NISTIR 4515

Proceedings of a Workshop on Evaluation, Repair, and Retrofit of Structures—U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, MD, USA, May 12–14, 1990

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Proceedings of a Workshop on Evaluation, Repair, and Retrofit of Structures—U.S.-Japan Panel on Wind and Seismic Effects, UJNR, Gaithersburg, MD, USA, May 12–14, 1990

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April 1991

U.S. Department of Commerce
Robert A. Mosbacher, *Secretary*
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Gaithersburg, MD 20899

PREFACE

The evaluation, repair, and retrofit of existing structures is now viewed as one of the most critical problems in reducing earthquake hazards. Research is underway in many countries to address technical issues related to this general area. Design and construction practices are changing rapidly. To facilitate the exchange of information between engineers in the U.S. and Japan, a series of workshops have been held over the past 10 years in connection with UJNR activities. Meetings were held in May 1980 in Los Angeles, May 1981 in Sendai and Tsukuba, May 1982 in San Francisco, and May 1987 in Tsukuba. The exchange of research and design studies and data has broadened the experience of all the participants and the information presented has been compiled into proceedings for wider dissemination.

The financial support of the National Science Foundation through Grant MSM-9014734 to The University of Texas made it possible to hold the latest meeting in Gaithersburg in May 1990. The efforts of Dr. Ken P. Chong, National Science Foundation (U.S. Chairman of Task Committee C) in establishing the program, G. Robert Fuller, Dept. of Housing and Urban Development (U.S. Chairman of Task Group D) in making arrangements for the technical tours, Dr. Shin Okamoto, Building Research Institute (Chairman of Task Committee D, Japan), and Dr. Shinsuke Nakata, Building Research Institute, (Representative of Dr. Masaya Hirosawa who is Chairman of Task Committee C, Japan) in arranging the program for the Japanese participants is gratefully acknowledged.

The opinions, findings, conclusions, and recommendations expressed in this Proceedings are those of the individual contributors and do not necessarily reflect the views of the NSF or other private or governmental organizations.

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ABSTRACT

This report is the Proceedings of an international workshop on Evaluation, Repair, and Retrofit of Structures. The workshop was a joint effort of Task Committees C and D, "Repair and Retrofit of Existing Structures" and "Evaluation of Structural Performance" respectively of the U.S.-JAPAN Panel on Wind and Seismic Effects. The workshop was hosted by the National Institute of Standards and Technology during May 12-14 1990. The National Science Foundation provided the financial support. The subjects addressed included: evaluation of structures; performance of existing structures their repair and strengthening; and research on techniques for repairing and retrofitting structures.

KEYWORDS: evaluation, performance, repair, retrofit, strengthening, structures.

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SUMMARY

OF

WORKSHOP ON EVALUATION, REPAIR, AND RETROFIT OF STRUCTURES

May 12-14, 1990

Gaithersburg, Maryland

The workshop was organized by The UJNR Task Committees C and D. The workshop followed a similar meeting held in Tsukuba in 1987. Because the exchange of information at that time proved to be valuable to both sides, it was felt that continued exchange of research and design data was desirable. The meeting in Gaithersburg was attended by 24 designers, researchers, and government officials (12 from Japan and 12 from the U.S.). A total of 18 presentations were made during the 1-1/2 day workshop.

The workshop was divided into three sessions:

1. Evaluation of Structures
2. Performance of Existing Structures - Repair and Strengthening Needs
3. Research on Techniques for Repair and Retrofit of Structures

The presentations offered both sides an opportunity to assess recent developments in both countries. It is clear that there is a need for continued studies into:

1. Evaluation of risk posed by existing structures in various seismic regions
2. Techniques for retrofitting inadequate structures
3. Design code provisions for retrofitting

The participants agreed that future workshops should be organized to continue the exchange of personnel and results of various studies underway or planned in each country. While general discussions are useful, it was felt that a directed program for the next workshop, with a more narrowly defined scope, would permit the participants to focus on specific topics and to exchange data and ideas which could be transferred to practice quickly. There is a special need for translating some documents from Japanese into English -- especially those dealing with evaluation of damaged or undamaged existing structures.

AGENDA

WORKSHOP ON EVALUATION, REPAIR, AND RETROFIT OF STRUCTURES

UJNR Task Committees C and D
May 12-14, 1990
Gaithersburg, Maryland

May 12, 19901-5 pm at Holiday Inn, Gaithersburg

Session 1: Evaluation of Structures

Co-chairmen: G. Fuller, HUD
S. Okamoto, BRI

Reporters: K. P. Chong, NSF
S. Nakata, BRI

1-1. C. Rojahn, Applied Technology Council "Development and Application of ATC21 and 22--Evaluation Procedures for Existing Buildings"
1-2. S. Nakata, M. Hirosawa, II. Yamanouchi, and K. Okada, BRI "Recent Aseismic Countermeasure for Existing Buildings and Damaged Buildings in Japan"
1-3. M. A. Sozen, University of Illinois "Deferring Payments: Management of the Earthquake Risk in Central United States"
1-4. T. Ohkoshi, Nihon Sekkei Co., S. Okamoto, BRI, and T. Teramoto, Nikken Sekkei Co. "Comparison of Aseismic Design Between U.S. and Japan"
3-2. Y. Hosokawa, University of Tokyo and Y. Matsuzaki, Science University of Tokyo "Guideline for Repair and Retrofit Design of Existing Reinforced Concrete Buildings"
3-5. T. Akiyama, Tokyo Soil Research and Y. Shimizu, Tokyo Technical Institute "A Study on Shear Strength of Post-Installed Anchors"

May 13, 1990 9 am-5 pm - Study and cultural tours in Washington D.C. area
National Building Museum
Rehabilitation of Union Station
Visits to Museums on Mall

May 14, 1990 8:30am-5pm - at NIST, Center for Building Technology, Gaithersburg, MD

Session 2: Performance of Existing Structures-Repair and Strengthening Needs

Co-chairmen: K. P. Chong, NSF
S. Nakata, BRI

Reporters: G. R. Fuller, HUD
S. Okamoto, BRI

2-1. T. Kaminosono, BRI, T. Okada, University of Tokyo, and M. Hirosawa, BRI "Aseismic Performance and Strengthening of Existing RC Building Structures"
2-2. M. Phipps, H.J. Degenkolb Associates "Performance of Buildings for "Hi-tech" Industries in Loma Prieta Earthquake"
2-3. S. Okamoto and T. Kaminosono, BRI "Actual Examples of Seismic Judgement for Existing Buildings with or without Retrofitting"
2-4. R. Lew, City of San Francisco "Implications of Loma Prieta on Building Requirements and Regulations"
2-5. M. Okubo, Kyushu Institute of Design "Current System for Inspection/Evaluation/Restoration of Earthquake Damaged Buildings and its Background and Future Issues"

Session 3: Research on Techniques for Repair and Retrofit of Structures

Co-chairmen: E. Sabadell, NSF
S. Nakata, BRI

Reporters: G. R. Fuller, HUD
S. Okamoto, BRI

3-1. J. O. Jirsa, University of Texas at Austin "Overview of Research on Repair and Strengthening of Concrete Structures"
3-3. L.-W. Lu, Lehigh University "Overview of Research on Repair and Retrofit of Steel Structures"
3-4. Y. Yamamoto, Shibaura Institute of Technolgy and Y. Shimizu, Tokyo Technical Institute "Reinforced Concrete Buildings Strengthened with Steel Members"
3-6. T. Ohta, Horie Design Associates, T. Okada, Institute of Industrial Science, University of Tokyo "Seismic Strengthening of an Existing Reinforced Concrete Office Building in Tokyo, Japan"
3-7. Y. Shimizu, Tokyo Technical Institute, Y. Yamamoto, Shibaura Institute of Technology "Recent Research Results in Strengthening Methods by Steel Brace"
3-8. T. Katagiri, Nihon Drive-it Co., et. al. "Experimental Study on Mortar Joints Between Reinforced Concrete Frame and Steel Shear Brace"

REPORTS

WORKSHOP ON EVALUATION, REPAIR, AND RETROFIT OF STRUCTURES

UJNR TASK COMMITTEES C AND D

May 12-14, 1990, Gaithersburg, Maryland

1-1. "Development and Application of ATC21 and 22--Evaluation Procedures for Existing Buildings." C. Rojahn, Applied Technology Council; Charles Scawthorn, EQE, Inc; and Fred Willsea, Wiss Janney Elstner.

1-2. "Recent Seismic Countermeasure for Existing Buildings and Damaged Buildings in Japan." S. Nakata and M. Hirosawa, BRI.

1-3. "Deferring Payments: Management of the Earthquake Risk in Central United States." M. A. Sozen, University of Illinois.

1-4. "Comparison of Aseismic Design Between U.S. and Japan." S. Okamoto, BRI; T. Teramoto, Nikken Sekkei Ltd.; and, T. Ohkoshi, Nihon Sekkei Co.

2-1. "Aseismic Performance and Strengthening of Existing Reinforced Concrete Building Structures." T. Kaminosono, BRI; T. Okada, University of Tokyo; and M. Hirosawa, BRI.

2-2. "Performance of Buildings for "High-tech" Industries in Silicon Valley in the Loma Prieta Earthquake." M. Phipps, H.J. Degenkolb Associates.

2-3. "Actual Examples of Seismic Judgement for Existing Buildings with or without Retrofitting." S. Okamoto and T. Kaminosono, BRI.

2-4. "Implications of the Loma Prieta on Earthquake on Building Requirements and Regulations." F. Lew, City of San Francisco.

2-5. "Current System for Inspection/Evaluation/Restoration of Earthquake Damaged Buildings and its Background and Future Issues." M. Okubo, Kyushu Institute of Design.

2-6. "Seismic Diagnosis for Gymnasium Type Structures." H. Yamanouchi, BRI; I. Nishiyama, BRI; and K. Takanashi, University of Tokyo.

3-1. "Recent Research - Repair and Strengthening of Reinforced Concrete Structures." J. O. Jirsa, University of Texas at Austin.

3-2. "Post-Installed Anchor Bolts Subjected to Tension." Y. Hosokawa, University of Tokyo and H. Aoyama, University of Tokyo.

3-3. "Repair, Retrofit and Strengthening of Steel Buildings and Bridges." L.-W. Lu, and Ben T. Yen, Lehigh University.

3-4. "Ultimate Seismic Strength and Ductility Index of Reinforced Concrete Buildings Strengthened with Steel Brace and Steel Panel." Y. Yamamoto, Shibaura Institute of Technolgy and M. Hirosawa, BRI.

3-5. "Method of Strengthing and Designing Shear Connectors Between Existing Reinforced Concrete Frame and Infilled Steel Brace and Panel." Y. Yamamoto, Shibaura Institute of Technology, and H. Aoyama, University of Tokyo.

3-6. "Ultimate Shear Strength of Reinforced Concrete Members with Extremely Small Shear Span Ratios." Y. Yamamoto, Shibaura Institute of Technology.

3-7. "A Study on Shear Strength of Anchor Bolts." T. Akiyama, Tokyo Soil Research; M. Hirosawa, BRI; Y. Shimizu, Tokyo Technical Institute; and T. Katagiri, Japan Drive-it Co., Ltd.

3-8. "Seismic Strengthening of an Existing Reinforced Concrete Building." T. Okada, Institute of Industrial Science, University of Tokyo; M. Murakami, University of Chiba; Y. Matuzaki, Science University of Tokyo; S. Hayashi, Shibaura Institute of Technology; and T. Ota, Horie Engineering and Architecutral Research Institute.

3-9. "Recent Research Results in Strengthening Methods by Steel Brace and Panel on Existing Reinforced Concrete Frames." Y. Shimizu, Tokyo Technical Institute and Y. Yamamoto, Shibaura Institute of Technology.

3-10. "Experimental Study on Mortar Joints Between Reinforced Concrete Frame and Steel Shear Brace." Y. Shimizu, Tokyo Technical Institute; Y. Yamamoto, Shibaura Institute of Technology, T. Katagiri, Japan Drive-it Co., Ltd.; and T. Akiyama, Tokyo Soil Research Co., Ltd.

3-11. "Applications of Retrofit Method with Carbon Fiber for Existing Reinforced Concrete Structures." H. Katsumata, K. Kimura, and Y. Kobatake, Technical Research Institute; and K. Yagi, T. Tanaka, and T. Sawanobori, Mitsubishi Kasei Corporation.

WORKSHOP PARTICIPANTS
EVALUATION, REPAIR, AND RETROFIT OF STRUCTURES
UJNR TASK COMMITTEE "C" & "D"
MAY 12-14, 1990, GAITHERSBURG, MARYLAND

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1.	**Shin Okamoto**	Director of International Institute of Seismology & Earthquake Engineering (IISEE) Building Research Institute (BRI), Ministry of Construction
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11.	**Takayuki Teramoto**	Director of Struct. Dept., Nikken Sekkei Co.
12.	**Toshio Ohkoshi**	Director of Struct. Dept., Nihon Sekkei Co.

United States

1.	**G. Robert Fuller**	Department of Housing and Urban Development
2.	**Ken P. Chong**	National Science Foundation
3.	**Mete A. Sozen**	University of Illinois
4.	**James O. Jirsa**	University of Texas
5.	**Eleanora Sabadell**	National Science Foundation
6.	**Le-Wu Lu**	Lehigh University
7.	**Franklin Lew**	City of San Francisco
8.	**Maryann T. Phipps**	H. J. Degenkolb Assoc., San Francisco
9.	**Christopher Rojahn**	Applied Technology Council
10.	**H. S. Lew**	National Institute of Standards and Technology
11.	**James Hill**	Department of Energy
12.	**Richard Marshall**	National Institute of Standards and Technology

WORKSHOP PARTICIPANTS

TOUR OF NATIONAL BUILDING MUSEUM

TOUR OF REHABILITATED
UNION STATION

SECTION ONE

EVALUATION OF STRUCTURES

Applied Technology Council Methodologies for Rapid and Detailed Seismic Evaluation of Existing Buildings

by

Christopher Rojahn
Applied Technology Council
Redwood City, California

Charles Scawthorn
EQE, Inc.
San Francisco, California

Fred Willsea
Wiss Janney Elstner
Emeryville, California

In April 1987 The Federal Emergency Management Agency (FEMA) awarded the Applied Technology Council (ATC) a three-year contract to develop two handbooks on seismic evaluation of existing buildings: (1) a handbook for rapid visual screening of buildings for potential seismic hazards (ATC-21 Handbook); and (2) a handbook on seismic evaluation of potentially hazardous buildings (ATC-22 Handbook). The intent of the ATC-21 Handbook (ATC, 1988a) is to provide a standard rapid visual screening procedure to identify those buildings that might pose potentially serious risk of loss of life and injury, or of severe curtailment of community services, in case of a damaging earthquake; the handbook has been written to be used primarily by local building officials, but could be used by persons ranging from interested citizens to professional engineers. Buildings identified as potentially hazardous using the ATC-21 Handbook should then be evaluated in detail using the ATC-22 Handbook (ATC, 1989a). The ATC-22 Handbook, which is written for design professionals and local building officials, provides a standard methodology to evaluate buildings of different types and occupancies in areas of different seismicity throughout the United States.

ATC-21 Handbook for Rapid Visual Screening of Buildings for Potential Seismic Hazards

As the initial step in the development of the ATC-21 Handbook, ATC evaluated sixty-two existing rapid screening procedures (RSPs), RSP-related methodologies and/or actual building review projects (ATC, 1988b). Based on this review as well as the general experience of the project participants in conducting numerous field surveys and analyses of existing buildings, it was found that an "ideal" RSP would include (i) explicit definition of the ground motion, (ii) consideration of all building types, not just an a priori "hazardous type" (e.g., unreinforced masonry), (iii) a procedure whereby the degree of seismic hazard is quantitatively determined, (iv) a rational analytically based framework for this procedure, (v) national applicability, (vi) probabilistic concepts, in recognition of inherent uncertainty, and (vii) consideration of various factors, such as age, condition, and vertical and horizontal irregularities. In summary, it was found that few, if any, methods incorporated all of the attributes of an ideal RSP.

Based on the above review and identified attributes of an ideal RSP, the ATC-21 RSP was developed, to be applied on the basis of a "sidewalk survey" (i.e., without benefit of entry to the building, or review of structural drawings). The ATC-21 RSP is fundamentally based on classifying the building on the basis of its primary structural lateral force resisting system (LFRS) and assigning a Basic Structural Hazard (BSH). Twelve building types are employed, generally corresponding to those in a related study on detailed evaluation of seismic resistance of buildings (ATC, 1987). The LFRS is inferred on the basis of available previously collated information, such as assessor's files and age of construction, as well as visual cues. When two (or more) LFRS's cannot be reasonably eliminated, both are admitted as possibilities and "scored". The BSH is then modified by adding or subtracting Performance Modification Factors (PMF), to arrive at a final Structural Score S. The

PMF's relate to significant seismic-related defects the inspector observes. The BSH, PMF and final Structural Score S all relate to the probability of the building sustaining major life-threatening structural damage:

$$S = -\log_{10} \text{Prob (Damage} > 60\%\text{)} \qquad (1)$$

Sixty percent was selected as the threshold of potentially life-threatening collapse and/or major damage, the point at which many structures are demolished rather than repaired. Final S scores typically range from 0 to about 6, with higher S scores corresponding to better seismic performance. Based on prevailing levels of acceptable risk as inferred from present building codes, the project team recommended that buildings which "score" less than about 2 should be reviewed by a professional engineer experienced in seismic design. In the above, BSH's are based on data in the ATC-13 Report (ATC, 1985), modified for non-California buildings on the basis of a limited sampling of experienced structural engineers from other parts of the nation. The methodology is completely presented in the ATC-21-1 Report (ATC, 1988b), with applicable computer code, so that a particular jurisdiction can modify the BSH's and PMF's to correspond to a level of damage other than 60%, if desired.

The ATC-21 Handbook contains sections on the nature of earthquakes and seismicity of the United States, earthquake behavior of buildings, general survey implementation, the RSP Method and Data Collection Form, and interpretation of Structural Scores. Three Data Collection Forms are provided, corresponding to areas of high seismic hazard (National Earthquake Hazards Reduction Program (NEHRP) Map Areas 5,6,7), moderate (Areas 3,4), and low (Areas 1,2). Each Data Collection Form (Fig. 1) includes space for sketches and a photograph of the building, information on occupancy, and a matrix showing Structural Scores and Modifiers for each building type considered by the methodology. The handbook also provides a quick reference guide (Fig. 2), for use with the Data Collection Form.

ATC-22 Handbook for Seismic Evaluation of Existing Buildings (Preliminary)

The ATC-22 Handbook (ATC, 1989) provides guidelines and procedures for the seismic evaluation of existing buildings nationwide. The methodology focuses specifically on life safety and enables the structural engineering practitioner to assess the adequacy of individual buildings and identify those characteristics, or weaknesses, that make a safety hazard (defined as the potential for structural and non-structural damage that would cause death or injury to the occupants, or cause their entrapment by destroying or blocking entry and exit paths). The methodology is based on that developed under the ATC-14 project, "Evaluating the Seismic Resistance of Existing Buildings" (ATC, 1987); it also follows the criteria specified in the 1988 *NEHRP Recommended Guidelines for the Development of Seismic Regulations for New Buildings* (BSSC, 1988). The methodology includes: (1) procedures for estimating expected seismic loading on a site-specific basis; (2) a building classification system that allows the user to classify existing buildings into one of 15 model building types; (3) procedures for qualitative evaluations that involve the identification of characteristics that make buildings vulnerable to seismic damage; and (4) procedures for detailed evaluation of buildings having such characteristics. All major building types prevalent in U.S. construction are considered.

ATC-22 provides a methodology that will serve to guide but not restrict the evaluating engineer so that consistent and fairly complete thinking can be brought to bear on each seismic evaluation. The methodology includes qualitative and detailed evaluation procedures and begins with data collection procedures that are required to gather the information necessary to classify the building and perform the evaluations. Appropriate procedures are outlined for determining the applicable seismic loading criteria, using existing design documents (available plans and calculations), performing site investigations, and testing the structural materials.

Based on a brief field inspection and review of the available drawings, each building is classified according to one of 15 model building types, which exhibit the structural and performance characteristics typical of the total building inventory. Once a building has been so classified, the building is evaluated utilizing a procedure that considers performance characteristics based on past earthquake observations, expected building loads and load paths, and characteristics, or weaknesses, that imply that a life-safety hazard may exist. If any such potential life-safety hazards are identified, an appropriate detailed evaluation is recommended with permissible demand/capacity ratios suggested for each element of concern. Through this procedure, not only are weak links in the structural system identified, but the severity of the life-safety hazard is assessed.

Seismic Loading Criteria. The ATC-22 seismic loading criteria utilizes the A_a map of Effective Peak Acceleration (Fig. 3) and the A_v map of Effective Peak velocity related acceleration (developed under the ATC-3 project (ATC, 1978). For buildings in the medium-period range, where acceleration is dependent on period, the base shear for the evaluation is reduced to 67% of the base shear in the NEHRP provisions for the design of new buildings. For buildings in the short-period range the base shear for evaluation is 85% of that for the design of new buildings. The rationale for using lower force levels for the evaluation of existing buildings (compared to the design of new buildings) is considered to be intuitively acceptable: we can tolerate less conservatism in an existing building because it can be strengthened only with substantial cost and disruption of use.

Building Classification System. The building identification system employed in the methodology is based on determining the material or type of construction employed in the principal gravity and lateral-force resisting elements. A total of 98 different building types were originally identified in the ATC-14 project that represent the possible combinations of materials and structural systems economical enough to be widely used. From these 98 building types a subset of 15 model building types were identified that exhibit the structural and performance characteristics typical of the total building inventory. By reducing the number of buildings to these basic 15 model building types, it was possible to develop a seismic evaluation methodology that treats each of the model building types uniquely. To further simplify the evaluation procedure, these model buildings have been further categorized by their structural materials (Fig. 4).

In those cases where the lateral-force resisting systems for the two principal directions are different, two model building types would be selected and a separate evaluation would be performed for each principal direction. Procedures are also provided for dealing with buildings that do not fall into any specified model building type.

Evaluation Procedures. The heart of the ATC-22 methodology is a collection of evaluation statements for each model building type that identify various vulnerability areas in the structural system requiring specific consideration. The evaluation statements are written such that a positive or "true" response to a statement implies that the building is adequate in that area. If a building passes all statements with true responses, it can be passed without further evaluation, i.e., it is deemed not to be a life-safety hazard. For statements that are "false", additional evaluation is required. A false statement does not necessarily imply that a complete structural evaluation is necessary, or that the building is automatically deficient; it simply flags an area of concern for the evaluating engineer and implies that a life-safety hazard may exist. Example statements for steel-frame buildings with concrete shear walls are shown in Figure 5.

In addition to and following the list of Evaluation Statements for each model building type, the Handbook provides procedures and commentary that assist the user in evolution the building. The procedures and commentary describe the concerns that give rise to the Evaluation Statements, provide guidance for evaluating the conditions, and describe the deficiencies which, when remedied, would allow the Statements to be considered true. The procedures provide acceptance criteria for three types of element behavior: elements that behave in a ductile manner; brittle elements, and semi-ductile elements. If all significant elements meet the basic acceptance criteria, no further analysis is needed. Elements that do not meet the acceptance criteria are the remaining deficiencies that should be addressed in a rehabilitation program.

The Handbook has been organized to be compatible with the FEMA Publication, *Techniques for Seismically Rehabilitating Existing Buildings (Preliminary)* (URS, 1989), which identifies and describes seismic rehabilitation techniques for a broad spectrum of building types and building components. Both the URS document and the ATC-22 Handbook are currently undergoing a consensus review by the Building Seismic Safety Council (BSSC). A re-issue of both documents is planned after BSSC completes its work.

References

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ATC, 1988b, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation*, Applied Technology Council ATC-21-1 Report, Redwood City, California.

ATC, 1989, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, Applied Technology Council ATC-22 Report, Redwood City, California.

URS, 1989, *Techniques for Seismically Rehabilitating Existing Buildings (Preliminary)*, URS Corporation, San Francisco, California.

ATC-21/ (NEHRP Map Areas 5,6,7 High)

Rapid Visual Screening of Seismically Hazardous Buildings

Address ____________________
____________________ Zip ________
Other Identifiers ____________________
No. Stories ________ Year Built ________
Inspector ________ Date ________
Total Floor Area (sq. ft) ____________________
Building Name ____________________
Use ____________________

(Peel-off label)

INSTANT PHOTO

Scale:

OCCUPANCY	
Residential Commercial Office Industrial Pub. Assem. School Govt. Bldg. Emer. Serv. Historic Bldg.	No. Persons 0–10 11–100 100+

Non Structural Falling Hazard ☐

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data

DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	C1 (MRF)	C2 (SW)	C3/S5 (URM INF)	PC1 (TU)	PC2	RM	URM
Basic Score	4.5	4.5	3.0	5.5	3.5	2.0	3.0	1.5	2.0	1.5	3.0	1.0
High Rise	N/A	-2.0	-1.0	N/A	-1.0	-1.0	-1.0	-0.5	N/A	-0.5	-1.0	-0.5
Poor Condition	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Vert. Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-0.5	-0.5	-1.0	-1.0	-0.5	-0.5
Soft Story	-1.0	-2.5	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-2.0	-2.0	-1.0
Torsion	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Large Heavy Cladding	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
Short Columns	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
Post Benchmark Year	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 & 6 to 20 stories	N/A	-0.6	-0.8	N/A	-0.6	-0.8	-0.6	-0.8	N/A	-0.8	-0.6	-0.6

FINAL SCORE

COMMENTS

Detailed Evaluation Required?

YES NO

Figure 1.- ATC-21 Data Collection Form.

STRUCTURAL TYPE:		*Benchmark year:*
W	wood frame	________
S1	steel moment resisting frame	________
S2	steel brace frame	________
S3	light metal frame	________
S4	steel frame with concrete shear wall	________
C1	concrete moment resisting frame	________
C2	concrete shear wall	________
C3/S5	steel or concrete frame with masonry infill	________
PC1	tilt-up	________
PC2	precast frame	________
RM	reinforced masonry	________
URM	unreinforced masonry	________

OCCUPANCY LOAD:	*sq. ft. per person*
residential	100-300
commercial	50-200
office	100-200
industrial	200-500
public assembly	varies, 10 minimum
school	50-100
government bldg	100-200
emergency service	100

MODIFIERS:	
High-Rise -	8 stories and taller except URM, URM above 4 stories
Poor Condition -	showing cracks, damage, settlement, etc.
Vertical Irregularity-	steps in elevation, inclined walls, discontinuities in load path, building on hill
Soft Story -	open on all sides of building, tall ground floor, discontinuous shear walls
Torsion -	eccentric stiffness in plan, (e.g. corner building, wedge shaped building with one or two solid walls and all other walls open)
Plan Irregularity -	"L", "U", "E", "T" or other irregular building shape
Pounding -	floor levels of adjacent buildings not aligned and less than 4" of separation per story
Large Heavy Cladding -	many large heavy stone or concrete panels. glass panels and masonry veneer do not qualify
Short Columns -	some columns restrained by half walls or spandrel beams
Post Benchmark Year -	building designed after certain key year when code requirement was increased - different for each building type and municipality
Soil Profile: SL1 -	rock, or stiff clay less than 200 feet overlying rock
Soil Profile: SL2 -	cohesionless soil or stiff clay greater than 200 feet deep
Soil Profile: SL3 -	30 or more feet of soft or medium stiff clays (use if do not know soil profile)
SL3 & 8 to 20 Stories -	8- to 20-story building on SL3 soil profile
Non-Structural Falling Hazard -	masonry cornices, veneer, small cladding, overhangs especially on older structures. Wood and sheet metal ornaments do not qualify

Figure 2.- ATC-21 Quick Reference Form for Rapid Visual Screening.

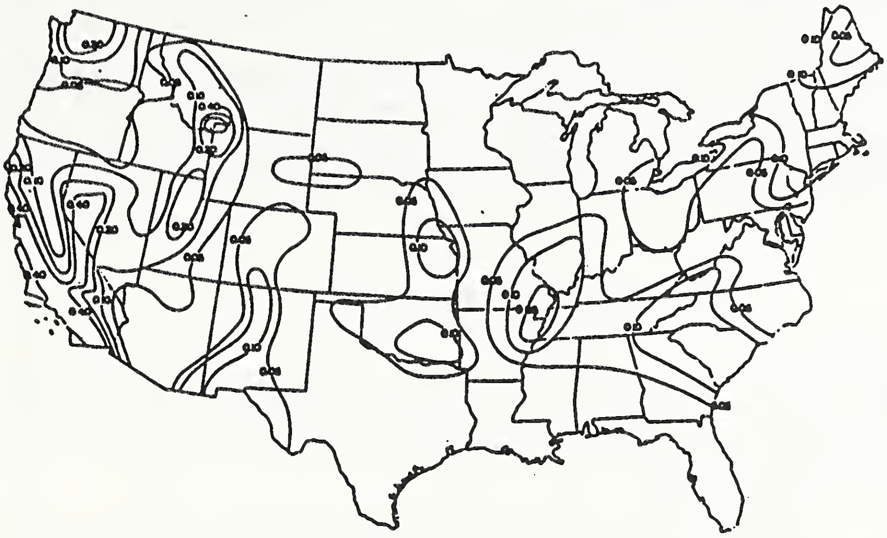

Figure 2-1: Contour Map for the Acceleration Coefficient A_a for the Continental United States

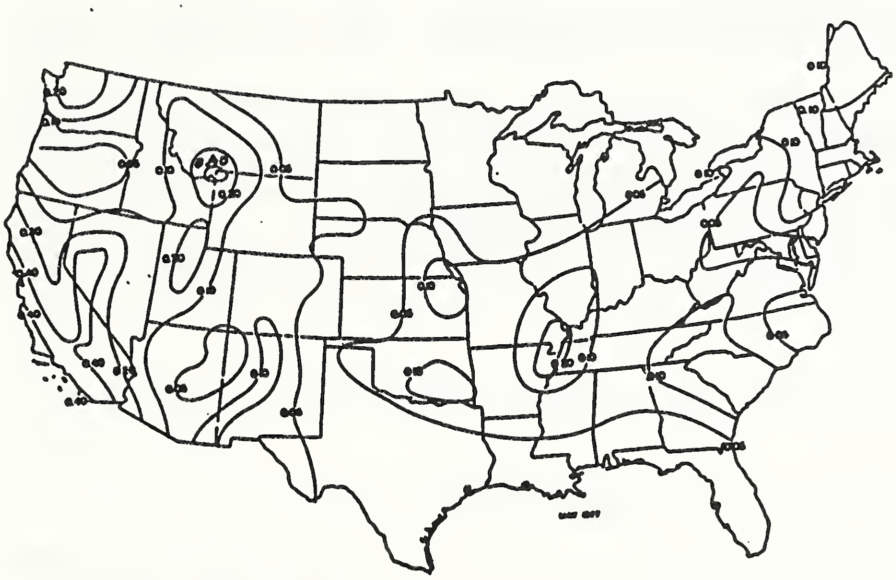

Figure 2-3: Contour Map for the Velocity-Related Acceleration Coefficient A_v for the Continental United States

Figure 3.- ATC-22 Maps of the Continental United States Showing Contours for Acceleration Coefficient A_a (top) and Acceleration Coefficient A_v (bottom).

TABLE 3.1: THE LIST OF BUILDING TYPES

Wood Buildings

1. Wood, Light Frame
2. Wood, Commercial and Industrial

Steel Buildings

3. Steel Moment Frame
4. Steel Braced Frame
5. Steel Light Frame
6. Steel Frame with Concrete Shear Walls
7. Steel Frame with Infill Shear Walls

Cast-in-Place Reinforced Concrete Buildings

8. Concrete Moment Frame
9. Concrete Shear Walls
10. Concrete Frame with Infill Shear Walls

Buildings with Precast Concrete Elements

11. Precast Concrete Tilt-Up Walls
12. Precast Concrete Frames with Concrete Shear Walls

Reinforced Masonry (RM) Buildings

13. RM Bearing Walls with Wood or Metal Deck Diaphragms
14. RM Bearing Walls with Precast Concrete Diaphragms

Unreinforced Masonry Buildings

15. Unreinforced Masonry Bearing Wall Buildings

Figure 4.- ATC-22 Model Building Types.

Address the following Evaluation Statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of the Handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation refer to the Handbook section whose number is indicated in parentheses at the end of the Statement.

T F

BUILDING SYSTEMS

__ __ WEAK STORY: There are no significant strength discontinuities in any of the vertical elements in the lateral force resisting system: the story strength at any story is not less than 80% of the strength of the story above. (4.1)

__ __ SOFT STORY: There are no significant stiffness discontinuites in any of the vertical elements in the lateral force resisting system: the lateral stiffness of a story is not less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. (4.1)

__ __ GEOMETRY: There are no significant geometrical irregularities: there are no setbacks, i.e. no changes in horizontal dimension of the lateral force resisting system of more than 30% in a story relative to the adjacent stories. (4.1)

__ __ MASS: There are no significant mass irregularities: there is no change of effective mass of more than 50% from one story to the next, excluding light roofs. (4.1)

__ __ VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (4.1)

__ __ TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20% of the width of the structure in either major plan dimension. (4.1)

__ __ DETERIORATION OF STEEL: There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical or lateral force resisting systems. (4.3)

__ __ CONCRETE WALL CRACKS: All diagonal cracks in the wall elements are 1.0 mm or less in width. (4.3)

SHEAR WALLS

__ __ SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the shear walls. (6.1)

__ __ REINFORCING STEEL: The area of reinforcing steel for concrete walls is greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes, at a maximum spacing of 18 inches. (6.1)

(continued)

Figure 5.- ATC-22 Evaluation Statements for Steel Frame Buildings with Concrete Shear Walls.

T F

__ __ REINFORCING AT OPENINGS: There is special wall reinforcement placed around all openings. (6.1)

__ __ OVERTURNING: All shear walls have h_w/l_w ratios less than 4. (6.1)

__ __ COLUMN SPLICES: Steel column splice details in shear wall boundary elements can develop the tensile strength of the column. (6.1)

__ __ WALL CONNECTIONS: There is positive connection between the shear walls and the steel beams and columns. (6.1)

__ __ CONFINEMENT REINFORCING: For shear walls with h_w/l_w greater than 2.0, the boundary elements are confined with spirals or ties with spacing less then 8 d_b. (6.1)

__ __ COUPLING BEAMS: The stirrups in all coupling beams over means of egress are spaced at d/2 or less and are anchored into the core with hooks of 135 degrees or more. (6.1)

DIAPHRAGMS

__ __ PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (8.1)

__ __ REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (8.1)

__ __ OPENINGS AT SHEAR WALLS: The diaphragm openings immediately adjacent to the shear walls constitute less than 25% of the wall length, and the available length appears sufficient. (8.1)

__ __ DECK TOPPING: All metal-deck floors and roofs have a reinforced concrete topping slab. (8.3)

CONNECTIONS

__ __ TRANSFER TO SHEAR WALLS: Diaphragms are reinforced for transfer of loads to the shear walls. (9.3)

__ __ WALL REINFORCING: All vertical wall reinforcing is doweled into the foundation. (9.4)

__ __ SHEAR-WALL-BOUNDARY COLUMNS: The shear wall columns are substantially anchored to the building foundation. (9.4)

Figure 5. (Continued)

RECENT SEISMIC COUNTERMEASURE
FOR
EXISTING BUILDINGS AND DAMAGED BUILDINGS IN JAPAN

by

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Workshop on Seismic Performance & Repair and Retrofit of Buildings
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1. GENERAL OUTLINE

1.1 INTRODUCTION

Around ten years ago , the guidelines for seismic index of existing buildings and their retrofitting were at first published . This year ,the revised edition of these guidelines are publishing very soon. At the same time , following book is also publishing : technical criteria for inspection method of damaged buildings and methods of their repair and restrengthening .

Tokachi-oki earthquake 1968 caused a strong impact to try to establish above mentioned guidelines . This earthquake also made to establish new seismic design standard in Japan.

In 1978 Miyagiken-oki earthquake , many low-rise and medium-rise buildings were damaged and were discussed their repair and reuse through analytical and experimental researches by technical engineers and academy groups. Using this occasion , The Ministry of Construction initiated the national research project " the Development of Restore Technique for Damaged Structures" . (1981-1985)

Such procedure of administrative countermeasure are tabulated in Table 1.

Summarizing many test results and strengthening examples until this time ,including the experiences of Armenian earthquake 1988, we Japanese side had a chance to revise the guidelines on the seismic evaluation and retrofit of existing buildings ,and that of damaged buildings.

1.2 REVISION OF THE GUIDELINE FOR EXISTING BUILDINGS

Following items were revised in the guidelines of seismic inspection and repair of existing buildings .

a. In steel buildings , new simplified inspection method for large space building like school gymnasium was devised . This guide book will be published as an appendix to existing one set (three books).

b.New design method strengthened by steel brace and its calculation examples were provided ,and further more the following revisions were also included; Increasing of numbers of new beam-column frames to exisiting frames and strengthening of girders. The guidelines which is composing of three books for RC will be all revised including above mentioned items.

c.In case of steel reinforced concrete (SRC) ,only an inspection method of seismic diagnose index was once published . The revision for SRC buildings will be done in accordance with that for reinforced concrete buildings.

d) No revision will be done in case of timber structure. Nor the simplified inspection too.

1.3 Contents of New Guideline for Damaged Buildings

National project "Development of the Restoration Technology for Earthquake Damaged Structures" covered steel reinforced buildings, reinforced concrete buildings ,timber buildings and land for houses. An inspection method of damage grade and repair method for reuse of those buildings were developed through analytical and experimental research works in this five years project. And then such research development project was summarized as " Design Manual of Repair and Retrofit to Damaged Buildings". In this manual, the back data base were also provided as well as technical guideline. However its technical grade was a little higher to general engineers, and so its revision covers simplified design method for the engineers to be easily understood. From now, central and local public bodies have to discuss on its actual application of this manual. Main items of this manual are as follows,

1) Damage Inspection Criteria (Fig. 2)

a. Damage inspection criteria is composed of two categories; emergency risk inspection and damage grade classification

b. Emergency risk inspection is for inspection of the risk of collapse by aftershocks. The building engineers can advise its reuse or not by the results of such inspection.

c. The inspection method of land for housing has two cases; one is for private house and another for group of houses.

2) Recover Technique Criteria

a. Recover technique criteria is different each other among types of structures. In reinforced concrete structures and timber structures, emergency and permanent recover technique is described. However emergency recover technique does not cover steel structure and land for housing.

b. Emergency recover technique for reinforced concrete and timber structure aims to prevent collapse or additional damage extend by aftershocks, and provides some emergency strengthening devices.

c. In the permanent recover design, following items are covered; evaluation method of remaining seismic performance after damage, new target of seismic performance, actual strengthening devices based on such new target and their seismic evaluation method.
d. In case of land for housing ,several recover devices and their actual application examples are introduced.

2. Main revised points in Seismic Evaluation Criteria and Repair Design Criteria for RC Structure

2.1 Seismic Evaluation Criteria of Existing Buildings and Criteria for Retrofit

1) Seismic Evaluation Criteria

Since the first edition was published around ten years ago, many related structural tests were done and new empirical equations for strength evaluation were proposed. And the numbers of actual seismic evaluation examples were increased. At the same time it was required to simplify the evaluation system. Considering such situation followings revisions were done.
a. Index of soil condition, G was one factor of synthetic seismic index Is as shown below,

$$Is\ (= E_0 \times G \times S_D \times T)$$

and its value was usually 1.0. In this revision, G value fluctuates by the soil condition.
b. In the provision of former manual, the values of seismic index I_{so} for safety criteria were expressed as 0.9 and 0.7 to the first and second phase evaluation respectively.
New seismic index is expressed as follows;

$$I_{so} = Es \cdot Z \cdot G \cdot U$$

where Es ; basic seismic index
Z ; zone factor
U ; use category factor

Here the basic index value of the first and the second phase for safety criteria were introduced as 0.8 and 0.6 respectively in the text.
c. Evaluating shear capacity of T-shaped girders, it was approved to take into account of the effect of floor slab concrete. Then the maximum shear capacity of girders raised to around 1.2 times as that before revision.
d. In an approximate calculation of ultimate strength of shears wall, it was approved to be assumed that the distribution of external force changed to uniform from reversed triangle without any special analysis.
e. F - index decided by ductility in shear wall can be estimated to be 10 % large than that in the old edition in case that stress distribution is clear at hinge mechanism.

f. C-index decided by average shear strength in the first phase diagnose can be evaluated in relation to the value of (fc' /200). It was based on that actually measured strength of concrete is always smaller than design strength.

g. Considering that many school buildings have been inspected their seismic indices and higher level indices were diagnosed, simplified method third phase diagnose was established. The amount of job was much decreased by such simplified method.

2) Design Criteria for Retrofit

In many actual retrofit works, strengthening by steel brace (light weight) was popular so as to redesign easily foundation of buildings. Following items were reversed reflecting actual design and analysis.

a. The contents of Section 2. Plan of Retrofit was intensified.

b. Required seismic index Is was expressed as follows;

$$Is = a \cdot Iso$$

Here the value of a was recommended as more than 1.2. This value taken as 1.0 1.2 in the new edition.

c. In the old edition, only brief recommendation for steel brace was described. However the its detailed calculation method and the structural detail were added in new edition.

d. In addition to conventional mechanical steel anchor, resin anchor was available . Evaluation method of its pull-out strength and shear capacity was also newly established .

e. Strength evaluation method for punching shear ofcolumn and shear wall installed in a frame was changed. Its evaluation of anchor strength was separately defined by the location in shear wall. And design method of shear reinforcement was also revised.

2.2 Inspection Criteria of Damaged Buildings & Guideline for Restoring

1) Damage Inspection Criteria

Two contents are the main parts here :

* judgement method of emergency risk at aftershocks
* judgement of requiring strengthen or not

The outline of such judgement are as follows :

a. The scope of this criteria is for reinforced concrete frame buildings and walled frame concrete buildings . However it is applicable to precast concrete buildings and reinforced concrete block buildings .

b. In the emergency risk judgement, if it is afraid that something will fall down from the building, such non-structural elements risk is also included as the risk.

c. Emergency inspection should be done from outside following three items : inclination , settlement and outward damage of building. (public buildings are inspected inside also.)
d. Damage grade is classified in five categories. Inspected buildings are judged their repair method and strengthening method or demolish .
e. Damage inspection is done at the most severe story , and all of columns are checked there.

2) Restoring Technique Criteria
a. In the criteria, emergency recover for aftershocks and permanent recover were described .
b. For emergency recover , after inspecting damage condition , especially the countermeasure works from following check points should be done : bearing capacity of axial force and horizontal force of columns, total stiffness and risk of collapse.
c. For permanent recover, following items are described , evaluation method of remaining seismic performance after damage, required seismic performance for recover and its evaluation and construction method .
d. By classification of damage grade , value of total seismic diagnoses Is should be decreased.
e. The target Is value for permanent recover is recommended as greater than that for existing buildings.
f. The general construction works and its detail of actual repair and recover works for columns , beams , non-structural elements and foundation are introduced. And as actual repair example after earthquake damage , design and recover works of hospital building is introduced.

3. Steel Buildings

3.1 Main Revision Points for Steel Buildings

The criteria itself was not changed . An repair and retrofit device for actual works is newly introduced . As an appendix, Simplified evaluation method large scale steel buildings was introduced as new edition.

1) Inspection of Gymnasium type Structure
In Japan , schhool gymnasium is oftten used as accommodations for the people who take refuge at big earthquake . So it is important to evaluate exactly its seismic performance before earthquake. In revised edition , new simplified seismic inspection method was added. Diagnose index, Is is composed of shear capacity Qu, F-value (ductility factor) etc. And

inspecting Diagnose Et is composed of zone factor Z, soil factor Rt and general base shear coefficient Co . By comparison of the value Is / Et , its seismic performance can be judged.

2) Restoring Design Criteria

After Miyagiken-oki earthquake 1978, Many engineers and researchers showed experimental research works of steel structure on repair and retrofit . In the new edition , following Actual strengthening devices were added summarizing such test results ;

a. cover plate for H-shape column
b. box type column
c. H-shape girder
d. Installation of brace
e. column base
f. local buckling

3.2 Damage Inspection Criteria and Recover Guideline

1) Damage Inspection Criteria

Damage inspection criteria is composed of emergency risk inspection and damage classification . The outline is :

a. The scope of this criteria is less than 45 meters height and exclude big span structure ,three dimension truss structure and suspension structure.

b. In emergency inspection , damage grade (A,B and C) are classified by such as check items , settlement of foundation , inclination of structure ,damage of structural members etc.

As an appendix , an actual steel building was introduced which was damaged in Miyagiken-oki earthquake.

2) Recover Guideline

a. Requirement of repair and retrofit can be discussed from the results of damage inspection and classification of damage grade.

b. If damaged building is judged as reuse after repair , the detailed discussion will be required : remaining seismic performance after damage.

4. Timber Buildings

In timber buildings also , damage inspection criteria and recover guideline were established.

1) Damage Inspection Criteria

Damage inspection criteria in timber structure consists of emergency risk inspection and damage classification and its outline is as follows;

a. Emergency risk is judged by settlement , inclination of first story and damage condition of finishing material.

b. Damage classification is judged by following check : foundation , floor slab, column ,beam , external wall and roofing . And damage grade of building is classified into five categories.

2) Recover Guidelines

Outline of recover guideline is almost similar to those of steel buildings: countermeasure at emergency inspection and permanent recover devices are described.

5 Conclusions

Earthquake Countermeasure of existing buildings is now being closed up by many people in the world whose buildings are not enoughly safe to earthquakes. In Japan , recent research and develop works on repair and retrofit of buildings caused to develop the design criteria on seismic diagnoses of existing buildings and recover manual of damaged buildings.

Such outline and their revisions were introduced here. In order to proceed successfully more this design criteria and guidelines considering prevention of severe damage from earthquakes, it is required to strengthen the countermeasure of political administration.

Both of U.S. and Japan should contribute towards mitigation of earthquake damage through this workshop.

Table 1 ADMINISTRATIVE COUNTERMEASURES AND RELATED EARTHQUAKES

EARTHQUAKE	RELATED PROJECT	FOR NEW BLDG.	FOR EXIST.BLDG	FOR DAMAGED BLDG
Noubi (1891)				
San Francisco (1906)				
Kanto (1923)		Design Code for Urban Buildings (1919) Revision above C = 0.1(1924) Standard for Struc. Calc. of RC Bldg. (1933)		
El Centro (1940)				
Fukui (1948)		The Bldg. Standard of Law (1950) C = 0.2		
Niigata (1964)	Group Research on Liquefaction			
Tokachioki (1968)		Revision of Ensuring Struc. Safety of the Code shear reinf.. (1970)	Check of public buildings Seismic Index of Exist.. Bldg. Repair & Retrofit (1977,1978)	
Miyagiken oki (1978)	New Aseismic Design Code Design Seismic Load Discussion of Design Philosophy (1972-1976)	Drastic Change above (1980)		
Nihonkaichuubu (1983)	National Project on Repair & Retrofit (1981-1985)			Manual for Damage inspection & Recover (1988)
Mexico (1985)	Design of Non-struc. Elements	Design criteria of Non-struc. Elements	Revision above	Establishment of Guideline above

Table 2 Outline of Revision and New Guideline for Seismic Diagnose and Retrofit

Type of *1 Structure	Existing Buildings				Damaged Buldings		
	Diagnose	Retrofit	Manual	Simplified	Inspection	Retrofit	Pamphlet
RC	△	□	□*3	○*2	▲	▲	●
SRC	(△)*4	—	—	—	—*5	—*5	—
S	X	X*6	X	○*6	▲	▲	●
W	X	X	—	X	▲	▲	●
Soil	—	—	—	—	—	—	●

—: guideline not available now nor plan of new guideline
△: small revision of existing manual
□: medium level revision
○: new establishment
X: no revision
▲: a little change or simplifing from results of national project
●: new establishment from results of national project

*1: Inspection and strengthening of non-structural members are described in each structural type.
*2: Simplified edition of third diagnose was deviced and installed in manual.
*3: Example of diagnose and retrofit were newly replenished.
*4: A little revision will be done as same as that in RC
*5: Inspecton and Retrofit of RC can be available.
*6: Simplified edition for gymnasium is published .

DEFERRING PAYMENTS:

Management of the Earthquake Risk in Central United States

by

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for presentation at the
Workshop on Evaluation, Repair, and Retrofit of Structures
UJNR Task Committees C and D

12-14 May 1990

A direct comparison of the consequences of the 1972 earthquake in Managua, Nicaragua, (14) and the 1985 earthquake in Vina del Mar, Chile, (12) makes a convincing argument for preparedness. It is true that the damage potential of earthquakes depends on more than the magnitude and the distance to the epicentral region. But it is undeniable that the dramatically better performance of the construction in Vina del Mar was due primarily to careful and patient planning. In Vina del Mar, the payments were made in advance and they paid off by limiting the cost of the damage to a fraction of what might have occurred otherwise.

One of the critical ingredients for planning to contain earthquake damage is a reliable technology to assess earthquake vulnerability. The appended report by Gavlin et al addresses the question of what needs to be done to develop a satisfactory earthquake vulnerability assessment technology for central U.S. and regions of comparable seismicity. The immediately following text discusses the earthquake risk in central U.S. and a possible reason for the slow pace of the preparations to reduce the threat of a devastating earthquake.

The focus of the earthquake risk in Central United States is at New Madrid, a town near the southeast corner of Missouri, which has given its name to the earthquakes of December 1811 and January and February 1812. The Modified Mercalli intensities in the epicentral region from all three events are reported to have reached or exceeded IX (10). The estimated Richter (m_b) magnitude from the December 1811 earthquake has been calculated from historic records to be 7.2 and magnitudes of 7.1 and 7.3 have been assigned to the January and February earthquakes of 1812 (11). The reported damages in St Louis, a densely settled area within 150 miles (240 km) of the epicentral region, provide a directly comprehensible if distorted measure of the effects of the 1811/1812 earthquakes. The maximum MM intensity in St Louis, Mo., was VIII for the December 1811 and IX for the February 1812 events (10).

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A strong earthquake with its epicenter near New Madrid is expected to cause strong shaking in at least seven states: Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee. The extent of the damage distribution from a repeat of one of the 1811 or 1812 events shown in Fig. 1 in terms of MM intensities (7). Although the specific locations of the boundaries of the regions with different damage intensities may be contested, there is little disagreement about the expectation that a large energy release in the New Madrid zone will affect a considerable portion of the central United States. As would be expected, expectation of the timing of the next major earthquake is more uncertain. A collection of direct quotes from reports and announcements about the seismicity of the New Madrid region will provide a perspective:

> "At New Madrid, for example, some workers estimate that major earthquakes, as large as those of1811 and 1812, could recur as often as every 500 years." (6)

> "A damaging earthquake, 6.0 or greater on the Richter scale, has a 50-50 chance of happening by the year 2000."(2)

> "A major earthquake, 7.5 or greater on the Richter Scale, has a 10 percent probability of occurrence in the next 20 years." (2)

> "Scientists agree that the region may be overdue for an earthquake in the range of 6.0 to 7.0+ magnitude on the Richter scale. The experts estimate there is a 50% chance that an earthquake of this magnitude could occur by the year 2000, and a 90% chance of one occurring in the next 50 years." (3)

> "The Center for Earthquake Studies at Southeast Missouri State University predicts that there is a 50-percent probability of a New Madrid area earthquake of 6.0 on the Richter scale by 2000. A 7.5-level quake has a 25 percent chance of occurrence in the same time period (8)."

> "A magnitude 7.6 earthquake [on the New Madrid fault] is my estimate of the largest possible earthquake in the near future. The probability of occurrence of an earthquake this size by the year 2000 is 7 percent" (9).

The statements are made in relation to different frames of reference and it is difficult to elicit a single conclusion out of them. However, it is clear that there is no disagreement about a definite expectation of strong shaking in the New Madrid area. Dr. Nuttli's estimate of a probability of 7 percent for a major event before the year 2000 in the New Madrid region appears quite credible against the background of other opinions even with the caveat that all researchers are likely to have been influenced by his pioneering work. Extent of damage, stated in terms of MM intensities, estimated by Dr. Nuttli is illustrated in Fig. 2.

A cursory look at construction in any one of the seven affected states would suffice to convince that much of the existing construction is vulnerable to earthquake. It is not unreasonable to conclude that the occurrence of the expected event in the New Madrid region will result in very serious losses.

The direct loss from a major earthquake in the New Madrid area is estimated to be approximately $70 billion (8) with the total loss, including indirect losses, amounting to approximately $140 billion. Considering the state of the art in loss estimation (12), the estimate may be off by a factor of two, in either direction.

Risk tolerance of society or individuals is neither a consistent nor a crisply definable matter. There are many examples of activities and products which society would not accept if the risk of injury or loss associated with it was seven percent in a fifteen-year period. In the building industry, if it was known that the risk of serious damage to a particular type of construction is seven percent over a fifteen-year period, past experience would indicate that the type of construction would not be built.

In the light of what is acceptable in building, the general state of inaction with respect to the potential damage from the New Madrid earthquake appears incongruous. It is conceded that the probability of occurrence assumed above may not be generally supported. It may be argued successfully to be less. But it is unlikely to be reduced by an order of magnitude, and the acceptance of a 0.7% probability of a 140-billion dollar loss in the foreseeable future is questionable. The comfort that may be derived from the position of relying on a stationary return period of not less than 500 years is too obtuse to ascribe to any responsible professional or public official. The apathy is difficult to rationalize. There is, however, another consideration that may identify the cause of if not rationalize inaction.

Table 1 contains a compilation of the losses from selected earthquakes experienced in the U.S. during the twentieth century. To facilitate comparison, the losses have been normalized to 1990 dollars using the Engineering News Record building-cost index.

Five of the events dominate the rest in terms of loss. They are San Francisco (1906), Long Beach (1933), Alaska (1964), San Fernando (1971) (Table 1, Fig. 3), and Loma Prieta (1989). Unfortunate as these events were, it may be observed that the U.S. losses to earthquake were surprisingly low considering the seismicity of its land mass and the total value of the construction.

Figure 4 compares the losses from the five events with the estimated direct loss from the next New Madrid earthquake. The comparison emphasizes that there has been no earthquake catastrophe experience in the U.S. comparable to the potential of the expected event in the New Madrid region. The cited amount for the New Madrid earthquake may pale in relation to the cost of a past war, the losses from a banking folly, or the annual budget of a department of the Federal Government, but three features of the earthquake catastrophe make such comparisons invalid. Earthquake loss is sudden. It will occur within the U.S. The losses have to be compensated quickly if society is not to suffer permanent setbacks.

Cognitive science has turned to memory to provide an explanation for thinking (4). One of the dominant models for explaining the thought process is "information processing" or reshuffling stored information to arrive at a conclusion. Thus, reasoning may depend critically on stored information. The more stored analogs exist of a new event or idea, the easier it is to comprehend it. If none exists, it may be very difficult, but not impossible, to understand the experience or the threat. There is no compelling reason for believing that the collective thinking process of a community can differ radically from that for its individuals. Recent U.S. experience does not include an earthquake disaster comparable in cost to the next New Madrid event. Central U.S. experience does not include any serious earthquake damage. The 1811/1812 events occurred at a time when there was little at risk near the epicentral region. It is not cynical to conclude that even if the Nuttli seven percent probability of occurrence before the year 2000 is found credible by decision makers who understand the reasons for the estimate, the devastating consequences of the earthquake may not be believed until it actually takes place.

Until we fully understand the consequences of the New Madrid earthquake, it is very likely that the payments toward earthquake preparedness will be deferred, even though experiences of other societies confirm that there is no better demonstration of the simple proverb "a stitch in time saves nine" than in the case of earthquake preparedness. A steady trickle of investment in preparedness may reduce the potential disaster to a minor event. That is why it is essential that we improve our technology in evaluation of structural vulnerability to earthquakes. As long as the damage estimates remain fuzzy, it will be difficult to convince decision-makers about the size of the risk in central U.S.

