.

F.3 Composite Concrete - Concrete Construction

This form of construction usually involves use of precast concrete elements acting in an integral manner with cast-in-place concrete. With appropriately designed reinforcement at the interface, this form of construction should perform as well as a fully cast-in-place system. The current ACI Code (Chapter 17) covers the design requirements for concrete composite flexural members. Further development of this type of construction, with

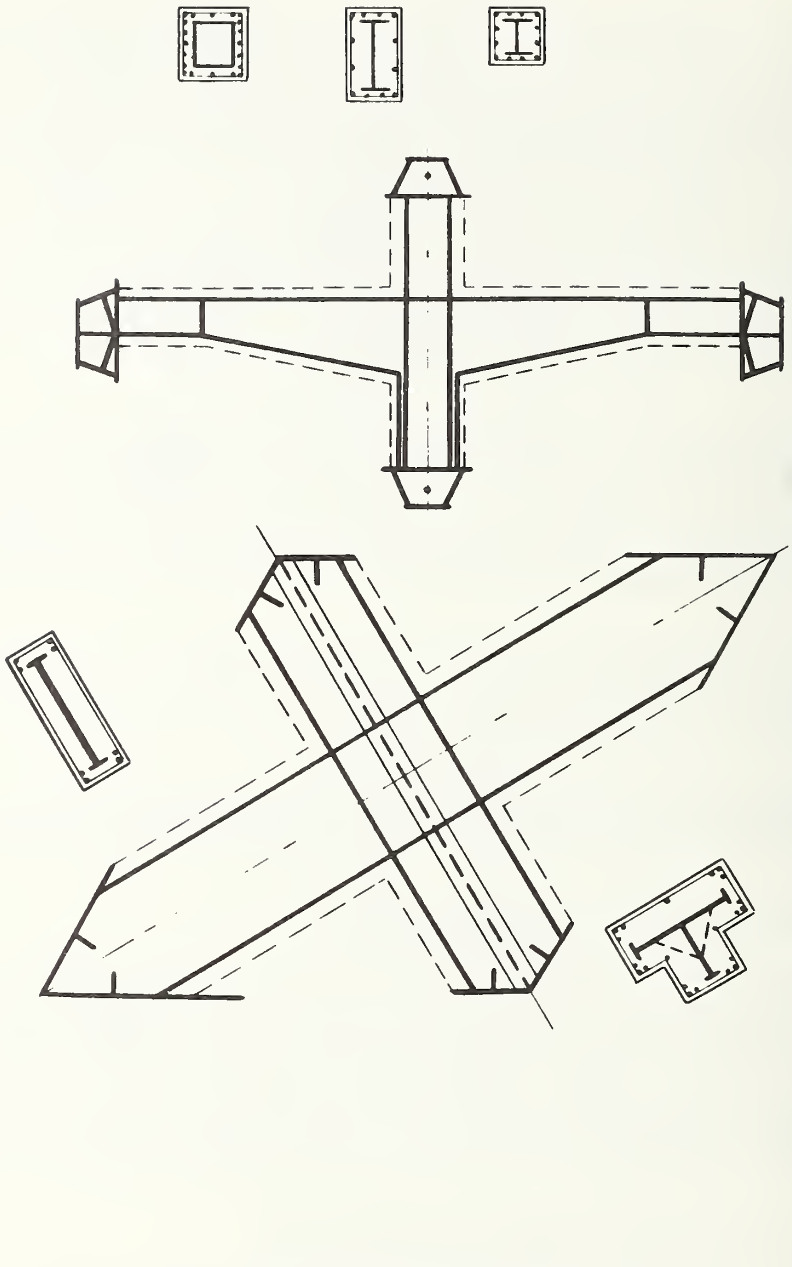


Fig. F.1 Concrete Encased Structural Steel Subassemblies.

proper attention to composite action of the frame-slab-wall systems is of particular importance.

G. Masonry

G.1 Reinforced Masonry

Unreinforced masonry buildings have demonstrated poor performance in numerous earthquakes and in other exposures to severe loadings. However, performance of reinforced masonry is considerably better. Reinforced masonry, from a basic point of view, is only a special case of reinforced concrete: it deals with certain size and shape of precast elements, joined together with steel reinforcement and cast-in-place concrete infill framework and interface.

The principal questions regarding proper design of reinforced masonry systems are: where and how much reinforcement must be provided, and what are the required cast-in-place concrete infill and interface joints. Some recent tests indicated that under essentially monotonic loading reinforced concrete masonry may have adequate strength and ductility. Its performance under cyclic reversal of loading still requires further study.

H. Conclusions and Recommendations

H.1 General Remarks

The review summarizes current knowledge regarding behavior of structural elements under seismic and similar extreme conditions, suggests some improvements in design criteria and discusses briefly some innovations for better performance of structural elements under repeated reversed loadings in the inelastic range.

Emphasis was placed on factors to be considered in design and on the evaluation of the relative importance of these factors.

The scope of the discussion was limited by space limitations. Therefore in addition to references cited directly in the text, additional sources of data are given in the bibliography.

H.2 Design

Review of the commonly used basic structural design codes (AISC, ACI, and SEAOC) reveals varying degrees of specific provisions for seismic or other extreme loadings. AISC sets very few guidelines for design under such conditions. ACI included provisions for seismic design in a special Appendix for the first time in 1971. The provisions of this Appendix are based largely on two sources - the Blume, Newmark, Corning [13] and the SEAOC (1968 version). In addition, results of studies based on investigations of damage in recent catastrophic earthquakes (Skopje-1963, Anchorage-1964, and Caracas-1967) were used in formulating the provisions of the Appendix. A number of the recommendations in the ACI Appendix and the SEAOC recommended requirements are similar, although some differences exist, particularly between the ACI 1971 and SEAOC 1971 versions. In the absence of other guidelines SEAOC has provided the leadership in developing special provisions for seismic design, and their recommendations provided the basis for UBC seismic design criteria. Unquestionably the ACI Appendix is a major step forward in the recognition of the special problems associated with earthquake-resistant design of concrete structures. Use of these design criteria, particularly the 1971 SEAOC recommendations, should result in structures which can withstand severe loadings without collapse.

Although there has been only limited damage (with few remarkable exceptions) in recently designed major structures - one can not rely on this as a measure of design adequacy, because large urban centers have not experienced extreme loading conditions. Some suggestions for design modifications which would further improve performance of structures under extreme conditions are outlined in the preceding sections and are summarized below.

Gaps in the knowledge of the extreme loadings and of the structural response to some of these preclude general agreement on the appropriate criteria for design which would minimize damage under these conditions. Because some of the design recommendations are based on extrapolation of "evidence" and on subjective conclusions (judgment), they are often unacceptable to large professional groups responsible for design specifications and codes. The dilemma most often faced by such groups is the extent to which additional cost of extra caution ("extra" - in a sense that it may not be necessary) in design is acceptable in the light of risk of severe damage (loss of life and property).

In the absence of adequate design criteria, the designer must rely heavily on his understanding of structural behavior under the extreme loading conditions and must accept the responsibility for subjective decision based on this special knowledge.

In making the design decisions, the individual designer must take full cognizance of the following: dynamic nature of the extreme loading conditions, inelastic response of the structure, behavior of the structure with some zones of limited damage (cracking, local buckling, crushing, permanent distortion, etc.), means of providing sufficient ductility and energy dissipation (i.e. eliminating conditions which may result in brittle behavior of an element), means of preventing imperfections in the structure (quality control in construction), and "repairability" of certain types of damage.

Some of the steps the designer can take which would help him in arriving at a judicious decision are the following:

(a) perform a reasonably realistic analysis of the structure, which would indicate the range of load variation (maximum and minimum loads),

(b) investigate local ductility of critical regions, taking into account possibility of loading reversals if so indicated by the analysis; distinction should be made as to full, partial, or no reversal conditions,

(c) avoid forms of construction and/or provide special reinforcement to prevent brittle modes of failure,

(d) prescribe control tests on materials to be used in construction which would ensure the desired material properties and quality of workmanship.

<u>Reinforced Concrete</u>

<u>Shear</u>. To prevent brittle shear failures the element must be designed for the maximum probable shear force. In ductile moment resistant frame elements this shear must not be less than that required for developing a "mechanism" action. In addition where the load is repeated a number of cycles, the shear capacity of concrete shall be reduced as follows:

(a) full reversal - concrete shear resistance is reduced to zero;

(b) no reversal - concrete shear resistance is reduced to 1/2 that for static loading.

Local Buckling. Sufficient ties should be used to prevent local buckling of each steel bar subjected to compression stress. Where large inelastic strains are anticipated (in the strain-hardening range) or where full stress reversals in the inelastic range are anticipated, the spacing of the ties will be of the order of 7D to 5D. Hoops and ties must be suitably hooked or prevented from opening after outer cover has spalled. They also must be tied closely to the longitudinal bars which are braced by them.

Crushing Strain. Concrete should be capable of maintaining a compressive stress of not less than 0.80 x compressive strength (f'_c) at a strain of 1.5 ε_o corresponds to stress f'_c. This requirement on the ability of concrete to develop a descending branch of the stress-strain diagram is minimal for ductile behavior of reinforced concrete. Laboratory tests should be performed to establish fulfillment of this requirement.

Reinforcing Steel. While generally ASTM A615 reinforcing steel may be adequate to provide the necessary ductility - special requirements on yielding prior to strain hardening, and on strain hardening characteristics are desirable. It must be emphasized that "over-strength" of steel reinforcement (due to higher yield or strain-hardening at strain below 0.003) may allow development of moments in excess of calculated yield or plastic moments, and thus increase shear loads on the element. This may change the failure mode from ductile to brittle. Therefore, in specifying reinforcing steel, it is advisable to set limits on both minimum and maximum yield strength values.

Structural Steel Elements:

Allowable Stresses. Because of possible distress due to low-cycle fatigue in elements subjected to stress reversals the allowable stresses should be well below the yield values. In the light of the uncertainties regarding the intensities of extreme loading conditions (including earthquakes), the 33% increase in allowable values seems unconservative for the local critical section where full stress reversal takes place under earthquake loading. The present AISC Specifications are ambiguous on this point. In Sec. 1.5.6 fatigue conditions (including low-cycle fatigue) are excluded from the 33% increase in allowable value, but stresses produced by wind or seismic loading are allowed such an increase.

Until substantiated or proved unwarranted by appropriate tests, the 33% increase in allowable values should be clearly disallowed in all cases where full stress reversal may occur in the inelastic range.

Width-thickness Ratios. In critical regions where significant reversals of stresses can occur, special provisions must be made to minimize local buckling and low-cycle fatigue fractures. The width/thickness ratio used in compact sections are adequate to prevent (or minimize) local buckling at plastic hinge locations under monotonic loading. Further reductions in width-thickness ratios are indicated in regions where full reversal of stress takes place under extreme conditions.

Until substantiated or proved unwarranted by appropriate tests, a 25% reduction in width-thickness ratios specified in Sec. 2.7 of AISC Specification is suggested where full stress reversal may occur in the inelastic range.

Welded Connections. Properly designed and welded connections have demonstrated excellent performance under reversed cyclic loading. However, defects either in design or in welding quality can significantly reduce the capacity of connections subjected to cyclic reversed loads. To achieve consistent high performance of welded connections requires proper design and execution (welding sequence, etc.) of all welds and limitation of plate thicknesses in welded joints in accordance with weldability of different steels.

Non-destructive inspection of welds should be specified in critical regions, particularly in connections with thick steel plates.

Panel Distortion. Excessive distortion and local buckling in the panel region of a frame joint can substantially increase the rotations at the joint and translational displacements of joints. To prevent excessive shear distortion doubler plates or diagonal stiffeners may be required.

Until substantiated or proved unwarranted by appropriate tests the shear stress caused by (D+L+E) loads in the joint panel surrounded by stiffeners and flanges on all four edges shall not exceed 0.3 F_y (a 25% reduction in the normally allowed shear stress).

P-Δ Effect. Frame deformation and joint displacements must be considered in evaluating the forces and moments at critical section. This can be done using "second-order" theory for analysis, or using approximate methods (e.g. an iterative procedure, or amplification factors).

H.3 Research

Considerable research has been carried out on behavior of structural elements under repeated and reversed loadings of high intensity. These investigations greatly increased the understanding of the problems and provided a basis for some improvements in design. However, the existing information has three major limitations:

(1) significant gaps exist in our knowledge of behavior of certain types of elements,

(2) laboratory studies have been largely limited to tests on isolated elements, rather than on full scale structures or on full scale subassemblies,

(3) loading histories used in the studies reported to date have not been adequately related to the actual actions (load histories) experienced by the structural systems under real earthquakes or other extreme conditions.

Therefore among the highest priorities in additional research would be studies on more realistic subassemblies (modeling portions of full scale structures). For example, the nature and magnitude of the contribution made by the floor slab to the behavior of the frame (beam and column) subassembly are essentially unknown. Further studies of this interaction may lead to more economical design.

Even more important, behavior of real buildings as systems subjected to realistic extreme loadings must be studied. For this purpose a special program must be developed, which would provide for coordinated analytical, laboratory and field studies.

Choice of loadings is equally important. Fig. H.1 illustrates some typical loadings used in studies of some steel connections. The magnitudes of peak loads and the sequence of cycles was selected somewhat arbitrarily. While no one set could be prescribed as a standard, guidelines should be developed for selecting loading conditions representing idealizations of real structures' response.

Furthermore, a number of special problems require further study. It is difficult to list these in a precise order of priority. Some of the high priority problems are indicated below.

Ductility and Limited Damage. The concept of ductility, while useful in some aspects of design, does not adequately reflect the dynamic (time-dependent) nature of special loading conditions and it does not properly reflect energy dissipation characteristics. It may also be misleading as in some cases it requires extensive local damage to develop reasonable overall ductility.

Prescribed ductility factors should be reviewed, their definition clarified, and particularly their relationship to local damage closely examined.

Guidelines for Evaluation of Safety of Existing Buildings. Many existing buildings while not entirely conforming to new codes reflecting current structural forms and practices, may nevertheless be safe - as a consequence of contributions from non-structural portions of the system. Guidelines for evaluation of their safety would be useful to communities wishing to minimize overall hazards to people and structures during extreme loadings.

Performance of Welded Thick Steel Plate Elements. Some concern has been expressed for potential vulnerability of such structural elements to low-cycle fatigue. Insofar as these elements are now widely used in high rise structures - evaluation of the performance of such elements is particularly important.

Residual Safety of Structures. Major earthquakes are likely to damage many structures. In view of the uncertainty of repair effectiveness - safety of residual (repaired or unrepaired) structures is particularly important. For example, changes in the local rigidity (stiffness) of joints may have decisive influence on stability of the structure during subsequent extreme, or even moderate, lateral loadings.

In addition many details of design, material behavior, imperfections inherent in fabrication and construction, and other characteristics of specific structural elements and systems require further study. Some of these have been identified in preceding sections; a complete list is entirely beyond the scope of this survey.

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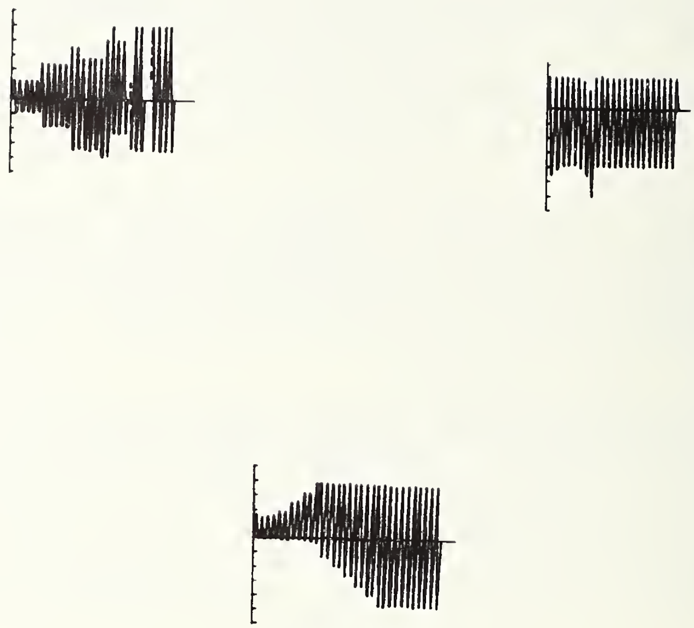

Fig. M.1 Loading Histories for Selected Tests

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BEHAVIOR OF STRUCTURAL SYSTEMS UNDER DYNAMIC LOADS

by

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1. Introduction

Structural systems may be subjected to many different types of dynamic loads. These may include wind, blast, and impact loading, earthquake ground motion, marine wave action, sonic boom pressures, and machinery-induced vibrations. Of these loadings, only the earthquake ground motion can affect, nearly simultaneously, all components of a structure including its contents. The techniques and procedures which have been developed for analysing earthquake-induced motions in structures can also be utilized in determining structural response to other types of dynamic loading. Therefore, this paper is devoted to presenting a comprehensive review of current and state-of-the-art procedures used in seismic analysis and design of various types of structural systems. Available research results are evaluated in light of current practice and areas needing improvement or research are discussed.

The advent of the computer age has made it possible to better analyse and design structures subjected to dynamic loading. Detailed mathematical modeling techniques have been developed so that almost any structural configuration can be modeled to realistically represent its physical characteristics. The model can then be subjected to various types of dynamic loads to compute its response. Once the response has been obtained, the individual members or components can be checked to ensure their adequacy to resist the imposed loads. If certain members are inadequate, they can be modified, and the analysis and checking repeated. Thus the complete design process for earthquake loadings consists of analysis-design cycles as do the processes for other more conventional loadings.

Considerable attention has been given to dynamic analysis with respect to the design of high rise structures; but the structures in which perhaps 90 percent of the people live, work, or sleep - the low rise and often non-engineered structures - have largely been ignored. With this in mind, the considerations affecting the behavior of this type of structures are discussed in this paper in addition to the more exotic or glamorous high rise buildings.

This paper presents first a general review of analytical methods, followed by a discussion of structural configurations and modeling procedures. Applications of the analytical methods are then presented, and many of the factors to be considered in the dynamic analysis and design of a structure are discussed. As equipment and systems supported in structures are often important to public safety, equipment-structure interaction and suggested analytical techniques are also discussed. Finally, present philosophy in practice and trends in design, together with possible simplified design techniques, are reviewed.

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II. General Review of Seismic Analysis Methods

Earthquakes are a global phenomenon as seismic activity is not confined to any one country or even one continent. Until about 50 years ago, earthquakes were largely considered to be acts of God, and no specific provisions were written into any code of practice to protect against earthquake hazards. Since that time, many methods of analyzing structural systems for seismic loads have been devised and are presently in use by engineers around the world. Most earthquake-prone countries have adopted building codes requiring some form of seismic analysis to guard against earthquake losses associated with the ground shaking hazards. These codes generally accept approximate equivalent static analysis methods.

The introduction of the high-speed digital computer in the sixties spurred the development of many computer programs which are capable of predicting the dynamic behavior of linearly elastic structural systems subjected to a prescribed earthquake ground motion. Subsequent program development has taken place that enables engineers to predict the inelastic behavior of idealized structures when vibrating under the action of predetermined seismic loads. Presently, new methods and techniques are being developed which assume earthquakes as being random events and as such, probabilistic predictions of dynamic behavior can be made. To date, however, most transfer functions used in making predictions of this nature have been assumed deterministic and linear in themselves. Before reviewing methods of dynamic analysis, the methods used to determine equivalent static seismic forces will be discussed.

A. Design Seismic Forces

The behavior of a structure when vibrating under the influence of seismic motions is dynamic in nature. Without the aid of a computer it is impractical, if not impossible, to determine the detailed dynamic behavior of structures under such conditions. Thus, in the past, engineers, while recognizing the dynamic nature of the problem, have sought to simplify and reduce the problem to a static one; dynamic forces are represented by code-prescribed equivalent static horizontal forces. The criteria are based largely on uniform type structures with symmetrical loadings. Present code-prescribed design seismic forces are not the seismic forces that can be expected in a structure during a major earthquake. Factors such as structural damping, ductility, participation of "non-seismic" elements, and the customary safety factors used in design are relied upon to ensure survival of structures in seismic events. The objectives of such designs are:

- to prevent structural damage and minimize nonstructural damage in small to moderate earthquakes
- to prevent structural collapse during great earthquakes
- to generally reduce hazards to public safety.

The general procedures used in the determination of code seismic lateral forces and the distribution of such forces throughout the structure are of interest. Static seismic design forces generally are equal to a certain percentage of the total weight of the structure and are applied, non-currently, along each of the principal horizontal axes of the structure. Many factors have been introduced that influence the magnitude of such lateral forces. These factors include dead plus other real loads, seismic zone factors, building type factors, fundamental periods of vibration of the structure, site soil factors, structure importance or occupancy factors, and foundation type factors.

Two methods are in general use for distributing the seismic lateral forces over the height of a structure. The first method requires that the total lateral force be computed for the structure using an appropriate base shear coefficient. This total force is then distributed, vertically, such that the sum of the static lateral forces applied at each level equals the total lateral force or base shear on the entire structure. The pattern of this distribution is usually governed by the overall dimensions of the structure or the nature, size, and location of any prevailing setback conditions.

The second method requires that the individual story lateral forces be directly determined using a prescribed lateral seismic coefficient for the particular level or story concerned. Normally, such lateral coefficients vary over the height of the structure in an attempt to account for the dynamic behavior of the structure during an earthquake. In this case, the total base shear is the summation of the individual story forces.

Once the individual story lateral forces are determined, the distribution of such forces to the various elements in the lateral force resisting system is usually accomplished by computing individual element forces in proportion to their relative rigidities. In some analyses, the rigidity of the horizontal bracing system or diaphragm is considered in the distribution. This implies that in the case of very flexible horizontal diaphragms, the distribution is done essentially on a tributary area basis. There are a number of approximate method for determining such distributions as discussed in the following paragraphs.

B. Approximate Static Analyses

Once the code forces are determined, it is necessary to perform analyses of the lateral force resisting elements, frames or shear walls, to obtain the forces to be resisted by each so that their adequacy can be ascertained. Many approximate methods have been devised and implemented for distributing the lateral forces. These will be discussed briefly.

For rigid frame structures, there are a number of approximate analysis methods, and these may be classified into three groups:

Group 1: Portal Method
Cantilever Method

Group 2: Spurr Method
Modified Portal Method

Group 3: Joint Coefficient Method
Factor Method

Group 1

The Portal Method and Cantilever Method incorporate basic approximations which disregard frame dimensions and sizes. They assume that points of contraflexure exist at the mid-points of both girder and column elements when subjected to lateral loads alone.

The portal method further assumes that all interior columns carry the same shear load and that exterior columns carry only half the shear of an interior column. Thus, the portal method implies that column rotations at the joints above and below are the same as that of the floor level under consideration, stiffnesses of all columns are the same, stiffnesses of all girders are the same, columns are very rigid in comparison to the girders, and vertical supports are unyielding.

The cantilever method does not assume shear distribution directly. It assumes that column vertical loads due to overturning moments vary as the distances of the columns from the centerline of the frame. Horizontal shear distribution in columns is then computed from the resulting vertical shears in the girders. Inherent in this method is the assumption that the girders are infinitely rigid.

The rationale for using the portal rather than the cantilever method depends greatly on the nature of the frame being analyzed. Most frequently, these methods are used only in preliminary sizing of elements.

Group 2

The second group of methods involve modifications to the portal and cantilever methods. Modifications include partial provisions for frame dimensions, element sizes, and deflections.

The Spurr Method modifies the Cantilever Method to size elements so as to improve the accuracy of the assumptions of the cantilever method. First, vertical shears in girders are adjusted to reflect actual column areas. The girders are then sized to produce equal girder drift contributions under the established shears. The method features a basic advance from Group 1 methods by considering drift and shears together. However, it represents a backward approach to the column-girder compatibility because it assumes the column vertical deformations to be of primary importance.

The Modified Portal Method presents an improvement in that points of contraflexture are not necessarily assumed at the mid-height of the columns located in the top and bottom stories, but usually at 0.6 of the story height.

Group 3

The third group represents a higher order of shear distribution methods for use in slide rule or desk calculator computations. Approximate provisions for individual element sizes, frame dimensions and deflections are included.

The Joint Coefficient Method assumes points of contraflexure at the midpoints of all girders and columns, and shear is distributed in proportion to joint coefficients computed for each joint. This method is perhaps the one most commonly used in slide rule computations. Care has to be exercised in using this method for irregular frames, especially at points of discontinuity.

The Factor Method is a refinement of the joint coefficient method. A partial carry-over system is used to provide for variations in the location of points of contraflexure.

For shear wall or box type structures the relative rigidities of each of the piers and wall elements comprising the lateral force resisting system are usually computed. Such rigidities generally include the summation of components associated with both shear and flexural deformations of individual members and the wall as a whole. Wall openings are, of course, always to be accounted for.

C. Detailed Static Analyses

The previously discussed methods of static analysis for code seismic analyses are suitable for sliderule calculations or desk-top calculator procedures. Recent trends indicate that more accurate computer solutions using matrix methods are being used to distribute horizontal shears between lateral force resisting elements and to determine structural member internal forces. This may reflect, in many cases, recent more stringent code requirements that require interaction between shear wall and frame elements to be considered. Finite element techniques utilizing truss, beam-column and plate elements are sometimes used.

The basic analytical matrix procedure used for these static analyses is the displacement (or stiffness) method. This method is used for linearly elastic structures, and is well documented in the literature [48, 54, 70, 75]. The basic assumptions of the displacement method are that the materials are linearly elastic (i.e., obey Hooke's law), the displacements of the structure are small (all computations can be made on the basis of the original geom - try), and that the effects of axial-flexural interactions can be neglected. Using these assumptions, the equations of joint equilibrium (or superposition equations) for a static analysis can be solved by any of a number of procedures (such as decomposition) to obtain the unknown joint displacements. Once the joint displacements are obtained, the member internal forces can be calculated by conventional means.

D. Deterministic Dynamic Analysis

An analysis is deterministic when both the configuration of a structure and the applied loads can be uniquely described. It is now generally accepted that mathematical models can be devised to reasonably represent real structures when subjected to strong earthquake ground motions. An analysis is considered to be dynamic if it recognizes that both loading and response are time dependent and employs a suitable method capable of simulating and monitoring such time dependent behavior. The following is a brief review of the various aspects involved in a dynamic analysis.

The dynamic behavior of any structural system is governed not only by the nature of the applied loading but also by the dynamic characteristics of the structural system itself. The dynamic characteristics of a linearly elastic structure are commonly represented by the natural frequencies and mode shapes of the structure. For every degree of freedom in a linearly elastic structure, there exists a unique natural frequency of vibration and an associated mode shape. It is necessary to obtain these dynamic characteristics if the modal superposition method is used to compute the dynamic response.

The fundamental frequency (or period) of the structure may be estimated by the use of code-prescribed approximate formulae, where the period is usually expressed as a function of building dimensions. When more accuracy is required, then computer programs which provide for the automated calculation of natural frequencies and mode shapes must be used. The calculation of such values by hand is usually only practical for very small problems.

Because earthquake ground motions are erratic in nature, they cannot be represented mathematically by a single continuous function for which a closed form solution can be obtained. They can be described in digital form. Most analytical methods that can utilize digitized load functions fall into two broad categories; Modal Superposition Methods or Step-by-Step Integration Methods.

The Modal Superposition Method recognizes that for linearly elastic systems vibrating within the elastic range, there exists a unique characteristic mode of vibration corresponding to each degree of freedom in the system. Furthermore, because the principle of superposition holds, the total response of the system can be represented by the sum of the individual responses from each of the characteristic modes. The method allows the equations of motion to be "uncoupled" and treated as a number of independent equations, each of which is analogous to the equation of motion for a single degree of freedom system. The modal equations of motion are then solved for each of these independent systems. Each of the modal equations are integrated to determine the response parameters for the modes under consideration at each time step, and the response parameters for the physical structure at each time step can be obtained by transforming the modal quantities to physical coordinates. Once the displacements of the structure are calculated, the remaining response parameters such as element forces or stresses can be computed by conventional means. The modal superposition, time history method is well accepted, efficient, and widely used by many engineers.

As an alternative, the response spectrum method can be used to solve the equations of motion. This method provides an estimate of the maximum response of a structure and does not require the computation of the complete time-history of the response. A response spectrum is a plot of the maximum values of some parameter, such as acceleration, experienced by a family of single-degree-of-freedom systems, when the systems are subjected to a time-history of input ground motion; the maximum values are expressed as a function of the natural period or frequency and damping of the systems. The engineering significance of the response spectrum lies in the fact that once the spectrum for an earthquake is known, it is possible to compute the forces induced in a single-degree-of freedom system by that earthquake. It is possible to extend the usefulness of the response spectrum to multi-degree-of-freedom systems by utilizing the modal superposition method of dynamic analysis.

Spectra are usually generated for single-degree-of-freedom (SDOF) linear elastic systems. They are occasionally developed for multi-degree-of-freedom elastic systems and non-linear systems. In addition they can be calculated with time as a third dimension to monitor variations in response maxima with time.

For simple structures such as elevated tanks or single story structures, SDOF response spectra can be used directly to obtain the maximum structural response. For more complicated multi-degree-of-freedom structures the maximum response in each mode is computed. The maximum values of the various response parameters must then be estimated because the time at which the maximum response occurs is not preserved in the spectral curves. Since the maximum values of individual modes do not necessarily occur at the same time nor have the same sense, the exact way in which various modes combine cannot be precisely determined. Commonly suggested methods of modal combination are to compute the sum of the absolute values of all significant modes or to compute the square root of the sum of the squares of each individual modal response. Other methods that have been used include the computation of the square root of the sum of the squares of the largest individual modal contribution plus one half of the sum of the squares of all other contributions or the sum of the absolute values of two or more of the significant modes. The first method yields an upper bound. The other methods should be used with care because they can yield unconservative results in certain instances.

The Step-by-Step Integration Methods involve the direct numerical integration of the equations of motion. These methods incorporate general techniques and as such are applicable to elastic or inelastic systems. It is not necessary to solve for frequencies or mode shapes nor is it necessary to transform the system of equations to normal coordinates. Changes in geometric, material, or connectivity properties of the structure may be made at any time step by modifying the appropriate mass, damping, or stiffness parameters.

The cost of implementing an analysis of this type relates directly to the size of the time step used in the integration. In general, the larger the time step the greater the likelihood of obtaining an unstable or inaccurate solution. Thus, a balance between cost and accuracy is frequently a principal concern, especially in the case of large structures.

The development of numerical integration methods has attracted much attention recently. Attention is now being directed toward developing criteria to define stability limits for various integration schemes. Efforts have been directed toward general solution procedures that are applicable to dynamic analyses of assemblages of structural elements exhibiting various types of nonlinearities. Some of the more widely used numerical integration methods are: Taylors Series Expansion Method, Newmark & Beta Parameter Method, Runge Kulta-Gill (4th order) Method, and Linear Acceleration Method. Details on these methods can be obtained from the References [26, 49, 51, 52].

E. Probabilistic Methods of Seismic Analysis

The type of probabilistic method of seismic analysis to be used is largely determined by the form in which the loading is specified. Statistical predictions of seismic loadings can be made in terms of probabilistic seismic coefficients or by defining the ground motions in terms of probabilistic response spectra or stochastic processes. The probabilistic analysis can be extended to incorporate the uncertainty in the structural characteristics. Probabilistic modeling of the structure almost invariably requires a simulation-type of approach, such as Monte Carlo simulation, because analytical solutions become extremely difficult if not impossible. However, the structures are generally considered deterministic because the structural uncertainty is relatively small and the probabilistic modeling results in complications described above.

In the case of probabilistic seismic coefficients or response spectra and a deterministic structure, the analysis can be performed as described in the section on deterministic analysis except that the seismic structural responses are probabilistically determined by appropriate probability combinations and deviations. If the seismic ground motions are modeled as stochastic processes, the so-called random vibrations approach becomes necessary. The stochastic processes can be stationary or non-stationary and are defined by such characteristics as spectral density and correlation functions, probability distributions and moments, etc. as pertinent. The characteristics are postulated on the basis of the past ground motion data and convenience in analysis. The structure is generally modeled as a linear-time-invariant system and the concept of transfer function (which can also be used advantageously in some deterministic analyses) is used to probabilistically determine the structural response. Nonlinear and time-variant systems can also be analyzed but with increased difficulty and effort. Details of the random vibration analysis can be found in the literature [2, 16, 17, 20, 21, 35, 42, 53].

The probabilistic methods of analysis are perhaps better suited for seismic and other similarly erratic loadings because the uncertainty of the phenomena is treated explicitly. However, these methods have not been used extensively, so far, because they are still being formulated and developed to be useful in specific applications.

III. Review of Structural Configurations and Modeling Procedures

A. Engineered Structures

Building structures may be of many types and configurations, and the possible approaches to the modeling of such structures are equally varied. The following text describes a number of the more widely used approaches to the representation of the physical characteristics (or the modeling) of building structural systems. The discussion is directed primarily to the analysis of structures for dynamic loads. Normal static vertical load analysis, while implicitly included in many cases, is not explicitly discussed.