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TABLE 1

LOSSES CAUSED BY SELECTED U.S. EARTHQUAKES AFTER 1900
(Adapted from Reference 1)

	Year	Loss in Millions	ENR Index	Loss in 1990 Dollars Millions
San Francisco	1906	$524.0	100	14006.5
Puerto Rico	1918	4.0	159	67.2
Long Beach	1933	40.0	148	722.4
Helena	1935	4.0	166	64.4
Cornwall, NY	1944	2.0	235	22.7
Puget Sound	1949	25.0	352	189.8
Wilkes-Barre	1954	1.0	446	6.0
Hegben lake	1959	11.0	491	59.9
Alaska	1964	500.0	612	2183.8
Puget Sound	1965	12.5	627	53.3
Dulce, NM	1966	0.2	650	0.8
San Fernado	1971	553.0	875	1689.3
Whittier	1985	110.0*	2410	122.0
Loma Prieta	1989	5600.0**	2619	5715.5
	1990		2673	

Loss amounts converted to 1990 dollars on the basis of the Engineering News Record Building-Cost Index

*Reference 16

**Reference 15

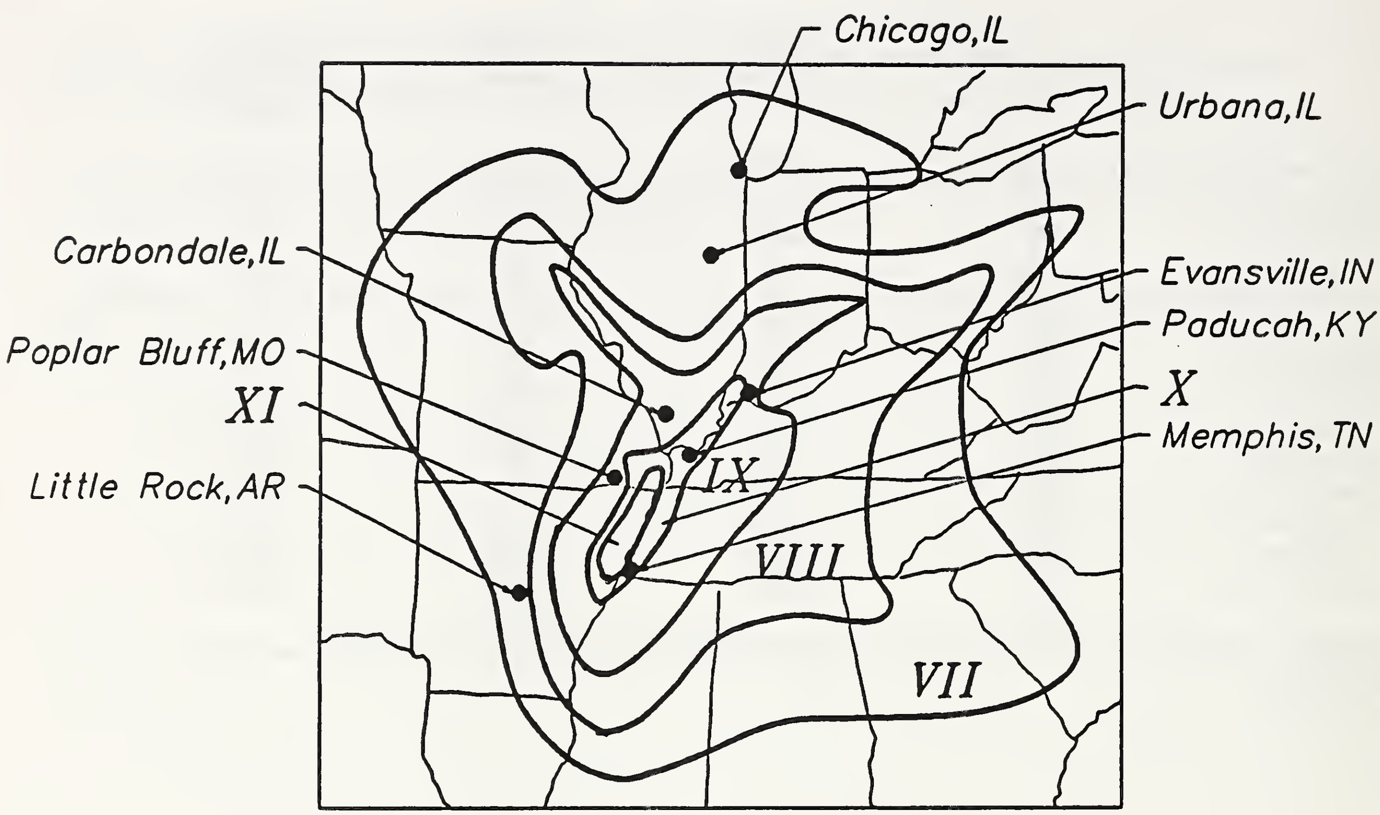


Figure 1 Estimated MM Intensities for a Repeat of the 1811/1812 Earthquakes in the New Madrid Region (Ref. 7)

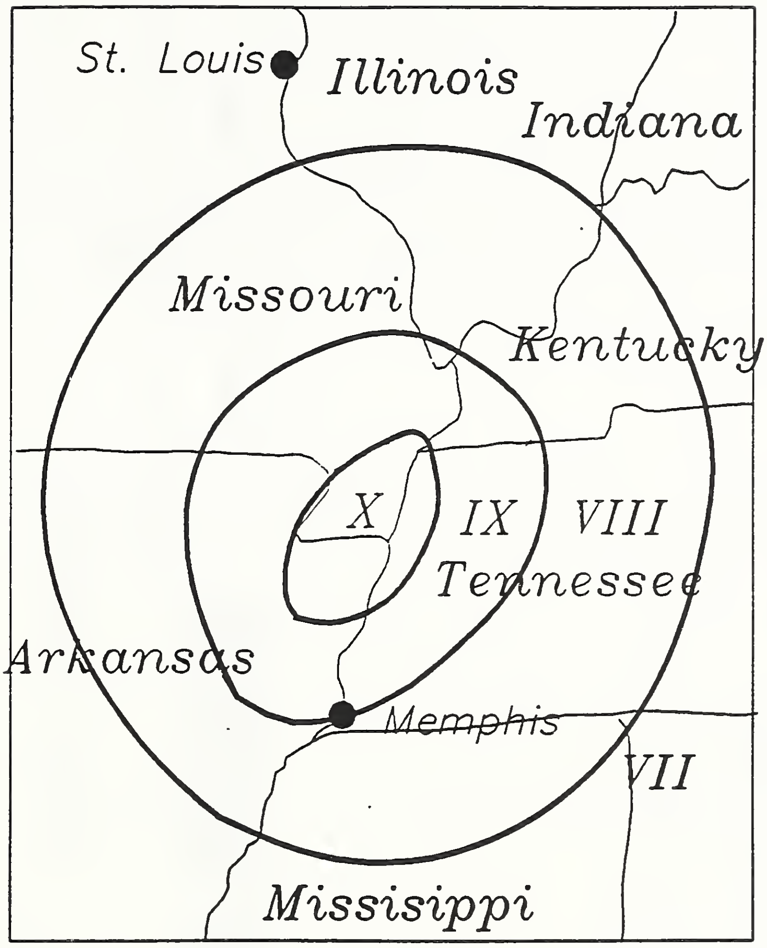


Figure 2 Distribution of MM Intensities from a Postulated Earthquake in the New Madrid Region (Ref. 9)

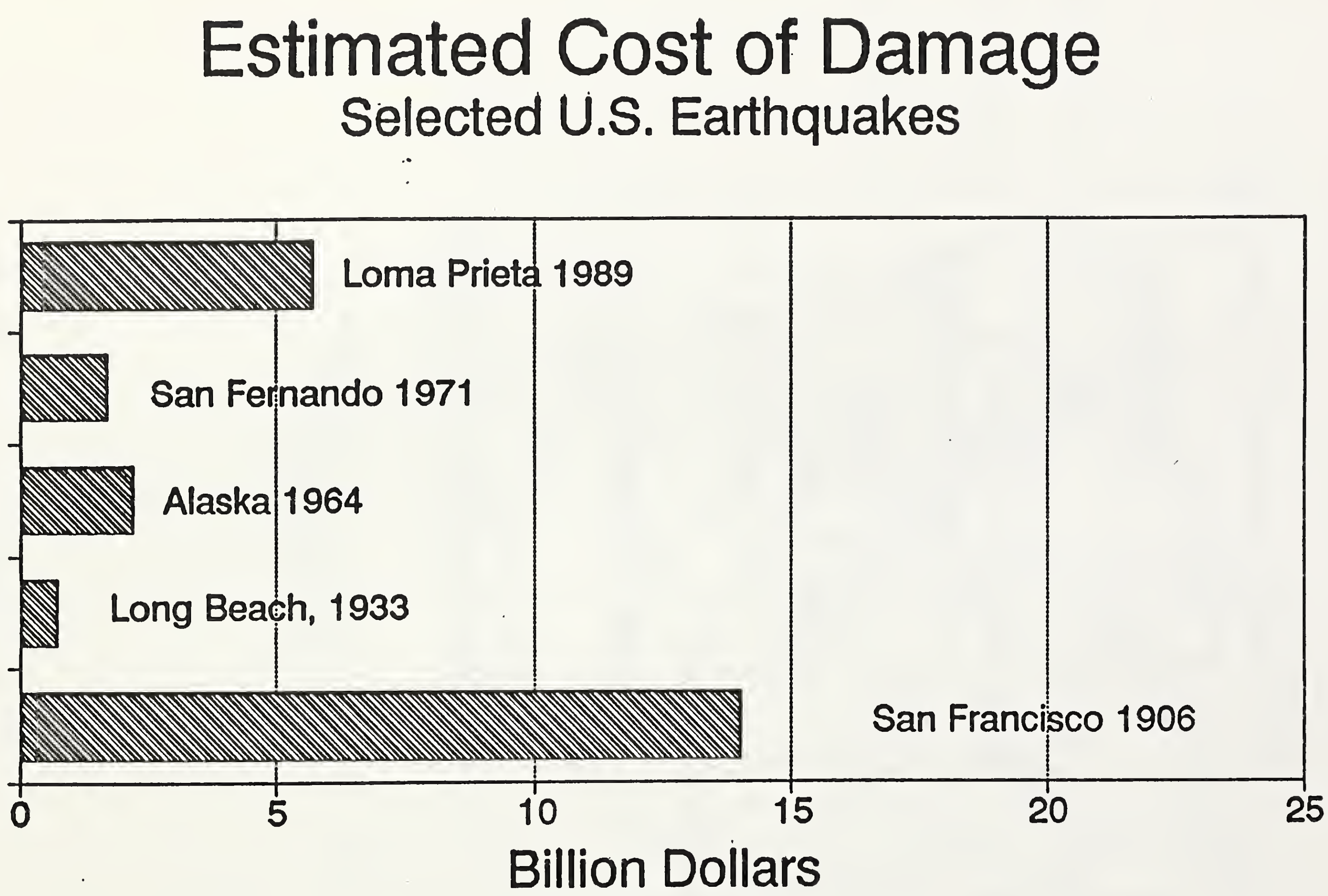


Figure 3 Losses in Five Selected U.S. Earthquakes (See Table 1)

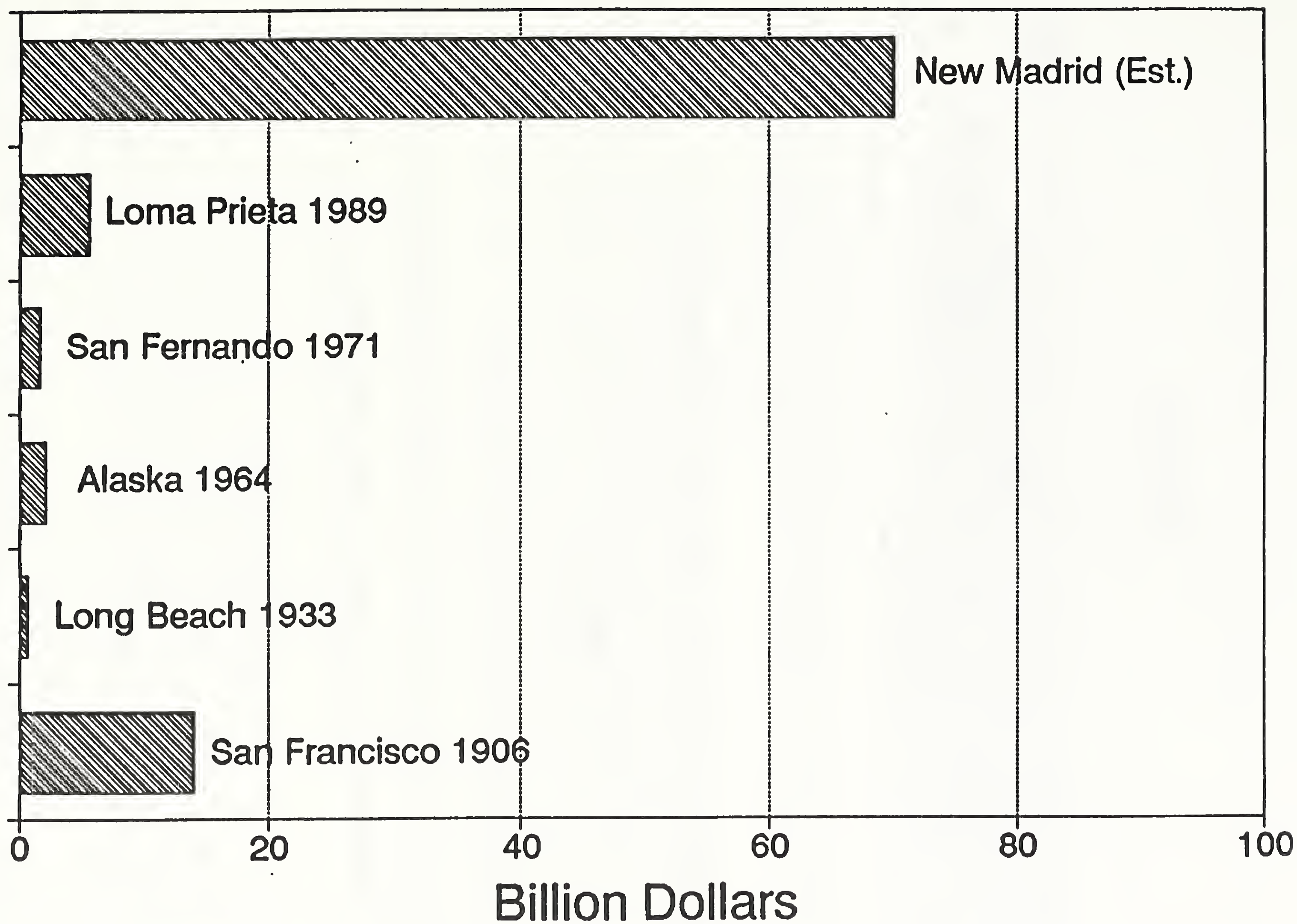


Figure 4 Estimated Loss from the Next New Madrid Event Compared with Losses from Five Previous Earthquakes

A RESEARCH AND DEVELOPMENT PLAN FOR UNDERSTANDING AND REDUCING EARTHQUAKE HAZARD TO CONSTRUCTION IN REGIONS OF LOW TO MODERATE SEISMICITY

by

Nancy L. Gavlin, Mete A. Sozen, James K. Wight and Sharon L. Wood

based on

The National Science Foundation Workshop

VULNERABILITY OF CONSTRUCTION TO EARTHQUAKE HAZARD

IN REGIONS OF LOW TO MODERATE SEISMICITY

NSF Grant No. BCS 89-17674

9-10 November 1989

University of Illinois

Urbana, Illinois

SUMMARY

It is generally accepted that strong ground motion will occur, as it has in the past, somewhere in central and eastern U.S. Furthermore, there is relatively little disagreement in the building professions about the fact that much of the construction in those regions is vulnerable to shaking. However, there is no generally accepted set of procedures for estimating how much of the construction will suffer given a certain ground motion.

This is a report on a National Science Foundation Workshop held to develop a research and development plan toward the definition and reduction of the earthquake hazard in regions of low to moderate seismicity.

A program is outlined for (a) adapting and improving the existing vulnerability-assessment technology to suit the needs of regions with low and moderate seismicity and (b) increasing the competence of building professionals in earthquake-hazard issues of those regions. The program comprises the four components listed below. Percentages of the total resource assigned to each component are indicated in parentheses:

1. Building Inventory (*2%*)

2. Behavioral Research (*50%*)

3. New and Improved Methods for Nondestructive Testing (*18%*)

4. Codes, Design Resources, and Training Courses (*30%*).

Research and development is recommended for building structures made of masonry, concrete, and steel. Earthquake damage to timber structures was considered to be a relatively minor threat to life safety in seismic regions of central and eastern U.S. It is recommended that 50% of total available R & D resources be used for masonry, 30 % for concrete, and 20 % for steel.

For each material type, different research activities and directions are suggested. A five-year program is proposed at an average annual support of approximately $6,000,000. In view of the immense direct and indirect losses possible from an earthquake in Central or Eastern U.S., the recommended support level is based on an estimate of its efficient use by available research and development sources rather than in relation to the cost of construction and lives at risk.

INTRODUCTION

The events that led to the collapse of the elevated portion of Route 880 in Oakland, CA, provide a metaphor for construction in regions of United States with low and moderate seismicity. There was little disagreement about the fact that strong ground motion would occur in Oakland. Among professionals, there was little disagreement that the structure for the elevated highway was vulnerable. However, there must have been disagreement about when and how to fix the structure.

Fig. 1 Relative Seismicity, Central & Eastern U. S.

There is little disagreement about the expectation that earthquakes will occur in regions of United States with low and moderate seismicity. There is little professional disagreement about the fact that much of the construction in those regions is vulnerable to shaking. There is, however, apparent disagreement about how to assess vulnerability and what to do about it.

The disagreement is compounded by the vicious cycle that current methods for evaluating vulnerability are calibrated for regions of high/frequent seismicity. They are binary (options between yes and no are ignored) and tend to exaggerate the risk. An exaggerated risk makes the mitigation costs appear overwhelming. And the perception of an overwhelming cost defers action.

The first essential step toward the implementation of a cost effective program to manage the earthquake risk in central and eastern U.S. is the development of methods suitable for vulnerability assessment of construction in the region and the training of the professional community to use the methods. To outline a research and development program for needed improvements in the practice of vulnerability assessment, a National Science Foundation Workshop was held in Urbana, Illinois, on 9 and 10 November 1989. The workshop brought together a group of 33 professionals, most of them from central and eastern U.S. The focus of the workshop was on structure. Seismological, geotechnical, and architectural issues were not considered directly. Figure 1 provides a qualitative description of the regions considered and their relative seismicities.

In preparations for the workshop, construction was considered in four categories according to the material providing (by design or incidentally) lateral resistance: masonry, concrete, steel, and timber. Discussions during the initial phases of the workshop led to the decision to concentrate on the first three categories. Issues related to buildings with their lateral strength provided by timber were eliminated from the agenda because that type of construction was considered to represent a relatively minor risk during earthquakes to life safety in central and eastern U.S.

The need for research and development toward improvement of earthquake hazard assessment in central and eastern U.S. results from the fact that the current evaluation technology evolved primarily in relation to types of construction and risks in regions of high seismicity. For evaluating buildings in an environment where strong ground motion is frequent, it is justifiable to ignore questions of dynamic response related to structural materials and systems that hold little promise for survival in a great earthquake. On the other hand, in regions where the expected ground motion is moderate, the earthquake has a return period exceeding 300 years, and there exists a large body of construction not designed to resist lateral forces, evaluation cannot afford to be as demanding. For example, it is understandable that unreinforced concrete masonry construction must not remain as built in a region of high seismicity. However, in a region of low seismicity, there may be a valid basis for tolerating existing construction in unreinforced masonry. To rationalize that action, it is necessary to develop evaluation methods for such construction subjected to low or moderate excitation. To develop such methods, behavioral research is required on dynamic response of masonry, a topic summarily and justifiably ignored for research related to needs of highly seismic regions. Parallel examples exist for construction in concrete and steel.

It was the consensus of the workshop participants that a research and development program on evaluation techniques would have a substantial impact on life safety and economical issues related to earthquake risk in regions of low and moderate seismicity by (a) enabling realistic damage estimates, (b) reducing the volume of constructed facilities that would be slated for demolition or strengthening if evaluated on the basis of current practice, and by (c) identifying efficient strengthening techniques.

The research and development plan outlined in the following sections is based on the discussions held during the two-day workshop. The research and development plan is not intended to start from ground level but to enhance and modify the existing evaluation technology to suit needs of regions with low and moderate seismicity and to increase the competence of building professionals working in those regions in the practice of vulnerability evaluation.

A preliminary "building classes" list was developed in order to serve as a framework for discussions during the workshop. The list is included in Appendix A. Summaries of the discussions held during the workshop are in Appendices B and C.

Peer Review

In developing the program outline, it was recognized that the proposed research and development activities would be carried out in the environment of the peer-review system. Topics of research and development were identified in broad categories. The creativity that is a natural part of the peer-review system will identify the specific topics and directions. At the same time, it is expected that peer reviews will prevent, in most cases, unnecessary repetitions of work already done.

Inventory

One of the first if relatively modest requirements for a balanced program of earthquake risk management in seismic regions of central and eastern U.S. is a building inventory that will permit informed decisions about directions of the research and development plan.

It is proposed that the building inventory be developed in several phases with different breadths and depths of coverage. Phase 1 is intended to discover and consolidate existing inventories compiled by Federal and State Governments, trade organizations, insurance firms, and corporations. Activities in Phase 2 will depend on outcome of Phase 1. Its objective is to develop a building inventory that would improve general planning for resource allocation. Phase 3, which may be initiated concurrently with Phase 2, is to provide detailed information about the structural characteristics of buildings in specific locations. Its main objective is to help decide resource allocations to specific research and development projects.

Phases 1 and 2 are to be carried out by research and development organizations using, where convenient, nonprofessional help. Phase 3 must be carried out by engineers with training and experience in earthquake resistant design. It is anticipated that this portion of the program will cost approximately 2% of the total.

Overall Distribution of Resources

In the initial phase of the program, it is proposed that the research and development resources be assigned approximately in the proportion indicated in Fig. 2 to the material types considered. These relative allocations are recommended on the basis of the assumed relative volumes of buildings exposed to risk as well as estimated return, in terms of reducing the risk, on invested resources. They are likely to be changed as the results of inventory surveys become available.

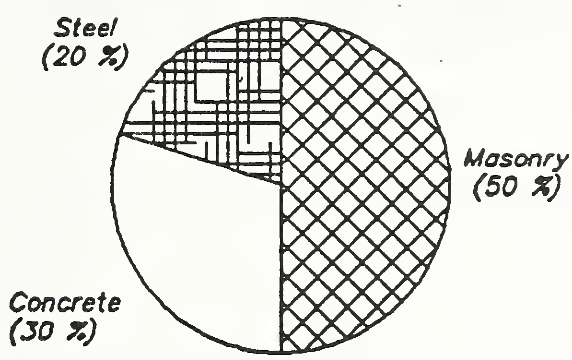


Fig. 2 Distribution to Materials

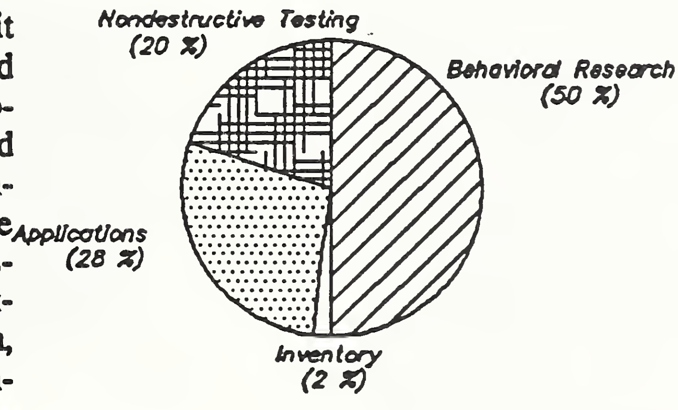


Fig. 3 Distribution to Tasks

Research and Development Activities

The proposed research and development activities are divided into three main categories:

- Behavioral Research
- Improvement of Nondestructive Testing Methods
- Applications

For each building material type, these categories include different tasks, but the objectives for each category are similar.

Behavioral research includes experimental and analytical studies leading to understanding of earthquake response of constructed facilities. Experimental projects may involve tests of materials, structural components, component assemblies, and actual structures (field tests). Experimental projects on response of structural systems are expected to include extensive analytical work with the object of modeling the observed phenomena. The work is expected to be accomplished by a mix of research laboratories (university) and development laboratories as suggested in Fig. 4. It is recommended that approximately one half of the total resources be assigned to behavioral research (Fig. 3).

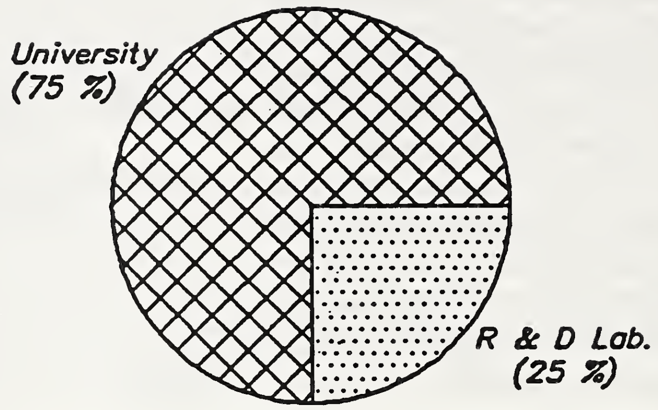


Fig. 4 Distribution to Sources, Behavioral Research

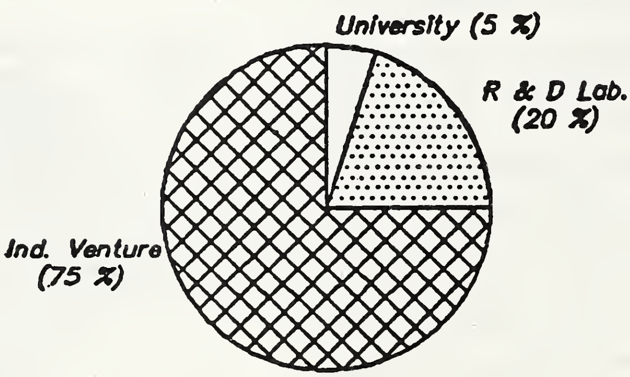


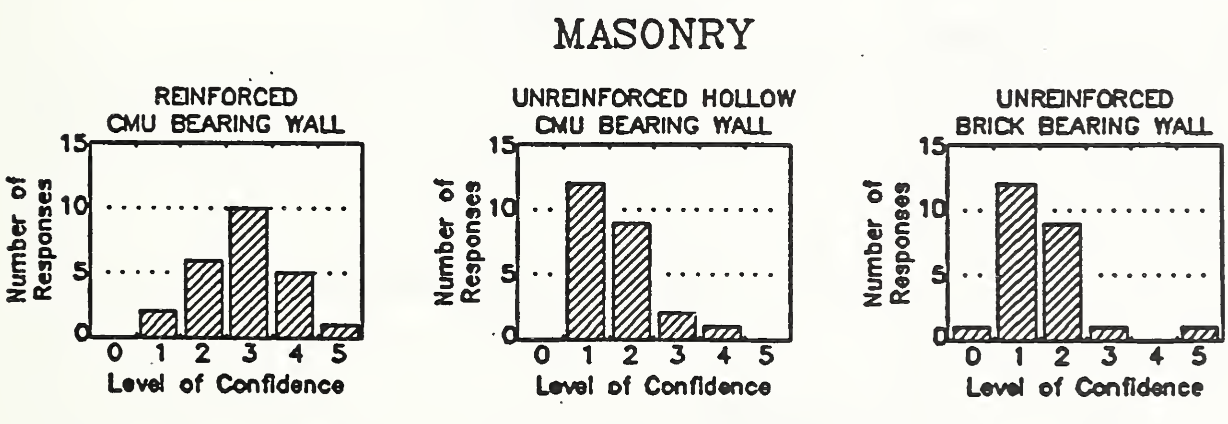
Fig. 5 Distribution to Sources, Nondestructive Testing

Work toward improvements in nondestructive testing is considered to be the proper domain of venture industries encouraged by incentives from federal government. The function of research and development laboratories is seen as one of defining the needs for new and improved techniques and evaluating the venture products. The relative allocation of resources indicated in Fig. 5 is based on that perspective. It is recommended that this component of the program be assigned approximately 18% of the total resources (Fig. 3).

The scope of applications includes development of technical manuals, building codes, and education of students and building professionals. These activities are to be carried out by architectural/engineering firms, professional institutes, and universities. It is anticipated that a substantial portion of the operation will be done by consortia bringing individuals from firms, institutes, and universities together in activities related to design of manuals and dissemination of information. Approximately 28% of the total resources should be assigned to this task (Fig. 3).

MASONRY

In central and eastern U.S., reinforced masonry buildings represent a relatively small portion of the building inventory. Furthermore, a reinforced masonry building is likely



Column "0" indicates no opinion.

Fig. 6 Confidence Level in Vulnerability Assessment, Masonry to pose a much smaller threat in the event of being subjected to an earthquake. And current evaluation methods promise a higher degree of success in identifying vulnerability in reinforced masonry than in unreinforced masonry. Therefore, the emphasis of research and development in vulnerability assessment is placed on unreinforced masonry.

Figure 6 shows the results, for masonry, of an opinion poll conducted before the workshop among the workshop participants. A general lack of confidence is indicated in current methods of practice for estimating degree of damage. That observation coupled with the information that the inventory of unreinforced masonry construction is large in regions of interest leads to the conclusion that it is proper to assign a substantial portion of available research and development resources to unreinforced masonry as indicated in Fig. 2. In the event of a strong earthquake, life and property losses caused by collapse of unreinforced masonry construction are likely to be heavy.

Behavioral Research

Because it has been correctly considered to be unsuitable for highly seismic regions, behavioral research in unreinforced masonry has been meager. There is a considerable amount of work to be done on the fundamental aspects of in-plane response under static and dynamic loading of unreinforced masonry. Much of the work is in the experimental field with appropriate analytical support to generalize the results. The needed work ranges from investigations into basic behavior of solid wall panels to studies of the effects of openings, joints, and mortar quality. Available research on out-of-plane response needs to be enhanced. Opportunities to test to failure actual buildings and their components should be fully exploited. There is a strong need to understand three-dimensional response of unreinforced masonry.

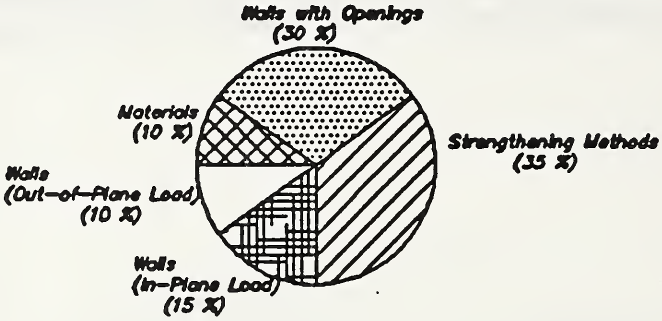


Fig. 7 Distribution of Behavioral Research, Masonry

Because it is believed that a large fraction of existing masonry will require remedial action, it is proposed that some of the research work be coupled directly with work toward methods of repair and strengthening (Fig. 7).

Nondestructive Testing

Methods to determine material properties of in-place block, brick, and mortar need to be improved. Equipment and methods are needed for determining conveniently whether and how in-place masonry is reinforced and for locating wall ties in masonry cladding.

Applications

There is a need for catalogs that define probable characteristics of masonry built during various epochs of construction in the affected regions. The profession should be provided with methods to evaluate the degree of deterioration in existing masonry.

A reference book containing annotated photographs of recurrent cases of damage to unreinforced masonry subjected to moderate and low ground motion would play an important role in the education of the engineering and architectural communities.

Manuals should be developed describing (1) possible load paths for lateral-force resistance, (2) methods of evaluating overall resistance, and (3) probable causes of serious damage. These manuals should also address questions on behavior of diaphragm-wall connections.

CONCRETE

Concern about behavior of low and medium-rise reinforced concrete buildings in regions of low to moderate seismicity arises primarily from the perception that such buildings are seldom designed for lateral loads, they often have bashibazouk framing that does not fit into the canon of framing types for earthquake resistance, and they lack detail needed for continuity. The state of the art for vulnerability assessment of such structures is not that far away from that for unreinforced masonry. Research and development efforts may demonstrate the feasibility of continued use, under certain ground-motion demands, of a substantial portion of the reinforced concrete inventory that would be considered to be unsuitable on the basis of current assessment practice. Figure 8 indicates a measure of the current level of confidence, based on a poll of the workshop participants, in vulnerability assessment for various general classes of construction in concrete.

Behavioral Research

Research is needed on behavior of frames with "typical" details required under various versions of the ACI Building Code. Emphasis on research on earthquake resistance of reinforced concrete has been concerned primarily with structural systems having adequate details and on development of such details. There is very little

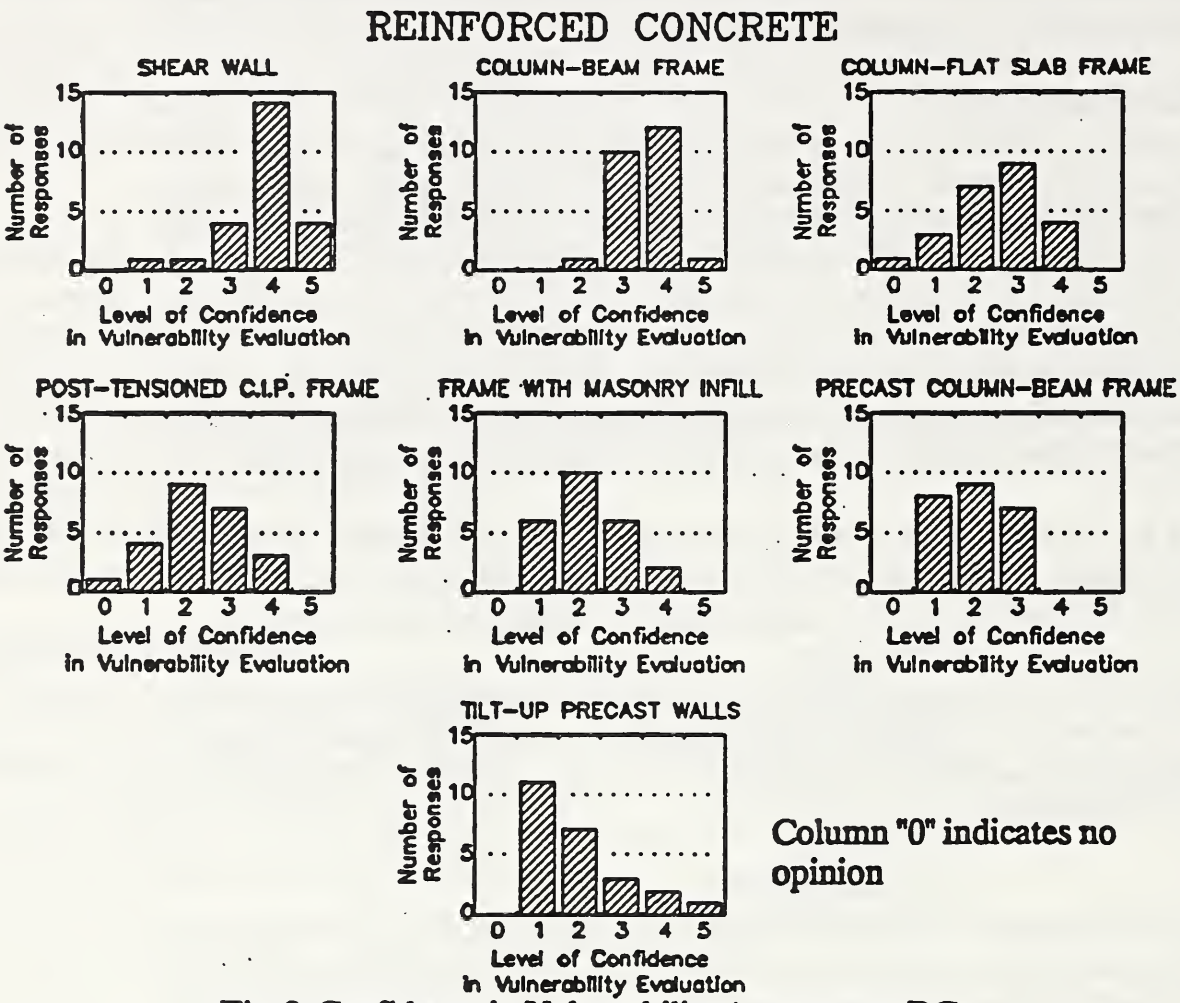


Fig. 8 Confidence in Vulnerability Assessment, RC

experimental information available on behavior under lateral static/dynamic loading of structural systems that would be considered substandard by current concepts of earthquake resistance and their connections.

Precast and post-tensioned systems, especially those with "dry" connections or connections that are not made using cast-in-place concrete, need considerable behavioral research in order to provide the appropriate base of experimental and analytical information for development of evaluation tools.

Frames with masonry infill represent another important topic of needed research. The focus should be on frames with details that are typical for existing construction.

As in the case of unreinforced masonry, tests to destruction of existing construction, whenever there is an opportunity to test a "typical" structure at an appropriate cost, are recommended. Instrumentation of existing buildings to capture strong-motion response is an option that deserves consideration.

Based on the evaluation of confidence in existing practice and estimates of relative populations of construction types, it is proposed that behavioral research and development resources be committed in the proportions shown in Fig. 9.

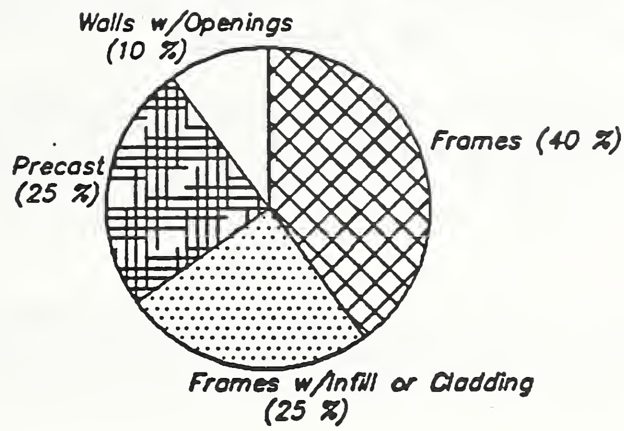


Fig. 9 Distribution of Behavioral Research, Concrete

Nondestructive Testing

To establish the load resistance mechanisms of reinforced concrete structures, it is essential to have reliable information about the arrangement, location, and amount of reinforcement. Industry should be encouraged to develop portable devices that will provide the needed information about reinforcement in an existing building. Combination of existing radar or other technologies with image-processing methods may provide a solution to this important problem.

Methods and devices for determining concrete strength in existing construction should also be improved.

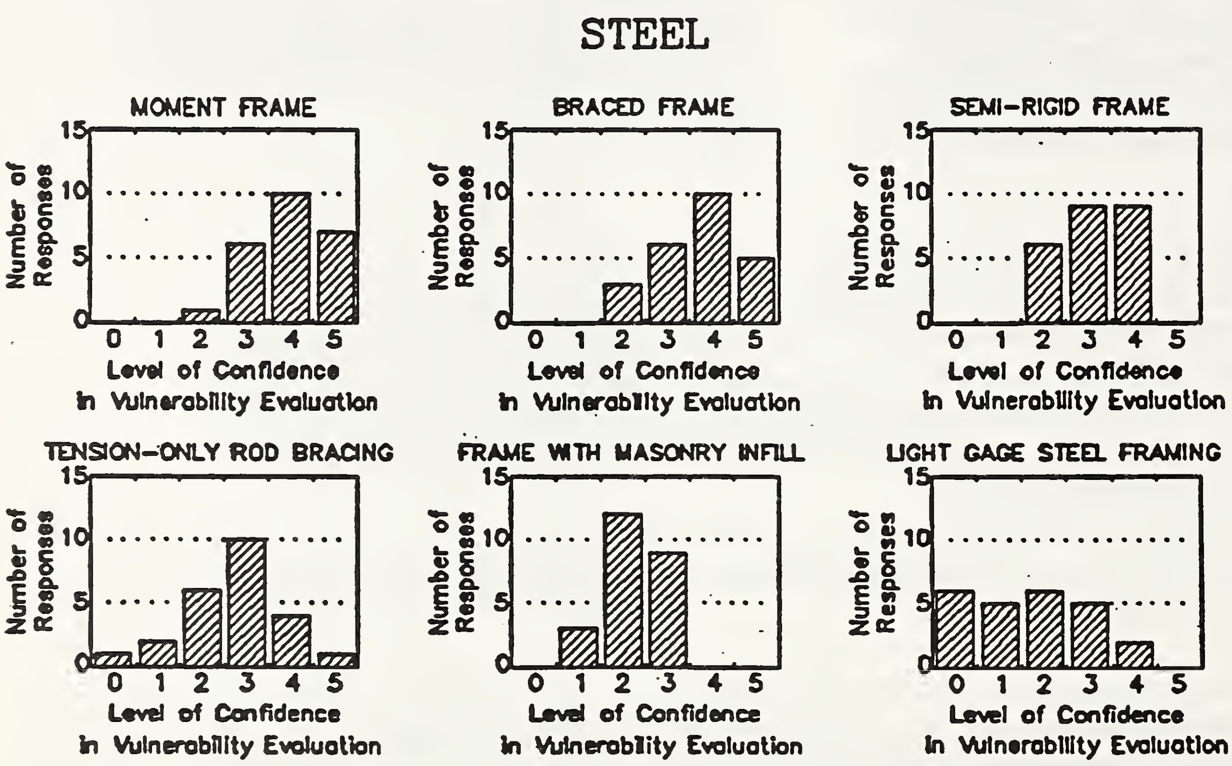
Applications

Because reinforced concrete construction has gone through a rapid and strong evolution, current practitioners are not always in a position to be familiar with the structural details of an existing building. A valuable practical resource for evaluating reinforced concrete buildings is a series of manuals/catalogs that describe the critical structural characteristics and material strength ranges of reinforced concrete buildings built at different times. The manuals should also provide the engineers with procedures of observation and/or testing to determine effects of deterioration as well as alerting the engineer to conditions under which certain types of construction is likely to be susceptible to aging/exposure effects. This series of manuals should be complemented by a set of manuals for detailed evaluation.

As in the case of masonry, effective applications in this topic require education of the professional community as well as the development or re-orientation of courses at universities.

STEEL

Many construction types and details used in central and eastern U.S. have never been evaluated for response to earthquakes. To make improvements in applications, it is essential first to develop information on behavior of such structures under reversals of



Column "0" indicates no opinion.

Fig. 10 Confidence in Vulnerability Assessment, Steel lateral load. Figure 10 shows the results of a poll conducted among the workshop participants to determine their confidence in the success of current practice to evaluate earthquake hazards for steel construction.

Behavioral Research

It is proposed that experimental information, under conditions simulating earthquake effects, be developed on behavior of flexible connections, concrete-encased steel sections, light bracing elements, built-up sections, diaphragm connections, and base-plate connections. Effects of deterioration with age should be considered in the investigations.

Behavior of frames with masonry infill and with attached cladding need to be investigated.

Tests to destruction of actual structures should be conducted.

Nondestructive Testing

Improvements are needed in methods for determining structural properties such as steel type, steel condition, and weld quality.

Because steel structures in central and eastern U.S. tend to be flexible, low-amplitude vibration tests are proposed primarily to raise the consciousness of the engineering community about the likelihood of serious nonstructural damage.

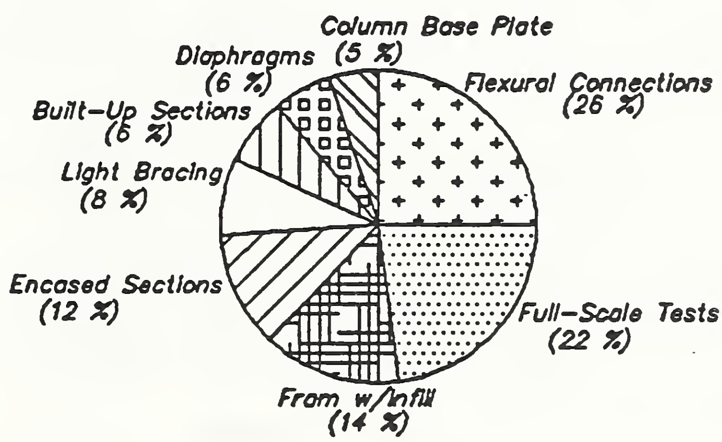


Fig. 11 Distribution of Behavioral Research, Steel

It is proposed that available support for behavioral research be distributed in the proportions shown in Fig.11.

Applications

There is a need for documents containing information on recurring and anticipated structural problems with the types of steel construction used at various times in central and eastern U.S. need to be produced. These documents must also address questions of deterioration caused by age and exposure.

Manuals and supporting courses need to be developed to transmit the technology to practicing professionals as well as to students.

SCHEDULE

The rate of progress of the proposed research and development program depends not only on the availability of support and urgency of the need but also on the capacity of the existing laboratories and professional firms willing and capable to implement the program. Although it is not essential, it is desireable that most of the work, especially the developmental part, be carried out in the regions where the results will be applied. Fortunately, various components of the National Earthquake Hazard Reduction Program have involved many laboratories and firms in central and eastern U.S. The

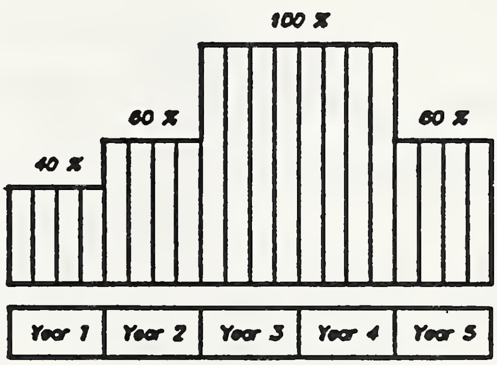


Fig. 12 Schedule

facilities in the region for research and development in problems related to earthquake effects are adequate. The program could be initiated in full size during its first year. But in any such undertaking there are bound to be startup problems as well as redefinitions of the goals. Therefore, it is proposed (Fig. 12) that the program reach its maximum level of $8 million (1989 dollars) in the third year. Whether it is reduced in the fifth year will depend on needs and accomplishments of the first four years. It is important to note that the proposed amount represents a trivial fraction of the cost of the existing construction at risk. The suggested support was based on an estimate of efficient utilization of funds by current research and development sources.

COMPARISON OF ASEISMIC DESIGN BETWEEN U.S. AND JAPAN

by
Shin OKAMOTO[1], Takayuki TERAMOTO[2], and Toshio OKOSHI[3]

ABSTRACT

This paper presents comparative designs of two steel buildings and three reinforced concrete buildings which were originally designed in accordance with U.S. codes. As regards steel buildings, the current Japanese code imposes the design shear force of 2 or 3 times as large as U.S. codes, while the drift requirements by U.S. codes are severer than those by the Japanese code. But, the resulted steel quantities of the U.S. and Japanese designs do not make so remarkable different as might be expected only through the design shears. And as regards reinforced concrete buildings, the seismic coefficients due to the Japanese Code in the examples vary from about 40% greater to more than twice as great as those from the ATC 3. Then the design base shear vary from about 60% greater to almost three times as great. So the variation in the total concrete volume for the columns is about 40% greater to about twice the volume resulting from ATC 3. The reinforcing amount does not go up proportionally. The beam concrete volumes vary from about 30% greater to the about 100% greater than those resulting from ATC 3 design, with beam reinforcing from 20 to 30% greater.

KEYWORD: A seismic design; ATC 3; Japanese Code; Quantity; Seismic design coefficient; UBC

1. INTRODUCTION

Some comparative seismic designs using ATC 3-06[1] and UBC 1982[2] were carried out by ATC, and two steel structure buildings and three reinforced concrete buildings were redesigned by JSCA (Japan Structural Consultants Association) using the Japanese current code. These comparative deisgns were reported at "The Second U.S.-Japan Workshop on Improvement of Seismic Design and Construction Practices", and the five papers dealing with this comparative designs exercise were presented at "the 9th World Conference of Earthquake Engineering" in Japan, August 1988. The analyses of these comparative designs were reported at "The Third U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices" in Japan, July 1988.

In the achievement of the structural design, regulations and codes have played an important role with regard to safety. Since the regulations and codes are the result among the latest analytical study, past experiences and practical requirements, they have been occasionally revised. In the region of high seismicity, actual experiences of earthquake damage of structures have imposed the revision of seismic regulations and requirements.

2. STEEL BUILDINGS

Redesigns of a 10-story and a 19-story steel buildings in Los Angeles are presented in accordance with current Japanese codes. The 10-story building which was designed by using ATC 3-06 is redesigned. The 19-story buildings was originally designed using the 1964 City of Los Angeles Building Code and was examined as an example model for redesign to comply with the 1982 UBC and ATC 3 requirements. It was of interest to redesign the same building originally confirmed to U.S. codes using current Japanese aseismic codes. This paper clarified the differences between U.S. and Japanese aseismatic codes, especially how the strength and drift requirements are dominated to tall building of steel structures. Quantities of steel required are also compared between U.S. and Japanese codes designs.

Aseismatic design of building structures in Japan necessities both the allowable stress design based on elastic analyses to moderate earthquake motions and the limit design based on plastic analyses to severe earthquake motions, where the standard shear coefficient shall be not less than 0.2 and 1.0, respectively.

2.1 A 10-Story Steel Building

2.1.1 Description of Building

This structure is a 10-story office building, 125 feet x 180 feet in plan. The bay sizes are 25 feet in the transverse direction and 30 feet in the longitudinal direction (See Fig. 2.1). The first story height is 22'-6" and the upper nine stories are 13'-6". The total building height is 144 feet.

The floor and roof decks are 3-inch standard weight concrete over steel decking supported on steel beams, girders and columns.

2.1.2 Framing

The steel frames along the perimeter of the building are ductile moment resisting frames. All the interior horizontal and vertical framing is designed to support vertical loads only. The floor construction is designed as a diaphragm to distribute the lateral forces to the perimeter frames. The exterior and interior walls and

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partitions are of metal studs, providing no lateral load resistance capability.
In the Japanese design, all the columns are designed to resist lateral forces in both transverse and longitudinal directions.
Although a dual system of design incorporating both rigid frames and shear walls are common for such buildings in Japan, design for a rigid-frame-only building was provided for the purpose of comparison in this study.

2.1.3 Basic Seismic Loading Factors

The basic loading factors for each code is as follows:

Los Angeles City Code

Zone	4
Importance Factor I	1
Soil Factor S	1.5
Coefficient K	.67

Fundamental Elastic Period (Formula 23-3A)
Longitudinal N/A Transverse N/A
Fundamental Elastic Period (calculated)
Longitudinal 2.38s Transverse 2.38s
Coefficient CS:
Longitudinal .065 Transverse .065
Coefficient ZIKCS:
Longitudinal .043 Transverse .043
Base Shear ZIKCSW:
Longitudinal 1017 KIPS Transverse 1017 KIPS

ATC 3-06

Map Area	7
Seismic Hazard Exposure	Group II
Seismic Performance Category	C
Soil Profile Type	S2 or S3
Response Modification Coefficient R	8

Approximate Fundamental Period Ta:
Longitudinal 1.46s Transverse 1.46s
Calculated Fundamental Period T:
Longitudinal 1.79s Transverse 1.79s
Seismic Design Coefficient Cs:
Longitudinal .062 Transverse .062
Base Shear CsW:
Longitudinal 1453 KIPS Transverse 1453 KIPS

1988 Uniform Building Code

Zone	4	Z = 0.4
Importance Factor I		1
Soil Factors		1.5
Structural System Rw		12

Approximate Foundamental Period T:TA = 1.45s
TB = 1.79s
Coefficient C 1.27
Seismic Design Coefficient ZIC/Rw
Longitudinal .042 Transverse .042
Base Shear ZIC/RwxW
Longitudinal 991 KIPS Transverse 991 KIPS

Japanese Building Code

Zone	1	Z = 1.0
Soil Profile Type		Tc = 0.65
Design Spectral Coefficient		Rt=0.727
Standard shear coefficient		Co = 0.2

Approximate Fundamental Period T
Longitudinal 1.32s Transverse 1.32s
Seismic Design Coefficient ZRtAiCo
Longitudinal .1454 Transverse .1454
Base shear
Longitudinal 4347 KIPS Transverse 4347 KIPS

In Japan a check of ultimate strength of horizontal shear for sever earthquakes is required of all buildings greater than 31 meters in height. The ultimate strength of a story is defined as the total strength when the ends of girders or columns in the story reach the full plastic moments. The ultimate strength of this building is determined mainly by the yielding of the ends of girders. Table 2.1 shows the ultimate strength of each stories. The ratios of the ultimate strength Qu to the necessary ultimate strength Qun indicate that this building has enought strength to withstand severe earthquake motions.

2.1.4 Tonnage of Structural Steel

The tonnage of structural steel required to construct this 10-story building by the L.A. City Code is estimated to be 1,329 tons. The ATC 3-06 redesign requires 1,627 tons of steel. The Japanese Code redesign requires only 25% more steel.

The standard method of fabricating and erecting such buildings in Japan involves a significant number of small framing segments due to limited access and transportation facilities in most regions of the country. Short end sections of the girders are welded to the columns in the shop on the remaining filler section of the girders are then bolted to the stubs in the field. This practice offers the designer the opportunity to reduce the size of girders at the filler sections and reduce the tonnage of steel.

However, this practice together with the all-moment-frame concept greatly increases the number of costly connections as shown below.

Estimated number of welded and bolted connection per floor

	ATC 3-06	JPN Code
Welded Moment Connections	40	142
Bolted Moment Connections	-	142
Bolted Shear Connections	102	-

Estimated number of members fabricated and erected per floor

	ATC 3-06	JPN Code
Fabricated Pieces	113	255
Erected Pieces	113	113

2.2 A 19-Story Steel Building

2.2.1 Description of building

This structure is a 19-story office building, 110 feet x 240 feet in plan. The typical bay size is 30 feet in the E-W direction and 40 feet in the N-S direction (see Fig. 2.5). The first story height is 30' and the upper 18 stories are 13'4". The total building height is 270 feet. A two story mechanical penthouse occurs over a portion of the tower. This penthouse is a steel braced structure and in the analyses the weights were included as roof loads.

2.2.2 Framing

The seismic resistance in the tower is provided by four moment resisting frames in the E-W direction, and a dual system of five X-braced frames and moment resisting frames in the N-S direction. This dual system is coincided with both systems sharing common vertical and horizontal members.

2.2.3 Basic Loading Factors

The basic loading factors for each code is as follows:

1988 UBC

Zone	4	Z=0.40	
Occupancy Load		8,000 persons	
Occupancy Category		III	
Important Factor I		1.0	
Soil Profile Factor	S_2	1.2	
Building Type Rw			
E-W (M-R Frame) 12		N-S (Dual) 10	
Structural Period (Method A: $T=Ct(hn)^{3/4}$)			
E-W	2.33s	N-S	1.33s
Structural Period (Method B)			
E-W	3.20s	N-S	3.27s
Factor C (Method A)			
E-W	0.85	N-S	1.24
Factor C (Method B)			
E-W	0.69	N-S	0.68
(For drift determination, design factor C=Method B)			
Minimum Base Shear for Strength Design Vmin			
E-W	1,293k	N-S	2,225k
Minimum Base Shear for Deflection Design Vmin			
E-W	1,293k	N-S	1,529k
Concentrated Force at the Top for Strength Design Ft			
E-W	290k	N-S	165k
Concentrated Force at the Top for Deflection Design Ft			
E-W	290k	N-S	350k
Vertical Distribution of Seismic Forces			
Triangular distribution of V-Ft			
Torsion		5%	
Drift limitation	$0.03/Rw \leq 0.004$ times the story height		
E-W	0.0025	N-S	0.0030

Japanese Building Code

Foundamental Period			
E-W	3.16s	N-S	2.37s
Accelerogram data	El Centro	1940 NS	
	Taft	1952 EW	
	Osaka 205	1963 EW	
Damping Ratio		2%	

Elastic Dynamic Analysis
- Maximum Velocity of Input Motion 25 cm/s (0.82 ft/s)
- Story Drift Angle
  - E-W $1/211 \leq 1/200$ (14F)
  - N-S $1/201 \leq 1/200$ (7F)
- Strength and Ductility
  - E-W $0.95 \leq 1.0$ (1F)
  - N-S $0.95 \leq 1.0$ (6F)

Elasto-Plastic Dynamic Analysis
- Maximum Velosity of Input Motion 40 m/s (1.31 ft/s)
- Story Drift Angle
  - E-W $1/132 \leq 1/100$ (14F)
  - N-S $1/126 \leq 1/100$ (8F)
- Strength and Ductility
  - E-W $1.51 \leq 2.0$ (1F)
  - N-S $1.52 \leq 2.0$ (6F)

2.2.4 Tonnage of Structural Steel

The tonnages of structural steel required to construct this 19-story building by the 1982 UBC, 1988 UBC, ATC 3-06 and Japanese Code are shown in Table 2.3.

3. REINFORCED CONCRETE BUILDINGS

An easy comparison is to compute the effective seismic base coefficients on the mass of the building, and compare the coefficients from different codes. However, this does not yield an accurate comparison when utilizing reinforced concrete. The Japanese Code and the American Code treat the strength design of concrete quite differently in the application of various load factors.

U.S. concrete design utilizes ϕ-factor that reduces the strength values of the concrete and reinforcing steel, based on possible variations or deficiencies in the quality of the materials or workmanship in installation. The Japanese Code appears not to have such a reduction, but an increase in the nominal yield strength of reinforcing of 10%. The U.S. concrete design code also incorporates additional load components. Therefore, it is difficult to compare the seismic coefficients as an accurate indication of what the effects will be on the final designed structure.

A unique feature of the Japanese Code is that the seismic design of major building involves a two step procedure. The first step is the design phase based on resisting a moderate earthquake with only minor building damage. The second step requires that an elastoplastic analysis be performed on the designed structure to verify that the building will be able to survive a major earthquake without sudden collapse. This step entails reviewing the collapse mechanism at ultimate resistance to assure that their beams yield prior to development of shear yielding in the walls or columns.

Redesigns of a 9-story, 10-story and 20-story reinforced concrete buildings are presented in accordance with current Japanese Code.

3.1 Results of Comparative Design

Results of comparative designs of the three building are shown in Table 3.1.

3.2 Framing System

Framing systems appear to be very different in U.S. and Japan. In Japan, all columns, girders and walls are designed to be seismic elements, and so all frames are utilized to resist the lateral forces. In U.S., the frames are divided into the latteral resisting frames and the frames which are designed for vertical force only. In the usual case, th outer frames and shear walls in the core are utilized as the lateral resisting frames, and the inner columns and girders are small in size and support vertical forces only.

The main reason for these differences is considered to be the great difference in seismic forces allowed for in the two countries. For example, as the shear wall seismic forces becomes after large using the Japanese Code, uplift tension occur at the bottom of shear walls. Therefore their foundation design becomes uneconomical, or it becomes impossible to design these foundations. As a result, the other frames are designed to resist more seismic forces in order to reduce the seismic forces on shear walls.

Many other reasons may also exist, and the differences can be assumed also to issue from the different design and construction practices in the two countries.

3.3 Base Shear Coefficient

The Japanese Code requires larger base shear coefficients. This can be rationalized in various ways. A primary factor is probably the fundamental difference in the two societies. In the U.S. there has been a reluctance to increase the design forces unless substantiating evidance is overwhelming. Use of relatively low coefficients goes back more than 50 years. Without any significantly tragic modern earthquake damage, this reluctance is likely to continue. For that reason, the United States had tended to emphasize ductile detailing provisions and the insertion of additional load factors in the design of components. On the other hand, Japan has experienced more damaging moderate earthquakes in the urban areas than other developed countries. As a result they are justifiably more conservative than the United States. Emphasis in the Japan Code is that the buildings are to survive moderate earthquake with minimal material damage. In the United States, it has traditionally been the object to maintain life safety without great regard for material damage.

The seismic coefficients due to the Japanese Code vary from about 40% greater to more than twice as great as those from the ATC 3. This increase is amplified when comparing the base shears due to the generally more massive buildings resulting from Japanese design practice The design base shears vary from about 6C greater to almost three times as great.

3.4 Quantities

The variation in the total concrete volume for the columns is about 40% greater to about twice the volume resulting from ATC 3. The reinforcing amount does not go up propotionally probably because the Japanese practice generally use lower concrete stresses so that the reinforcing steel does not increase proportionally. Similarily,
the beam concrete volumes vary from about 30% greater to about 100% greater than those resulting from ATC 3 design.

CONCLUSIONS

Comparative designs of the steel structure by the U.S. and the Japanse codes have been discussed in this paper through the case studies of a 10-story and a 19-story buildings. Quantities of the steel needed have been also estimated and compared between U.S. and Japanese code design. It should be emphasized that the current Japanese code imposes the design shear force of 2 or 3 times as large as U.S. codes, while the drift requirements by U.S. codes are severer than those by the Japanese code. The resulted steel quantities of the U.S. and Japanese designs, therefore, do not make so remarkable differences as might be expected only through the design shears.

From comparative designs of the reinforced concrete structure by ATC 3-06 and the Japanese code, it appears that the Japanese code requires designs to meet higher lateral forces that ATC 3-06, so large structural members result using the Japanese Code.

Direct comparison of codes, design and construction practices in the U.S. and Japan is very difficult and could be misleading. Local requirements and actual field condition play a very significant role in evaluating the relative merits of any system.

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d. Robert K. Burkett, "Summary of Comparative Case Studies of Reinforced Concrete Structures."

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Table 2.1 Ultimate Strength

Story	Ultimate Strength Qu (ton)	Necessary Ultimate Strength Qun (ton)	Qu／Qun
10	1925	456	4.22
9	1600	878	1.82
8	1842	1222	1.51
7	2265	1813	1.25
6	2647	2109	1.25
5	2885	2362	1.22
4	3098	2140	1.45
3	3582	2281	1.57
2	4423	2388	1.85
1	3484	2459	1.42

Table 2.3

Design code	Seismie System	Non-seismie System	Total Tons
1982 VBC	3342	1647	4989
1988 VBC	3695	1647	5342
ATC 3-06	5624	1647	7271
Japanese Code	4540	2403	6943

Table 2.2 COMPARISON OF CODE REQUIREMENTS

DIRECTION	BASE SHEAR COEFFICIENT Cs	BASE SHEAR S_B (KIPS)	ALLOWABLE STORY DRIFT R	Cs/R	S_B/R
CODE	BOTH DIRECTIONS	BOTH DIRECTIONS	BOTH DIRECTIONS	B.D.	B.D.
JAPANESE BUILDING CODE	0.1454 (1.0)	4347 (1.0)	1/200 (1.0)	29 (1.0)	394×10^3 (1.0)
L.A. CODE	0.043 (0.3)	1020 (0.23)	1/300 (0.67)	12.7 (0.44)	139×10^3 (0.35)
ATC 3	0.062 (0.43)	1453 (0.33)	1/370 (0.54)	23 (0.79)	240×10^3 (0.61)
1988 UBC	0.0423 (0.29)	991 (0.23)	1/300 (0.67)	12.7 (0.44)	136×10^3 (0.35)

Table 3.1 Results of Comparative design (1)

Comparative Items (ATC 3-06 / Japanese Code)	Building-A	Building-B	Building-C *1
(Dir.)			
1. Number of stories	9+Basement 1	10+Basement 1	20
2. Location	Los Angeles Tokyo	San Jose Tokyo	Los Angeles Tokyo
3. Framing system (X)	Dual System Dual System	Dual System Dual System	Dual System Dual System
(Y)	Shear wall Dual System	Moment Resisting Frame System Moment Frame System	
4. Natural period (X)	T=0.53 sec. T=0.72 sec.	0.69 (0.44) 0.76	1.37 1.40, 1.80 *2
(Y)	T=0.81 sec. T=0.72 sec.	0.93 (0.71) 0.76	
5. Design base shear coef. 1st-stage (X)	0.16 0.25	0.124 0.197	0.0584 0.127
(Y)	0.12 0.25	0.103 0.197	
2nd-stage (X)	Non 0.3	Non 0.394	Non 0.18
(Y)	Non 0.35	Non 0.295	
6. Vertical distribution of shear force	Inverted triangular distribution Ai-distribution	Inverted triangular distribution Ai-distribution	Inverted triangular distribution Ai-distribution
7. Horizontal distribution of shear force (X)	Ext frame 100% Int frame 0% Ext frame 50% Int frame 50%	Ext 95% Int 25% Ext 60% Int 40%	Ext 100% Int 25% Ext 40% Int 60%
(Y)	Ext shear wall 100% Int frame 0% Ext shear wall 50% Int frame 50%	Ext Int Ext 50% Int 50%	Int 50%

Note :*1 Building-C has a symmetric plan, so the values at Y-dir. are same to that at X-dir.

*2 Value 1.40 for elastic stiffness, 1.80 for reduced stiffness

Table 3.1 Results of Comparative design (2)

Comparative Items (ATC 3-06 / Japanese Code)	Building-A	Building-B	Building-C
(Dir.)			
8. Building weight (t)	10.572 11.645	11.113 18.233	26.400 35.300
9. Member section			
1) Girder (cm×cm) (X)	Int frame (5F) 35×70 50×90	(8F) 30×59 (2F) 30×90 (8F) 50×120 (2F) 65×130	(2F) 68×90 60×110~120
(Y)	Ext frame (5F) 35×110 50×150	(Int) 65×60 (Ext) 45×120 (8F) 40×120 (2F) 55×130	(2F) 45×60 60×90
2) Column (cm×cm)	Int frame (4F) 50×50 80×80 Ext frame (4F) 70×70 80×80	(Int) 60×90 (Ext) 45×180 (7F) 100×100 (1F) 120×120	(1F) 90×90 100×100
3) Shear wall (cm) (X)	30-40 20-30	30 20	30-35-40 25-35-40
(Y)	30-40 20-30		
10. Quantities Forms (m²)			69,800 82,400
Concrete (m³)			12,100 15,700
Rebar (t)			1,320 3,240

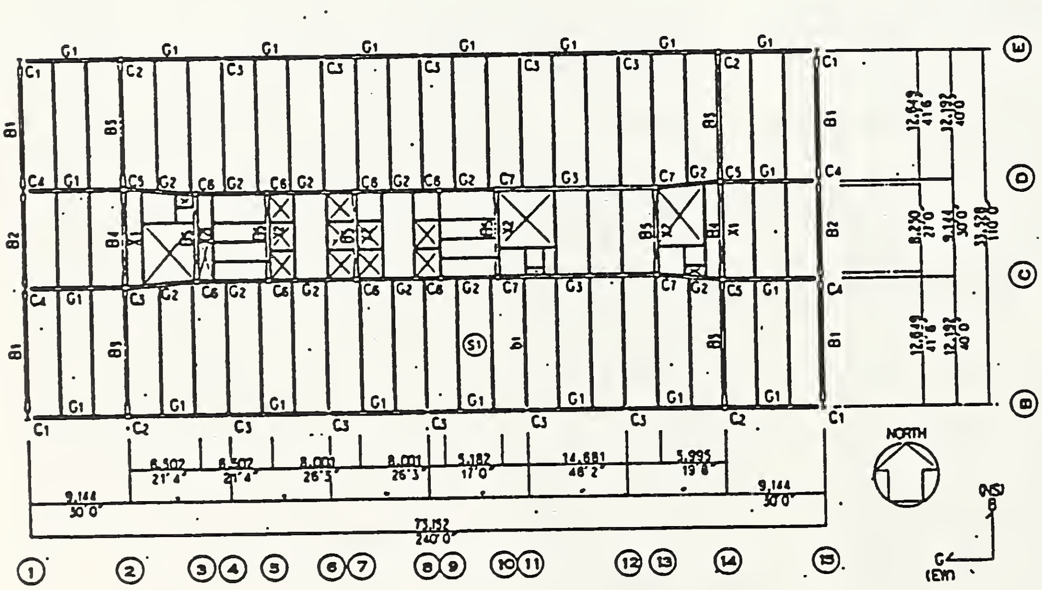

Fig. 2.5 Typical Floor Framing Plan

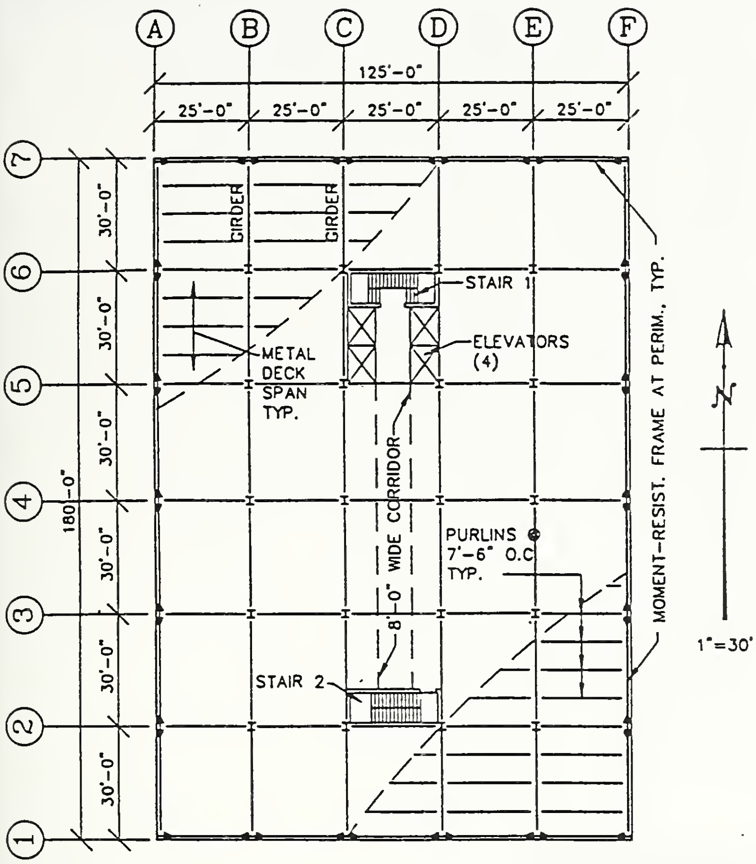

Fig. 2.1 Typical Floor Framing Plan

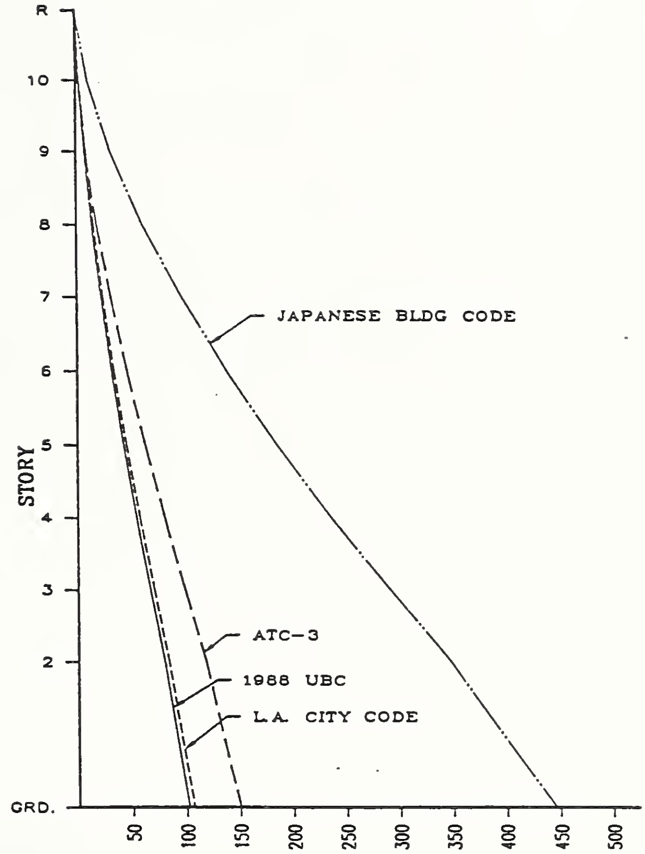

Fig. 2.3 Overturning Moments

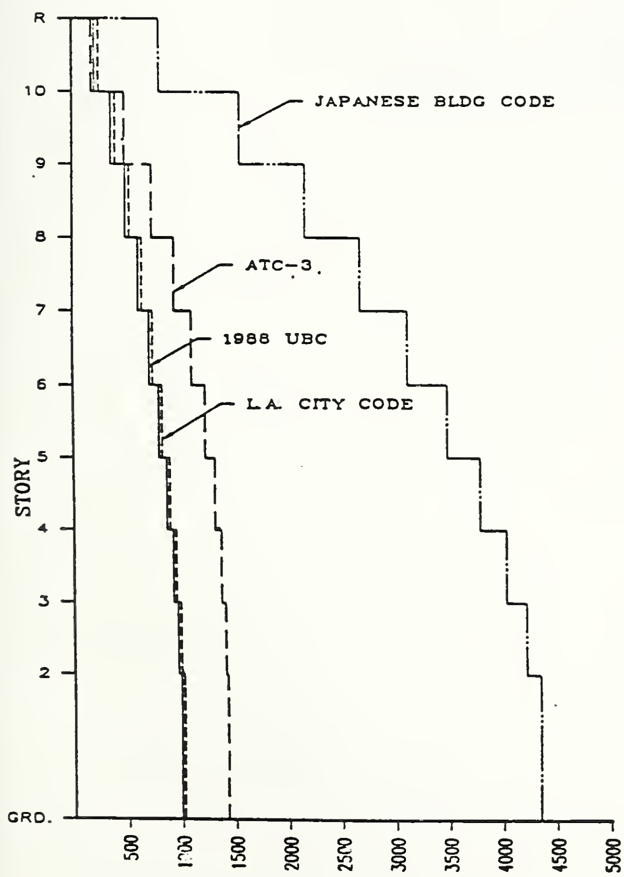

Fig. 2.2 Story Shear

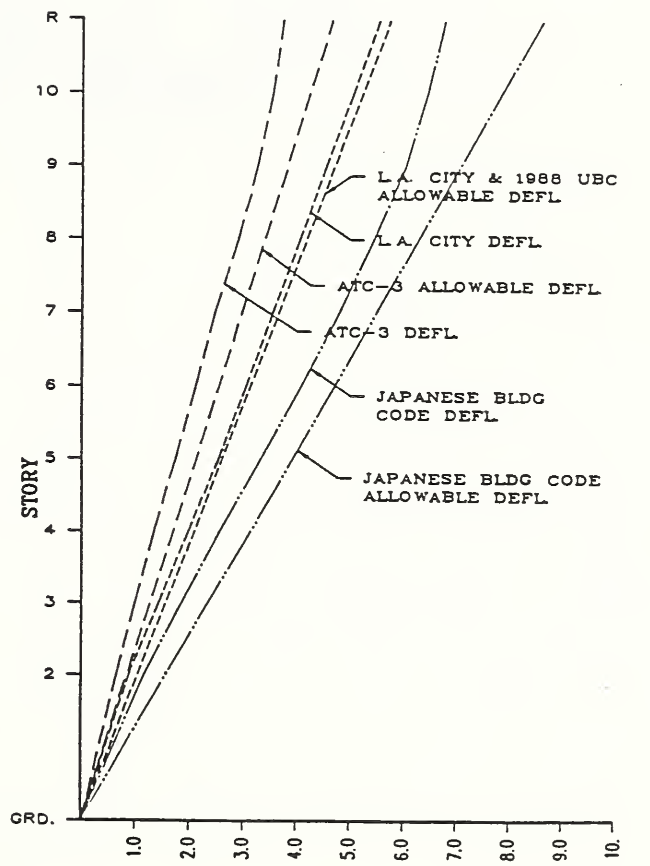

Fig. 2.4 Deflection

SECTION TWO

PERFORMANCE OF EXISTING STRUCTURES -- REPAIR AND STRENGTHENING NEEDS

Aseismic Performance and Strengthening of Existing Reinforced Concrete Building Structures

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Aseismic Performance and Strengthening of Existing Reinforced Concrete Building Structures

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INTRODUCTION

"Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings" was proposed by the special Committee chaired by Professor H. Umemura under the sponsorship of the Ministry of Construction, Japanese Government, in 1977.

This standard may be used to evaluate the seismic capacity of existing low-rise reinforced concrete buildings and consists of three different level screening procedures; the first level screening procedure, the second level screening procedure, and the third level screening procedure.

Now, revised version of the standard is being prepared by the Committee for revision. Revised portions are mainly as follows;

1)Add new concepts and equations to calculate the strength of structural members and to evaluate the F-index, corresponding to the deformation capacity of members.

2)Add a simple procedure of the third level screening to evaluate seismic capacity of school buildings up to three stories.

In this paper, item 1) is described and discussed.

OUTLINE OF THE STANDARD

Seismic safety of building may be estimated by the following two numerical indices and the larger value indices indicates the higher seismic safety of buildings.

Seismic Index of Structure : Is
Seismic Index of Non-Structural Elements : In

Is-index shall be calculated by Eq.(1), independently to the longitudinal and ridge directions at each floor of the building under consideration. However, the following G-index, T-index, and SD-index in the first level screening procedure are independent of the floor location of the building.

$$Is = Eo \times G \times SD \times T \tag{1}$$

where Eo : Seismic Sub-Index of Basic Structural Performance
G : Seismic Sub-Index of Ground Motion and Soil Condition
SD : Seismic Sub-Index of Structural Design
T : Seismic Sub-Index of Time-Depended Deterioration

In order to calculate the Is-Index, any of the first, the second, and the third level screening procedure may be used.

i)The First Level Screening Procedure

Eo-Index is calculated from the horizontal strength of a building, based on the sum of the horizontal cross sectional area of columns and walls and on the their average unit strength.

ii)The Second Level Screening Procedure

Eo-Index is calculated from the ultimate horizontal strength, failure mode and ductility of columns and walls with assumption of infinitely strong floor system.

iii)The Third Level Screening Procedure

Eo-Index is calculated from the ultimate horizontal strength, failure mode and ductility of columns and walls, based on failure mechanism of frames with consideration of strength of beams and overturning of walls.

The sub-index Eo is the most basic index to evaluate the structural seismic performance. In estimation Eo-Index, the ultimate strength and ductility to lateral force, type of failure, total number of stories, and the story are under consideration.

Basically, Eo-index is proportion to the value of C-index multiplied by F-index, as equation (2).

$$Eo \quad C \times F \tag{2}$$

where C : Strength Index
F : Ductility Index

In the third level screening, C-index and F-index are estimated by the following procedure.

Strength Index C

i)General

Strength index C for third level screening may be estimated by the following procedure:

a)Calculate flexural strength Mu and shear strength Qsu of columns, walls and beams by the method shown in the following item ii).

b)Calculate ultimate lateral strength of story by a simplified limit analysis: nodal distribution method for frames and limit analysis based on an assumed failure mechanism for wall-frame structure, based on the member strengths estimated above, and determine failure mode and shear force of each vertical structural member.

c)Classify all vertical structural members into three or less groups based on their failure modes and F-index and estimate C-index of each group.

ii)Member strength

a)Flexural strength and shear strength of columns and walls may be calculated by Eqs. (8), (9), (10), (12), (13), (14), and(15) same as in the second level screening.

b)Flexural and shear strength of beam section are computed by Eqs.(8), (9), (10), (12), (13), (14), and (15) with substituting N = 0 or $\sigma o = 0$ into these equations. Here, the effect of the reinforcement arranged in slabs may be considered for the computation of the flexural strength of beams. For the computation of the flexural strength of sections of beams, the following simple equation (3) is also applicable.

$$bMu = 0.9\ at\ \sigma y\ d \tag{3}$$

where at : Area of tensile reinforcement (cm2)

σy : Yield strength of tensile reinforcement (kg/cm2)

d : Effective depth of a beam cross-section (cm2)

iii)Determination of the lateral load carrying capacity of vertical members and the failure mode of the members.

a) Columns

Columns are classified into the three following types; flexural failure column, beam yield type column and beam shear type column. Based on the ultimate strength of columns and beams, the maximum end moments of all members at all nodal points are determined. comparing the sum of the end moments of beams and that of columns at each nodal point, the ultimate moment distribution of columns is determined by nodal limit analysis. The ultimate strength of the column, cQu may be computed by the following equation, considering the above moment distribution.

cQu = (Sum of ultimate moment capacity at top and bottom end of column)/(clear height of column)

b)Walls

Walls are idealized by cutting off from the other framing members at the mid-span of connecting beams. The lateral load carrying capacity of the idealized walls may be taken as the least of the three following lateral loads determined under inverse triangular distribution of lateral loads; flexural yield strength, shear strength or overturning capacity. The failure modes of walls for the mechanism are based on the determination of their lateral load carrying capacity.

iv)F-index and C-index

Vertical structural members should be classified into three or less groups of ductility F-index. And estimation of C-index of each group can be done by the similar manner to the second level procedure.

Sub-index of Ductility, F

i)General

The sub-indices of ductility of members except flexural columns and flexural walls can be fixed to the constant values.

ii)F-indices of Flexural Columns and Flexural walls

a)F-index of flexural columns may be obtained by equation (4), based on the ductility factor of the columns determined by Eq.(6) in the following item iii).

$$F = 2\mu-1 \ / \ (0.75(1+0.05\mu) \qquad (4)$$

b)F-index of flexural walls is obtained by Eq. (5) using their shear strength wQsu and flexural strength, wQmu.

$$\begin{array}{lll} \text{when} & wQsu/wQmu >= 1.3 & : F = 1.0 \\ \text{when} & 1.3 < wQsu/wQmu < 1.4 & : F = -12.0 + 10\,(wQsu/wQmu) \qquad (5) \\ \text{when} & wQsu/wQmu >= 1.4 & : F = 2.0 \end{array}$$

iii)Ductility Factor of flexural columns

Ductility Factor of flexural columns may be computed by Eq.(6). However, F-index should be 1.0 proved that any one of conditions described in Eq.(7) is corresponded.

$$\mu = \mu o - k1 - k2 \qquad \text{here } 1 =< \mu <= 5 \tag{6}$$

where μo : = 10(cQsu/cQu - 1)

k1 : = 2.0 (k1 may be zero provided that shear reinforcement spacing is less than eight times the diameter of longitudinal reinforcement.)

k2 : = 30(cτu/Fc - 0.1) >= 0

cQsu : Shear strength of column

cQu : Shear force working on the column at yielding.

cτu : = cQu / b j

b : Width of column

j : Distance between compressive resultant forces and tensile resultant forces.

Fc : Compressive strength of concrete

F-index should be 1.0 in the conditions as bellow (7);

Ns/bDFc > 0.4,	(Ns corresponds to axial forces of columns at their failure mechanism)	
cτu/Fc > 0.2		(7)
Pt > 1%,	where Pt is tensile reinforcement ratio.	
ho/D =< 2.0,	where ho is clear height of columns.	

ULTIMATE STRENGTH OF MEMBERS

Strength index C is estimated on the basis of the ultimate strength of members, such as beams, columns, and walls. Hereafter, equations for calculation of the ultimate strength of members are listed. Accuracy of the equations is discussed.

1) Ultimate strength of beams

For the flexural strength of rectangular beams, the equation (3) may be used. The effects of a monolithic slab and those of intermediate reinforcing bars in case of beams with more than two layers of longitudinal reinforcement, may be taken into account.

For the shear strength of rectangular beams, the empirical equation (8) derived by Prof. Arakawa is mainly used in Japan.

$$bQsu = (0.053pte\hat{\ }0.23(Fc+180)/(M/(Qd)+0.12) + 2.7\ \ pwe\sigma wy\ )\ be\ j \tag{8}$$
$$\text{here } 1=< M/(Qd) =< 3$$

where pte : Tensile reinforcement ratio (%)

pwe : Shear reinforcement ratio. When pwe exceeds 0.012, pwe should be taken as equal to 0.012.

σwy : Yield strength of shear reinforcement (kg/cm2)

d : Effective depth of beam, = (D-5) (cm)

M/Q : May be taken as equal to ho/2. ho is clear length of beam. (cm)

be : = ΣAg/D , be =< 1.2(width of beam) (cm)

The equation (8) gives the lower limit value of shear strength of beam. And equation with the coefficient 0.068 instead of 0.053 in the first part of the equation (8) gives the average value of shear strength of beams.

2) Ultimate strength of columns.

Equation (9) may be used for evaluation of the flexural strength of columns.

For $N_{max} \geq N > 0.4bDF_c$

$$cM_u = (0.8 a_t \sigma_y D + 0.12 b D^2 F_c)((N_{max}-N)/(N_{max}-0.4bDF_c)) \qquad (9)$$

For $0.4bDF_c \geq N > 0$

$$cM_u = 0.8 a_t \sigma_y D + 0.5ND(1-N/(bDF_c))$$

For $0 \geq N \geq N_{min}$

$$cM_u = 0.8 a_t \sigma_y D + 0.4ND$$

where N_{max} : Ultimate strength of columns under axial compression (kg) $= bDF_c + a_g\sigma_y$

N_{min} : Ultimate strength of columns under axial tension (kg) $= -a_g\sigma_y$

N : Axial load of column (kg)

a_t : Area of tension reinforcement of column (cm)

a_g : Gross area of longitudinal reinforcement of column (cm)

b : Width of column (cm)

D : Depth of column (cm)

σ_y : Yield strength of reinforcement (kg/cm2)

F_c : Compressive strength of concrete (kg/cm2)

Equation (10) gives the lower limit value of the shear strength columns. Equation (8) was modified to Eq. (10) in order to include the effect of axial force.

$$cQ_{su} = (0.053 p_t^{0.23}(F_c+180)/(M/(Qd)+0.12) + 2.7 \sqrt{p_w \sigma_{wy}} + 0.1\sigma_o)\, b\, j \qquad (10)$$

here $1 \leq M/(Qd) \leq 3$

where p_t : Tensile reinforcement ratio (%)

p_w : Shear reinforcement ratio. When pwe exceeds 0.012, pwe should be taken as equal to 0.012.

σ_{wy} : Yield strength of shear reinforcement (kg/cm2)

d : Effective depth of column, = (D-5) (cm)

M/Q : May be taken as equal to ho/2. ho is clear height of column. (cm)

b : width of column (cm)

σ_o : Axial stress of column (kg/cm2) When σ_o exceed 80kg/cm2, σ_o should be taken as equal to 80kg/cm2.

Equation (10) with the coefficient 0.068 instead of 0.053 gives the average value of the shear strength of columns.

Equations (8) and (10) are proposed on the basis of the shear loading test data. But, theoretical equation (11) is proposed on the basis of resisting mechanism that is combination of truss model and arch model. In using this equation, it is necessary to check the column not to be failed in bond splitting failure mode, because the equation (11) is proposed with assumptions that all reinforcement for shear force should be yielded and bond between longitudinal reinforcement and concrete should be sound until columns are failed in shear.

$$V_u = b\, j_t\, p_w\, \sigma_{wy} \cot\phi + \tan\theta(1-\beta) b\, d\, \nu\, c\sigma_b/2 \qquad (11)$$

where jt : Distance between longitudinal reinforcements (cm)
L : Height of column (cm)
ν : Effective modules of concrete strength
ϕ : Angle of concrete strut of truss model
$\tan\theta$: = (L/D) ^2+ 1 - L/D
β : $(1+\cot^2\phi)$ pw $\sigma wy/(\nu\ c\sigma b)$
$c\sigma b$: Compressive strength of concrete (kg/cm2)

3) Ultimate strength of walls

Equations (12) and (13) may be used for evaluation of the flexural strength of walls.

$$wMu = 0.9at\sigma yD + 0.4aw\sigma wyD + 0.5ND(1 - N/(BcDFc)) \tag{12}$$

$$wMu = at\sigma ylw + 0.5aw\sigma wylw + 0.5Nlw \tag{13}$$

where at : Area of longitudinal reinforcement of tension side column (cm2)
σy : Yield strength of longitudinal reinforcement of column (kg/cm2)
aw : Area of vertical reinforcement of wall (cm2)
σwy : Yield strength of vertical reinforcement (kg/cm2)
Bc : Width of compression side column (cm)
lw : Length of wall; measured center to center of columns (cm)
D : Length of wall; measured out to out of columns (cm)
Fc : Compressive strength of concrete (kg/cm2)
N : Axial force (kg)

There is no description about the applicable range of the axial force N. But, considering that the axial force is not large in actual shear walls, calculated values show relatively good agreement with tested values within 20% difference.

Equations (14) and (15) can be applied to evaluate the shear strength of walls with boundary columns. These equations were proposed on the basis of experimental study. Equation (14) gives the average values of the tested results, and equation (15) gives the lower limit values.

$$wQsu = (0.068pte^{0.23}(Fc+180)/(M/(QD)+0.12) + 2.7 \sqrt{pwe\sigma wy} + 0.1\sigma o)\ be\ j \tag{14}$$

$$wQsu = (0.053pte^{0.23}(Fc+180)/(M/(Qd)+0.12) + 2.7 \sqrt{pwe\sigma wy} + 0.1\sigma o)\ be\ j \tag{15}$$

where be : Equivalent depth of wall, = A/l (cm)
A : Sum of sectional area of wall and columns (cm2)
l : End to end distance between columns (cm)
pte : 100at/(be l) (%)
at : Total sectional area of axial reinforcements of tension side column (cm2)
pwe : Equivalent lateral reinforcement ratio of wall, = aw/(be S)
aw : Sectional area of lateral reinforcement of wall (cm)
S : Spacing of lateral reinforcement of wall (cm)
σwy : Yield strength of lateral reinforcement of wall (kg/cm2)
σo : $\Sigma N/(be\ l)$ (kg/cm2)
N : Axial forces of columns and wall (kg)
j : May be taken as lw or 0.8 l

Equation (16) was proposed for evaluation of shear strength of walls on the basis of theoretical modeling.

$$wVu = t\ lwb\ pwh\ \sigma wh\ \cot\phi + \tan\theta(1-\beta)\ t\ lwa\ \nu\ c\sigma b/2 \qquad (16)$$

where pwh : Ratio of horizontal reinforcement of wall
σwh : Yield strength of horizontal reinforcement of wall (kg/cm2)
lwa : Equivalent length of wall for truss model (cm)
lwb : Equivalent length of wall for arch model (cm)
hw : Height of wall (cm)
ν : Effective modules of concrete strength
ϕ : Angle of concrete strut of truss model
$\tan\theta$: = (hw/lwa) ^2+ 1 - hw/lwa
β : $(1+\cot^2\phi)\ pwh\ \sigma wh/(\nu\ c\sigma b)$
$c\sigma b$: Compressive strength of concrete (kg/cm2)

4) Accuracy of equations.

Table 1 shows the data related to the accuracy of equations (3), (8) - (16). It is generally considered that equations for the flexural strength have good accuracy. But equations for the shear strength were not accurate compared with equations for the flexural strength. Mean values of equations (10), (11), and (15) in Table 1 are over 1.3 and the standard deviations are almost 0.3. These values mean that the equations (10), (11) , and (15) evaluate the lower limit value of shear strength of members. Therefore it is considered that use of these equations to evaluate the shear strength of members is safety side of evaluation.

Fig.1 shows the relationship between tested shear strength (Qexp) and calculated shear strength (Qs) by equations (8) with 0.068, (10) with 0.068, and (11), using the same data of beam and column tests. Y-axis and X-axis are Qexp and Qs nomalized by the calculated flexural strength (Qf).

Among the equations for the strength of reinforced concrete members, equations for the shear strength have larger deviation. It is necessary to discuss on the accuracy of the equation for the shear strength.

DEFORMATION CAPACITY

In order to give good deformation capacity to reinforced concrete members, it is necessary to yield the members in flexural and prevent brittle failure before the members reach large deformation with inelastic hinge. So, not only to prevent the shear failure and the bond splitting failure of members, but also to confine the compressive part of concrete, to prevent shear failure of beam - column joint panels.

In Japan, engineers have done effort on research related to the prevention of shear failure, because the members failed in shear were observed in the earthquake damage. And also, many trials to make clear the relationships between the ratio of shear strength to flexural strength and deformation capacity have been performed. Furthermore, bond splitting failure was observed in the tests of RC members which were reinforced enough to prevent the shear failure, and research on the method to prevent such failure was performed.

On the other hand, research on the evaluation method of deformation capacity has been started on the basis of theoretical study of shear failure after flexural yield. In this method, it is necessary to prevent the bond splitting failure before the shear failure because the formation of truss model is assumed.

There are some proposed evaluation methods of deformation capacity. But the evaluation method of deformation capacity is now on progress, so we have no accurate evaluation method. All evaluation methods has their variation, but they gives conservative values for deformation capacity of members. Therefore, the evaluation methods of deformation capacity are able to apply the design of the ductile members.

Experimental data of short span column with alternative loading are plotted in Fig. 2 with X-axis of the inverted value of shear strength divided by flexural strength (cQmu/cQsu) and Y-axis of inelastic ratio (μ). μ increases with decrease of Qmu/Qsu. This relationship means that μ increases with increase of Qsu/Qmu. In "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings", the relationship was simplified to equation (6).

CONCLUSINS

Conclusions are listed as follows.

1)Revised edition of the "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings" is being prepared now.
2)Revised edition is added new concepts and equations for evaluation of strength and deformation capacity of members
3)New concept for evaluation methods for strength and deformation capacity is proposed on the basis of theoretical arch and truss model.

REFERENCES

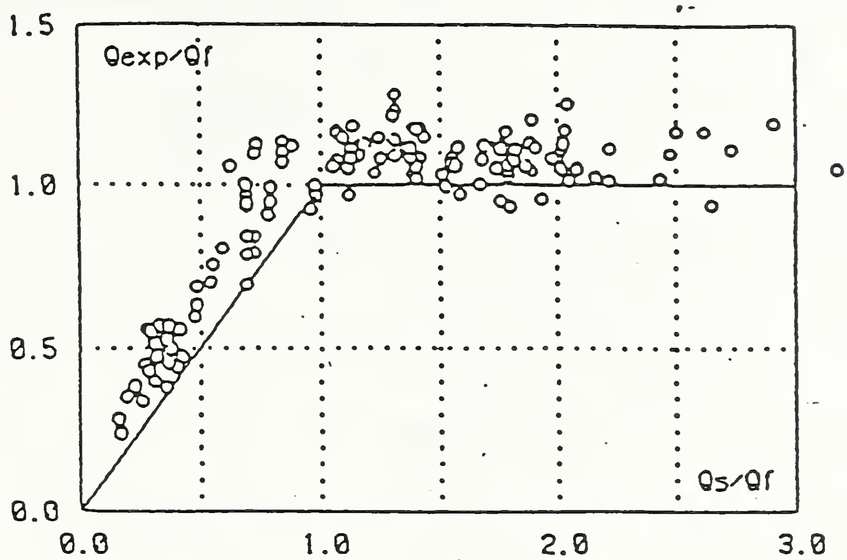
1)"Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings", Japan Building Disaster Prevention Association, April 1977
2)"Design Guideline for Earthquake Resistant Reinforced Concrete Building Based on Ultimate Strength Concept", Architectural Institute of Japan, May 1988

Table 1 Comprison of Experimental value and Calculated Value

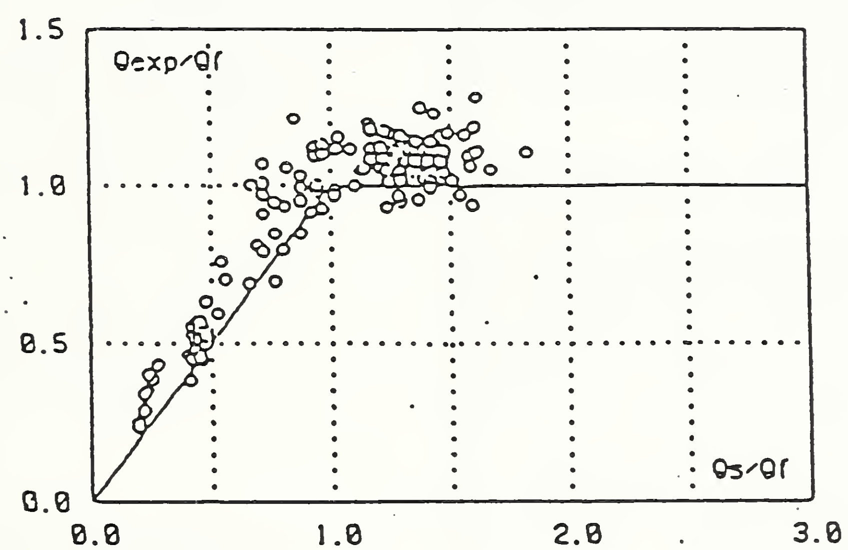
Calculation		Number of Datas	Minimum	Maximum	Mean Value	Standard Deviation	Coefficient of Variation
Bending Strength of Beam	bMu (3)	95	0.745	1.484	1.063	0.112	0.106
Shearing Strength of Beam	bQsu' (8)+0.068	70	—	—	0.955	0.085	0.089
Shearing Strength of Beam	bQsu' (8)+0.068	138	—	—	0.95	0.079	0.085
Shearing Strength of Beam	bQsu (8)	70	0.942	1.368	1.142	0.091	0.080
Bending Strength of Column	cMu (9)	48	0.85	1.30	1.08	0.09	0.083
Shearing Strength of Column	cQsu' (10)+0.068	218	0.57	1.90	1.09	0.27	0.248
Shearing Strength of Column	cQsu (10)	171	0.709	2.86	1.346	0.342	0.256
Shearing Strength of Column	Vsu(11)	77	—	—	1.333	0.247	0.185
Bending Strength of Bearing Wall	wMu (12)	13	0.942	1.508	1.083	0.136	0.125
Bending Strength of Bearing Wall	wMu (13)	13	0.929	1.462	1.055	0.125	0.119
Shearing Strength of Bearing Wall	wQsu (14)	83	0.365	1.969	1.046	0.262	0.250
Shearing Strength of Bearing Wall	wQsu (15)	83	0.495	2.670	1.313	0.351	0.267
Shearing Strength of Bearing Wall	wVu (16)	48	—	—	1.13	0.149	0.132

note: (-) in Table Indicates of Vagueness.

A-Method of Design Guideline Formula(AIJ 1988). (11)



Modified Arakawa mean Formula. (10) + 0.068



$Q_{exp}/Q_f - Q_s/Q_f$ Relation.

Fig.1 Qexp/Qf - Qs/Qf Relation

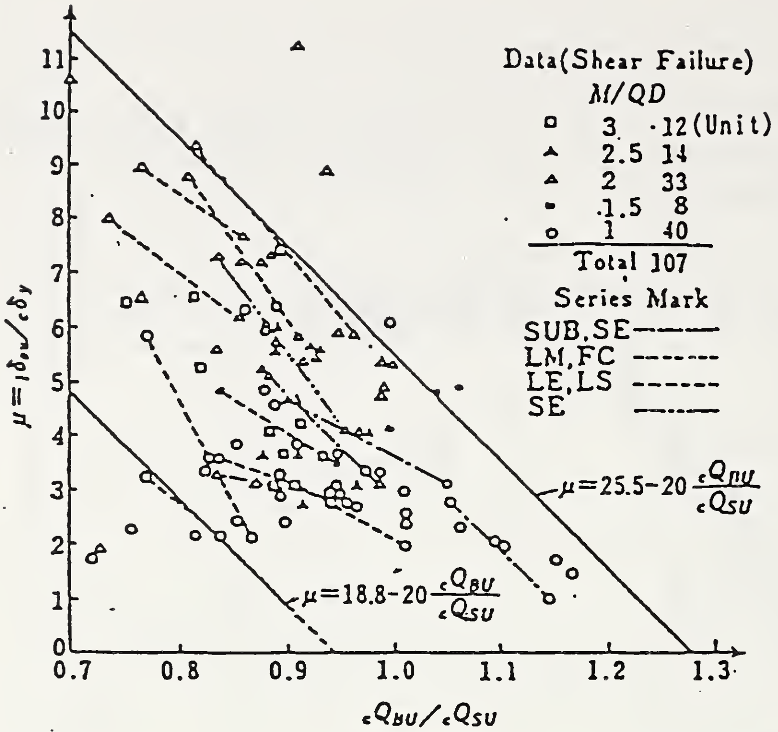


Fig. 2 Ductility Factor - Strength Ratio Relation

PERFORMANCE OF BUILDINGS FOR "HIGH-TECH" INDUSTRIES IN SILICON VALLEY IN THE LOMA PRIETA EARTHQUAKE

by

Maryann T. Phipps[1]

ABSTRACT

Silicon Valley is vital to the economic health of the San Francisco Bay Area. It is home to one of the nation's centers of high-technology industrial activity. Consequently, earthquake damage to the area has the potential for severe regional and national economic impacts. Located 40 km north of the epicenter of the Loma Prieta Earthquake, Silicon Valley experienced ground accelerations up to 0.38g during the event. Limited structural and nonstructural damage resulted. The types of damage experienced in high-tech facilities which led to a temporary loss of productivity are discussed.

KEYWORDS: Building Performance, Earthquake Damage, High-Tech Industries, Loma Prieta Earthquake, Silicon Valley.

1. INTRODUCTION

Silicon Valley is the economic powerhouse of the San Francisco Bay Area. The region is a recognized center of manufacturing, research and development for the electronics industry, defense, and biomedical technology. Twelve Fortune 500 companies are headquartered here. Silicon Valley, located in Santa Clara County, ranks second in California and fifth in the nation as measured by values of shipments. In total, the county's industries shipped over $24 billion in 1988 (San Jose Metropolitan Chamber, 1989). The area's manufacturing firms produced more than 44 percent of the nation's output of computer terminals, more than 29 percent of total U.S. production of ordnance, including tracked vehicles and guided missiles, nearly 23 percent of the nation's computers, 18 percent of computer storage equipment, and almost 16 percent of U.S. production of semiconductors (San Jose Chamber, 1988/89).

On October 17, 1989, a magnitude 7.1 earthquake occurred due to approximately 40 km rupture along the San Andreas Fault. The epicenter of the 20 second earthquake was located near Loma Prieta in the Santa Cruz Mountains, about 40 km south of Silicon Valley. In the wake of the disaster, industrial operations ceased temporarily. Within six days after the earthquake, however, industrial activities were nearly restored to pre-earthquake levels.

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The potential regional and economic impact which could result from extended loss of productivity in Silicon Valley is great. As such, it is valuable to study the effects of the Loma Prieta Earthquake on industrial enterprises in order to better prepare for future events. This paper reflects on the performance of buildings which house the "high-tech" industrial activities of Silicon Valley. The types of structural damage observed are described, and examples of each type of damage are presented. For each example, repair and strengthening measures are illustrated. The types of nonstructural damage observed are also described.

2. SEISMOLOGIC SETTING

Silicon Valley lies between the San Andreas Fault Zone to the west and the Hayward and Calaveras Fault Zones to the east. As such, it is an area highly vulnerable to seismic activity. During the Loma Prieta Earthquake, peak ground accelerations up to 0.38g were recorded (Figure 1).

Most industrial activities in Silicon Valley are located in areas where bedrock is located over 200 feet below the ground surface. While potential for liquefaction exists in a large portion of the area, none was reported as a result of the Loma Prieta Earthquake.

3. PERFORMANCE OF BUILDINGS

Thousands of buildings dating from the late 1940s to the present house the industries of Silicon Valley. Most buildings, however, were constructed since 1960. While building construction varies, most construction is limited to one or two stories. A large percentage of the buildings is of tilt-up construction. Other common building types include steel moment resisting frames, steel braced frames and concrete shear wall systems.

In general, the performance of industrial buildings in Silicon Valley was good. There was some damage to buildings that are of known hazardous construction. Some buildings of more modern construction were damaged as a result of construction deficiencies. Others designed to modern codes were damaged as a result of poor detailing. Examples of each type of building damage follow.

3.1 Buildings of Known Hazardous Construction Type

The inventory of industrial buildings in Silicon Valley includes old tilt-up buildings, nonductile concrete moment resisting frame structures, and other structural systems lacking a deliberate lateral force resisting system. Many of these buildings suffered little or no observable structural damage in the earthquake. This apparent good performance cannot be attributed to the adequacy of the structural systems, but rather is a result of the relatively low levels of ground accelerations experienced at most sites in Silicon Valley and the relatively short duration of the shaking.

Some buildings of known hazardous construction did suffer some structural damage, however. One tilt-up building in San Jose partially collapsed as a result of a lack of positive connection between the roof and the walls. More common were buildings structurally weakened, but still standing. One example involves a San Jose warehouse of 194,000 square feet constructed in 1975. The structural system consists of tilt-up walls at the exterior of the building and a plywood roof diaphragm. Connection between the tilt-up walls and the roof diaphragm consists of 2-3/4 inch x 12 inch anchor bolts at the top of each panel connecting to glued laminated roof beams as shown in Figure 2. No special confining reinforcement is provided around the anchor bolts.

After the Loma Prieta Earthquake, the connection between the roof diaphragm and walls showed some signs of distress. The concrete around the anchor bolts connecting the glulam beams to the wall panels was prominently cracked at nearly each connection. A more detailed investigation of the condition indicated that the reinforcement in the pilasters did not engage the anchor bolts at all. Given the high threat of aftershock, the condition was judged to be unsafe and the building was evacuated.

Because this warehouse ships on the order of $1 million worth of computer equipment daily, prompt repair and strengthening measures were needed to minimize economic losses to the company. A positive tie between each glulam beam and tilt-up panel was designed. The strengthening scheme was designed the day after the earthquake based on discussions with the contractor who would be performing the work. Such cooperative effort permitted the development of a repair and retrofit scheme which could accommodate availability of materials and fabrication operations. The selected scheme consisted of the installation of a positive wall anchor at the top of each tilt-up wall pilaster as shown in Figure 3. The new structural ties to the tilt-up walls were installed within 5 days after the earthquake. Full warehouse operations resumed on Monday, October 23.

3.2 Buildings with Construction Deficiencies

Several instances of building damage were reported to have resulted from construction deficiencies. One such example occurred in a 27,000 square foot, two-story building constructed in 1973. The lateral force resisting system consists of precast walls resisting forces in the transverse direction and steel braced frames in the longitudinal direction (Figure 4). The roof diaphragm consists of metal decking. The second floor diaphragm consists of metal decking and concrete fill reinforced with light welded wire mesh. Steel columns, steel beams, and open web steel joists provided the principle gravity load carrying system.

This building was located in an area of Palo Alto which appears to have experienced somewhat greater shaking than that generally felt in Silicon Valley. Peak ground accelerations were possibly between .2g and .3g. As a result of the earthquake, the building suffered considerable structural and nonstructural damage. The most noticeable structural damage observed was a crack in the second floor diaphragm, approximately 1/2 inch wide, which ran transversely across the building. The location of the crack coincided with the stairwell opening on one side of the building and plan offset on the other side.

The original design included reinforcing bars at the location of the stair and plan offset. The bars were designed to resist the chord force in the diaphragm due to earthquake forces in the transverse direction.

Upon close investigation after the earthquake, the chord bars which had been included in the original construction drawings were found to have been omitted during construction.

Repair to the second floor diaphragm included the removal of concrete at the crack. New welded wire mesh and concrete were installed to restore the shear strength of the diaphragm at this location. In order to provide a structural chord member at the locations of discontinuities in the second floor slab, an angle was installed at the underside of the diaphragm as shown in Figure 5.

The detailed post-earthquake structural investigations, structural repairs and reconstruction of all tenant improvements caused this building to be out-of-service for over 6 months following the earthquake.

3.3 Buildings Designed to Modern Codes

In general, buildings located in Silicon Valley which were designed and detailed in accordance with recent seismic codes in California were not structurally damaged in the earthquake. While this is not a full test of our modern codes and design practices due to the limited magnitude and duration of the earthquake, it does provide some measure of confidence in the direction that our codes have taken.

Minor exceptions to the good performance of modern engineered buildings could be found, however. One interesting example involves a research and development building located in Palo Alto. Based on two nearby strong motion records, the peak ground acceleration at the site is estimated at approximately 0.3g. This 220,000 square foot building suffered significant damage in the Loma Prieta Earthquake and was vacated for approximately 5 months during repairs.

The building is a two-story plus basement reinforced concrete and steel frame building which is divided into two seismically separate structures consisting of a central core and a perimeter section. A key plan is shown in Figure 6. The building is roughly 277 feet by 268 feet in plan and about 44 feet high. The perimeter area is isolated from the core by a 2 inch separation. Both sections have a complete attic space in the upper level and interstitial space in the lower level.

The perimeter section consists of two stories of structural steel frame over a reinforced concrete basement. Both stories are framed with steel trusses which provide support for the attic and interstitial floors at the lower chords. Seismic forces in both principal directions are resisted by steel braced frames in the upper two stories and reinforced concrete walls in the basement. Diaphragms consist of a metal deck with concrete fill at the second floor, a metal deck without fill at the roof and a combination of plywood and concrete on metal deck at the attic and interstitial levels. The ground floor consists of a reinforced concrete waffle slab, supported on concrete columns and walls. Foundations are primarily reinforced concrete spread footings.

The core section consists of reinforced concrete waffle slab construction at the first story and steel trusses at the upper story. The interstitial floor consists of steel framing and plywood while the attic level consists of a plywood floor supported on the lower truss chords. Lateral forces are resisted by concrete shear walls at the lower level and basement and steel braced frames at the upper level. Foundations consist primarily of spread footings.

Earthquake damage to the structural system included significant structural damage to fourteen steel braced frames in the perimeter section of the building (Figure 7). The documented failures to the steel braced frames can be considered to fall within three general groups: twisting of steel beams at brace-to-beam intersections, buckling of steel braces, and weld failures at brace connections.

Figure 8 illustrates one type of braced frame with the following general characteristics: it contains diagonal braces which intersect a horizontal beam within the span of the beam and are not accompanied by another diagonal brace framing into the same joint. In the original design, the two braces framing into the interstitial level were assumed to be laterally supported by the framing at the interstitial level. It is apparent, however, due to the twisting failure of the interstitial level beam, that the bottom flange of the beam was unable to provide adequate lateral bracing for the upper end of the first level brace. Figure 9 shows a sketch of the failed condition. As the interstitial beam twisted out-of-plane, the compressive capacity of the brace was lost and, consequently, lateral forces could no longer be carried by this type of frame. This condition led to an increase in the forces in other braced frames which did not have a lateral instability problem. This increase in the load to these braced frames is believed to have caused the buckled braces and failed welds.

Post-earthquake repair of the lateral instability condition involved the installation of beams in the interstitial level which connected into the joint of the brace intersection. The building was out of service for approximately 4 months during repairs.

4. <u>NONSTRUCTURAL DAMAGE TO BUILDINGS</u>

The real distinction between "typical" buildings and those housing high-tech industrial activities is generally not the structural system, but rather, its contents. It is the nonstructural elements of the building which provide it with its functional characteristics. Nearly every business in Silicon Valley suffered some damage to nonstructural elements during the Loma Prieta Earthquake. A sample of the types of nonstructural elements generally effected and the range of damage observed is presented here.

Most modern buildings in Silicon Valley include large areas with suspended ceilings. Some damage to suspended ceiling systems was experienced in most buildings. Extent of damage to these systems ranged considerably, however. In some cases, only a few lightweight ceiling tiles shook loose. In other systems where the ceiling framing was outdated and the ceiling was unbraced, both ceiling tiles and framing came down over large areas. This damage required substantial time to clean up and reinstall, thereby, preventing full, uninterrupted occupancy of the space until repairs were complete. Some ceiling damage was further aggravated because of the presence of asbestos which was released from above the ceiling. Clean up in these cases prevented beneficial use of some ares for durations ranging from days to months.

While, in general, suspended ceiling damage did not pose a serious threat to life safety, overhead items supported by suspended ceilings did pose such a threat. In many instances, overhead fluorescent lighting fixtures which were supported solely by the ceiling framing were shaken loose during the earthquake. While there are no reported deaths or injuries as a result of falling overhead fixtures, the potential for such bodily harm was quite high.

A wide range of office

furnishings were damaged during the earthquake. Tall lateral file cabinets tilted, drawers came open, some blocking means of egress. Tall bookcases toppled. Demountable partitions overturned where they were not adequately restrained by cross walls. Such damage led to some reported injuries and limited "downtime" during cleanup activities.

Equipment which was well anchored to a structural system generally performed well. Many examples of poorly anchored or unanchored items could be found, however. These items became dislodged, moved and sometimes overturned. Mechanical units on spring isolators without lateral restraints were commonly damaged. No serious damage to computer equipment was reported.

Some damage to piping systems was reported. Damage to sprinklers, for example, often resulted from swaying pipes which contacted nearby obstructions. Others were damaged as a result of a lack of flexible joints at seismically separate portions of a building.

5. CONCLUSIONS

Silicon Valley survived the Loma Prieta Earthquake with only minor physical and economic losses. An estimated 95% of industrial activities were resumed within six days following the earthquake. Buildings for the most part performed well. Noted exceptions include some buildings of known hazardous construction type, buildings with construction deficiencies, and building of modern construction with poor detailing. Nonstructural damage was the greater cause for loss of productivity immediately after the earthquake. Damage to ceilings, office furnishings, equipment and piping systems was observed.

6. REFERENCES

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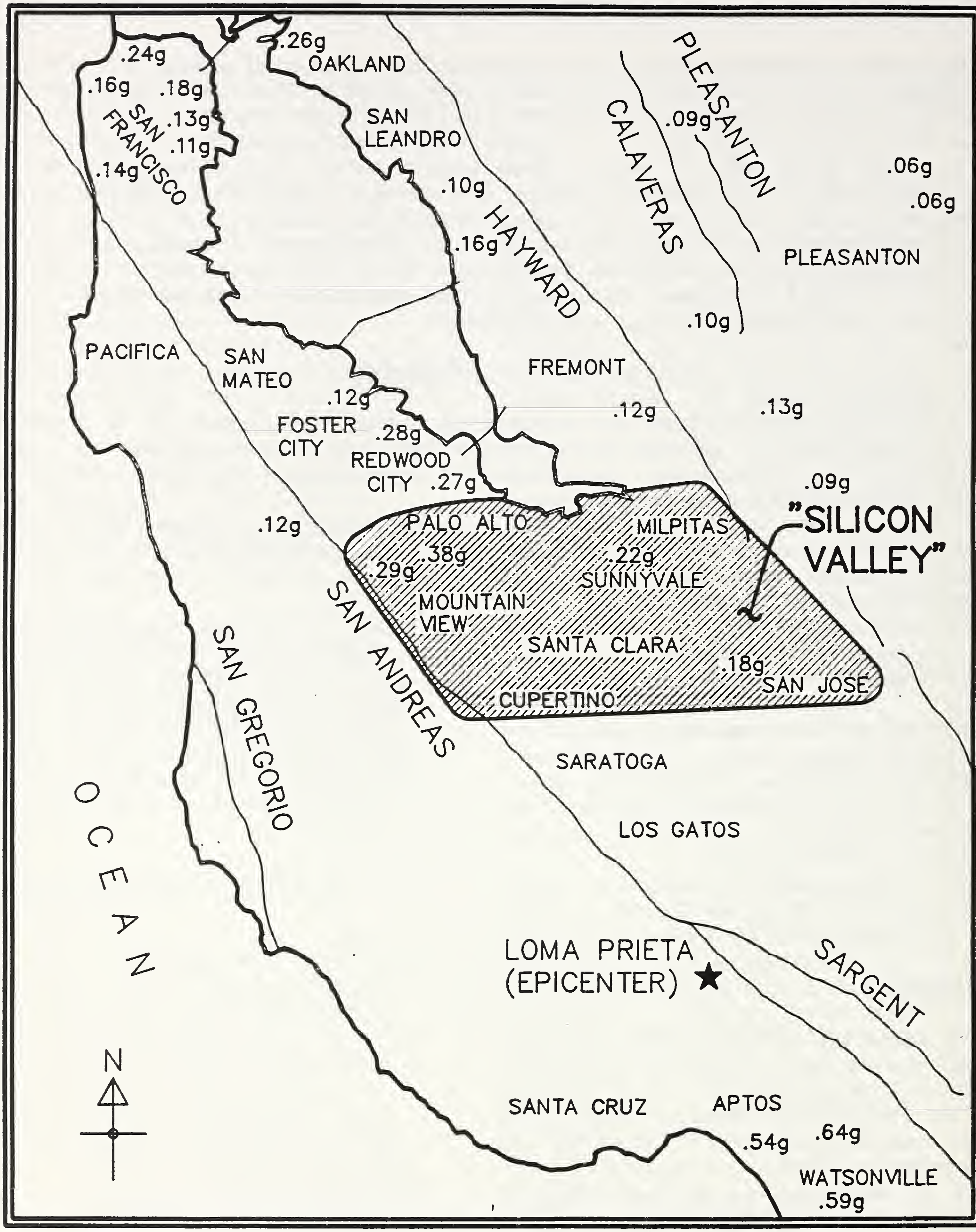


FIGURE 1 - GROUND ACCELERATIONS RECORDED DURING THE LOMA PRIETA EARTHQUAKE

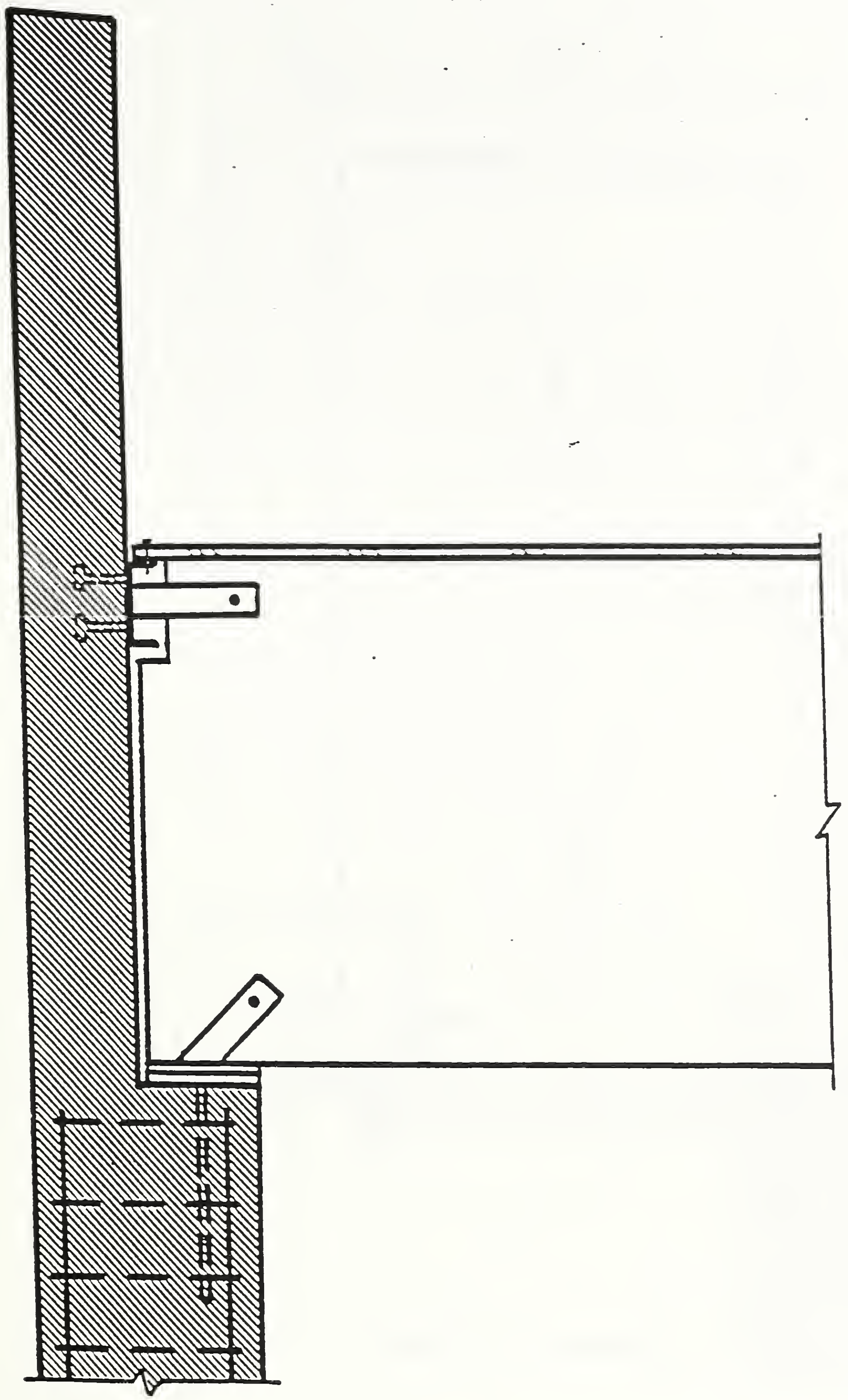


FIGURE 2 - ORIGINAL CONNECTION BETWEEN GLULAM BEAM AND TILT-UP WALL

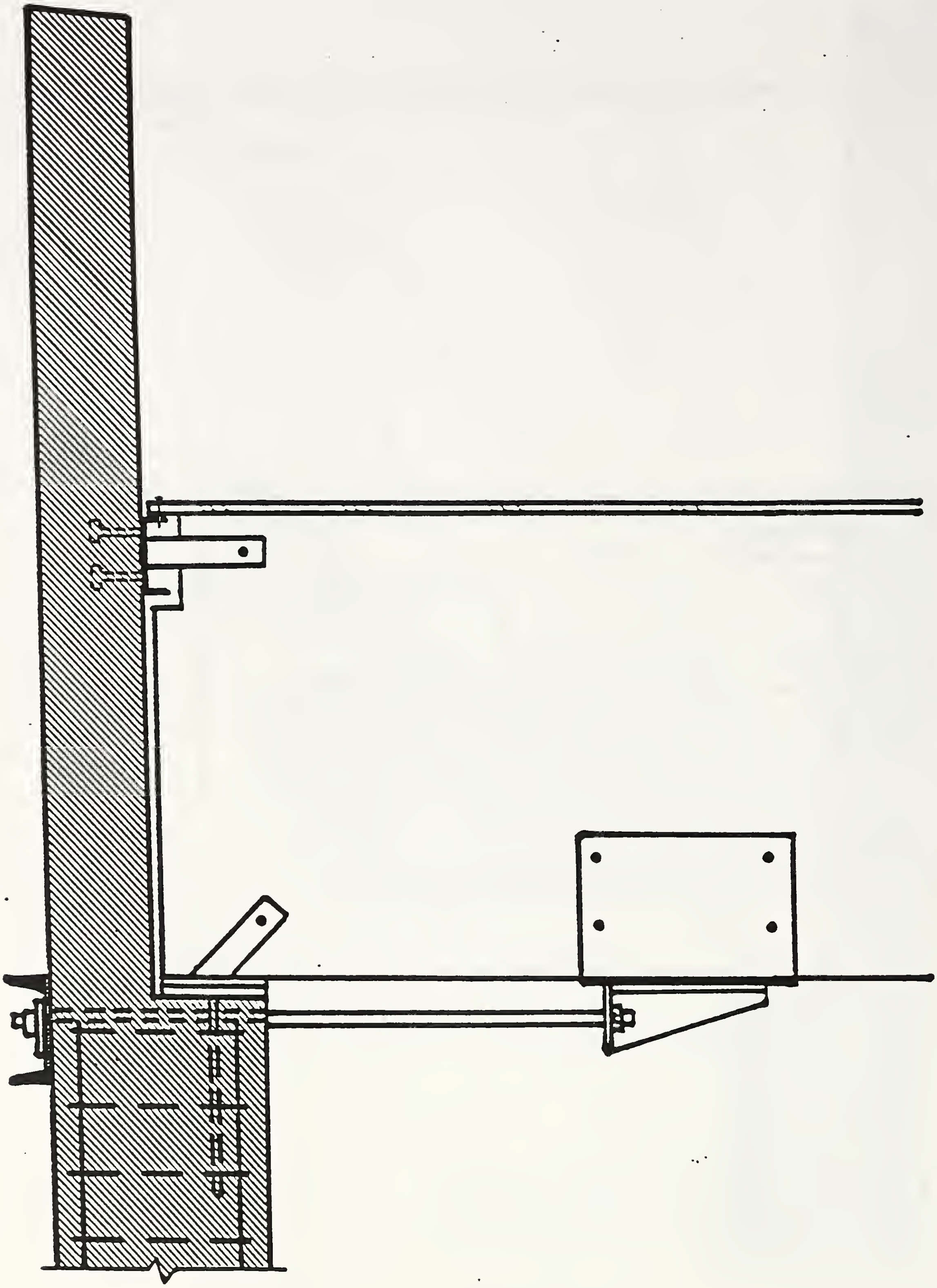


FIGURE 3 - RETROFITTED CONNECTION BETWEEN GLULAM BEAM AND TILT-UP WALL

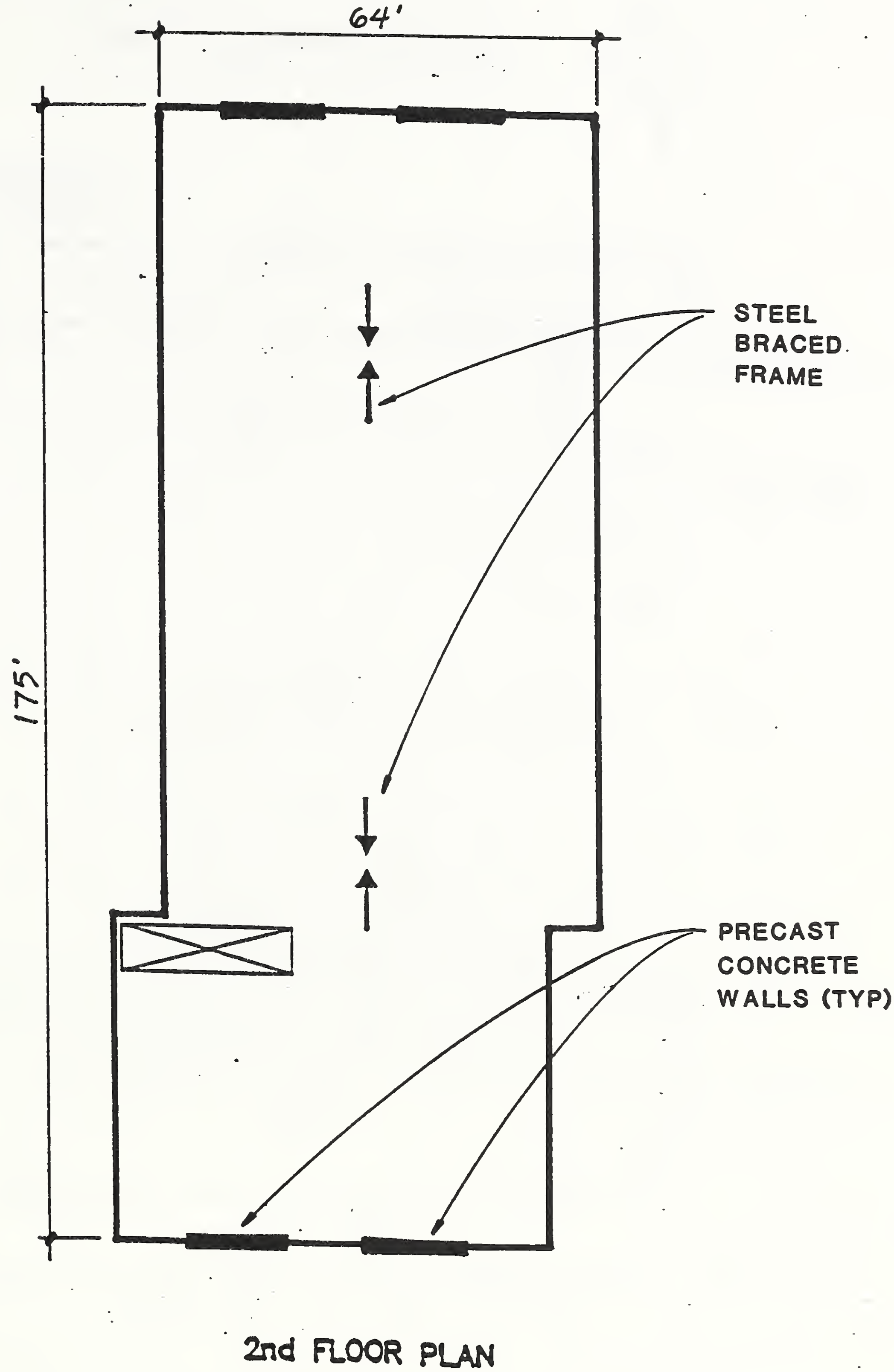
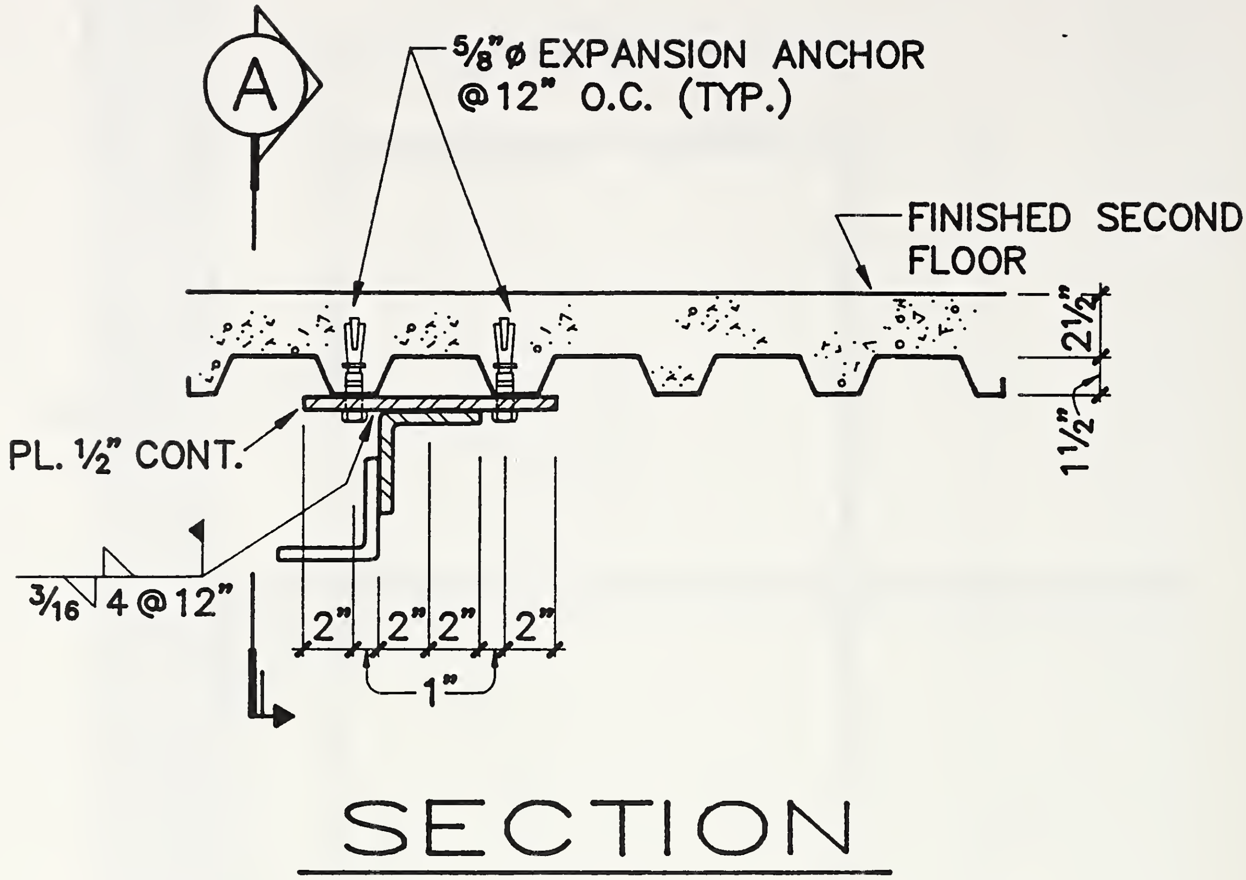