Prior to the advent of the high-speed digital computer, the engineer was forced to rely on approximate techniques that could be performed either by hand or by means of a desk calculator. Examples of these techniques are the Cantilever and Portal methods and modifications thereof as discussed in the previous section. In these cases, certain assumptions are made regarding the distribution of internal forces within the members of the structure, which result in reducing a statically indeterminate problem to a statically determinate one which can be readily solved. Other approaches such as those based on the slope deflection equations and moment distribution method have also been employed. These methods, while still used by some engineers, have largely been precluded by more efficient techniques based on the displacement (of stiffness) method of analysis and the use of digital computers.

Modern structural dynamic analysis has been made possible by the development of large, high-speed digital computers and by the systematizing of the calculations in matrix form. The two complementary methods - force (or flexibility) method and the displacement (or stiffness) method - can readily be programmed. The latter method is actually more suitable for computer generalization.

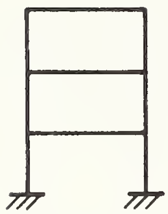
The lateral-force resisting elements of a structure may be one type or a combination of two basic types - frame (moment resisting, braced, tube), or shear wall. In the analysis of structures of these types, it is generally assumed that the floor diaphragms are rigid in their own plane. In most of the initial efforts, which were directed towards planar structures, it was assumed that the floor slabs only translate and do not rotate. This assumption is valid when the structure is reasonably symmetrical. If the structure is not symmetrical, then the building must be modeled three-dimensionally. In this case, the floors are assumed to translate in a horizontal plane and rotate about a vertical axis. Examples of two- and three-dimensional building models are shown in Figures 1 and 2, respectively.

A.1 Framed Structures

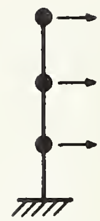
The dynamic analyses of linearly elastic moment-resisting framed structures and braced frame structures, both planar and three-dimensional, have been well documented in the literature in the past few years [5, 7, 10, 22, 23, 35, 48, 54, 70, 71, 72, 73, 74, 75]. Many commercial and private computer programs exist that are capable of analysing these cases. Most engineers should, as a minimum, have access to programs capable of performing such analyses. The engineer should satisfy himself as to the validity and the applicability of any program he uses.

Considerable attention has been directed toward the static and dynamic nonlinear analysis of framed structures in recent years. In numerous cases it has been observed that the computed elastic response of a structure due to earthquake ground motions is considerably greater than code design response, and yet buildings designed for code forces have withstood rather severe earthquakes. This disparity has been attributed, at least in part, to the energy absorption provided by deformation of members beyond their elastic limits. Attempts to more realistically model this phenomenon have generally involved the utilization of elasto-plastic, bi-linear, or Ramberg-Osgood moment-curvature functions for the beams and columns of the structure, and the performance of step-by-step analyses to determine the response of the structure. Thus a considerable portion of the effort in the development of nonlinear analysis techniques has been concentrated in better modeling of members stressed beyond their elastic limits, although axial-flexural interaction (the so-called P-delta effect) has also been investigated.

The foregoing discussions have pertained primarily to two-dimensional (planar) structures. Nonlinear analyses of space frames or three dimensional framed buildings have been somewhat hampered by lack of test data and adequate theory for yielding of members intersecting at a complex three-dimensional joint. Work in this area is progressing.

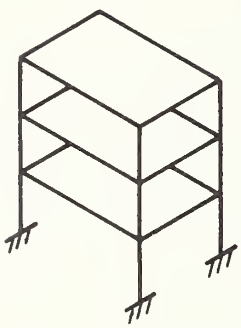


Planar Structure

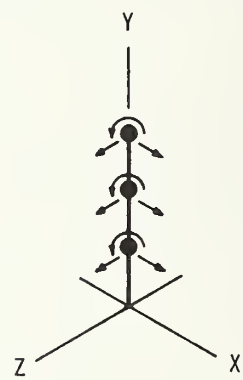


Mathematical Model

FIGURE 1   EXAMPLE TWO-DIMENSIONAL MATHEMATICAL MODEL



3-D Structure



Mathematical Model

FIGURE 2   EXAMPLE THREE-DIMENSIONAL MATHEMATICAL MODEL

Other approaches to reconcile observed behavior during earthquakes have included the Reserve Energy Technique [4] which provides an approximate method of correlating observed and predicted damage due to ground motions. This method, which is based on the concept of an energy balance between input kinetic energy and energy losses associated with the vibrating structure, is useful in estimating the total structural resistance capability and the nature and extent of damage, if any, to various types of structures.

A.2 Shear Wall Structures

The analysis of buildings with shear walls has received considerable attention in recent years, and numerous analytical techniques have been utilized to determine the deformations of and stresses in the walls due to lateral loads. The state of the art of research in this area was summarized by Coull & Smith in 1966 [24] and by the American Concrete Institute in 1971 [1]. A brief review of the methods of analysis that have been employed follows.

The most basic or elementary method is to use beam bending theory, which can be used for a single shear wall without openings that is not connected to other walls or to frames. When interconnected walls or walls combined with frames occur, more advanced theories must be used.

One method for treating interconnected (or coupled) shear walls is the elastic continuum approach, which has received considerable attention and has been reported in the literature in numerous instances. In brief, this approach, in its most elementary form, assumes that the elastic properties of two walls connected by beams at floor levels remain constant and that the connecting beams can be replaced by a continuous connection of elastic laminae. The basic limitation of this approach is the restriction on the geometry of the configuration of the wall system. Only the simplest of cases can be treated. ACI Committee 442 [1] presented their opinion on the continuum approach when that stated that the continuum approach has received a great deal of attention and would seem to have reached a state where further development in analytical methods for this approach, while of theoretical interest, is not likely to result in greatly improved design techniques.

A significant advance in the analysis of buildings with shear walls came with the development of the high speed digital computer and matrix techniques of structural analysis. One resulting approach to the analysis of frames and walls is to use frame analysis programs which have the capability of treating finite joint size. In this approach, the walls are analyzed as frames, where the connecting elements between walls are treated as beams, and the walls are treated as equivalent columns. The finite widths of the equivalent columns are treated by assuming that the beams are infinitely stiff in the region from the center line of the column to the edge of the opening. This approach has been described by numerous authors [22, 23, 43].

An alternate approach is to utilize the existing frame analysis programs which do not have the capability of representing finite joint size. In this approach, the columns are assumed to be of the same dimensions as the walls and equivalent members are used to represent the beams. A member of equivalent stiffness is derived to represent a beam with infinitely stiff end regions.

The two computer approaches described above have been termed frame analogies or wind column analogies and are often used by the practicing engineer.

Most of the procedures described in the literature for the treatment of buildings with shear walls are for two-dimensional or planar problems. The three-dimensional problem has been investigated by investigators [34, 71] who have sought to determine both the lateral and the torsional stiffness

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of the frame and shear wall structure. Examples of these approaches have included the application of non-uniform torsion theory of thin-walled members to determine the wall stiffness properties.

The development of the finite element method of structural analysis, documented in excellent tests [48, 75] has made more detailed investigations into the behavior of shear walls possible. The finite element method permits investigation of complex shear wall configurations and does not require the idealizations and approximations required by the methods previously described. The application of the finite element method to the analysis of shear walls has been discussed in numerous instances in the literature [31, 44].

A.3 Frames with Filler Walls

In addition to shear walls, filler walls (or infilled walls or panels) or partitions may occur in a structure. Filler walls and partitions are not generally considered by the structural engineer to carry vertical load. They often serve only to partition space or to enclose a structure, although in some instances they may be designed to resist lateral loads. A filler wall usually fills the space created by the beams and columns of a building frame and is usually in or slightly offset from the plane of the frame. A partition is any nonstructural wall that is used to divide space within a building, and it may be offset from the frames. The importance of filler walls to structure response is generally recognized and has been pointed out by several authors [5, 9, 25, 28, 30, 40].

Various approaches have been used to model the effects of filler walls on frames; one such approach is the equivalent brace or strut method. In this method, the wall is replaced by a diagonal strut in the frame. Formulae based partly on test data and partly on analytical studies to determine the stiffness of the equivalent strut have been developed [8, 64, 65, 66].

As previously mentioned, the development of the finite element method has had a considerable impact on the field of structural engineering, and a logical application of the method is to model the effects of filler walls on the response of building frames. This approach has been utilized and presents a rational basis for the determination of the dynamic response of a structure with filler panels [40, 47]. An example of an analytical model used for a four-story frame structure with filler panels is shown in Figure 3 [40].

B. Non-Engineered Structures

Building codes presently do not cover the detailed seismic design of all buildings. The SEAOC Recommended Lateral Force Requirements [62] exclude all Type V buildings of Group 1 occupancy which are less than twenty-five (25') in height, and exclude minor accessory buildings. Type V buildings with Group 1 occupancy include dwellings and lodging houses. The UBC specifies that all buildings or portions thereof which are constructed in accordance with the conventional framing requirements specified in Chapter 25 (Wood) [36] shall be deemed to meet the seismic design requirements.

This approach by codes for dwellings and similar structures is not surprising. Detailed seismic analysis for each small building is generally uneconomical because of the design cost for the owner and the extra cost for building officials to check the calculations. The code requirements for such structures have usually been derived from empirical experience and generally have been satisfactory for structures built in the past 15 to 20 years. Present code requirements and design procedures, however, do not provide uniform factors of safety because the dynamic response characteristics of these structures are largely ignored. Ties or anchorage of walls and other

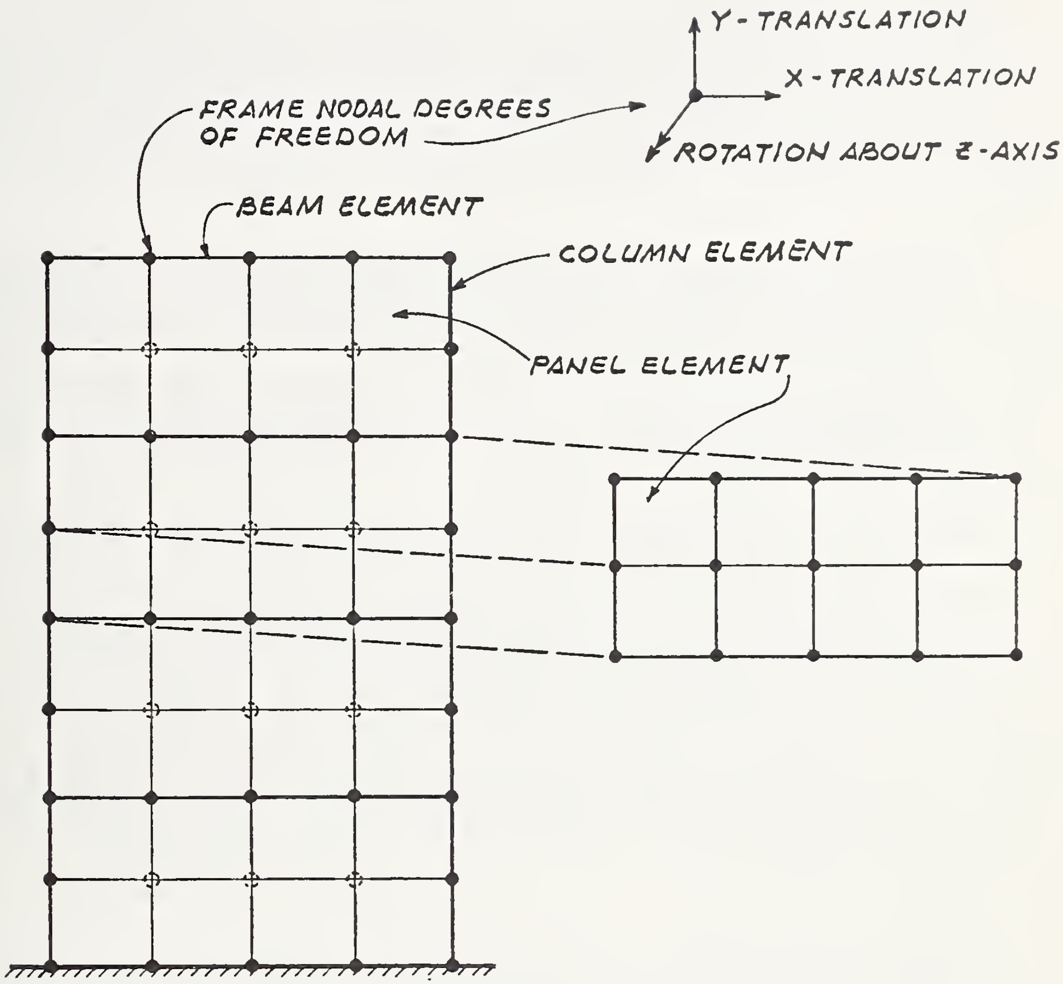

FIGURE 3 FRAME STRUCTURE WITH FILLER PANELS

components often are specified based on calculations of typical situations. Residential damage at Santa Rose in 1969 and at San Fernando in 1971 mostly occurred to structures older than 15 years, although many new or relatively new homes in areas of severe ground shaking suffered considerable damage or collapsed during the San Fernando earthquake.

Damage to residences and other small non-engineered structures often occurs due to exterior walls pulling away from the roof and floors and thus causing serious stability problems or collapse. Other types of damage include failure of chimneys, building sliding off of foundation piers, or excessive lateral deformations causing cracking of plaster and breaking of windows or collapse. In addition to structural damage, many bric-a-brac and similar items have been damaged by sliding or falling off of shelves or furniture.

Observations of many incidences of damage and specific response measurements indicate that the dynamic response characteristics of non-engineered structures can significantly affect their response to seismic ground motion. Vibration tests of residential buildings indicate that their fundamental periods fall in the range of approximately 0.05 to 0.2 seconds [57]. One-story masonry buildings generally fall in the lower end of this range and two-story wood frame buildings fall at the higher end. Vibration test data also show that soil-foundation stiffness influences the dynamic response characteristics of low rise structures.

The detailed consideration of system dynamic response characteristics in the analysis of such structures would be too costly in nearly all cases.

An approach whereby simplified procedures can be used must be developed. Such procedures should consider the structure stiffness - whether it is rigid, semi-rigid, or flexible; the type of construction - all wood, wood and plaster or stucco, wood and gypboard, or masonry; height of structure - whether one or two-story; and foundation soil properties - for example, seismic shear velocity V_s > 3,000 to 4,000 fps, $V_s \geq 1,500 \leq 3,000$ fps, or V_s < 1,500 fps (or some other values). Tabulations of structure response characteristics for each of the above could be developed and appropriate lateral force criteria specified. Of course, connection details, ties, anchors, etc. should also be specified.

C. Computer Applications

Numerous computer facilities, methods of communication with computers, and computer programs are available to the structural engineer today. The choice of which system to use is perhaps most often based on such factors as: program availability, economics, convenience, and service. This section presents a brief discussion of various aspects and factors involved in computer usage and is directed primarily towards digital computers because of their wide-spread use and versatility.

C.1 Machines and Facilities

There are many types of machines and facilities available, ranging from the small, relatively inexpensive desk-top computers suitable for office use to the large, high-speed machines available at the various commercial data centers.

The small desk-top calculator/computers, which are capable of operation as a calculator and can be programmed for a limited number of instructions, have a distinct place in the structural office. They find use in the solution of small problems of a repetitive nature for which it is uneconomical and inefficient to punch data cards for use on a larger machine. Output from these calculator/computers is often displayed on cathode ray tubes or may be printed on paper tape. Some machines have capabilities for plotting.

The larger machines are, of course, more suitable for large problems. Some of the more familiar large computers are the IBM 360 series, the UNIVAC 1108, and the CDC 6400 and 6600. At the top of the list in terms of computational and storage capabilities are perhaps the CDC 7600 and the IBM 360/195. Certain of the available machines have been developed primarily for business applications and others have been developed for scientific usage. Machines of the latter type are generally more satisfactory for engineering applications because of fewer difficulties with numerical precision.

C.2 Communication with Computers

Communication with the smaller calculator/computers is generally by means of a keyboard or paper tape; there are numerous methods and languages available for communication with the larger machines. These methods can perhaps be categorized as two basic types: conversational and batch.

The conversational mode of communication with computers is generally implemented by use of typewriter-like, low-speed keyboard terminals. The terminal may be at the computer center, or may be at the user's office and linked with the computer center by telephone lines. Using such a keyboard terminal in a timesharing environment, the engineer can instruct the computer by means of a specially developed language, such as BASIC or APL. Program execution can be started, interrupted, or stopped; program statements or values can be altered; and execution can be resumed upon command by the user. Such systems usually include libraries of mathematical and engineering programs and subroutines. The keyboard terminals are suitable for problems that require more programmed instructions than can be used in the desk-top type devices, and yet do not require large quantities of input or output or long programs.

Other more elaborate conversational systems utilizing keyboard terminals have expanded ranges of capabilities - from conversationally developing a program to changing a program already stored - or to simply executing a stored program. Such systems are capable of conversing in the well known languages such as FORTRAN and COBOL, and some systems permit optional input or output at either the terminal or data center.

Communication with computers in the conventional batch mode simply means the input of the program and data by means of cards, tape, or disk. Operational instructions are provided to the system by means of control cards, and the computer programs themselves are usually in one of the more common languages, such as FORTRAN. This type of service is oriented toward the processing of large production jobs through a variety of types of high-speed terminals.

The high-speed terminals, which may be installed in the engineer's office or at a central location within a city, vary in type and complexity. They may consist of a cathode ray tube and keyboard for command and query purposes, card reader and/or punch, a high or low-speed printer, plotting capabilities, tape or disk drives, and a small computer which permits some calculations at the terminal. Hybrid modes of operation are also possible. A data base may be developed interactively and executed in a batch processing mode.

C.3 Computer Programs

There are many programs for the static or dynamic analysis of building structures that are commercially available or that can be obtained from various sources. Some of the more popular and important of these programs are:

DYNA - STARDYNE - Static/dynamic structural analysis system for static, stability, and dynamic analysis of linear elastic structures.

EASE - Performs static structural analysis of linear three-dimensional systems using beams, membranes and plate structural elements.

ELAS - Finite element program for static three-dimensional structure analysis.

FRAN - HOUSEM - Framed structure analysis program for large three-dimensional frames.

ICES - Integrated civil engineering system supporting, among others:

DYNAL - Dynamic structure analysis

STRUDEL - Framed structure analysis

NASTRAN - Static and dynamic analysis of general three-dimensional structures.

SAMIS - Structural analysis and matrix interpretive system for computing forces, displacements, stresses, mode shapes, and frequencies of generalized structures.

SAP - Structural analysis program for linear elastic analysis of three-dimensional structural systems.

STRESS - Structural engineering system solver for frames, trusses, culvert sections, etc.

IV. Factors to be Considered

The response of any structure to dynamic ground motion is a complex phenomenon. Many factors have to be considered - often they are interdependent. A number of important considerations which can affect the response of a structure are discussed in this section.

A. Damping and Energy Losses

Determination of the damping coefficients to be used is one of the most important and difficult steps in the seismic analysis. There are relatively few applicable test data to support an accurate estimate of the true damping of a structure. Most available test results are based on very small amplitude distortions or on component tests, and the results probably do not accurately reflect the damping that might be expected for the large amplitude motions associated with a severe earthquake. And yet, small changes in assumed damping may significantly change the calculated response of a structural system. It is apparent, therefore, that considerable effort is required in the research field to provide test data that will minimize the uncertainties in the damping values currently used. The following is a discussion of several aspects of the damping question. Topics covered are: analytical conditions, test results, and possibilities for isolation or increased damping.

A.1 Analytical Considerations

The literature on damping contains frequent reference to the fact that damping in real structures, especially those of composite materials, is some unknown combination of energy absorption by internal elastic and inelastic deformation of the materials including foundation soils, and by rubbing or friction between various elements and materials. Damping is no doubt a combination of various forms, such as Coulomb, viscous, etc. The Coulomb or frictional damping force is constant under velocity changes, the viscous damping force is proportional to velocity, and there may well be quadratic or other powers of velocity that would best express the real damping forces developed during building vibration. In spite of these uncertainties it is customary, and always convenient, to assume that damping is of viscous form.

In the case of more complex multi-degree-of-freedom systems, the energy absorption capabilities of a structure are also treated by convenient mathematical techniques. In linear elastic analyses by the modal superposition method, damping is usually assumed to be simple viscous modal damping. In linear or nonlinear direct integreation analyses, damping is often assumed to be proportional to the mass and/or stiffness of the structure. The validity of this latter approach is questioned by many investigators. In the direct integration case, other procedures have also been utilized to rationally determine the damping matrix [40].

All of the above techniques account for damping in an overall sense. No detailed attempt is made to specifically define the distribution of the energy absorbing capabilities. The energy absorbing capabilities of a structure are generally a function of the types of construction and materials used for structural members and for nonstructural walls and partitions. A structure that is relatively brittle will have little strength beyond its elastic limit and hence little capability to absorb energy and resist overload. On the other hand, a relatively ductile structure that can withstand numerous cycles of deformation into the inelastic range will have considerable energy absorption capacity.

These concepts led to the development of moment-resisting ductile concrete [13] and steel frames. The basic idea is to ensure the capability of the structures to deform past the yield limit. It is important to execute the design so that any yielding will initially occur in the beams or girders to preclude possible instability of the columns. In using this approach, the energy absorption capabilities of the structural frame are, in a rough sense, distributed in proportion to the energy absorption capability of the individual members. The distribution of energy absorbing components can significantly effect the response of a structure. For example, shear walls supported by columns can produce discontinuities which result in extremely high forces and stresses. As another example, earthquake motions can produce high torsional motions in unsymmetrical buildings. Nonstructural partitions and panels can contribute to the energy absorption capability of a structure, although the degree of contribution may be small in modern structures. These energy absorption capabilities may be distributed throughout the structure.

The total kinetic energy of a vibrating system may be equated to heat gain, stored or potential energy, and work done. Although linear systems are usually assumed, many systems are not truly linear for various reasons including hysteresis, yielding, cracking, deterioriation, failing, etc.

For inelastic analyses, the concept of equivalent damping [37] is useful in that the inelastic work is combined with hysteresis to provide a measure of effective damping for possible inelastic excursions. However, this leads to complications including variable values under different cycles and to errors under large nonlinear deformations.

The Reserve Energy Technique [4, 13] is a concept of analysis under which the damping is confined to hysteresis or heat loss with no deterioration or distress to the structure under repeated cycles; i.e, the damping is restricted to what would normally be considered linear response. The work done in the inelastic range plus the stored energy is considered to be represented by the area under the force-deformation curve, adjusted for any deterioration under repeated cycles. Thus non-damaging hysteresis and damping inelastic excursions can be kept separate in design or analysis.

A.2 Test Results

Damping values are generally determined experimentally by one or more of several available means; steady state forced vibration or near-steady state forced vibration, various means of inducing free vibration such as pull and release tests, and the spectral response reconciliation method [6]. The assumption is usually made that the effects are the same as if the damping were truly viscous and the damping force is proportional to the relative velocity of a single-degree-of-freedom system. There may be some error associated with these assumptions, but there are distinct advantages to the simplicity and tendency toward standardization of this approach.

Other considerations affecting the damping ratio are the variations with amplitude or stress level, and variation with the prior history of motion. Comparisons should consider these parameters, when and if data are available.

A comprehensive review of test results reported in the literature and other services was performed [11]. This work included the compilation of data on 171 structure types or components for which there were a total of 409 damping determinations. Of these, there were data on 74 buildings (230 damping determinations). The results of this compilation for buildings is shown in histogram form on Figure 4. It would be expected that the damping ratios would increase in many cases under greater amplitude of motion than that for the tests reported. In general, the higher damping values are associated with composite construction of many materials and elements, and with greater amplitudes. The low damping values are generally associated with non-composite and more simple construction, and with lower amplitudes. The prior history of motion is also an important parameter in reinforced concrete framing and composite construction.

One factor that seriously complicates the damping problem is that results of various tests suggest that damping in a complex structure is dependent on both relative and absolute velocity, displacement, and frequency. The fact that damping may be displacement-dependent is particularly vexing because this tends to cast doubt on the results of all low amplitude vibration tests as far as damping is concerned.

In order to study the effect of displacement on damping as well as to obtain other basic data, two full-scale, four-story reinforced concrete frame test structures were constructed at the AEC Nevada Test Site near Las Vegas, Nevada. These structures were designed and tested by John A. Blume & Associates Research Division for the Structural Response Program, Office of Effects Evaluation, Nevada Operations Office, United States Atomic Energy Commission [9, 30, 50]. Some tentative results of this testing program are shown in Figure 5.

For these four-story structures, partitions of various types were installed, subjected to ground motions, and then removed. In general, damping is higher for the structures with frame and partitions than for the frame alone (Figures 6 and 7). However, it has also been observed that the damping can decrease in a structure with frame and panels, after the structure has been subjected to numerous cycles of ground motions. This can be attributed, at least in part, to the destruction of the bond or connection between frame and panel and subsequent ineffectiveness of the panel in the dissipation of energy.