FIGURE 4 - FLOOR PLAN OF BUILDING DAMAGED DURING THE LOMA PRIETA EARTHQUAKE



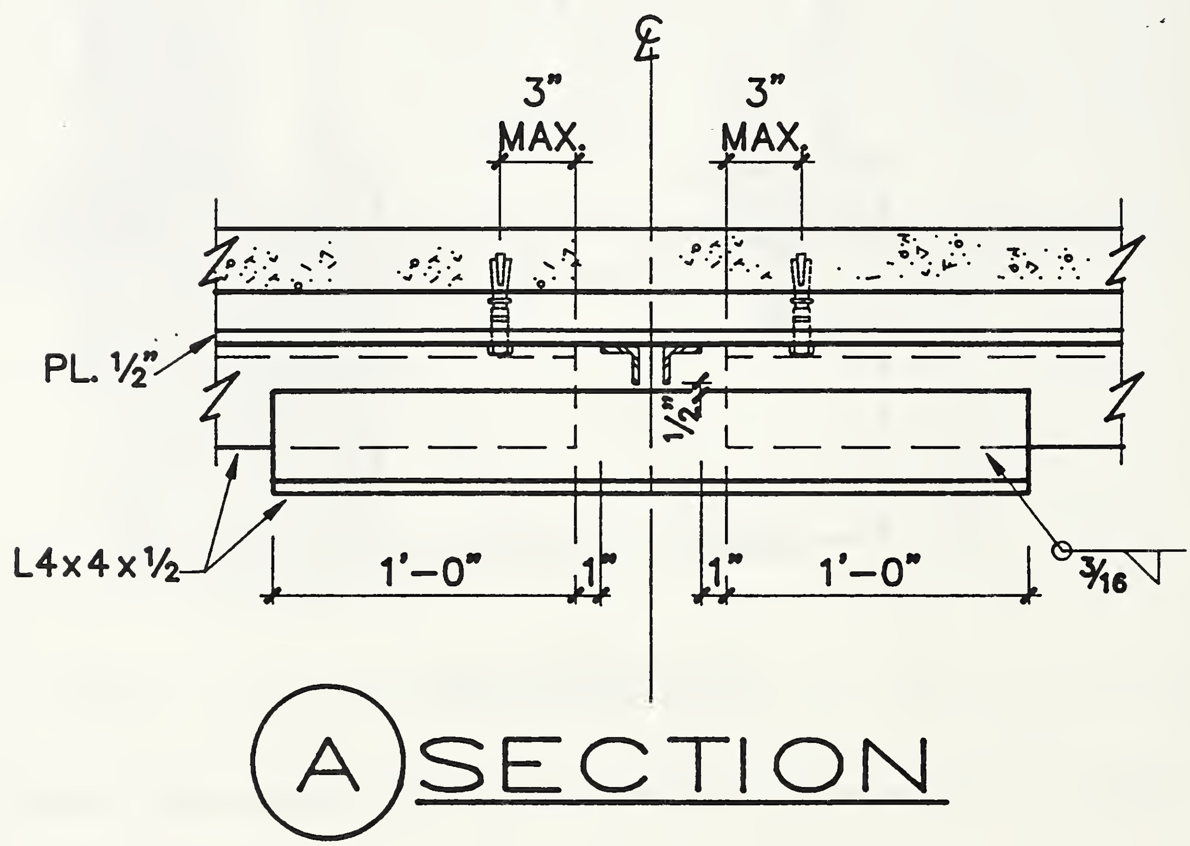


FIGURE 5 - RETROFIT OF SECOND FLOOR DIAPHRAGM TO ACCOMMODATE MISSING CHORD BARS

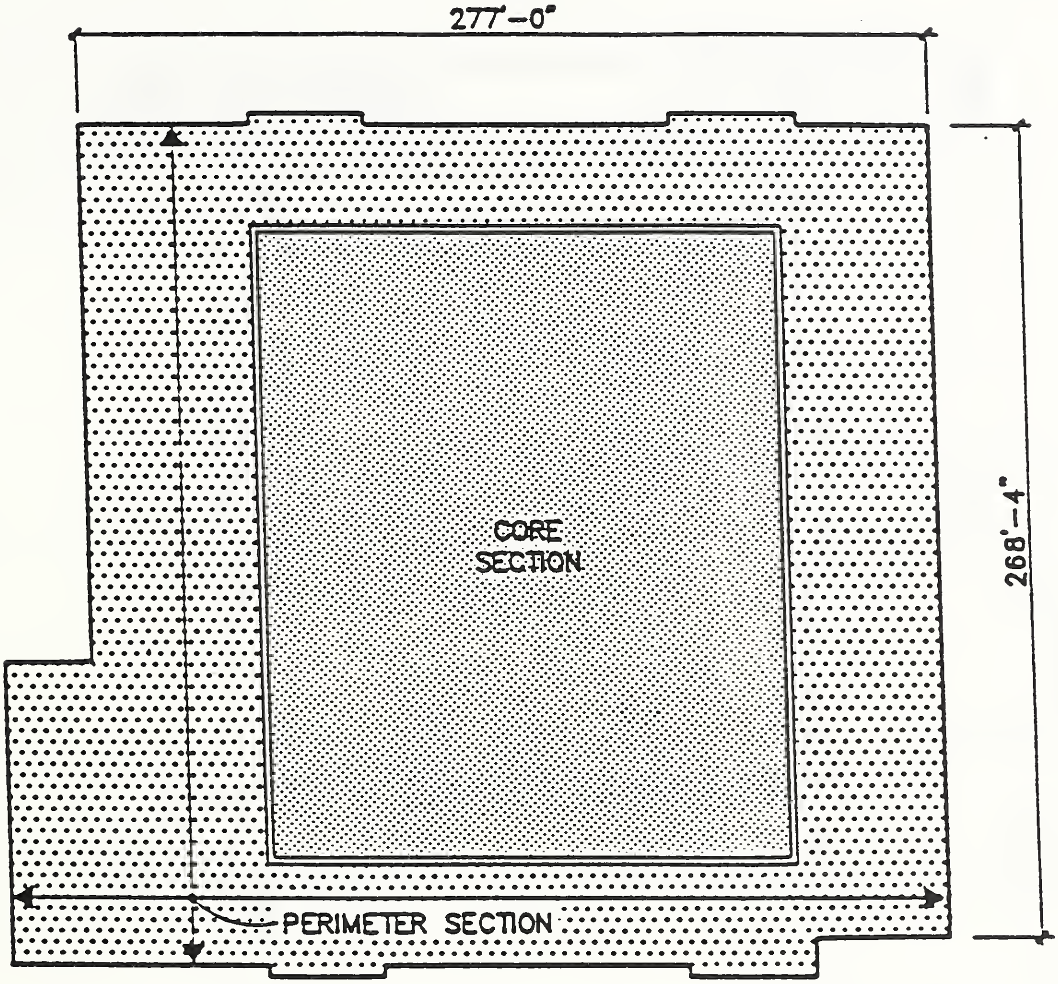


FIGURE 6 - SCHEMATIC PLAN OF BUILDING DAMAGED
DURING THE LOMA PRIETA EARTHQUAKE

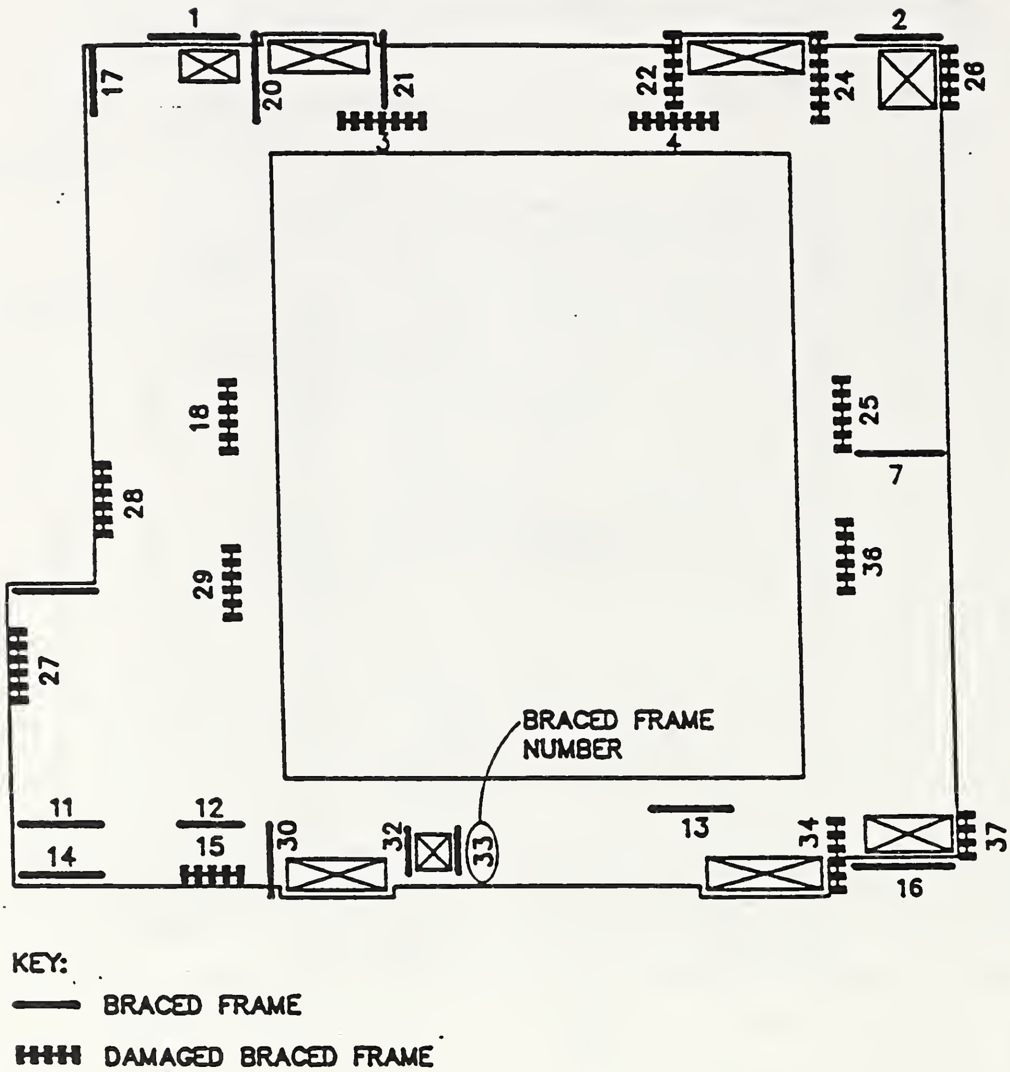


FIGURE 7 - BRACED FRAME LAYOUT

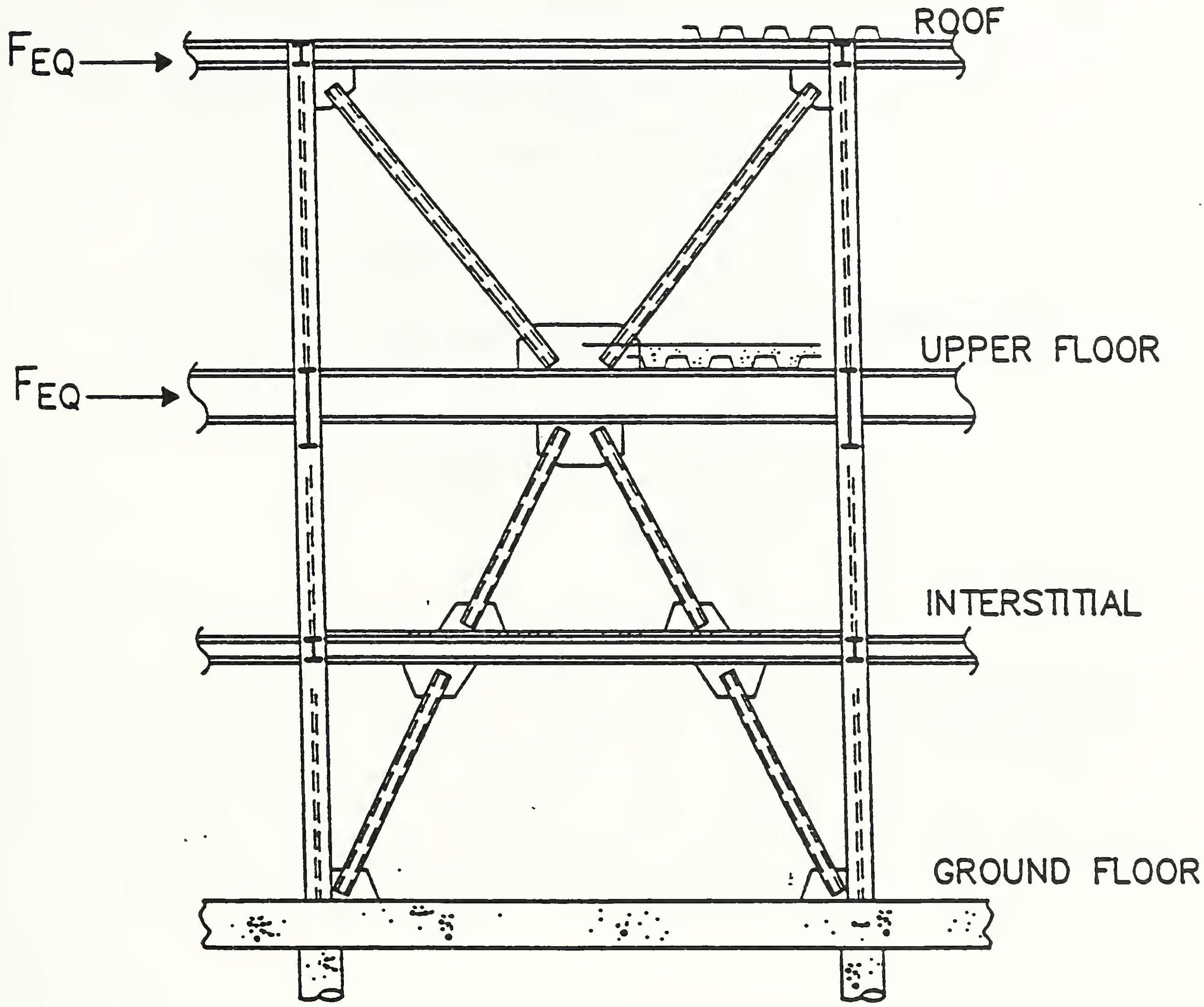


FIGURE 8 - BRACED FRAME DAMAGED DURING THE LOMA PRIETA EARTHQUAKE

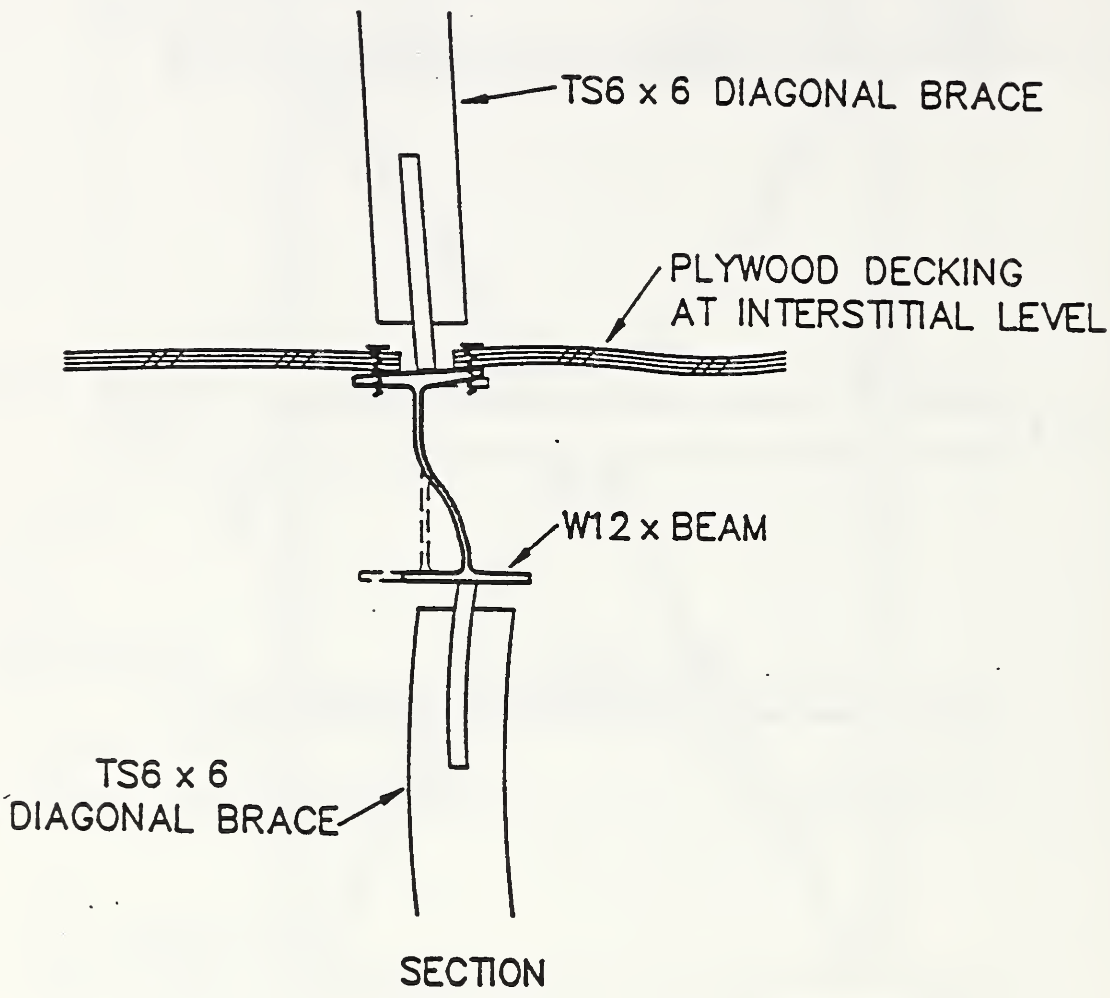


FIGURE 9 - FAILED CONDITION OF BRACED FRAME DAMAGED IN THE LOMA PRIETA EARTHQUAKE

Actual Examples of Seismic Judgement for Existing Buildings with or without Retrofitting

by

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Workshop on Evaluation, Repair, and Retrofit of Structures
UJNR Task Committees C and D
May 12 - 14, 1990
Gaithersburg, Maryland, U.S.A.

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Actual Examples of Seismic Judgement
for Existing Buildings with or without Retrofitting

S. OKAMOTO and T. KAMINOSONO

INTRODUCTION

"Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Building" is published by the Japan Building Disaster Prevention Association. This standard was proposed by the special Committee chaired by Prof. H. UMEMURA under the sponsorship of the Ministry of Construction, Japanese Government, in 1977. This standard may be used to evaluate the seismic capacity of existing low-rise reinforced concrete buildings.

Judgements were carried out on 41 buildings by the Evaluation Committee for Seismic Performance of RC buildings, chaired by Prof. H. Aoyama, Japan Building Disaster Prevention Association from 1979 to 1989.

JUDGEMENTS

Here, we report such items from the seismic judgement reports for the evaluated buildings as follows;

a. kind of constructin
b. use of buildings
c. construction year
d. motive of inspection
e. number of stories (original and after change)
f. retrofitting method
g.applied evaluation method
h.obtained values of seismic index of structure

The results of evaluation are listed if Table 1.
Main items made clear by Table 1 are as follows.

1) Construction kind of the inspected buildings was mainly of reinforced concrele (RC) building, except 6 buildings were constructed of steel-reinforced concrete (SRC).

2) Kinds of their use are mainly school, office, and department store. The buildings of these kinds of use occupies 29 cases out of 41.

3) Their construction years are mainly from 1957 to 1980. The oldest building is No.34 constructed in 1929. Buildings have been constructed before 1971, when related seismic code was revised, are 26. Three buildings from No.9A to No.9C were inspected during construction.

4) Inspections were carried on in 25 cases to evaluate the seismic safety of building which would expand upward or horizontally. Remains are insections for remodeling of building in 7 cases and simple inspections in 8 cases.

5) Numbers of stories of inspected building are mostly from 3-story to 5-story, 25 cases in total, and the highest building is 11-story.
6) As retrofitting methods being planned, retrofitting only by the infilled wall was selected in 15 cases. The methods with infilled walls and

Items to be Discussed

A) Motive of Inspection and Evaluation
a.Dicision by the provincial government for the public buildings etc.
b.Extension preliminary or newly planned
c. Large scale remodel or change of building use
d. building where some damage occure due to permanent, fire and /or earthquake load

B)Evaluation Method to be Applied
a.Necessity and adaptability of the simplified method
b.Problems related to the 2nd level screening method
c.To use plural evaluation methods to one buildings
d.To use the static screeing methods more fruently than dynamic response analysis method

C) Retrofitting Method
a.To use retrofitting by walls more often than ones of columns or by side walls
b.Merits and Demerits of retrofitting method of steel buildings
c.Typical retrofitting method of steel buildings
d.Ductirity of the members retrofitted by each method
e.Estimation Method of Retrofitting Effect

D)Judgement
a.Propriety for Iso of each level screening method
b.Items and the criteria in order to judge the results by dynamic response analysis
c.Consideration to ground condition and local topography

other methods was applied in 8 cases. Therefore infilled wall was used in 23 cases out of the retrofitting cases of 29. Retrofitting by reinforcing columns only or by steel braces is few.

7) Applied evaluation methods

a. The second and /or third level screening evaluation methods are mainly applied to many cases. Dynamic response analysis method and the revised seismic design method were adopted in the 9 cases of discussion out of 55 cases (Fig. 1).

b. Compairing with the evaluations before retrofitting, the higher level screening methods were applied to the evaluation after retrofitting. (Fig. 1)

c. The cases where two or three evaluation methods were applied to the same building are more than the cases only one method was used.(Fig. 1)

8) Final Judgement

a. The direction of building with Is is more than 0.7 by the 2nd level evaluation will be usually judged safe. and the direction of building with Is is more than 0.6 by the 3rd level evaluation will be usually judged safe. Remains will be usually recommended to be retrofitted (Fig.2a, 2b).

9) Others

a. Is values of building after retrfitting are larger than the value of 0.6. (Fig. 3)

b. Only one evaluation method were applied to the inspections after No.32 performed after 1987.

Number of Building	Kind of Construction	Use of Building	Construction Year	Motive of Inspection	Number of Stories (/Under Ground)		Retrofitting Method						Direction	Evaluation Method		Is Value
					Original	After Change	a. Reinforcing Columns	b. Infilled walls	c. Thickened walls	d. Side walls	e. Steel Braces or Truss	f. Structual Slit		Original	After Retrofit	
1(6)	RC	Dept.	'71	U.Ex'+R	5/1	7/1		0					Longi	2,3	(3),D	(0.12)
													Ridge	2,3	(3),D	(0.12)
2	RC	Sch.	'89	Rem.	4	--	--	--	--	--	--	--	Longi	2,3	--	0.42/0.38
													Ridge	2	--	0.83/ --
3	RC	Sch.	'61	Ins.	3	--	--	--	--	--	--	--	Longi	3	--	0.60
													Ridge	3	--	1.25
4	RC	Sch.	'61	Ins.	4	--	--	--	--	--	--	--	Longi	3	--	0.48
													Ridge	3	--	1.40
5	RC	Offi.	'70	H.Ex+R	7	7+7	0	0					Longi	(3)	3,(D)	(0.63)
													Ridge	(3)	3,(D)	(0.81)
7	RC	Dept.	'64	Rem.+R	5	--					0		Longi	--	--	--
													Ridge	--	3	(1.06)
8	RC	Offi.	'63	Ins.	4/1	--	--	--	--	--	--	--	Longi	1,2	--	0.39
													Ridge	1,2	--	0.62
9A	RC	Sch.	'80	Ins.	3	--	--	--	--	--	--	--	Longi	1,2,3	--	0.50/0.65
													Ridge	1,2,3	--	1.19/1.86
9B	RC	Offi.	'80	Ins.	3	--	--	--	--	--	--	--	Longi	1,2,3	--	0.89/1.06
													Ridge	1,2,3	--	0.57/0.53
9C	RC	Gym.	'80	Ins.	3	--	--	--	--	--	--	--	Longi	1,2,3	--	0.65/0.83
													Ridge	1,2,3	--	0.67/0.85
10	RC	Sch.	'69	U.Ex+r	3	4 (S1)		0	0	0			Longi	2	2,R	0.55 (0.85)
													Ridge	2	2,R	0.66 (0.67)
11	RC	Dept.	'78	U.Ex'+r	2/1	4/1		0					Longi	(3),R	3,R	(0.81)
													Ridge	(3),R	3,R	(0.85)
12	RC	Offi.	'68	Ins.+R	3	--		0					Longi	(1,2)	(1),2	0.48 (1.04)
													Ridge	(1,2)	(1),2	0.70 (1.25)
13	RC	Sch.	'71	Ins.+R	4	--						0	Longi	(1,2)	(1),2,(D)	0.53 (0.98)
													Ridge	(1,2)	(1),2,(D)	1.21 (—)
14	RC	Dept.	'70	Rem.	5/1	6/1 (S6)	--	--	--	--	--	--	Longi	1	--	1.24
		Park											Ridge	1	--	1.34
15	RC	Hosp.	'68	U.Ex+R	3	5 (S2)		0					Longi	2	2	0.74 (—)
													Ridge	2	--	0.98 (—)
16'1	RC	Dept.	'78	U.Ex'+R	2/1	4/1		0		0			Longi	--	3	(0.77)
													Ridge	--	3	(0.78)
17	RC	Offi.	'69	U.Ex'+R	2	4 (S2)		0		0			Longi	3	3	0.39 (0.80)
		Res.											Ridge	3	3	0.51 (1.03)
18	SRC	Sto.	'69	Rem.+R	7/3	--	0	0					Longi	(1,2)	1,2	(0.78)
													Ridge	(1,2)	1,2	(0.72)
19	RC	Offi.	'71	U.Ex'+R	2	5 (S3)		0					Longi	--	3,R	(1.05)
													Ridge	--	3,R	(0.88)
20	RC	Hotl.	'73	Rem.+R	5/1	--		0					Longi	--	(1),2	0.67 (0.67)
													Ridge	--	(1),2	0.56 (0.61)
21	RC	Offi.	'67	H.Ex+R	3	3+6	0						Longi	--	2,3	(0.45)/(1.08)
													Ridge	--	2,3	(0.68)/(0.81)
22	RC	Hall	'76	U.Ex'+R	2	4	0	0					Longi	3	3	0.39 (0.63)
													Ridge	3	3	0.67 (—)
23	RC	Kim.	'74	H.Ex'+R	3	3+3		0					Longi	2	2	0.64 (0.99)
													Ridge	2	--	0.96 (—)
24	RC	Fac.	'74	U.Ex'	1	2 (S1)	--	--	--	--	--	--	Longi	(1,2)	--	1.15
													Ridge	(1,2)	--	1.12
25	SRC	Dept.	'57	H.Ex+R	7/2	7/2+7/2		0	0		0		Longi	--	3,(D)	(0.82)
													Ridge	--	3,(D)	(0.82)
26	RC	Sch.	'68	Ex'+R	2	4 (IS2)		0					Longi	(1)	1,3	(0.86)
													Ridge	(1)	1,3	(0.89)
27	RC	Off.	'67	H.Ex+R	5	5+5 (S1)	0						Longi	--	(2),3,D	(1.00)
		Res.											Ridge	--	(2),3,D	(1.06)
28	RC	Hos.	'68	U.Ex+R	4(S2)	5 (S3)	--	--	--	--	--	--	Longi	(1,3)	--	(1.36)
													Ridge	(1,3)	--	(1.05)
29	RC	Offi.	'84	Rem.'+R	4/1	4/1 (IS1)		0					Longi	--	1,3	(1.41)
													Ridge	--	1,3	(1.02)
30	RC	Hall	'76	U.Ex+R	2	3 (S1)		0	0				Longi	(2)	2	(0.75)
													Ridge	(2)	2	(0.73)
31	RC	Sch.	'72	U.Ex'+R	2	4		0					Longi	--	3	(1.52)
													Ridge	--	1,3	(1.09)
32	RC	Hos.	'69	U.Ex+R	2	3 (S1)		0					Longi	1,2	2	0.61 (0.96)
													Ridge	1,2	2	0.92 (0.92)
33	RC	Hos.	'71	U.Ex'	3/1	4/1 (S1)	--	--	--	--	--	--	Longi	2,3	--	0.57/0.82
													Ridge	2,3	--	0.75/0.79
34	SRC	Offi.	'29	Ins.+R	6/1	--	0						Longi	--	2	(0.57)
													Ridge	--	2	(0.68)
35	RC	Gym.	'66	U.Ex+R	2/1	3/1 (S1)		0					Longi	--	3	(0.73)
													Ridge	--	3	(0.64)
36	SRC	Offi.	'69	U.Ex'+R	4	8 (S4)	0	0					Longi	--	2	(0.66)
													Ridge	--	2	(1.41)
37	RC	Offi.	'72	U.Ex'+R	3/1	5/1 (S2)					0		Longi	--	2	(0.69)
													Ridge	--	2	(0.70)
38	RC	Offi.	'64	U.Ex'	5/1	5/1 (S5)	--	--	--	--	--	--	Longi	1	--	1.05
													Ridge	1	--	1.98
39	SRC	Offi.	'84	H.Ex	11/4	--		0	0				Longi	--	3	(0.88)
													Ridge	--	3	(0.58)
40	SRC	Offi.	'78	U.Ex	3	5 (S2)	--	--	--	--	--	--	Longi	2	--	0.78
													Ridge	2	--	0.81
41	RC	Offi.	'71	Rem.+R	4/1	4/1 (S4)		0					Longi	--	1	(0.97)
													Ridge	--	1	(3.46)

Explanation for tables and figures

1. Number of Inspected Buildings : 41Buildings
(No.1=No.6. No.11=No.16)

2. Kind of Constructions :

RC : Reinforced Concrete	27+6=33
SRC : Steel framed Reinforced Concrete	3+4=7

3. Use of Buildings (after change of use)
Kind of Use after change (number of buildings)

Sch. (school)	; 9	.Gym. (gymnasium)	; 1+1
Offi.(office)	; 8+7	.Park. (Parking Place)	; 1
Dept.(department store)	; 4	.Hotel	; 1
Hos. (hospital)	; 2+2	.Factory	; 1
Hall.	; 2	.Store	; 1
		.Kin. (Kinder garden)	; 2

4.Construction Year, (number of Buildings) *Means Under Construction

a(- 1965) , 6+3 d(1976 -1980) , 3+1
b(1966 - 1970) , 12+2 eUnder Construction, 3
c(1971 - 1975) , 7+3

5. Motive of Inspection (31 cases in Total)

. U.Ex*+R (Scheduled up-ward extention with retrofit) : 6+2
. U.Ex+R (Up-ward extention with retrofit) : 4+2
. U.Ex* (Scheduled up-ward extention without retrofit) : 1+2 17+8
.H.Ex+R (Scheduled or not-sche. horizontal extention with retrofi : 5+1
.Rem.(+R) (Remodel with or without retrofit) : 6+1
.Ins. (Simple inspection) : 6
.Ins. +R (Inspection and Retrofit) : 2+1
.Ex +R (Remodel from 2 - story to 4 - story without hight extension ; 1

6(7).Number of stories
(Note)
3 - 4(S1) ; The plan is to extend one story by steel constrction above
4/1 -4/1 (IS1) ; The plan is to construct floor slabs of steel in the Fukinuke of the existing 4 story buildings

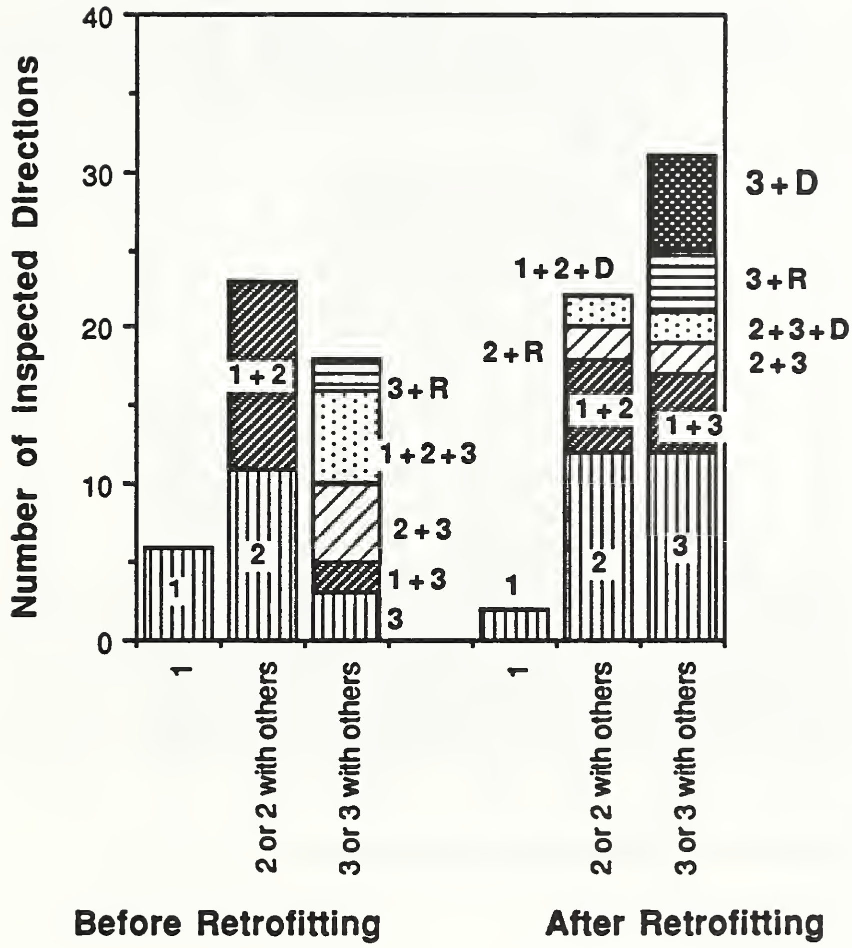
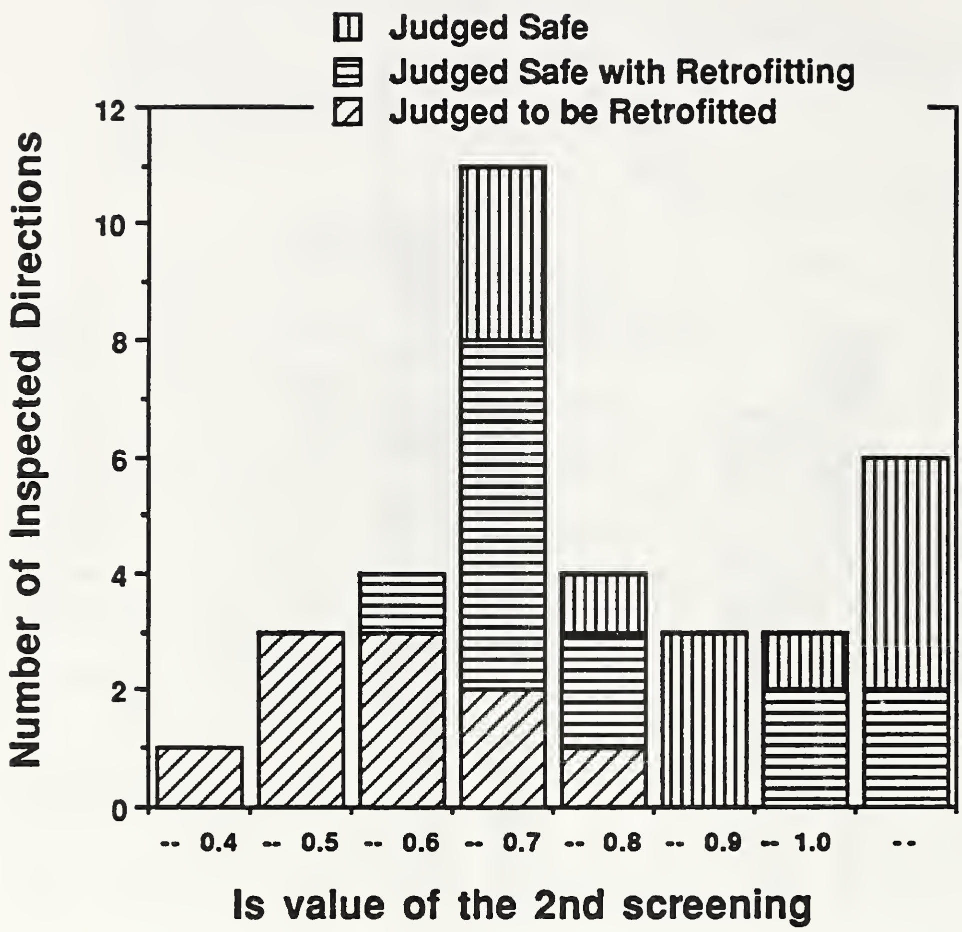

Number of Inspected Directions
0
10
20
30
40
1
2
1+2
3
1+3
2+3
1+2+3
3+R
1
2
1+2
2+R
1+2+D
3
1+3
2+3
2+3+D
3+R
3+D
1
2 or 2 with others
3 or 3 with others
1
2 or 2 with others
3 or 3 with others
Before Retrofitting
After Retrofitting

Fig. 1



Judged Safe
Judged Safe with Retrofitting
Judged to be Retrofitted
Number of Inspected Directions
0
2
4
6
8
10
12
-- 0.4
-- 0.5
-- 0.6
-- 0.7
-- 0.8
-- 0.9
-- 1.0
--
Is value of the 2nd screening

Fig. 2 - a

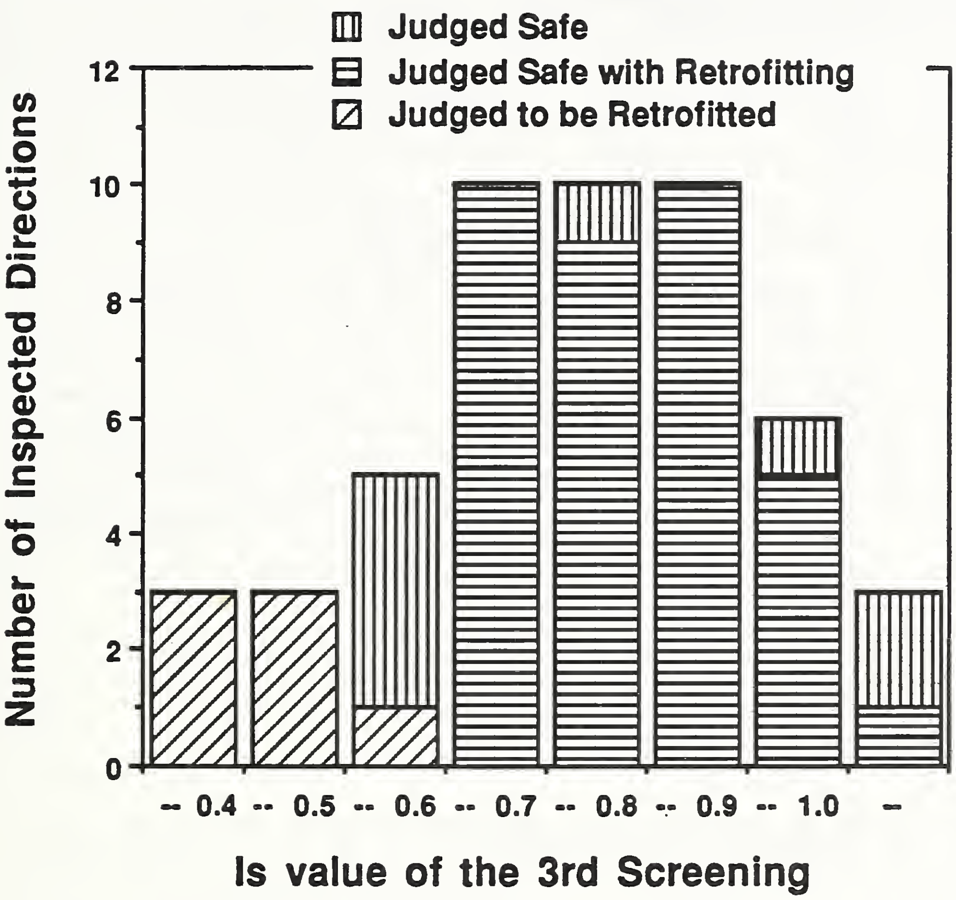
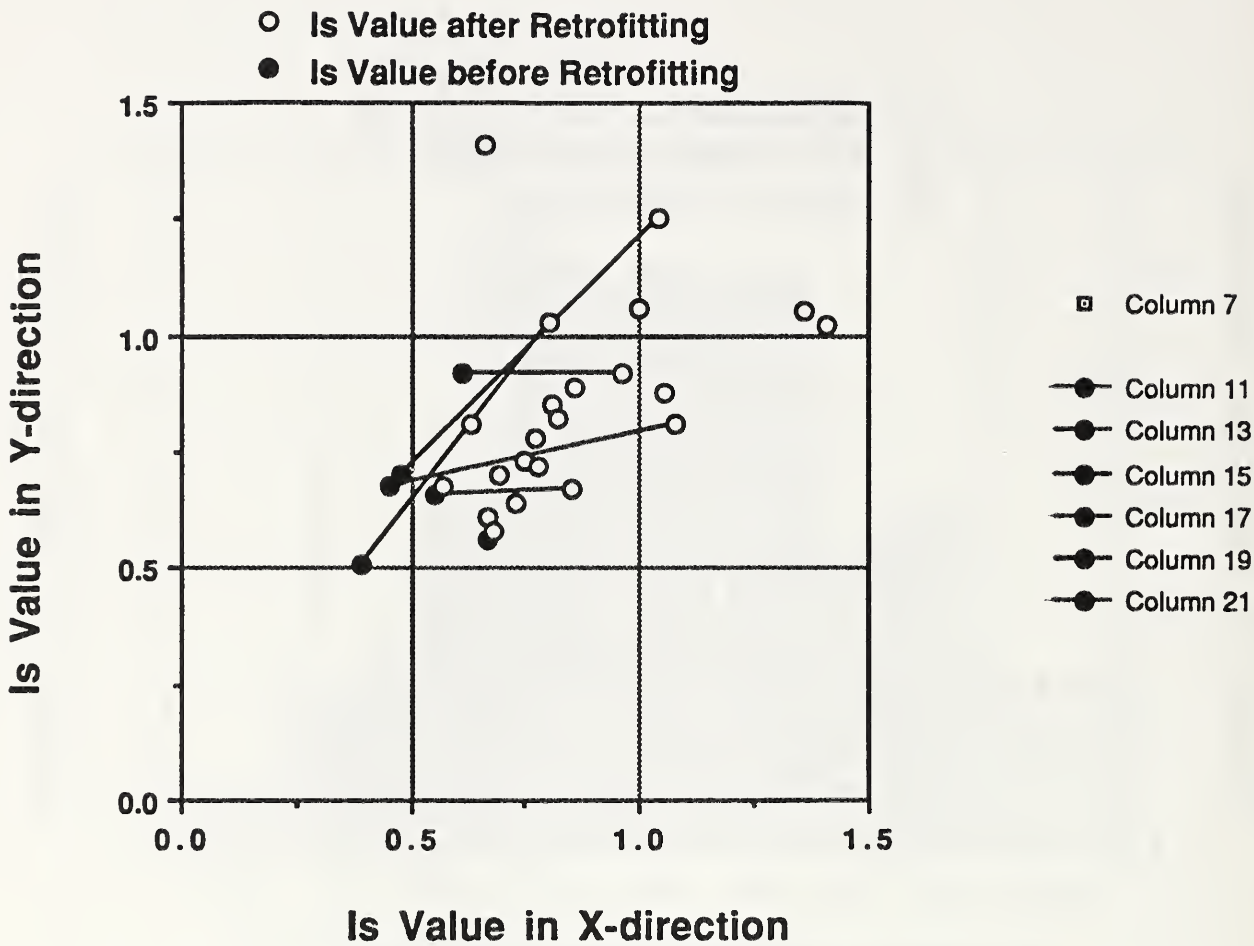

Judged Safe
Judged Safe with Retrofitting
Judged to be Retrofitted
Number of Inspected Directions
0
2
4
6
8
10
12
-- 0.4 -- 0.5 -- 0.6 -- 0.7 -- 0.8 -- 0.9 -- 1.0 --
Is value of the 3rd Screening

Fig. 2 - b



Is Value after Retrofitting
Is Value before Retrofitting
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
Is Value in Y-direction
Is Value in X-direction
Column 7
Column 11
Column 13
Column 15
Column 17
Column 19
Column 21

Fig. 3

Implications of the Loma Prieta Earthquake on Building Requirements and Regulations

by

Franklin Lew[1]

ABSTRACT

The casualties and property damage caused by the 1989 Loma Prieta earthquake have heightened public awareness of the potentially immense losses a major earthquake in California can cause, and have prompted State and local governments to consider legislation to facilitate and/or mandate programs aimed at seismic hazards reduction and seismic preparedness. Many bills have been introduced, and some already have become law. These efforts likely will have significant implications for building owners, designers, and code enforcement agencies. The potential hazards posed by an estimated 30,000 unreinforced masonry (URM) buildings in California have received considerable attention. Many jurisdictions have adopted, or are developing, hazards reduction programs for URM buildings.

KEYWORDS: Earthquake; unreinforced masonry; URM; seismic hazards; strengthening.

1. INTRODUCTION

The 1989 Loma Prieta earthquake damaged existing buildings in patterns and extent that could have been predicted based on past earthquakes. Most of the injuries and property losses occurred in older buildings built before the advent of code requirements for seismic design. While some newer buildings also sustained major damage, analyses showed that the inadequate performances generally were caused by building features or design approaches which would require special consideration, or are no longer allowed, by the 1988 Uniform Building Code and the 1987 SEAOC Blue Book. Loma Prieta, then, did not reveal any significant inadequacies in current seismic code requirements for new buildings, although the quake was not a sufficiently severe test for that purpose.

The earthquake was an effective reminder of the need for governments to encourage and/or mandate seismic hazards reduction programs in the existing building stock. Significant progress in advancing the agenda of seismic safety often comes in the immediate aftermath of a damaging earthquake, when the public, policy makers and elected officials are more receptive to bearing the economic and social costs for mitigation efforts. Loma Prieta is continuing this pattern. Mitigation efforts are in progress or being considered in San Francisco and many other cities in California. At the State level, many bills have been introduced in the legislature which address seismic safety and preparedness.

2. THE SAN FRANCISCO EXPERIENCE

2.1 Building Stock Profile

The City's 49 square miles contain 130,000 parcels of land and an estimated 150,000 buildings. The predominant type of construction is wood-frame (144,000 buildings). Some 2,000 are unreinforced masonry (URM) bearing wall buildings, containing 35 million square feet. The remainder is a mixture of structural frame with masonry in-fill walls, reinforced concrete/masonry, steel, tilt-up, high rise, etc. Prior to Loma Prieta, the City had compiled a database on the URM buildings preparatory to considering a mandatory retrofit program for them. Some interesting data are shown in Figures 1-3 which illuminate some issues and concerns that must be addressed in a program. First, the URM building stock is old (by West Coast standards). A majority of them were built during the four years after the 1906 quake (less than 20 predate the quake, and few were built after the 1933 Long Beach earthquake). The median year of construction for all existing URM buildings in the City is 1909, and most buildings show the ravages of time. Many factors contribute toward weakening the nominal lateral force resistance capabilities these buildings had when new, including alterations over the years that degraded the stress paths, differential settlements that created locked in stresses or cracks in masonry walls, and deferred maintenance that have weakened building materials and components, particularly the mortar. Many of these buildings are vulnerable even to moderate ground shaking. Had Loma Prieta's duration not been so unusually short, URM buildings as a

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group likely would have sustained much greater damage. Second, one-third of these buildings are used primarily for residential purposes and contain some 26,000 dwelling units. The rents for these units are among the lowest in the City. Vacancy rates are below one percent. Many tenants would find it difficult to relocate during the strengthening work, and to afford the rent increases were the landlord to pass through the costs for the work. These negative socio-economic impacts have impeded the development of a mandatory seismic strengthening program for URM buildings.

The 4,000 non-URM, non-wood frame buildings in San Francisco include many with designs and construction materials that have not perform well in other earthquakes. These include buildings having non-ductile concrete frames, unreinforced masonry infill walls, lift slabs, tilt-up walls, etc. No data on their numbers are available at this time.

2.2 <u>Damage to buildings</u>

Due to the 100 kilometer distance from the epicenter to San Francisco, ground accelerations in the City in areas of competent soil typically were only in the 8 to 10% of gravity range. However, in areas of Bay Mud overlaid by non-engineered fill, ground shaking was amplified two to four times. Many of the damaged URM buildings were located in these poor soils areas (Figure 4). The greatest loss of life occurred at a 4-story URM building when a portion of the front wall collapsed outward and killed 6 people on the sidewalk and street.

Damage to wood-frame buildings was widespread in the Marina district. The unconsolidated and saturated sand fills in the area amplified the ground shaking several-fold. Most of these non-engineered buildings in the area had inadequate lateral stiffness at the ground floor due to wall openings for garage doors. Many sustained significant damage, and several collapsed.

Engineered buildings designed in the last 20 years performed well, although as mentioned previously, Loma Prieta was not a significant test. A notable exception was a building built in the mid-70's and having concrete shearwalls augmented by concrete frames. The building, which had been strengthened before the quake as a result of a third party evaluation, sustained notable damage and had to be partially vacated for repairs to be made. The performance appears to validate the changes that have been made to the codes since the building was designed, changes which penalize designs with plan irregularities and non-ductile frames. Overall, Loma Prieta did not reveal any significant shortcomings in the seismic provisions in the current UBC. The issues of ground shaking amplification and soil-structure interaction, which was the focus of much attention after the Mexico City earthquake, are being addressed again, as discussed below, and may result in a change to the City's building code.

The City's Bureau of Building Inspection (BBI) made use of volunteer engineers and inspectors to inspect all the URM buildings. In addition to the ATC-20 datasheet that was filled out for all inspected buildings, a datasheet was developed specifically to record damage to URM buildings (Figure 5). A tabulation of some of this data is shown in Tables 1 and 2. Much of the data has yet to be analyzed with other information in the URM building database, but preliminary work have shown some apparent relationships. Buildings on poor soils sustained greater damage, as did buildings with taller first story heights.

Over a hundred URM and other non-wood frame buildings had been seismically retrofitted prior to the earthquake to the base shears of the 1973 Uniform Building Code. These buildings apparently performed well, with most sustaining nominal or no damage. It is likely NSF will fund a more detailed study of the performance of these buildings. The results will help to calibrate and/or validate the effectiveness of the retrofit requirements that have been in the San Francisco building code since 1973.

Parapets on hundreds of URM buildings had been braced earlier under the Parapet Safety Program. The value of the program was demonstrated not only by the relative absence of damage to parapets, but also by the number of instances where wall-to-floor separations and wall damage at the upper stories indicated the roof-to-wall anchors and the bracing installed under the program had prevented out-of-plane wall failures. Clearly, bracing parapets is the most cost-effective measure that can be taken to reduce seismic hazards in URM buildings, and such work is the minimum that should be mandated in high seismic regions.

2.3 Repairs to earthquake-damaged buildings

After a damaging earthquake, people tend to make whatever repairs it takes to quickly resume normal activities and use of a building. Owners, having sustained losses from the damage, lost business, lost rents, etc, often are not receptive to performing seismic strengthening on their buildings as part of the repairs. In San Francisco, the problem is compounded because owners of URMs are aware that a mandatory strengthening program is likely within the next couple of years, and they don't want to over or under spend for upgrades. Many are choosing to restore their buildings to the pre-quake state. They will get several more years of use and rents from the building before compliance deadlines are reached. Many URM buildings are nearing economic obsolescence, and owners may choose to demolish a building when the deadline for retrofit draws near.

The City certainly wanted owners to go beyond damage repairs. However, codifying a reasonable set of standards proved difficult. During attempts to develop mandatory upgrades for damaged URM buildings, questions arose which showed that requirements must of necessity be highly judgmental in nature. A few examples, just for the masonry wall itself, illustrate this point. How much of a building's lateral force resistance capacity must be damaged before Section 104(f) (the City's standard for seismic strengthening when required by other code provisions) is triggered? How is the capacity determined for a masonry wall with a crack? How big can a crack be, in terms of crack width, length or offset from a plane before the wall should be dismantled and rebuilt instead of repaired by gluing back together with epoxy injections? Should a damaged portion be allowed to be replaced with reinforced masonry, thereby creating 'hard spots' in the wall that could be detrimental in some situations? The right answers to these questions depend on the building and circumstances involved. In the end, BBI left the decision to repair or retrofit to the owner and his engineer. This may appear to be an abdication of responsibility, but BBI's perspective is that a building that is not retrofitted now will be caught in a few years when a mandatory retrofit program is implemented. Most responsible engineers are recommending Section 104(f) retrofits to owners.

An issue that has received ongoing debate is whether and/or how to account for the ground acceleration amplifications that were observed in areas of poor soils. As mentioned above, a statistically meaningful correlation was found to exist for levels of building damage and poor soils. Using UBC provisions, base shear calculations are not sensitive to the site coefficients until the building period exceeds about 0.3 seconds. Thus, upping the soil classification from sat S1 to even S4 would not result in higher lateral force requirements for most URM buildings. There have been discussions of a larger-than-unity multiplier for the 'Z' parameter in areas of poor soils. The foundation subcommittee of the Joint SEAONC/ASCE Building Code Committee is looking into the issues.

3. SEISMIC HAZARDS REDUCTION EFFORTS

3.1 California Legislature

In 1986 the Legislature enacted SB 547. It required cities and counties to perform a census to identify all URM buildings in their jurisdiction. And it required that the local governments develop and implement a program of seismic hazards mitigation for these buildings. As a minimum, the program could consist of notifying URM building owners that their buildings are potential seismic hazards. By last October, more than one-half of the cities and counties had performed the census. Less than two dozen jurisdictions had passed legislation to require strengthening.

Loma Prieta has spurred activity at both the local and State levels. More cities have passed, or are in the process of considering, legislation for mandatory retrofit programs. The California Seismic Safety Commission has developed a bill that would require all jurisdictions to adopt programs to mandate strengthening of URM buildings to a specific standard (the standard is described in detail in Section 3.2). The bill would also require all strengthening work to be completed by the year 2000. At this writing, an amended version is in the legislative committee process. The prognosis is guarded.

Loma Prieta has prompted the introduction of over one hundred bills that focus on seismic safety, hazards mitigation, earthquake insurance, and financial assistance to owners and local government for seismic hazards mitigation costs. The financial element is absolutely critical to any program. Nobody is against safety. The question is who is going to pay for it. In many cases, lenders won't make loans for this type of work (keep in mind that the avowed purpose of seismic hazards reduction programs is focused on life safety and not property preservation). Ultimately, government is going to have to step in and help spread the pain. General obligation and revenue bonds, and outright grants and tax credits are being explored.

3.2. Structural Engineers Association of California

The Structural Engineers Association of California (SEAOC) is completing a multi-year project to develop a set of seismic strengthening provisions for URM buildings. [SEAOC is the organization that developed and updates the 'Blue Book', which contains seismic design provisions that are adopted into all three model building codes in the United States]. The URM provisions are expected to be endorsed by the California Seismic Safety Commission (SSC) for use by all California cities and counties. These same provisions, in codified format, are expected to be adopted by the International Conference of Building Officials into the 1991 edition of the Uniform Code for Building Conservation. A commentary to the provisions also has been developed with funding by the SSC, and it may be published by SEAOC.

SEAOC has formed a hazardous buildings committee to address not only URM buildings, but also other construction types that have exhibited poor performance, such as non-ductile concrete frame and tilt-up buildings. This work was underway before Loma Prieta, and it has gained new priority since the quake. There is a bill in Sacramento that would extend SB 547 to cover such buildings, and while the bill has been stalled for this legislative session, it could be revived. SEAOC wants to anticipate developments and have some recommendations available if a law is enacted, instead of reacting as it did with the passage of SB 547. SEAOC plans to publish a volume of recommended provisions and commentary for seismic retrofit of existing buildings (dubbed the 'green book'). This document is likely to be as influential with designers and code officials as the blue book has been for new buildings. The completion and publication dates are uncertain at this time.

3.3 City of San Francisco

The City had in the past given attention and resources to seismic hazards reduction. However, it was somewhat of a hit-or-miss affair, with efforts being made only in conjunction with other projects. Three years ago, the author was given the task of providing a coordinated and systematic focus on seismic safety. The Seismic Safety Program has two major components:

1) Reduce seismic hazards in over 450 City-owned buildings. Seismic evaluations are made to assess probable performance. Deficiencies are prioritized for correction. Financing for the work is obtained. And detailed design and construction are performed. To date, some 170 buildings have been evaluated, mostly by engineering consultants and using the approach outlined in ATC-14. Each building is given a hazards rating on a scale of 1 to 4, corresponding to minor, moderate, and major damage, and potential collapse. Since the average age of City-owned buildings exceeds 50 years, it is not surprising that more than one-third of the buildings evaluated to date have ratings of 3 and 4. Mitigation work is prioritized by the hazards rating, but also by other considerations such as the post-earthquake need for the building to remain functional, and the size and occupant load in the building. General obligation bonds will be used to fund the strengthening work, and also other ancillary work such as asbestos removal and disabled access. A $60 million bond issue was placed on the ballot before the earthquake occurred. The voting took place two weeks after the quake, and the measure received 82% approval. A second bond measure was quickly assembled for the June 1990 election to take advantage of the electorate's increased awareness of seismic hazards and its willingness to fund mitigation work. The measure would provide $332 million to continue the program, and would strengthen most of the buildings in the Civic Center area, including the City Hall, Opera House and Civic Auditorium. (Postscript: the measure received 78% approval).

2) Develop a program to address seismic hazards in privately-owned buildings. The first phase has focused on the 2,000

URM buildings. The significant socio-economic problems have slowed progress in developing the ordinances needed to create a mandatory strengthening program. The City is at the mid-point in writing a major environmental impact report and a socio-economic impact report for the program. It is uncertain at this writing whether the strengthening requirements will be similar to the SEAOC recommendations. Many people have said tenants and owners cannot afford that level of strengthening (estimated to be 500 to 750 million dollars). Loma Prieta has changed the balance somewhat, and there is now less focus on reducing strengthening requirements and more effort on developing local and state government financial assistance for affected building owners.

4. CONCLUSIONS

The Loma Prieta earthquake was not a major test of modern seismic code requirements, and few changes in the technical requirements are likely. The earthquake again reminded us that the greatest seismic risks comes from existing buildings which were designed to earlier seismic codes, or which had no seismic design at all. The heightened public awareness of these hazards hopefully will translate into changes in public policies on seismic safety. To that extent, we can expect changes and additions in our codes and regulations that will reduce these risks.