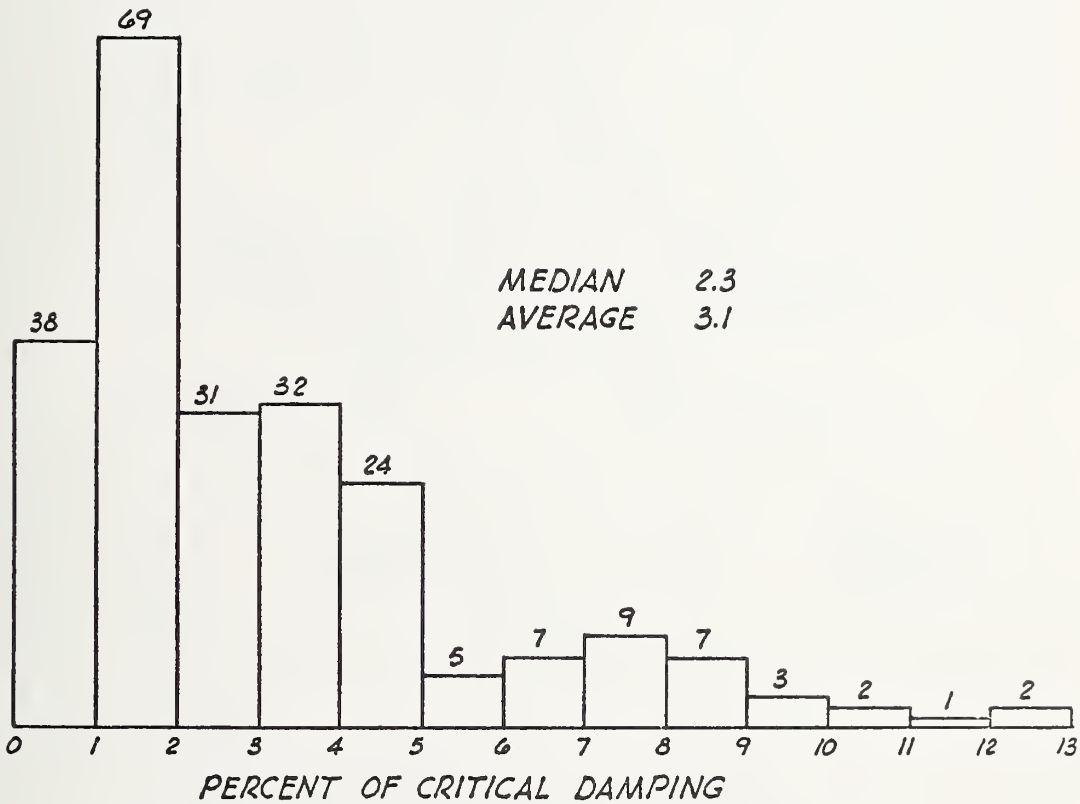

FIGURE 4 HISTOGRAM OF 230 DAMPING DETERMINATIONS FOR BUILDINGS

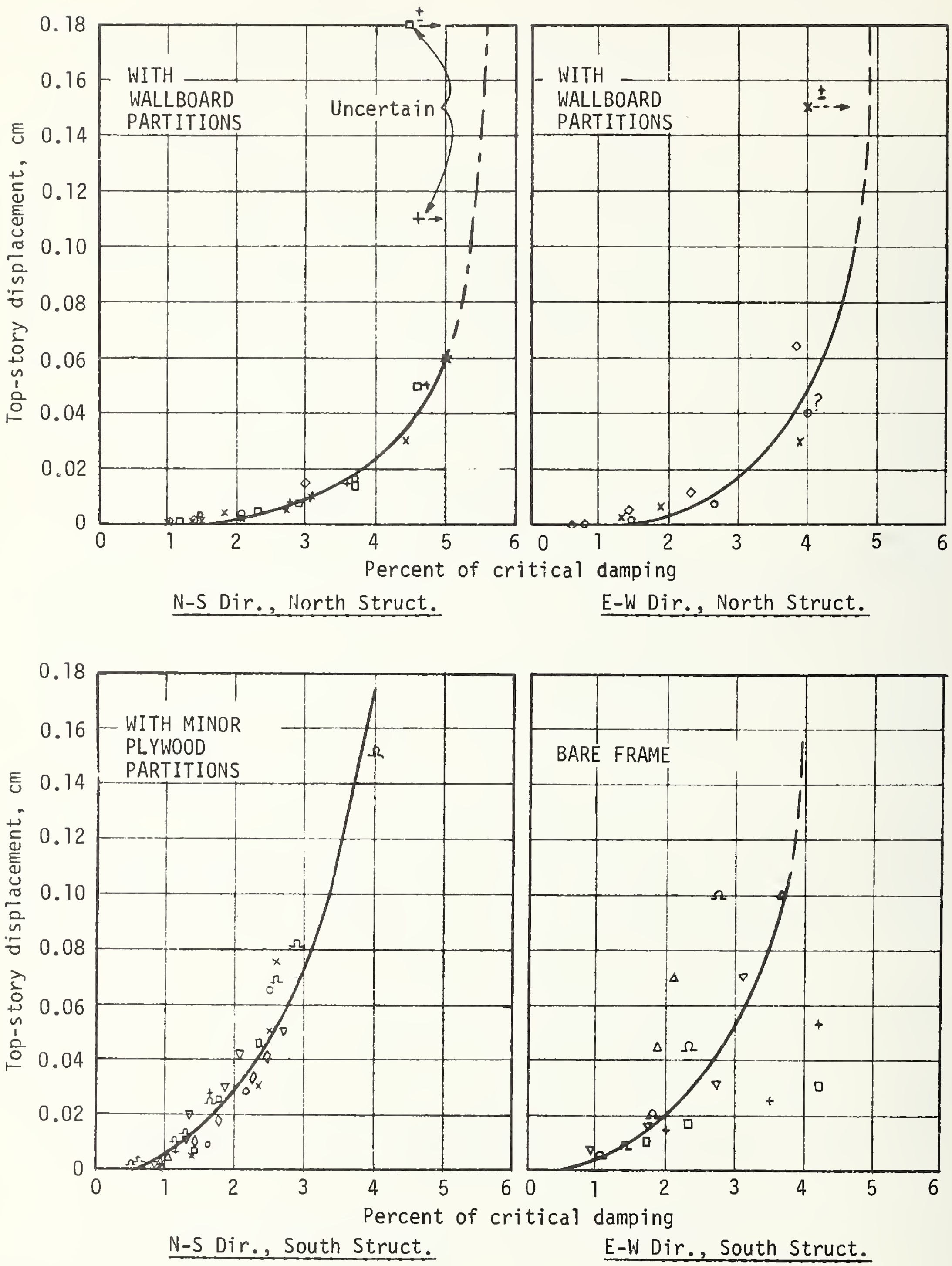

FIGURE 5 DISPLACEMENT VERSUS PERCENT OF CRITICAL DAMPING

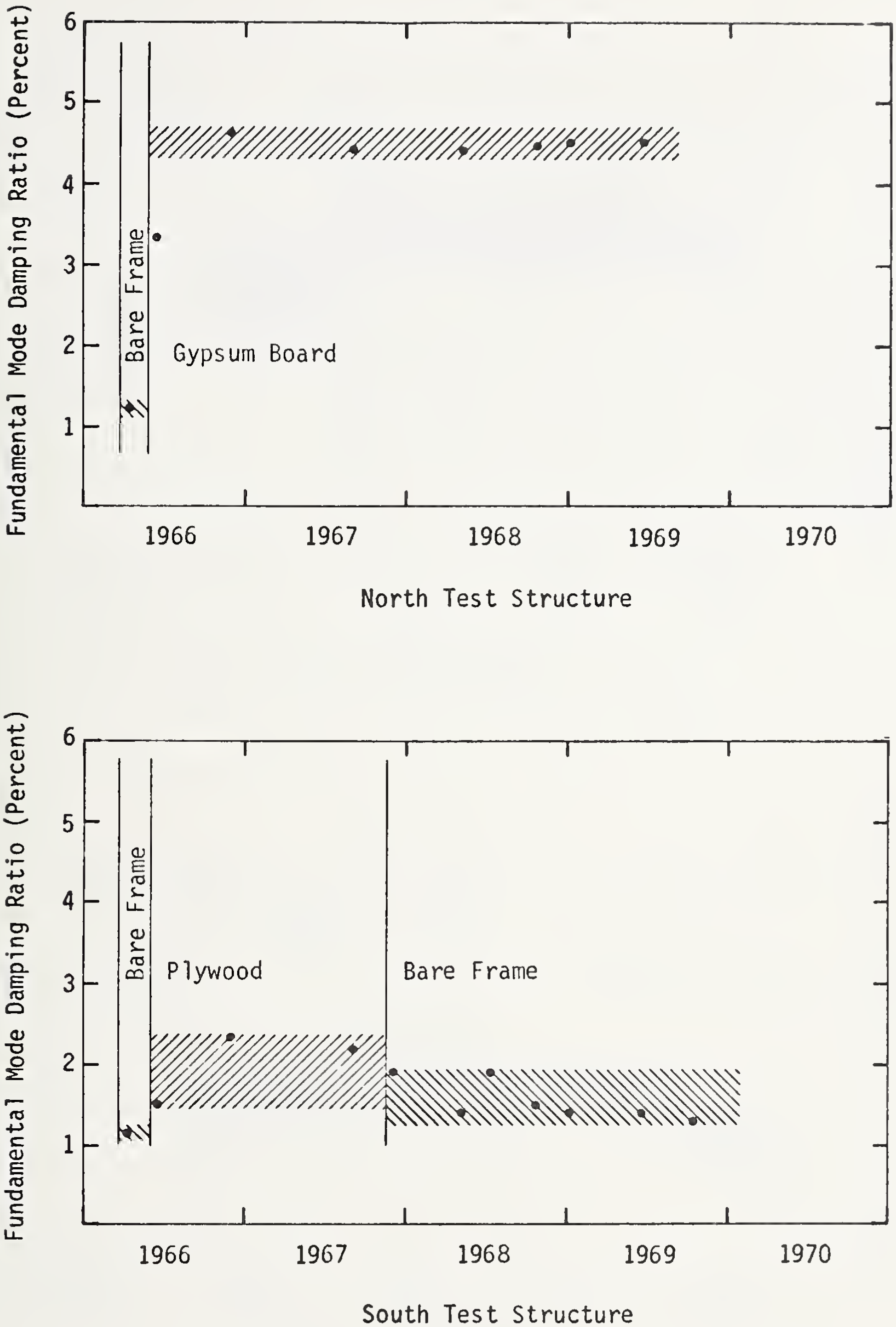

FIGURE 6 EXPERIMENTALLY OBTAINED FUNDAMENTAL MODE DAMPING RATIOS FOR LONGITUDINAL MOTIONS

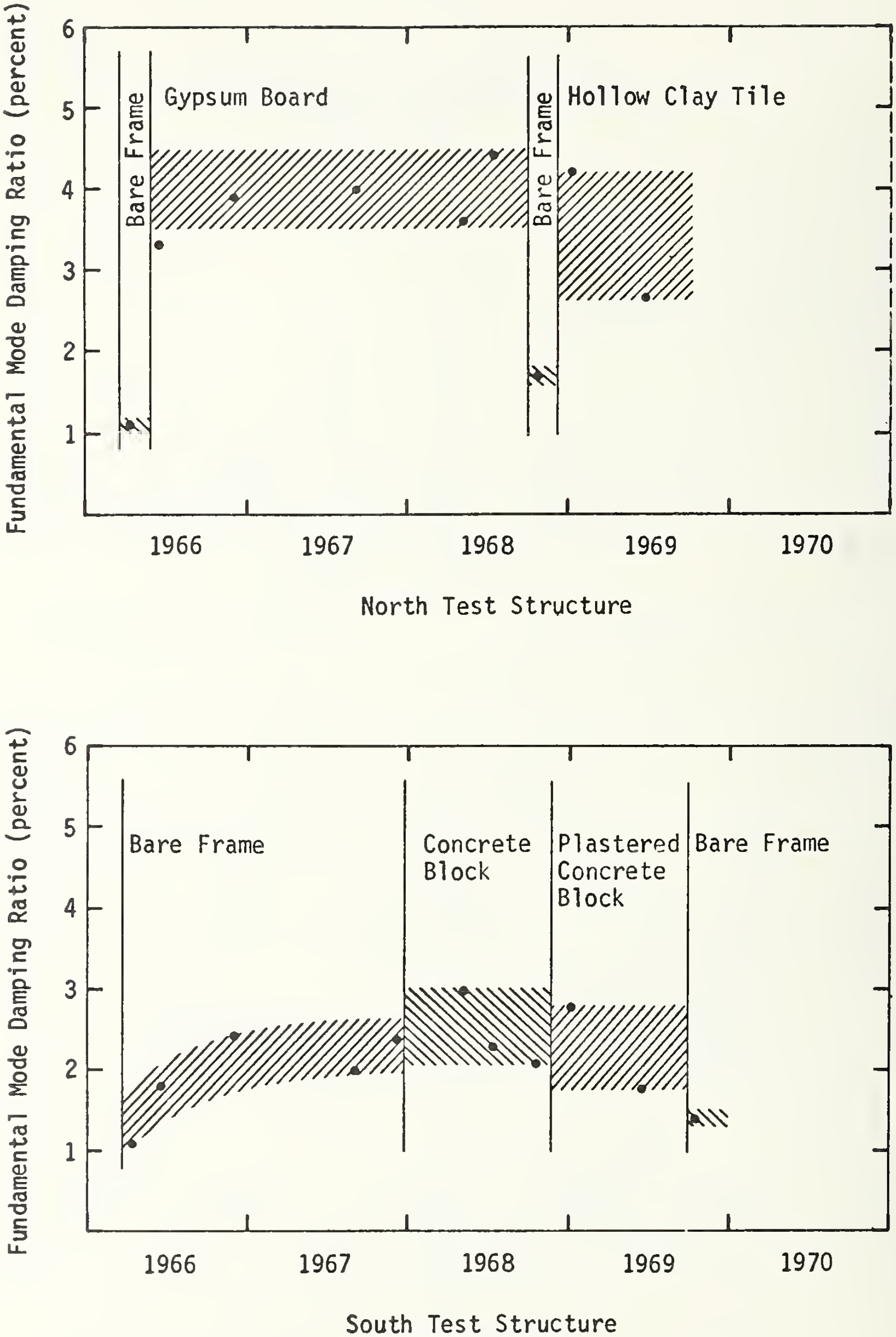

FIGURE 7 EXPERIMENTALLY OBTAINED FUNDAMENTAL MODE DAMPING RATIOS FOR TRANSVERSE MOTIONS

Studies of data recorded from the San Fernando Earthquake for several high rise structures indicate that damping can vary considerably [12]. For structures that responded within the elastic range, some low damping values were obtained - on the order of 1% to 3% (of critical viscous damping). For two similar concrete structures, which were excited to roughly the same levels, quite different damping results were obtained. For one structure, damping of approximately 5% to 10% was obtained, whereas damping of 2% to 5% was obtained for the other structure. Both of these structures showed a marked change in response characteristics after about six seconds of earthquake induced motion - which indicates a change in structural properties possibly due to excursions into the inelastic range or due to failure of the connections between partitions and the structure.

Tests have also been performed on a series of one-story and two-story houses [57]. These houses, located in Colorado, were constructed of adobe, wood frame, or concrete block, and were generally in poor condition in that their structural integrity was not comparable with modern construction. The tests performed on these structures were of the pull-release type, in which a static lateral force was applied on each building near the eave line to deflect the building. The force was then released and the resulting free vibration motion recorded. The experimentally determined fundamental periods of vibration of these structures were in the range of 0.09 to 0.20 seconds. The damping in these structures was determined from the decay rate of the recorded motion. Values obtained are summarized in Table 1. The data indicate an average damping of from 5% to 6% for these houses.

It can be concluded from a review of past test results that data to adequately define the magnitude and mechanism of damping are still definitely needed. Until these data become available, the engineer is forced to use his best judgment in the selection of damping values. Conservative values should be selected to ensure satisfactory and safe performance of the structure.

Table 1

OBSERVED BUILDING-AS-A-UNIT MODAL DAMPING

Structure Number	Structure Description	Test Direction	Damping (Percent of Critical) Variation	Damping (Percent of Critical) Average
1	One-Story Adobe House	Transverse	4 - 7	5.7
2	Two-Story Frame House	Transverse	4 - 10	6.0
3	One-Story Frame House	Longitudinal	0 - 30	*
4	One-Story Concrete House	Longitudinal	**	5**
5	Free-standing Chimney	Longitudinal	2.5 - 4.0	3.0

* Approximately 30% damping observed in first cycle, thereafter damping was 0 - 2%

** Approximated -- record too noisy to obtain damping for individual peaks.

A.3 Isolation and Increased Damping

Numerous ideas have been set forth by various authors to isolate a structure from the earthquake-induced ground motions. These ideas have generally involved the concept of increasing the fundamental period of the structure to the point where the structure is so flexible that its response to the ground motions is extremely small. One method by which this can be accomplished is by the use of the so-called flexible first story, where the columns of the first story are relatively flexible [27, 33, 45]. Another approach, which has been employed on several recent buildings, is to "hang" the structure from a central core (or cores) by means of cable systems, thus greatly increasing the flexibility of the system. Other proposed methods are to use mechanical devices, such as rollers [18], or to use flexible materials between column base plates and the foundation system.

Various schemes have been proposed to increase energy absorption capabilities of a structure by mechanical means. These schemes have generally involved the use of physical dashpots (viscous dampers) or the use of frictional dampers. These are sometimes proposed for use in conjunction with the mechanical isolation devices mentioned previously. The use of mechanical damping (and isolation) devices has one disadvantage that is often cited; the requirement for periodic maintenance during the life of the structure - and this maintenance could be easily forgotten after a few years or after a change in ownership of the building.

The use of isolation devices and the various means of increased damping is an intriguing concept and certainly warrants future research. As of yet, the full effectiveness of this approach and the practicability of such devices has not been demonstrated. However, the effects of these types of devices on the dynamic response of structures can be determined by employing advanced analytical techniques. A series of proper analytical studies, which include vertical and horizontal motion response, etc., coupled with detailed designs of such devices, would be a significant contribution to the field of earthquake engineering. The results should be confirmed with vibration testing of models that are as large as possible.

B. Effects of Nonstructural Elements

The walls of a building may be of many types and configurations. They may be part of the engineering design of the structure or they may be architectural or nonstructural in nature. In many cases, the walls of a structure are included in the design of the building to resist lateral forces induced by wind or earthquake. Such walls are generally called shear walls and may also serve a load bearing function. Other types of walls are called filler walls (or infilled walls or panels), or partitions. These walls are usually constructed of reinforced or unreinforced concrete block, clay tile, brick, wood or metal, and are not designed as an integral part of the structural system of the building.

Filler walls and partitions, even in the cases where they are nonstructural can have a considerable influence on the lateral response of a structure. Investigations of high-rise buildings and four-story test structures [8, 9, 30, 40] have shown that the presence of filler walls and partitions must be considered in the analyses of structures for lateral response. In these investigations, it has been shown that the presence of filler panels can have a significant effect on the frequency characteristics and dynamic response of a structure.

Investigations of response data from high rise structures from the San Fernando earthquake have shown marked changes in the response characteristics of structures at certain times during the response. This can be partially attributed to the changing characteristics of partition-structure interaction during the response time-history.

One particularly vexing problem that often occurs with filler walls and partitions is that they may have only limited strength relative to the frame of the structure. That is, they may fail at some point in the loading time history, and the frames and any remaining walls must then resist the entire lateral force. An additional problem -- sometimes created by lax construction practices and techniques -- is incomplete connection between the wall and frame. In these cases, mortar or grout may be of low strength and reinforcement, if any exists, may not provide continuity between wall and frame. Also, prior motion of the building due to wind or ground motions may have damaged the wall to the extent that the connections between the wall and frame are partially destroyed. These effects have been shown to be quite important in determining the response of a structure [40].

In addition to nonstructural partitions and filler panels, exterior pre-cast concrete fascia panels can significantly effect the response of a structure. These panels may in come cases be more significant than interior partitions. They are often designed with sliding or deformable joints so that they do not inhibit the movement of the frame structure. However, if the joints are not properly designed, or if they should bind during seismic motions, the pre-cast walls may greatly increase the stiffness of the structure. This can result in increased seismic forces imparted to the structure or a redistribution of forces, which in turn could decrease the safety of the structure. In addition, the panels or connections themselves could fail, which could present a serious hazard to any persons or property in the areas adjacent to the building [63b].

It should be pointed out that damage to partitions and exterior panels and glass can represent a serious economic loss, since these items may represent a significant portion of the total cost of the structure, and, in fact, may cost more than the structural frame.

C. Site Soil Effects on Structure Response

The term soil-structure interaction is often used to designate a number of different phenomena; modification of the site ground motion, interaction of the soil-structure system, or effect of foundation soil deformation due to dynamic loading on building response. The modification of site ground motion was covered under the review article Earthquake Hazards for Buildings. The other effects will now be examined.

Since most structures are supported on soil or rock formations and since ground motion waves are transmitted to structures through the underlying foundation materials, it is reasonable to assume that structure response to ground motion is affected by local soil or rock conditions. Numerous observations concerning the influence of local soil conditions on structure damage caused by earthquakes have been documented - starting with the 1906 San Francisco earthquake. In addition, some pertinent information is available from studies made during underground nuclear explosion experiments.

From studies of these observations, it is possible to identify several important factors which have a direct effect on structure response and are affected by local soil conditions:

- Site ground motion modification (Review article, Earthquake Hazards for Buildings),
- Stiffness characteristics of foundation soils,
- Dynamic response characteristics of structure,

- Dynamic soil consolidation potential,

- Liquefaction potential of site soils, and

- Slope stability.

The effects of stiffness characteristics of foundation soils were noted as early as the 1906 San Francisco earthquake. Many buildings on bay fill suffered much more extensive damage than those on hard rock. At Kanto, Japan, in 1923, rigid buildings founded on rock were more severely damaged than those on deep alluvium. In the 1957 San Francisco earthquake, local soil conditions both amplified as well as attenuated ground motions.

The dynamic response characteristics of structures appear to be affected by the site soil conditions. As noted at Kanto, rigid buildings on hard rock were damaged more severely than those founded on alluvium. This could be explained on the basis that the ground motion intensity in the alluvium was less for periods coincident with the periods of the rigid buildings on alluvium than for rigid buildings on rock. It is also possible that the soil-structure system damping was greater in the softer soils.

Recent studies by Scholl [56], Seed [59] and others lend credence to another explanation - the structure response is dependent on the relative periods of the soil and structure, and on the damping of the soil-structure system. Observations of response to ground motion induced by underground nuclear detonations by Blume [7] and Scholl [57] substantiate this hypothesis. Figure 8 is a comparison of response spectra calculated for motions recorded free field and in the basement of high-rise structures. In these cases the stiffnesses of the sites were greatly different than those on the buildings. It is of interest to note that the Imperial Hotel in Tokyo designed by Frank Lloyd Wright and completed in 1921, was designed to "float" during an earthquake; the building was much stiffer than the underlying soils. The hotel withstood the 1923 Kanto earthquake with essentially no damage. There are also a number of theoretical studies of soil-structure response [32, 38, 56, 69]. In summary, there are empirical evidence and theoretical considerations to justify the following conclusions:

- Local soil conditions do effect the dynamic response of structures,

- The amount of the effect is primarily a function of the frequency content of the ground motion and the relative stiffnesses of the soil and structure,

- In many cases local soil conditions may be more important to low rise buildings than for high rise structures.

Three other effects of site soil conditions should also be briefly considered: the potential for dynamic soil consolidation, liquefaction, and slope stability. Considerable damage has been caused by these factors; San Fernando (1971), Niigata, Japan (1963), and Fukui, Japan (1948) produced numerous examples of damage.

Observations of dynamic soil consolidation caused by earthquake vibratory loads have been made on numerous occasions. In addition, vibratory loading is used to improve building sites by compacting loosely consolidated sands. The results of field work and field and lab testing show that the degree of consolidation is a function of loading intensity, frequency and duration, and of the characteristics of the soil, i.e., whether it is cohesionless or cohesive soil. The consolidation of cohesionless soils is also a function of their relative density and the confining pressure.

The effects of liquefaction were perhaps most spectacular at Niigata. Liquefaction is a "quick" or liquid condition developed in cohesionless soils

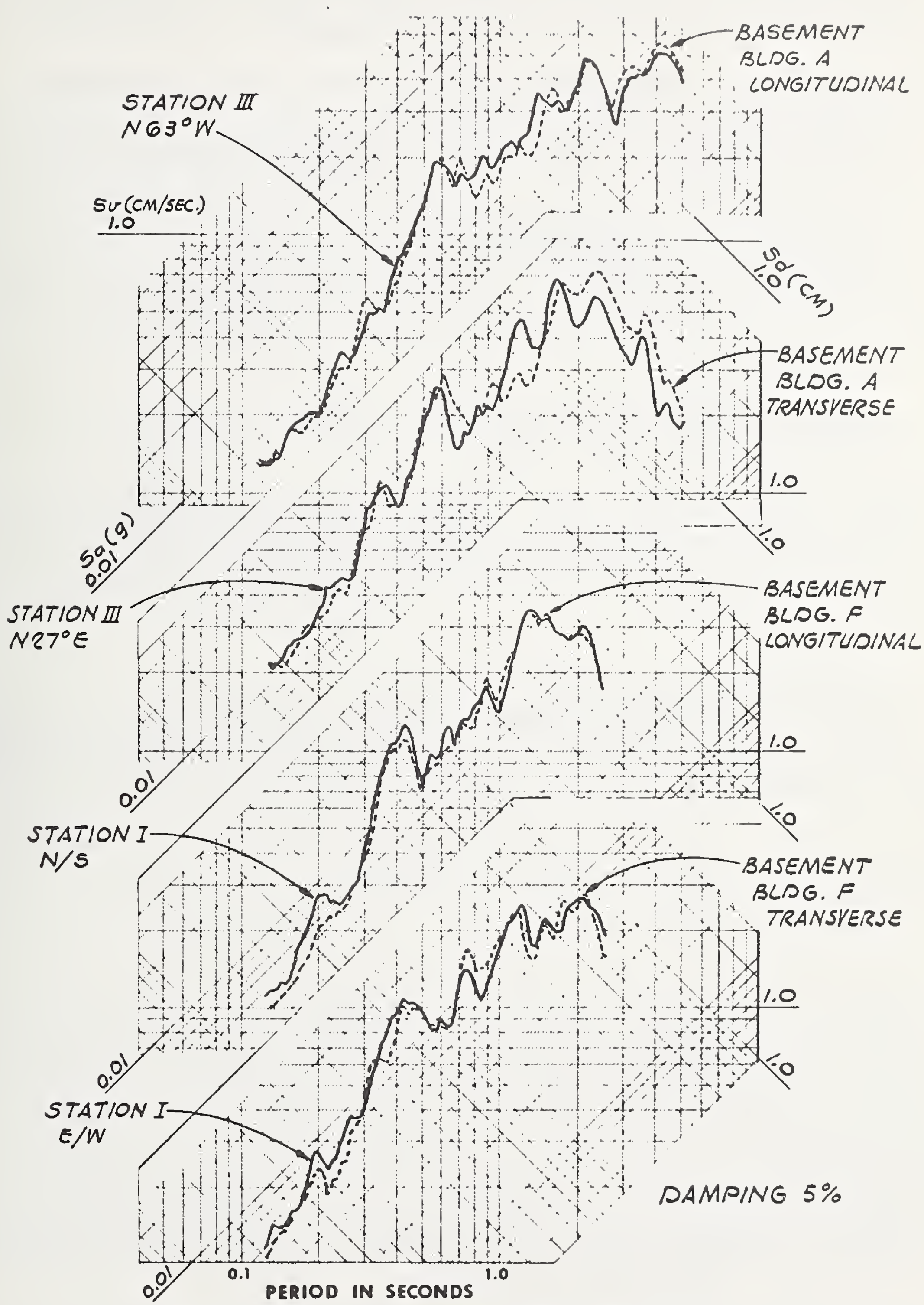

FIGURE 8 COMPARISON OF BASEMENT AND "FREE FIELD" RESPONSE SPECTRA, EVENT BENHAM

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under cyclic loading - excess pore pressure decreases the intergranular pressure to the point where the material has zero strength. Field tests by Florin and Ivanor [29] showed that liquefaction could be produced by blasting. Seed and Idriss [60] and Kishida [39] described methods for evaluating liquefaction potential. The required conditions include:

- Cohesionless soil, uniformly graded - sand is more susceptible to liquefaction than gravel,
- Relative density - loose sands have higher liquefaction potential than dense sands,
- Initial confining pressure - liquefaction potential decreases with increasing confining pressure,
- Ground motion intensity - liquefaction potential increases with increasing intensity,
- Duration of ground motion - liquefaction potential increases with increased duration,
- Depth of water table - liquefaction potential decreases with increased depth to water table.

Another factor that should be evaluated is the potential for <u>slope instability</u>. The Turnagain Heights failure (Alaska, 1964) [61] is perhaps one of the most spectacular. Slope failures also occurred at San Fernando and in other earthquakes [58]. Approximate quantitative evaluations of slope stability can be made based on proper soils engineering investigation, experience and judgment.

D. Vertical Acceleration Effects

Vertical accelerations currently are given little or no consideration in the design of conventional structures. However, there may be instances where vertical accelerations could contribute to the failure of a structure under seismic conditions. Vertical accelerations could contribute to the compressive failure of a column, reduce the factor of safety against overturning, or cause a critical condition in prestressed beams by reducing the effective dead load on the beam. Vertical motions have also contributed to the collapse or severe damage to marquees or entrance canopies.

Investigations of the acceleration - time history records of a number of recorded earthquakes [41] have shown that peak vertical accelerations ranged from 0.36 to 0.67 times the peak horizontal accelerations. The Parkfield earthquake was of particular interest in this respect as accelerations were recorded at varying distances from the San Andreas fault trace. The vertical and horizontal accelerations decreased rapidly as the distance from the fault increased. This rapid decrease in accelerations is important because accelerograms for the other major earthquakes have been recorded at distances from 7 to 30 miles from the fault trace. Thus structures close to the moving fault or epicenter may be subjected to higher ground motions than indicated by most records. It appears that vertical accelerations close to the fault may be much higher than those shown on available records. This observation is also confirmed by experience with underground nuclear explosions. In this case, for locations relatively close to the point of detonation, vertical accelerations are a significant percent of the horizontal accelerations, and in some cases exceed the horizontal accelerations.

Investigation of the horizontal and vertical records for the Olympia, Taft, and Golden Gate Park earthquakes indicate that while the peak vertical accelerations may not occur at exactly the same time as the peak horizontal accelerations, major vertical motions do occur within the same general time as the major horizontal accelerations. Thus the vertical and horizontal accelerations could be conservatively assumed to act simultaneously. This assumption is generally applied in the earthquake design of nuclear power plants and other critical structures.

Single-degree-of-freedom response acceleration spectra computed from the vertical acceleration time-history records for the Olympia, Taft, El Centro, and Golden Gate Park earthquakes show significant amplification in the low period (or high frequency) range [41]. The spectra from Taft show amplification in the period range 0.05 to 0.5 seconds, and all of the vertical spectra obtained show amplification in the 0.05 to 0.25 range. The El Centro spectra show a definite attenuation of the long period components. It is extremely difficult to generalize as to the shape of the vertical spectra versus the shape of the horizontal spectra on the basis of the few records investigated. It is reasonable to assume that the shape will be a function of distance to the fault, type of fault, intervening geologic structure, soil type, etc. In general, the vertical spectra show an attenuation of the long period components of the earthquake records.

Numerous analyses of structures in the vertical direction have been performed to determine their natural frequencies, mode shapes, and dynamic response [41]. In almost all cases it was found that buildings are not rigid in the vertical direction as is commonly assumed. They are, of course, much stiffer in the vertical direction than in the horizontal direction. As an example, the fundamental period of vibration in the vertical direction of a 51-story structure was calculated to be 0.60 sec., and for a 24-story building was 0.32 sec. It is important to note that the vertical periods were within the range of amplification indicated by the response spectra developed from the vertical earthquake records.

The dynamic response of the structures to input vertical motion was also determined. It was found that the peak vertical acceleration at the top of the 24-story structure was 0.42g for the Taft earthquake, which is 3.4 times the peak ground acceleration. The peak vertical acceleration at the top of the 51-story structure was 0.38g for the same earthquake, which is 3.1 times the peak ground motion. It should be emphasized that these amplifications are due to axial deformations of columns only. No response of beams or slabs was included.

Examination of records obtained during the 1957 San Francisco and 1971 San Fernando earthquake bear out the above results. Amplifications vertical accelerations were recorded during the 1957 San Francisco event varying from 3 at the 15th floor of one building to 5.67 at the 14th floor of another building [17a].

During the San Fernando earthquake two nearly identical seven-story reinforced concrete buildings [12] had peak recorded vertical accelerations of 0.18g and 0.086g at the ground floor and 0.24g and 0.140g respectively at the eighth level (roof). A twelve-story reinforced concrete moment resisting frame building had 0.108g peak vertical acceleration at ground level versus 0.150g at the 13th level (roof). For these three structures the amplification of vertical motion was in the order of 1.5. However, a 21-story reinforced concrete moment resisting frame had peak vertical accelerations of 0.087g at ground floor and 0.260g at the 20th floor - an amplification of 3. Similarly, a 32-story moment resisting steel frame building showed an amplification of 3.8 (0.059g at ground floor and 0.224g at roof). It is therefore evident that significant magnification of vertical accelerations does occur in structures. The commonly held opinion that structures are rigid in the vertical direction is not valid; they are very stiff but are not rigid.

In conclusion, it should be noted that amplification of vertical ground motion in a structure may contribute to the failure or overstress of a structure under seismic conditions. Vertical accelerations could increase or decrease the load on columns during an earthquake by 20% or more. This may be critical when combined with the loads from the lateral response of the structure. Vertical accelerations therefore could contribute to the compressive failure of a column or reduce the factor of safety against overturning.

E. Effects of Aging and Prior Loadings of Materials

Aging of some materials can increase their strength up to a point - concrete is an example. Conversely, concrete or masonry structure strength can be reduced by cracking due to shrinkage, thermal cycles or differential settlement. Steel frames, on the other hand, should maintain a relatively constant strength. However, the usual greater flexibility of steel frames may make their exterior and interior walls more susceptible to damage from dynamic loadings.

Prior loading of a structure can in some cases significantly affect its response. It has been observed that if a concrete structure has been deformed beyond the yield limit, a significant change in period of vibration can result. This has been attributed to cracking of concrete in the tension zones and the resulting more flexible cracked sections. Interestingly enough, it has been determined in certain test structures that the fundamental period of vibration of a structure, after lengthening over a period of time due to loading, has actually decreased after a period during which the structure was not loaded. This could perhaps be attributed to "cementing" of the cracks due to dust, wind, and rain.

A structure with filler panels or partitions may have quite different physical characteristics before and after loading. The panels or partitions may have only limited strength relative to the frame of the structure. That is, they may fail at some point in the loading time history, and the frames and remaining panels and partitions must then resist the entire lateral force. This phenomenon can represent a significant change in the physical properties and corresponding response characteristics of a structure [12].

F. Effects of Variations in Parameters

Regardless of what analytical techniques or procedures are used in the seismic analysis and design of structures, it should be recognized that none of the procedures are exact. Uncertainties always exist. These uncertainties, which are in the parameters used in the analyses, can be categorized as input motion, mass, damping, and stiffness - all of which can influence the dynamic response of a structure.

Uncertainties in the input motion can in part be treated by the use of relatively smooth response spectra for a site, as opposed to the rough or jagged spectra from historic earthquakes. The use of the historic time history records may result in significant changes in response resulting from small changes in structure frequency. Therefore, if time history analyses are performed, time histories should be developed which produce relatively smooth spectra that closely match the spectra postulated for the site. Procedures are currently in use to develop such time histories [46, 68]. The range of effects of local soil conditions on the input motion should also be considered.

The mass of the structure is perhaps the parameter that can be most accurately estimated. A reasonable variation from the calculated values might be plus or minus ten percent.

As previously discussed, the damping in a structure is subject to much conjecture and discussion. However, one point remains clear: the damping values should be selected conservatively.

Variations in the stiffness of the structure will be a function of the complexity of the structure and the modeling techniques used. The stiffness of a simple framed structure can be determined quite accurately, whereas it may be very difficult to determine the stiffness of a complex frame or shear wall structure. Generally, the stiffness of a structural system is assumed to be constant with time for the specified loading. In cases where material or geometric nonlinearities exist, this assumption is not justified and may lead to inaccurate results. Some sources of nonlinearity which produce a change in stiffness are

- Once a structure reaches the yield stress level, the stiffness to resist further loading is greatly reduced.
- Once a structure undergoes large displacements, linear theory based on the assumption of small displacements is not valid. As a consequence, loads acting on the structure should be applied to the deformed structure. In building structures, this phenomenon is often referred to as the P-delta effect, and is often interpreted as reduction of stiffness.
- Another factor influencing the stiffness of a structure is the contribution of the non-structural components (partitions, windows, etc.).
- The nonlinear stiffness properties of the soil materials underlying the structure may influence the overall structure response.

In addition to the uncertainties in the analysis, the site soil effects as previously mentioned may be significant in the design of relatively stiff structures. For tall flexible structures the soil effects often are less significant.

The designer must recognize the above possible sources of uncertainties in the analyses and must incorporate in his design the effects of these uncertainties on the response of the structure. Seismic analysis is not an analysis with one set of input parameters, but requires a combination of analysis and experienced engineering judgment.

G. P-Delta Effects

P-Delta effects are generally defined as those effects that are attributed to changes in physical characteristics when a structure is subjected to large displacements. For linear elastic structures which undergo small displacements, all loads can be applied to the undeformed structure without introducing significant errors. With large displacements, it is necessary to apply the loads to the deformed structure. For a typical rectangular building frame, the gravity loads have no effect on the overturning moment of the undeformed structure. In the deformed structure, however, the overturning effects due to the gravity loads become increasingly more important as the story displacements increase. Because such large displacements are generally not possible without at least some members undergoing inelastic deformations, the P-delta effect will generally be negligible for buildings responding in the linear elastic range.

A formulation of the P-delta effect, which is sufficiently accurate for most practical purposes [15] can be derived for a typical column element in the displaced configuration, which leads to the so-called geometric stiffness matrix. In this stiffness matrix, certain terms associated with the axial force in the member have a reduction effect on the member stiffnesses.

The essential features of the P-delta effects may be summarized as follows: As a consequence of the geometric element stiffnesses, the total structure stiffness is reduced. With increasing axial column forces and increasing displacements, the frame may become unstable and buckling may result. Moreover, with increasing axial forces, the yield capacities of column members are reduced, resulting in much earlier formation of plastic hinges and hence further weakening of the structure.

Without further elastic and inelastic parametric studies, it is difficult to predict the net change on the dynamic behavior of a structure that might result if the P-delta effect is considered. The relative significance of many P-delta effects have not yet been assessed, either in respect to each other or in respect to other response parameters.

H. Drift Control

It is often impractical and ineffective to attempt to control the seismic performance of a structure primarily by regulation of stress. Equally, if not more important is distortion control, which is essential to preventing both structural and nonstructural earthquake damage. If distortion is adequately controlled, then ductility requirements are usually satisfied, P-delta effects become insignificant, and damage potential is reduced considerably.

Distortion is generally measured in terms of interstory drift coefficients. These coefficients are defined as the maximum relative lateral movements between adjacent floor levels divided by the appropriate story height; interstory drift limitations are seldom specified in static codes. In codes where they are referenced, drift limitations ranging from 0.002 to 0.005 are usually specified. Whatever drift limitation is set, it should, of course, be consistent with the level of the prescribed forces. One reason that drift control has not been defined in codes is that suitable envelopes of design earthquakes and resulting responses have not been defined. Code lateral forces greatly underestimate the level of forces anticipated during a great earthquake. Therefore, it is important to predict the order of real distortions during periods of strong ground shaking. In lieu of computing the dynamic response of a structure to gain such knowledge, it has been proposed that another static force formula be devised to represent a more realistic force level that might be anticipated during a great earthquake. Such a formula could be used for drift computations, assuming an elastic response in all cases.

The need for drift control is considerably greater in structures that rely on inelastic energy dissipation to survive a major earthquake. Many structures in existence today fall into this category. During an earthquake, energy is imparted to a structure, the amount of which is both dependent on the characteristics of the ground shaking and of the structure itself. If an earthquake delivers more energy to a structure than the structure can absorb and ultimately dissipate, the structure will collapse. Without adequate drift control, certain elements within a structure may be called upon to dissipate considerably more energy than other elements even though such other elements may have equal or greater energy dissipation capacities. If the energy demand is too great for these heavily loaded elements and the other elements cannot share or relieve the overload because of geometric or stiffness constraints, then some form of failure will result. The application of consistent uniform drift criteria generally results in energy being dissipated uniformly throughout a structure, and more elements share in the energy dissipation process. Also, ductility demands on any one element are considerably reduced.