Continued research on effective retrofit measures are important. However, this knowledge will have the most impact when incorporated into mandatory retrofit programs. It is here that the greatest difficulty will be encountered. It will be difficult to transfer the results of research to mandatory building codes and regulations unless the socio-economic-political impacts are adequately addressed. Safety is not only a technical issue. Engineers need to articulate their case well because cogent arguments can and will be made for other claims on society's finite resources. Loma Prieta has made our task easier.

San Francisco URM Buildings
Selected Data

Age Distribution for 2015 Buildings

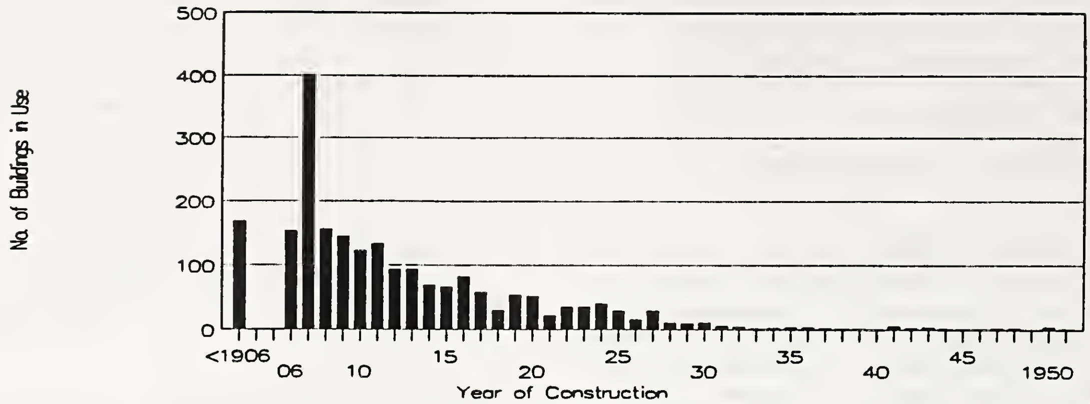


Figure 1

Height Distribution for 2015 Buildings

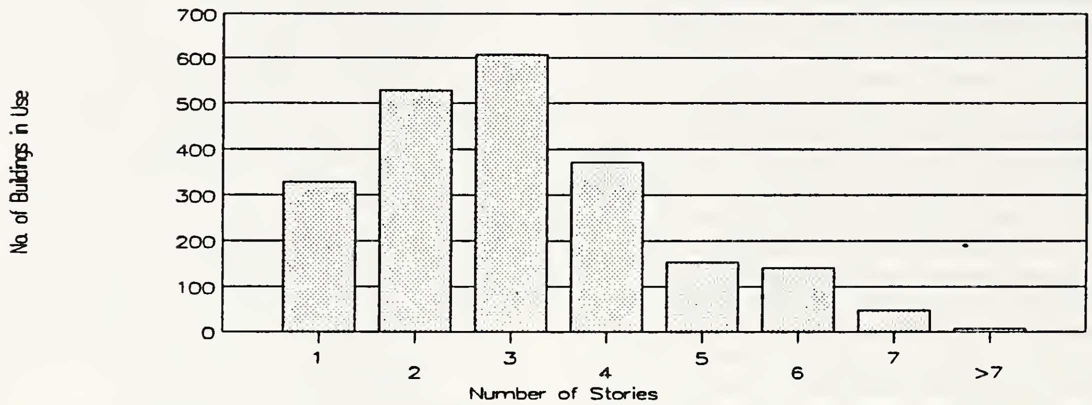


Figure 2

(730 Residential Bldgs — 26,400 Units)

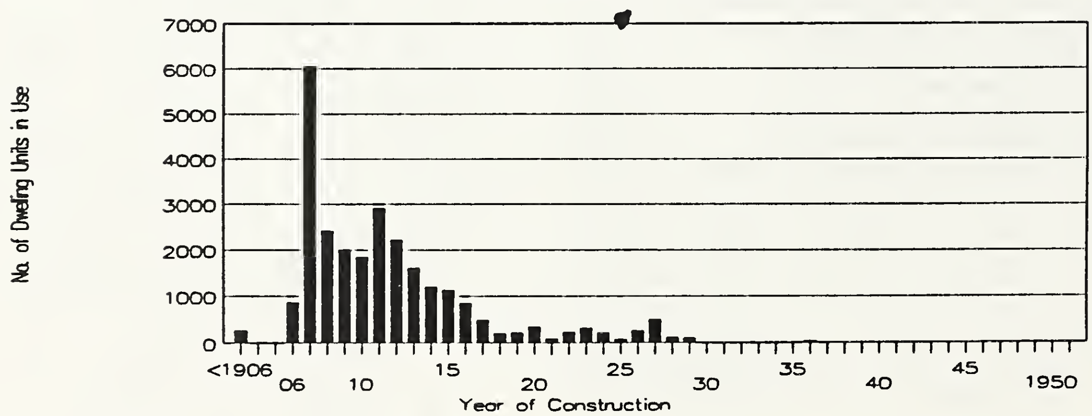
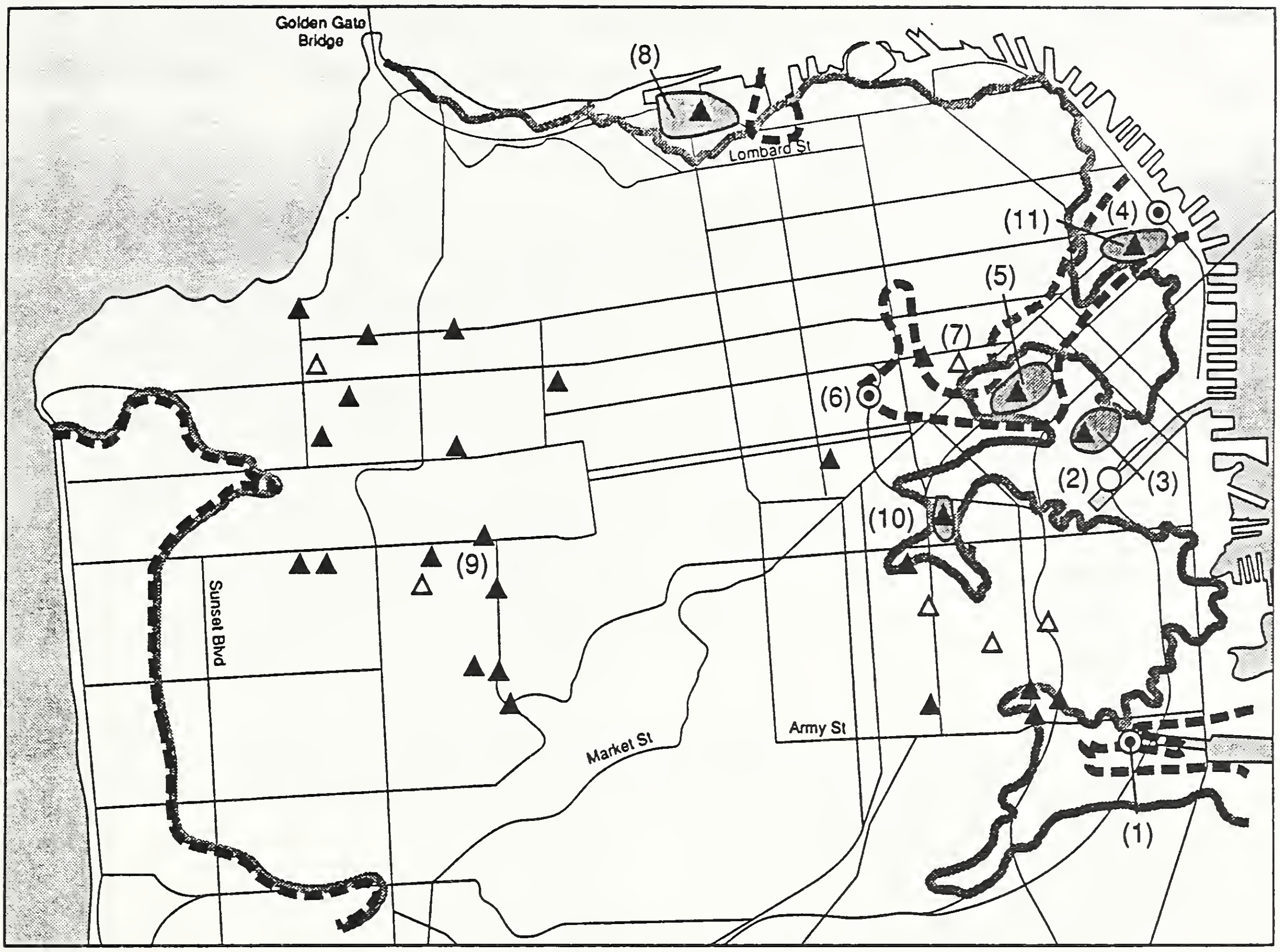


Figure 3



Outer limit of bay mud deposits

Coutour of 60m (200 ft) depth of unconsolidated deposits

Area of damage concentration

Major damage to highway structures

Minor damage to highway structures

Major damage to buildings

Minor damage to buildings

Figure 4

Correlation of Loma Prieta damage in San Francisco and areas with filled or poor soil

(From NIST Publication 778)

BUILDING IDENTIFICATION		
Address: ______________________		Location in Block:
Block: ______	Lot/Suffex: ______	Street: ______
Date: ______	Time: ______	Street: ______

DAMAGE DESCRIPTION				
Description	Front Face	Left Face	Right Face	Rear
Falling from Building				
Wall not Visible	☐	☐	☐	☐
No Falling Elements	☐	☐	☐	☐
Individual Units or Trim	☐	☐	☐	☐
Veneer (Sheet - Type Failure) or Delamination	☐	☐	☐	☐
Parapet	☐	☐	☐	☐
Portion of Wall	☐	☐	☐	☐
Entire Wall	☐	☐	☐	☐
Masonry Cracking				
No Cracking Visible	☐	☐	☐	☐
At Corner of Opening(s)	☐	☐	☐	☐
X Cracking of Spandrel	☐	☐	☐	☐
Vertical Cracking at Edge of Spandrel	☐	☐	☐	☐
Piers or Walls (X or Stepped Cracking)	☐	☐	☐	☐
Horizontal at Top/Bottom of Pier	☐	☐	☐	☐
Corner Distress - 1st Level	☐	☐	☐	☐
Corner Distress - Above 1st Level	☐	☐	☐	☐

Other Damage	Comments:
Damage from Debris from Adjacent Building ☐	
Roof or Floor Failure Due to Movement of Exterior Wall ☐	

OVERALL DAMAGE ESTIMATE (Estimate Using Both Scales)

ATC-13 Scale Percent of Replacement Cost		
None	0 %	☐
Slight	0 - 1 %	☐
Light	1 - 10 %	☐
Moderate	10 - 30 %	☐
Heavy	30 - 60 %	☐
Major	60 - 100%	☐
Destroyed	100 %	☐

Wailes and Horner Scale		
A	Undamaged or Minor Cracking No Significant Structural Damage	☐
B	Parapet Failure or Separation of Veneer; Major Cracking and Interior Damage	☐
C	Failure of Portion of Exterior Wall. Major Damage to Less Than 50% of Walls	☐
D	Major Damage to More Than 50% of Walls	☐
E	Unreparable Damage. Demolition Probably Appropriate	☐

Evidence of Pre-Earthquake Seismic Strengthening (X Bracing, etc.; Do <u>Not</u> Include Parapet Bracing)	Yes ☐	No ☐

Figure 5

City of San Francisco
UMB Supplementary Damage Collection Form

(Developed jointly by Rutherford & Chekene and the City)

Table 1

Types of Damage to 1935 URM Buildings in San Francisco

TYPE OF DAMAGE	NUMBER OF BUILDINGS
Falling Objects	
Individual Units or Trim	103
Veneer or Delamination Failure	101
Parapet Failure	99
Portion of Wall Failure	61
Entire Wall Failure	36
Cracking	
Corner of Openings	253
"X" Cracking of Spandrel Edges	125
Vertical Cracks at Spandrel Edges	181
"X" Cracks in Piers or Wall	204
Horizontal Cracks at Top/Bottom of Pier	202
Corner Distress at First Level	167
Corner Distress Above First Level	170
Other	
Damage from Debris from Adjacent Building	7
Roof or Floor Failure Due to Movement of Exterior Walls	13

Table 2

Levels of Damage to 1970 URM Buildings in San Francisco

Damage Class	Central Damage Ratio	Number Of Bldgs	Damage Weighted by Bldg	Square Footage	Damage Weighted by Sq.Ft.
None	0.000	1238	0.0	20,388,484	0
Slight	0.005	406	2.0	7,866,706	39,334
Light	0.055	187	10.3	3,440,805	189,244
Moderate	0.200	103	20.6	2,589,129	517,826
Heavy	0.450	19	8.6	563,362	253,513
Severe	0.600	17	10.2	610,828	366,497
Totals		1970	51.7	35,459,314	1,366,413
Total Percent of Damage			2.62		3.85

Source: Rutherford & Chekene Draft Technical Services Report 2/23/90

CURRENT SYSTEM FOR INSPECTION/EVALUATION/RESTORATION OF EARTHQUAKE DAMAGED BUILDINGS AND ITS BACKGROUND AND FUTURE ISSUES

by

Masamichi Ohkubo[1]

ABSTRACT

If a building suffered an earthquake-damage, the safety/danger of the building is investigated and the judgment for either repairing or demolishing the building is taken action on the basis of the result investigated. The judgment often has been controlled by the criteria of the persons themselves who conducted the damage investigation. Many countries including Japan have recently developed the system to inspect and evaluate the damage degree under an uniform specification after an earthquake toward getting more correct and speedy results.

The outlines of the "Guidelines for Post-Earthquake Inspection, Evaluation and Restoration of Damaged Reinforced Concrete Buildings - popular edition", which was established in 1989 in Japan, is presented. The historical circumstance to the development of the Guidelines is also reviewed and the issues for an actual application of the Guidelines in future are discussed.

KEYWORDS: Damage degree evaluation; Earthquake damage; Guidelines; Reinforced concrete buildings; Restoration.

1. INTRODUCTION

A large number of masonry buildings used brick suffered severe damage in 1923 Kanto Earthquake in Japan. At that time, there were also a lot of reinforced concrete buildings which were designed mainly under the gravity load. However, the damage is not so severe comparing with the masonry brick buildings. Since the Kanto Earthquake, masonry brick building has disappeared, and reinforced concrete building and steel building have come into wide use. After the earthquake the then Japanese Building Code added the seismic design requirements using the 10 percent load of the building weight as a lateral seismic design load. After the 2nd War, the seismic design load was increased to the 20 percent concerning with the increase of allowable material strength.

A large number of reinforced concrete buildings designed by the Seismic Requirements have been constructed with the economic growth after the 2nd War, and the level of seismic capacity of buildings has become higher receiving the benefit of an advance in earthquake engineering. In particular, it seems that the buildings constructed after 1980, when the Seismic Design Requirements were revised extensively, have quite enough seismic capacity. However, the recent experience of earthquake damage and the current knowledge on earthquake engineering suggest us that a part of the existing reinforced concrete buildings has not enough seismic capacity.

Table-1 presents the circumstance on the earthquake damage experiences and the earthquake engineering development for the last some decades. The Nigata Earthquake in 1964 featured as the damage by liquefaction of sand soil. Many reinforced concrete buildings without piles inclined or settled, but the building did not have not any structural damage by vibration.

In 1968 Tokachi-oki Earthquake, several reinforced concrete buildings suffered very severe damage, so that people including professional on the seismic engineering were shocked by the damage and worried about the seismic performance of reinforced concrete buildings. Since then, the importance to reconsider the then Seismic Design Requirements and to develop the new seismic design system, as well as the importance to evaluate the seismic capacity of existing buildings have been strongly recognized. The intensive researches to develop the new seismic design system were commenced as the First Synthetic Research Projects promoted by the Building Research Institute, Ministry of Construction in 1972. The research results were reflected for the development of new seismic design system in 1980, and also reflected for the development of seismic performance evaluation technique of existing reinforced concrete buildings.

The 1978 Miyagiken-oki Earthquake attacked Sendai-city, one of the big cities, and quite a few reinforced concrete buildings again suffered damage. The experience of earthquake pointed out us the necessity for the development of post-earthquake inspection technique which was available for the rapid damage evaluation and restoration. The Second Synthetic Research Projects was commenced in 1981. The Guidelines for Post-Earthquake Inspection and Restoration Techniques was developed on the basis of the research results in 1985.

The experience on the damage evaluation and the restoration of Namioka Town Hospital by the Nihonkai-chubu Earthquake in 1983 very served

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the development of the post-earthquake evaluation and restoration system during the research projects. The developed system was also made to apply to the damage evaluation of the 1985 Mexican Earthquake.

The system developed has been known only for the related researchers or official after that, but it has not come widely known for the public. It was revised as the popular edition of the guidelines for the general engineers or public in 1989, and the Japan Building Disaster Prevention Association is scheduled to publish it in 1990. (Ref. 1)

This report presents the outlines of the guidelines revised, and its issues for the wide application in future are discussed.

2. PURPOSE OF DAMAGE EVALUATION AND THE PAST DAMAGE CLASSIFICATION

The damage evaluation after an earthquake has basically two purposes ;
(1) to evaluate the safety or danger of a building immediately after a damaging earthquake, and to prevent the secondary damage, in particular for human life, by possible aftershocks, and
(2) to evaluate the remaining seismic performance of a building damaged, and to decide whether any structural strengthening to reuse the building should be required or not.

There has never been any evaluation system applying immediately after an earthquake. The damage classification has ever been conducted after an earthquake, but there was not any clear criteria to evaluate the damage.

The Report of Damage Investigation at the Kanto Earthquake in 1923 shows the category of five damage degrees used to be ; collapse, partial collapse, severe, small and none, with some remarks that "severe" has considerably conspicuous damage on the main structure and "small" has visible cracks on the exterior walls or the main structure. (Ref. 2)

In 1964 Nigata Earthquake which featured on liquefaction of sand, two different damage classification were used. The Architectural Institute of Japan, (AIJ), adopted the methodology in which the damage was classified under the failure pattern. (Ref. 3) The Building Research Institute, (BRI), adopted the different method from the AIJ for damage degree classification, in which both damage for soil including foundation and for structure were inspected independently and the final damage degree was identified using its matrix. (Ref. 4)

In 1968 Nigata Earthquake, the AIJ used the category of five damage degrees; collapse, severe, medium, small and none, and they have thefollowing brief comments. "Collapse" should be recommended to demolish, "severe" to strengthen or demolish, "medium" to repair or strengthen, and "small" to repair. (Ref. 5)

In the 1978 Miyagiken-oki Earthquake, the AI adopted the similar damage degree classification as the 1968 Tokachi-oki Earthquake adding the failure patterns illustrated. (Ref. 6)

The past evaluation methods for damage have been focussed on the damage classification, and they were on the sensuous evaluation by the inspector. The damage of the buildings investigated in the early stage spent a long inspection-time, and also the rather severe result was given. But one investigated in the later stage spent sometimes a short time, and the rather slight result was given by the way practiced too much in an earthquake.

The past evaluation methods also were not quantitative, but only ranking. The method which we aim in future should be a successive and a numerical evaluation related the seismic performance remained after an earthquake. The method to evaluate the damage, developed in Japan recently, is a checklist type with many inspection items. The result is represented by a percentage at first, and is identified finally into a corresponding damage degree.

3. THE OUTLINE OF THE GUIDELINES FOR POST-EARTHQUAKE DAMAGE EVALUATION

3.1 The Relation Between The Guidelines and The Other System Concerned

The technique to evaluate the post-earthquake performance of a building and the basic policy of the restoration by the Guidelines for post-earthquake evaluation is essentially based on the same concept as designing a new building. Table-2 shows the relation between the various guidelines adapted and the scope for each evaluation. The guidelines for post-earthquake evaluation is related to many requirements, and the building which is strengthened in the restoration is required the same level of seismic performance as a new building required by the seismic code.

3.2 The General Flow of Post-earthquake Evaluation and Restoration

The guidelines consist of (1) the techniques of inspection and evaluation for the buildings damaged, and (2) the techniques of restoration for them. The general flow from the beginning inspection to the completing restoration is illustrated in Fig. 1.

4. EMERGENCY EVALUATION AND RESTORATION

4.1 Emergency inspection

The emergency inspection is adapted immediately after a damaging earthquake. The purpose is on the mitigation or prevention of secondary

damage by after-shocks. The task of emergency inspection is as follows ;
(1) to inspect possible falling or overturning objects and the possible collapse of a building damaged, and to post the necessary actions such as caution, off-limit, etc., and
(2) to inspect the safety or adaptability of the public buildings used as evacuation places for homeless people.

It is aimed that the emergency inspection and evaluation should be completed for a couple of days after a damaging earthquake, so that one team consisting two persons who may be structural engineers, but ordinary building or civil engineers of the local government, might finish the inspection for approximately one hour or less using the prescribed form. See Table-3.

4.2 Inspection Items

The inclination and settlement of a building and the damage of structure should be inspected, and the possible falling or overturning risk of finishing materials, building equipments or sub-structures such as chimneys, fence, etc. also should be inspected. The ordinary building is looked for the outside of a building, but the public building for an evacuation place must be looked for both of the outside and inside. The results of each item inspected are ranked to three categories, A, B, or C in order of small to severe damage. The directions for a building is recommended finally depending upon the number of rank. (See 4.4)

4.3 Evaluation

4.3.1 Overall settlement of a building

The maximum settlement in either corner of a building is defined as the overall settlement, if the settlement due to soil deformation, liquefaction or failure of piles is observed. The damage rank is identified using Table-4.

4.3.2 Overall inclination of a building

The maximum inclination angle on either exterior wall of a building is defined as the overall inclination, if it is due to soil or foundation deformation. The damage rank is also identified using Table-4.

4.3.3 Damage of building structure

The damage rank of the structure is decided directing attention to the failure of columns in case of frame structure, but the bearing walls in case of wall-type structure. The damage levels of each column or bearing wall are evaluated at first using Table-5, and the final structural damage rank is classified by Table-4 depending upon the occupying-percentage of the members corresponding to damage level IV and/or V. The members of damage level III also should be checked, because the occupancy of such a member in a building should be reflected on the final evaluation.

4.3.4 Possible falling or overturning objects

The finishing materials attached to the exterior walls such as window glass, mortar, tile, stone, etc., the building equipments such as water tank, cooling tower, transformer, and the sub-structures such as signboards or advertising pillars, eaves, parapets, chimneys, etc. are evaluated using Table-4. The possible overturning objects in the outside of building, such as an exterior staircase, a concrete-block fence, gates, a vending machine, etc., are also evaluated using Table-4. The objects nearby the building entrance must be carefully checked particularly, but the objects fallen or overturned already is not dangerous.

4.4 Decision of Risk Level and Emergency Actions

The final risk level of a building is decided as follows, and the appropriate treatment for the building is recommended as well ;

<DANGER> : If there is more than one Rank "C", or two Rank "B", in either of the overall settlement, the overall inclination, the damage of structure, and the damage of falling or overturning objects, the entire building should be identified as "DANGER". The entrance into the building and the approach to the building should be basically prohibited.

<CAUTION> : If there is more than one Rank "B" in either of the inspection items, or if there is any structural member of the Damage Degree III in a building, the building should be identified as "CAUTION". The warning against entrance or approach should be recommended to the building identified as "CAUTION".

<SAFETY> : If the result inspected is not applicable to both of the above, the building may be identified as "SAFETY". The public building identified as "SAFETY" may be used as an evacuation place.

4.5 Emergency Restoration

The emergency restoration should be applied to the buildings identified as "DANGER" or "CAUTION". The restoration is basically a temporary one, but the reliable technique shall be applied. The technique being able to add the bearing capacity for gravity load should be mainly required, but sometimes the technique being able to increase the capacity for lateral loads also may be required as well.

There are some examples used steel wire-rope or steel strip in the emergency restoration for The Namioka Town Hospital in 1983 (Ref.7), but the available researches for emergency restoration are quite few. (Ref. 8)

5. PERMANENT EVALUATION

5.1 The Guidelines for Evaluation of Permanent Treatment

The damage classification guidelines is applied to judge the permanent treatment of a building as shown in Fig. 1. The inspection may be begun after the confusion due to an earthquake cooled off, or after the after-shocks become rather inactive. The Inspection and evaluation should be conducted by the structural engineers using the prescribed form. See Table-6.

The inspection items are the overall settlement, the overall inclination of a building, and the damage of structural members such as columns and shear walls. If the damage of beams is severe than the columns, the beams should be inspected, because the damage degree evaluation of a building is executed by using the damage degree of the members damaged more severely.

The result evaluated is classified into either category of NONE, SLIGHT, SMALL, MEDIUM, SEVERE, or COLLAPSE.

5.2 Damage Evaluation for Overall Settlement

The evaluation for the overall settlement of a building should be executed as follows, using the maximum settlement, S (meter).

NONE :	$S = 0$
SMALL :	$0.0 < S \leq 0.2$
MEDIUM :	$0.2 < S \leq 1.0$
SEVERE :	$1.0 < S$

5.3 Damage Evaluation for Overall Inclination

The evaluation for the overall inclination of a building should be executed as follows, using the maximum inclination angle, θ (radian).

NONE :	$\theta = 0$
SMALL :	$0 < \theta \leq 1/100$
MEDIUM :	$1/100 < \theta \leq 3/100$
SEVERE :	$3/100 < \theta \leq 6/100$
OVERTURNED :	$6/100 < \theta$

5.4 Damage Evaluation for Structure

The damage evaluation for an entire building should be executed using the maximum value of the sum of "D_i" which is called the "DAMAGE RATIO OF STRUCTURE". The value "D_i" is calculated by Equation (1).

NONE :	$\Sigma D_i = 0$
SLIGHT :	$0 < \Sigma D_i \leq 5$
SMALL :	$5 < \Sigma D_i \leq 10$
MEDIUM :	$10 < \Sigma D_i \leq 50$
SEVERE :	$50 < \Sigma D_i$
COLLAPSE :	$\Sigma D_5 = 50$

$$\left.\begin{array}{ll} D_1 = 10*B_1/A & \text{(not more than 5)} \\ D_2 = 26*B_2/A & \text{(not more than 13)} \\ D_3 = 60*B_3/A & \text{(not more than 30)} \\ D_4 = 100*B_4/A & \text{(not more than 50)} \\ D_5 = 1000*B_5/7A & \text{(not more than 50)} \end{array}\right\} \text{Eq. 1}$$

where, A is the total number of columns inspected, and B_i is the number of columns corresponding to each "DAMAGE LEVEL" shown in Table-5, respectively. In case of the wall-type structure, A and B_i should be replaced into the length of wall.

5.5 Judgment on Permanent Treatment (Strenghtening or Not)

The category of damage degree for an entire building is identified to the either maximum one in the overall settlement, the overall inclination, and the damage of structure.

The judgment on the permanent treatments after evaluation should be executed using Table-7. The term of repairing represented by "RP" in Table-7 is defined as that the structural performance of the structure should be recovered to the state before the earthquake damaged. The term of strengthening "ST" is defined as that the member damaged should be repaired and the seismic performance of the building should be rehabilitated and increased to the level which required by the Japanese Uniform Building Code as well. The advanced evaluation technique is applied, if the judgment is uncertain, as shown by the mark-"UN" in the Table-7.

6. AN ADVANCED EVALUATION TECHNIQUE FOR THE DAMAGE DEGREE OF STRUCTURE

If the judgment about the requiring any structural strengthening or not is uncertain, the technique as shown in Table-8 may be applied.

The damage degree Index Φ as shown in Table-8 is calculated by Equation (2).

$$\Phi = (1 - I_s. / I_s) \times 100 \quad (\%) \qquad \text{Eq. 2}$$

where, Index I_s represents the seismic capacity of a building estimated with the state before damage, using "The Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings", (Ref. 9). Index I_s. represents the remaining seismic capacity of the building after damage. Index I_s. is also estimated by the above standard, using the capacity reduction factors as shown in Table-9 for the members damaged.

The back data for the judgment used Eq. 2 and Table-8 is shown in Fig. 2. (Ref. 10) The buildings with the index of more than approximately 50% have been strengthened by some techniques or demolished in the past earthquakes.

7. PERMANENT RESTORATION FOR STRUCTURE

The building identified as "REPAIRING" in Table-7 or Table-8 may be repaired without any strengthening. The building identified as "STRENGTHENING" should be repaired or strengthened. The prescribed seismic performance should be required for the strengthening of such buildings, and it is basically same performance as a new building designed by the current Japanese Uniform Building Code.

The retrofitting techniques for the existing buildings, such as in-filling shear walls, steel braces, jacketing by steel plate or concrete with welded wire-fabric, and also injecting epoxy resin into the cracks, are available for repairing or strengthening the building damaged.

8. CONCLUDING REMARKS

Followings, as some future issues, should be discussed among the related administration, official, researchers, or engineering professional to make better application of the evaluation system for post-earthquake.

(1) The damage degree evaluation techniques have been partially applied at the 1985 Mexican Earthquake on a kind of research-viewpoints. The system, however, has never yet been applied to the damage by a big earthquake which attacked a large city, so that the establishment of application system, in which the voluntary structural engineers may be expected particularly for emergency evaluation after an earthquake, should be urged. A kind of special legislation may be required for more effective application, because the application of the system to the private buildings may be very difficult.

(2) The brief self-evaluation technique by the owner or resident of the building may be effective for the emergency inspection after an earthquake. The Shizuoka Prefecture has studied the establishment of such a system in 1989. It is considered that the owner or resident should report the results written in the questionnaire form to the local government after an earthquake and they should wait the instructions or recommendation from the local government. See Table-10. (Ref.11)

(3) The education about the seismic weakness or failure patterns of buildings may be required to the general public. The campaign as how to do immediately after a damaging earthquake or the seminar as how to check the safety/danger of a building may also be required to the general public. The Emergency Services Organization of the Unified San Diego County distributes the EARTHQUAKE SAFETY CHECKLIST at the some general public places. (Ref.12) Such kind of instructions should be spread more widely in the earthquake prone areas.

(4) The improvement of insurance system for repairing the buildings damaged should be required. The current insurance system is valid only for the residential buildings. The system should be extended to the other kind of buildings in future. The possible system for earthquake subsidy also should be proposed by the governors.

9. REFERENCES

(1)"GUIDELINES FOR POST-EARTHQUAKE INSPECTION, EVALUATION AND RESTORATION OF DAMAGED REINFORCED CONCRETE BUILDINGS - popular edition", Japan Building Disaster Prevention Association. 1990. (Japanese)

(2)"THE REPORT OF DAMAGE IN THE 1923 KANTO EARTHQUAKE", The Civil Engineering Institute of Japan, reprinted in 1984. (Japanese)

(3)"THE REPORT OF DAMAGE IN THE 1964 NIIGATA EARTHQUAKE", The A.I.J., 1964. (Japanese)

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(11)"THE REPORT ON DEVELOPMENT OF THE MANUAL FOR EVALUATING THE RISK OF DAMAGED BUILDINGS", Japan Building Disaster Prevention Association, 1989. (Japanese)

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ACKNOWLEDGMENT

The work to revise the Guidelines for Post-earthquake Inspection, Evaluation and Restoration was done by the Sub-committee chaired by author on Reinforced Concrete Buildings arranged temporally in the Japan Building Disaster Prevention Association. The members of the sub-committee are T. Okada, M, Murakami, M. Hirosawa, T. Kaminosono, K. Takahara, M. Yoshimura, and author.

Table-1 : THE EARTHQUAKE DAMAGE FOR THE LAST SOME DECADES AND THE SEISMIC ENGINEERING DEVELOPED

Year	Earthquakes	Seismic engineering developed
1964	Niigata Earthquake (M 7.5); Damage by liquefaction (settlement and inclination of buildings).	
1968	Tokachi-oki Earthquake (M 7.9); Shear failure of reinforced concrete columns.	
1970		Revision of the Japanese Uniform Building Code (increasing shear reinforcement ratio to R/C columns).
1972		Beginning The Synthetic Research Projects for development of new seismic design methodology.
1977		Development of The Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings and The Guidelines for Retrofit of Existing R/C Buildings.
1978	Miyagiken-oki Earthquake (M 7.4); Quite a few R/C buildings suffered damage.	
1980		Extensive revision of the Japanese Uniform Building Code (New Seismic Design Methodology).
1981		Beginning The Synthetic Research Projects for development of techniques on damage evaluation and restoration to the buildings damaged.
1983	Nihonkai-chubu Earthquake (M 7.7); Damage of Namioka Town Hospital.	
1985	Mexican Earthquake (M 8.1); Applied to the tentative guidelines for post-earthquake evaluation.	Development of The Guidelines for Post-earthquake Inspection and Restoration Techniques to Damaged Buildings.
1988	Armenia Earthquake (M 7.0); Many precast concrete buildings suffered severe damage.	Development of The Earthquake Resistant Design Guidelines for R/C Buildings Based on Ultimate Strength Concept.
1989	San Francisco Earthquake (M 7.1); Damage of free way's R/C frames.	Revision of the 1977 Standard for Evaluation of Seismic Capacity of Existing R/C Buildings and the 1977 Guidelines for Retrofit of Existing R/C buildings. Publishing the popular edition of the Guidelines for Post-earthquake Inspection and Restoration of R/C damaged Buildings.

Table-2 : THE CURRENT VARIOUS STRUCTURAL REQUIREMENTS vs. EACH SCOPE

		Current Code/Guidelines, etc.						
		A	B	C	D	E	F	G
Existing Buildings	Evaluating Seismic Performance	*						
	Retrofitting Design	*	*			*		
Earthquake Damaged Buildings	Evaluating Seismic Performance Remained	*		*				
	Restoration Design	*	*		*	*		
Design for New Buildings Which Will be Constructed From Now						*	*	*

Note)
A : "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings", revised in 1990. Japan Disaster Prevention Association.
B : "Guidelines for Seismic Retrofitting of Existing Reinforced Concrete Buildings", revised in 1990. Japan Disaster Prevention Association.
C, D : "Guidelines for Post-Earthquake Inspection, Evaluation and Restoration of Damaged Reinforced Concrete Buildings - popular edition", Japan Disaster Prevention Association. 1990.
E : Japanese Uniform Building Code (required seismic performance)
F : "Guidelines of Structural Design for Buildings", Japan Building Center.
G : "Recommendation for Structural Analysis of Reinforced Concrete Structures", Architectural Institute of Japan. 1988.

Table-3 : THE FORM FOR POST-EARTHQUAKE EMERGENCY INSPECTION (FOR REINFORCED CONCRETE BUILDINGS)

FORM FOR POST-EARTHQUAKE EMERGENCY INSPECTION
<FOR REINFORCED CONCRETE BUILDINGS>

<u>TIME AND DATE OF INSPECTION</u>

TIME : ☐a.m.☐p.m.________ DAY ______ MON ______ YEAR ________

<u>NAME OF INSPECTOR</u> : ______________________________

<u>1. DESCRIPTION OF BUILDING INSPECTED</u>

1.1 NAME : ______________________________

1.2 LOCATION : ______________________________

1.3 OWNER : ______________________ TEL ____________

1.4 CONTACT PERSON : ______________________ TEL ____________

1.5 USE :

☐PRIVATE USE : ☐RESIDENCE ☐APARTMENT ☐OFFICE ☐STORE
☐WAREHOUSE ☐FACTORY
☐OTHER ______________________

☐PUBLIC USE : ☐SCHOOL ☐NURSERY ☐CITY HALL ☐PUBLIC HALL
☐POLICE ST ☐FIRE ST ☐HOSPITAL ☐GYMNASIUM
☐ASSEMBLY HALL ☐BROADCAST ST
☐OTHERS ______________________

1.6 NUMBER OF FLOORS : ABOVE THE GROUND ______________
BASEMENT __________ PENT HOUSE __________

1.7 STRUCTURAL SYSTEM : ☐ MOMENT RESISTING FRAME ☐ FLAT SLAB
☐ WALL (BOX) TYPE STRUCTURE

1.8 CLADDING : ☐ MORTAR ☐ TILE ☐ CURTAIN WALL ☐ BRICK
☐ SHEET METAL ☐ NONE ☐ OTHERS ______________

<u>2. INSPECTION</u>

<u>2.1 INSPECTION FOR STRUCTURES</u>

<u>(EXTERIOR DAMAGE)</u>

	DAMAGE RANK (A)	(B)	(C)
* OVERALL INCLINATION (Deg)	☐<1	☐ 1 - 2	☐>2
* OVERALL SETTLEMENT (m)	☐<0.2	☐ 0.2 - 1	☐>1.0

* DAMAGE TO STRUCTURAL MEMBERS AT THE FLOOR NO. : __________

(a) IN CASE OF FRAME STRUCTURE / FLAT SLAB
TOTAL NUMBER OF EXTERIOR COLUMNS : ______________
NUMBER / RATIO OF EXTERIOR COLUMNS TO DAMAGE RANK :
RANK-IV ________ / ☐<10 % ☐ 10 - 20 % ☐>20 %
RANK-V ________ / ☐<1 % ☐ 1 - 10 % ☐>10 %

(b) IN CASE OF WALL (BOX) TYPE STRUCTURES
TOTAL LENGTH OF EXTERIOR WALLS : ______________ (m)
LENGTH / RATIO OF EXTERIOR WALLS TO DAMAGE RANK :
RANK-IV ________m / ☐<10 % ☐ 10 - 20 % ☐>20 %
RANK-V ________m / ☐<1 % ☐ 1 - 10 % ☐>10 %

<u>(INTERIOR DAMAGE)</u>

(a) IN CASE OF FRAME STRUCTURE / FLAT SLAB
TOTAL NUMBER OF EXTERIOR COLUMNS : ______________
NUMBER / RATIO OF EXTERIOR COLUMNS TO DAMAGE RANK :
RANK-IV ________ / ☐<10 % ☐ 10 - 20 % ☐>20 %
RANK-V ________ / ☐<1 % ☐ 1 - 10 % ☐>10 %

(b) IN CASE OF WALL (BOX) TYPE STRUCTURES
TOTAL LENGTH OF EXTERIOR WALLS : ______________ (m)
LENGTH / RATIO OF EXTERIOR WALLS TO DAMAGE RANK :
RANK-IV ________m / ☐<10 % ☐ 10 - 20 % ☐>20 %
RANK-V ________m / ☐<1 % ☐ 1 - 10 % ☐>10 %

<u>2.2 POSSIBLE RISK OF OVERTURNING OR FALLING OBJECTS</u>

	RISK LEVEL (a) SAFETY	(b) CAUTION	(c) DANGER
<u>(EXTERIOR)</u>			
* WINDOW GLASS	☐	☐	☐
* SIGN-TOWER	☐	☐	☐
* SIGNBOARD	☐	☐	☐
* STAIRCASE	☐	☐	☐
* CLADDING	☐	☐	☐
* EAVES	☐	☐	☐
* BALCONY	☐	☐	☐
* PARAPET	☐	☐	☐
* ELEVATED TANK	☐	☐	☐
* COOLING TOWER	☐	☐	☐
* CHIMNEY	☐	☐	☐
* PENTHOUSE	☐	☐	☐
* OTHERS : ______________	☐	☐	☐
<u>(INTERIOR)</u>			
* CEILING	☐	☐	☐
* LIGHTING APPARATUS	☐	☐	☐
*OTHERS : ______________	☐	☐	☐

<u>3. RESULTS OF INSPECTION</u>

* TOTAL NUMBER OF RISK LEVEL-(c) : ________
* TOTAL NUMBER OF RISK LEVEL-(b) : ________

<u>4. DECISION AND SUGGESTION</u>

	BUILDING	BUILDING-SURROUNDINGS
* SAFETY	☐	☐
* CAUTION (ENTRY LIMITED)	☐	☐
* DANGER (OFF LIMITS)	☐	☐

Table-4 : EVALUATION OF DAMAGE RANK FOR EMERGENCY INSPECTION

Inspection Items		Damage Rank		
		A	B	C
<OVERALL SETTLEMENT>				
Maximum settlement, S (meter)		$S \leqq 0.2$	$0.2 < S \leqq 1.0$	$1.0 < S$
<OVERALL INCLINATION>				
Maximum inclination, θ (rad.)		$\theta \leqq 1/60$	$1/60 < \theta \leqq 1/30$	$1/30 < \theta$
<STRUCTURAL DAMAGE>				
Occupation ratio, D(%)	The member of Damage level-IV	$D \leqq 10$	$10 < D \leqq 20$	$20 < D$
	The member of Damage level-V	$D \leqq 1.0$	$1.0 < D \leqq 10$	$10 < D$
<DAMAGE OF FALLING & OVER-TURNING OBJECTS>				
Damage state observed		none	uncertain	dangerous

Table-5 : CLASSIFICATION CRITERIA FOR THE MEMBER DAMAGED

Damage level	Damage state observed on the structural members
I	Visible narrow crack of concrete surface, (crack-width ; smaller than 0.2 m/m)
II	Visible clear crack of concrete surface, (crack-width ; approximately 0.2 - 1.0 m/m)
III	Local crush of covered concrete and/or considerably wide crack, (crack-width ; approximately 1.0 - 2.0 m/m)
IV	Remarkable crush of concrete with exposing re-bars and/or widely spalling of covered concrete
V	Flexure of rebars, crush of core concrete, visible vertical deformation of column or wall, and/or visible settlement or inclination of floor

Table-6: THE FORM FOR DAMAGE DEGREE CLASSIFICATION
<FOR REINFORCED CONCRETE BUILDINGS>

TIME AND DATE OF INSPECTION

TIME : [ ]a.m. [ ]p.m. ________ MONTH_____ DAY_____ YEAR__________

1. DESCRIPTION ON BUILDING INSPECTED

1.1 BUILDING NAME : ______________________________

1.2 LOCATION : ______________________________

1.3 OWNER : ____________________ TEL : __________

1.4 CONTACT PERSON ____________________ TEL : __________

1.5 USE :

[ ]PRIVATE USE : [ ]RESIDENCE [ ]APARTMENT [ ]OFFICE [ ]STORE
[ ]WAREHOUSE [ ]FACTORY

[ ]PUBLIC USE : [ ]SCHOOL [ ]NURSERY [ ]CITY HALL [ ]PUBLIC HALL
[ ]POLICE ST [ ]FIRE ST [ ]HOSPITAL [ ]GYMNASIUM
[ ]ASSEMBLY HALL [ ]BROADCAST ST
[ ]OTHERS ______________________

1.6 NUMBER OF FLOORS : ABOVE THE GROUND ______________________
BASEMENT __________ PENTHOUSE __________

1.7 STRUCTURAL SYSTEM : [ ]MOMENT RESISTING FRAME [ ]FLAT SLAB
[ ]WALL (BOX) TYPE STRUCTURE

1.8 FOUNDATION SYSTEM : [ ]WITH PILES [ ]WITHOUT PILES

1.9 CIRCUMSTANCE AT SITE : [ ]FLAT [ ]SLOPE [ ]HEIGHTS [ ]DEPRESSION
[ ]TOP OF PRECIPICE [ ]BOTTOM OF PRECIPICE
[ ]SEASIDE [ ]LAKESIDE

1.10 CLADDING : MORTAR [ ]TILE [ ]CURTAIN WALL [ ]BRICK [ ]SHEET METAL
[ ]NONE [ ]OTHERS ______________________

1.11 DOCUMENTS ON DESIGN OR CONSTRUCTION : [ ]PRESERVED [ ]NONE

2. EVALUATION OF OVERALL SETTLEMENT, S : MAXIMUM SETTLEMENT (METER)
[ ]NONE($S=0$) [ ]SMALL($S \leq 0.2$) [ ]MEDIUM($0.2 < S \leq 1.0$) [ ]SEVERE($S < 1.0$)

3. EVALUATION OF OVERALL INCLINATION, θ : MAXIMUM INCLINATION (RADIAN)
[ ]NONE($\theta=0$) [ ]SMALL($\theta \leq 1/100$) [ ]MEDIUM($1/100 < \theta \leq 3/100$)
[ ]SEVERE($3/100 < \theta \leq 6/100$) [ ]OVERTURNED($\theta < 6/100$)

4. EVALUATION OF STRUCTURAL DAMAGE ; the result on the story at where the most severe damage was observed shall be represented here.

4.1 THE STORY EVALUATED : ____/____ , DIRECTION OF FRAME : [ ]X or [ ]Y

4.2 TOTAL NUMBER (LENGTH) OF COLUMNS (WALLS), A_0 : ______

4.3 TOTAL NUMBER (LENGTH) OF COLUMNS (WALLS) INSPECTED, A : ______

4.4 INSPECTED RATIO, A/A_0 : ________

4.5 THE NUMBER (LENGTH) OF COLUMNS (WALLS) WITH EACH DAMAGE DEGREE :

DAMAGE DEGREE	0	I	II	III	IV	V
NUMBER (LENGTH)	______	______	______	______	______	______

4.6 CALCULATION FOR DAMAGE RATIOS OF STRUCTURE, D_I AND THE SUM :

DAMAGE LEVEL I : $D_1 = 10B_1/A =$ ______ (not greater than 5)
DAMAGE LEVEL II : $D_2 = 26B_2/A =$ ______ (not greater than 13)
DAMAGE LEVEL III : $D_3 = 60B_3/A =$ ______ (not greater than 30)
DAMAGE LEVEL IV : $D_4 = 100B_4/A =$ ______ (not greater than 50)
DAMAGE LEVEL V : $D_5 = 1000B_5/7A =$ ______ (not greater than 50)
THE SUM OF D_I, $\sum D = \sum D_1 \sim D_5 =$ __________

5. IDENTIFICATION OF DAMAGE DEGREE FOR THE ENTIRE BUILDING ;
[ ]NONE [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE

6. DAMAGE OF SUB-STRUCTURES ;

PENTHOUSE : [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE
EXTERIOR STAIRCASE : [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE
CHIMNEY : [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE
EXTERIOR PASSAGE : [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE
EXPANSION JOINTS : [ ]SLIGHT [ ]SMALL [ ]MEDIUM [ ]SEVERE [ ]COLLAPSE
THE OTHERS : ______________________

7. DAMAGE OF FOUNDATION ;

PILES : [ ]DAMAGED [ ]NOT DAMAGED [ ]UNCERTAIN
LIQUEFACTION : [ ]OCCURRED [ ]NOT OCCURRED [ ]UNCERTAIN

8. REMARKS OR MEMO ; ______________________
____
____

Table-7 : EVALUATION CRITERIA FOR PERMANENT TREATMENT

Seismic intensity by J. M. A.	Identified damage category				
	SLIGHT	SMALL	MEDIUM	SEVERE	COLLAPSE
Smaller than V	RP	UN	ST/DM	ST/DM	ST/DM
V	RP	RP	UN	ST/DM	ST/DM
Bigger than V	RP	RP	RP	UN	ST/DM

Note) J. M. A. : Japanese Meteorological Agency. RP : Repairing. UN : Uncertain, the application of advanced evaluation technique should be required, see 6. ST/DM : Strengthening or demolishing.

Table-8 : LOWER LIMIT OF INDEX- Φ FOR STRENGTHENING
- Advanced Evaluation Technique -

The year constructed	Seismic intensity by J. M. A.			
	Smaller than IV	Lower V	Upper V	Greater than VI
Before 1971	20 %	30 %	40 %	50 %
After 1971	30 %	40 %	50%	50 %

Note); If the result estimated is more than the percentage in above, the building should be strengthened or demolished. If the hoop-spacing is less than 10 cm, the building may be applied to the category after 1971. The ground acceleration of 150 gal. may be applied as the boundary between "lower V" and "upper V"

Table-9 : CAPACITY REDUCTION FACTOR FOR THE MEMBER DAMAGED
- Advanced Evaluation Technique ; phase-1 screening -

Damage level of the member	Kind of member		
	Column-F	Column-S	Wall
I	1. 0	1. 0	1. 0
II	1. 0	0. 8	0. 9
III	0. 6	0. 4	0. 6
IV	0. 3	0	0. 3
V	0	0	0

Note) Column-F : column failed by flexure,
Column-S : column failed by shear.

Table-10 : SELF-INSPECTION FORM (for R/C buildings)

<QUESTIONNAIRE> ; Please check one for each question.

Q-1: Do you find either of landslide, landslip, earth-crack, or liquefaction?
- [] I: none
- [] II: yes
- [] III: yes severe

Q-2: Did the building settle, or did the surrounding ground of building sink?
- [] I: none
- [] II: yes, more than 10 cm.
- [] III: yes, more than 20 cm.

Q-3: Did the buliding incline?
- [] I: none
- [] II: it seems inclining
- [] III: inclining clearly

Q-4: Do you find any fracture of floors?
- [] I: none
- [] II: sinking a little
- [] III: sinking severely

Q-5: Do you find any fracture of columns?
- [] I: none
- [] II: concrete spalling
- [] II: cracking severely
- [] II: exposing steel bars
- [] III: fractured completely

Q-6: Do you find any fracture of concrete walls?
- [] I: none
- [] II: concrete spalling
- [] II: cracking severely
- [] II: exposing steel bars
- [] III: fractured completely

Q-7: Do you find any mortar spalling on the exterior walls?
- [] I: none
- [] II: spalling
- [] III: spalled off

Q-8: Did the roof-tiles drop?
- [] I: none
- [] II: disarranging
- [] III: dropped

Q-9: Did the doors of inside or outside fracture?
- [] I: none
- [] II: not open or slide smoothly
- [] III; not open or slide completely

Q-10: Do you find the window-glass broken?
- [] I: none
- [] II: broken a few
- [] III: broken many

Q-11: Did the electric light or ceiling drop?
- [] I: none
- [] II: just dropping
- [] III: dropped

Q-12: Please record the other damage, such as fence, water supplying, a gas leak, overturning furniture, etc. ______________________

____

<SELF JUDGMENT>

Even if there is one item of rank-III, it may be dangerous. If there is the rank-II in the questionnaire No. 1 - No. 8, you should also take care of the circumstance. Please call the engineering official of the nearest local government. Some professional will consult with you about the treatment required.

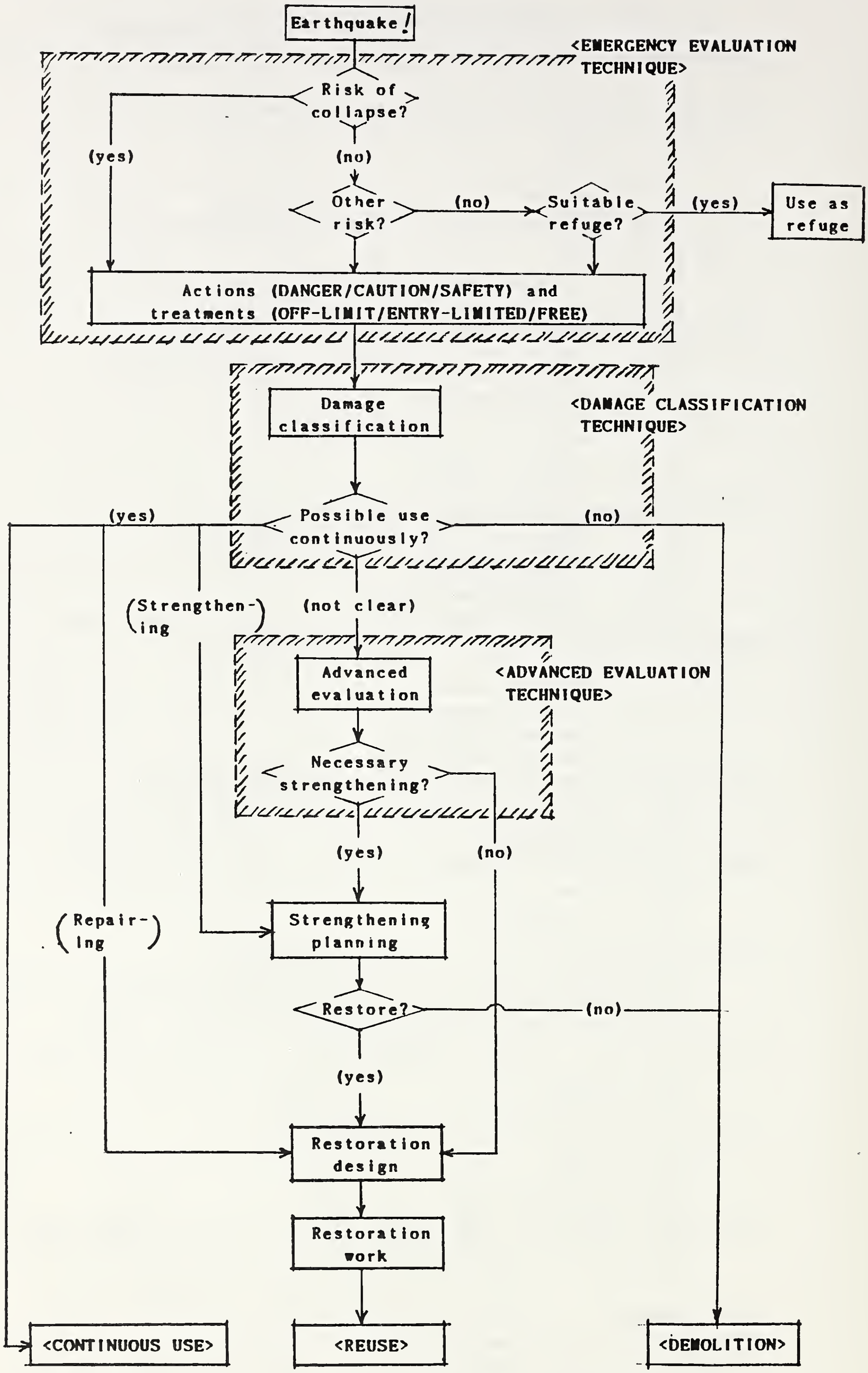
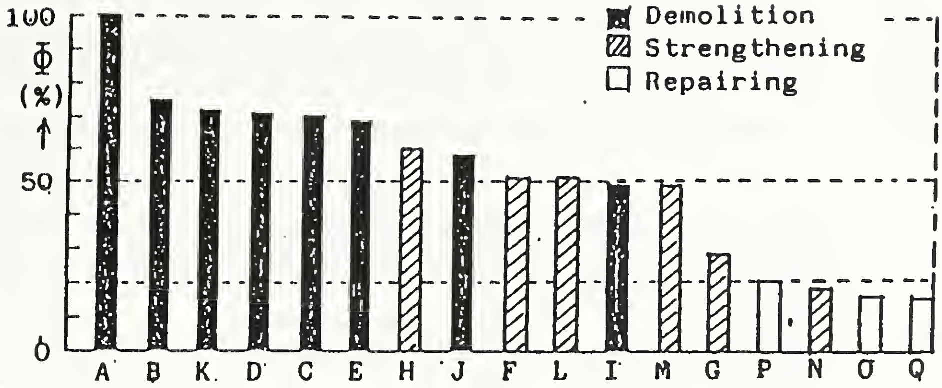


Fig. 1 : GENERAL FLOW-DIAGRAM FOR POST-EARTHQUAKE DAMAGE INSPECTION, EVALUATION AND RESTORATION



Note) Above A - Q represent the name of building suffered damage by the past earthquakes in Japan.

Fig. 2 : EARTHQUAKE DAMAGE INDEX Φ vs. ACTUAL TREATMENTS AFTER DAMAGE (Ref. 10)

SEISMIC DIAGNOSIS FOR GYMNASIUM TYPE STRUCTURES

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INTRODUCTION

Gymnasiums can be used for accommodations for the inhabitants who take refuge before or after big earthquakes. To utilize gymnasiums for a place of refuge, the seismic structural performance of them should be inspected in advance. For this purpose, a seismic diagnosis for gymnasium type structures has been developed.

The characteristics of the seismic diagnosis are:1) Easy operation thanks to the limited scope of application (only applicable to gymnasium type structures), and 2) Simple expression of strength index which is easily understandable for general structural engineer because of the low redundancy of gymnasium type structures.

The procedure of the seismic diagnosis can be expressed as follows:

(1) Preliminary Survey

The outline of the structure will be surveyed such as use, scale, structural system, and so on. It also includes the inspection of deflections, cracks, and corrosions.

(2) Preliminary Evaluation

The applicability of the seismic diagnosis will be evaluated based on the results of the preliminary survey, especially on the deterioration of the structure. The diagnosis can not be applied to the structure having a remarkable deterioration.

(3) Detailed Survey

The actual conditions of the structure will be inspected. The detailed survey results will be compared with those in the design documents.

(4) Calculation of Ultimate Lateral Shear Strength and Strength Index

The ultimate lateral shear strength of the structure will be calculated based on the results of the detailed survey. The strength index will be estimated considering the weight, the deformability, the dynamic characteristics of the structure, and the ultimate lateral shear strength as well.

(5) Evaluation of Seismic Structural Performance and Judgement of Safety

The strength index of the structure will be compared with the structural safety criterion index established in this diagnosis. The structure will be finally classified into four categories as follows: 1) safe; 2) almost safe but better to be strengthened; 3) not safe and to be strengthened; and 4) danger and to be much strengthened. Necessary comments should also be given to the final judgement.

PRELIMINARY SURVEY & PRELIMINARY EVALUATION

The purpose of the preliminary survey is to inspect and record the actual conditions of the structure and of its surroundings. In case that the surveyed conditions are unpreferable, further application of the seismic diagnosis to that structure should be given up.

The structures, whose structural actual conditions preliminary-surveyed agree with at least one of the following items, can be considered to be not suitable for accommodations for refuge inhabitants without further examination.

(1) The story drift angle of the structure is more than or equal to 1 / 120 radian.

(2) The foundation settlement angle of the structure is more than or equal to 1 / 120 radian.

(3) Remarkable overall or local buckling deformation exists in columns or girders.

(4) Cracks or remarkable local kinks can be seen in joint portions.

(5) Remarkable corrosion over the entire surface of members and joints can be seen. Here, the word of "remarkable" means that a loss in sectional area due to corrosion is more than or equal to 10% of the original one.

The followings should also be inspected in the preliminary survey and the observed results should be reported in the final document of the seismic diagnosis results.

(1) Land cracks and landslides

Soundness of the land under and around the structure should be inspected. It includes the inspection of the near by dangerousness of the hills, the retaining walls, and so on.

(2) Seismic structural performance of attached portions

Corridors, eaves and other portions which are attached to the structure should be examined carefully, because the refuge inhabitants frequently pass through under them.

(3) Joints of the interior and exterior finishings

Generally, the joints of the finishings in a gymnasium structure are simple connections using round bars or nails. In particular, the deterioration of these joints in higher portions of the structure should be inspected, since the failing down of the finishings will directly bring about the injury of the refuge inhabitants.

(4) Failure of floors

It should be confirmed that the floor is supported on a rigid foundation beam. Otherwise, the inclination or the sinking of the floor will be expected during the earthquake, which will bring about fear to the refuge inhabitants.

(5) Falling down of ceilings and lighting tools

The strength of the attachment of the ceilings and the lighting tools are not always designed for dynamic force by earthquakes. Therefore, the joint strength should be examined. In case that the joint strength is not enough, it should be strengthened or some treatment should be given for preventing the falling down.

DETAILED SURVEY

The purpose of the detailed survey is to obtain the fundamental data on the actual conditions of the structure, which is necessary to calculate the ultimate lateral shear strength.

(1) Number of the survey locations and their places in the structure

To grasp the actual conditions of the structure within the limited number of survey locations, they should be reasonably selected in plan and elevation. At least three locations of column-to-beam junctions should be selected in the principal lateral shear resisting frames both in longitudinal and transverse directions. The sizes of the members adjacent to the junctions and the execution quality of work should be surveyed.

(2) Items of the survey

a. Measurement of the dimensions of members

The dimensions of columns, beams, cross-braces should be measured and compared with those in the design documents.

b. Inspection of the joints

The followings should be surveyed.

b-1. Welding type

1) It should be inspected whether the welding of the beam flange to the face of the column flange is full penetration welding or fillet welding. The welding type can be considered to be full penetration welding when there exist end tabs, backing plates and rat holes simultaneously. On the other hand, it can be considered to be fillet welding if

all of them are not found. In other cases, a more detailed inspection should be needed to judge the welding type.

2) Size of the fillet welding may be considered to be 5 mm as the minimum size when it cannot be measured.

b-2. Bolted joints

1) The kind of bolts (high-strength bolt or low carbon steel bearing bolt), the size and the number of them should be inspected and compared with those of design documents.

b-3. Diaphragms

1) It should be confirmed that the locations of the diaphragms in the structure agree with those in the design documents. If the diaphragms are not arranged at appropriate locations, the local stress effects should be considered.

c. Survey of the corrosion in members and joints

When the entire surfaces of the members and joints are covered with corrosion, the reduction of the section due to corrosion should be measured.

(3) Assumptions used for the evaluation of member and joint strength

a. When the dimensions of the members and the actual conditions of the details are coincided with those in the design documents, and the corrosions on the entire surface of members and joints are not observed, the member strength and the joint strength should be estimated based on the design documents.

b. When the dimensions of the members are not accordance with those in the design documents, the member size can be assumed as follows: 1) All the members has the same tendency of commonness in member size difference between actual and documents, in case that the structural members are covered by the finishings; 2) Same as 1) but the number of the survey locations should be increased, in case that the structural members are exposed.

c. When only the actual conditions of the details do not satisfy those required in the design documents, the joint strength should be estimated based on the following assumptions: 1) All the weldings should be considered to be 5 mm size fillet welding; 2) All the bolted connections should be considered to have the smallest joint efficiency among the joint efficiencies obtained from the survey. Here, the joint efficiency is defined as the ratio of the joint strength estimated based on actual details and that estimated based on design documents, and it is evaluated independently for beam-to-beam joints, beam-to-column joints, brace end joints, and so on.

d. When the corrosions on the entire surface of the members and joints are observed, the strength should be estimated considering the reduction in sectional area due to corrosion.