Inelastic drift predictions have been generally based on extrapolations of linearly elastic drift computations and on the non-linear dynamic responses of simple idealized structures. However, it is now possible to calculate inelastic drifts directly using recently developed computer software. Limited studies on several structural systems have indicated that inelastic drifts are, in general, less than those predicted for identical linear elastic systems.

I. Other Code Considerations

Code approaches generally include other considerations such as provisions for horizontal torsion, overturning moments, and special lateral force requirements on parts or portions of structures.

Overall or interstory planwise rotations of a structure result in additional lateral forces that are induced in the lateral force resisting elements. This twisting, or torsional, motion is attributed to non-coincidence of the centers of mass and rigidity of the stories of the structure as a whole. Most present-day codes now require that such torsion be analyzed and the resulting torsional forces distributed to lateral force resisting elements in proportion to their rigidities. Frequently, a minimum torsional eccentricity is specified.

The overturning moments as computed by some code formulae include a reduction factor that supposedly accounts for higher mode participation. Recent studies and events have led to the elimination of this reduction factor in many codes. Incremental changes in the prescribed overturning moment at any level are generally required to be distributed to the various resisting elements.

Certain portions or elements within a structure are often exposed to much larger force levels during an earthquake than the force level associated with the structure as a whole because a resonance condition between the basic structure and such elements is induced. Whiplash at the top of a structure is evidence of this phenomenon. Parapets, appendages, partitions, chimneys, tanks, equipment, etc., are generally classified as requiring special consideration under seismic loads. A more comprehensive discussion of equipment analysis is given in Section V.

J. Earthquake Recording Instrumentation

Awareness of the need to have more and higher quality data on actual earthquake ground and building motions has resulted in the requirement of several codes that strong motion seismographs be installed within buildings. By analyzing records of building response to strong ground shaking, seismologists, engineers, and code-writers can gain a more meaningful understanding of the seismic problem as related to structural response.

V. Equipment Supported in Structures

Equipment supported in building structures may consist of conventional mechanical and electrical equipment - heating and air conditioning, lighting and control panels - or it may consist of more exotic and valuable items such as computer systems. Presently, little or no consideration is given to the seismic design of such equipment. Conventional codes, such as the UBC, can be construed to apply to equipment by using the horizontal force factor, Cp, for ". . . parts or portions of buildings . . .," although no specific mention of equipment is made. Thus, this portion of the seismic design is left solely to the discretion of the individual designer.

If no provisions are made for seismic resistance of the equipment, then the equipment may be damaged or rendered inoperable if an earthquake should occur. This could result in injury or loss of life, or an economic loss due to inoperability of the facility and the costs of repairs. Examples of damage to equipment resulting from the San Fernando earthquake are shown in Figures 9, 10 and 11. Much was learned regarding equipment design from the San Fernando earthquake [63]:

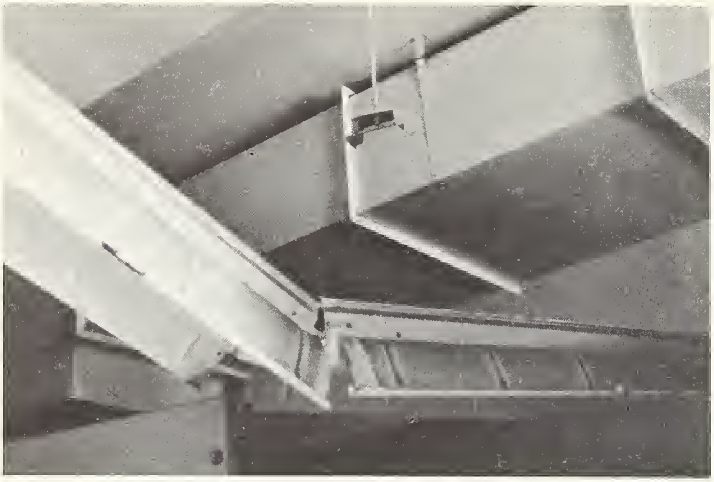

FIGURE 9. DAMAGE TO A PENDANT-TYPE LIGHT FIXTURE. ALTHOUGH HORIZONTAL MOTION WAS UNDOUBTEDLY SEVERE, VERTICAL EARTHQUAKE FORCES MAY HAVE PLAYED A SIGNIFICANT PART IN CAUSING THIS FIXTURE TO PULL AWAY FROM ITS SUPPORTING BRACKET. MANY LIGHTING FIXTURES ATTACHED TO SUSPENDED CEILINGS OR HUNG ON RODS OR WIRES FROM FLOORS AND ROOFS WERE OBSERVED DAMAGED. DAMAGE NOT ONLY RESULTED IN MALFUNCTION OF THE FIXTURES BUT ALSO PRESENTED A HAZARD TO PERSONNEL.

FIGURE 10. THIS SUSPENDED CEILING, LOCATED IN THE CONTROL ROOM OF A LARGE ELECTRICAL POWER FACILITY, COLLAPSED DURING THE EARTHQUAKE, INJURING ONE MAN. WHILE LITTLE DAMAGE TO CONTROL CONSOLES AND EQUIPMENT WAS REPORTED, THE FALLEN CEILING HINDERED EMERGENCY OPERATIONS.

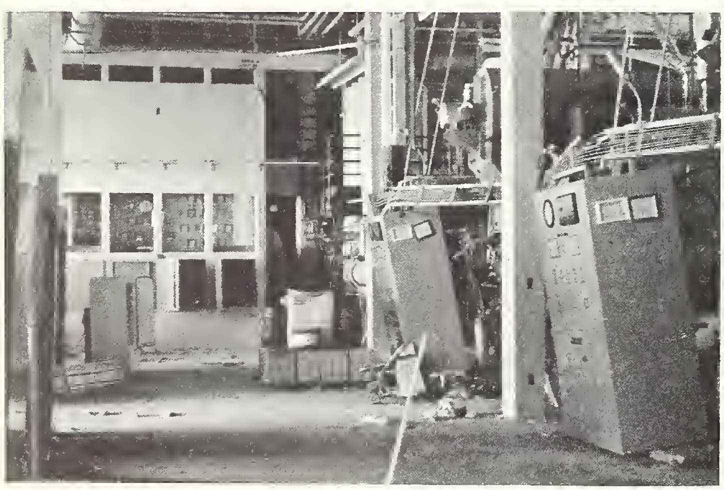

FIGURE 11. DURING THE EARTHQUAKE THESE BOILER CONTROL PANELS WERE DAMAGED WHEN STRUCK BY SLIDING BOILER UNITS. OTHER PANELS, INSTALLED WITHOUT ADEQUATE ANCHORAGE, OVERTURNED OR SLID ON THEIR FOUNDATIONS.

- Mechanical and electrical equipment, and their supports and connections, must be designed to withstand realistic seismic forces if sliding or overturning of equipment, disruption of utility service, or other possible costly damage is to be avoided or minimized.

- Suspended ceilings, light fixtures, and other "non-structural" elements must be designed to resist seismic forces without collapse.

- Proper seismic design should consider the simultaneous effects of both horizontal and vertical earthquake forces.

- Elevator equipment and emergency power supply systems should be given particularly detailed treatment in seismic design.

- For equipment located in structures, the possibility of damage resulting from the interaction of the structure and equipment should be considered. Components of large diameter piping systems, especially pipe hangers, that cannot accept the loads imposed on them as a result of interstory building displacements are particularly vulnerable to this type of damage.

- Equipment having supporting members and connections of brittle materials, such as insulating porcelain, are especially susceptible to damage.

From the above, it is evident that equipment supported in a structure must receive a rational seismic design. One approach to the seismic design is to estimate the peak horizontal acceleration of the floor on which the equipment is suspended and then design the equipment for this acceleration applied as a static coefficient. This approach is valid provided that the means used to estimate the building acceleration is rational, and provided that the item of equipment itself is rigid, that is, it does not amplify the floor motion due to its own dynamic response.

Where the equipment is flexible and is supported by a flexible structure, the equipment-structure interaction becomes a critical factor and must be considered to ensure an adequate design. For a solution to this problem, it is desirable to draw on procedures developed for the seismic design of critical piping and equipment supported in nuclear power plant structures [14, 63a]. These procedures account for equipment-structure interaction and provide a rational basis for design. These procedures are summarized in the following paragraphs.

If the mass of the equipment is significantly less than the mass of the floor on which the equipment is supported (as often the case), then the equipment and structure can be uncoupled. That is, the equipment and structure can be analyzed as separate items. An analysis of the supporting structure can be made by either the time-history, modal superposition method or the direct integration method to determine the time histories of motion at the various floor levels of the structure. These time histories can then be used to compute single-degree-of-freedom damped response spectra for each of the floor levels. Examples of the mathematical model for a reactor containment and resulting response spectra are shown in Figure 12 [63a]. Once the response spectra have been determined in this manner, it is then possible to design the equipment to resist the appropriate seismic forces. Several approaches to accomplish this can be used: approximate analyses, detailed analyses, and testing.

Approximate analyses can be performed as follows. Regardless of the natural frequencies of the equipment, the equipment can be designed for a static coefficient equal to some factor, say 1.5, times the peak of the floor

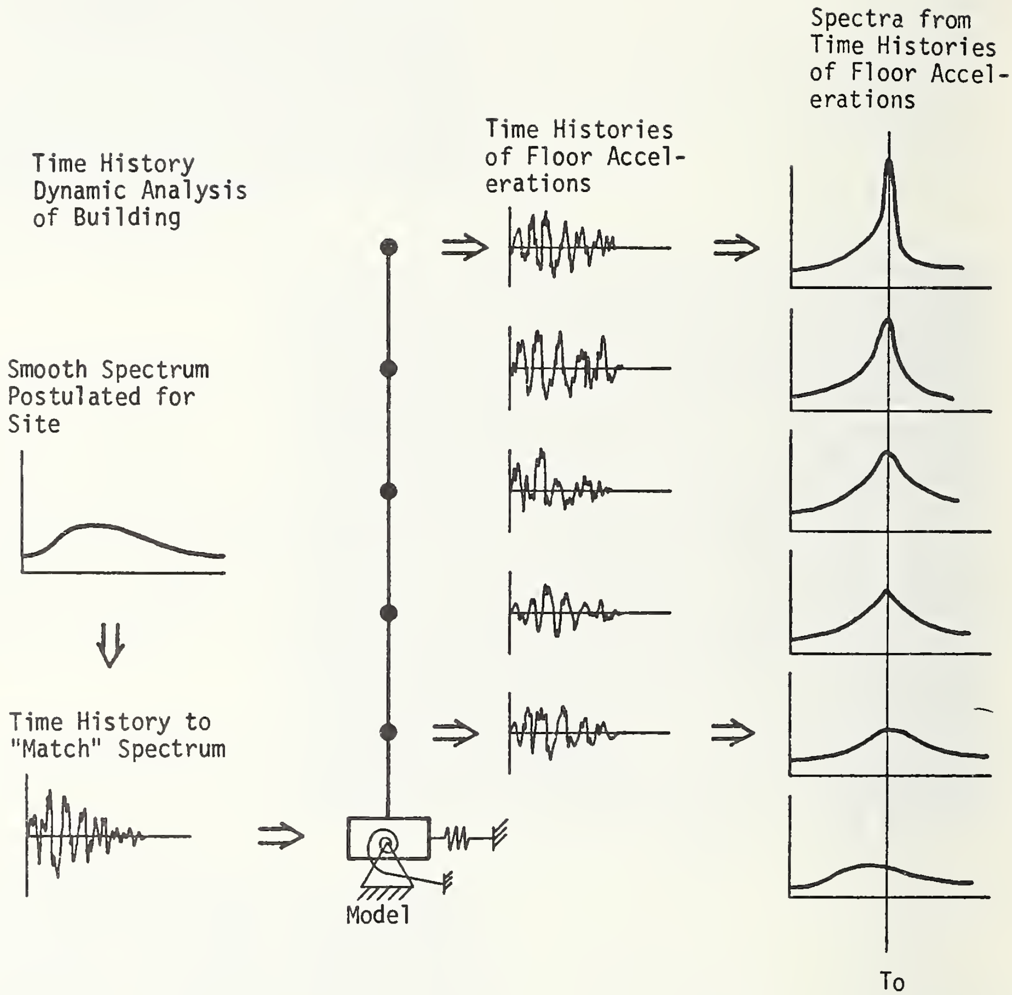

FIGURE 12. SEISMIC ANALYSIS STEPS

response spectrum. This will ensure a safe design. This approach has the advantage that it is not necessary to compute any equipment frequencies, but the disadvantage is that the seismic loads may be quite high in some cases. This of course is not a significant disadvantage if the equipment has high inherent strength.

If it is not elected to perform approximate analyses, then the engineer must resort to detailed dynamic analysis procedures. The first step in such an analysis is to compute the natural frequencies (and mode shapes) of the equipment. If the equipment is quite rigid (with a fundamental natural frequency higher than, say, 25 to 30 cps), it can be designed for a static coefficient equal to the peak acceleration of its supports. If the equipment is flexible, then the dynamic response can be obtained by the response spectrum, modal superposition method.

If the mass of the flexible equipment is not significantly less than the mass of the floor on which it is mounted, then the equipment and structure must be analysed together as a coupled system. This approach is usually followed in the analysis of the nuclear steam supply system and supporting structure in a nuclear plant, but most likely would not be required for equipment in conventional building structures.

In lieu of the aforementioned analytical methods of ensuring seismic adequacy, testing of equipment may be substituted. The basic requirement of any testing program is that the testing machine should be capable of simulating the levels of response indicated by the response spectra applicable at the points of support of the item to be tested.

A number of special precautions warranted in the seismic design of equipment are listed in Table 2 [63]. The application of the above approaches described in this section, including approximate analyses, detailed analyses, or testing, provide a rational basis for ensuring the seismic adequacy of equipment supported in buildings.

Table 2

SPECIAL PRECAUTIONS AND RECOMMENDATIONS

1. Design Equipment Supports for Toughness and Ductility. Support materials (such as mild steel) and support arrangements should be designed to provide tough performance. Brackets, anchors, etc., should be ductile so that they can bend and not break. They can thus continue to carry load and absorb energy.

   In special situations, non-ductile materials such as insulating porcelain must be used to support equipment, especially electrical substation equipment. Because of the lack of ductility these materials exhibit, the suggested coefficients may not be large enough. Dynamic analysis can be employed to determine forces and displacements, or design loads can be increased. Some means of attenuating the seismic motion may also be needed to protect the equipment.

2. Connections are Important. Many engineers consider it good seismic design practice to balance member strength with connections as strong as the member itself or to provide connections that bend or give rather than break.

3. Consider the Interaction of Piping and Structure. Pipes attached to relatively massive, rigid appurtenances and structures can be broken or damaged by differential displacements due to foundation rocking or

differential foundation settlements or displacements caused by vibration-induced soil consolidation.

Distress to piping and hangers can also result where a stiff piping system tends to resist the sideway of the more flexible structure to which it is attached. The stiff piping may tend to act as a lateral force resisting element of the flexible structure - something piping is generally not designed to do.

4. Avoid Resonance Problems - Mount Equipment Rigidly. Equipment flexibly mounted in buildings or structures can have its response to earthquake motion considerably amplified. When the period of vibration of the equipment system is approximately the same as one of the predominant periods of vibration of the building, amplification of 10 times the building response or even greater can result. Possible resonance problems can be avoided by securing or bracing equipment rigidly.

5. Consider the Effect of Interstory Building Displacements. Equipment secured between two floors of a building may be forced to accept large interstory displacements under earthquake motion. This situation can result in distress to equipment and supports, particularly when the equipment system is rigid in comparison to the building and tends to act as a lateral force resisting element of the building. Care should be taken by the designer to assure that the equipment supporting system can accommodate realistic interstory displacements, which may be on the order of two to four times greater than those calculated from code seismic forces.

6. An Experienced Engineer Should be Responsible for Seismic Design of Equipment. Seismic design of equipment supports should be accomplished by an engineer familiar with structural analysis techniques and experienced in structural design in seismic areas.

VI. Design Procedures

Design procedures have been covered in some detail in the preceding text (Sections II and III). Present philosophy of building codes in the United States has been discussed by Pinkham. However, it should be noted that the trend of future codes appears to be in direction of requiring, in design, more consideration of all aspects of the earthquake engineering problem: probable earthquake motions at a site or in a given area, effects of geologic structures underlying and surrounding a site, effects of site foundation materials, dynamic characteristics of structures, effects of horizontal and vertical motions, and effects of earthquake-induced motions on nonstructural elements, equipment, and systems. Indications of this trend are evident in the proposed new earthquake code for the City of Los Angeles [19] and in the Position Paper recently prepared by the SEAOC Seismology Committee [67].

Future codes must consider the aspects listed above, but they should be codified and written in a manner that can be understood and utilized by the practicing design professionals and the building officials who will be charged with administering them. The dynamic characteristics of earthquake motions and of the soil-structure systems must be taken into account in building designs. This does not necessarily mean that all buildings need to be designed by dynamic analysis methods. It does mean, however, that recognition must be given to all of the factors involved.

It is hoped that as more is learned about computer analyses and simulation of building response, standardized procedures may be developed. That is

Structures that are relatively uniform and symmetrical in arrangement could be analysed using simpler models than for those more irregular in shape. Some attempts have been made to categorize buildings by structural type - moment resisting ductile space frame, moment resisting space frames, moment resisting - partial shear element buildings, and shear wall structures. Present code provisions tend to recognize the decrease in response of longer period high rise buildings, however, for low rise structures the possibly higher response of flexible framed structures as compared to stiffer shear-type buildings is not recognized in the present codes.

It would seem possible that soil effects for a given city or region could be investigated, correlated, and published. The effects of each type of site, considering construction type and fundamental period of structure, could be determined and codified. Lateral and vertical seismic force factors could then be assigned for typical small buildings. A similar approach could be followed for medium-rise buildings.

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SURVEY AND EVALUATION OF EXISTING BUILDINGS

by

Frank E. McClure*

I. Introduction

Objectives

The objective of this paper is to present the state-of-the-art concerning the "Survey and Evaluation of Existing Buildings."

This review includes:

1. Evaluation of practices in current use and identification of the best current practices.

2. Identification of opportunities and priorities for developing improved practices from documented research findings.

3. Recommendations for high priority research activities to fill gaps in knowledge.

Background

Buildings constructed with earlier building practices require special evaluation of hazards from earthquakes, extreme winds, and similar loadings. In emergency and post emergency situations, evaluations of deteriorated and damaged buildings are particularly needed to determine whether they are safe for immediate use and whether repairs are economically feasible. Screening and detailed evaluation procedures both must be used. Relatively simple screening procedures may be capable of identifying many buildings as either clearly adequate or clearly unduly hazardous. Since strengthening or condemnation is very costly, procedures and criteria for evaluation of questionable buildings may be more refined than those used in conventional design.

The post-disaster period is considered to be the period of emergency relief, recovery and rehabilitation programs. The pre-disaster period is the period after these programs have been completed or the period before the disasters. The pre-disaster period is longer than the post-disaster period. It exists in those areas of the United States which have a potential for natural disasters even if these disasters have not occurred in historic time.

The procedures and criteria for the post-disaster and pre-disaster periods differ because public policies change with the immediacy of the natural disasters.

Work Statement and Scope

The scope of work for this paper includes the following:

1. Review published and unpublished work of others in the survey and evaluation of existing buildings.

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2. Review applicable portions of reports prepared by the author in his pre-disaster and post-disaster studies.

3. Contact engineers, government and military representatives, contractors and others with first-hand experience to compile and review their recommendations on damage surveys and demolition or reconstruction work following disasters caused by earthquakes, extreme winds, and similar loadings. The sampling, though concentrated in the Pacific Coast "earthquake country," will endeavor to include in its scope representative findings in areas of the nation subjected to other destructive forces.

4. Prepare a paper based on the experience of others and the author's previous work leading to recommendations for damage survey and repair procedures, plus conclusions regarding the enforcement of governmental ordinances.

The main subject topics presented in this paper are:

1. Post-disaster procedures for mobilizing inspection teams.

2. Procedures for evaluation of the condition of structures.

3. Procedures for repair and the degree of rehabilitation attainable.

4. Rehabilitation ordinances and their effectiveness.

5. Pre-disaster inspection of existing structures to assess their potential hazard risk and pre-disaster ordinances.

6. Recommendations for Federally sponsored studies on legal controls of rehabilitation.

7. Compilation of a bibliography of available papers and articles on building damage surveys and repair.

Post-disaster procedures and ordinances are discussed first because their pre-disaster counterparts usually develop from the post-disaster experiences. Proper implementation of pre-disaster procedures will mitigate the need for some of the post-disaster abatement programs.

II. Post-Disaster Procedures--Earthquakes

Introduction

Recent natural disasters caused by earthquakes, hurricanes, tornadoes and extreme winds have tested procedures for the mobilization of inspection teams to survey and evaluate existing buildings. The work of surveying and evaluating is usually under the jurisdiction of the local building departments. These inspections are made in two phases. Phase 1 inspections are made during the emergency period immediately following such disasters. Phase 2 inspections after the immediate emergency is past provide the opportunity for more complete evaluation of the condition of the buildings preparatory to actual design of repair and rehabilitation work and the implementation of actual construction.

This section of the paper discusses the experiences of various building departments during the emergency period. Other portions present their procedures for evaluation of the condition of the buildings during the recovery period.

The recent 1971 San Fernando earthquake is a logical starting point in such a review of mobilization procedures. The experiences of building departments in the 1969 Santa Rosa, 1964 Alaska, 1954 Eureka, 1952 Kern County and 1933 Long Beach Earthquakes provide additional background in reviewing the history of mobilization and evaluation plans.

A building department's approach to damage inspection and rehabilitation is usually based on a four-phase program: 1) Immediate survey by inspectors to determine which buildings are damaged and to post those buildings which are unsafe for occupancy, 2) Resurvey by building department engineers or private consulting structural engineers hired by the local jurisdiction to re-evaluate the doubtful buildings and verify those buildings that are posted as unsafe, 3) Withholding occupancy until the owner's private consulting engineer renders a report that the building has been restored to its original vertical and lateral load resistance, or to other criteria developed immediately after the disaster, and 4) Plan-checking and field inspection of the rehabilitation work.

Immediately after the disaster, in the emergency period, the building department's primary mission is to obtain an overall evaluation of the damage to aid the local elected governing bodies in making decisions as to whether or not to request aid under state and Federal Disaster programs. Their next most immediate mission is to survey buildings to determine whether they are safe or unsafe for occupancy. With the advent of state and Federal Disaster programs, building departments are requested to provide statistical data in terms of dollars of damage and the number of buildings and people who have been affected by the disaster. Unfortunately, most building departments have not been adequately prepared to provide the various types and detailed statistical information requested by Federal, state and local agencies. This paper will provide recommendations to overcome this deficiency.

"Unsafe" buildings are often occupied after earthquakes, because additional hardship to disaster areas due to unemployment and cessation of business activity must be minimized. This places an additional burden on the building departments to permit immediate occupancy of shored-up "unsafe" buildings while permanent repairs are worked out.

San Fernando Earthquake

The mobilization of inspection teams for the large jurisdictions, such as the City of Los Angeles and the County of Los Angeles, and the smaller cities were different. The larger jurisdictions usually had pre-disaster emergency plans, which were set forth in Disaster Orders, that were developed from their atomic bomb attack response programs. Some of these disaster plans were more than just "paper plans" and represented actual training and disaster drill exercises, in addition to actual disaster plan implementation under fire, floods, and other similar disasters. Few of the smaller cities had such well-prepared disaster plans. Some had to rely on State Mutual Aid Disaster Programs or plans developed immediately after the earthquake.

The experience of the City of Los Angeles Department of Building and Safety in this earthquake represents the state-of-the-art for the mobilization of inspection forces by a large city with a pre-disaster plan in the event of a moderate earthquake. Immediately after the initial shock, the Department was faced with the realization that there had been an earthquake, but they did not know where the damage was concentrated or the extent of the damage. The Department divided the City into 100 districts and sent inspectors into each district to survey the conditions and report back within three hours. The inspectors were given copies of the "Earthquake Damage Survey Guide [8] prepared by the Earthquake Engineering Research Institute in 1964, which the Department had on hand in accordance with their pre-earthquake disaster plan. Inspectors sent to the heavily damaged areas were able to enter the areas early in the morning but could neither telephone nor leave the areas because of the breakdown in telephone communication (they had no emergency radio communication capability) and because, in many cases, the areas had been barricaded by police. It was not until 12 hours after the earthquake that the Department had an appreciation of the magnitude of the disaster and knew where the damage was the heaviest. Helicopters were used to a limited extent to attempt to identify areas of heavy damage. It is recommended that in future disasters more use be made of helicopters in conjunction with a training program involving the Department's engineers and inspections, so their use could be maximized. Aerial photographs taken at low altitudes are useful in documenting the damage.

The second day the inspectors were sent to specific addresses in disaster areas based on requests for inspection by the owners. By the fourth day there were 2,600 unanswered requests for inspections and it became apparent that simply answering called-in requests for inspections would not be adequate to insure public safety. A house by house, block by block survey was commenced in the heavily damaged areas.

The inspectors were given emergency building inspection forms and unsafe building posters and were told that they were to determine if buildings could be safely occupied or should be vacated and posted "unsafe." They were to make this decision based on their evaluation of the extent and type of damage and the possibility of after shocks which could further damage the buildings. There was no time for a complete write-up of the damage for each damaged building. Within two weeks, all buildings which had been ordered vacated by the initial inspections were re-inspected by the Department Structural Engineers to determine the correctness of the initial decision to vacate unsafe buildings. Many owners who had never experienced an earthquake refused to return to their homes until an inspector assured them it was safe to reoccupy.

The biggest challenge, which was underestimated in the pre-disaster plans, was trying to furnish information and statistics requested by as many as 20 outside agencies. The emergency building inspection forms were not designed to collect information concerning the dollar losses. More than 25,500 buildings were inspected, and since the damage inspection forms were not coded for computer application, all statistics had to be compiled manually. One serious problem developed when the same owner would call back several times requesting an inspection. There were no adequate provisions for keeping an up-to-date record of whether or not the inspection had been made. This resulted in duplicate inspections of many buildings.

The work accomplished was possible only because the telephone service did not break down except in the heavily damaged areas. The Department's efficiency would have been reduced to 10 per cent if the earthquake had been of a greater magnitude and had the telephone service failed. It is recommended that an emergency communication system, probably radio, be established for use by the Department. Without Federal aid the possibility that this will be done is very remote, due to lack of local funds.

Because of a four-day weekend, the Department was able to make all the critical inspections without calling in outside private consulting engineers. At one point, however, the Department was seriously considering, under the State Mutual Aid program, a request to the cities of San Diego and San Francisco for emergency help. Had the earthquake been of greater magnitude, and had such outside help been needed, some plans for the use of inspectors who were not familiar with the local areas should have been prepared in advance and made ready for use. In a great disaster, the local building departments cannot expect too much help from local structural engineers and architects who will be committed to work requested by their clients.

The City of Los Angeles, Department of Building and Safety, has recently made a critical review of their response to the San Fernando earthquake and has made revisions in their 1967 Disaster Orders and their Emergency Inspection Forms. The new forms are formulated for computer analysis. A detailed study of the experience of this Department and the basis for the revised Disaster Orders and their Emergency Forms is strongly recommended.

The County of Los Angeles' experience was similar to that of Los Angeles City. The Building and Safety Division had a standard procedure for inspection and reporting of damage as a result of fire, flood, earthquake or other disaster. The Division had on hand standard report forms for reporting of damage, including provisions for reporting the estimated dollar value of the damage. The information on these report forms was combined with data obtained during the resurveys on computer data cards which could be read directly by the computer without punching another set of data cards. The computer program was flexible so that the information on a particular building could be easily updated, depending on the re-inspections and changed status of demolition or repairs. It provided for the initial inspection as well as re-inspections and the close-out inspection. The computer capability provided for calculation of dollar damage statistics, as well as daily monitoring of the status of each building and the status of the total rehabilitation program. This provided the information which was very helpful for the County Supervisors and others in decision-making positions.

After considerable experience in the preparation and the review of other damage survey forms used in previous earthquakes, the author was most impressed with the computer damage survey data card developed by Los Angeles County. The success of their computer capability in providing immediate information concerning the overall damage as well as constant updating of the data strongly recommends further study of the application of computers in disaster surveys.

The experience of the building departments in smaller cities in the Los Angeles area was typical of similar cities in previous earthquakes. The small building departments do not have sufficient staff to make the necessary inspections and relied on assistance from the State Division of Housing, State Office of Architecture and Construction, and the State Office of Emergency Services. They reported the lack of adequate communications, the lack of unsafe building posters and inspection forms, the need for improved methods of presenting damage statistics to governmental agencies and private insurance and lending agencies, and the need for improved procedures for requiring building demolition. A serious need was to have an up-to-date predisaster written plan with provisions to cover specific disaster situations.

More detailed discussion is presented in References 1 and 2 concerning the pre-disaster plans, response, problems and recommendations of the building departments in the Los Angeles area as a result of the San Fernando earthquake. References 3, 4, 5, 6 and 7 review the experiences of building departments following previous earthquakes.

III. Post-Disaster Procedures - Other Natural Disasters

Introduction

For the purpose of this paper, "other natural disasters" are hurricanes, tornadoes and extreme winds as well as explosions and accidents which produce effects similar to earthquakes. Although the main emphasis in this paper is on earthquake disasters, the procedures and recommendations for earthquake disasters are applicable to the other natural disasters. The author's professional practice has been almost exclusively in the earthquake field; however, he experienced two typhoons while on Okinawa in 1945. Review of the reference material for these other natural disasters points up the similarity of the response of buildings during typhoons and their response to other meteorologically-caused disasters.

Tornadoes, with their whirling winds, are the most violent weather phenomena known to man. During the past 50 years U.S. tornadoes and severe windstorms have killed amost 18,500 persons, while hurricanes and floods have killed approximately one-half that number during the same time period. The hurricane brings devastation by wind, flood-producing rain and the storm surge which is the most lethal. Considering only the damage caused by the wind effects of such disasters, the losses to property are approximately equally divided among ordinary gales, tropical hurricanes, and tornadoes [24].

Procedures

The areal extent of these other natural disasters and their dollar losses are usually so great that detailed post-disaster building investigations ordinarily are not made. The inspections are often undertaken by Federal agencies and their consultants who have responsibilities for the prediction and warning of these disasters. These agencies, obviously, have the best understanding of the mechanisms which cause these disasters, but they seldom have the in-house capability to assess the structural performance of the buildings except in general terms. The literature is very limited in detailed damage statistics by type and quality of construction. Few studies similar to that of K. C. Mehta and A. J. Sanger in 1971 [22] have been made. Most studies for the larger disasters, such as the 1969 Hurricane Camille, [21] have been made by airplane. The basis and detail for the dollar loss statistics customarily are not given in the reports of such disasters. Those Federal agencies which have a responsibility for the review of the performance of materials and the review of the adequacy of building code provisions have not been involved in post-disaster surveys to the extent that they should for adequate disaster mitigation through improved building practices.

Damage Evaluation

Although the author has personally observed only the damage caused by typhoons, a review of the descriptions and photographs of the damage caused by other disasters leads to the conclusion that in many respects the damage caused by earthquakes and the damage caused by these other disasters is quite similar. Unreinforced masonry walls collapse, while reinforced masonry wall construction performs quite well. Wood frame dwellings which are not bolted to the foundations are often destroyed while dwellings with adequate anchorage to the foundations perform well. The degree of damage depends more on the quality of construction than on the magnitude of the tornado and hurricane wind forces [21, 22, 23]. This is not to say that the other disasters subject the buildings to the same type of loadings as do earthquakes, but buildings that perform well in these other disasters have many of the characteristics that are important in earthquake-resistant construction. These other disasters subject the buildings to negative and positive pressures and uplift forces due to negative air pressure gradients. However, a building which is tied together so that it functions as a unit and has a rational system for the resistance of lateral forces, using materials of adequate strength and ductility, will perform well in other wind-caused disasters as well as in earthquakes.

Disaster Mitigation

The present state-of-the-art for disaster mitigation from other disasters is largely based on adequate public response to prediction and warning programs. These programs have been very effective in reducing the loss of life. Until weather modification programs can appreciably reduce the intensity of these other disasters, however, buildings must continue to be subjected to the forces from these disasters. Unfortunately, without complete disaster damage statistics by type and quality of construction, it is generally assumed that the forces from tornadoes, hurricanes, and violent windstorms are so great that few buildings can be economically designed to resist these forces. This is not true and the few structural surveys have been made support the conclusion that improved building practices can mitigate the effects of these other disasters [21, 22, 23].

The recommendations for improved building procedures given in References 21 through 27 to make buildings more wind-resistant are basically the same recommendations for making buildings earthquake-resistant. Engineers experienced in earthquake-resistant design will recognize the similarity in these recommendations, because these recommendations are included in their earthquake-resistant designs almost as a matter of course.

The main impediments to the implementation of these recommendations are inadequate building codes, inadequate enforcement of these codes, and lack of understanding by the local authorities, local architects, engineers and building contractors of the benefits to be derived from the application of the simple principles of lateral force design.

It is recommended that an appropriate Federal agency be given the responsibility to encourage local application of the present state-of-the-knowledge for disaster mitigation through adequately enforced and improved building codes. Federal agencies which have statutory responsibilities for the funding of Federally-supported building projects and the review of the adequacy of construction materials must play a more active role in the investigation of other disasters as well as the testing of building components to resist the forces from these disasters.

IV. Evaluation Procedures

Introduction

During the emergency period, the main question is whether the building is safe or unsafe for occupancy. This decision is usually based on a cursory inspection of the building and an on-the-site judgment whether the building is adequate for vertical loads and lateral forces which might occur prior to the start of repair work. After the emergency period, when there is time for a more complete evaluation of the damage, the evaluation procedures vary according to: 1) The type of construction of the building, 2) The degree and the causes of damage, and 3) The building department's criteria for the degree of rehabilitation required.

Except for the "Earthquake Damage Survey Guide," [8] no publication is generally available covering how to evaluate the damage to buildings caused by lateral forces--wind and earthquakes. Inspectors, who do not have formal training in structural engineering, must rely on their observations of the observable evidence of the strains in the building and make judgments as to the severity of the damage based on their practical knowledge of good construction. Earthquake damage, short of collapse, is not spectacular and it takes an experienced investigator who has inspected previous earthquake damage to detect the subtle indications of such damage. Structural Engineers, who have experience in earthquake-resistant design and who have either made previous earthquake damage inspections or who have read the reports of previous earthquake damage, know where to look for earthquake damage and how to evaluate this damage.

The state-of-the-art of detailed evaluation of damaged buildings is not fully developed for adequate protection of the public because the relative infrequency of major disasters prevents effective updating of the art and because of the limited number of experienced professionals who have made such investigations.

Procedures

Before any detailed evaluation of the damage to individual buildings (except hospitals, emergency relief centers and other buildings with high socio-economic importance) is started by governmental agencies, an inventory of all damaged buildings based on cursory inspections must be compiled. The damaged buildings should be reported on simple, one-page forms or on computer data cards similar to those used by the County of Los Angeles in the San Fernando earthquake. A more detailed report form is required for the second-phase, detailed evaluation of the damage. Although private consulting engineers working directly for building owners will probably start prior to the completion of the inventory of all damaged buildings by governmental agencies, it is recommended that these engineers use the same standard, second-phase report forms provided by these agencies. This procedure will result in a consistent method of reporting the damage by the private engineers and will aid in expediting the approval by the building departments of plans for the repair. This is not to preclude the private engineers from preparing their own detailed evaluation reports in formats reflecting their individual professional practices, but if the standard, second-phase reports were appended, this would lead to a consistent approach and aid the building departments in their evaluation of the private engineers' reports.

The 1964 EERI "Earthquake Damage Survey Guide" [8] and the 1970 Report by the Structural Engineers Association of Northern California, "Emergency Earthquake Damage Survey Procedures and Guidelines for Earthquake Damage Repair" [9] outline detailed procedures for more complete evaluation of the condition of structures. The author contributed to both these reports. Based on a knowledge of the background and thinking that went into these reports, it is recommended that these reports be reviewed in light of the experience in the San Fernando earthquake and that a combined report be prepared. The new report should be broadened to cover all disasters; it should include mobilization plans coordinated with State and Federal Mutual Aid and Disaster plans; it should include the two-phase approach: Phase 1, the emergency phase, to determine the overall extent of damage and the unsafe buildings with more information on the dollar-damage statistics, and Phase 2, the post-emergency phase, to establish procedures for more detailed evaluation of building damage. The two-phase report forms should be developed to provide for both computer and non-computer data analysis similar to the Los Angeles County approach, and the report should be revised to include an appendix which could be quickly read to aid those not familiar with disaster building damage evaluation on what to look for and how to fill out the forms.

The interest and concern of even the most devoted professionals, except those directly involved on a day by day basis in disaster programs, tend to diminish between disasters. Although the Structural Engineers Association of Southern California continues to have a Disaster Study and Report Committee, similar committees have either been abandoned or have not been formed in the other three SEAOC Associations. There is a serious need to establish a cadre of concerned professionals in all professional building construction associations who will annually meet and review their response programs to future disasters.

The space limitations of this paper do not allow a complete discussion of the procedures for full evaluation of the condition of structures after the emergency period. The state-of-the-art of such evaluations is represented by References 8 and 9.

V. Repair Procedures

Introduction

The terms "repair," "rehabilitation," and "reconstruction" are often used interchangeably. According to Webster, "repair" is to put back in good condition after damage; "rehabilitation" is to restore or to put back in good condition; and "reconstruction" is to construct again, rebuild, or make over. For the purposes of this paper and for brevity, the word "repair" is used for all three terms. Strictly speaking from a construction view-point, to "repair" or to "rehabilitate" means to put back or restore what was in the original construction. To "reconstruct" is taken to mean to construct again and to add materials which were not in the original construction.

Little has been written in the American literature and very little emphasis has been given to the engineering criteria for the repair of damaged buildings after disasters. No aspect of structural engineering tests the ingenuity of the Structural Engineer more than does the preparation of a design for the shoring, temporary bracing, and permanent strengthening of an already damaged structure. When a disaster strikes a community, after the emergency period has passed, the primary concern of the community is to restore its economy to its pre-disaster condition. Buildings must be repaired and placed in service as soon as possible and there is little time or concern for the niceties of design or the general philosophy of safety. In many instances, buildings have been repaired using the "patch and paint" approach to cover up the evidence of the damage which precludes the proper investigation of the extent of damage [10].

There are two broad categories of damage: 1) Non-structural and 2) Structural damage. The non-structural damage--the cracked plaster, shattered partitions, dropped ceiling, fallen light fixtures, and displaced mechanical and electrical equipment--is usually repaired as rapidly as possible. In the general confusion after the disaster, there is a loss of control by the building departments and minor repairs are completed without governmental control and inspection. Usually little consideration is given to the remaining structure if it was not seriously damaged [10].

The structural damage might not be evident without the removal of ceilings, plaster fire-protection, and other finishes which can hide the damaged structural elements. It requires a very conscientious structural engineer to require the removal of finish materials so he can make very careful examination of the evidences of damage to the structure. Liberal sampling by means of test cores to establish the strength of concrete or masonry and test specimens to establish the strength of reinforcing is recommended particularly when the engineer did not design the original building or drawings and specifications are no longer available. Without the construction drawings, the engineer must rely on his knowledge of construction practices at the time of construction as well as his engineering evaluation of what the construction that he can see actually represents [9].

Extent of Repair and Reconstruction

One of the most difficult decisions to be made is to what level the building should be repaired. If the building collapsed, there is no problem. If the building is seriously damaged, however, and there are no pre-disaster building code requirements concerning the degree of rehabilitation, what level should the engineer use in his design of the repair? Five possible approaches are:

1. Repair to the extent necessary to provide for stability under vertical loading only.

2. Restore the structure to its original strength. This is usually the highest level attempted by building departments without pre-disaster criteria to the contrary. The argument for this level of strength is that it brings the building up to the legal level of safety under which the building was originally designed. It overlooks the fact that the building was proved weak in a disaster and will probably suffer similar damage in another disaster.

3. Repair the major structural elements to their original level of strength and reinforce the failed members to a higher degree of strength.

4. Reconstruct to the level of strength and safety set by the latest current pre-disaster standards of design and construction being enforced by the building department for new buildings. The structure will probably be stronger, but the design of the reconstruction will not include the new knowledge that may have been developed after the disaster.

5. Reconstruct to the highest level based on the new knowledge or revised post-disaster standards of design and construction. This could result in some delays due to the time to codify the new knowledge while the owner is pressuring the engineer to finish his design and get the building back in operation as soon as possible. This procedure was used by the Venezuelan building authorities by means of provisional standards which were developed soon after the earthquake to guide the engineers responsible for the repairs [10].

Henry Degenkolb was one of the first to present these concepts in the report, "The Venezuela Earthquake, July 29, 1967" [10]. It must be recognized that the owner possibly faces very large repair costs to bring the damaged building up to a safe lateral and vertical force requirement. The 1970 Uniform Building Code has specific provisions concerning the degree of repair which vary with the percentage of the dollar value of the repair to the value of the building. It also has very specific provisions in Volume IV, "Dangerous Buildings," for the abatement or reconstruction of buildings damaged by disasters. After the 1952 Bakersfield earthquake and the 1969 Santa Rosa earthquake, similar pre-earthquake provisions were not enforced because of the potential adverse economic impact on those cities and modifications of the provisions were voted by the local city councils. (See section "Post-Disaster Rehabilitation Ordinances.")

When California public schools are reconstructed under the provisions of the Field Act, the reconstructed schools must be designed and reconstructed to the level of safety provided by the current provisions of the Field Act for new schools. This has been the law since 1933 when the Field Act was first enacted. Hundreds of pre-Field Act schools have been demolished and replaced, however, because - in addition to questions about the schools' adequacy educationally - the cost of reconstruction has been so high compared with the cost of new construction that it was deemed prudent to replace rather than reconstruct.

Methods of Repair

Methods of repair will necessarily vary with the type of construction, the type of structural element being repaired and the degree of damage. In wood or steel structures, where adequate shoring is possible, removal and total replacement of damaged members is a relatively simple procedure. Repair of cracked or spalled concrete members is more difficult, particularly if the reinforcing steel has elongated. Replacement of cast-in-place or pre-cast elements usually involves the breaking out of the members and the repouring and repacking of the same member. According to the 1970 Uniform Building

Code, unreinforced masonry walls are not permitted to be used if they resist seismic forces. Therefore, they must be strengthened by other materials such as reinforced masonry, gunite, or poured concrete.

Strengthening of a structure by the addition of concrete shear walls, unless properly located, can introduce torsion problems where none existed and the building could have a lower total strength than it had before reconstruction. Addition of new lateral and/or vertical load-resisting frames or elements can lead to problems of overloading of existing footings due to additional dead loads and overturning forces from lateral loads. If no soils information is available, soil investigations might be required to review the adequacy of existing and new foundation systems [9].

Degree of Rehabilitation Attainable

The most important influence on the degree of rehabilitation attainable is the ability to repair or reconstruct the building so that the final structure has a rational system for the resistance of the vertical and lateral loads using materials of adequate strength and ductility. A high degree of repair can be achieved in wood-frame and light steel-frame structures. The future behavior of such structures is influenced by whether or not the structural steel frame members and connections have gone into yield and have lost a portion of their ductility. If these members and connections have yielded and some have not been identified and replaced, the degree of rehabilitation is appreciably reduced [10].

Relatively highly shattered concrete members can be almost completely repaired by the introduction of epoxy under pressure into the cracks, by breaking out shattered concrete and replacement of this concrete with drypack epoxy mortar, or the bonding of newly-poured sections to existing concrete by troweled-on epoxy. Repair of concrete members is greatly influenced, however, by the adequacy of the workmanship and the proper use of these special epoxy materials. It is estimated that the pressure epoxy-grouted repair members can attain a strength of 80 per cent of the original members, but this requires continuous inspection including test coring to check the penetration of the epoxy [9].

Gunite (pneumatically applied concrete) is one of the traditional materials used in the repair of damaged concrete members or the introduction of new shear walls. One of the disadvantages of this type of repair is the extreme difficulty of proper placement in areas of congested reinforcing. If the initial preparation and application is properly performed, it is reasonable to expect that members so repaired or added will perform adequately [9].

The repair of damaged buildings is a new, relatively untried art that has generally been neglected by engineers. The need for research in this field is vitally important to the economy and public safety in areas that will experience disasters [10]. The research should include: 1) Testing of members and connections which have been stressed to yield and above to determine the degree of degradation of strength and ductility when subject to repeated above-yield loadings, 2) Testing of repaired members and connections which have first been stressed to yield and/or failure to determine the degree of restoration obtained in terms of strength and ductility, and 3) Testing of various methods of repair for various construction materials to compare the relative degree of restoration achieved. Examples are the testing of various methods of application of epoxy to repair concrete members, the testing of welds to repair cracked or insufficient welding, and the testing of insert connections.

VI. Post-Disaster Rehabilitation Ordinances

Introduction

This section discusses the various ordinances and their enforcement which were in effect prior to specific disasters and ordinances which were passed soon after these disasters, compared with ordinances which were passed after the recovery and redevelopment period. Disasters in the first category of ordinances are the 1933 Long Beach, 1952 Kern County, 1969 Santa Rosa, and 1971 San Fernando earthquakes. Ordinances in the second category were usually passed years after the disasters that focused attention on the need for such ordinances. These latter ordinances and their enforcement are discussed under "Pre-Disaster Procedures and Ordinances."

1933 Long Beach

The California State Legislature was in session on March 10, 1933, when the Long Beach earthquake caused serious damage to many public schools in Southern California. In order to protect the children and their teachers against possible death and injury from unsafe public schools, the Legislature enacted the Field Act [28], effective April 10, 1933, one month after the earthquake. The Field Act vests with the State of California, Office of Architecture and Construction, the authority and responsibility to pass upon plans, specifications, and construction of any new public school building constructed since April 10, 1933, regardless of cost, or if the estimated cost of reconstruction, alteration of, or addition to any public school building exceeds $10,000. <u>The Field Act was not and still is not retroactive</u>. It does not specifically apply to pre-1933 public schools unless they are altered, repaired, or reconstructed. The Field Act requires that any construction must be designed and constructed to provide a level of safety equivalent to new construction. The provisions of the Field Act have been strictly enforced and any violation of the Act is punishable as a felony. School buildings constructed and reconstructed under these provisions have been subjected to the 1940 El Centro, 1952 Kern County, 1969 Santa Rosa, and 1971 San Fernando earthquakes, and these schools have performed very well.

1952 Kern County

The earthquakes of July 22 and August 21, 1952, destroyed or damaged numerous buildings in the City of Bakersfield, California. The City was operating under the 1949 Uniform Building Code which required that if repairs to a building exceed 25 per cent and were less than 50 per cent of the value of the building within any twelve month period, the entire building would not have to be made to comply with the requirements for a new building. If the repairs exceeded 50 per cent of the value of the building within any twelve month period, then the entire building would have to be made to comply with the requirements for a new building. The repair work itself had to comply with the requirements for the building code then enforced. If the repairs were less than 25 per cent, lesser restrictions were required [6, 7].

The City Council passed an ordinance which lessened the code requirements for certain structural features used in the rehabilitation of earthquake-damaged buildings. This ordinance came about through pressures brought to bear upon the City Council by owners, building contractors, engineers and architects to bring an economical solution to rehabilitate earthquake-damaged buildings to a minimum safe condition. The results of this ordinance brought about the strengthening of the damaged buildings to an extent greater than the original structural strength, but not entirely up to the pre-earthquake code. This ordinance allowed the use of existing roof and diaphragms and unreinforced brick shear walls which would have had to be reconstructed under the pre-disaster code.

After rehabilitation was started, however, many of the buildings had additional hidden costs besides the cost of structural repairs - in the replacement of obsolete plumbing, electrical and mechanical features. Many owners would not have gone into the repair work had they known the complete costs for this additional work. Instead they would have demolished the buildings and constructed new ones.

The Building Departments in Kern County deserve a word of praise for the corrective work that was done, compared to the previous "patch and paint" procedures used after earthquakes in other areas. The Kern County earthquakes showed that it was feasible to demolish badly damaged structures and repair others effectively, providing good existing codes are not emasculated. One of the keys to this success in Bakersfield was that a registered Civil Engineer was appointed as Building Official soon after the earthquake and he insisted on enforcing the building code during the reconstruction period [6].

1969 Santa Rosa

On October 1, 1969, Santa Rosa, California, a city of 50,000 population, was damaged by a magnitude 5.6 earthquake, which caused more than 5 million dollars worth of damage to commercial buildings in the downtown area. The situation in Santa Rosa was similar to Bakersfield, in that the City Council faced the dilemma that strict enforcement of the pre-earthquake provisions of the City Building Code - the 1967 Uniform Building Code and the 1967 Dangerous Building Code - would create further economic hardship for property owners, displaced businesses and employees, as well as the general public. The City Council appointed a Board of Consultants to advise the Council on interim emergency and long-term policies concerning the hazardous buildings. On October 9 and 28, and November 4, 1969, the Council adopted ordinances which dealt with the required minimum levels of structural safety for emergency repairs or demolition of buildings. The political policy behind each of these ordinances was that it was in the public interest to have a balance between the interests of the private property owners and consideration for the public safety and welfare of the citizens of Santa Rosa. This policy was not to amend or alter the City Building Code but rather to establish an emergency policy to implement an orderly procedure to handle buildings damaged by the earthquake [13].

The ordinances were quite detailed, and limitations on the length of this paper do not allow discussion of all items in these ordinances. The following are some of the more important ones:

1) No occupancy shall be allowed in a building declared to be unsafe by the Chief Building Official and which is open to and used by the general public until the code requirements for vertical loads are met and immediate hazards such as loose parapets and cornices are repaired or removed,

2) Emergency criteria for use of and bracing of inadequate unreinforced or inadequately reinforced concrete or masonry were given,

3) Owners were given 44 days to comply with the emergency procedures,

4) Provisions for varying degrees of lateral bracing for the resistance of earthquake forces were to be provided at a future date, and

5) A Structural Review Board was established to hear appeals by owners concerning the decisions of the Building Inspection Department and to advise the City Council on what action should be taken.

These 1969 ordinances were a compromise between rather strict enforcement of the pre-disaster City code advocated by the Board of Consultants and amendments to the City code to eliminate the Dangerous Building Code provisions advocated by the property owners. Enforcement of these ordinances was influenced by the retirement, due to poor health, of the building official who was in office at the time of the earthquake, the time interval to find a registered structural engineer replacement, and the usual problems which a small building department faces in attempting to enforce such ordinances. Owners were somewhat reluctant to start work on the older buildings until the details of the degree of lateral bracing were set forth in future City Council resolutions.

Since 1969, the same Board of Consultants has been retained by the City Council to develop criteria for the safety investigations of older buildings within the City and suggestions for the reinforcement requirements for those buildings found to be so substandard as to create an unacceptable level of safety. These requirements are based on considerations of the economic health of the community to develop a balance between the acceptable degree of risk and the cost of reinforcement or demolition. These criteria are set forth in Degenkolb [13] and were adopted by City Resolution 9820, October 12, 1971. Some of the important items in these documents are:

1) Buildings to be reviewed and priority for review,

2) Scope of preliminary investigation, to be conducted by representatives of the City Building Department at public expense,

3) Criteria and scope of further investigation by the property owners and their engineers,

4) Building requirements for continued long-term use of structures,

5) Criteria for occupancy for a term not to exceed five years,

6) Criteria for buildings which fail to meet the requirements under 5 above, which shall be abated within one year,

7) Abatement of buildings which fail to meet the requirements for one-year occupancy under 6 above, and

8) Other criteria for buildings which may be occupied for a period of one to five years.

Time has not been sufficient since enactment of the 1971 ordinances to determine the effect. The recently appointed structural engineer building official hired a structural engineer in mid-1972 to start work on making the preliminary reviews of the buildings. Once this has been completed, the other phases of the work under the 1971 ordinances should progress more rapidly than did the implementation of the requirements of the 1969 ordinances. The 1971 ordinances represent a "break-through" in ordinances for the abatement of hazardous buildings and the City Council and its consultants are to be commended for developing and adopting such ordinances. Their successful implementation can have far-reaching influence on State of California legislature for the abatement of similar buildings in all of California.

1971 San Fernando

The extent of the problems concerning the demolition and degree of repair experienced after the 1952 Bakersfield and 1969 Santa Rosa earthquakes, did not occur after the 1971 San Fernando earthquake except in the City of San Fernando. The San Fernando earthquake was a moderate one, and the percentage of seriously damaged buildings was not as great as in the two previously cited earthquakes.

Building codes enforced in most of the Southern California cities and counties are basically the recent editions of the Uniform Building Code which have adequate provisions for the control of repairs to damaged buildings. The Dangerous Building Code [12] is not always included in local building codes, but there are similar provisions in the City of Los Angeles and the County of Los Angeles Building Codes. It is estimated that there are more than 15,000 old masonry-walled structures in the City of Los Angeles which represent "clear and present danger" in the event of moderate and major earthquakes. Though abatement is clearly provided for by the pre-earthquake building codes of Los Angeles and other areas of Southern California, there is no public policy concerning the abatement of these structures that is acceptable to the political and economic segments of the city.

Repair of damaged structures was generally executed under the plan-checking and inspection procedures of existing codes. In some instances, repair work was started immediately, before a complete investigation of the damaged buildings could be made. This repair work covered up some of the damage. In several buildings, due to the closer surveillance possible after the emergency period, this repair work had to be redone. It must be remembered that this was a moderate earthquake. In the event of a major earthquake, with thousands of seriously damaged buildings, a considerable increase in the capability of the building departments to control the repair and abatement work will be required.

VII. Pre-Disaster Procedures and Ordinances

Introduction

Each area of the United States is subject to potential disasters. It is important to consider the time frame for the pre-disaster and post-disaster periods. The post-disaster period is considered to be the period of emergency relief, recovery and rehabilitation programs. After these programs have been completed, the area returns to the pre-disaster period. Those areas of the United States which have not experienced certain types of disasters in historic time may be said to be in their pre-disaster period.

The level of disaster mitigation through identification of existing hazardous buildings and rehabilitation or demolition of those buildings varies with the time period under consideration. If several decades have passed since the last disaster and there has been no active governmental program, the level of public concern is usually very low. This was the case after the earthquakes of 1906 San Francisco, 1925 Santa Barbara, 1933 Long Beach, and to some extent 1952 Bakersfield. Examination of existing buildings for their earthquake resistance was practically non-existent prior to the 1952 Bakersfield earthquake, even though there had been increasing concern about the earthquake-resistant design of new buildings since 1925. It was not until 1967 that there were meaningful provisions for the investigation of public school buildings in California built prior to the 1933 Long Beach earthquake. A program for the investigation of California hospitals did not begin until 1956.

Since the 1971 San Fernando earthquake proved the validity of the recommendations of Steinbrugge et al, (1970), "Earthquake Hazard Reduction," [14] relating to the survey of non-earthquake resistant structures, activity has been increased in the pre-disaster inspection of existing buildings. Without an active Federal program for surveying existing buildings to evaluate their resistance to natural disasters, the level of activity in this field of disaster mitigation will decrease with increased public apathy. This occurred in the late 1940s in California between the 1933 Long Beach and the 1952 Bakersfield earthquakes when legislation was introduced not only to eliminate the investigation of existing school buildings for earthquake resistance, but

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to eliminate the provisions of the Field Act which had proven to be one of the best programs for the design and construction of earthquake-resistant structures. Fortunately, it did not pass.

The following portions of this paper discuss the experience in California of pre-disaster inspection of buildings and the results of plans for implementation of programs for the elimination of hazardous buildings. Programs related to the public schools, universities, hospitals, and programs for various cities are presented as well as several of the Federal programs for the identification of hazardous buildings.

California Public Schools

The 1933 Field Act provides for inspection of school buildings when requested by at least 10 per cent of the parents having children in the school district or when requested by the school district. This is clearly not an overall retroactive provision for the investigation of all public schools. Relatively few schools were inspected under the provisions of the Field Act [28].

In 1939, the Garrison Act [29] provided that if pre-1933 public schools were inspected and found to be unsafe for use, the school district must take action to repair, replace or abandon the schools. Again, this was not retroactive nor did it require all schools to be investigated. After the 1952 Bakersfield earthquake, school board members expressed increasing concern over their personal liability if they had not required an inspection of pre-1933 schools and if there were to be death or injury in these schools because of an earthquake. In 1966, the State Attorney General ruled that the school board members were personally liable in such cases. In 1967 legislation was passed, requiring that these schools be inspected and requiring the school board members to take definite steps with specific time deadlines to avoid such individual liability. The reports of such investigations were required to state whether the pre-1933 public schools were "safe" or "unsafe" based on the sole consideration of the protection of life and prevention of personal injury at a level of safety equivalent to that established by the Field Act, which would be expected from one disturbance of nature of the intensity used for design purposes in the Field Act. Few, if any, pre-1933 schools were reported "safe." No public school buildings declared "unsafe" for school use can be used for school purposes after June 30, 1975, unless it is reconstructed to a level of safety equivalent to new school buildings designed and constructed under the provisions of the Field Act.

In 1972, 1,593 schools reported "unsafe" were still in use. The cost of their reconstruction or replacement is estimated to be more than $600 million. Construction bond issues for their reconstruction or replacement have failed each time since 1970 in the larger cities such as San Francisco, Oakland, San Diego and Los Angeles. The first Los Angeles bond issue failed even though the election was held so soon after the 1971 San Fernando earthquake that aftershocks were still being felt at the time of the election.

California State Universities, State Colleges and Private Schools

State colleges above the junior college level, State universities and private schools at all levels are not covered by the provisions of the Field Act. Junior colleges are under the jurisdiction of the Act. No pre-disaster inspection and evaluation procedures are in effect for the private schools similar to the 1967 Garrison Act.

State colleges and State universities are designed under the applicable editions of the Uniform Building Code and other local building codes. The buildings on the nine campuses of the University of California are currently being investigated to evaluate their earthquake resistance using the 1970 Uniform Building Code - Title 24, State of California Building Standards - as the evaluation criteria. Private consulting structural engineers are performing the investigative work and are preparing estimates of the costs of rehabilitation. The need for these investigations developed out of concern of the University of California Board of Regents for the earthquake safety of the buildings under their jurisdiction as a result of the 1971 San Fernando earthquake. No known program of similar investigation of the other State colleges and State universities is being undertaken.

California Hospitals

Following the 1952 Kern County earthquakes, the Department of Public Health became concerned about the earthquake resistance of many of the old hospitals constructed prior to the inclusion of lateral force requirements in the local building codes. In 1955, the Department engaged a structural engineer to make cursory inspections of more than 600 such hospitals. He reported that approximately 100 were of such construction that more detailed investigations should be made to determine their structural condition. In 1956, the Department adopted regulations in Title 17, California Administrative Code, which required that if the Department determined that an evaluation of the structural condition of the hospital were necessary, then the licensee was required to submit a report prepared by a structural engineer or architect to establish the basis for alterations to the building to eliminate or correct the structural conditions which might be hazardous to the occupants. Letters were written to the licensees putting them on notice of the need for such investigations. Most notified licensees were informed by their attorneys that they could be personally liable in the event of injury or death to patients or staff caused by a damaging earthquake. The inspections and reports were prepared by private consulting engineers. The progress under this program has been good though there is no deadline for compliance with the recommendations in the consultants' reports similar to the provisions of the Garrison Act for public schools.

This program did not require mandatory inspection and/or repair of all hospitals similar to pre-1933 public schools. Generally, only those judged to be hazardous by the Department's Structural Engineer were required to comply. Hospitals built under codes which had lateral force provisions or earlier hospitals that were not considered hazardous generally were not included. Twenty hospitals have been demolished and ninety have been reinforced under the program.

Parapet Ordinances

More people in the United States have been killed by falling parapets, ornamentation and unanchored masonry walls during earthquakes than from any other earthquake cause. The City of Los Angeles recognized this problem and in 1949 the Parapet Correction Ordinance was enacted. This ordinance was specifically aimed at those building elements which are most hazardous and it makes it mandatory that they be corrected. The wisdom of this ordinance was re-emphasized in the 1952 Kern County, 1969 Santa Rosa and 1971 San Fernando earthquakes. The Los Angeles program was adequately funded and administered by engineers with special public relation training. This program has been very successful, being both economically and politically feasible [15]. Other cities in Southern California enforce similar ordinances which proved their merit in the 1971 San Fernando earthquake.

A parapet ordinance was passed recently in San Francisco, but has not been implemented because of budget limitations for hiring the necessary inspectors to carry out the program. Similar ordinances have been proposed in the cities of Berkeley and Oakland, but these ordinances have not been adopted.

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The author attempted, with the cooperation of the Structural Engineers Association of California, to have the City of Eureka and the County of Humbolt pass ordinances similar to the Los Angeles Parapet ordinance, following his inspection of the 1952 Eureka earthquake damage. To date no known action has been taken by these authorities. Certainly immediate near term benefits would accrue if cities would pass ordinances to remove these "clear and present" falling hazards. It is recommended that a Federal program be implemented to encourage local jurisdictions to adopt ordinances for the removal of hazardous projections from buildings.

Long Beach

Prior to the 1933 Long Beach earthquake practically all buildings built in the State of California were designed for vertical loads only. In 1959 the Long Beach Building Code was amended to give the Building Department the authority to condemn earthquake-hazardous buildings. Letters of condemnation for 118 buildings and letters requesting demolition or repair for 149 buildings were sent by June 1969. After years of opposition by property owners, on November 3, 1969, a moratorium period was declared by the City Council to allow for further study of the provision for condemnation of earthquake-hazardous buildings.

In July of 1971 these regulations were amended by a City Ordinance which deals exclusively with the problem of existing hazardous buildings. Approximately 1,000 buildings built prior to the 1933 Long Beach earthquake came under this ordinance. As of May 1972 the Department has inspected six buildings and has sent all six owners of the buildings notices of excessive hazard. The 1971 ordinance has the following features: 1) Priority for inspection of pre-1933 buildings according to decreasing occupancy risk, and 2) Provides owners' options to demolish, to strengthen the building for a given life, to abandon and demolish the building within a specified time, to reduce the occupancy risk to an acceptable level thereby possibly increasing the life of the building, or combinations of the above. The 1971 ordinance is very specific and covers such items as: 1) Dynamic Soil Condition Testing, 2) Hazard Grading, 3) Calculation of Actual Lateral Force Withstanding Capability, 4) Minimum Tolerable Lateral Force Carrying Capability, 5) Owner's Options, and 6) Procedures for Filing of Notices for Abatement. The Long Beach program is a unique example of abatement of buildings declared to be hazardous in the pre-earthquake period. The success of this program is due almost entirely to continued efforts of a very dedicated building official, Edward M. O'Connor, City of Long Beach. The 1971 ordinance was developed based on the study by Wiggins and Moran [16], and the work of O'Connor [17]. The concept of a hazard grading system and the provision of options for the building owners have considerable merit. The 1971 Santa Rosa ordinance is more straight-forward and is based on less theoretical methods of analysis, and perhaps can be better understood by the building owners. It is recommended that these two 1971 ordinances be reviewed and a model ordinance developed using the better portions of each ordinance.

Federal Agencies

Space limitations in this paper do not allow for a full discussion of the pre-disaster plans for the survey and evaluation of existing Federal buildings. A recent publication from the Office of Emergency Preparedness, Disaster Preparedness, Vol. 1, Part IV [19] discusses this subject in detail. Many Federal agencies, HUD, HEW, GSA, VA, NBS and the National Science Foundation have varying responsibilities in this area. Federal agencies perform surveys of the buildings they own and lease to gather information on performance and maintenance needs, but the effort is neither comprehensive nor continuous. Such activity is quite expensive, though no specific cost data are available. These surveys are not "safety" surveys in the terms of future disaster loss mitigation. Post-disaster surveys are performed on an ad hoc basis without proper criteria for evaluation of risk, and there have been duplications of surveys by various Federal agencies in a given area.

There is a vital need for these "safety" surveys and evaluations, as they represent one of the most effective means of extending regulation to existing construction. One of the best methods of disaster loss mitigation to existing structures is to require that there be an inspection and evaluation, prior to the granting of loans and other assistance for the alteration of these structures.

Since the 1971 San Fernando earthquake, the Veterans Administration has started an extensive program of investigation of the earthquake resistance of their hospitals. They are starting with hospitals located in seismic risk zones 2 and 3, Uniform Building Code. There are 81 V. A. hospitals in these seismic risk zones. Replacement of these hospitals could cost several billion dollars. The Administration has engaged a Board of Consultants which includes experts in seismology, structural engineering and other aspects of earthquake engineering. This Board of Consultants provides guidance on which hospitals should be investigated, what type of special geological and soils investigations should be made, and the procedures used to evaluate and reconstruct the older hospitals. Local architects and structural engineers are performing the actual investigation and evaluation of the hospitals as well as preparing the plans and specifications for their reconstruction.

The Defense Civil Preparedness Agency (formerly "Office of Civil Defense") in its National Fallout Shelter Survey has surveyed buildings to establish fallout protection factors. The information for each investigated building is coded on a "Fosdic" form which is read directly by the computer. Unfortunately, the buildings in this survey usually do not have wood roof and floor construction because of the low protection factors provided by these light weight construction materials. These uninvestigated buildings would include Type III buildings - Uniform Building Code - which have experienced high damage losses during previous disasters, such as earthquake and wind. Exterior and interior walls are coded according to their weight in pounds per square foot, but the type of wall construction is not given. Also, the type of construction is not given on the forms. Since the earthquake resistance of buildings is influenced primarily by the type of construction and wall materials - McClure [7] - it would be difficult to rate the data compiled on the Shelter Survey Forms.

Information is surprisingly lacking concerning the inventory of buildings in the United States. The author's experience has been that except for an inventory of the one-to-four-unit dwellings given in the U.S. Census data, no inventory of buildings, even by occupancy, is readily available in the United States. It is recommended that the Federal Government make a preliminary study of the feasibility of gathering building inventory data by occupancy and type of construction. It would be a tremendous task to gather the data for all buildings, excluding dwellings, but this data is needed to provide input data for disaster loss and loss mitigation studies.