CALCULATION OF ULTIMATE LATERAL SHEAR STRENGTH AND STRENGTH INDEX

ASSUMPTIONS

(1) The ultimate lateral shear strength should be estimated on each group of frames, which are rigidly jointed with roof diaphragm. However, the frames which have different loading condition or different structural system from that of the other frames in the group should be considered separately as independent group. Then, the stress transmission between the groups of the frames whose ultimate lateral shear strengths are independently estimated should be assured by horizontal braces.

(2) The ultimate lateral shear strength of the group of the frames should be estimate as the sum of the ultimate lateral shear strengths of the frames of the group. Here, each frame can be considered to be a bar-and-joint model.

(3) The ultimate member strength can be considered to be the full plastic moment or the buckling strength.

(4) The yield strength of the structural steel specified by JIS (Japanese Industrial Standards) can be increased 10 % to evaluate the ultimate lateral shear strength.

(5) The joint strength should be calculated based on the surveyed results.

(6) The ultimate lateral shear strength of the frame in a dual system can be estimated as the sum of that of the moment resisting frame and that of the braced frame. Here, the ultimate strengths of the columns and beams in the braced bay should be reduced considering the axial thrusts induced by the braces. When the uplifting of the foundation is anticipated, the uplifting strength should be considered to be the ultimate lateral shear strength of the dual system frame.

CALCULATION OF THE WEIGHT

The weight of the representative frame of each group of frames should be calculated in both longitudinal and transverse directions. The calculated weight is used for the evaluation of the seismic force.

CALCULATION OF THE FUNDAMENTAL PERIOD

The fundamental natural period can be estimated assuming the gymnasium type structure to be one mass system, where the mass can be considered to concentrate on the roof level. When the lower portion of the structure is made of the reinforced concrete (RC) or the steel structure encased in the reinforced concrete (SRC), RC or SRC portion can be neglected in the estimation of the fundamental natural period because of their high rigidity. The shear spring constant of the steel portion can be estimated based on the elastic frame analysis.

From the fundamental natural period, the response amplification coefficient along the height of the structure (A_i) can be estimated. The design spectrum coefficient (R_t) can also be estimated; it is used to evaluate the structural safety criterion index (E_t).

CALCULATION OF THE ULTIMATE LATERAL SHEAR STRENGTH

(1) Ultimate lateral shear strength in the longitudinal direction

The structural type in the longitudinal direction of gymnasium structures is usually a braced frame. The critical failure modes of a brace frame are as follows: 1) The buckling of the beams; 2) The buckling of the columns; 3) The tensile yielding of the braces; and 4) The joint fracture. The least strength given by the above failure modes can be considered to be the ultimate lateral shear strength of a braced frame. The formulas of the strengths of the members and the joints are given by the equations in the next section.

(2) Ultimate lateral shear strength in the transverse direction

The structural type in the transverse direction of the gymnasium structure is usually a one-story one-bay moment resisting frame. The column base can be considered to be hinged even if it is embedded in RC or SRC.

The ultimate lateral shear strength may be estimated by the plastic hinge method. Here, the effect of the gravity load and the effect of the applied location of the seismic forces should be appropriately considered. The critical sections where the buckling or the joint failure dominates are assumed to behave like plastic hinges for simplicity.

STRENGTH OF MEMBERS

(1) Beam

Flexural strength M_{Cr} (ton-cm) shall be estimated by the following equations.

$$M_{Cr} = M_P \quad \text{in case} \quad \lambda_b \le {}_p\lambda_b$$

$$M_{Cr} = M_P \cdot \left\{1.0 - 0.4 \cdot \frac{(\lambda_b - {}_p\lambda_b)}{({}_e\lambda_b - {}_p\lambda_b)}\right\} \quad \text{in case} \quad {}_p\lambda_b < \lambda_b \le {}_e\lambda_b$$

$$M_{Cr} = M_P / (\lambda_b)^2 \quad \text{in case} \quad {}_e\lambda_b < \lambda_b$$

where, $\lambda_b = \sqrt{M_P / M_e}$,

${}_p\lambda_b = 0.6 - 0.3 \cdot \alpha$,

${}_{e}\lambda_b = 1 / \sqrt{0.6}$,

M_P : full plastic moment, $M_P = F \cdot Z_P$ (ton-cm),

F : standard strength (ton / cm^2),

Z_P : plastic section modulus (cm^3),

$M_e = C \cdot M_{e0}$ (ton-cm),

M_{e0} : lateral buckling moment under uniform bending (ton-cm) ,

$C = 1.75 - 1.05 \cdot \alpha + 0.3 \cdot \alpha^2 \leq 2.3$,

$\alpha = M_2 / M_1$, M_2 and M_1 are the smaller bending moment and the larger bending moment, respectively, about the strong axis at the ends of a member subjected to buckling. M_2 / M_1 takes a positive value in the case of single curvature and a negative value in the case of double curvature. Where moment on the center of the portion subjected to buckling is larger than M_1, C is taken as unity.

(2) Column

a. For H-shaped members which are subjected to bending about their strong axis and rectangular hollow sections

Flexural strength M_{CB}(ton-cm) should be the smaller one of M_{C1}(ton-cm) or M_{C2}(ton-cm).

$M_{C1} = M_P$ in case $N / N_y \leq 0.15$

$M_{C1} = 1.18 \cdot (1 - N / N_y) \cdot M_P$ in case $N / N_y > 0.15$

$M_{C2} = 1.18 \cdot (1 - N / N_C) \cdot M_{Cr}$

b. For open-web members

Flexural strength M_{CB}(ton-cm) should be the smaller one of M_{C1}(ton-cm) or M_{C2}(ton-cm).

$M_{C1} = (1 - N / N_y) \cdot M_P$

$M_{C2} = 2.30 \cdot (1 - N / N_C) \cdot M_{Cr}$

c. For H-shaped members which are subjected to bending about their weak axis

$M_{CB} = M_P$ in case $N / N_y \leq 0.4$

$M_{CB} = 1.19 \cdot \{1 - (N / N_y)^2\} \cdot M_P$ in case $N / N_y > 0.4$

d. For tubular steel members

$M_{CB} = M_P$ in case $N / N_y \leq 0.2$

$M_{CB} = 1.25 \cdot (1 - N / N_y) \cdot M_P$ in case $N / N_y > 0.2$

where, N: axial force (ton),

N_y : axial yield force (ton), $N_y = A \cdot F$,

N_C : buckling force under concentric axial load (ton),

$N_C = N_y$ in case $\lambda_C \leq {}_{p}\lambda_C$,

$$N_C = N_y \cdot \{1 - 0.5 \cdot (\lambda_C - {}_p\lambda_C)/({}_e\lambda_C - {}_p\lambda_C)\} \quad \text{in case} \quad {}_p\lambda_C < \lambda_C \leq {}_e\lambda_C,$$

$$N_C = N_y / (1.2 \cdot \lambda_C{}^2) \quad \text{in case} \quad {}_e\lambda_C < \lambda_C,$$

$$\lambda_C = \sqrt{N_y / N_e},$$

$${}_p\lambda_C = 0.15,$$

$${}_e\lambda_C = 1 / \sqrt{0.6},$$

$$N_e = \pi^2 EI / L_C{}^2,$$

L_C : column height (cm),

E : Young's modulus, E = 2100 (ton / cm^2),

I : moment of inertia about buckling axis (cm^4).

(3) Brace

Tensile strength ${}_bN_y = A \cdot F$ (ton)

Compressive strength ${}_bN_u$ (ton), post-buckling stable strength

$${}_bN_u = N_y \quad \text{in case} \quad \hat{\lambda} \leq 0.15$$

$${}_bN_u = N_y / (11\hat{\lambda} - 0.65) \quad \text{in case} \quad 0.15 < \hat{\lambda} \leq 0.30$$

$${}_bN_u = N_y / (6\hat{\lambda} + 0.85) \quad \text{in case} \quad 0.30 < \hat{\lambda} \leq 1.39$$

$${}_bN_u = 0 \quad \text{in case} \quad 1.39 < \hat{\lambda}$$

where, $\hat{\lambda}$: non-dimensionalized slenderness ratio, $\hat{\lambda} = \lambda\sqrt{F/E} / \pi$,

λ : slenderness ratio.

(4) Built-up bending member

Flexural strength of built-up members

$$M_C' = N_C \cdot h \text{ (ton-cm)}$$

where, N_C : buckling strength of one of the chords.

The buckling length ${}_kL_C$ should be taken to be the interval length of the lateral supports. The column buckling strength formulae can be used. In case that the yielding or buckling of the web precedes the flange buckling, the flexural strength should be reduced appropriately.

STRENGTH OF JOINTS

(1) Column-to-beam junction panel-zone

The strength of the column-to-beam junction panel-zone shall be estimated by the following formula,

$${}_pM_u = \mathrm{MIN}\,[({}_bM_1 + {}_bM_2), ({}_cM_1 + {}_cM_2), {}_pM_p]$$

where, $\mathrm{MIN}[a, b, c\cdots]$: minimum value among a, b, c···

${}_bM_1$, ${}_bM_2$: flexural yield strength of beams adjacent to panel-zone (ton-cm),

${}_cM_1$, ${}_cM_2$: flexural yield strength of columns adjacent to panel-zone (ton-cm),

$_pM_p = 4/(3\sqrt{3}) \cdot V_e \cdot F$ (ton-cm),

V_e : effective panel volume (cm^3), as follows.

a. For H-shaped sections

$V_e = h_b \cdot h_C \cdot t_w$

b. For Box-shaped sections or rectangular hollow sections

The sum of the volumes of the two webs parallel to the plane of action of bending should be the effective panel volume. When the section is box-shaped, square and with uniform thickness, $V_e = V/2$.

c. For tubular sections

$V_e = V/2$

d. For cross-shaped sections

$V_e = \phi \cdot V_W$

where, $\phi = \{\beta^2 + 2.6 \cdot (1 + 2\gamma)\}/(\beta^2 + 2.6)$,

$\beta = h_b / b$,

$\gamma = A_f / A_W$,

$A_f = b \cdot t_f$,

$A_W = b_C \cdot t_W$,

$V_W = A_W \cdot b_h = h_b \cdot h_C \cdot t_W$,

: volume of the panel of H-columns directly connected to be beam,

V : total volume of the joint panel-zone (cm^3),

h_b : web height of the beam (cm),

h_C : web height of the column (cm),

t_f : flange thickness of the column (cm),

t_W : web thickness of the column (cm),

b : flange width of the column (cm).

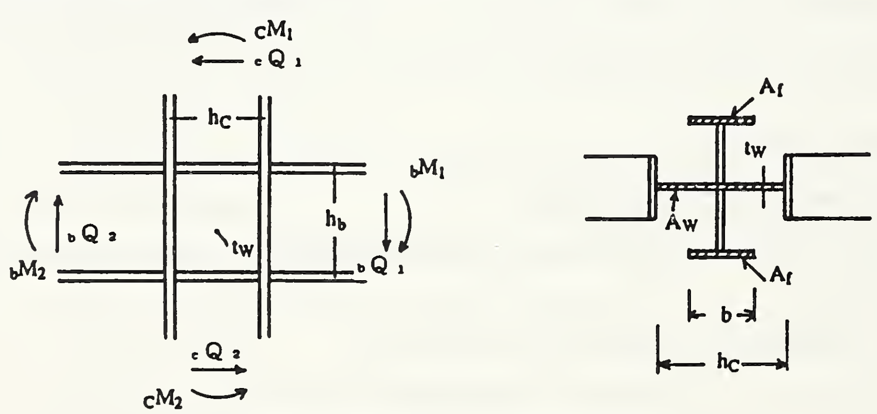

Figure 1 Effective Panel Volume

(2) Brace joint

The strength of the brace, whose end joints are bolted, shall be estimated as follows.

$P_u = \mathrm{MIN}[P_1, P_2, P_3, P_4]$ and $P_u \geq 1.2 \cdot N_y = 1.2 \cdot A \cdot F$

a. Bolt shear strength

$P_1 = 0.62 \cdot m \cdot n \cdot {}_fA_s \cdot {}_fF_u$

where, m : number of shear planes,

n : number of bolts,

${}_fA_s$: sectional area of bolt,

${}_fF_u$: specified minimum tensile strength of bolts (ton / cm^2).

b. Tensile strength of effective brace section

$P_2 = A_e \cdot F_u$

where, A_e : effective sectional area of brace,

F_u : specified minimum tensile strength of brace.

b-1. For angle and channel section braces (see Figure 2)

$A_e = A - (d_o \cdot t_2 + h_n \cdot t_1)$

where, d_o : diameter of bolt hole,

t_1 : thickness of the unconnected leg,

t_2 : thickness of the connected leg,

h_n : height of reduced area of unconnected leg (see Table 1).

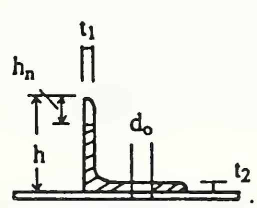

Figure 2 Effective Sectional Area of Angle Brace

Table 1 Height of Reduced Area of Unconnected Leg (h_n)

	number of bolts in line of stress	1	2	3	4	5
h_n	angle section	$h - t_2$	0.7h	0.5h	0.33h	0.25h
	channel section	$h - t_2$	0.7h	0.4h	0.25h	0.2h

b-2. For flat bar braces

$A_e = A - d_o \cdot t_2$

c. Strength of edge distance

$P_3 = n \cdot e \cdot t \cdot F_u$

where, n : number of bolts in line of stress (number of bolt line should be one),

e : edge distance in line of stress,

t : thickness of the connected leg of brace or the gusset plate.

d. Strength of gusset plate effective section

$P_4 = {}_gA \cdot F_u$

where, ${}_gA$: effective sectional area of gusset plate (see Figure 3).

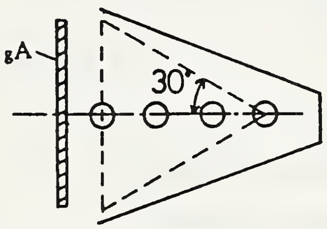

Figure 3 Effective Sectional Area of Gusset Plate

STRENGTH INDEX I_S

(1) The strength index I_S is estimated by the following equation.

$I_s = (Q_u \cdot F) / (A_i \cdot W)$

where, Q_u is the ultimate lateral shear strength, $F = 1 / D_S$, D_S is the structural coefficient specified by the Recommendations For Structural Calculation (Building Center of Japan 1988), Ai is the response amplification along the height of the building stipulated by the Enforcement Order of the Building Standard Law (EOBSL), and W is the weight of the structure.

(2) When the gymnasium is two storied structure with RC or SRC at its 1st story, 1.5 times of the estimated A_i should be used to estimate the earthquake loads. This should be considered in the estimation of the ultimate lateral shear strength in the transverse direction.

EVALUATION OF SEISMIC STRUCTURAL PERFORMANCE AND JUDGEMENT OF SAFETY

(1) Evaluation of the seismic structural performance is done comparing the strength index I_S and the structural safety criterion index E_t,

I_s / E_t

where, the structural safety criterion index E_t can be calculated by the equation below,

$E_t = Z \cdot R_t \cdot C_o$

and where, Z is the zone factor stipulated in EOBSL, R_t is the design spectrum coefficient stipulated in EOBSL, and C_o is equal to or larger than unity.

(2) The final safety judgement is done according to the following criterion:

a. The structure is safe, when the ratio (I_s / E_t) is more than or equal to unity;

b. The structure is almost safe but is better to be strengthened, when the ratio (I_s / E_t) is more than or equal to 0.9 and less than unity;

c. The structure is not safe and should be strengthened, when the ratio (I_s / E_t) is more than or equal to 0.7 and less than 0.9; and

d.The structure is danger and should be much strengthened immediately, when the ratio (I_s / E_t) is less than 0.7.

(3) It should be recommended that some counter-measure should be taken care when the followings are observed in the preliminary survey even if the final safety judgement is "safe": 1) The story drift angle is more than or equal to 1 / 200 radian; 2) The foundation settlement angle is more than or equal to 1 / 200 radian; 3) The loss in section due to corrosion is less than 10%, but corrosions on the entire surface of the members and joints can be seen; and 4) Other defects such as an inadequate fastening of the high strength bolts, which will cause the reduction in strength or deformability of the structure.

(4) The followings should be written down in the final judgement document: 1) I_s / E_t ratio; 2) The ultimate lateral shear strengths of moment resisting frames and braced frames; 3) The failure modes which determined finally the ultimate lateral shear strength; and 4) The structural coefficient (D_s) and the basis for the determination of D_s.

APPENDIX --- EVALUATION OF LATERAL BUCKLING STRENGTH

The lateral buckling strength can be expressed by the following equation.

$$M_e = C \cdot M_{e0}$$

where, $C = 1.75 - 1.05 \cdot \alpha + 0.3 \cdot \alpha^2 \leq 2.3$,

$$M_{e0} = \sqrt{\frac{\pi^4 E^2 I_y I_W}{{}_kL_b{}^4} + \frac{\pi^2 E I_y G J}{L_b{}^2}}.$$

From above equations, the slenderness ratio for lateral buckling can be estimated as follows.

$$\lambda_b = \sqrt{\frac{M_P}{M_e}} = \sqrt{\frac{M_P}{C \cdot M_e}} = \frac{1}{\sqrt{C}} \frac{\sqrt{F \cdot Z_P}}{\sqrt[4]{\frac{\pi^4 E^2 I_y I_W}{{}_k\lambda_b{}^4} + \frac{\pi^2 E I_y G J}{L_b{}^2}}}$$

Substituting $I_W \approx I_y \cdot h^2 / 4$ and ${}_kL_b = k \cdot L_b$, the above equation becomes;

$$\lambda_b \approx \frac{1}{\sqrt{C}} \frac{1}{\pi} \sqrt{\frac{F}{E}} \cdot \frac{k \cdot L_b}{i_y} \left\{ \frac{1}{4} \left(\frac{A \cdot h}{Z_P} \right)^2 + \left(\frac{G}{E} \right) \cdot \frac{1}{\pi^2} \frac{A \cdot J}{Z_P{}^2} k^4 \cdot \left(\frac{L_b}{i_y} \right)^2 \right\}^{-1/4}$$

Neglecting the web of the section, the above equation can be more simplified as below;

$$\lambda_b \approx \frac{1}{\sqrt{C}} \frac{1}{\pi} \sqrt{\frac{F}{E}} \cdot \frac{k \cdot L_b}{i_y} \left\{ 1 + \frac{4}{3} \frac{1}{\pi^2} \left(\frac{G}{E} \right) \cdot \left(\frac{t_f}{h} \right)^2 \cdot k^4 \cdot \left(\frac{L_b}{i_y} \right)^2 \right\}^{-1/4}$$

$$\approx \frac{1}{\sqrt{C}} \frac{1}{\pi} \sqrt{\frac{F}{E}} \cdot \frac{k \cdot L_b}{i_y} \left\{ 1 + 0.05 \cdot \left(\frac{t_f}{h} \right)^2 \cdot k^4 \cdot \left(\frac{L_b}{i_y} \right)^2 \right\}^{-1/4}$$

The k-value can be considered to be 0.55 in case for a beam, whose both ends are rigidly connected to the columns. It can be considered to be 0.75 in case for a beam, whose end is supported by a column or a lateral buckling support member.

SECTION THREE

RESEARCH ON TECHNIQUES FOR REPAIR AND RETROFIT OF STRUCTURES

Recent Research - Repair and Strengthening of Reinforced Concrete Structures

by

James O. Jirsa[1]

ABSTRACT

For the past 8 to 10 years there has been some activity in the US in the area of repair and strengthening of existing hazardous structures in seismic zones. Recent earthquakes in California (Whittier, 1987; Loma Prieta, 1989) have demonstrated the need for an acceleration of these efforts and for development of a coordinated national program. The purpose of this paper is to review briefly some of the ongoing work at the University of Texas and to outline plans for a more concerted national research effort to mitigate the hazard posed by existing buildings.

KEYWORDS: Repair; strengthening; reinforced concrete structures; research

1. RESEARCH ON R.C. FRAME STRUCTURES

One structural type which has been known for a long time to pose a special problem is the broad category of reinforced concrete frame systems designed primarily for gravity loads, and with only cursory attention to seismic forces or details. Such structures have a number of easily identified weaknesses; low stiffness (drift problems), low lateral force capacity, and poor detailing (inadequate beam and column transverse reinforcement, lack of continuity in longitudinal beam or column reinforcement).

In the research program at the University of Texas, two strengthening techniques for "non-ductile" moment resisting frames have been studied: member jacketing or encasement and the addition of infill walls. Some of the tests were a direct outgrowth of observations from the 1985 Mexico City earthquake where many structures with flexible, poorly-detailed columns failed. Member jacketing is particularly appropriate where the basic lateral load resisting system must be maintained for architectural or occupancy requirements or where the existing foundations cannot be easily modified to accommodate a new or alternate lateral load resisting system. If the structure and foundation can accommodate the force concentration produced by the construction of infill walls in selected frames, the cost may be reduced because construction associated with rehabilitation can be concentrated to a few locations in the structure.

2. JACKETING OF FRAME ELEMENTS

An interior joint representing a prototype structure was designed according to American and Mexican design practices of the 1950's. The structure was not detailed for ductile response. The floor system was designed for large gravity loads and the columns were not designed to carry significant lateral forces. As a result the system consists of a joint with weak columns and strong beams. Four R/C frame connections were tested after being repaired and/or strengthened by jacketing only columns or both columns and beams. One specimen was tested to failure (ME1), repaired by jacketing the column with bundled longitudinal reinforcement, and retested (ME1-R). A second identical specimen was built and jacketed without damage to permit a comparison of the effect of a damaged column core on the response (ME2). To compare the influence of bundles, the column of a third specimen was jacketed distributing the longitudinal reinforcement around the column (ME3). In the fourth specimen, the column was jacketed using distributed bars, and the beams jacketed to increase moderately their flexural capacity (ME4). The experimental program is summarized in Table 1.

The dimensions of the specimens are shown in Fig. 1. The original model had two perpendicular beams with 8 x 20-in. section and a square column with 12-in. sides. The slab was 5-in. thick. The reinforcement consisted of Grade 60 deformed bars. Details of the reinforcement are shown in Fig. 2. According to the design practice of the times (1950's), no ties were placed in the column within the joint region. The lower column, beams and slab were cast in a first stage; the upper column was cast a few days later. Specimen ME1 was tested to failure (Martinez, 1988; Alcocer and Jirsa, 1990), repaired and retested (ME1-R). The beams and slab were not modified. With the concrete jacket, the column section increased to 20-in. sides. To

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improve bond between the existing column concrete and the jacket, the surface was roughened with a chipping hammer. The column reinforcement was placed through perforations of the slab. Transverse steel for the column was made with two #4 L-shaped ties that overlapped in diagonally opposite corners and spaced to meet the requirements of ACI 318-83. For ME3 and ME4 additional #3 cross-ties were placed between bars adjacent to the beams and located in the same quadrant. Beams in specimen ME4 were also jacketed to increase the section to 15 x 23-in. with reinforcement as shown in Fig. 2. Top bars crossed the orthogonal beams through holes and bottom bars were placed under the soffit of the original beam on each side of the original column. Transverse steel consisted of U-shaped ties fixed to the top jacket bars, and of inverted U-shaped ties placed through perforations on the slab and that overlapped in the top bars of the jacket.

To confine the joint concrete not confined by transverse beams, and to confine the column bars, a structural steel cage was welded around the joint (Fig. 3). The cage consisted of A36 structural steel angles and steel straps welded in situ. The cage eliminated the need to drill holes through the beams for placing the ties.

Specimens were tested applying a bi-directional cyclic load history. The hysteretic response in the EW direction of each specimen is shown in Fig. 4. Envelope of the hysteretic response are shown in Fig. 5.

Specimen ME1 failed by plastic hinging in the columns, with almost no flexural cracking in beams and slab. The ratio of column-to-beam flexural capacities was 0.31. The specimen exhibited a very poor performance with severe degradation of stiffness and energy dissipation. The repaired specimen (ME1-R) exhibited improved response in terms of strength, stiffness and energy dissipation. Specimens ME2 and ME3 had similar behavior and performed better. In both cases beam hinging was developed. There was no obvious detrimental effect of column bar bundles (ME3) but detailed analysis of the results indicates that the 'components of deformation, flexure in the beam and columns and shear in the joint, were different in the two tests.

In ME4, large shear deformations of the joint were evident but beam hinging was produced. The good performance of the joint can be attributed to the wide beams framing into the joint.

Envelopes of the response in the EW direction for the five tests are shown in Fig. 5. The poor behavior of ME1 in relation to the other jacketed specimens is clear. Comparison of ME1-R and ME2 shows that by jacketing the most damaged element, the column, the strength and stiffness were 35% and 45%, respectively, of the values obtained in the undamaged specimen. As expected, ME4 was the stiffest, strongest and toughest specimen, since the beam were also jacketed.

3. ADDITION OF INFILL WALLS

To assess the performance of infill walls, five specimens were tested. The existing frame system selected was one in which the columns are quite flexible and are poorly detailed. The spacing of transverse reinforcement in the columns was based on tied column requirements and resulted in spacings equal to the side dimension of the column. In addition, the column longitudinal reinforcement was spliced at the bottom of the column and the splice length was designed to meet compression requirements only. The details of the existing frame columns can be seen in Fig. 6. The test frame was a 2/3 scale model of a prototype frame. The lower beam had a large cross-section and was bolted to the lab floor to simulate a rigid base condition. The upper beam had a width equal to that of the column but a depth greater than that of the prototype. The depth was selected to produce a boundary condition that would more closely simulate a multi-story frame with infills at all levels in the span selected. Lateral loads were applied to the heavily reinforced blocks at the top of the beam as shown in Fig. 7.

Three specimens were strengthened with shotcrete infill walls; one solid wall, one with a window opening (4'-8" x 2'-8" centered in wall), and the third with a door opening (3'-4" x 4'-8"). The other two specimens had cast-in-place walls; one with a door opening but with added vertical wall reinforcement (Fig. 8) adjacent to the column, and the other with a solid wall cast against the side face of the columns and with a jacket around the columns (Fig. 8). A summary of the test program is given in Table 2 and all details in Reference 3.4.5.

The specimens were subjected to cyclic lateral loading. A typical lateral load-drift curve is shown in Fig. 9. Since the curves were similar only the envelope curves for the specimen will be shown for comparison. In Fig. 10, load-drift envelope curves are shown for the three specimens with shotcrete infill walls. In the solid wall (SC-S) the peak load was reached when the splice in the column in tension under lateral load failed. Under continued increasing deformation the crack which developed at the top of the splice extended into the wall just above the dowels connecting the all reinforcement to the lower beam (base). This failure was noted under loading in both directions (Fig. 11). In the specimen with a window (SC-W), failure was produced by diagonal cracking which formed from an upper corner to the opposite corner at the base. Large cracks penetrated through the upper end of the column in tension. In the specimen with a door (SC-D) peak load was reached when the splice failed as in SC-S. However, the walls on each side of the door worked independently with tension failure in the column splice in one wall segment and at the dowels to the trim steel around the door opening (Fig. 12).

The infill for specimen CIP-D was cast-in-place. The added wall steel carried tensile forces which could not be developed by the splice in the column. As a result, the section at the base of the wall reached flexural capacity with crushing in the segment where the wall adjacent to the door was in compression. A comparison of the load-drift response for the cast-in-place and shotcrete walls with doors are shown in Fig. 13. In the specimen with a cast-in-place eccentric wall (CIP-S), the wall reached is full flexural capacity with vertical wall and column reinforcement yielding at the base (Fig. 14). Diagonal cracking was uniformly distributed over the wall and there were indications that local crushing along the diagonal was imminent in some sections of the wall. The capacity of the wall was slightly higher than that of the companion shotcrete infill because the column or boundary elements were larger and had additional longitudinal steel. The column jacket also served to confine the column splice region and prevent a splice failure in tension.

4. FUTURE RESEARCH

Since 1987 when the last US-Japan Workshop on repair and strengthen of structure was held in Tsukuko, there has been activity underway to develop a coordinated national program. Recently NSF announced an initiative for a 5-year program on "Repair and Rehabilitation of Research for Seismic Resistance of Structures." The primary objectives of this program activity are:

- to provide technical information for realistic evaluation of existing structures for various levels of seismic excitation, and
- to develop and document cost-effective construction techniques for repairing or strengthening structures found to be hazards.

Research proposal are being encouraged to study key problems in the following topic areas:

- Performance evaluation of existing buildings and foundations,
- Load-transfer mechanisms,
- Retrofitting criteria and techniques,
- Problems and solutions applicable to seismic zones nation-wide, and
- New materials, methods and devices for seismic retrofitting, and their design, manufacture, fabrication, and field installation.

Ordinary concrete structure designed without seismic provisions or with older provisions are included in the group which appear to represent the greatest hazard. Unreinforced masonry infill walls in concrete frames are specifically cited as representing a problem. The projects supported under this initiative are expected to contribute directly to the development of a comprehensive technical summary document including design requirements and details for engineering practice.

5. CONCLUSIONS

1. Jacketing was shown to change the structural system from a strong beam-weak column to a strong column-weak beam system. By jacketing the most damaged element, the column, the strength and stiffness were 35% and 45%, respectively, of the values obtained in the redesigned undamaged structure. With adequate confinement and a strong column (relative to beam strength), bundled column bars did not have a negative effect on the behavior of the specimens. The joint cage fabricated from the steel straps and angles provided to be effective in confining the joint concrete. No loss bond was detected between the old and new concretes. The addition of beam jacketing produced satisfactory performance but construction of the cages for the jacketed column and beam was difficult and probably too expensive for US application.

2. The successful use of infill walls to modify the lateral load resisting system is dependent on the correction of deficiencies in the existing frames. Ductile response was obtained when the tensile capacity of the infill wall boundary elements was assured by jacketing the column or by adding vertical wall reinforcement next to the columns.

3. An NSF initiative to begin in 1990 should provide impetus for new research in evaluation, repair and strengthening of existing hazardous structures. The ultimate goal is to produce recommendations for design of systems to reduce the risk posed by existing structures.

6. ACKNOWLEDGEMENTS

The support of the National Science Foundation through grants ECE-8416147, ECE-8610946 and ECE-8701316 is gratefully acknowledged. The contributions of many students: P.J. Gaynor, S.M. Shah, L.R. Jimenez, S.M. Alcocer, J.E. Martinez, and faculty: M.E. Kreger, R.L. Carrasquillo, are responsible for the progress that has been made in the projects described in this paper. The collaboration with the staff at H.J. Degenkolb Associates has been most helpful in developing test specimens and interpreting results.

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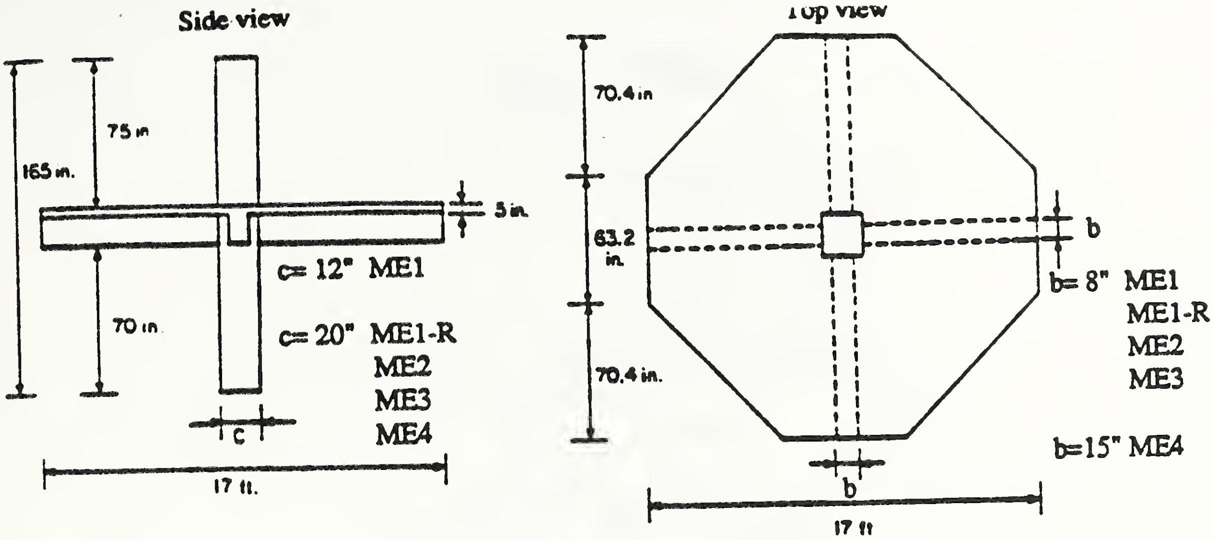


Figure 1. Dimensions of the specimen

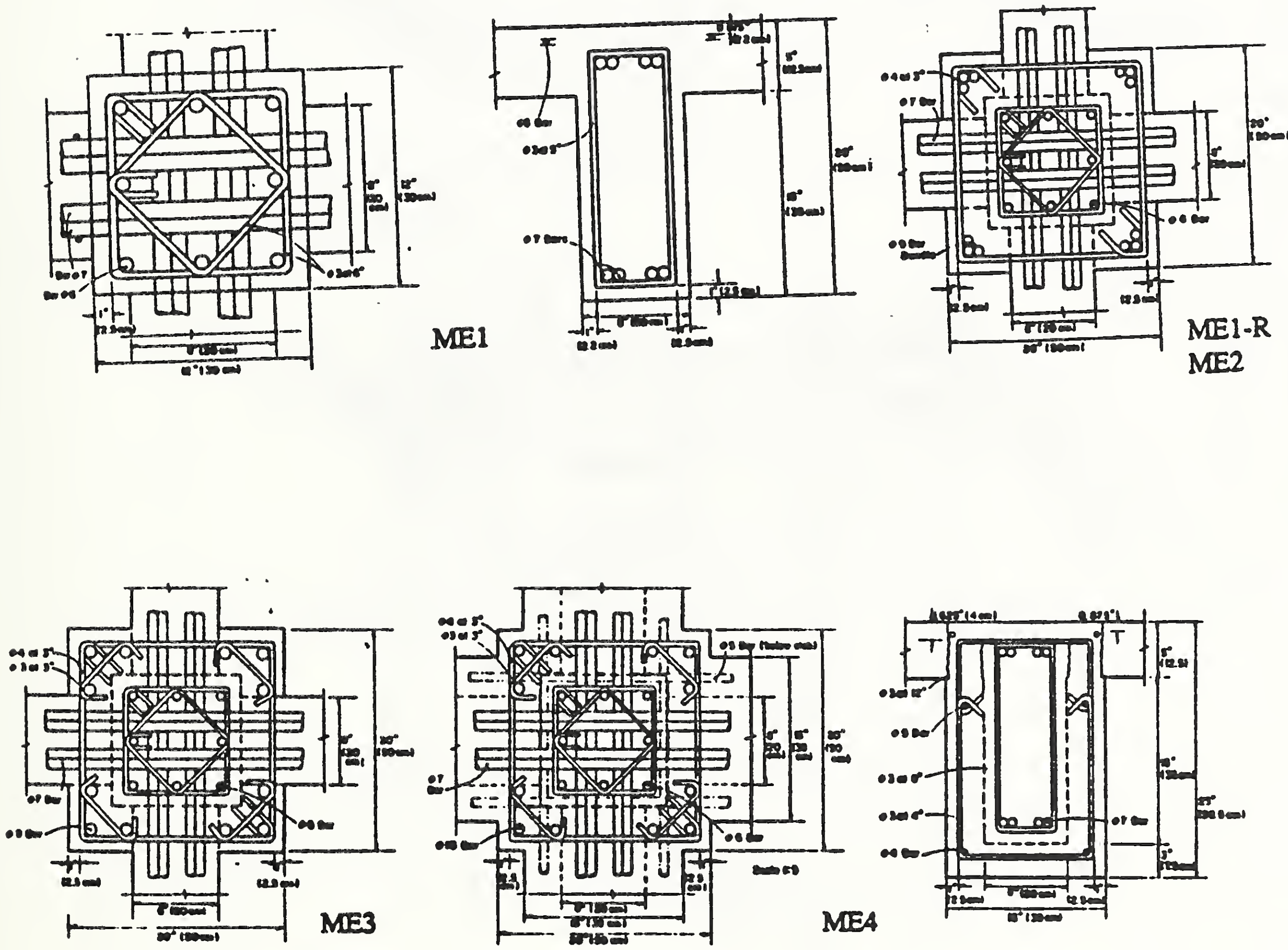


Figure 2. Details of the reinforcement

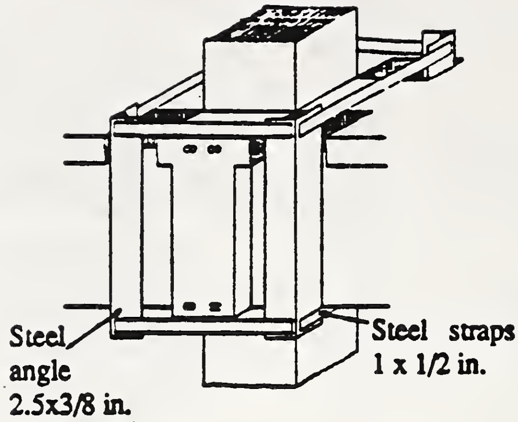


Figure 3. **Structural steel cage assembled in the joint**

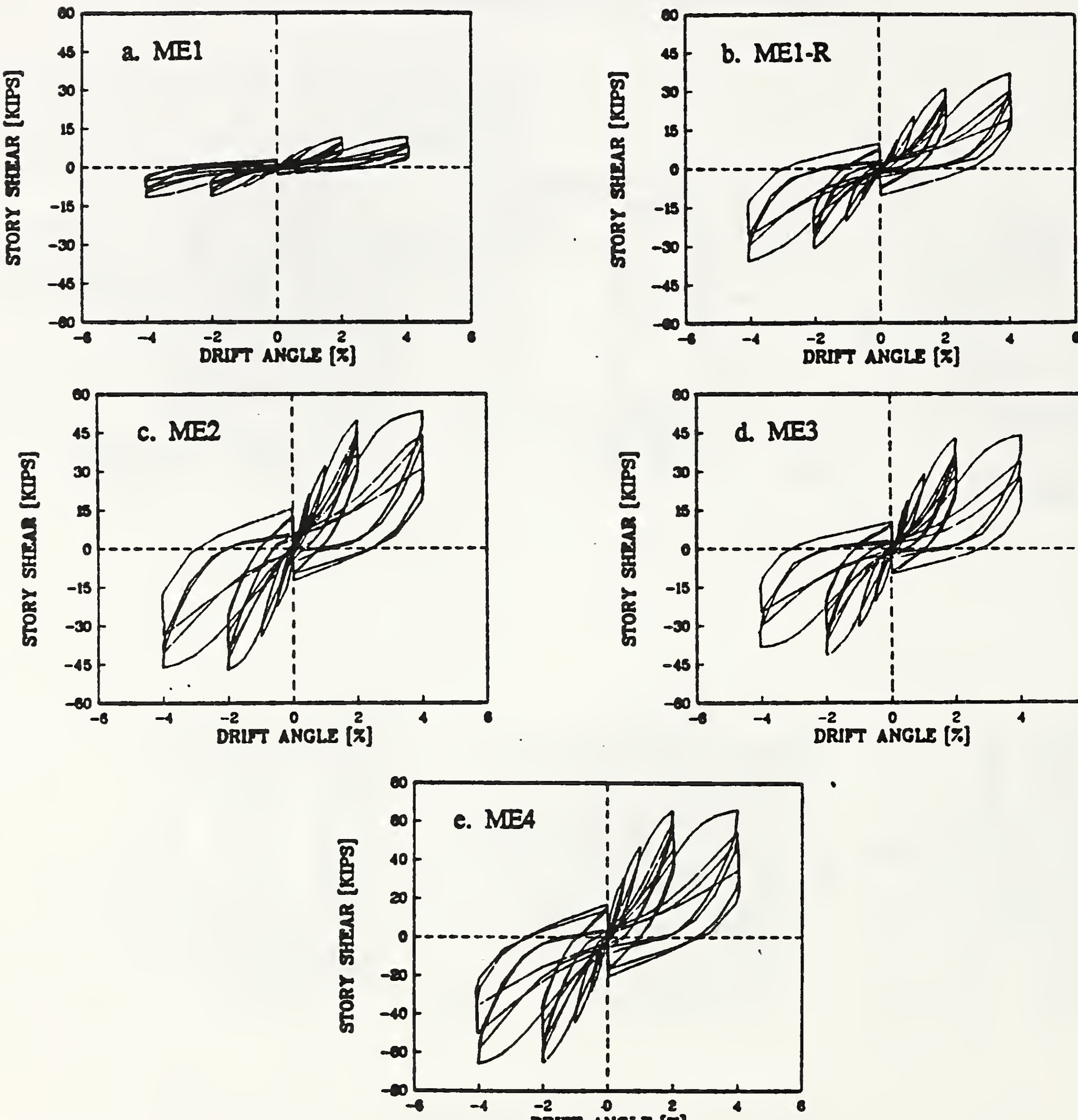


Figure 4. Hysteretic response of the specimens (EW direction).

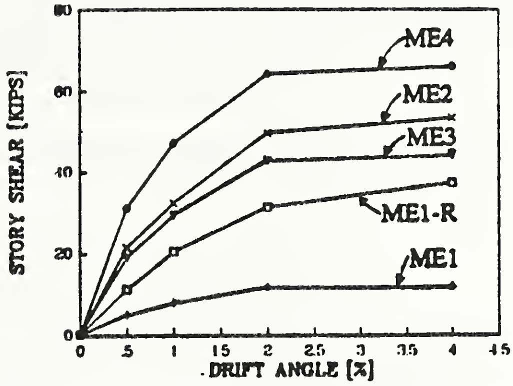


Figure 5 Envelopes of the response of the models (EW direction).

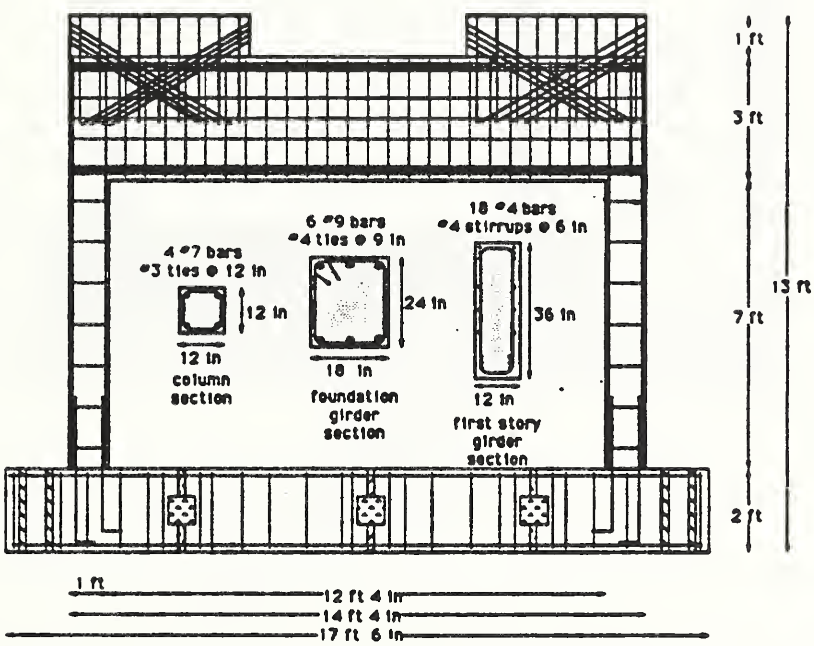


Figure 6. Model of existing frame

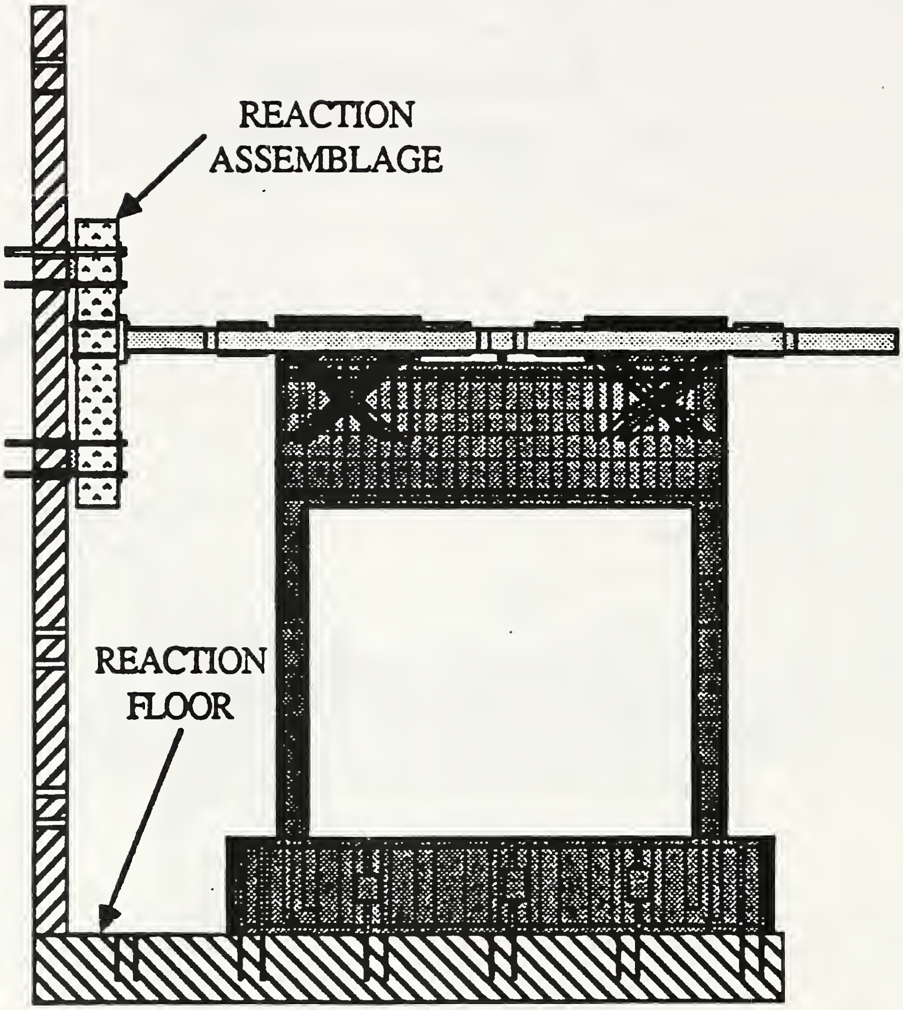
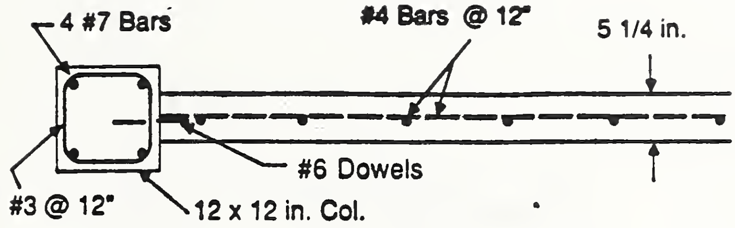
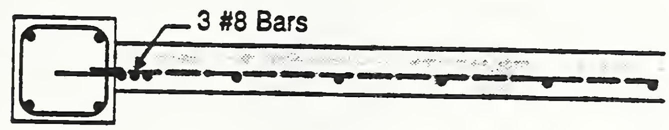


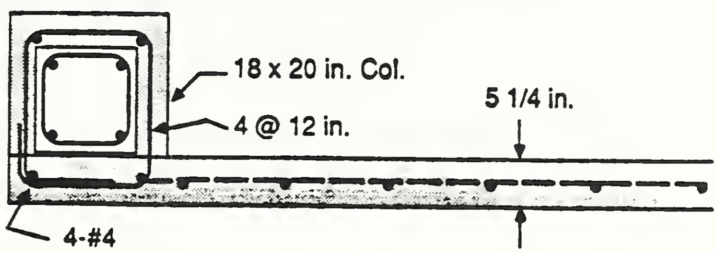
Figure 7. Loading System for Infill Wall Tests



(a) Shotcrete walls; Solid, door, window



(b) Cast-in-place with door, added wall bars



(c) Cast-in-place, eccentric wall

Figure 8. Column and wall details

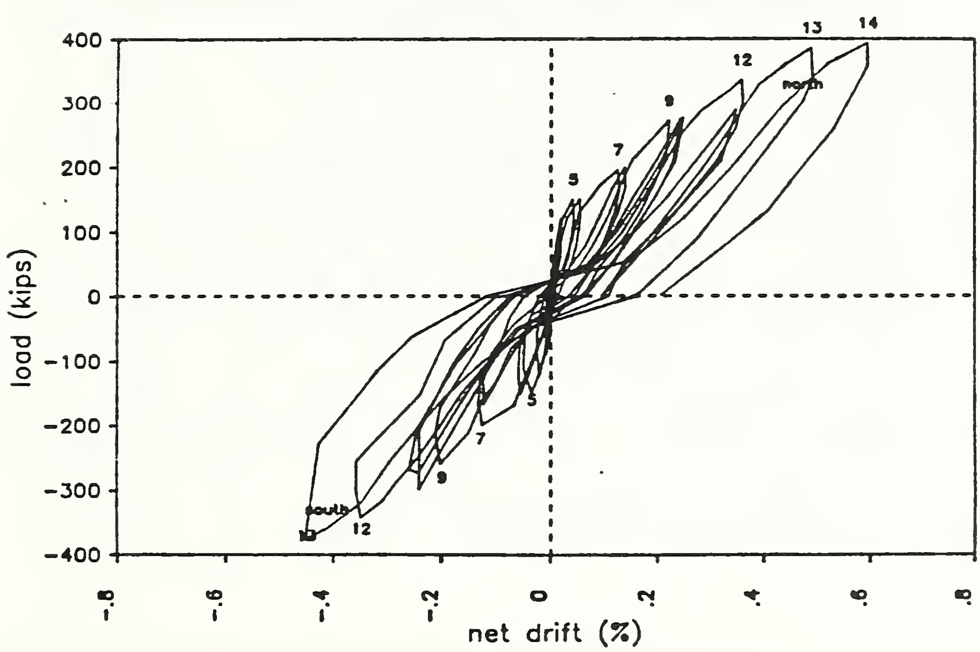


Figure 9. Measured Load-Drift Response Curve (Test CIP-S)

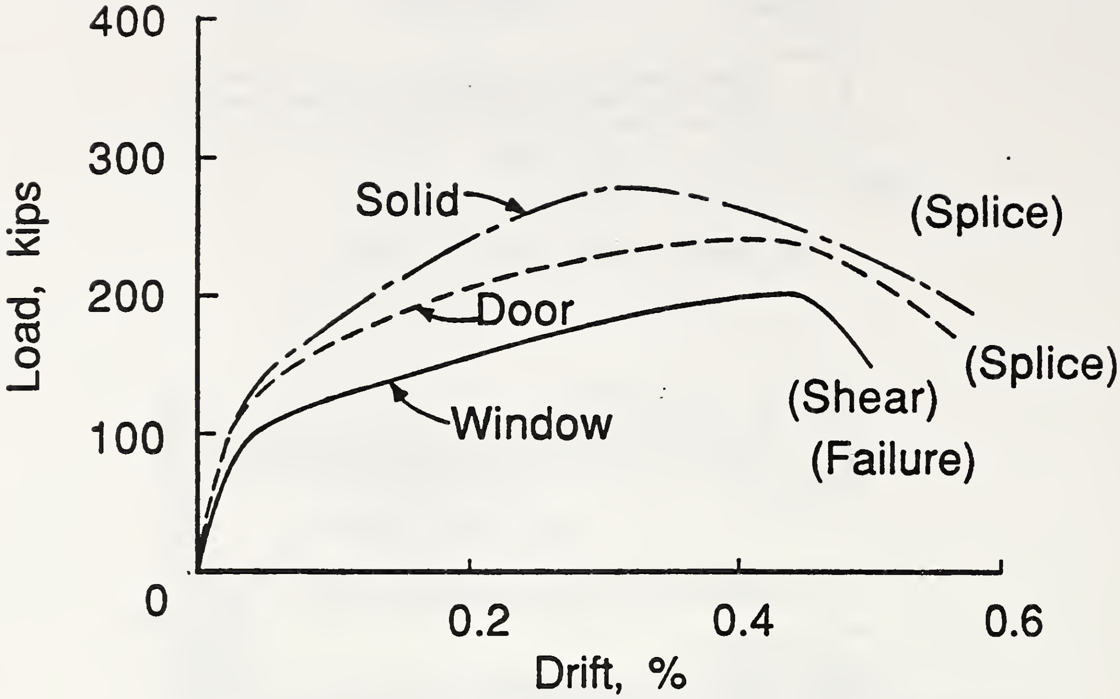


Figure 10. Envelope curves - shotcrete infill walls

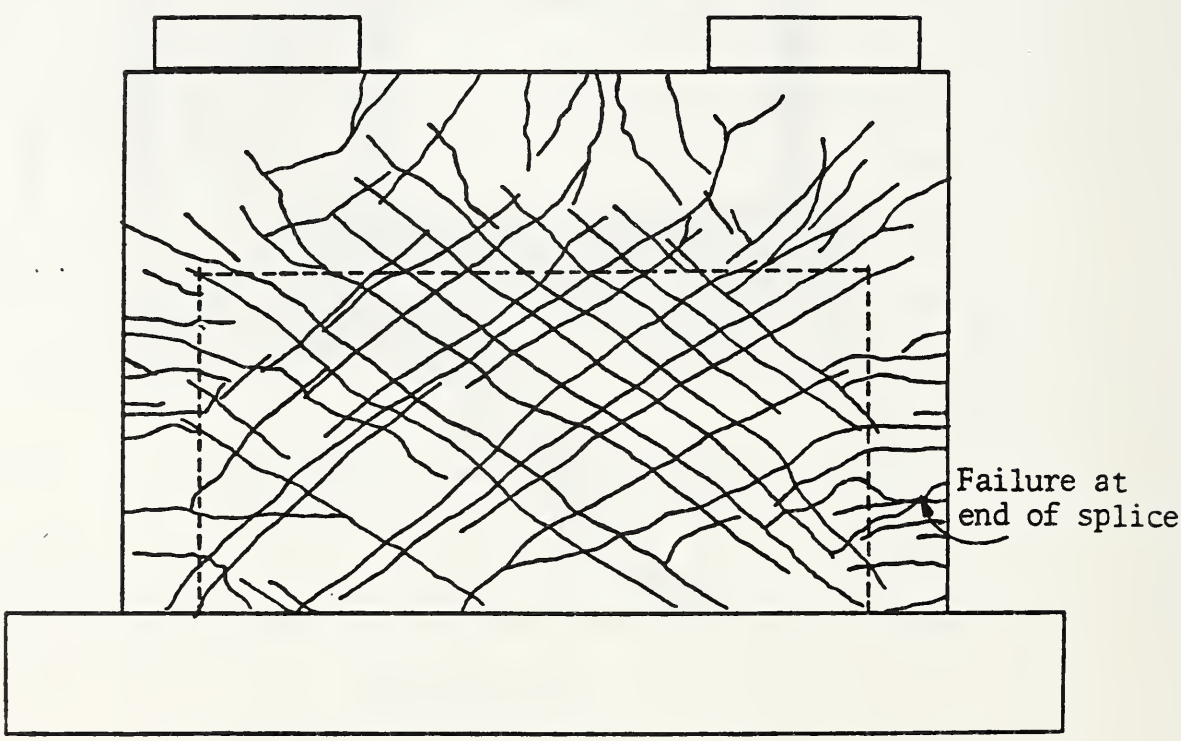


Figure 11. Crack pattern at failure, SC-S

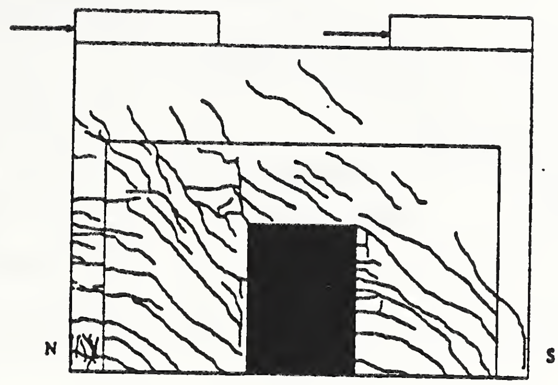


Figure 12. Crack pattern at failure, SC-D, south loading

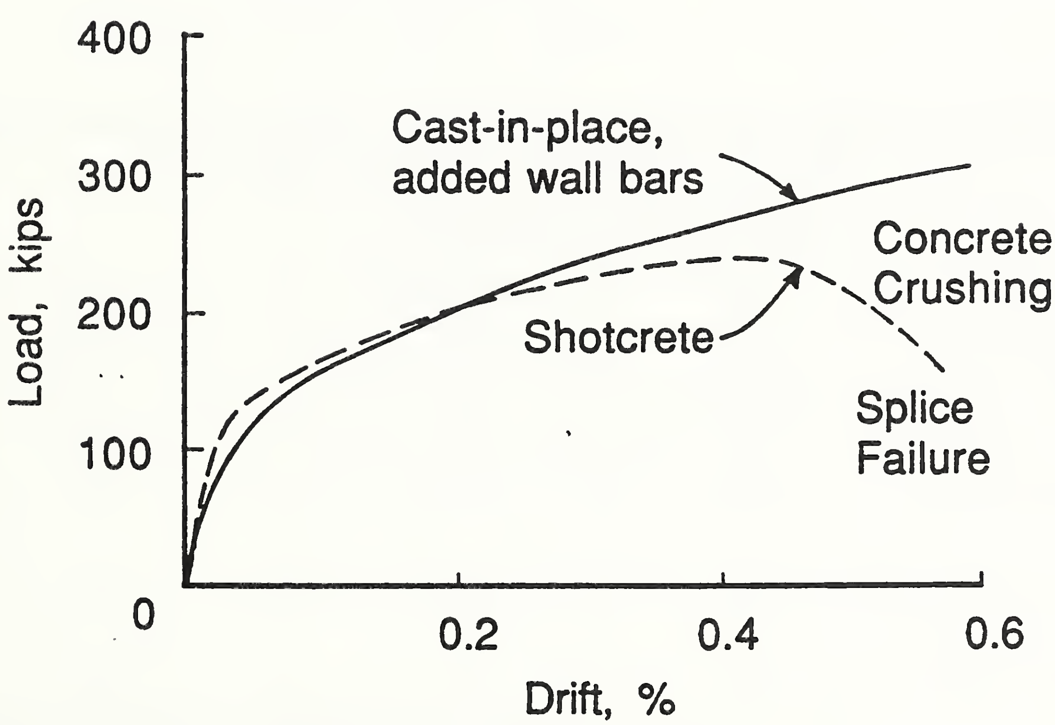


Figure 13. Envelope curves for tests CIP-D and SC-D

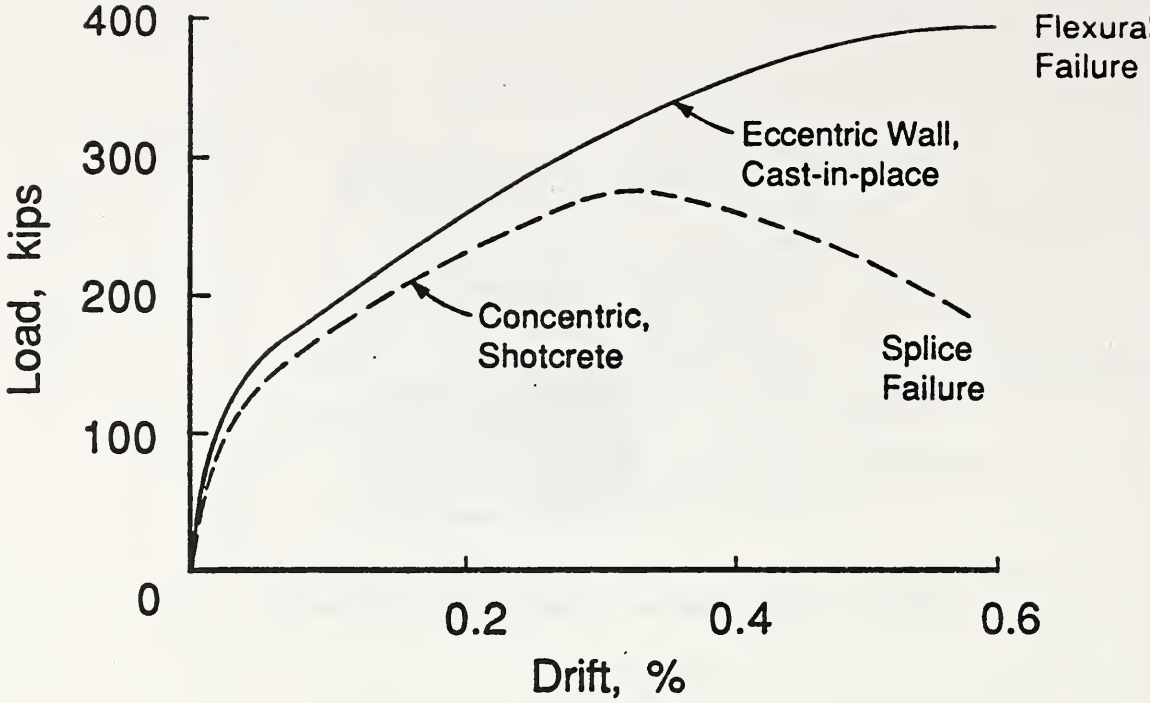


Figure 14. Envelope curves for tests CIP-5 and SC-S

Table 1. Summary of Test Program - Jacketing						
Test Name	Specimen Before Test	Jacketing		Bundled	f'_c	
		Columns	Beams		Existing	Jacket
ME1	Undamaged	----	----	----	2.3	----
ME1-R	Damaged	X		X	2.3	5.1
ME2	Undamaged	X		X	4.0	6.0
ME3	Undamaged	X			2.7	5.6
ME4	Undamaged	X	X		2.7	5.8

Note: All reinforcement Grade 60

Table 2. Summary of Test Program - Infill Walls			
Specimen	Concrete Strength at Time of Testing		Mode of Failure
	Existing Frame f'_c	Infill Wall f'_c	
SC-S	4.15	3.23	Column Splice
SC-W	5.58	3.41	Shear
SC-D	4.80	3.41	Column Splice
CIP-D	4.70	3.92	Flexural Compression
CIP-S	4.20	4.50	Flexural Yielding

Reinforcement: Grade 60, f_y = 63 ksi (#4) and 70 ksi (#7)

POST-INSTALLED ANCHOR BOLTS SUBJECTED TO TENSION

BY

Youji HOSOKAWA[1] and Hiroyuki AOYAMA[1]

ABSTRACT

The performance of post-installed anchors, which have commonly been used in seismic retrofit of existing reinforced concrete buildings by adding new in-filled shear walls, was investigated in a series of experimental efforts. Reliability of metal expansion anchors in their pull-out stiffness and strength was improved by application of pre-loading at their installation. Pull-out stiffness and strength of adhesive chemical anchors were improved by cleaning their installing holes with nylon or wire brush. The new installation technique of the post-installed anchors is shown to increase the lateral resistance of a structure retrofitted with post-cast shear walls.