A first step could include only emergency centers and other structures with high socio-economic values that are located in identified high risk land in the larger urban areas which are most vulnerable to natural disasters [19].

VIII. Conclusions and Recommendations

Deficiencies exist in the state-of-the-art for the surveying and evaluation of the hazards represented by existing buildings and buildings damaged by disasters. Current procedures used for the abatement of the hazards represented by these buildings do not reduce these hazards to an acceptable level of public safety. With the increased involvement of the Federal government in disaster loss indemnification programs, the Federal government has a vital interest in the development of new programs for disaster mitigation to existing buildings. In the past this has been left almost entirely up to the state and local governments, but this has resulted in an inconsistent approach to the problem throughout the United States.

Improving building practices for disaster mitigation constitutes but one step in a more comprehensive approach required to abate hazardous existing buildings and buildings damaged by disasters. Mitigation of the potential disaster dollar losses to these buildings and the life hazard to the occupants of these buildings must be based on: 1) Public recognition of the magnitude of the hazard these buildings represent, 2) Characterization of the hazards based on appropriate kinds of investigation, 3) Evaluation of the risk represented by these hazards, and 4) Alternative public policy decisions based on judgments of acceptable levels of risk and feasible abatement programs which consider the social, political, and economic factors involved.

Federally sponsored programs must be based on:

1) Adequate inventory data on the number of existing buildings by occupancy, type of construction, and other parameters which identify their hazard potential.

2) Consistent criteria used to survey and evaluate the damage potential represented by existing and damaged buildings.

3) Adequate disaster damage surveys that report reliable dollar loss statistics.

4) Currently acceptable public policies expressed by laws and ordinances concerning abatement procedures.

5) The degrees of rehabilitation which can be achieved using present repair procedures.

6) New knowledge which has developed from recent disasters, the potential for new knowledge in future disasters, and the ability to disseminate this new knowledge to the intended users including the governmental agencies.

7) The need for research to evaluate current repair procedures and research to develop new methods of repair.

8) Recognition of the political and economic considerations and other problems in application of retroactive abatement procedures and the lack of public policies to resolve these problems effectively.

9) Evaluation of alternative plans for Federal funding to provide financial inducements for the abatement of hazardous buildings.

In the context of the above, the following conclusions and recommendations for Federally sponsored studies are based on the findings presented in this paper and the recommendations given in Steinbrugge, et al. (1970), "Earthquake Hazard Reduction" [14]. Although this latter report was directed to the reduction of earthquake hazards, many of the recommendations are applicable to mitigation of losses from other natural disasters, such as hurricanes, tornadoes, and extreme winds.

- 1. There is a need for an inventory and data on all buildings that are located in potentially disaster hazardous urban areas of the United

States. The magnitude of the work of gathering these data is so large that this information should first be gathered for buildings with high socio-economic values - emergency centers, hospitals, and other occupancies with higher disaster support responsibilities. The appropriate Federal agency should make a feasibility study of how to undertake such an inventory which would provide data on the type of occupancy, type of construction and other details of construction needed to make viable potential risk evaluations. The magnitude of such an inventory could exceed the National Fallout Shelter Survey program and, therefore, a feasibility study is recommended first rather than a recommendation to undertake such an inventory study immediately.

- 2. There are no accepted criteria for the procedures to be used in pre-disaster surveys and evaluations of existing buildings. The limited number of such surveys have been conducted by various Federal, state, and local agencies which has resulted in duplication and overlap. Results of such surveys are not readily available nor even mutually known. One Federal agency should be given the responsibility to develop these criteria, publish guidelines, and study the feasibility of making these reports more readily available. This work should be done in cooperation with other Federal agencies which have statutory responsibilities to conduct such surveys and archive these reports.

- 3. Previous damage surveys have lacked adequate dollar damage statistics reported in a consistent manner. It is too simplistic to recommend that one "standard" form be developed for all disasters, but certainly the present procedures can be improved using "Fosdic" type data forms to gather, analyze, and report damage statistics. Processing of the data should be done by computer at the local level as well as the Federal level. Further study of the procedures used by Los Angeles County in the San Fernando earthquake is recommended. Review of recently revised inspection forms designed for computer analysis by the City of Los Angeles is also recommended. The same Federal agency given the task under recommendation 2 above should be given this responsibility.

- 4. There is no accepted criterion for the evaluation of the hazards created by damaged buildings or by buildings constructed without adequate lateral force resistance. The Uniform Building Code's "Dangerous Building Code," Part IV, 1967, represents one approach to this problem, but the "Dangerous Building Code" is not generally adopted or enforced. The appropriate Federal agency should develop criteria for such evaluations and develop model ordinances for adoption by local jurisdictions. Approval of Federal funds for disaster loss indemnification and other programs for the abatement of hazardous buildings should be continguent upon adoption of such model ordinances. These ordinances should include the new approaches now being used in Santa Rosa and Long Beach which provide for various owner options. Important items in these ordinances must be provisions to prevent the lowering of local safety standards after the disasters and adequate monitoring of repair work by Federal and local agencies. For such ordinances to be effective there must be corresponding Federal and local government agencies to participate in these abatement programs.

- 5. Improved building practices for the design and construction of structural components are developed from theoretical non-testing studies, the testing of these members and the observation of the performance of these components in disasters. There is a need for more full-size scale testing of structural members to better understand their failure mechanisms so that their degree of damage can be readily evaluated in damage surveys. Most tests have been performed on small scale models and it is only after disasters that opportunities are available to observe the performance of full-size members when subject to yield point and failure loadings. Unfortunately, immediate repairs to these members do not allow for the proper evaluation of their performance during disasters. There is a need to develop criteria based on full-sized tests for appropriate guidelines to evaluate the levels of damage to structures and their components which are not immediately obvious. One Federal agency should be given the responsibility to develop test programs and to obtain funding for testing of full-scale structural components from which the above needed criteria can be developed.

- 6. Little testing or other research has been conducted concerning the repair of damaged buildings. The methods of repair have not changed appreciably except for the use of epoxy-type adhesives. It is necessary to develop simple methods of repairing damaged structures based on tests to insure continued acceptable levels of safety in the event of future disasters. At present, the repair of damaged buildings is a relatively new and untried art that has long been neglected by engineers. It is left primarily to the skill and ingenuity of the engineer, on an ad hoc basis, with little to guide him but his judgment. Repair of damaged buildings has been performed on this basis without adequate test data to verify the degree of rehabilitation attained. There is a vital need for a program for the testing of structural members and their connections, which have been stressed above their yield point and then repaired, to evaluate their resultant strength and ductility. One Federal agency should be given the responsibility to review the adequacy of present repair methods as well as to develop new methods of repair. Testing programs should be developed and funded in cooperation with construction material associations and structural engineers in private practice. The test programs should include full-sized tests performed on large shaking tables and in wind tunnels, and must include cyclic, three dimensional dynamic loading as compared with the more traditional simple reversal of loading procedures. This testing is too important to be left to either the construction materials associations, the academic theoretical researchers, the practicing structural engineers, the private testing laboratories, or to the Federal government agencies, but must be conducted using an inter-disciplinary approach. More effective procedures must be developed to make the new knowledge from such test programs more readily available to the practicing engineers so that it can be more quickly incorporated in new codes and construction.

- 7. Immediate near term benefits in disaster mitigation will result from the removal of unreinforced brick and masonry parapets, ornamentation, improperly anchored exterior walls, and other appendages which represent "clear and present" hazards in the event of earthquake and extreme winds. A first step toward the mitigation of these hazards would be a survey of buildings with these hazards in the high risk urban areas. Implementation of local programs, based on federally developed model ordinances, should be based on the procedures used in the City of Los Angeles. Such programs must include a public education program combined with a federally supported financial incentive program.

- 8. Viable programs for building hazard reduction must be based on cooperative Federal, state and local government disaster plans. The survey and evaluation of damage <u>immediately</u> after the disaster is vital to proper disaster mitigation. In the past there has been the traditional "patch and paint" approach which covers up the evidences of damage prior to an adequate investigation of the damage. The appropriate Federal agency should be given the task to see that the states have real natural disaster response plans - not "paper" plans - and that these plans are periodically tested as a condition for Federal funding of such state programs. Review of the experience in the San Fernando earthquake and the recent hurricanes is an obvious recommendation.

- 9. In the emergency period additional funds are needed for additional inspectors, clerical and administrative staffing, for hiring of private consulting engineers to make special studies, and for the use of special equipment, such as helicopters and emergency communications systems. It was not clear after the San Fernando earthquake whether or not funds from Federal disaster programs were available for such needs and local agencies were reluctant to spend their own money for these vitally needed services. A re-evaluation of funds allocated to civil preparedness (civil defense) programs is needed. Apparently facilities and equipment and equipment furnished through these programs are not available to local building departments during disaster emergency operations. Pre-disaster funding for emergency, portable hand-operated two-way radio communications for building departments should be given the highest priority.

- 10. After previous earthquakes and other disasters there has been a lack of coordination between Federal agencies which have statutory responsibilities or interests in making immediate surveys of the damage during the emergency period. This has resulted in duplication and incomplete reporting by some agencies. Their studies have been "mission" oriented, performed too late, and the new knowledge which develops out of each disaster is lost or not made readily available to the state and local agencies or to those in the construction industry who could benefit from these early studies. One appropriate Federal agency should be given the responsibility to develop pre-disaster response plans for the various Federal agencies and non-governmental institutions to make meaningful immediate investigations of the damage. Federal funds should be made available on a stand-by basis to provide for adequate studies and preparation of reports for distribution at the earliest practical date. These investigations should be coordinated to cover all aspects of the disaster and require a multi-disciplinary oriented approach. Hopefully, only a few comprehensive reports would be prepared to minimize the duplication and number of different "mission" oriented reports. The use of preliminary reports is strongly recommended to expedite the dissemination of the new knowledge to those who have an immediate need for this information.

- 11. Development and implementation of disaster mitigation programs by the Federal Government can take place only if there are accepted public policies at all levels of government supporting such programs. The public is generally not fully informed concerning the possibility for and the consequences of natural disaster until after they have occurred. A coordinated Federal Government program is needed for public education on a continuing basis concerning the potential damage which can be caused by natural disasters and what can be done to minimize their effects. An informed public will aid in making and supporting public policy decisions which weigh the economic and social impact and life hazard posed by these disasters against the cost of reducing these hazards to an acceptable level of risk. It is recommended that an increased public education program concerning disasters and disaster mitigation possibilities be undertaken by a Federal agency with adequate funds for continuing implementation by all levels of government.

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ABNORMAL LOADING ON BUILDINGS AND PROGRESSIVE COLLAPSE

by

Norman F. Somes

I. Introduction

Since 1968, there has been growing international concern that multistory buildings are frequently designed without explicit consideration for abnormal loading conditions. On May 16 of that year, there occurred the much-publicized collapse of apartments in the Ronan Point building in London. The building has 22 stories of precast concrete panel construction above a cast-in-place concrete podium. A typical floor layout is shown in Figure 1 in which the structural walls are shown solid. The collapse was triggered by an accidental explosion of gas that leaked from the connection of a gas range located in an apartment on the 18th floor. The Report of the Inquiry into the Collapse [1] states:

> "The explosion blew out the non-load-bearing face walls of the kitchen and living room, and also, unfortunately, the external load-bearing flank [end] wall of the living room and bedroom of the flat, thus removing the support for the floor slabs on that corner of the nineteenth floor, which collapsed. The flank walls and floors above this collapsed in turn, and the weight and impact of the wall and floor slabs, falling on the floors below, caused a progressive collapse of the floor and wall panels in this corner of the block [building] right down to the level of the podium."

The podium and the extent of the collapse is shown in Figure 2; an adjacent building of identical construction shows the appearance of the building prior to the collapse. From Figures 1 and 3 it may be seen that collapse affected both the living room and bedroom above the 16th floor, while below this level, the collapse was limited to the living room. Four people were killed in the collapse and seventeen people were injured.

> "The loss of life and injury might well have been very much worse. At 5:45 a.m., mercifully, most tenants were in their bedrooms . . ."

The Report also documents that, by a fortunate chance, of the four apartments directly above the one in which the explosion took place, only one was occupied at that time.

The Report of the Inquiry drew international attention to several deficiencies in existing codes and standards, particularly as they applied to multistory buildings. Interim additional criteria [2] were quickly implemented in the United Kingdom having regard to the appraisal and strengthening of existing buildings and the design of new structures. Several other countries in Europe introduced additional design criteria to explicitly deal with the risks exposed by the Ronan Point incident.

To date, with one exception, the U.S. codes- and standards-writing bodies have not published criteria taking account of abnormal loads and progressive collapse. The exception is the 1972 American National Standards Institute A58, <u>Minimum Design Loads in Buildings and Other Structures</u> [3] which provides a short statement drawing the attention of the designer to the problem. Several standards-writing bodies have established technical committees to consider the problem, however.

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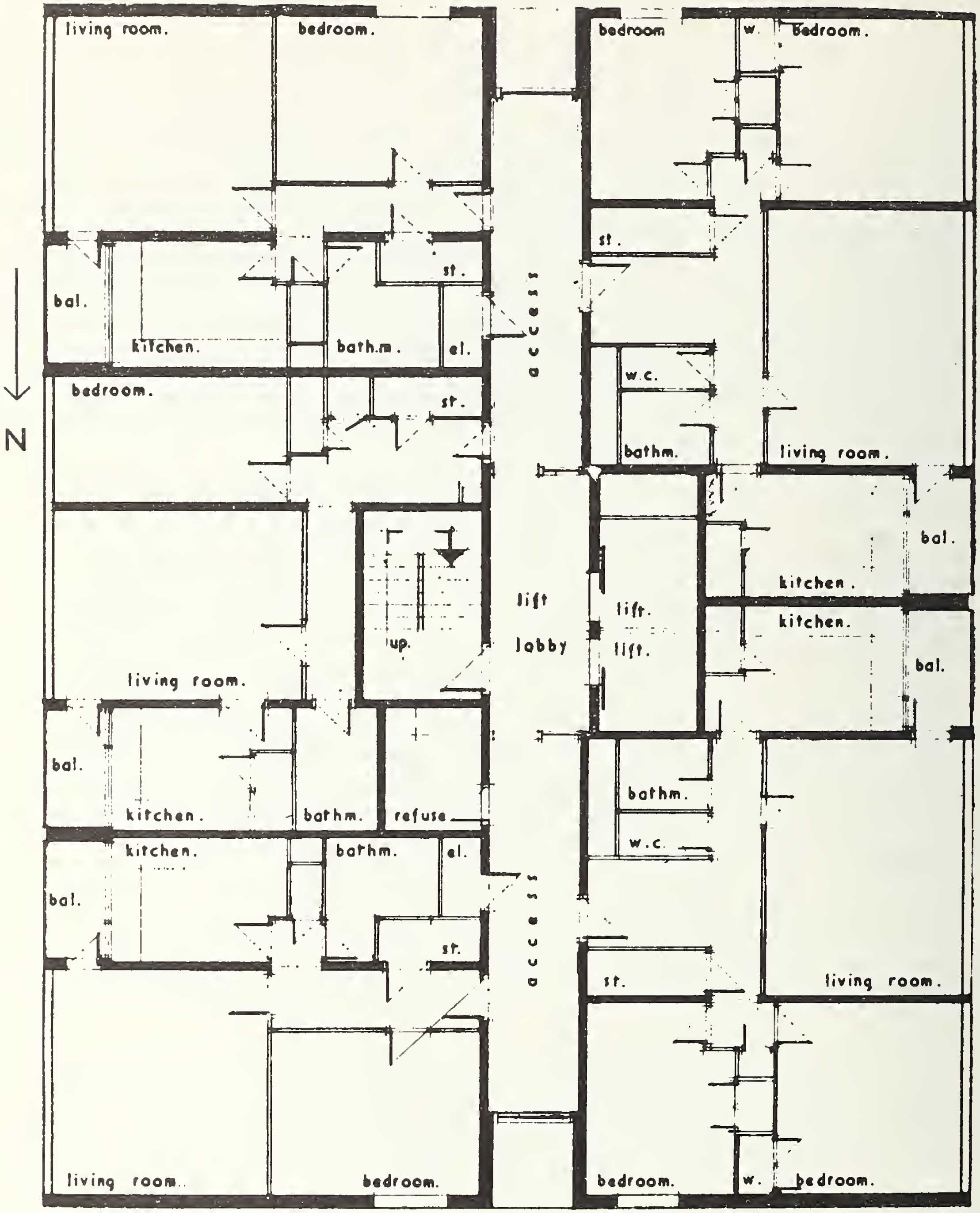

Figure 1 Typical floor layout of Ronan Point apartment building (explosion occurred in a SE-corner apartment). Reproduced from [1].

Figure 2 Ronan Point apartment building after the collapse, with a second identical building in the background. Reproduced from [1].

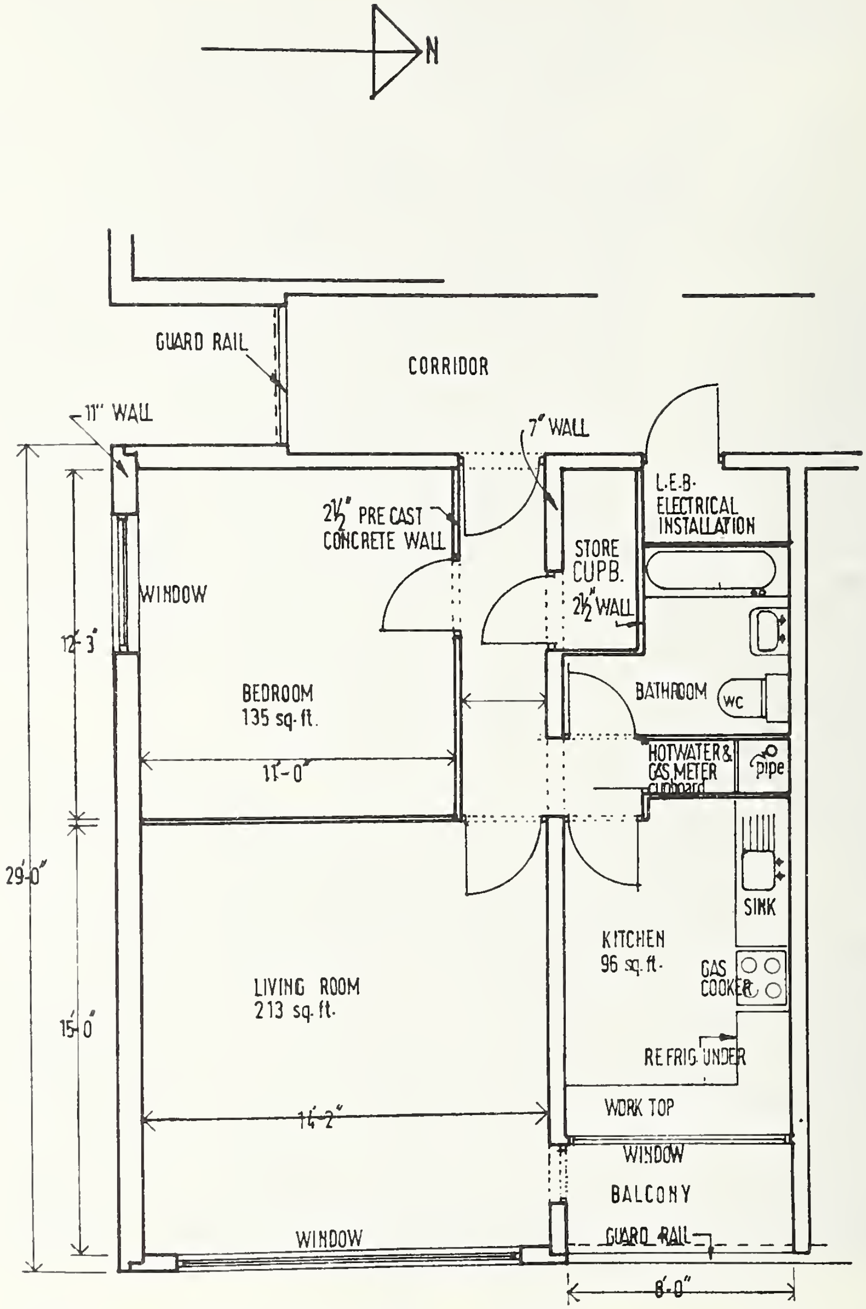

Figure 3 Layout of the SE-corner apartment on the 18th floor where the explosion originated. Reproduced from [1].

The prime mover in the matter of progressive collapse in the USA has, to date, been the Department of Housing and Urban Development (HUD). In August 1969, provisions against progressive collapse were included for the first time in a Structural Bulletin [4] issued to the producer of a precast concrete housing system by HUD's Federal Housing Administration (FHA). Criteria against progressive collapse were included in HUD's Guide Criteria documents [5] prepared by the National Bureau of Standards (NBS) early in 1970 and implemented in HUD's experimental housing program Operation Breakthrough. In October of 1971 the FHA circulated its draft document "Provisions to Prevent Progressive Collapse" [6] for review by certain trade associations, standards-writing bodies and members of the design profession. This document expressed criteria intended for application by FHA in evaluating multistory buildings for which Federal mortgage assurance is required. The document has not been circulated in final form but the draft has served as a starting point for much discussion. In each of these three preceding instances, the criteria reflected the United Kingdom requirements at that time.

In November 1971, the National Bureau of Standards, at the request of HUD, commenced a detailed study of abnormal loading on buildings in the USA and the many aspects pertinent to the avoidance of progressive collapse. The results, to date, provide the basis for this interim progress report, which firstly attempts to relate the many parameters of the problem, to classify and discuss the various sources of abnormal loading and to quantify their frequency insofar as the currently available U.S. data permits. The implication of these findings for the USA are then discussed. The response of buildings and building elements to abnormal loading is then reviewed, including cases of progressive collapse. Several alternative approaches for the introduction of criteria are presented and, finally, conclusions are given with respect to the problem posed in the U.S.A.

II. Problem Definition

Progressive collapse may be defined as <u>a chain reaction of failures following damage to a relatively small part of a structure</u>.

An abnormal loading may be defined as <u>a condition of loading which a designer, following established practice, does not include in the normal analysis and design of a particular structure</u>. It is a loading condition of sufficient severity and probability of occurrence to be a cause for concern, but still of such a relatively rare nature as to be outside of normal design-life expectancy. This definition goes beyond that of static and dynamic forces and includes such conditions as the dislodgement of a bearing wall panel, and the development of a weld failure in a steel connection.

Recent reports [7, 8] confirm the growing view that studies of the problem of progressive collapse should deal generally with multistory construction and not be limited to high-rise construction or simply that using precast concrete panels. However, the view is widely held that framed buildings are more tolerant of local damage and have more resistance to progressive collapse than load-bearing structures. It is reasoned that this is due to the fact that continuity between members is more easily accomplished in framed buildings than in load-bearing structures, and that the former have greater ability for developing alternate paths for forces in the event of the loss of a critical member. This viewpoint is supported by the documented experience of engineers during World War II bombings [9].

Occupants of multistory buildings, whatever type of construction is used, have a right to expect adequate and consistent levels of safety. The user requirement may be expressed as an <u>adequate protection from extreme loads</u>. Expressed as a performance requirement, this corresponds to <u>adequate strength</u> namely, compliance, with a specified load capacity. Present U.S.

design standards specify load capacity in terms of severe combinations of dead, live, snow, wind, or earthquake loads. They do not specify load capacity with respect to abnormal loading conditions.

In a general sense, adequate safety is achieved by insuring that, at loads less than the specified load capacity, there is no loss of static equilibrium resulting in:

Local Collapse, or
Extensive Collapse.

The Ronan Point incident was clearly due to an abnormal loading condition and the collapse was extensive. Had the damage been confined to the apartment in which the explosion originated, it is doubtful whether the accident would have received more than local newspaper coverage. Such explosions occur somewhere everyday and arouse little reaction from society at large. It is only when they produce extensive collapse that they generate international attention.

The foregoing discussion serves to delineate the NBS study which is concerned with abnormal loadings on multistory buildings of any type of construction, and the need to prevent progressive collapse.

III. Public Acceptance of Risk

Risk is a function of the probability of occurrence and the consequence of a particular event. Zero risk in the face of all possible conditions and hazards can never be achieved. By assessing the statistics of all foreseeable hazards and evaluating their consequence, an acceptable level of safety can be achieved, acceptable safety at an acceptable cost. The acceptable risk to life and property is probably best decided by representatives of the community at large.

Otway et al [10] have provided one basis for considering the risks with which society is prepared to live. For this purpose, they use the U.S. accidental death statistics for 1966. The probability of death per person per year is given for a series of types of accidents in the following table:

Table 3.1

Motor Vehicle	2.7×10^{-4}
Falling	1.0×10^{-4}
Fire	4.0×10^{-5}
Drowning	2.8×10^{-5}
Firearms	1.3×10^{-5}
Poisoning	1.1×10^{-5}
Earthquake	8.0×10^{-7}
Lightning	5.5×10^{-7}

The paper points out that situations providing hazards on the order of 10^{-3} deaths per person per year are uncommon. When a risk approaches this level, immediate action is taken to reduce the hazard. This level of risk is unacceptable to everyone in society. At an accident level of 10^{-4} deaths per person per year, people spend money, especially public money, to control the cause. Risks at the level of 10^{-5} deaths per year are still significant to society. Accidents with a probability about 10^{-6} deaths per person per year are not of great concern to the average person. He may be aware of them but he feels they will never happen to him. There is a general lack of concern about accidents having a mortality risk of less than 10^{-6} per person per year.

Some qualification should be made with respect to this last point and the statistic for earthquakes. In fact, a considerable amount of money is spent on earthquake protection and this would appear to conflict with Otway's last general comment. However, Otway divided incidents by the total U.S. population whereas the earthquake risk is geographically concentrated. If, for example, the population living within seismic Zone 3 were used, the probability of death per person would be increased by a factor of approximately 5, to 4.0×10^{-6}, thereby appearing to remove the above conflict. Another factor contributes to the considerable U.S. expenditure on earthquake protection; in addition to deaths and injuries, serious earthquakes are accompanied by large property losses.

Public expenditure to provide protection against fire in buildings is very considerable and this is in response to a probability of 4×10^{-5} deaths per person per year together with heavy property damage. It would be useful to assess the frequency of abnormal loading incidents that, for people living in buildings susceptible to progressive collapse, would constitute a risk of death from that cause equal to that due to fire. It is important to differentiate between buildings that are susceptible and those not susceptible to progressive collapse. Figure 4 shows a speculative plot of the total number of U.S. housing units, increasing with time, and having a value of Y units at a particular point in time. There is a reason to believe that, in the absence of new criteria to minimize the risk of progressive collapse, the number of buildings that are susceptible, will otherwise increase. A second plot shows the number of susceptible units increasing with time from a relatively insignificant level to a very significant fraction of the total number of housing units. Of course, it is with the objective of stopping any such growth, that the NBS study and related studies are underway. Nonetheless, it is assumed that, in the absence of specific criteria, the number of housing units susceptible to progressive collapse have risen to X at the point in time previously referred to, X units out of total of Y.

The following assumptions will be made in order to carry out a comparison of the respective risks due to progressive collapse and fire:

1. The total number of abnormal loading incidents affecting U.S. housing units in a year corresponding to the selected point in time is N.

2. The probability of occurrence of an abnormal loading on a housing unit is the same irrespective of the type or location of the unit.

3. The abnormal loadings in question are of a severity that in the case of susceptible buildings, progressive collapse could occur.

On the basis of assumption 2, the probability of an abnormal loading per year in a housing unit susceptible to progressive collapse is N/Y.

In what follows, the result will be seen to depend upon the size and architectural arrangement of the multistory building considered. Consider a 100 housing unit building with a central service core, four housing units per

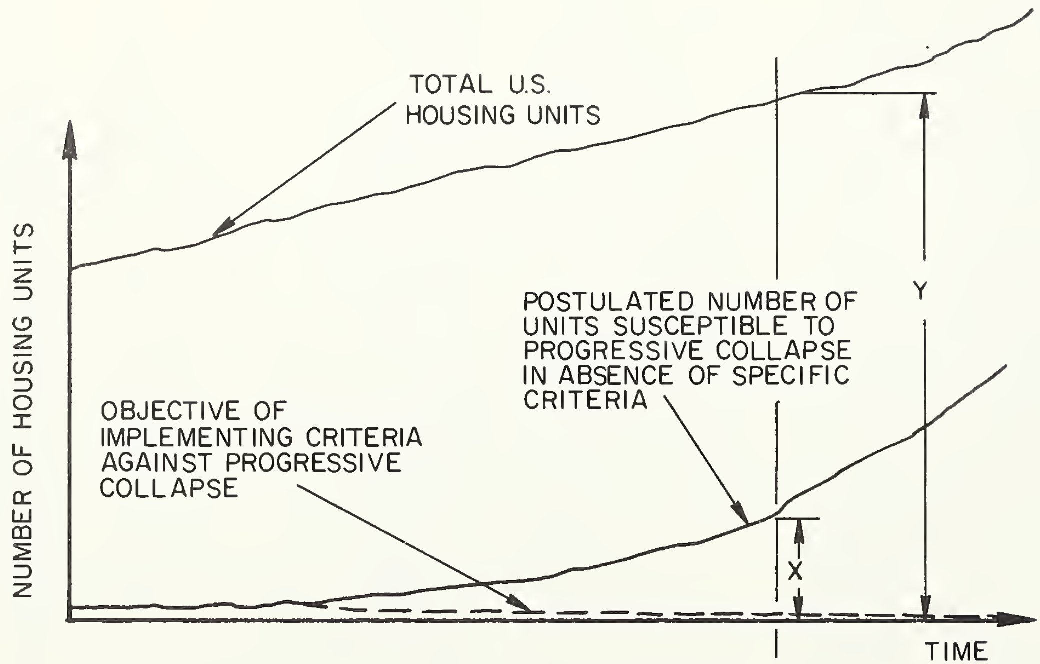

Figure 4 Postulated growth of number of housing units susceptible to progressive collapse in the absence of additional criteria.

story and an average occupancy rate of 4 persons per unit. If, as in the Ronan Point building, the collapse affected one quarter of the building in plan, then an estimate of life loss would have to be based upon some fraction of the number of occupants of that quarter. For part of each 24 hours, people will be absent from the building e.g., at work or school or in recreational activity. Furthermore, it is doubtful if more than one-half of the casualties would result in death. There would be a number of severely injured, less injured, etc. When both factors are considered, a reasonable estimate of the number of deaths would appear to be 25, namely 1/16 of the total number of building occupants.

The following discussion will be confined to units susceptible to progressive collapse.

The probability of an abnormal loading in a 100 unit building per year is

$$\frac{N}{Y} \times 100$$

The risk of death due to progressive collapse per person per year in that 100 unit building is

$$\frac{N}{Y} \times 100 \times \frac{1}{16}$$

Equating this risk of fatality to that due to fire

$$\frac{N}{Y} \times 100 \times \frac{1}{16} = 4 \times 10^{-5}$$

If Y is taken to correspond to the U.S. housing unit total as given by the 1970 U.S. Housing Census [11], namely 67.7 million

$$N = \frac{4 \times 10^{-5} \times 16 \times 67.7 \times 10^{6}}{100}$$

$$= 433 \text{ incidents}$$

This result states that, for the conditions considered, an annual U.S. total of 433 abnormal loadings on housing units would result in a risk of fatality that would correspond to the general risk of fatality in fire. Clearly the result is a function of the architectural layout of the building considered and hence the estimation of the ratio of possible deaths to the total number of occupants of the building.

It is shown that the risk of fatality due to progressive collapse per year could be written

$$\frac{N}{Y} \times \text{No. of units} \times \frac{1}{16}$$

For a given architectural layout, and given values of N and Y, the risk appears to be directly proportional to the number of units, namely the number of stories.

The use of the figure of 433 incidents should be qualified to account for the assumptions made and for its dependence upon architectural layout and size of building. Nonetheless, the figure of 433 incidents establishes an order of magnitude that is useful in assessing the significance of the statistics for abnormal loadings that are discussed in Chapter 5 and summarized in Chapter 6. In Chapter 6 it will be shown that a lower bound estimate of the number of abnormal loadings per year on housing units is 702.

IV. Classifications of Abnormal Loadings

Publications such as the Engineering News Record regularly document engineering failures as well as successes. The authoritative work of Feld [12] has described many building failures. Allen and Schriever [7] have summarized reported failures of recent years in North America. Two conclusions are drawn from works such as these: Only a small minority of building failures occur due to a loading of a type explicitly considered in the design. The great majority of failures result from loading conditions to which current codes and standards give little or no guidance and which, as a consequence, are not considered in design. Such loading conditions are termed abnormal loadings in this report. The second conclusion is that there is a large variety of abnormal loadings and no classification of them could reasonably be expected to be complete. One important contributing factor is that, with ever advancing technology, new sources of abnormal loadings can be expected to be generated. With these several thoughts in mind, the following classifications are deliberately limited to abnormal loadings for which the probability of occurrence seems significant. The first classification is an overall generic one:

violent change in air pressure

accidental impact

faulty practice

foundation failure

The subclassifications will now be discussed in more detail.

A. Violent Change in Air Pressure

These include:

Sabotage bombings
Service system explosions
Other explosions within the building
Explosions external to the building

Sabotage using explosives is a very serious form of abnormal loading. The motive for sabotage might concern only one person, a family, or an organization resident in the building, yet the bombing could affect many or all of the occupants. Service system explosions can originate in heating, cooling, and cooking systems, in high-pressure steam pipes and in boilers. Sources of other internal explosions include containers of liquified gases such as propane or butane or containers of gasoline. There are a number of sources of accidental explosion external to the building such as the shipment of hazardous materials through urban areas by truck, railroad, and waterway or by the rupture of gas transmission and distribution systems.

B. Accidental Impact

These include:

Highway Vehicles
Construction Equipment
Aircraft

Trucks and automobiles leaving the highway out of control are included in the first category. Accidents involving cranes and lifting devices of all kinds are included in the second category. In urban areas, construction frequently takes place on congested sites that have relatively small clearances from existing occupied buildings.

C. Faulty Practice

Past experience would indicate that when failures do occur, they are frequently the result of faulty practice. Whether or not local or extensive collapse results, is largely a function of the type of construction involved, i.e. whether it can tolerate local damage without extensive collapse.

Design Error
Construction Error
Misuse or Abuse by the Occupant

Misuse or abuse by the occupant can include ill-considered architectural changes or cutting of the structure.

D. Foundation Failure

The ASCE Research Council on Expansive Soils has documented [13] that building foundation failures and distress account for average annual property damage in the USA valued at $740 million. While this figure is not broken down into specific categories of failure, it is nonetheless indicative that present codes and standards may not provide adequate requirements for foundation design. Feld [12] has documented a number of instances in which foundation failure has produced severe building distress and even total collapse. It is apparent that foundation failures, including the following specific cases, can pose severe abnormal loadings:

Unforeseen Settlement
Foundation Wall Failure
Scouring Action of Floods on Foundations
Adjacent Excavation

An important factor is the growing scarcity of land in urban areas causing more and more buildings to be located on sites previously considered to be of marginal quality for construction purposes.

V. Studies of Abnormal Loadings

The NBS studies to date have revealed that statistics are compiled by appropriate authorities with respect to sabotage bombings, gas explosions, explosions of hazardous materials in transit, highway vehicle accidents and aircraft accidents. Efforts to locate data regarding other abnormal loadings will continue, however, it is recognized that, for certain incidents, statistics may either not be compiled or they may be collected in such a fragmented manner as to make them of little value.

A. Sabotage Bombings

Sabotage bombings are generally classified as explosive or incendiary but it is the former that are of particular relevance to progressive collapse.

A.1 Characteristics of Loading

There is no shortage of published technical information relating to the pressures, rise time, and distribution relationships for explosive charges. Organizations such as the U.S. Department of the Army have developed and distributed materials that serves as a guide in the use of explosives in the destruction of military obstacles and in certain construction projects. The Army Field Manual [14] provides information including type, characteristics, and uses of explosives and auxiliary equipment, preparation, placement and firing of charges, and charge calculation formulas.

Figure 5 due to Granstrom [15] expresses pressure-time curves at selected distances from a 1 kg (2.2 lb) charge of T.N.T. The ordinates express the static peak pressure in atmospheres. As an illustration, at a distance of 1 meter (39.4 in) from the detonating charge the peak static pressure would be approximately 11 atmospheres corresponding to approximately 160 psi. This positive pressure pulse would have a duration significantly less than 1 millisecond. A pressure of 160 psi is so large in comparison with the normal resistance of walls and floors as to make their destruction a certainty. At a distance of 10 meters (32.8 ft) from the charge the peak static pressure would be approximately 1.5 psi while the positive pressure pulse would last approximately 4 milliseconds. Pulses of these durations are so short, in comparison to the natural period of building elements such as walls and floors (20-40 milliseconds) as to require any structural analysis of the element in question to be a dynamic one.

The characteristics of explosive charges differ considerably from those of flammable gases. According to Rasbash [16].

"As a rule, gas and vapor explosions take place substantially more slowly than explosions involving high explosives such as T.N.T. The most explosive mixture of a fuel vapor and air in a volume of $30m^3$ (1050 ft^3) will contain about 2.5 kg (5.5 lb) of fuel. The energy potential of this will be equivalent to that of 20 kg (44 lb) of T.N.T., but the pressure pulse with the gas explosion would last several hundred milliseconds and with T.N.T. only about 1 millisecond." For comparison, Figure 6 is included to show the pressure-time curves for typical vented gas explosions.

A.2 Probability of Explosive Bombings

Two organizations have gathered national statistics for sabotage bombing in the USA, namely the Federal Bureau of Investigation and the International Association of Chiefs of Police.

The Federal Bureau of Investigation (FBI) commenced its program to collect and classify bombing incidents at the start of 1972. The Bureau issues monthly bulletins [17] summarizing the data reported by its Field Agency; it is understood that the first annual report, containing statistics for 1972, will be issued in the spring of 1973. Figures for the 10 month period, January through October 1972, are given in Table 5.1. The term "actual" is used to denote that detonation of an explosive or ignition of an incendiary material actually occurred, whereas "attempted" denotes that detonation or ignition of the bomb did not take place.

Because reports continue to trickle in long after the reporting period has passed, the FBI cautions the user of the data that the figures are subject to revision (in an overall upward sense). The figures of the most recent months are likely to change most. The monthly totals should therefore not be used in an attempt to define trends.

Referring firstly to Lines 1 through 5 of Table 5.1, it is seen that, during the 10 month period, there was a total of 608 actual explosive bombings out of a total of 1689 incidents of all categories. The ratio of actual explosive bombings to actual and attempted, explosive and incendiary incidents is 608/1689, namely 0.36.

The FBI uses a number of categories with which to describe the target.

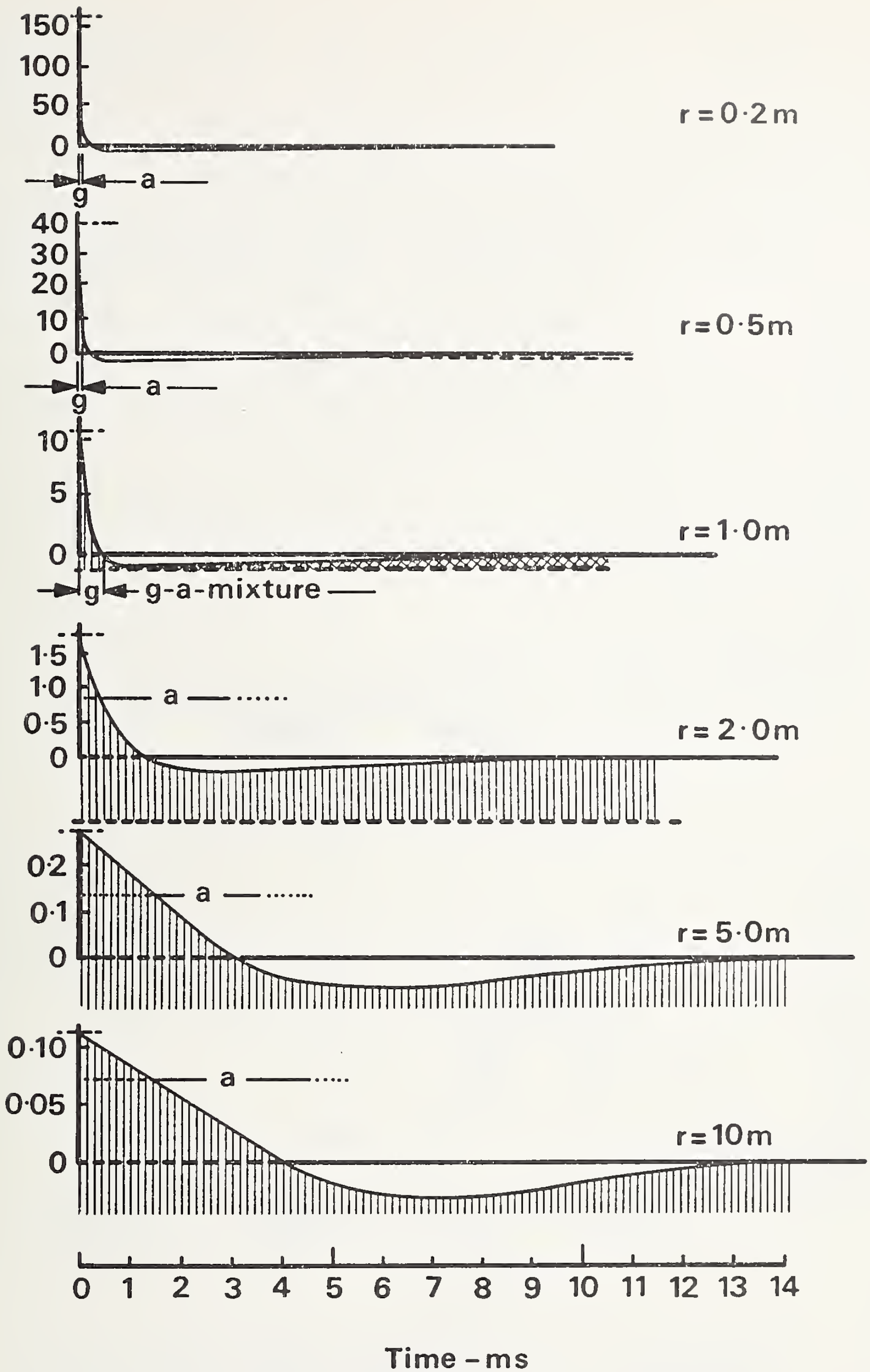

Figure 5 Pressure-time curves at selected distances (r) from a 2.2 lb (1 kg) charge of TNT. The ordinates give the pressures in atmospheres above atmospheric pressure. The horizontal dashed lines correspond to a total vacuum. g = gas, a = air. (Due to Granstrom, reference 1.5, reproduced from [70]).

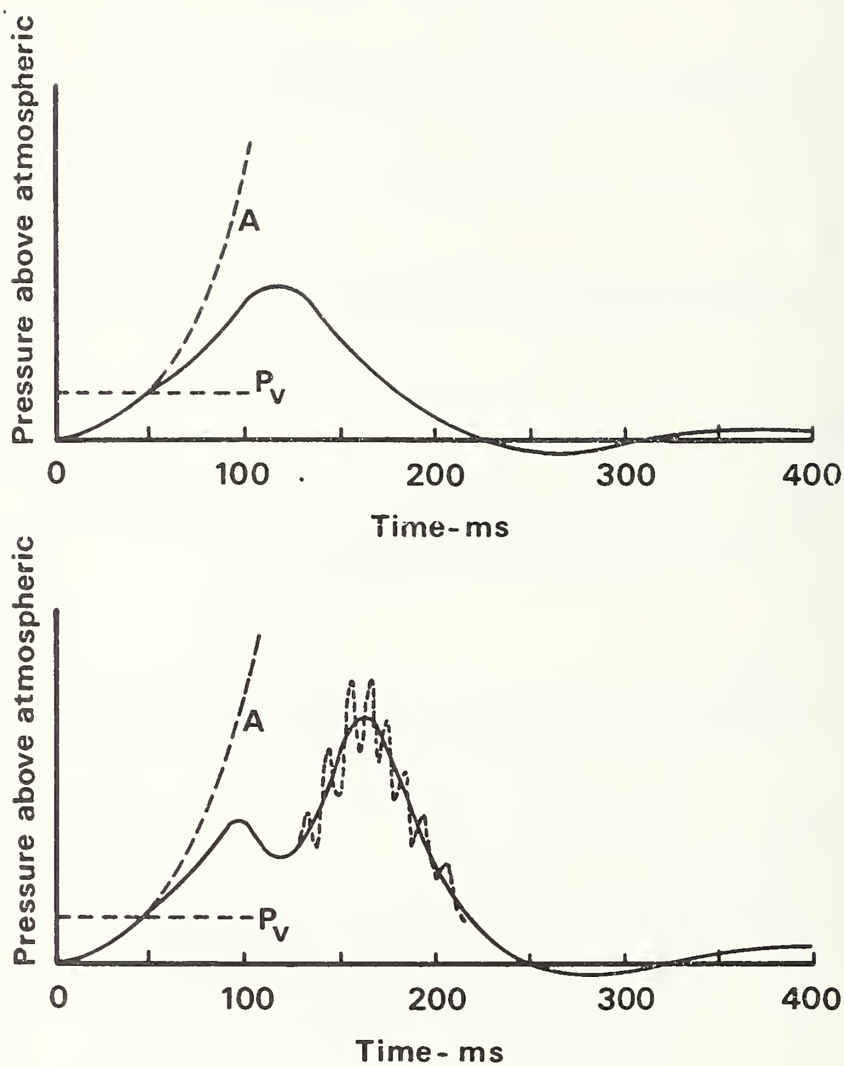

Figure 6 Pressure-time curves for typical vented gas explosions. P_V = pressure at which vents fail. Broken line A denotes the subsequent pressure rise in the absence of vents. Upper curve shows the effect of smaller vents. Time scale is only approximate. (Due to Mainstone, reference 70).

Table 5.1 Sabotage Bombing Statistics Provided by the Federal Bureau of Investigation

Line	Category of Reported Incident	Jan. 1972	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	FBI Totals for 10 Month Period	Authors Extrapolat Totals For 1 Yea
1	Actual Explosive	78	63	59	62	52	72	54	59	59	50	608	703
2	Actual Incendiary	81	50	62	80	83	79	99	83	50	41	708	805
3	Attempted Explosive	28	28	8	24	18	14	24	12	27	18	201	241
4	Attempted Incendiary	12	20	16	19	27	13	20	14	20	11	172	206
5	Actual & Attempted, Explosive and Incendiary	199	161	145	185	180	178	197	168	156	120	1,689	2,027
	Various Targets												
6	Commercial Buildings	40	29	34	41	27	35	40	47	27	24	344	413
7	Office Buildings	4	5	8	7	1	7	3	6	1	3	45	54
8	Automobiles	20	18	15	13	10	13	19	23	11	15	157	188
9	Residences*	55	53	41	49	45	55	58	47	52	26	471	566
	Sub-Categories of Residences												
10	Private Residence	50	52	36	47	38	52	48	43	39	21	426	512
11	Apartment House	5	1	5	2	7	3	10	4	3	5	45	54

*The FBI include sheds, garages, and other private property in their figure under this heading. In this case, it is simply the sum of lines 10 and 11.

NOTE: Lines 6 through 11 cover all incidents whether they be actual or attempted, explosive or incendiary.

For brevity only four main categories are given; those having the most bearing on the subject in hand. Lines 6 through 9 show figures respectively for the main categories: commercial buildings, office buildings, automobiles and residences. It is important to note that the listed category of target is that in which the incident took place. Thus, if an office unit within a residential building were the target of an incident, the occurrence would be listed under category "Office Building" not "Residences." If the unit had been a store rather than an office, the entry would be under "Commercial Building." Similarly, if an automobile parked in a basement garage to the residential building was the object of an attack, the incident would be listed under "Automobiles." This suggests that inasmuch as they could constitute abnormal loads for the building as a whole, certain incidents listed on Lines 7 through 8 might contribute to the figures defining the probability of explosive loading on buildings i.e., line 9 might constitute a lower bound to the frequency of abnormal loadings. At this time however, the readily available FBI data do not permit these incidents to be identified, and for this reason, only the figures on line 9 will be used further.

The FBI uses the term "Residences" to cover private residence, apartment house and other private property such as sheds, and garages adjoining the residential building or in its vicinity. The figures for other private property are not included in the table, however the 10-month total for this subcategory is 26.

At the present time, the FBI makes no attempt to report the number of stories in the building subjected to the attack, or the severity of the attack in terms of structural damage to the building. Such information would greatly add to the value of the statistics, at least as far as building safety is concerned. It would also be valuable to have a clear definition of the difference between a private residence and an apartment house with respect to the FBI's classification of incidents.

Probably the most useful figure to use, in order to assess the frequency of abnormal loadings due to sabotage bombing, is the 10-month total for residences, 471. To arrive at an estimate of the number that were actual explosive incidents this number is multiplied by 0.36 giving 170. If this is converted to a yearly estimate the result is 204. It appears therefore, that at the present time, 566 sabotage bombing incidents per year occur where the direct target is a housing unit. Of these, an estimated 204 can be expected to involve explosives with which detonation takes place. For reasons discussed earlier, this figure may be a lower bound; it is also conceivable that not all incidents were recorded by the FBI during this initial 10-month period of their data collection.

Corroborating evidence to support the FBI data is provided by the ICAP [18] and, on a State basis, by the California Department of Justice [19]. In the period July 1971 through February 1972, the IACP operated the National Bomb Data Center under the auspices of the U.S. Department of Justice. Since that time, the functions of the Center have been transferred to the FBI. The monthly summary reports of the Center, during the period July 1971 through February 1972, provide a brief description of each incident in addition to a statistical treatment of all bombings for the month in question. One description, taken from the January 1972 report [20], serves to illustrate the possible scale of the risk:

> *"January 5, Las Vegas, Nevada. An explosive device, consisting of 40 sticks of dynamite, was placed in the laundry room of an apartment building. The blasting cap detonated, but improper assembly of the device, and the age of the dynamite combined to produce only partial detonation. No damage resulted."*

Had the explosion occurred and had the building been of a design similar to that used at Ronan Point, the United States may have had its first "Ronan

Point" incident, with considerable political and professional repercussions.

B. Gas Explosions

This subsection deals with accidental loadings arising from explosions occurring accidentally as a result of the distribution and use of gas. Because of Ronan Point, the dynamics of gas explosions and their interaction with building construction of various forms have received considerable study in the United Kingdom. The greater part of U.K. gas is still manufactured, yet a rapidly increasing percentage is of natural origin from North Sea sources. In contrast to the U.K., the U.S. production of manufactured gas amounts to only a few percent of the total, natural gas providing almost all the needs.

B.1 Time-Magnitude-Distribution of Load

Rasbash documented [16] the maximum fundamental burning velocities of some gas-air mixtures under atmospheric conditions. Table 5.2 shows manufactured gas to burn approximately three times as rapidly as natural gas. Such data has caused certain members of the engineering profession to make some distinction between the two types of gas when considering the risk of progressive collapse.

Table 5.2 Fundamental Burning Velocities (Maximum) of Some Gas-Air Mixtures Under Atmospheric Conditions Due to Rasbash [16]

Gas or Vapor	Burning Velocity ft/s	Burning Velocity m/s
Methane (natural gas)	1.2	0.37
Butane	1.3	0.40
Hexane*	1.3	0.40
Propane	1.5	0.46
Ethylene	2.3	0.70
Manufactured Gas**	4.0	1.2
Acetylene	5.8	1.8
Hydrogen	11.0	3.4

* Similar to gasoline.

**Approximate value for manufactured gas - depends on composition.

Alexander and Hambly [21] discussed various means of reducing the consequences of gas explosions in buildings; such as, complete removal of the explosive source, forced ventilation, and control of the maximum flow of gas from the supply. They provided a qualitative description of the nature of a gas explosion in terms of gas/air mixture, presence of venting, pressure rise, turbulence, and other related parameters. They also provided a method of analysis of structures subjected to these dynamic loads.

In another paper, Alexander and Hamley [22] developed a method for design of structures to withstand dynamic loading from gaseous explosions similar to that which caused the collapse at Ronan Point. The loading pressure pulse was described only qualitatively; a precise description could not be given at that time due to the absence of relevant experimental data. The response of the structure to such dynamic loads was discussed and a method of design presented, with examples for floor, slabs, and load-bearing walls. Proposals were given for research that should be carried out to determine the design pressure pulse and to check the validity of the assumptions made.

Stretch [23] examined how explosions, caused by vapor-phase reactions between common inflammable solvents or sources of energy and air are controlled by particular features of domestic buildings, and consequently strain or damage the structure during their passage. He concluded that the general use of materials such as gas is a safe and convenient element in contemporary society, so that, today, homes are considered fit for human occupancy only if they are designed to withstand and contain, within tolerable limits, the risks incumbent on a high standard of living. He showed how inherent features of established structural systems have obscured the necessity of special precautions in more recent systems. Stretch gave simplified forms of the pressure waves and used these to analyze the behavior of buildings of traditional brick design, framed construction, and concrete panel construction, respectively.

Rasbash [16], in a paper accompanying that presented by Stretch [23], discussed the influence of potential relief of explosion pressures provided by external windows and doors during gas and vapor explosions. He provided a quantitative approach for estimating these pressures.

To define experimentally, those data needed in the mathematical modeling of gas explosions in buildings, Rasbash, Ralmer, Rogowski and Ames [24] carried out experiments in which they exploded mixtures of air and manufactured gas or natural gas respectively. These explosions took place in a strong chamber with partitions simulating the division of a building into rooms.

Further experiments were carried out by Astbury, West, and Hodgkinson [25] to investigate the effect of different gas layering conditions in a pair of rooms. During the tests, different layers of both gas and a gas/air mixture were used. In two of these experiments, the most explosive (stoichiometric) manufactured gas/air mixtures were obtained and the resulting explosion caused, in one case, minor, and in the other case, major damage to the 3 1/2-story building which was of load-bearing brick. These experiments were a repeat of earlier tests in which an attempt was made to demonstrate the effects of turbulence. Turbulence has the effect of increasing the pressures developed when an explosion proceeds from one room to another filled with gas, namely, the "cascade" effect. The test demonstrated the effectiveness of venting in limiting the maximum pressure developed in an explosion. Despite suffering the extensive damage, the brick building could be safely propped and no progressive collapse occurred.

West [26] carried out tests involving the effect of gas explosions on windows of various details in order to study their effectiveness in providing venting. Specimens included single-glazed windows of 32 oz glass and double-glazed units of the same thickness of glass. Because of their higher strength, the double-glazed windows provided ineffective venting. Further, the resistance of glass to short-term loads (defined as lasting 3 seconds) is more than twice that under sustained loadings. Repeated explosions that do not break the glass may be the cause of eventual failure at a lower pressure. The strength of glass decreases with time. For example, glass eighteen months old failed at loads some 20 percent less than those obtained with newly-manufactured glass. Finally, while the failure of glass may give an indication of the pressure developed in a real incident, care is necessary in interpreting test results since a distortion of the frame can cause the glass to break at a pressure less than its actual breaking strength.

Mainstone [27] reviewed the existing experimental data on the strength of glass under loads of very short duration. He provided the basis for a graphic presentation of likely breaking pressures, under gas-explosion loading, for particular sizes and thicknesses of window panes. An earlier review by Rasbash of data on the venting of gas explosions was then used as the basis for extending the graphic presentation to cover also the possible rise in pressure after the glass is broken by an explosion. The graphic presentations can be used directly for estimating the pressure reached in actual explosions from observations on the damage to glass windows; and they were prepared primarily to be used in the design of glazing as an explosion vent.

On the basis of a review of the above studies it is concluded that there is sufficient data available to allow satisfactory prediction of the characteristics of gas explosions in buildings providing the gas mixtures can be defined.

B.2 Probability and Consequence of Load

The probability of gas explosions has been studied both in the United States and the United Kingdom. Whereas the Ronan Point incident did not occur until 1968, the American Gas Association, which represents approximately 85 percent of the U.S. gas industry, had compiled statistics of gas incidents some years earlier. Before reviewing the results of the AGA studies, it will be useful to consider the studies in the U.K. to gain perspective.

In the Report of the Inquiry into the collapse at Ronan Point, Griffiths, Pugsley and Saunders [1] assembled the data available at that time dealing with the probability of gas explosions in the U.K. Table 5.3 is taken from the report of Griffiths et al and contains an analysis of explosions in housing for each of the years 1957 through 1966.

Structural damage is defined as damage to the structure over and above the mere blowing out of windows and window frames. It will be seen from Tables 5.3 and 5.4 that, of the known causes of explosions, manufactured gas is the principal hazard. In the year 1966 there were approximately 18 million housing units (apartments and houses) in the United Kindgom and, of these, approximately 12,260,000 were supplied with manufactured gas. The 1966 figures show that the frequency of explosions involving manufactured gas in premises supplied with gas is approximately 8 per million dwellings, of which 3.5 per million will be of sufficient violence to cause structural damage. Griffiths et al assessed the chance of a gas explosion in a high apartment building. In a building the size of Ronan Point, with 110 apartments and a life of 60 years, there is slightly more than a 2 percent risk that a gas explosion causing structural damage will occur in one of the apartments during the lifetime of the building, i.e., $3.5 \times 10^{-6} \times 110 \times 60 \times 100 = 2.31$ percent. In other words, the chances are that of every 50 such buildings, one will experience structural damage as a result of a gas explosion in its lifetime. They pointed out that, whereas it may be argued that it is cheaper to prohibit the use of gas in tall apartment buildings than to make the structures free from the risk of progressive collapse, they did not accept this argument. They reasoned that gas is justifiably regarded as a safe and acceptable fuel in domestic premises generally [28]. Furthermore, the banning of gas would not, of course, completely eliminate the risk of damage to the structure of a tall apartment building, resulting in progressive collapse, although admittedly it would remove the most likely cause (sic). There remained the possibility

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Table 5.3

Frequencies of Explosions in Housing Units Estimated from Samples of Fire Department Reports in the United Kingdom - Damage and Explosive Material Reproduced from [1]

Year	Sampling factor	Total explosive	Manufactured Gas			Liquefied Petroleum Gases			Liquids			Other and unknown
			Total	Superficial	Structural	Total	Superficial	Structural	Total	Superficial	Structural	
1966	1/1	213	97	55	42	14	8	6	33	25	8	69
65	1/1	181	76	40	36	14	8	6	29	20	9	62
64	1/2	168	80	28	52	8	—	8	18	16	2	62
63	1/6	216	84	54	30	6	—	6	18	12	6	108
62	1/2	234	70	20	50	8	2	6	34	28	6	122
61	1/2	198	46	28	18	10	4	6	38	30	8	104
60	1/4	144	72	36	36	16	—	16	12	8	4	44
59	1/4	148	88	24	64	—	—	—	24	20	4	36
58	1/4	192	64	28	36	12	—	12	48	32	16	68
57	1/1	195	70	41	29	8	3	5	44	35	9	73
Total	—	1,889	747	354	393	96	25	71	298	226	72	748

Table 5.4

Explosions in Domestic Premises in the United Kingdom

Reproduced from [1]

Total Explosions	Explosive Material		Damage		Fault	
213	Manufactured gas	97	Superficial	55	Installation	35
					User	20
					Unknown	0
			Structural	42	Installation	26
					User	9
					Unknown	7
	L.P.G. (Liquefied petroleum gases)	14	Superficial	8	Installation	3
					User	4
					Unknown	1
			Structural	6	Installation	5
					User	0
					Unknown	1
	Liquids	33	Superficial	25	Installation	6
					User	17
					Unknown	2
			Structural	8	Installation	0
					User	7
					Unknown	1
	Other and Unknown	69	---		---	

of explosions caused by substances other than manufactured gas, for example, gasoline or other volatile inflammable liquids, butane gas cylinders, electrical apparatus, and so on; as well as other forms of accidental damage (sic).

Prompted by the Ronan Point incident, the U.K. Construction Industry Research and Information Association (CIRIA) established a pilot survey [29] to establish the procedures for future and wider surveys on the frequency of gas explosions and the structural damage they can cause. Newspapers were used to obtain reports of explosions in residences and, where appropriate, visits were made to damaged properties and comprehensive investigation of the circumstances carried out. Findings showed that gaseous explosions in U.K. housing causing significant structural damage, occur at the rate of less than one per week.

Fry [30] examined U.K. fire incidents involving explosions of manufactured gas in dwellings during the 13 years, 1957 to 1969. The average annual incidence was shown to be approximately 90 but appeared to be increasing as the consumption of gas increases. The average rate of incidents is about 5.0 per 10^8 therms of gas sold. Approximately 48 percent of the incidents cause some structural damage and in 40 percent of these it was considered "severe." From reports involving manufactured gas and natural gas in 1969, it appeared that natural gas was more likely to cause explosions but that the explosions were of similar violence for the two types of gas.

A later report [31] of the field survey of damage, caused by gaseous explosions in the U.K. contained the conclusion that roughly one severe or very severe explosion occurs every two weeks. Furthermore, there is some evidence that the frequency is increasing.

It is now useful to discuss the probability of gas explosions within the Continental USA. The two principal sources of information on the gas industry are the American Gas Association and the Office of Pipeline Safety (OPS) of the U.S. Department of Transportation. The following is an attempt to evaluate information from each of these sources.

Within the Continental USA there are 915,000 miles of gas pipeline [32]. This total is composed of 288,000 miles of pipeline in gathering and transmission systems and 627,000 miles of pipeline involved in subsequent distribution of gas [33]. Since the former are located largely in rural areas, it is with gas distribution systems that this study is primarily concerned. It is significant that, within the gas distribution system, gas leaks are reported to occur at an annual rate of 1 per 1.1 mile of gas line; e.g., a total of more than 560,000 leaks in 1972. More than 300,000 of these leaks occurred as a consequence of normal wear and tear; i.e., corrosion, fatigue, material failure, etc. Leaks are reported [33] to occur with comparable frequency in both the mains and service lines and about two-thirds of all leaks were associated with the pipeline itself, the remainder being associated with the fittings and other attachments.

Including both single and multiple dwelling units, almost 34.7 x 10^6 housing units in the USA used gas for house heating in 1970 [32]. It is also estimated [5] that natural gas serves about 55 percent of all housing units as a fuel for residential space heating and it is significant that of all new customers for house heating in 1970, 37 percent of them were conversions to a gas service system. Because a single customer or meter or furnace may involve more than one dwelling unit, these figures may not be fully representative of the total number of dwelling units to which gas is supplied. The AGA estimates that, including appliance usage, natural gas is supplied to upwards of 60 percent of the total residential market.

A prime source of information on gas leak incidents is the summary to the research report "Public Safety and Gas Distribution" [34] prepared for the

American Gas Association by Arthur D. Little, Inc., and dated December 1967. This was a survey of the gas distribution industry in which 140 companies and systems participated, representing 83.3 percent of all gas distribution meters. The yearly averages for gas-related incidents are based on the 10-year period 1957-66. As far as incidents involving payment of compensation are concerned, the yearly averages are representative of the 7 year period 1957-1963. Both in terms of the time periods and the number of companies involved, this survey constitutes the most comprehensive, if not the only, study of the overall safety of the gas distribution system within the continental U.S.

With reference to Table 5.5, assembled from information contained in the A. D. Little, Inc., summary, it is evident that 1,508 gas explosions can be expected in an average year and, of these, 329 will require monetary compensation. A total of 151 (60 percent of 253) explosions will involve payments of more than $1,000. It is recognized that only a proportion of these explosions could have been severe enough to cause structural damage, but unfortunately, there is no record of:

1. The relative proportion of personal and property damage,

2. Whether or not one or more buildings were involved and, if so, the type of building, and

3. The relative severity of each of the incidents involving payment greater than $1,000.

It may be presumed that of the explosions incurring payment of less than $1,000, very few, if any, were likely to have involved significant structural damage. In order to evaluate a probability based upon the AGA statistics, it will be assumed that only one building was involved in each of the 151 incidents and that each suffered significant structural damage. This assumption can be seriously questioned as the following illustration from an OPS report [35] would indicate but in the absence of more specific data, it at least provides the basis of a conservative estimate:

> "Mobile Oil Corporation High Pressure Natural Gas Pipeline
> Houston, Texas, September 9, 1969 - Synopsis
>
> At 3:40 p.m. on September 9, 1969, the 14-in pipeline carrying natural gas at a pressure of more than 780 psig ruptured in a newly constructed residential subdivision 3 1/4 miles north of Houston, Texas. The escaping gas created a dust storm like condition and sounded like a jet engine. Electrical and telephone utility servicemen working in the area, with the help of local residents, immediately commenced to evacuate all residents in the vicinity of the rupture. About 8 or 10 minutes later, the escaping gas exploded violently. Thirteen houses, ranging from twenty-four ft to 250 ft from the rupture were destroyed by the blast. The leaking gas caught fire and continued to burn to a height of 125 ft for 1-1/2 hours until valves on the other side of the leak were closed by Mobil workmen dispatched to the valve locations. The fire abated at that time, but some gas burned for another five hours. In all, 106 houses were damaged and property damage was estimated at $500,000. Miraculously, there were no deaths but nine people were injured, two seriously."

Office of Pipeline Safety figures for 1971 [35] indicate that, of the total number of reported explosions, 50 percent occurred in residential

Table 5.5

Gas Related Incidents. Their Nature, Annual Incidence and Consequence (Taken from A. D. Little Summary Report to American Gas Association [34]

	10 YEAR AVERAGE		7 YEAR AVERAGE - 1957-63 INCLUSIVE				
NATURE OF INCIDENT	INCIDENT REPORTED		INCIDENTS INVOLVING PAYMENT*		SIZE OF PAYMENT		
					>$1,000	<$1,000 >$ 25.	< $25.
	NUMBER	%	NUMBER	%	%	%	%
EXPLOSION	1508	12.8	329	13.3	60	10	4
FIRE	1835	15.6	274	11.1	36	9	7
PRODUCTS OF COMBUSTION	1228	10.4	109	4.4	7	5	3
UNIGNITED GAS	3118	26.5	1498	60.7	12	66	69
FLASHBACK	4122	35.1	217	8.8	6	9	11
OTHER	344	2.9	167	6.8	3	7	8
		103.3+		105.1+	124+	106+	102+
TOTAL NUMBER OF INCIDENTS	11753		2466		253 / 10.3%	1607 / 65.1%	606 / 24.6%

* Total Payments by Gas Company exclusive of Company repair costs.