KEYWORDS: post-cast shear wall; post-installed anchor; metal expansion anchor; adhesive chemical anchor; pull-out; stiffness; strength

1. INTRODUCTION

Adding infilled shear walls to existing reinforced concrete buildings is a common seismic retrofitting method in Japan. Such post-cast shear walls are connected to existing frames with post-installed anchor bars or bolts. Details or numbers of post-installed anchors are designed according to the Guidelines for Repair and Retrofit Design of Existing Reinforced Concrete Buildings[1]. The strength of post-cast shear walls is estimated lower than that of monolithic shear walls in the guidelines because many tests on post-installed shear wall panels showed (a) slip failure along the boundary between a post-cast shear wall panel and the surrounding frame or (b) pull-out of connecting anchors from the surrounding frame. A design criterion of a post-cast shear wall is the resistance against the slip-failure, which was formulated on the basis of the direct shear tests of concrete elements reinforced with anchors.

However, recent tests of post-cast shear wall specimens demonstrated that the lateral resistance of the retrofitted frame-wall increased with the pull-out resistance of anchors. In addition, a series of experimental investigation was carried out to develop construction techniques and shapes of post-installed anchors, and demonstrated the improvement in the pull-out performance of post-installed anchors.

In this report, the performance of two types of anchor, i.e., metal expansion anchors and adhesive chemical anchors, is studied. Two

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techniques are introduced effective to generate sufficient pull-out stiffness and strength of the metal expansion anchor; (a) widening of expanding angle of the expander and (b) pre-loading technique at installation. The influence of cleaning method on the stiffness and strength is discussed for the adhesive chemical anchor.

2. BACKGROUND

Since the pull-out resistance of post-installed anchors was unreliable, such anchors have not been used in connections where they would resist pull-out actions. When post-installed anchors were used for the repair of seismically damaged structures or the seismic retrofit of existing structures, the number of anchors was determined on the basis of direct shear strength of the cross section of anchor bolt. The dowel resisting mechanism was solely expected as a role of anchors[2].

However, Shiohara et al.[3] pointed out the importance of the pull-out resistance of post-installed anchors on the basis of tests of post-cast shear wall specimens. The joints between a post-cast shear wall panel and its surrounding frame were carefully constructed to develop monolithic characteristics (Fig.1); (a) the column and beam surface was roughened more than 5 mm in depth before placement of the post-cast wall concrete, (b) no-shrink mortar was injected into the space under the beam after casting the wall panel, (c) the post-installed anchors were provided with sufficient lapping length within the wall panel and anchorage length within the boundary beams and columns. The post-cast shear wall specimens, constructed in this manner, developed the stiffness and resistance equivalent to the monolithic shear frame-wall specimen.

An analytical method was developed by Shiohara[4] to calculate the resistance of a monolithic shear wall with the boundary frame using the truss analogy with tie and strut forces. The same analytical method was applied in the analysis of post-cast shear wall specimens (Fig.2), and the pull-out resistance of the anchor bolts was a conclusive factor to determine the shear wall strength.

Thus, the post-installed anchors should be able to develop full yield stress of the connecting bars in tension to improve the performance of post-cast shear walls.

3. ANCHOR SPECIMENS FOR PULL-OUT TESTS

3.1 Metal Expansion Anchors

Three types of metal expansion anchors were tested.

The DE anchor (Fig.3.a) is an ordinary type in Japan. It resists pull-out action by friction and interlock mechanism between concrete and expander surface at the top of an anchor bolt. The expander was expanded by knocking the plug of an angle of 7 to 8 degrees. It was pointed out that the pull-out stiffness and strength of this type of anchors were likely to scatter, caused by concrete rigidity or the condition of expander.

The UC anchor (Fig.3.b) was developed to increase the interlock with concrete by enlarging the expanding angle of a plug. The bottom of installing hole was enlarged in a cone shape with a special drill so that the expander could easily expand to fit into the hole at a wider angle.

The BU anchor (Fig.3.c) was specially prepared to study the influence of the plug base length. The standard plug base length was 5 mm, while in the BU anchor, the length was deliberately made longer to 15 mm. With the ordinary type of anchor, pieces or powder of concrete crushed by knocking were sometimes observed to pile at the bottom of the hole and interfere with the expansion of the expander. With an additional length in stud of the BU anchor, the expander could be expected to expand normally to improve the anchorage.

The material properties in the tests are summarized in Table 1.

3.2 Adhesive chemical anchors

Adhesive chemical anchors usually used for seismic strengthening in Japan are of capsule type with polyester or epoxy compounds. The anchor embedded in concrete is illustrated in Fig.3.d.

Standard embedment length was chosen to be 8 times bar diameter. Deformed bars of grade SD30 (nominal yield strength: 30 kgf/mm^2) and a nominal diameter of 16 mm were used as anchor re-bars. The diameter of drill edge used in drilling installing holes was 20 mm. Assuming the pull-out resisting mechanism of adhesive anchors was similar to the bonding mechanism of deformed bars, the bond stress distribution along anchor re-bar was determined from the measured strains along the anchor. An embedment length of 12.5 times anchor diameter was given to some specimens to investigate an influence on bond characteristics.

The material properties in the tests are summarized in Table 1.

4. TESTING PROCEDURE

Anchor specimens were installed in a concrete block as shown in Fig. 4. The dimensions of the concrete block were 1200 x 1200 x 450 mm or 900 x 900 x 400 mm, large enough to avoid the influence by flexural cracks or partial cracks near an edge.

The loading system is illustrated in Fig. 5. Each specimen was pulled by a 20-ton center hole hydraulic ram. Pull-out displacement was measured by electric displacement transducers. The diameter of the reaction stand was wide enough to avoid confinement of the concrete in the region of a diameter equal to the depth of the anchor (Fig. 5.a).

Bond tests of adhesive chemical anchors were carried out by the apparatus shown in Fig. 5.b. Bond of the re-bar was cut from the concrete surface to 50 mm in depth in order to eliminate the influence of confinement from the loading apparatus near the concrete surface.

In the pull-out test of adhesive chemical anchors, three cleaning methods of the installing hole were chosen as a main parameter to influence the bonding stiffness and strength; i.e., (a) vacuuming the hole, (b) brushing the concrete surface in the hole with a nylon brush after vacuuming and (c) brushing with a wire brush after vacuuming. The diameter of a wire brush fitted to an electromotive drill was 5 mm larger than the diameter of an installing hole.

Effect of cleaning methods similar to the above has already been investigated by Luke et al.[5].

5. TEST RESULTS

5.1 Metal Expansion Anchor

PULL-OUT LOAD - DISPLACEMENT RELATIONSHIP: Typical pull-out load and displacement relations of the DE anchor and the UC anchor are compared in Fig. 6. In the DE anchor, large pull-out displacement was observed as load increased; displacement at maximum resistance was as large as 15 mm, and the decay in resistance after the maximum was drastic. While in the UC anchor, both initial stiffness and maximum resistance were high; the decay in resistance after the maximum was gradual. In both anchors, cracks in concrete surface were not observed at the maximum resistances. Initial cracks were observed at a pull-out displacements of 16 mm.

Influence of embedment length or concrete strength to pull-out load displacement relationship is demonstrated in Fig. 7. A horizontal line in the figure represents the yield strength of joining re-bar of the test. The resistance of the DE anchor did not reach the yield strength, when the embedment length was less than seven times the bar diameter. On the other hand, the pull-out resistance of the UC anchor exceeded the yield strength of the joining re-bar, when the embedment length was more than five times bar diameter.

The behavior of a UC anchor under one way loading and unloading simulating seismic situation is shown in Fig. 8. Loading and unloading were repeated five times at the design load and then five times at twice the design load. The load-displacement relation was almost linear during unloading and reloading. A draw-back of the metal expansion anchor is the scattering of its stiffness and strength values, but can be corrected by making use of the preloading to a given level.

ASSURANCE LOAD: Stiffness under reloading is compared in Fig. 9. The ordinate represents tension load and the abscissa pull-out displacement shifting residual displacements under one way loading reversals to the origin. The reloading stiffness up to the previous maximum load coincided with the initial elastic stiffness. This simple but important characteristic can be applied to correct unreliable magnitude in stiffness and strength of metal expansion anchors; i.e., after installing an anchor, the preloading should be applied to a certain load level before the usage. The load level, called assurance load, may be decided in relation to the design load.

ESTIMATION OF PULL-OUT STRENGTH: Elasticity under reloading as shown in Figs. 8 or 9 implies perfect interlock between concrete and anchor. An idealized interlocking condition is illustrated in Fig. 10. Friction force and bearing force were assumed to act on the surface between the anchor and concrete. The concrete around the anchor would fail in bearing under the pull-out action, but the ground concrete powder produced by crushing would be compacted and hardened enough to resist the pull-out action[6]. As the pull-out action increased, bearing region of the concrete would spread and finally the pull-out resistance would reach the maximum. Pull-out strength due to such bearing resisting mechanism T_1 was assumed to be presented by the following equation:

$$T_1 = N \cdot \mathrm{Sin}(\alpha + \phi)/\mathrm{Cos}\ \phi$$

Where, N: bearing strength of concrete (assumed equal to $12\sigma c$, in which σc: compressive strength of concrete), α : angle of the plug, and ϕ : Angle of friction (Tan $\phi = \mu$, μ was assumed equal to 0.4).

The pull-out strength could be defined for the cone failure given in the following equation:

$$T_2 = Ac \cdot \sqrt{\sigma c} \quad \text{(Unit: kgf, cm)}$$

Where, $Ac = \pi \cdot le \cdot (le + da)$,
$le = l - da$,
T_2: pull-out strength due to cone shape failure, Ac: effective projected area of cone shape, σc: compressive strength of concrete, l: embedment length, le: effective length, and da: anchor diameter.

Calculated pull-out strengths for the cone shape failure and the bearing failure are compared in Fig. 11. The abscissa represents the concrete strength. In the cone shape failure, the embedment length was varied from five to nine times anchor diameter. In the bearing failure, the angle of plug was varied 2 to 14 degrees. In case of shallow embedment and small plug angle, the pull-out strength was decided by the cone shape failure, while in case of deep embedment and large plug angle, it was decided by the bearing failure.

The change in observed pull-out strength and failure mode by embedment length is plotted in Fig. 12. Test data were grouped by the embedment length. Horizontal lines are calculated strength for each failure mode. For both DE and UC anchors, only cone shape failure was observed when embedment length was short. Fracture of anchor re-bar was observed when embedment length was 9 times diameter in the DE anchor and 7 times in the UC anchor. Bearing failure was observed only in the UC anchor when the embedment length was more than 7 times diameter. Therefore, the pull-out strength of UC anchor with deep embedment length must be calculated based on bearing failure .

INFLUENCE OF PLUG BASE LENGTH The pull-out load - displacement relation of a BU anchor is compared with that of a DE anchor in Fig. 13. The pull-out stiffness of the BU anchor was significantly low. This low stiffness was caused by concrete failure around the expander during the installation. The elongation of plug base was not effective to improve the stiffness and strength of a metal anchor. However, the preloading technique was confirmed effective to insure a required stiffness for such type of an anchor.

5.2 Adhesive Chemical Anchor

BOND BEHAVIOR The location of strain gauges to investigate bond performance is illustrated in Fig. 14.a. Observed strain distribution for each cleaning method is shown in Fig. 14.b and c. Strain of each specimen distributed linearly along anchor re-bar at lower load steps, demonstrating uniform bond stress. However, in higher load steps, the effect of cleaning method was revealed in the strain distribution. Anchor re-bar did not yield in the case of only vacuuming, while the re-bar yielded in the case of brushing with nylon brush or wire brush.

Pull-out load-displacement relations observed for each cleaning method are shown in Fig. 15. In the case of cleaning by vacuuming only, significantly large displacement occurred before anchor re-bar yielded. Anchors whose hole was cleaned by brushing developed high stiffness and strength. This accounted especially for the wire brush.

Average of bond strength for each cleaning method was 100 kgf/cm^2 when only vacuuming, 125 kgf/cm^2 when brushing with a nylon brush and 140 kgf/cm^2 when brushing with a wire brush.

<u>FAILURE MODE</u> Observed failure modes of adhesive chemical anchor under a pull-out action are generally classified as the following: (1) cone shape failure, (2) bond failure, (3) combined mode of the above (1) and (2), (4) adhesive fracture and (5) anchor re-bar fracture. In case (2), the pull-out strength was lowest.

The range of bond stress of specimens failed in combined mode of the cone shape failure and the bond failure in this series of tests was 108 - 119 kgf/cm^2 for an effective embedment length.

<u>ESTIMATION OF PULL-OUT STRENGTH</u> A cross section of specimens failed in the cone shape failure is illustrated in Fig. 16.a. The depth of a cone was measured by vernier at every 2 cm. Based on this observation, each length necessary to calculation of pull-out strength was defined as shown in Fig. 16.b.

Supposing the combined failure mode of the cone shape failure and the bond failure as a general failure mode, the pull-out strength of an adhesive chemical anchor P was defined as the sum of strengths of each failure mode:

$$P = Pc + Pa$$

Where, $Pc = \pi \cdot l_1 \cdot (l_1 + D) \cdot \sqrt{\sigma c}$,
$Pa = \pi \cdot D \cdot l_2 \cdot \tau a$,
$l_1/le = 0.46$ (observed average value),
Pc: pull-out strength due to cone shape failure, Pa: pull-out strength due to bond failure, l_1: cone depth, l_2: bond length, le: effective embedment length and τa: bond stress (assumed 125 kgf/cm^2 here).

Observed pull-out strength and calculated strength are compared in Fig. 17. The pull-out strength may be predictable by the method.

6. CONCLUSIONS

Post-installed anchors were difficult to use for seismic retrofit of important structures because of the lack of reliability in their performance. However, a series of pull-out tests proved that the reliability could be improved. Such improved anchors will contribute to the increase of seismic resistance of strengthened structures especially when used for post-cast shear walls.

Main findings obtained from pull-out tests of metal expansion anchors are summarized as follows;

(1) Hardening interlock between concrete and the top of anchor by pulling out up to a necessary load makes it possible for a metal expansion anchor to generate elastic stiffness surely in load range below the pull-out load. The finish of this loading work should be regarded as completion of anchor construction.

(2) The undercut type anchor (UC anchor) with wide angle expander installed into the hole whose bottom was drilled to spread in cone shape develops higher stiffness and strength.

(3) When estimating pull-out strength of metal expansion anchor, it is necessary to suppose not only cone shape failure but also bearing failure of concrete near the top of anchor.

Test results of adhesive chemical anchors are concluded as follows.

(1) In order to assure the adhesive chemical anchor develops pull-out strength up to yield strength of its re-bar, it is necessary to certify the bond strength is more than 120 kgf/cm^2 by field bonding test.

(2) Cleaning the installing hole is important to make the anchor generate sufficient stiffness and strength. Brushing with wire brush whose diameter is larger than the hole diameter is very effective.

7. ACKNOWLEDGMENT

Japan Decoluxe Co., LTD. and Japan Drive-It Co., LTD. assisted in carrying out a series of tests reported in this paper. Dr. Shunsuke OTANI, Associate Professor, Department of Architecture, University of Tokyo, gave helpful advices for test-planning. Mr. Akira TASAI, Research Associate, Department of Architecture, University of Tokyo, co-operated in preparing the manuscript. The authors acknowledge gratefully these contributions.

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Table 1 Properties of Material

Metal Expansion Anchors		
Compressive Strength of Concrete	kg/cm2	160, 170, 250, 300
Yield Strength of Anchor Bolts	kg/cm2	UC Anchor:5800 DE Anchor:6300
Diameter of Anchor Bolts	mm	16
Embedment Length	mm	48, 80, 112, 144
Minimum Cross Section	cm2	UC Anchor:145.6 DE Anchor: 99.8
Testing Method	Pull-out Test	
Adhesive Chemical Anchors		
Compressive Strength of Concrete	kg/cm2	183, 220
Yeild Strength of re-bar	kg/cm2	3720
Embedment Length	mm	80, 128, 200
Testing Method	Bond Test, Pull-out Test	

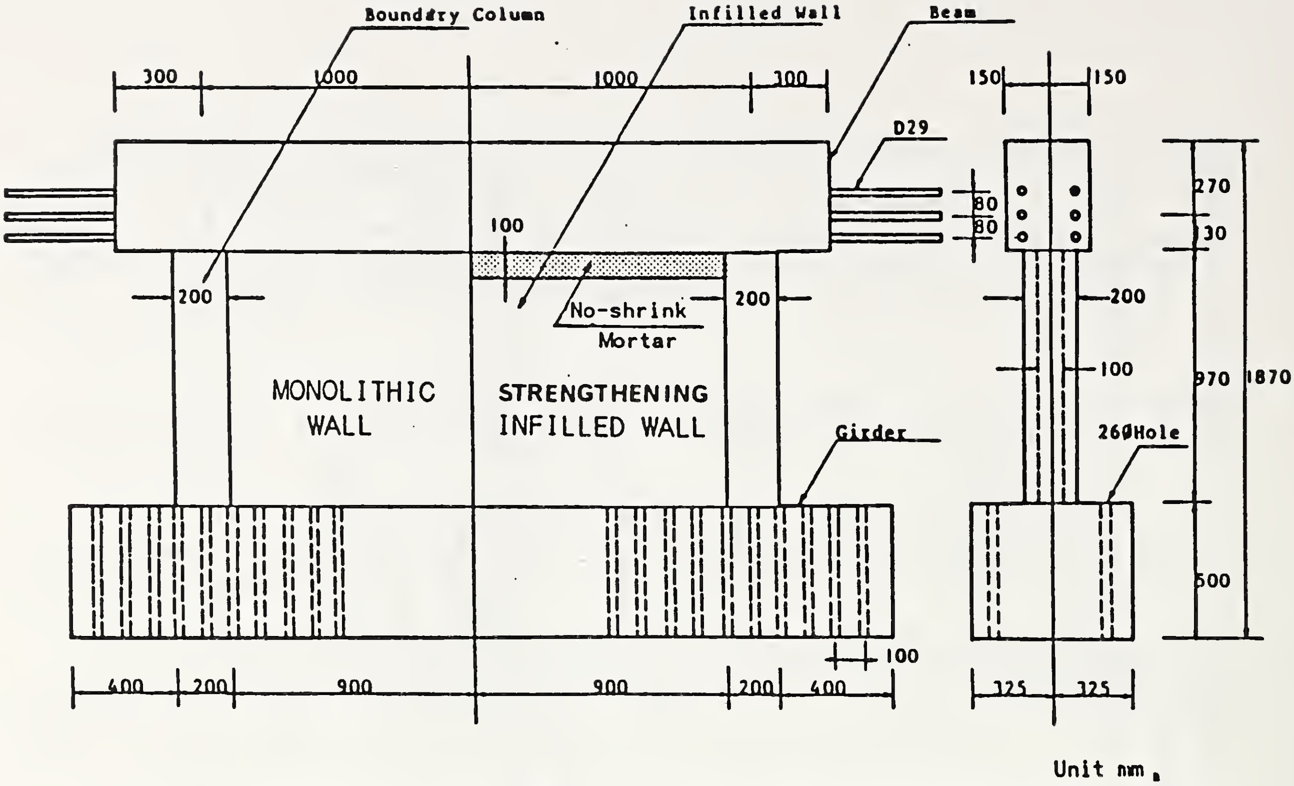


Fig. 1 (a) Elevation View of Monolithic and Post-cast Shear Walls

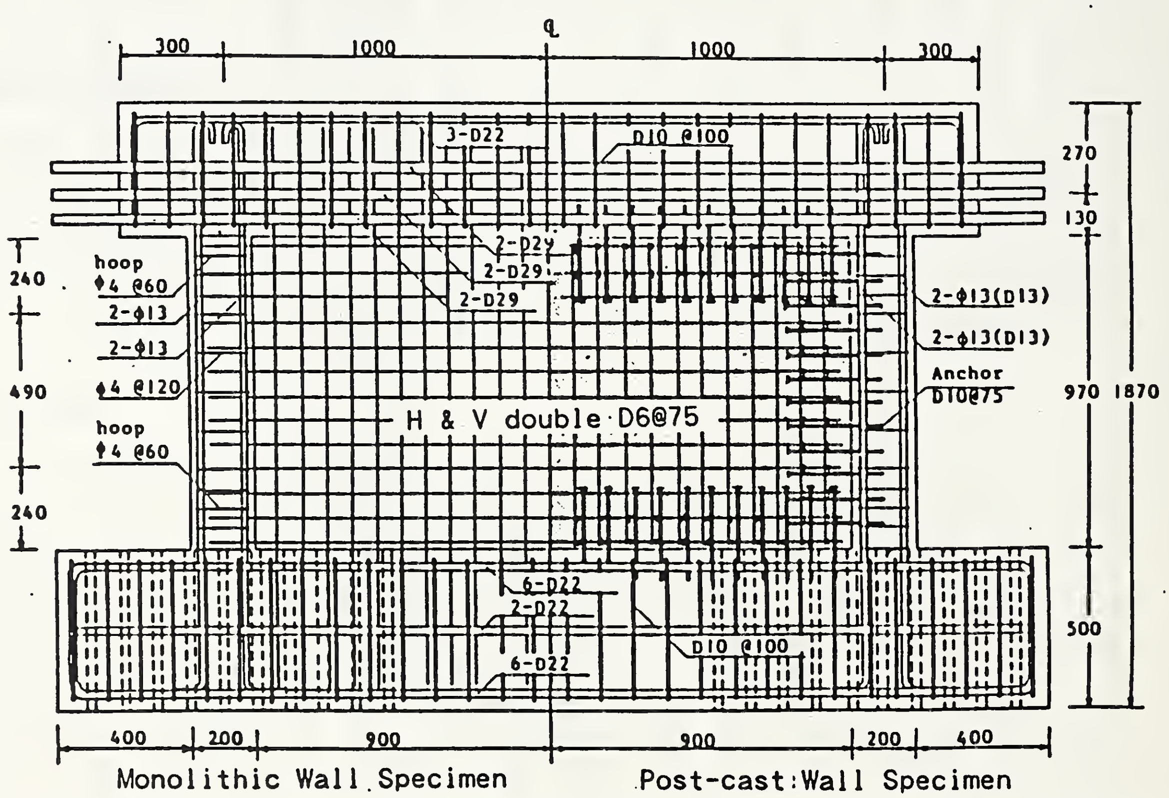
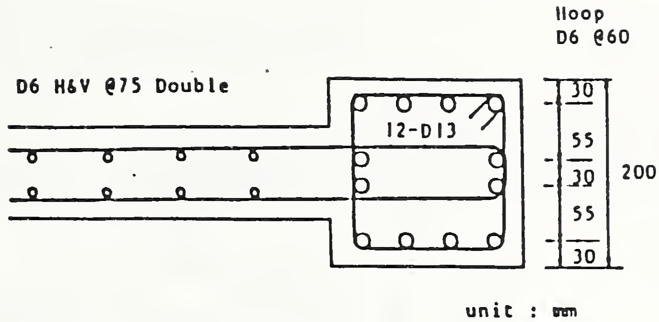
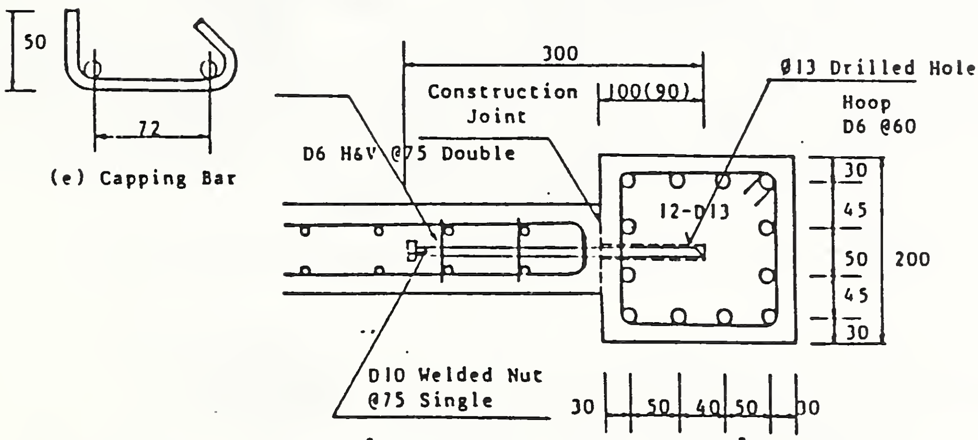


Fig. 1 (b) Reinforcement Details of Monolithic and Post-cast Shear Walls



[Monolithic Shear Wall]



[Post-cast Shear Wall]

Fig. 2 (a) Cross Section of the Two Types Shear Walls

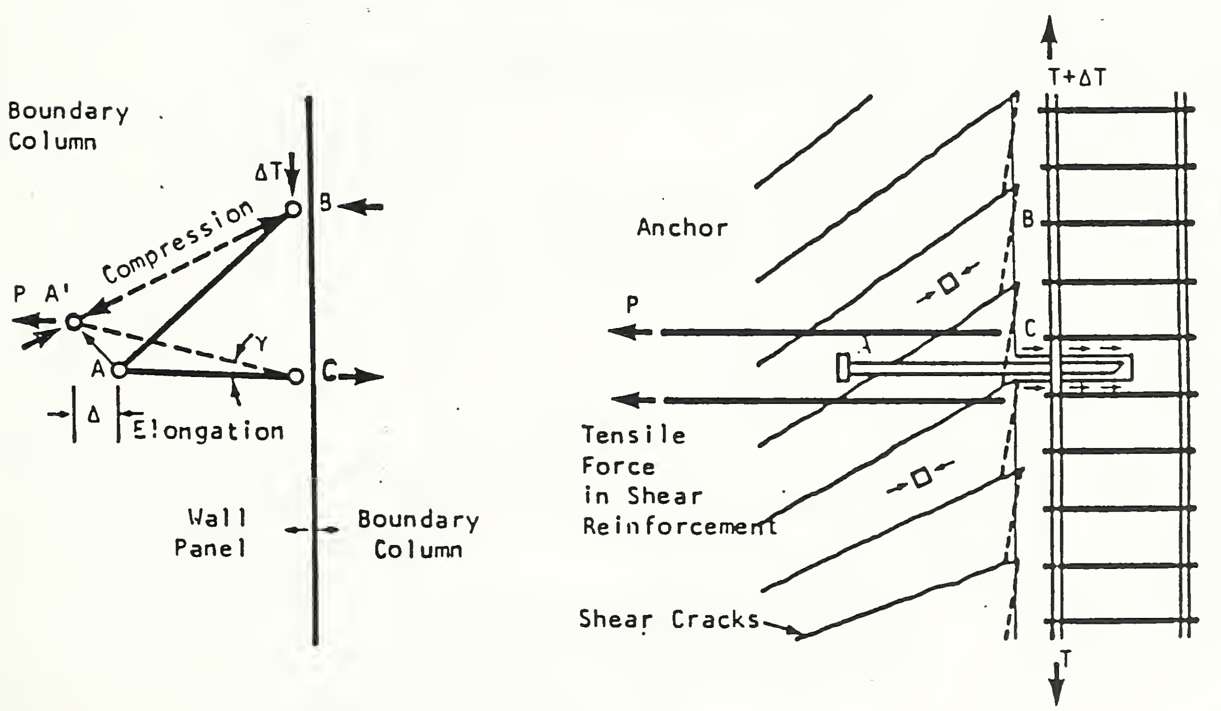
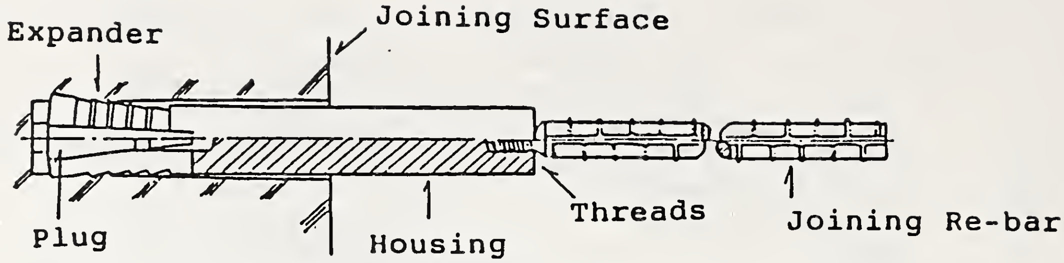


Fig. 2 (b) Transfer Mechanism of Tension



Terms in Metal Expansion Anchor

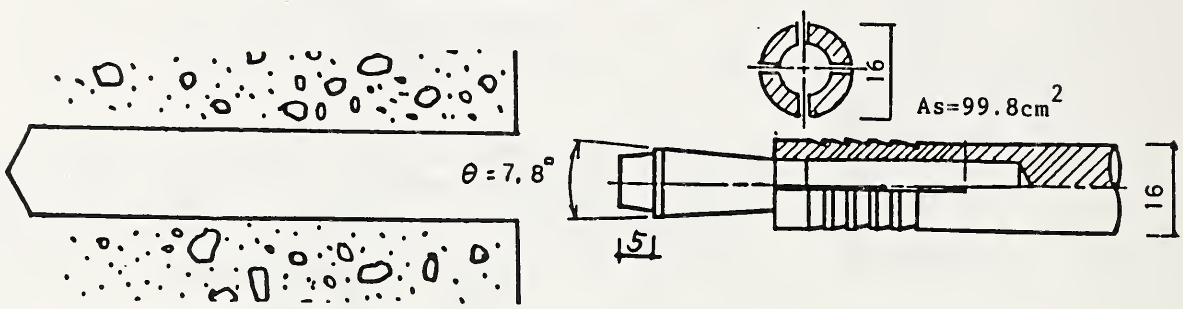


Fig. 3a Typical Metal Expansion Anchor=DE Anchor

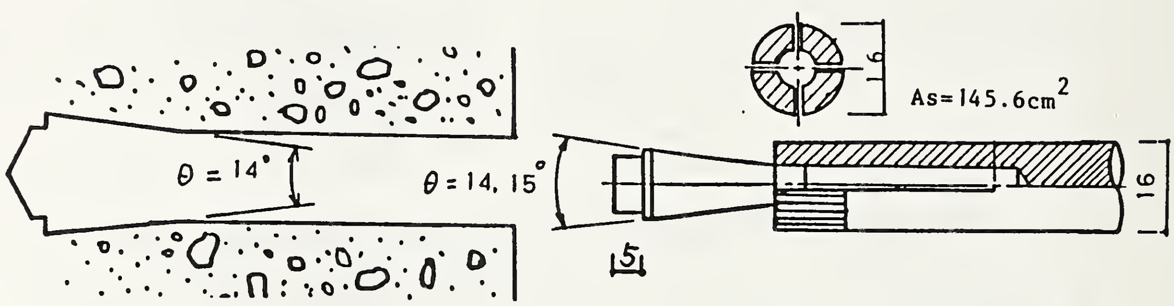


Fig. 3b Undercut Anchor=UC Anchor

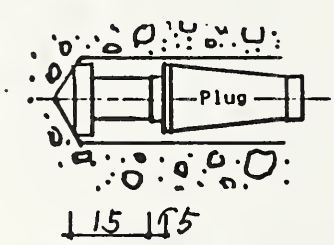


FIg. 3c BU. Tyoe Anchor

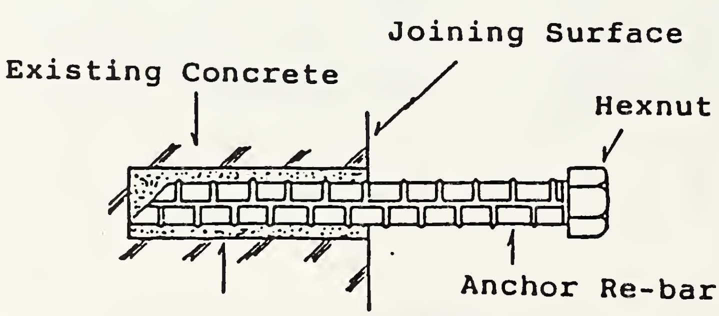
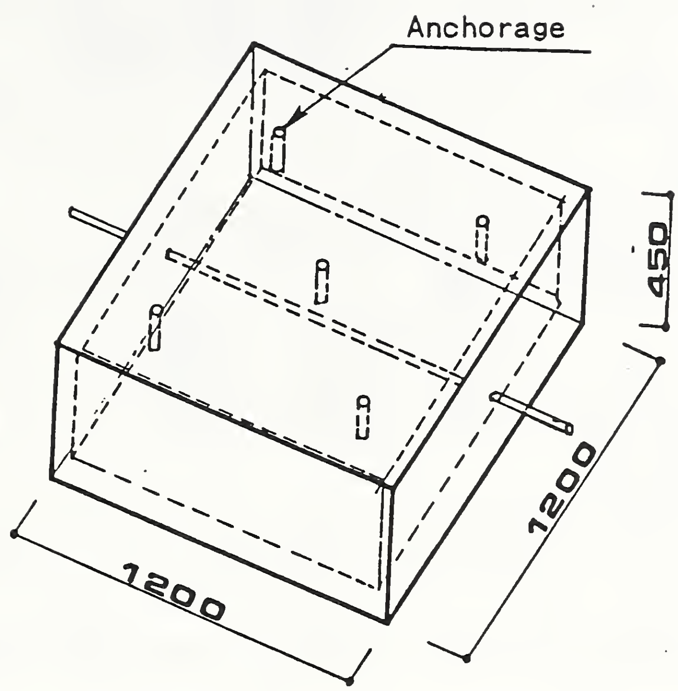


Fig. 3d Adhesive Chemical Anchor

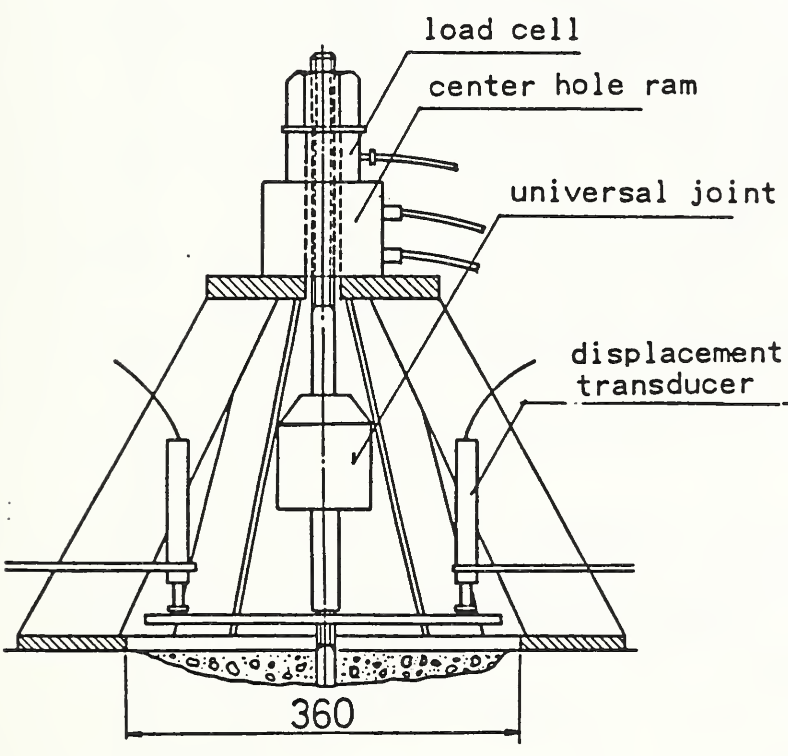
Unit mm

Fig. 3 Detail of Post-installed Anchor

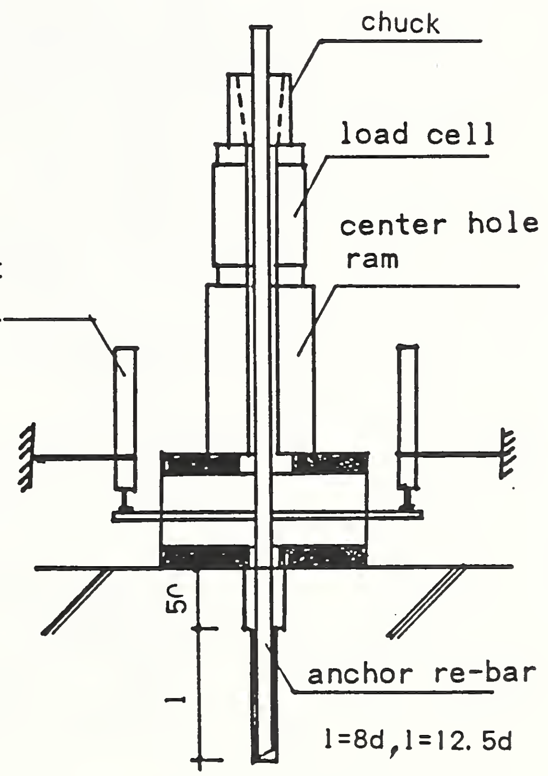


Unit mm

Fig. 4 Concrete Test Block Installed with Anchors



(a) Pull-out Tests



(b) Bond Tests

Unit mm

Fig. 5 Loading System

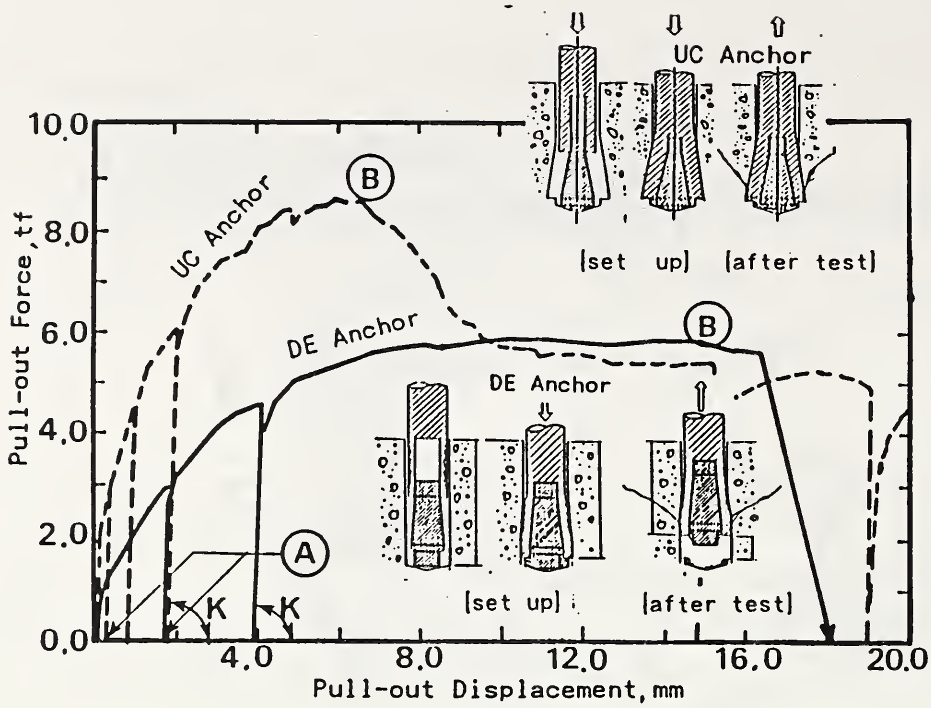
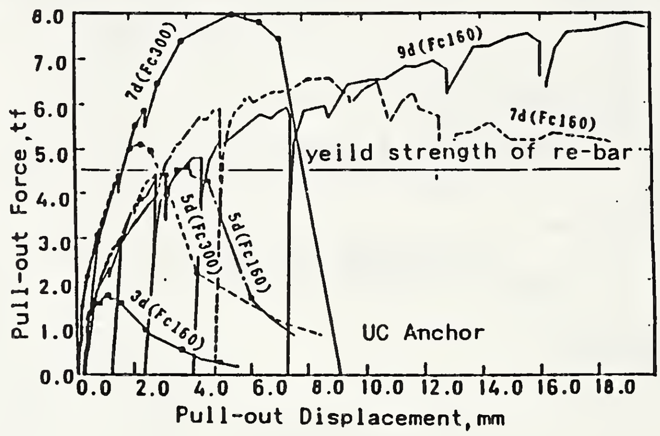


Fig. 6 Comparison between DE Anchor and UC Anchor



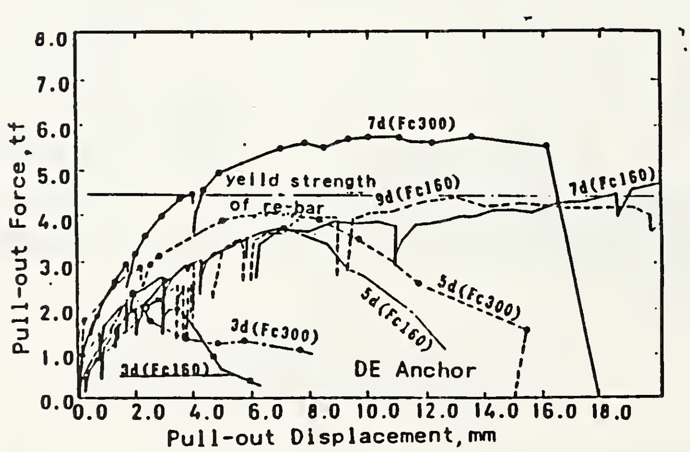


Fig. 7 Effect of Embedded Length and Concrete Strength

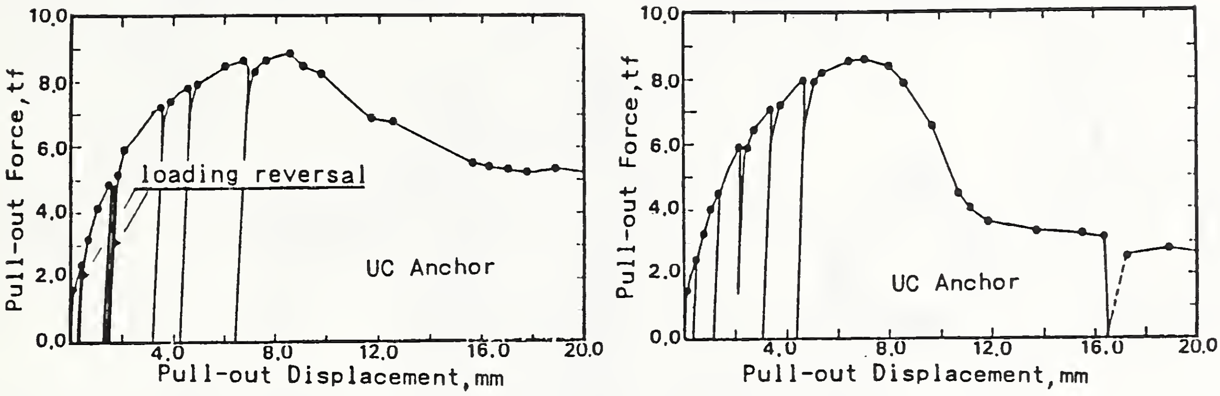


Fig. 8 Restoring Force Characteristics of Metal Expansion Anchor

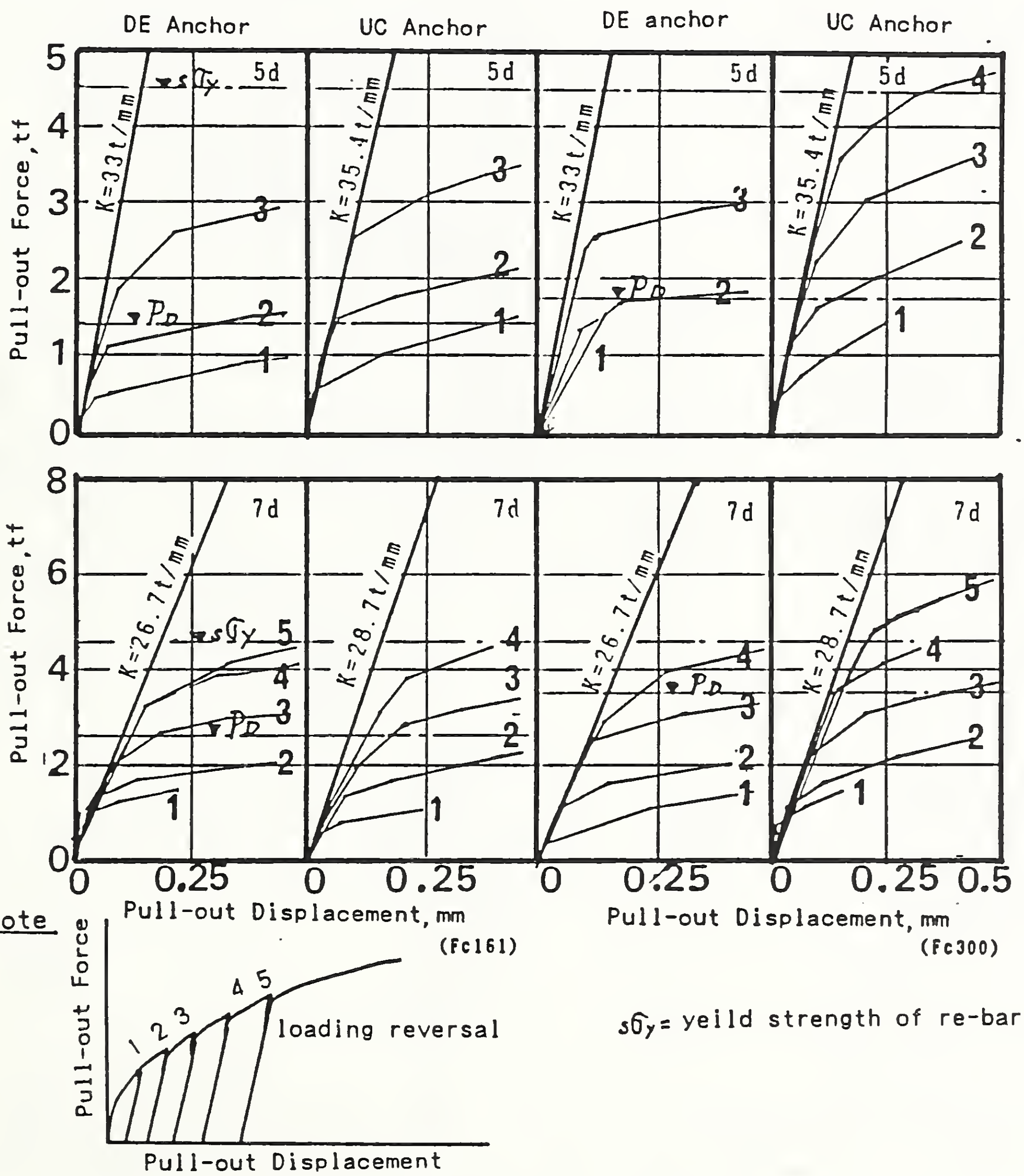


Fig. 9 Comparison of stiffening for Restoring Force

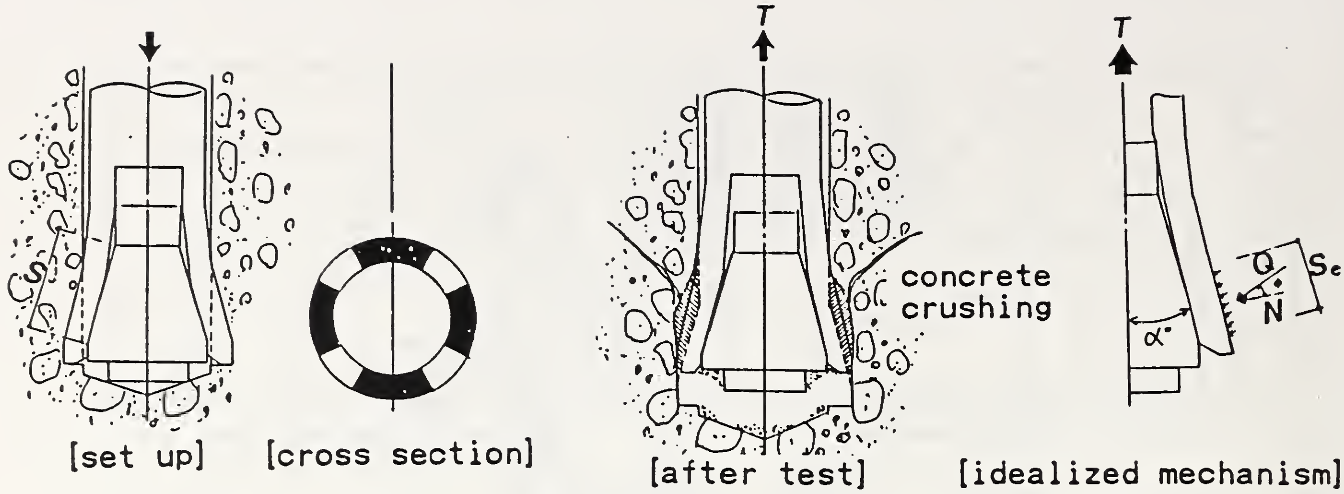


Fig. 10 (a) Tensile Mechanism of Expansion Anchor

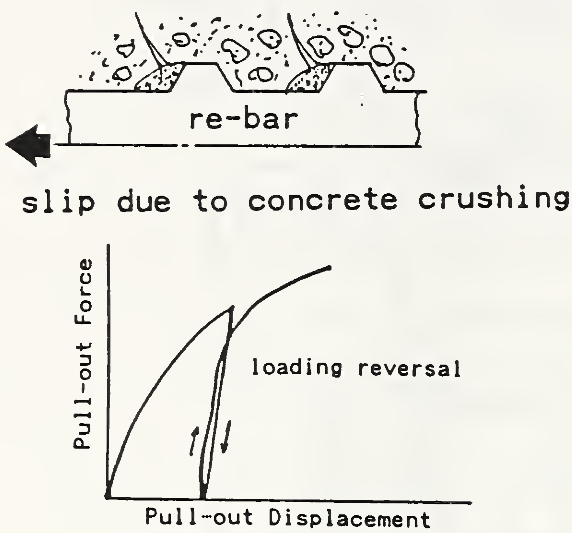


Fig. 10 (b) Bond Failure Mechanism

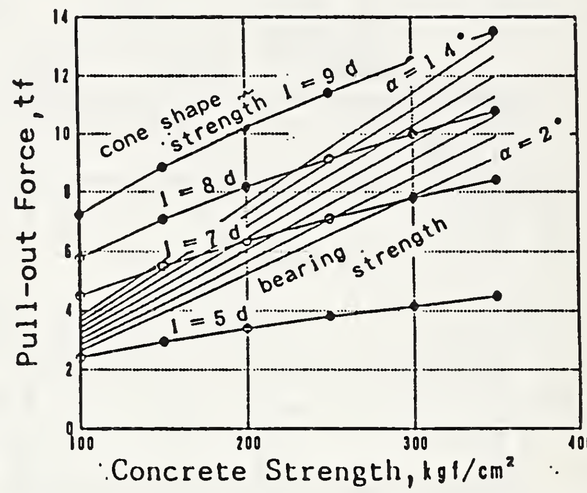


Fig. 11 Pull-out Load and Concrete Compressive Strength Relationship

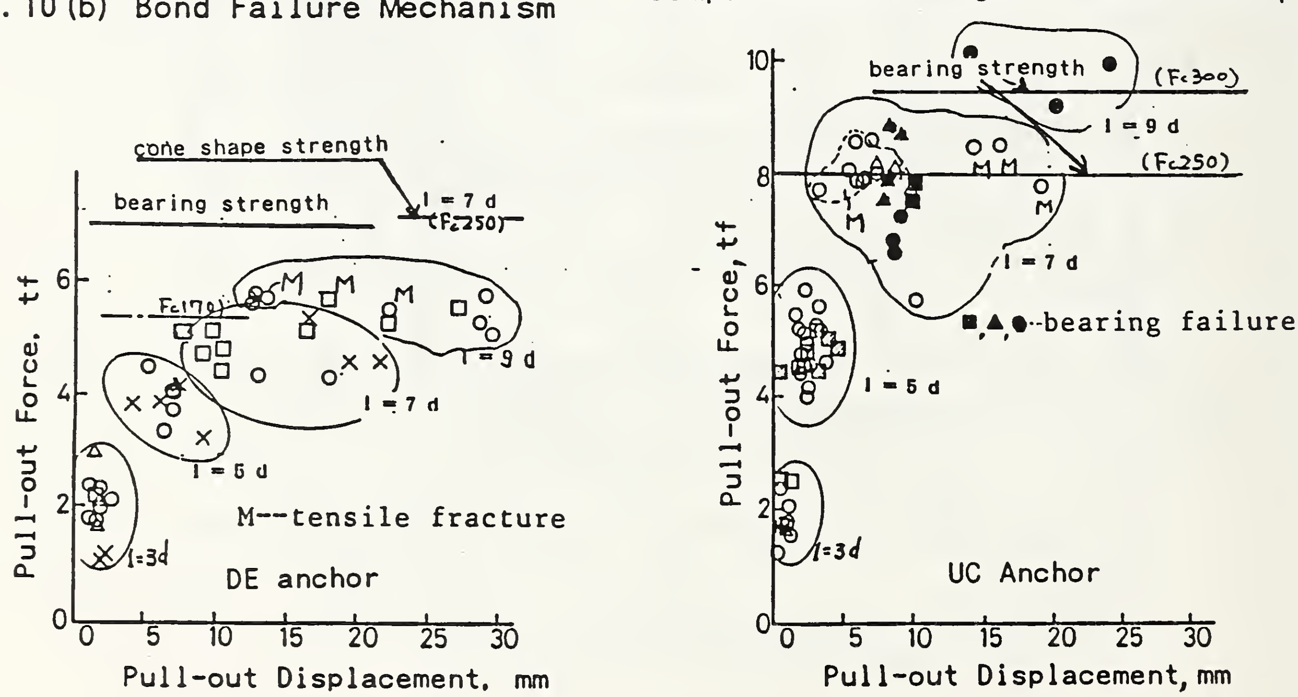


Fig. 12 Effect of Embedment Length on Pull-out Displacement at Maximum Resistance

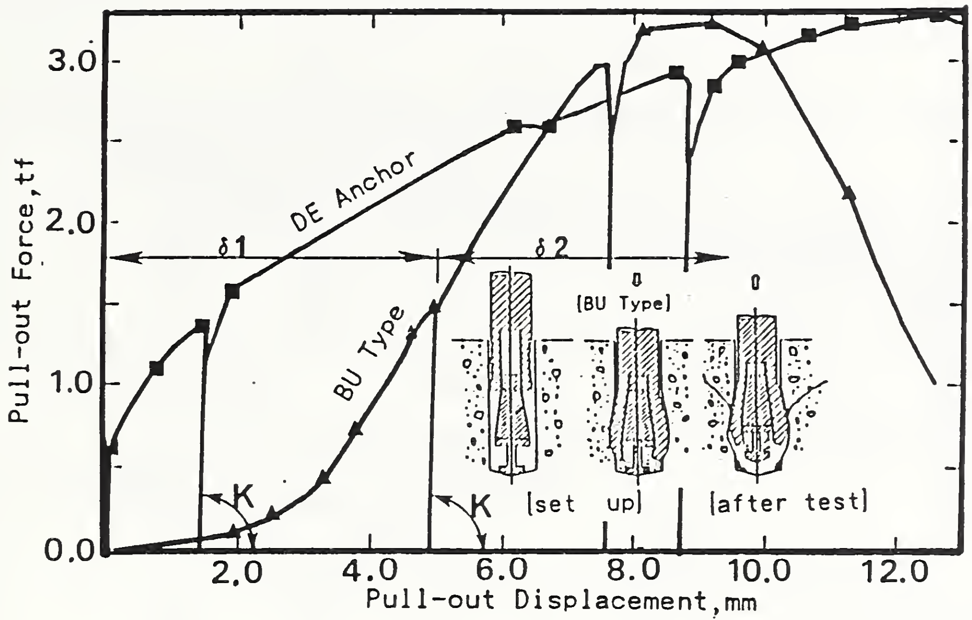


Fig. 13 Comparison of Load-slip Relationship for Typical Anchor and BU Type Anchor

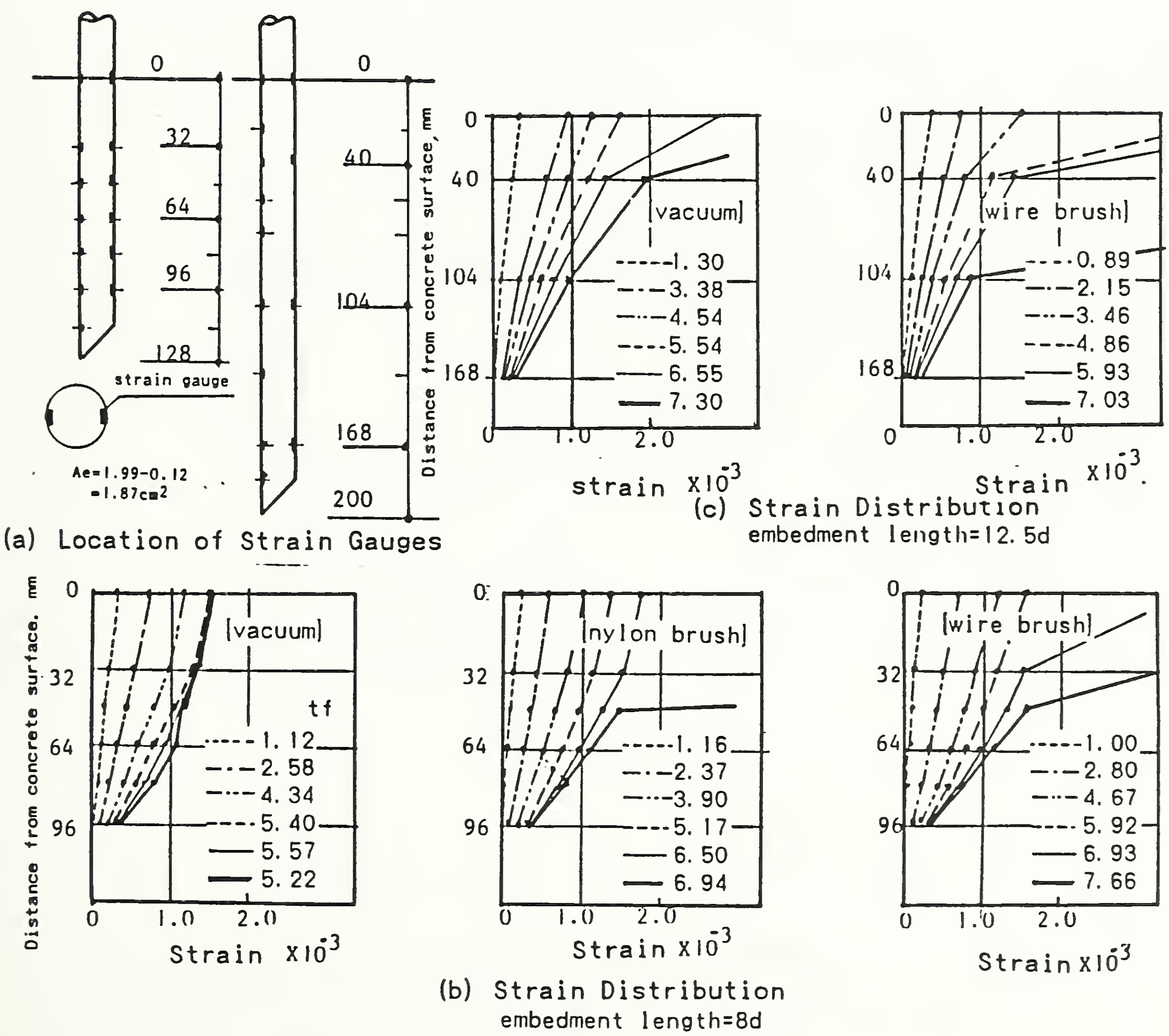
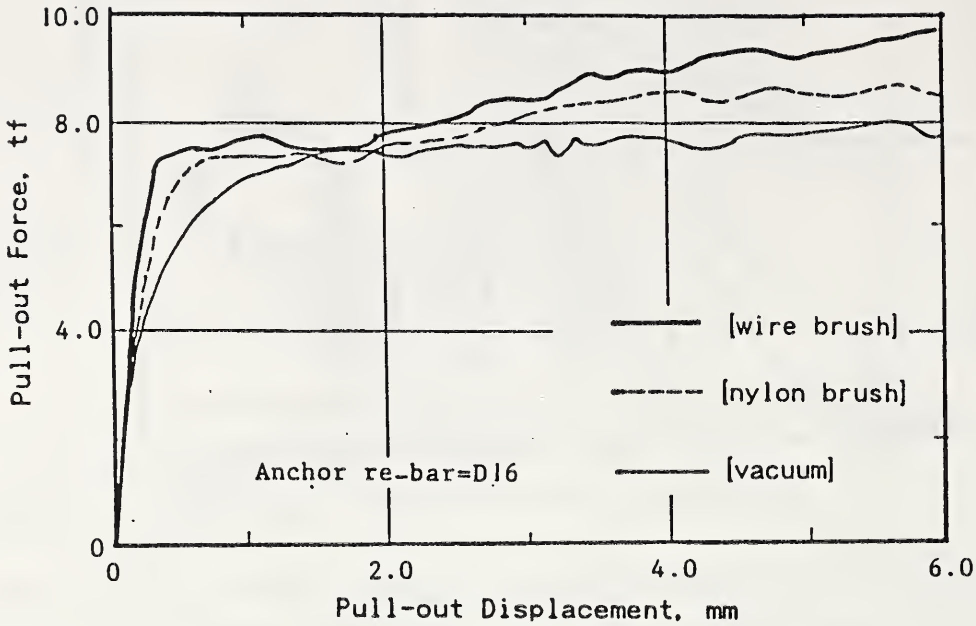
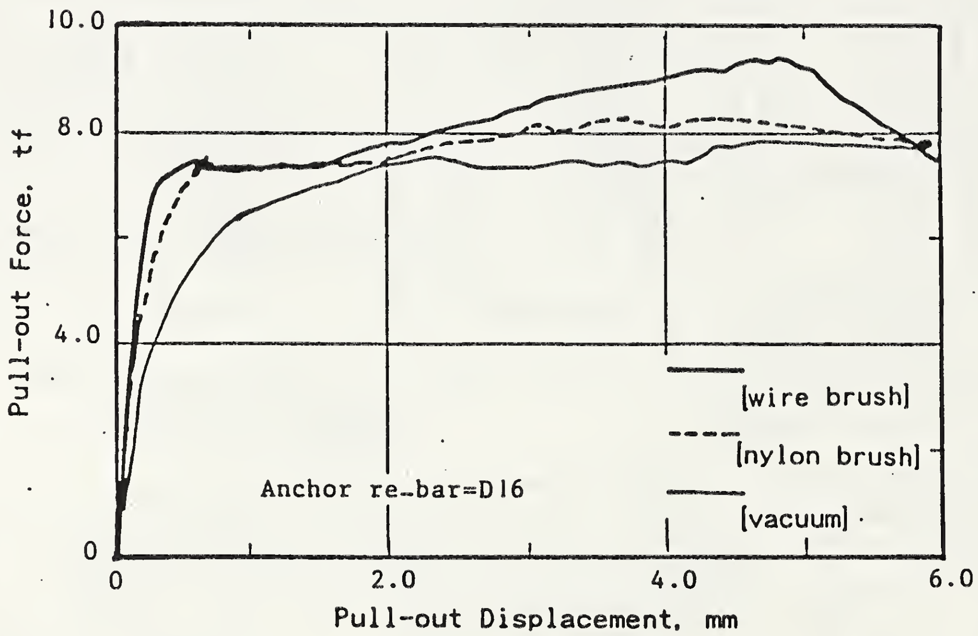


Fig. 14 Strain Distribution in an Embedded Re-bar



(b) Bond Tests



(a) Pull-out Tests

Fig. 15 Effect of Cleaning Methods on Load-slip Relationship Chemical Anchors

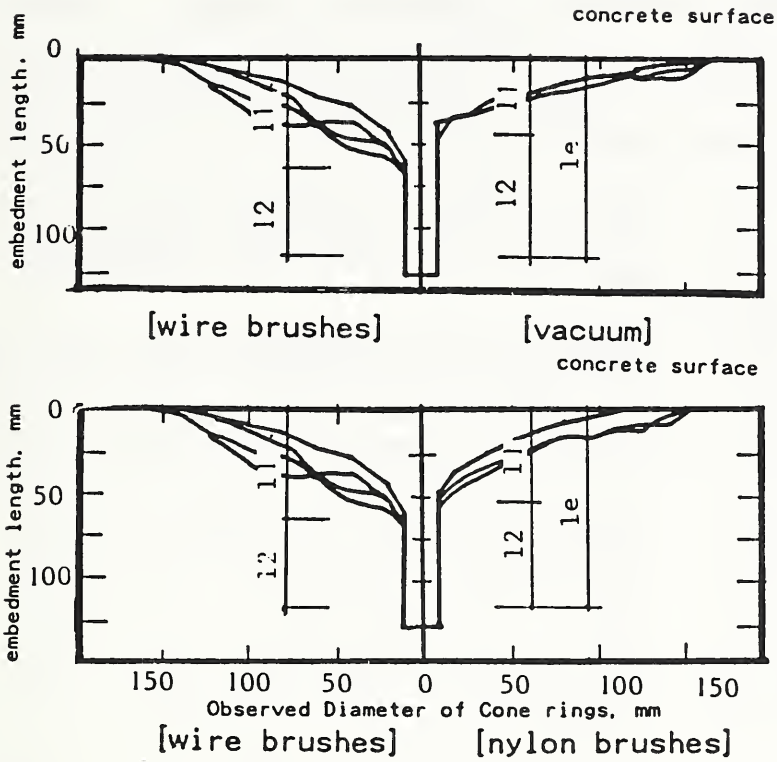
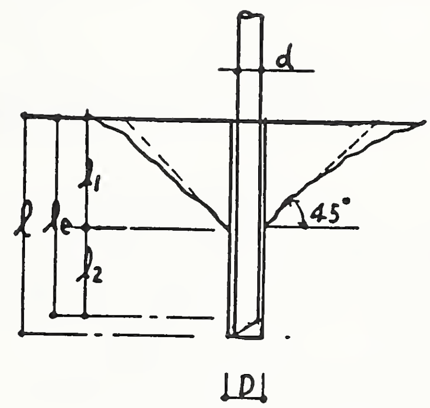


Fig. 16 (a) Observed Failure Patterns for Each Cleaning Methods



$$P = P_c + P_a$$

$$P_c = \pi \cdot l_1 \cdot (l_1 + D)\sqrt{F_c}$$

$$P_a = \pi \cdot D \cdot l_2 \cdot \tau_a$$

$$l_1 / l_e = 0.46$$

Fig. 16 (b) Idealized Combination Cone and Bond Failure

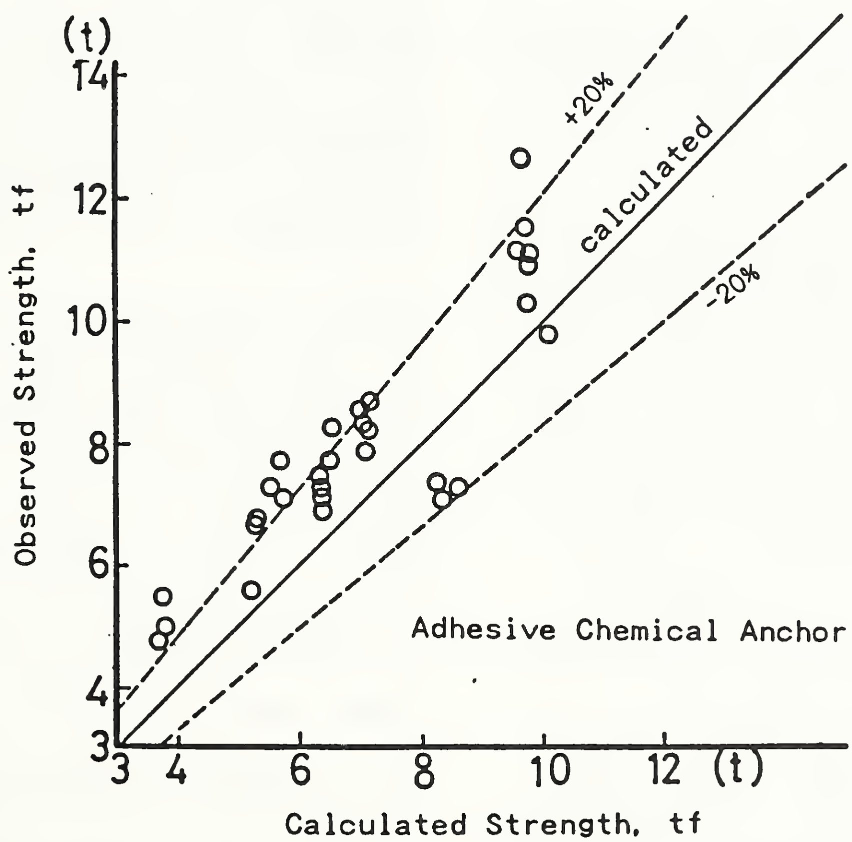


Fig. 17 Comparison between Calculated Strength and Observed Strength

REPAIR, RETROFIT AND STRENGTHENING OF STEEL BUILDINGS AND BRIDGES

by

Le-Wu Lu[1] and Ben T. Yen[1]

ABSTRACT

An overview of the problems and approaches related to repair, retrofit and strengthening of steel building and bridge superstructures is presented, with emphasis on experience gained in field applications. The methods and techniques that have been used to strengthen individual members, connections and frames in building structures are reviewed. Some of the phenomena of damage in bridges, including corrosion, fatigue and brittle fracture of components, and distortion induced fatigue cracking are examined, and a rational procedure for repair or retrofit is introduced. Several methods of bridge rehabilitation, which are to be applied in accordance with the cause of damage, are described.