\+ Some incidents may involve more than one of the listed phenomena.

buildings and 7 percent in commercial buildings. The remainder occurred in manholes, regulator pits, etc., and it is unlikely that these involved compensation in excess of $1,000 since, in most instances, these incidents occurred on or within gas company property, it is unlikely that they involved the gas company in claims for payment. Accordingly, it is presumed that 50/57, namely 87.5 percent of the 151 incidents, namely 131, were likely to have involved residential buildings.

The period over which the AGA statistics were gathered centers approximately on 1960. According to the 1960 Census of Housing [36], the occupied housing in the U.S. totaled 58,314,784 units. Accordingly, a crude estimate of the annual probability of occurrence of a structurally significant gas-related explosion in a residential unit is of the order of $131/58,314,784 \times 100 = .00022$ percent or 2.2 structurally significant explosions per million dwelling units per year.

During the period 1957-1963, the average annual natural gas sales to residential customers totaled 300×10^8 therms. In terms of energy, the annual rate of structurally significant gas-related explosions is 0.44 for every 10^8 therms of gas supplied. If all gas-related explosions and the total amount of natural gas sales are considered, the probability of explosion is 1.67 explosions per 10^8 therms per year.

In accordance with the provisions of the National Gas Pipeline Safety Act of 1968 [33], a nationwide reporting system for gas-related incidents was initiated in 1970 by the Office of Pipeline Safety (OPS). Detailed reports are required of individual incidents that involved one or more of the following criteria:

1. Caused a death or a personal injury requiring hospitalization;

2. Required any segment of transmission pipeline to be taken out of service;

3. Resulted in gas igniting;

4. Caused estimated damage to the property of the operator, or others, or both, of a total of $5,000 or more;

5. Required immediate repair and other emergency action such as evacuation of a building, blocking off an area, rerouting of traffic to protect the public; and

6. Were deemed significant but did not meet the criteria of 2, 3 or 4.

All gas companies with more than 100,000 customers (over 85% of the total number of gas customers) are required to submit reports within 30 days of the incident. Records for 11 months of 1970 and all of 1971 are publicly available [33].

In an NBS Technical Note entitled "Residential Buildings and Gas-Related Explosions" [37] the AGA and OPS statistics are fully discussed. It is shown that at this point in time the AGA and OPS statistics are in conflict and of the two the AGA statistics would appear to be the more representative since:

1. They attempt to cover all incidents, both upstream and downstream of the meter in distribution systems (see Table 5.6),

2. They represent a 10-year average whereas the first complete year of OPS statistics is 1971.

In that report, it is concluded that, though due regard has to be taken

Table 5.6

Relative Location of the Basic Cause of the Various Gas-Related Incidents in a Typical Year (Taken from A. D. Little Summary Report to American Gas Association [34])

	10 YEAR AVERAGE		7 YEAR AVERAGE	
LOCATION OF BASIC CAUSE OF INCIDENT	TOTAL NUMBER OF INCIDENTS		TOTAL NUMBER OF INCIDENTS INVOLVING PAYMENT	
	NUMBER	%	NUMBER	%
BEFORE OR AT THE METER I.E., UPSTREAM	3,122	26.5	1,780	72.1
AFTER THE METER I.E., DOWNSTREAM	8,296	70.6	639	25.9
OTHER	335	2.9	47	2.0
TOTAL NUMBER OF INCIDENTS PER YEAR	11,753	100%	2,466	100%.

for the limitations inherent in the available statistics, the probabilities of the occurrence:

1. Of a gas explosion capable of causing significant structural damage is 2.2 per million housing units per year.

2. Of a gas explosion capable of causing significant structural damage in a hundred unit apartment building during a 50-year service life is 0.011 or 1.1 percent.

C. Explosions of Hazardous Materials

The National Bureau of Standards is presently gathering statistics of explosive incidents arising from the normal transportation of hazardous materials in urban areas by road, rail, and waterway. Such materials include petroleum and its products, chemicals, explosives, and liquified gases. Approximately 20 percent of all hazardous materials transported in the USA are moved on waterway [38]. Waterway explosions of such a nature that buildings are damaged in the course of the incident are rare, yet the potential for such an incident is not only present but increasing, particularly as the tonnage of petroleum and petrochemicals shipped by water increases. Another source of potentially extreme loading exists where fuel transportation terminals are located. Such terminals provide a storage facility for large volumes of petroleum which have been transported on waterways. Storage tanks are now located in many densely populated areas including Providence, Boston, Staten Island, and Philadelphia; more are planned.

It would appear that the shipment of hazardous materials, by road and rail, poses a problem that is not negligible. The explosion of unconfined vapor clouds, produced by the dispersion of flammable liquid or vapor spills, is becoming a serious problem contends Strehlow [39]. He points out that this is mainly because of the increased size of the spills in recent years. Illustrating this point is the accident that occurred in East St. Louis, Missouri on January 22, 1972. Since the National Transportation Safety Board is currently investigating the incident, and the report is not yet released, the following information was obtained from a file of newspaper clippings maintained by the Safety Board; it should not be considered official:

The accident occurred in an East St. Louis railroad yard where a process known as "humping" was taking place. In this process, a railroad car is allowed to roll freely toward other coupled cars with sufficient momentum to allow the couplers to fasten. This is a process regularly used to make up trains. Alledgedly, in this instance not 1 but 4 already coupled cars were being humped. The lead car, filled with 500,000 lb (30,000 gallon) of propylene, a derivative of liquid petroleum gas (LPG), was travelling at approximately 15 mph, instead of the recommended 6-7 mph, when it bumped a hopper car at the end of the partly-assembled train. The total momentum of the impact was such that the hopper car coupler jumped over the coupler of the lead rail car and punctured its propylene tank. The cars continued to move for several hundred feet. The released propylene formed an unconfined vapor cloud which exploded 500 ft. from the location of the leaking car.

The explosion shook a 4 square mile area and shattered windows 8 miles away; the concussion was felt up to 20 miles from the scene of the accident. The explosion caused roofs and walls to collapse as much as 6 blocks away. At least 3 separate fires followed the explosion and involved 30 additional railroad cars. Apparently none of these additional cars contained hazardous materials, for *no further explosions were reported.*

Early reports estimated that 1,000 buildings had been damaged including 650 homes and 350 business buildings. Later information indicated that 868 buildings had been reported as damaged. One hundred families were left homeless. In all 176 people were injured and total non-railroad property damage was estimated to be 6.5 million dollars.

It is estimated by the National Transportation Safety Board that the report will be completed by January 1973. Strehlow sites a 1962 accident in the State of New York that involved a truck:

July 26, 1962 New Berlin, New York [40]. 7,000 gallon tank truck in a single truck accident. Tank ruptured catastrophically in town. Vapor cloud covered 200,000 square feet and was 80 ft deep before ignition. Explosion and following fire killed 10, caused $200,000 damage.

Strehlow states that the characteristics of the initial fire or explosion, which follows the ignition of a spill, depend on four things:

1) The nature of the fuel.

2) The rapidity of the spill coupled with the wind conditions, terrain and/or location of nearby buildings.

3) The delay before an ignition source is found.

4) The nature of the ignition source.

He concludes that current theoretical results are unable to predict the observed pressure-time behaviors.

Strehlow [39] has tabulated 108 accidental unconfined vapor cloud explosions that have been documented over the past 42 years. The list is not complete because it is limited almost entirely to explosions that have occurred in the USA and Germany, because the documentation of individual explosions has often been fragmentary, and because information about many on-site plant explosions are not accessible to the general public. Using Strehlow's data, Figure 7 has been plotted showing the average number of explosions per year over periods of 2, 5 or 10 years. It is seen that the number per year is increasing rapidly. In approximately two years, 1970 through January 1972, a total of 15 incidents occurred, causing damage estimated at 23-27 million dollars. It appears from Strehlow's paper that, with one exception, these occurred in the United States.

How many housing units were subjected to severe blast loading is not known with any certainty, but it is clear from the incident at East St. Louis, January 22, 1972, described earlier, that the number would be a significant one. The reports of the incident, which has been previously discussed, estimated that 650 homes were damaged and 100 families were left homeless. It would appear that a lower bound to the number of housing units subjected to abnormal loading due to vapor cloud explosion was 100. The non-railroad loss was 6.5 million dollars, out of an estimated total of 23-27 million dollars for the last two year period shown in Figure 7. If it is assumed that the number of housing units affected in two years can be arrived at by prorating, the figures for St. Louis then this number would be approximately

$$100 \times \frac{23}{6.5} = 354 \text{ housing units}$$

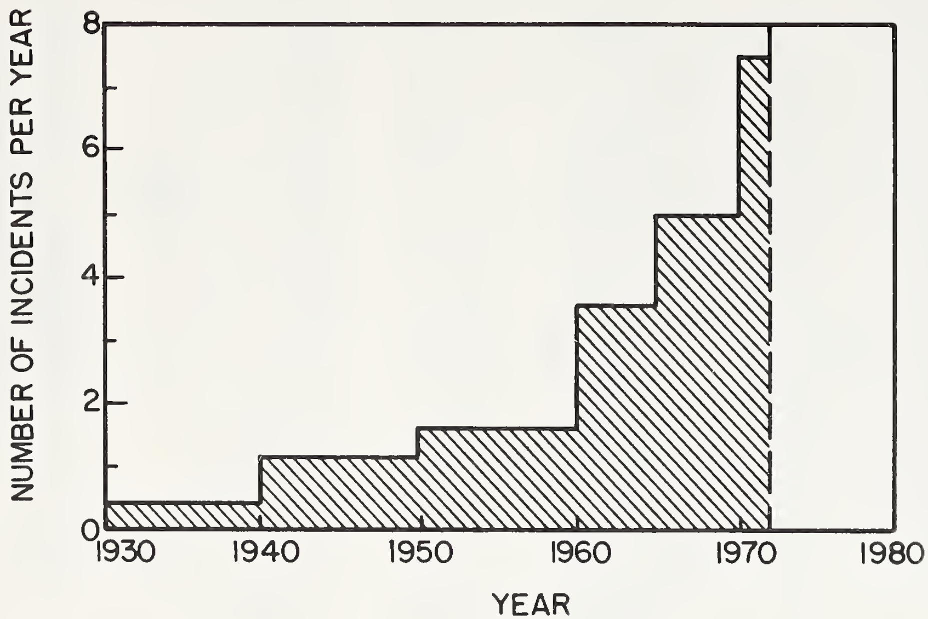

Figure 7 Histogram of unconfined vapor cloud explosions, 1930 through January 1972, based on data compiled by Strehlow [39] and principally based on incidents occurring in the United States and Germany.

On this basis, albeit a crude one, it could be expected that 354/2, namely 177 incidents occur per year unless some change in the incidence of accidental unconfined vapor clouds were to take place.

C.1 Highway Vehicle Impact

Sanders [41] has developed relationships to aid the structural designer in estimating the probability of a structural component being struck by an errant motor vehicle. An example of this type of incident would be a motor vehicle straying from its normal path and hitting a building column. The probability of failure of the column is derived from combining the probabilities: (a) of a vehicle striking the column and (b) that the vehicle has the mass, velocity, and stiffness to cause the failure of the column. The probability of a motor vehicle striking an object located near a traffic path is derived as a function of (a) the distance of the object from the traffic path and (b) the volume of traffic flow.

For the USA in 1970, the total number of motor vehicle accidents of all types was approximately 16 million [71]. Highway accident statistics are compiled by each State and the categories of information on the reporting forms frequently total as many as 60 items. In spite of this, incidents in which vehicles collide with buildings receive scant coverage. Indeed, it is the exception for the report to identify the object struck when a vehicle leaves the highway. The U.S. Department of Transportation has assisted NBS in identifying two states in which building strikes were considered in the data collection. These States are Oklahoma and Illinois, the former having relatively little urban area while the second has large urban concentrations. In each case the data analyzed were those for 1970.

Throughout the year 1970, there were 65,183 motor vehicle accidents in Oklahoma. In 50 of these a vehicle was reported to have collided with a building. In Illinois in 1970, there were 409,174 motor vehicle accidents and of these, the number in which a vehicle was reported to have collided with a building, is 1229.

If the following assumptions are made: 1) the Oklahoma data, added to the Illinois data, is representative of the USA as a whole; 2) the number of vehicle collisions with buildings (C) is proportional to the population (P), then based on 1970 data:

$$\frac{C_{US}}{P_{US}} = \frac{C_{ILL} + C_{OKLA}}{P_{ILL} + P_{OKLA}}$$

$$C_{US} = \frac{1279}{13,674,000} \times 203,212,000$$

$$C_{US} = 19,000 \text{ (approximately)}$$

This estimate is for the number of vehicle collisions with buildings for the USA as a whole in 1970, based upon combined Oklahoma and Illinois data.

It is interesting to differentiate between urban and rural situations, where urban is used to denote an incorporated area having a population of 2500 or more.

URBAN: $$C_{US} = \frac{1171}{10,970,000} \times 149,325,000$$

$$= 16,000 \text{ (approximately)}$$

$$\text{RURAL:} \quad C_{US} = \frac{108}{2,704,000} \times 53,887,000$$

$$C_{US} = 3000 \text{ (approximately)}$$

On this basis, it is estimated that, nationally, the 1970 figure for vehicle-building collisions is 16,000 in urban areas and 3,000 in rural areas.

So far the discussion has dealt with reported collisions of vehicles with buildings, with no consideration for what is meant by 1) vehicle collision, and 2) building. Vehicles vary widely in mass, and collisions can range from a gentle nudge to an impact capable of removing a structural element. The term building covers a range of structures from barns to multistory apartment houses. As far as this study is concerned, all standard reporting forms are unsatisfactory in these respects. Efforts to break out the particularly-relevant data are continuing; however, it seems unlikely that the probability of abnormal loading of housing units due to highway vehicle impact will be defined with any clarity unless: 1) existing data recording procedures used by the States are slightly modified, 2) a separate detailed study is made (this could be based on statistical sampling and involve relatively small areas of the country).

However, with due regard to the uncertainties inherent in the data available, a lower bound estimate can be made of the number of abnormal loads per year on residential structures subjected to highway vehicle impact. From a study of what detailed information is available, it is presumed that at the national level at least one tenth of the incidents will involve residences and, of these, one tenth will involve impact sufficiently large to be considered an abnormal loading.

On this basis a lower bound estimate of the number of abnormal loadings on residential buildings per year due to highway vehicle impact is 190 or, on the average, four per State.

C.2 Aircraft Collisions With Buildings

In comparison with other forms of abnormal loading the probability of such incidents is trivial. Using data furnished by the U.S. Federal Aviation Authority, the NBS study has shown that for buildings located further than 3 miles from the end of an airport runway the probability of collision with buildings per year is approximately 10^{-8}.

VI. Summary of the Probabilities of Abnormal Loadings And Their Implications for the USA

Chapter 5 discussed the available statistics from which preliminary estimates have been made of the annual incidence of five sources of abnormal loading on residential buildings. Reference to Chapter 4 will show several abnormal loadings for which data has not been presented. Efforts to quantify the probability of occurrence of these other loadings will continue; however, it should be recognized that for several reasons this quantification may never be satisfactorily accomplished.

In arriving at figures in Chapter 5, it has been the author's intention to err on the low side in cases of doubt about the available data. For this reason and because not all sources of loading are included, it is believed that the following summation provides a lower bound result.

Source	Number of Abnormal Loadings Per Year
Explosive Bombing	204
Gas Explosion	131
Explosion of Hazardous Materials	177
Highway Vehicle Impact	190
Summation	702

Chapter 4 discussed the national risk due to fire as a basis for assessing the incidence of abnormal loadings in housing units. It was shown that architectural layout and building size affected the number of abnormal loadings on housing units per year that, in buildings susceptible to progressive collapse, would pose the same risk as fire. For a 100 unit building, with 4 apartments per story, an average of 4 persons per apartment, and a central service core, this number correspond was 433, while for a 50 unit building of similar arrangement the corresponding number would be 866.

Comparing these figures with the lower bound estimate of 702 abnormal loadings a year, it appears that, for buildings susceptible to progressive collapse, abnormal loadings do pose a risk comparable to that of fire. In the case of the fire risk, specific design criteria are currently implemented.

The United Kingdom Authorities introduced criteria to minimize the risk of progressive collapse upon concluding that the frequency of explosions, involving gas in housing units supplied with gas, was approximately 8 per million housing units per year, of which, 3.5 per million were of sufficient violence to cause structural damage. It should be noted that this figure relates to only one source of abnormal loading.

In the USA, the frequency of abnormal loadings on housing units per year appears to be in excess of $702/(67.7 \times 10^6)$, namely approximately 10 per million. This data appears to provide adequate justification and need for standards-writing bodies to adopt criteria explicitly to deal with abnormal loadings and progressive collapse.

VII. Alternative Approaches for Criteria

Each of the following approaches, or some combination, may be used in the preparation of criteria to reduce the level of risk from abnormal loadings:

Eliminate Source

Reduce Magnitude

Limit Extent of Structural Damage

Resist Local Structural Damage

The banning of gas from buildings would illustrate an attempt to partially implement the first of these approaches. The second is illustrated by regulations that might call for buildings to be sited at distances from a highway and with such barriers that impact from vehicles becomes unlikely. The provision of alternate paths for loads in the event of the loss of a critical member illustrates the third approach. The fourth approach is the basis of the U.K. criterion in which the element is designed to resist 5 psi pressure in any direction, applied to the element and to those elements attached to it.

For the most part, the first two approaches are non-structural.

A. Non-Structural Design Criteria

It could also be feasible to consider introducing criteria in non-structural areas including the following:

Zoning, Siting, Planning
Regulations for Service Systems
Transportation Regulations

A.1 Zoning, Siting, and Planning Criteria

The following are examples of areas in which criteria could be considered for development or modification:

1. The location of gas mains in urban areas.

2. The location of multistory buildings with respect to highways, railroads, and waterways.

3. The location of multistory buildings with respect to similar buildings.

4. The zoning of land for residential use, taking into account the possibility of local external explosions.

A.2 Regulations for Service Systems

The following are features of service systems that might be studied with a view to reducing risk:

1. The location of high pressure gas riser mains in multistory buildings.

2. The location of boiler rooms and furnaces and HVAC plant in general.

3. Ventilation provisions for gas mains, boilers, and furnaces.

4. Specifications for pipe work and fittings.

5. Maintenance and inspection procedures for plumbing installations.

As an illustration, French regulations [42] prohibit the use of gas in buildings higher than 50 meters (164 feet), and this law was enforced before the Ronan Point collapse. Furthermore, when gas is installed in buildings, French codes have requirements for ventilation that are more stringent than most codes; e.g., the gas supply pipes are enclosed in a ventilated duct space.

A.3 Transportation Regulations

Considerable volumes of hazardous materials are transported through urban areas in the U.S. by means of truck, railcar, and waterway traffic. Minimization of the risk incurred might come about from new or improved regulations following a study of:

1. The type and volume of hazardous materials transported as one cargo.

2. The engineering specifications used in the regulation of the design and maintenance of vehicles used in this transportation, the standard procedures for the operation of these vehicles, and the procedures for the supervision of operatives.

3. The routes over which this transportation is permitted to take place and, in particular, the proximity to urban areas.

4. The statistics for explosions of hazardous cargoes in shipment.

VIII. Philosophies for Structural Criteria

Criteria to minimize progressive collapse are being implemented in several countries including the Nordic Countries, Countries of Eastern Europe, France, Italy, United Kingdom, the United States and Canada. A detailed review of the contents of these various criteria is beyond the scope of this paper. It is sufficient and more appropriate to identify the various philosophies that have been used in the preparation of these criteria:

1. The Cautionary Note: Whereby the attention of the structural engineer is drawn to the risk of progressive collapse and he is urged to take precautionary measures to deal with it, as in the National Building Code of Canada, 1970 Edition [43] which states that: "Buildings and structural systems shall provide such structural integrity that the hazards associated with progressive collapse, due to local failure caused by severe overloads or abnormal loads, not specifically covered in the section, are reduced to a level commensurate with good engineering practice." The 1972 American National Standards Institute A58, Minimum Design Loads in Buildings and Other Structures [3], also gives a cautionary note as follows: "Progressive Collapse - Buildings and structural systems shall provide such structural integrity that the hazards associated with progressive collapse such as that due to local failure caused by severe overloads or abnormal loads not specifically covered herein are reduced to a level consistent with good engineering practice."

2. The Alternate Path Approach: Criteria of this form call upon the designer to consider successively, the removal of structural elements or combinations thereof from the building in a systematic manner, and insure thereby through analysis and design, that the building remains capable of withstanding a specified combination of loads albeit with a small load factor.

3. Specified Abnormal Loadings: In this approach, an equivalent static loading consisting of a uniform pressure is assumed to envelope the loading states that might be produced by the various abnormal loadings on buildings. A structural element or combination of such elements, that may not be removed in accordance with the previous approach, is required to withstand the application of this pressure to its surface and those surfaces of elements attached to it (subject to the strength of their connections). An example of this approach is provided by the United Kingdom Regulations [44].

4. Specifications for Reinforcement and Reinforced Connections: This

approach removes from the structural designer the responsibility for considering successively the application of abnormal loadings to various portions of the structure. It does so by specifying reinforcement for elements and for their connection in such a way as to insure an adequate strength and ductility in the event of abnormal loadings. An example of this is provided by the CEB Regulations [45] used in France and in the soon-to-be ratified Unified Code in the United Kingdom [46].

5. Deemed to Satisfy Clauses: Finally, there have emerged a number of documents in which it is stated that design in accordance with certain national standards is deemed to satisfy the intent of other standards dealing explicitly with the risk of progressive collapse. Examples of this include BS 449, Design for Steel Structures, in the United Kingdom [47].

In the United States there have been two cases where criteria have been developed explicitly to deal with progressive collapse, albeit applied in a limited manner. One of these involved the preparation of the Guide Criteria [5] for use in the evaluation of the housing systems demonstrated as a part of the Operation BREAKTHROUGH program. These criteria were prepared by the National Bureau of Standards on behalf of the Department of Housing and Urban Development. Expressed in performance language, they were an adaptation of the Fifth Amendment to the U.K. Building Regulations 1970 [48], and expressed both the alternate path approach and the use (where absolutely necessary) of the specified abnormal loading approach (5 psi). These criteria were used in the evaluation of multistory systems in the BREAKTHROUGH program.

The second set of criteria [6] was prepared by the Federal Housing Administration of the Department of Housing and Urban Development for use in the preparation of Structural Engineering Bulletins for those industrialized systems being designed and built with a FHA mortgage guarantees. Essentially, FHA based these criteria on the British Standard Code of Practice 116, Addendum No. 1 for the Design of Large Concrete Panels [44].

There is no shortage of literature discussing the reaction of the building community to the introduction of criteria against progressive collapse in high-rise buildings. The reader is referred to Collins [49], Short and Miles [50], Rodin [51, 52], Creasy [53], Ferahian [54, 55], Lewicki [56, 57], and a collection of the discussions of structural engineers at a special meeting on the subject [58, 59]. It is a subject which has generated a great deal of interest among structural engineers.

IX. Building Response to Abnormal Loadings

This section will deal with the response of buildings to abnormal loadings from three standpoints: cases of progressive collapse, experimental studies, and analyses.

A. Cases of Progressive Collapse

A number of people including Griffiths [1], Rodin [52], and Ferahian [54] have provided considerable insight into the behavior of the Ronan Point building. Slack [60], has discussed two other instances of explosions in reinforced concrete buildings, albeit factory buildings. The first case study is concerned with an explosion in a 4-story cast-in-place framed structure, while the second illustrates the nature of blast damage within a single-story precast concrete framed building. In the cast-in-place structure the progression of the explosion pressure wave is traced and the resulting damage to the external cladding as well as fire damage are described. The precast

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concrete frame building suffered column damage due to the confinement of the primary explosion. In both of the buildings, large areas of cladding or their connections, were weaker than the main structural frame, resulting in limited overall damage to the buildings. From the first case, it appears that the framed cast-in-place structure had a greater inherent resistance to damage than would be indicated by a straight-forward analysis taking account of structural continuity. In the precast frame structure, it was found that a precast frame can have a greater resistance to collapse than would seem apparent from a basic reinforced concrete design analysis. However, it is Slack's recommendation that full or partial continuity at the cast-in-place joint be provided to give individual members greater resistance to damage.

A short report [61] was given on the bombing of an Army Officer's building in Aldershot near London in February 1972. The Irish Republican Army claimed to have carried out the bombing using 280 pounds of gelignite. The building did not collapse. The walls of one-half of the building were blown away completely, but the structural frame of reinforced concrete remained intact.

A gas explosion destroyed a row of 22 single-story shops in Clarkston, Glasgow, Scotland. Below the shops, were a series of basement voids into which gas seeped from a nearby broken main, and concentrated until it was ignited. The explosive pressure has been estimated to be at least 7.3 psi (and possibly as high as 14.6 psi). The maximum damage occurred about half way along the row. At the location of the explosion, floor slabs were lifted and the collapse of the front row of columns was attributed to the failure of 12 in by 18 in reinforced concrete beams and cross beams at ground level.

On the night of March 6, 1972, an explosion occurred in an 11-story apartment building in Barcelona, Spain, resulting in the collapse of a portion of the building (See Figure 8) and the deaths of 18 people and many injuries. It was concluded [8] that the collapse was the result of a conventional explosive. Basically, the building is a load-bearing brick structure. From evidence available, it would appear that the explosion occurred in an apartment on the fourth floor, destruction of this unit then leading to the progressive collapse of all stories above. Falling debris caused destruction of the rooms directly below the apartment. The opinions given in the report [8] are based upon evidence available at the time of its preparation in July 1972, and may not be the final word as to the cause.

If it is assumed that the apartment in question were occupied at the rate of 4 persons per apartment, then the death toll represents 18 out of 44 affected by the incident, 41 percent.

It was reported [62] that a car crashed into a five-story tenament building in New York dislodging a column and sending tenants and portions of the structure crashing into the street.

A considerable discussion of cases of progressive collapse in Canada and the U.S. in the years 1968 to 1972 has been given by Allen and Scriever [7].

B. Experimental Studies

It is appropriate to discuss these studies from the standpoint of the materials concerned.

B.1 Reinforced Concrete Panels

Ronan Point provided considerable impetus for construction companies in the United Kingdom to expand their already large experimental efforts and

Figure 8 Apartment building at Calle Santa Amelia, Barcelona, Spain following a progressive collapse on March 6, 1972.

to develop improved procedures for design. Accordingly, the large construction companies marketing concrete panelized systems carried out evaluations of their systems to determine whether they met the new criteria [2].

Much testing has been carried out in the U.K., by the Building Research Establishment and Imperial College, concerning the ability of concrete panel structures to bridge over the loss of structural elements.

Outside the United Kingdom, considerable experimental work has been carried out by Hanson and Olesen [63] in Denmark. Tests were made to determine the strength and stiffness of vertical-keyed shear joints between wall elements of prefabricated concrete. It is known that Olesen is continuing with a series of laboratory tests, in which are simulated two stories and two bays of a building that is of precast concrete panel construction. In these tests, wall and floor panels will be successively removed to determine the distress in the remaining structure.

Granstrom, in Sweden, has reported [64] model tests, to scale of 1:20, involving studies of forces in joints and framework deformations of buildings that have sustained local damage. The tests relate mainly to domestic and office buildings of precast concrete. The results show that the provision of joint connections, even of moderate strength, can reduce dramatically the probability of collapse of buildings. The shear resistance of vertical joints in large concrete panel construction, and the ability of the joints to transmit horizontal in-plane tensile forces, has been studied extensively and the various contributions are too numerous to list in this report. The same is equally true of horizontal connections between floor panels and the top of wall panels. The reader is referred to the considerable volume of information presented in March 1970, at a Symposium [65] for Joints in Precast Concrete Components held in the United Kingdom.

B.2 Load-Bearing Masonry

Wilton, Gabrielsen, Edmunds, and Bechtel [66] reported the early progress on a long range program with the objective of developing improved methods of predicting the structural response, failure modes, and debris characteristics of masonry wall panels. This information is required by the Office of Civil Defense in order to develop improved shelter systems. The study combines both analytical and experimental work and, in the period reported, the authors have developed a statistical failure theory for brick structures, a failure theory for wall panels, and the development of an analytical program whereby wall panel failure predictions may be used in the design of an experimental test program. Their research included testing of brick wall panels, interior gypsum board wall panels, concrete wall panels and a small number of tests to investigate the effect of air blast on shelter ventilating equipment. A later report by Wilton, Gabrielsen, and Morris [67] describes the results of the continued investigation of the response to blast loading of full-scale wall panels of relatively brittle materials, notably non-reinforced brick. Information such as element-failure times, energy transmitted to a building frame, and the influence of support conditions and other geometric factors were obtained from the tests. Loading tests were carried out on walls which completely closed a test tunnel, on walls with 17.5 percent doorway openings, and on walls with 16.7 and 27 percent window openings.

In the United Kingdom, it is now a requirement under the Building Regulations [48], that structures of five stories and more shall remain stable after the removal of a specified length of load bearing wall, although at a substantially reduced safety factor. Sinaj and Hendry [68] describe three experiments that had the objective of confirming that this could be achieved in a simple five-story brick cross-wall structure. In each experiment, a section of the main load bearing wall was removed at ground level to test the stability of the structure in a damage condition, as might occur following an internal explosion. Measurements were made of the applied loads, deflections and strains. The theoretical conclusion that the structure would remain stable under these conditions was confirmed and some information was obtained concerning the strength of 114 mm. (4.5 inches) thick wall panels subjected to lateral loadings.

C. Analysis

The most extensive work, in the area of analysis of building response to overpressure loading, is that of Newmark [69] who prepared a state-of-the-art report presented at the August 1972 Conference on the Planning and Design of Tall Buildings. In this paper, he discussed the effects on buildings of external loadings of a transient or impulsive nature where these loadings can arise from the detonation of explosives, including gas or other sources, from sonic boom from aircraft, or from accidental impact. Fundamental relations for developing blast resistance design procedures are also presented based upon the work of Newmark and others over a period of many years.

Other analytical procedures have been reviewed earlier in this paper in the context of their application.

X. Conclusions

This report is in the nature of a progress document, coming at the end of the first year of a study of abnormal loadings and progressive collapse. This study is being carried out by the National Bureau of Standards on behalf of the Department of Housing and Urban Development.

The report has discussed the state of knowledge with respect to abnormal loadings on buildings, the response of buildings and building elements to these loadings, and criteria by which the risk of progressive collapse might be minimized.

Probably very few buildings, of the type constructed in the past, would be susceptible to progressive collapse in the event of an abnormal loading. To date, few incidents of progressive collapse have occurred in the USA, and none approach the magnitude of the Ronan Point incident in the United Kingdom.

The patterns of siting, design, and construction of buildings are changing, however, and the frequency of occurrence of certain abnormal loadings on buildings is increasing. The NBS study is concerned not so much with U.S. buildings of a type constructed in the past, but with those types constructed now, and in the future. There is reason to believe that, in the absence of new criteria to minimize the risk of progressive collapse, the number of buildings, that are susceptible, will otherwise increase.

It should not be assumed that conventional systems are free from the risk of progressive collapse. Rather, studies should be made to determine the general degree of susceptibility to progressive collapse for multistory buildings of all construction types. There is also need to study certain abnormal loadings on buildings for which little knowledge is available at present, such as unconfined vapor cloud explosions and vehicular impact.

Several other countries have implemented criteria to minimize the risk of progressive collapse. The data presented and discussed in Chapters 5 and 6 of this report are believed to express a lower bound to the probability of abnormal loadings in the USA. Yet these data indicate that, for buildings susceptible to progressive collapse in the USA, the risk substantially exceeds that which prompted other countries to implement criteria. Furthermore, the risk of fatality appears to be on a par with that for fire.

It is concluded, therefore, that U.S. standards-writing bodies should adopt appropriate rational criteria as soon as possible.

XI. Acknowledgments

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The National workshop on building practices for disaster mitigation was concerned with earthquakes, extreme winds, and similar dynamic hazards. These proceedings present recommendations derived at the workshop and addressed to policy makers in government and industry, as well as practitioners in engineering, architecture, land use planning, and the earth and meteorological sciences. The recommendations evaluate current building practices, define opportunities for improving current practice from documented research findings, and recommend research to fill gaps in knowledge. Recommendations are made for implementation of improved practices at professional and policy levels. The objectives include avoidance of human suffering, reduction of property loss, and maintenance of vital function in buildings under conditions threatening disaster. Fifteen review articles were prepared by experts in the professions and research disciplines to define the state of the art in disaster mitigation and to guide discussions at the workshop; the articles are included in the proceedings.

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