KEYWORDS: Steel; building structures; bridges; repair; retrofit; strengthening; corrosion; fatigue; fracture.

1. INTRODUCTION

Structures in service are often repaired, retrofitted or strengthened for different reasons, under different circumstances, and by different techniques or approaches. Repair is to restore or regain the lost strength because of damage, decay, or aging. Retrofit has a broader meaning and usually implies a change in design or construction of a structure in service in order to incorporate later improvements. Strengthening is to upgrade or increase the strength and stiffness of an existing structure with a goal to satisfy a more stringent demand on safety or performance. For the purpose of this paper, these terms will be used interchangeably to indicate processes whereby the load-resisting characteristics of a structure, whether or not damaged, are improved. Also, the word "strengthening" will be used to describe all the three procedures because some increase in strength is likely to result.

Decision to strengthen an existing building and the level to be achieved are usually made after a careful study of many relevant factors, some of which are non-engineering. One of the factors is obviously the cost, another is the importance (functional, historical, etc.) of the building. In most instances, the cost of strengthening is compared with the cost of constructing a new structure at the same site. Except in some obvious cases, the decisions are usually based not solely on cost and such other factors as time required to complete the work, resulting improvements (structural as well as architectural and mechanical), and disruption of the building's services must also be evaluated. On the engineering side, the level of strengthening that can be achieved with a reasonable cost is an important issue and this level, in some instances, is affected by the load-carrying capacity of foundation or substructure.

Bridge superstructures are designed according to design specifications, constructed with available materials and using prevalent construction procedure, and accepted into service with anticipated maintenance of the bridges. In many locations, traffic volume and weight intensity have increased many folds in fifteen to twenty years and very little maintenance work was carried out, contributing to the rapid deterioration of the bridges. For some bridges, deficiency in design, fabrication or construction combined with inferior materials resulted in unexpect-

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ed damages.

This paper presents an overview of the methods that have been used or proposed to strengthen or rehabilitate steel building and bridge superstructures.

2. EVALUATION OF BUILDING STRUCTURAL CONDITION

Kaminetzky (1985) outlined the following steps in the process of evaluating the existing strength of a concrete building structure considered for rehabilitation:

1. Study the history of the structure.
2. Perform a condition survey that will identify defects such as excessive deflections, cracking, fracture, spalls, or corrosion of metals.
3. Establish or verify basic geometry of the frame and its members. Survey and measure in the field or obtain from existing plans (when available) supplemented with careful field verification.
4. Establish or verify shape, location, size and cross section of embedded elements, such as rebars or structural steel sections.
5. Establish strength of materials such as concrete, steel or masonry by reviewing existing construction records supplemented by a combination of non-destructive and destructive tests.
6. Compute the present structural capacity of the structural frame, its components and its foundations.
7. Perform a full-scale load test of elements or an entire structural section in the event questions and doubts still persist regarding the true and actual strength and the load-carrying capacity.
8. Establish the desired use and its live load requirements, and determine whether the existing structure has to be upgraded to higher live loads.
9. Select the best repair method and construction techniques suitable for rehabilitation.

Most of these are equally applicable to a steel building and to the situations where strengthening is to be carried out to increase the lateral load-carrying capacity.

3. REPAIR AND STRENGTHENING OF BUILDING STRUCTURES

3.1 Structural Members

In the process of evaluating the existing strength, there is often the need to know the distribution of stresses in certain critical regions of the structure or the loads being carried by certain members. It is difficult to obtain such information by experimental means. For steel structures, the blind-hole method has been used with limited success to determine the existing stresses in members. The results usually include the residual stresses due to manufacturing and fabrication processes as well as the stresses due to applied loads. The two types of stresses can not be easily separated with limited test holes drilled.

For beams, columns and braces in a steel structure, the usual method of strengthening is to add cover plates. In some cases the beams are strengthened by making them act compositely with the floor slabs. Tubular columns and braces may be strengthened by infilling with concrete (Liu and Goel, 1987). Welding of cover plates to beam or column flanges under load requires careful planning and execution. Some information on columns reinforced under load is available (Rao and Tall, 1963). Steel columns can also be reinforced by encasing them in concrete with added longitudinal steel.

3.2 Connections

The behavior of a connection often affects the performance of all the members it connects and is an important consideration in any strengthening work. In steel framed structures, the important connections are beam-to-column connections, bracing connections, and column base connections. Much of the attention has been on beam-to-column connections, which may be designed originally to transmit only shear. Unless the structure is properly braced, certain moment capacities at connections are normally required in order to achieve adequate overall lateral stability (Lu and Driscoll, 1975). To develop a partial moment capacity, some kind of flange connection is necessary. This can be accomplished by adding seats, top and bottom plates or angles. Knee braces may also be used to provide limited lateral stability.

In many instances, non-destructive tests are carried out to detect cracks or local distresses. Among the various test methods, the dye penetrant magnetic flux, and ultrasonic procedures are commonly used. Also, a simple hardness test, conducted with a portable tester, can give a good estimate of the yield stress of material.

3.3 Frames

Steel frames can be strengthened, especially for lateral-load resistance, by installing bracing members, precast concrete panels, and metal sheeting. Common bracing systems are: X, K, inverted K, and knee bracing. The members may be made from single angle, double angle, channel, structural tubing and wide-flange shape. The braces are connected to the beams or columns of the frame through gusset plates by bolting and/or welding. It is often necessary to check the strength of the adjoining beams and columns because the forces in these members may be higher than the forces that existed before installation of the braces. Also, they may require local strengthening in order to safely transmit the bracing forces. Other problems related to the addition of bracing systems to frames are described by Yamanouchi (1980).

Precast concrete panels have been used in Japan to increase the seismic resistance of open frames, as reported by Tani (1980). Embedded in the panels, in a diagonal pattern, are steel plates or flat bars, and connections to the frames are conveniently made through these bars. Strengthening by thin metal plates has recently been studied by Tromposch and Kulak (1987) with emphasis on flat plates without stiffeners. The plates developed the desired tension field action, but under cyclic load the hysteresis loops became significantly pinched at large deformations. A similar study using corrugated plate panels has been conducted at Lehigh University and the results show the importance of providing adequate connections between the panels and the surrounding frame members (El-Dakhakhni and Daniels, 1973).

Steel bracing systems may be used to increase the lateral-load resistance of reinforced concrete frames. The advantage of this approach is that the additional weight of the bracing system is usually small in comparison with the weight of concrete or masonry panels. This is an important consideration if the foundation or soil condition limits the amount of dead load that may be added to the superstructure.

4. EVALUATION OF CAUSES OF BRIDGE DAMAGE

The causes of damages in bridge superstructures can be placed in two groups: those due to deterioration of materials, and those resulting from high magnitude of applied stresses or deflections. In the first group is corrosion of steel members. The second group of damages include, among others, yielding, fatigue cracking and fracture of steel.

Corrosion of steel occurs on unprotected surfaces when there is moisture. When protective layers of paint wear off or are accidentally removed, moisture comes in contact with steel and corrosion starts. Very frequently, debris accumulate at bridge details such as bearings and joints, and corrosion becomes most severe in these areas.

The effects of corrosion are the reduction of member cross sections (and the associated geometrical properties), and the change of characteristics of the bridge members. For example, it has been observed that the bottom flange of beams was so corroded that it was subjected to stresses much higher than the designed value. It has also been observed that corroded pin-connections became "frozen" and generated high bending moment in bridge members at the pin, causing local failure.

The design of bridge members is normally based on assumed distribution of loads from bridge deck to the individual members. The factor of safety which has been incorporated in design rules is usually sufficient to cover uncertainties so that yielding of primary bridge members very seldom occurs. At junctions or connections of bridge members, localized yielding could take place when a bridge is subject to unusual overload. These localized yield zones, however, do not reduce the strength or carrying capacity of the bridge members. In fact, more and more design practices permit this phenomenon under overloads. The situation which can induce damage, and often did, is the frequent application of high stresses.

Fatigue of steel is a phenomenon in which cracking develops under repeated stresses at levels much below the yield stress of the steel. Studies in the last two decades have indicated that, for steel structures, the governing parameters are the magnitudes of live load stresses (stress ranges), the number of applications of these stresses, and the "initial flaw" at the crack. The yield stress of a material has little influence.

All steel structural members have flaws, particularly at the connections or details such as welded joints, rivet and bolt holes, pin-connections etc. These flaws are very small imperfections in material or due to manufacturing and fabrication of the structural member; some are in the order of one or two thousandth of an inch in size (0.05 mm). Depending on the size of initial flaws, the initiation stage of fatigue crack growth may be relatively shorter for bridge components than for precisely manufactured machine components.

Because the design of connections of bridge components are usually by average nominal stresses, local areas of connections often have higher stresses. These locations of higher stresses also are where initial flaws exist. Thus, two of the governing factors of fatigue are in existence together. When the number of application of stresses (stress cycles) is also high, fatigue cracks could propagate quite rapidly.

The number of stress cycles is directly related to the number of vehicles traveling over the bridge. High volume of traffic generates high number of stress cycles. High volume of heavy trucks generates high number of high live load stresses. In the United States, and in many other countries in the world where this condition exists, fatigue cracks have been detected a few years or even a few months after bridges were open to traffic.

Fatigue cracks reduce the effective cross-sectional area of steel bridge members and increase the local stresses further. Undetected and unrepaired, these cracks can lead to yielding of the material or to sudden brittle fracture of a member.

For bridge components, fracture usually is the final stage of fatigue crack growth. Whereas fatigue crack propagation is governed by the magnitudes of live load stresses, fracture is a function of the maximum tensile stress at the crack. Therefore, the sum of dead load stress, live load stress including impact stress, and residual stress due to fabrication, is the reference stress for the evaluation of fracture from a crack.

When the combination of the maximum tensile stress and crack length produces a "stress intensity factor" higher than the "fracture toughness" of the material with a crack, sudden fracture of the tensile component will occur. The bridge member loses most or all of its capacity to carry load. In one case of a bar-chain suspension bridge, the bridge collapsed, killing a larger number of people.

Fracture toughness of steel is influenced by temperature of the material, the rate of stress application, and other factors. Low temperature causes steel to have lower fracture toughness, or to become more brittle. In moderate and warm climatic areas, sudden fracture of bridge components is not of a serious concern so long as the cracks in the tensile components are not too long. This condition allows sufficient time for the repair of bridge components after detection of fatigue cracks.

5. ASSESSMENT OF BRIDGE MEMBER DAMAGE AND PERFORMANCE OF BRIDGES WITH DAMAGED MEMBERS

5.1 Stresses at Location of Damage

The design of bridge members by design provisions adopts the basic assumption that loads on a bridge deck are transmitted to the component members through some simple manner of distribution. As bridge superstructures become more and more complex, with more components interconnected to each other, the assumption of simple distribution of loads becomes less and less accurate. Most components actually carry less load and sustain lower stresses than the computed design value. On the other hand, a few components may sustain stresses much higher than the design values.

Results of evaluation of damages in a number of bridges have shown that it is the high magnitudes of actual stresses at bridge details, not the design stresses according to design provisions, that caused the damages. The nominal stresses at a point before damage must be estimated by a reliable analytical procedure, and preferably confirmed by actual measurement at the bridge at locations similar to the point of damage.

The most powerful and most commonly used analytical procedure for computation of stresses at points of damages is the finite element method. Bridge structures can be adequately modelled by finite elements, and loads representative of common or unusual vehicles can be applied to the model to provide good estimate of actual stresses.

The measurement of actual stresses at points in bridges has been done for many years. In earlier days, measurement was primarily to confirm that actual stresses in bridge members were within the allowable values, and to provide information for the development of simplifying assumptions for design provisions. In recent years, the use of stress measurement to gather information for the prediction of damages at suspected locations and for the assessment of existing damages has become more common. Measured are the live load stresses due to selected "test truck" loads, and the stress-time variations due to regular traffic for a long period of time for the determination of stress history at the specific points of the bridge.

These measured stresses can be compared with those computed by a finite element analysis, and the results can be used to predict possible damages.

5.2 Prediction of Damage

The prediction of damage is the estimate of a bridge component's safe life. A commonly accepted criterion is that when a bridge member develops a primary fatigue crack, the member has attained it's safe fatigue life. A rational procedure consists of the following steps:

(1) Determine the live load stresses at the weakest structural detail. Actual measurement of stresses in the field is more direct and provides more useful data.

(2) Evaluate the live load stress data to obtain an equivalent stress range magnitude.

(3) Estimate the safe life of the structural detail using the equivalent stress range and an appropriate fatigue strength curve (S-N curve).

(4) Review the traffic record of the bridge and the projected traffic volume of the future to estimate the time (year) when the safe life will be reached.

This procedure has been utilized quite successfully for estimating the useful life of bridges until their replacement.

The application of the procedure requires an appropriate S-N curve and traffic data, which may not be readily available. There appears to be sufficient fatigue strength curves for welded bridge details. More information for riveted and bolted steel bridge details and for prestressed concrete elements are needed.

5.3 <u>Evaluation of Bridge Behavior</u>

When damage is imposed on a bridge component, its strength and stiffness change with the extent of damage. A larger number of studies have been conducted to examine these changes, primarily with the member alone carrying loads. In actual bridge structures, the damage of one component and the reduction of its strength and stiffness may induce transfer of its load carry function to other components of the structure. This phenomenon depends on the redundancy of the bridge structure. A few bridges have had one of their primary members fractured extensively and the bridges continued to carry vehicular traffic without collapse.

The evaluation of bridge behavior after the damage of one or more components can be made through analysis of the total bridge structure incorporating the damages in the components. Case studies of some bridges have been made, and research examining the behavior of two-girder bridges which are often regarded as "nonredundant structure" has recently been completed.

All these case studies and research work on the behavior of bridge superstructures with damaged members indicate that ordinary bridges normally can sustain certain damages without a catastrophic consequence. From the detection of distress to the stage of fairly extensive damage of a bridge member, there usually is enough time to repair the member and restore the bridge.

6. <u>REPAIR AND RETROFIT OF BRIDGES</u>

The repair of damages in bridge members must be made according to the causes of the damages. These repairs can be arbitrarily separated in three groups. The first includes direct repairs to correct damages resulting from deficiency in materials, lack of maintenance, or localized defects of fabrication or manufacturing. The second group of repair adds similar components to the bridge members. In the third group are retrofits to change the behavior of the members at the damages.

6.1 <u>Direct Repair of Damages</u>

Moderately corroded steel members often need not be repaired. Cleaning and repainting usually is sufficient. In case of very severe corrosion and significant loss of member cross section, an analysis of the bridge with the damage should be made to evaluate the need of reinforcement. One common practice has been the addition of reinforcing parts to the corroded area by welding. Care must be given to the fatigue strength of the welded detail so that it will not cause short service life of the repaired member. This situation has been observed in some truss bridges.

Local fatigue cracks and fracture resulting from defects in primary members of bridges can often be repaired directly. Flanges of steel beams with cracks originated from flaws in butt welds are examples. Repair can be drilling of holes at the tips of short cracks to decrease the stress intensity at the tips of the crack, so that the crack will not grow. Repair can also be adding of splice plates to the flange at the crack, increasing the area of the flange and reducing the stresses in the plate with the crack.

6.2 Addition of Parallel Components

When damages are results of high stresses, either due to increase of load or underestimate of stresses by using design provisions, addition of parallel members or components can be effective. The addition of more bars to tension diagonal of truss bridges is an example. Another example is the addition of external prestressing bars to concrete bridge beams to increase the flexure strength and close the tensile cracks at the bottom of the beams. Widening of roadway width of beam-slab type bridges by adding more beams may also be considered as an example.

In all these cases of retrofitting by adding parallel members, the distribution of loads among the repaired and other members as parts of the complete structure should be examined to assure the adequacy of the addition.

6.3 Repair of Displacement Induced Damages

Damages or failure such as local yielding and fatigue cracking at connections, as mentioned earlier, often are the unexpected consequence of designing the connected members individually and designing the connections for strength. Displacement or deformation of the connections usually is not considered. This situation may lead to local stresses which often are not considered and are much higher than the nominal design value. Local damages of this type are the most frequent in structures. One of the most commonly detected damage is the fatigue cracking of connections between stringer beam and cross girder in plate girder bridges. Another is the cracking of plate girder web at the connection of interior diaphragm.

The repair of this type of local damages must be made to change the displacement and stress distribution characteristics of the local area. Simple replacement of damaged parts, such as broken rivets or cracked connection angles at the joint, does not change the relative stiffness of the components and does not remove the cause of the damage.

The retrofitting scheme for displacement induced damages can be either increasing or decreasing of the rigidity of the damaged zone. For example, the dimensions of the connecting angle and the number of bolts of a floor beam-to-girder connection may be reduced to make the connection more flexible and yet still capable of carrying the reactions with sufficient safety margin. In the case of interior diaphragms of steel box girders, the relative displacement between the component parts may be reduced by using full depth diaphragm plates and rigid connection between the diaphragm, the box girder webs, and the box girder flanges. In all cases, the analysis of the displacement and stresses at the local area is necessary, and actual measurement of these quantities after retrofit is highly desirable.

There have been many other cases of displacement induced damages in bridges due to live loads. A vertical member of a long truss fluctuated under sustained high wind and vibrated seriously, causing the member to crack at its connections. The retrofitting procedure was the addition of vibration dampers and direct repair of the damages. Addition of secondary chord members to the truss would be another procedure. In the cases of steel plate elements in bridge members vibrating due to high speed vehicles on the bridge, simple addition of stiffeners to the steel plates removed the unfavorable condition. These are but a few examples of repair according to cause and effect. Each case must be examined carefully with the goal of correcting the inherent difference between design and actual behavior.

7. SUMMARY

An overview of repair, retrofit and strengthening of steel building and bridge superstructures has been presented. Much of the work was carried out on an ad-hoc basis and intended to provide solutions to particular structures.

Building rehabilitation has become an

increasingly important aspect of engineering activities in many countries, especially those which are located in regions of high seismicity. Much work on seismic strengthening has taken place in Japan, Mexico, New Zealand and U.S.A. The effort in Wellington, New Zealand is the most impressive and, perhaps, is the best planned and coordinated (Smith, 1985).

Bridge superstructures have experienced more damages in recent years because corrosion, deterioration of inferior concrete, fatigue of material, and other time-dependent damages developed gradually to an alarm situation. The underlining controlling factor of damage and member strength, however, remains to be the stress at the points of damage.

An accurate evaluation of the stresses at the damages of components in actual bridge structures is essential. Measurement of stresses in these components of bridges in service is necessary. From the data of actual live load stresses, the safe life of the components can be estimated. Damage of one component in a bridge structure does not necessarily cause the bridge to fail or collapse. The evaluation of behavior of bridges with damaged components must be done through the analysis of the total bridge structure.

Similarly, the repair of damages of bridge components must be made with appropriate analysis of the stresses at the damage. Depending on the cause of the damage, change of structural details may be necessary in order to change the stress and deflection characteristics of the details.

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ULTIMATE SEISMIC STRENGTH AND DUCTILITY INDEX OF REINFORCED CONCRETE BUILDINGS STRENGTHENED WITH STEEL BRACE AND STEEL PANEL

BY

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1. OUTLINE

It has been proved by previous tests that seismic performance is improved by retrofitting a reinforced concrete (thereafter refered as RC) building with framed steel braces and panels when seismic strength and ductility of existing RC frame result in poor seismic performance. Recently in Japan, this seismic strengthening method with steel frames has been widely accepted. This method is a sort of composite construction of the existing RC frame directly jointed to framed braces or panels framework. There are two methods to frame the steel elements. While steel rim frames that enclose the circumfererance of steel braces or panels are indirectly jointed to the existing open RC frame in one method, steel braces or panels are directly jointed to the existing open frame in another. It may be stressed that the jointing method of different types of structural materials would determine seismic performance. Since the following strengthening method is effective in both compression and tension for lateral seismic force and has smoother force transmission, the indirect jointing method with steel rim is recommended. Illustration in Fig.1.1 is a typical example of this retrofitting method. The strengthening method under Revised Guidline for Repair and Retrofitting is outlined, focussing on how to obtain the failure modes, seismic strength, and ductility of the infilled steel walls with rims indirectly jointed.

2. CHARACTERISTICS OF RC FRAME STRENGHTENED WITH STEEL WALL

It is common to strengthen the existing RC building with post-cast RC walls when it does not have adequate seismic strength. However, strengthening with steel walls makes the design and construction possible with the characteristics mentioned below.

i) In some cases, light weight steel retrofitting is effective when the foundation does not have adequate allowable bearing capacity.

i) It helps increase not only the strength, but also the ductility of

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the whole frame. Moreover, the vertical steel rims can support the vertical load as the substitutes of columns when shear collapse occurs to the independent RC columns.

iii) Since it enables the builders to leave appropriate openings for such purposes as natural lightning and ventilation without decreasing seismic strength and causing deterioration to the indoor enviroment, RC frames retrofitted with steel is superior to post-cast RC walls.

iv) The method enables the builders to save the construction time and to increase the accuracy of the work by building up the steel materials at the factory.

Though retrofitting with steel has the advantages mentioned above, it is still inferior to post-cast RC walls in aspects such as fire resistance, durability and insulation of sound. Since there is little experimental data and few examples to analyse, the method should be adopted within the scope of limited circumstances. The outline of the construction process of common retrofitting is illustrated in Fig. 2.1. Compared with RC retrofitting, steel retrofitting has more strict conditions that should be met in site investigation, securing the accuracy in working processes at the site, and keeping the joint parts in good condition before building up the steel members of the framework.

3. FAILURE TYPES OF RC FRAME STRENGTHENED WITH STEEL WALL

Failures shown in destructive tests can be classified into three types illustrated in Fig. 3.1..

Type I: This type of failure causes tensile failure or buckling to the braces or shear failure to the panels and, at the same time, it also causes shear collapse or failure to the columns in the RC frame. This type of failure occurs only when the connector between the existing RC frame and the installed steel shear wall is strong enough to transmit the shear force.

Type II: In this case, shear deflection occurs to the joint. As the shear force that should be transmitted to the installed steel member from the existing RC frame is directly transmitted to the top of the existing RC column on the tension side, it causes punching shear slippage in vicinity of the top of the column. Furthermore, it causes shear collapse or bending failure to the column on the compression side.

Type II: The overturning moment causes tensile failure to the existing RC column on the tension side or compressive collaps to the column on the compression side and brings about bendin failure to the strengthened frame. It is difficult to increas the bending strength of the whole strengthened frame by increasing the seismic strength of the members retrofitted wi steel frames or joints for this type of failure.

4. CALCULATION OF THE ULTIMATE SHEAR STRENGTH OF THE INFILLED WALLS STRENGTHENED WITH STEEL

Important factors in calculating a retrofitted frame are mentioned in the following.

i) The ultimate shear strength, ${}_{w}Q_{su}$, of the infilled wall retrofitted with steel can be calculated using the following formulas. How it resists at the existing RC frame, the installed steel member of the framework and the connector should be taken int consideration.

$$
{}_{w}Q_{su} = \left.\begin{array}{l} {}_{s}Q_{u} + Q_{c1} + Q_{c2} \\ \\ Q_{j} + {}_{p}Q_{c} + Q_{c2} \end{array}\right\rbrack \text{ the smaller of the two } \text{——— (4.1)}
$$

herein, ${}_{s}Q_{u}$: The seismic strength (tf) of the installed steel membe (brace or panel)

Q_{j} : Shear strength of the connector along the under side o the beam(studs or post-infilled anchors)

Q_{c1} : Ultimate shear strength (tf) of the column on the tension side

Q_{c2} : Ultimate shear strength (tf) of the column on the compression side

${}_{p}Q_{c}$: Punching shear strength (tf) of the top of the column on the tension side.

ii) With few exceptions, the ultimate shear strength of each column in the RC frame is calculated on its section without considering th retrofitting members such as steel rims or mortar.

iii) In principle, the ultimate shear strength of the framed steel member such as brace is determined as the value where the full section of both compressive and tensile braces reach each limit state stress. Herein, critical compressive stress f_{cr} is given by the following formula.(figure 4.1)

$$f_{cr} = [1-0.4(\lambda/\Lambda)^2]F \quad \text{for } \lambda \leq \Lambda$$
$$= 0.6F/(\lambda/\Lambda)^2 \quad \text{for } \lambda > \Lambda \qquad (4.2)$$

Herein, Λ :Ultimate slenderness ratio $[=\sqrt{(\pi^2 \cdot E)/(0.6F)}]$

λ :Effective slenderness ratio

F :Specified strength of steel(tf/cm²)

E :Young's modulus of steel materials(tf/cm²)

iv) With regards to the seismic strength of the framed steel member the steel section must be designed to cause shear failure. For this it is necessary for the flange to have a section that does not caus flexural yielding and stiffeners which need to be arranged at appropriate intervals to prevent elastic buckling to the panels. As openings in the panel cause deformation to the frame, ultimate flexural strength has to be taken into consideration (see Fig. 4.2)

v) Between the two values, the tensile failure or compressive failure of the RC column when either one of them occurs, the smaller value is the bending strength of the infilled walls. In this case, it is assumed that the framed steel braces and panels do not contribute to the bending strength of the whole structure.

vi) Seismic strength of the foundation at overturning is calculated considering boudary beams, perpendicularly intersected beams, the axial load of the columns and the dead load of the foundation.

vii) Seismic strength of the connector is calculated with items related to it in Revised Guidline for Repair and Retrofit Design. The shearing force of each stud is calculated with the following formula.

$$q_s = 0.64\, \sigma_{max} \times a_s$$

herein, σ_{max}: The value of the tensile unit strength of the stud which is smaller than 4.1(tf/cm²)

a_s : Sectional area of the stud.(cm²)

However, it must be confirmed that the post-installed anchors have more seismic strength than studs after they have been arranged.

5. COMPARISON BETWEEN THE RESULTS OF PREVIOUS TESTS AND THE RESULT OF ULTIMATE SHEAR STRENGTH CALCULATION

The outline of the typical specimen(A-1) as an object for comparison is illustrated in Fig. 5.1. Each of the fourteen specimens is one third the size of the actual object with a single layer and single span (Fig. 5.2). The existing RC frame is 2.0m span, and the effective height of the columns is 0.97cm. Concerning the section of the columns, the breadth is 200mm and the depth is 250mm. As far as the ratio p_t of longitudinal bars is concerned, there are three types of columns, 3.44%, 2.39% and 1.02%. The ratio p_w of tied hoop of all the columns is as litle as 0.10%. Compressive strength of concrete, σ_B, of RC frame is between 195~ 291kgf/cm². Strength of grouting mortar σ_M, in the indirect joint is as high as 325~ 369kgf/cm². As illustrated in Fig. 5.1d, in principle, post installed anchors with heads and studs are alternately arranged along the whole circumference of grouting mortar. Only the number of the studs along the under side of the beam is shown in Fig. 5.2. The forms of the brace designed against both tension and compression, and the panel, are illustrated in the figure. In principle, constant axial pressure σ_0 of 30kgf/cm² is added to the columns. Ultimate shear strength Q_{max}, ultimate shear unit stress τ_{max} given by the test result, and the types of failure are listed in the first half of table 5.1 and in the latter half, calculated values that correspond to each style of failure are listed. Each value is calculated in the following way.

Q_{su} (cal.): Calculated ultimate shear strength of failure type I using formula (4.1)

Q_{su} (cal.) $= {}_sQ_{u,brace(or\ panel)} + Q_{c1} + Q_{c2}$ ———— (5.1)

herein, ${}_sQ_{brace}$ is the sum of the smaller value of the horizontal component of either yielding strength or buckling strength of the brace, and the horizontal component of yielding strength of the tensile brace. ${}_sQ_{panel}$ is the shear stress of the panel when the average horizontal shearing stress reaches $\tau_y=\sigma_y/\sqrt{3}$ (σ_y: yield strength of the steel member). Q_{c1} and Q_{c2} are respectively the ultimate shear strengths of the column on the tension side and on the compression side that are given by the calculation using the formula (13) in Criterion on the Evaluation of Seismic Safety.

Q_{pu}(cal.): Calculated ultimate strength of type II.

Q_{pu}(cal.) $= Q_j + {}_pQ_u + Q_{c2}$ ———————— (5.2)

Herein, Q_j is the ultimate shear strength(=$n \cdot q_s$) of the joints. And n is the number of studs that are arranged in a row along the under side of the beam. q_s is the shearing strength (2.28tf/piece) of the headed stud 9ϕ that was given by the direct shear test. $_pQ_c$ is the shear strength of the RC column on the tension side when punching shear failure occurs to it. However, punching shear strength is calculated using the influence factor k_{min}.

Q_{mu}(cal.):Calculated ultimated unit strength of type Ⅲ

$$Q_{mu}(cal.)=Mu/h \text{ ———————— } (5.3)$$

Herein, Mu is the ultimate bending moment concerning the neutral axis given from the balance of axial force of the whole strengthened frame at the bottom level of the column.

Among the three values of the calculation, the smallest is the theoretical ultimate shear strength Qu(cal.). This value is shown as the figures in frame □ in the table 5.1 . Excluding models P-1,P-2 and P-1-N that have shown high ultimate shear strength in the test, each failure mode obtained from calculated values comforms the tested ones. In the last column of the table, the ratios of the calculated value and test value Q_{mu}(test)/Q_u(cal.) are shown. Generally the ratio of connectors is close to 1.0. However, specimens like P-2-C and P-1-60, whose connectors receive large tension as well as shear strength, should be designed reducing the ultimate shear strength given by the calculation, otherwise, the value would exceed the safety level. In some cases, like P-2-C whose part of the indirect connector must be cut off for some reason, it is recommended that the ultimate shear strength be decreased by multiplying reduction factor α(=1- l_0/l_1, herein, l_0: cut off length of the joint member, l_1:clear span of the existing open frame) with the ultimate shear strength given by the formulas (5.1 to 3).

6. THE TYPES OF RESISTANCE AND DUCTILITY INDEX

6.1 The types of resistance

A strengthened RC frame consists of three parts of structures. They are existing RC frame, framed steel brae or panel and connectors. Types of resistance after retrofitting are classified into four resisting types (see Table 6.1). It is apparent that ultimate shear strength, ductility in this state and failure type as the whole strengthened RC frame are reflected by both ultimate strength and its ductility in each structure. Strength and ductility resisting type (type I) is recommended when retrofitting using framed steel shear walls is considered.

6.2 Other types of resistance

Resisting type of ductility (type Ⅴ), which absorbs the energy of seismic force by the overturning deformation of the foundation, can be recommended when overturning of the foundation occurs to the strengthened RC frame, however seismic performance of resisting type of strength and ductility may not be expected.

6.3 Examination of resisting types

The final resisting type of the strengthened RC frame is decided by the minimum of seismic strengths which are calculated from the ultimate shear strength concerning each resisting type and overturning type of the foundation shown in table 6.1. Ductility indices of each resisting type are set out in table 6.2, corresponding to the resisting types and overturning types of the foundation listed in table 6.1.

6.4 The ductility index of the RC frames strengthened with rimless frames

When the existing RC frame is strengthened with the rimless brace or panel member, the data is more likely to be dispersed than when it is strengthened with framed braces or panels. Therefore, the ductility index should be adopted with a wider safety rate than needed for retrofitting with framed one (see Table 6.3).

7. THE DUCTILITY INDEX SHOWN IN THE TEST RESULT

Fig. 7.1 shows the envelope curve concerning the load and deformation of the fourteen specimens classified by failure types. On the X-coordinate, the drift angle R is indicated together with a simplified ductility index $F=0.6+100R$(rad). This simplified index is obtained from the formal ductility index $F(=\sqrt{2\mu-1}/[0.75(1+0.05\mu)]$, assuming it has the same relationship with the value of the ductility index (herein, μ:plasticity ratio, see ref. 4). as it has with bending member of the column. On the Y-coordinate, the value Q/Q_{max} normalized tested horizontal load Q by the calculated ultimate load Q_{max} (cal.) whose is illustrated. Therefore, in the zone where the Y-coordinate value exceeds 1.0, the ratio of the tested value which exceeds designing value, namely the safety ratio that corresponds to the designing value, is indicated. As shown in Fig. 7.1, horizontal deformation at the ultimate strength occurs to each specimen only when drift angle exceeds approximately 1/150 without great influence of the difference of failure types. However, the shear strength increases when shear failure occurs to web plate of panel wall even if further deformation occurs (see Fig. 7.1b). The above makes it clear that the specimens that cause failures of type I have superior ultimate shear strength and ductility indices in both brace retrofitting and panel retrofitting. Especially the web panel designed to cause shear failure shows better ductility than the brace that causes buckling or tensile failure. Even among the specimens that cause failure type II, the number of spesimens fall below the designing value suddenly increases by repeated loads when the drift translation angle exceeds 1/150. Therefore, large ductility index can not be expected when punching shear failure occurs. In case of failure type III, the designed value is obtained from the assumption that only the existing RC frame resists to the bending of the whole strengthened frame and the framed steel member and connectors do not contribute to it. For this reason, the ultimate value of Q/Q_{max} becomes larger. However, as the ultimate shear strength suddenly decreases when the drift angle of connector exceeds 1/63.6(δ=18mm), it can not be said type III is much superior to the common post-cast RC walls with regards to ductility.

Under this Revised Guidiline for Repair and Retrofit Design, it is recommended that suitable retorfitting parts be found to cause failure type I. This type is a resisting type of strength as well as ductility with most advantageous characteristics of RC frame strengthned with framed steel braces or panels. Value F of type I that exeeds 3.0 shows it is much superior to other types. With regards to type II, value of F is close to the minimum value 1.27 of flexural failure of the RC column even when punching shear failure occurs to the RC column. As far as this test is concerned, failure type III is not superior to flexural failure of the common infilled RC walls in ductility. Therefore, the value F =2.0 is adopted in the table.

8. COMPARISON WITH THE HYSTERICIS CHARACTERISTICS OF INFILLED RC WALLS

When large seismic strength is needed, either RC frames strengthened with steel walls or infilled RC walls, is adopted. In the following article, hystericis characteristics of the strengthened frame is compared with that of infilled RC walls which have similar test scales and retrofitting purposes.

Figure 8.1a illustrates hystericis characteristics of the load and deformation of the top of the column of specimen A-1 of the frame strengthened with steel. On the other hand, figure 8.1b shows the load deflection characteristics of specimen CH2018 which is designed for retrofitting with post-installed RC wall and one third of the actual object in scale with a single layer and single span. The span of the existing open frame and the effective height of the columns are equal to the size of specimen A-1. However, the section of the column is $b \times D$ = 20cm × 20cm, the ratio of total sectinal area of the longitudinal hoop P_w is 4.53(%) and the reinforced ratio of hoop P_w of the column is 0.64(%). The compressive strength of the existing open frame is 261kgf/cm². Stud anchors D10 with heads are buried into the whole inner circumference of the open frame with @75. The strength of the post-installed wall which is 12cm thick and has no opening is 222kgf/cm². Both vertical and horizontal reinforcement (D6, @75) of walls are p_s = 0.71(%) and are double arranged. In principle, the constant axial pressure σ_0=30kgf/cm² is added to the column. Though the columns of specimen CH2018 have many bars and shear reinforcing bars, yielding occurs to the columns on both tension and compression sides at the ultimate load(Q_{max}= 118tf, R_u=1/120). Furthermore, a large shear crack occurs to the wall panel.

Ultimately, large shear cracks occur to the existing columns of both specimen, A-1 and CH2018, however, there are two major differences, mentioned below, between the two.

1) While the repeated loop of A-1 is spindle shaped, that of CH2018 is cocoon shaped, the latter middle part slightly thinner than the first.
2) While A-1 shows a minor decrease of load after drift angle R reaches the ultimate shear strength at the approximate values of 1/120 and CH2018 show a considerable decrease of load after R reaches the ultimate shear strength at the approximate value of 1/120.

It is shown in above that strengthening meathod with steel braces or panels absorbes greater amountof energy as the major difference between the hystericis characteristics of frame strengthened with steel and that of infilled RC walls.

9. CONCLUSION

The strengthening method of existing RC building with steel brace and/or panel were described above.

Under the deep understanding of the characteristics of steel frame, this strenthening method showed excellent strength and ductile behavior up to the large deformation which exceeds any practical estimation of responcse displacement for standard RC buildings. Especially, it is preferable to choose the appropriate existing RC frames reveal the type I failure mode.

This method will be adopted in the revised guidline for aseismic retrofitting in Japan.

10. ACKNOWLEDGEMENT

This proposal of revised guideline was completed under the endeavor of workshop members. We would like to express our appreciation to Mr. T. Akiama for assistance in preparing the manuscript.

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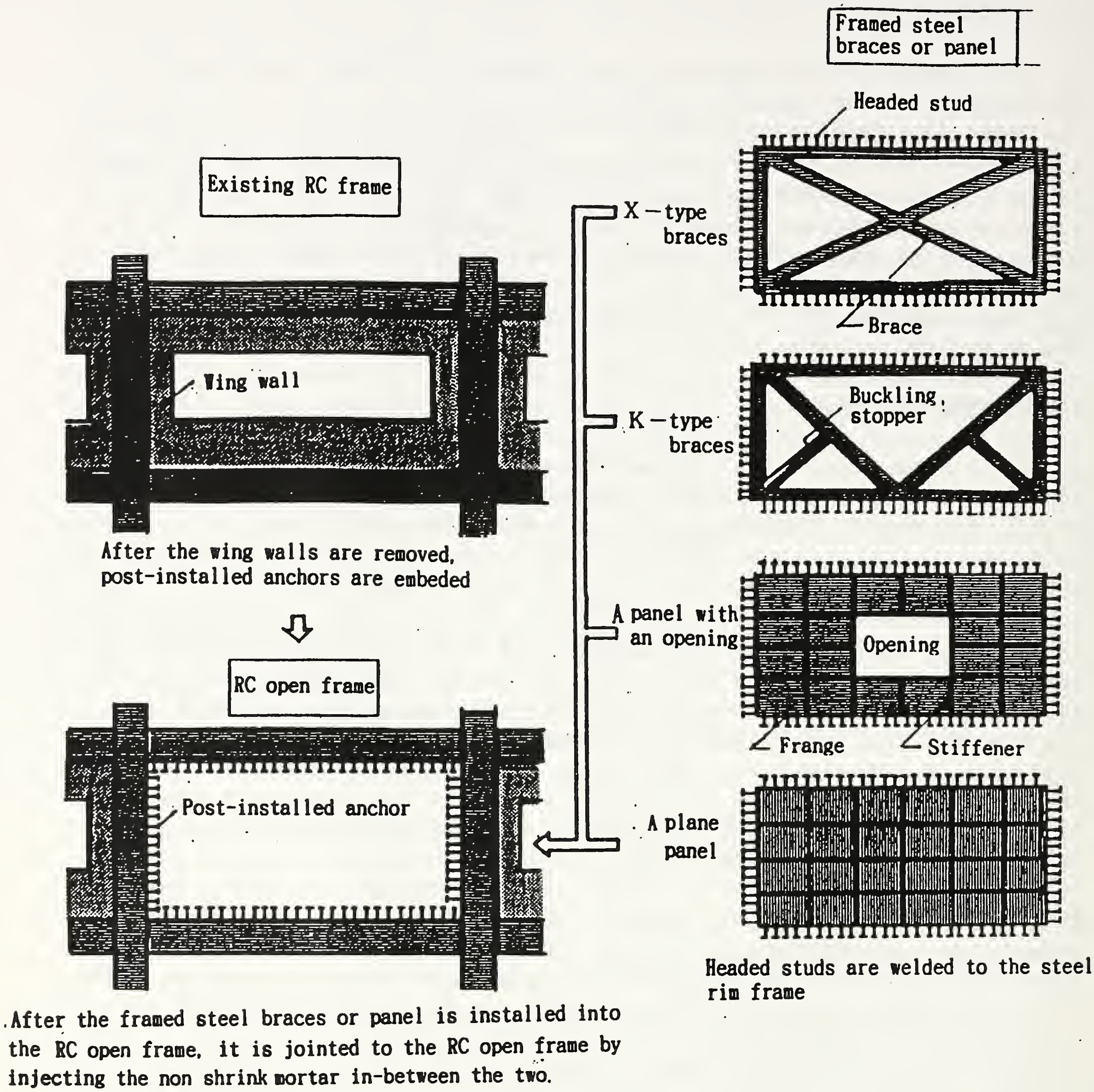


Fig. 1.1 Examples of RC open frames strengthened with framed steel(strengthened with rims frames)

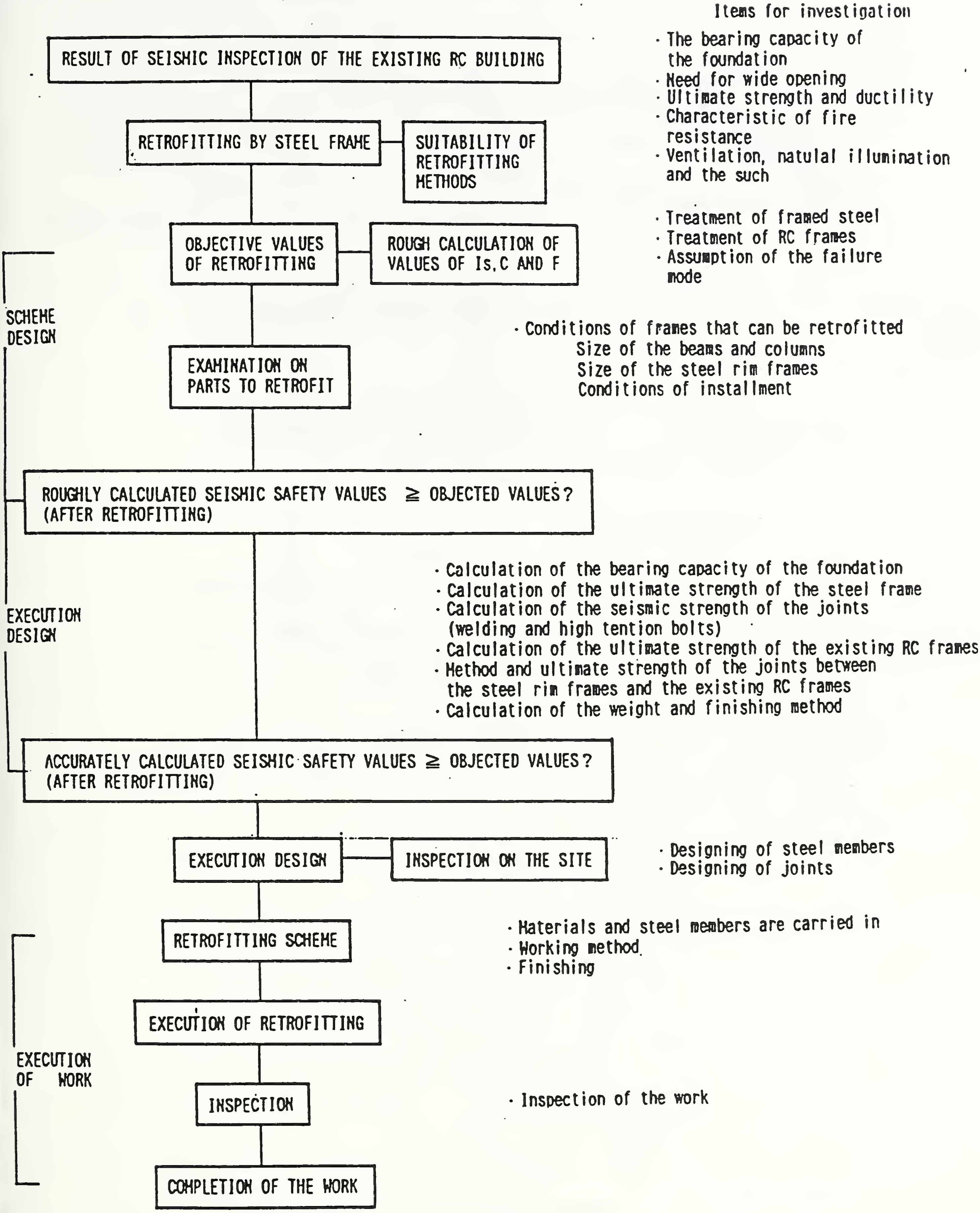


Fig. 2.1 Flow chart of designing and excution of retrofitting with framed steel

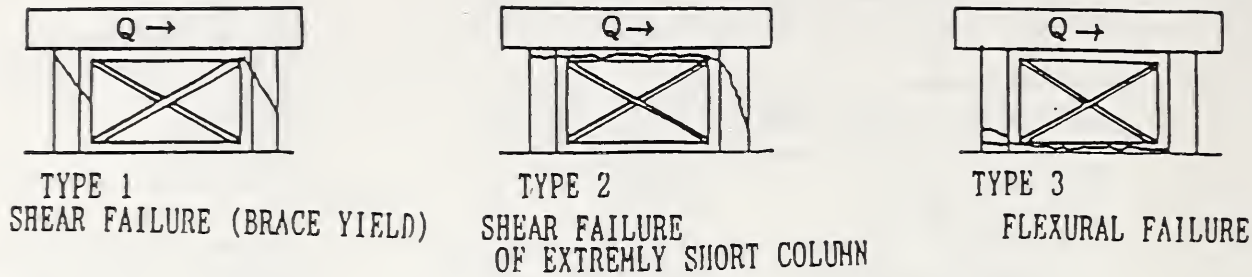


Fig. 3.1 Examples of RC open frames strengthened with framed steel(strengthened with rims frames)

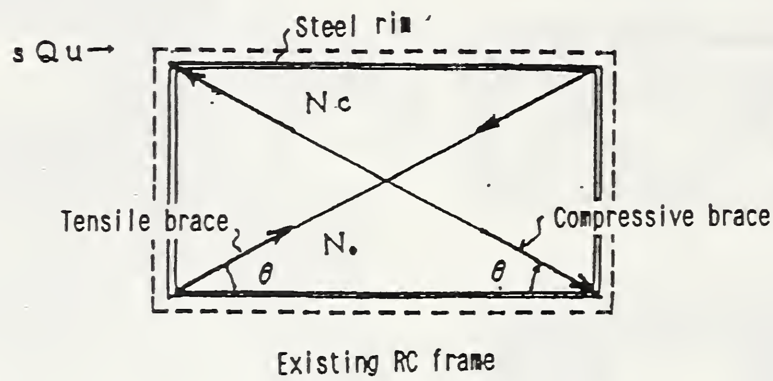


Fig. 4.1 Ultimate strength of the brace

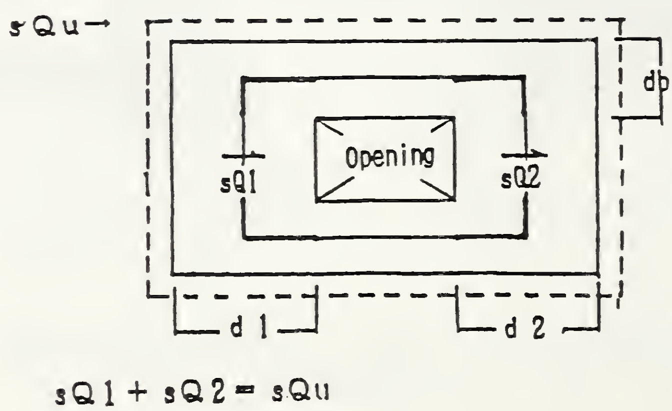


Fig. 4.2 Ultimate strength of the framed panel

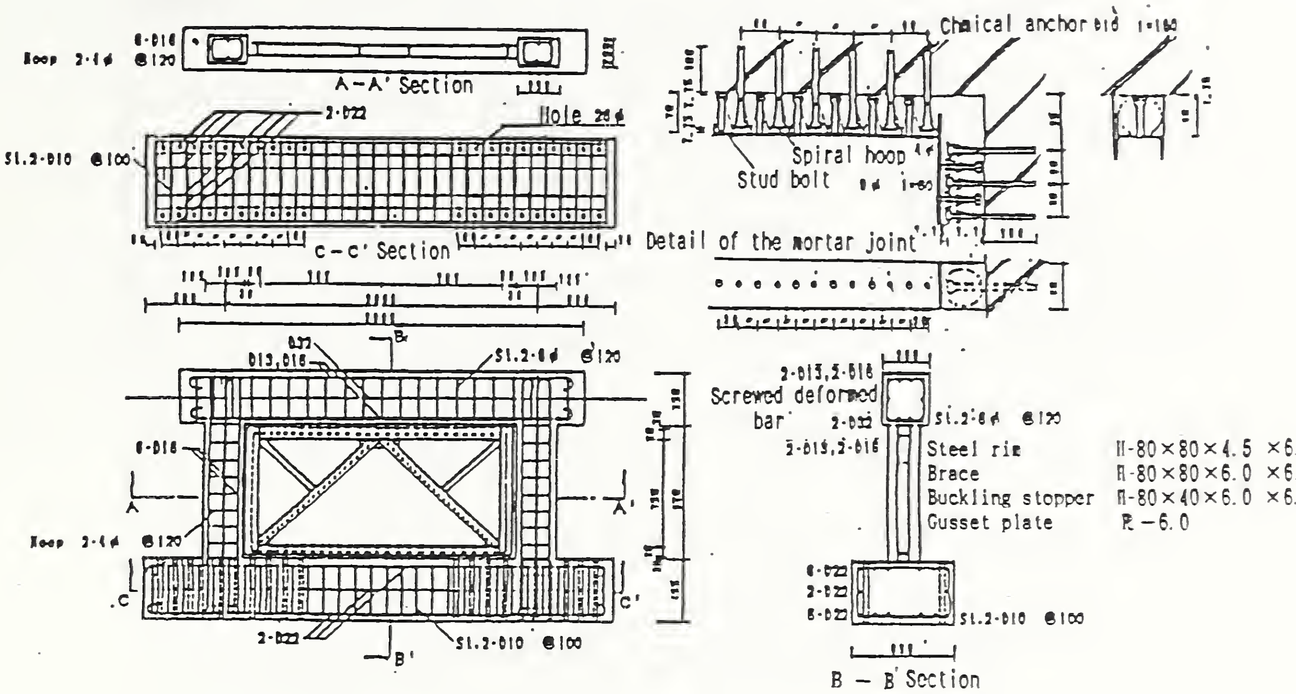


Fig. 5.1 Details of the test specimen(Λ−1)

NO	Spec-imen	σ_B (kg/cm²)	σ_M (kg/cm²)	Stud n	Sectional area of RC column	Shape of steel brace or panel	σ_0 (kg/cm²)
1	X−1	291	372	26	b=200 D=250 Ratio of longitudinal bars Pg=2.39% (3.44) Pt=0.80% (1.15) Reinforced ratio of hoop Pw=0.10% Values for P-1-N is in parenthesis	Brace H-80×80×4.5 ×6.0	30
2	X−2	291	372	26		Brace H-80×80×3.2 ×4.5	30
3	Λ−1	225	337	26		Brace H-80×80×6.0 ×6.0	30
4	Λ−2	225	337	26		Brace H-80×80×3.2 ×4.5	30
5	M−1	216	325	26		Brace H-80×80×6.0 ×6.0	30
6	P−1	247	325	26		Panel PL−4.5	30
7	P−2	247	325	26		Panel PL−4.5	30
8	P-1-0	209	407	14		Panel PL−4.5	30
9	P-1-S	209	355	20		Panel PL−4.5	30
10	P-2-C	195	400	20		Panel PL−4.5	30
11	P-2-G	195	339	26		Panel PL−9.5 、4.5	30
12	P-1-N	235	417	26		Panel PL−4.5	Variable ※1
13	P-1-86	240	469	18	Pg=1.02% Pt=0.51% Pw=0.10%	Panel PL−4.5	30
14	P-1-60	202	469	26		Panel PL−4.5	0(30)

※1 $\sigma_n=30\pm0.572Q$(Q is the horizontal load)is added, saauming a wall of two story buiding

Fig. 5.2 Properties of the test specimens

Table 5.1 Comparison with test results and calculated values

Test specimen	Test results			Calculated values			Qmax(Test) / Qu(cal.)
	Qmax(Test) (tf)	τ_{max} (kgf/cm²)	Failure type	Qsu(cal.) (tf)	Qpu(cal.) (tf)	Qmu(cal.) (tf)	
X − 1	90.4	56.5	Type I	$\boxed{81.9}$	83.7	94.2	1.10
X − 2	74.2	46.4		$\boxed{64.1}$	83.7	94.2	1.16
A − 1	78.1	48.8		$\boxed{72.2}$	81.1	94.2	1.08
A − 2	59.7	37.3		$\boxed{53.5}$	81.1	94.2	1.12
M − 1	79.3	49.6		$\boxed{65.1}$	80.7	94.2	1.22
P − 1	94.1	58.8		94.1	$\boxed{82.0}$	94.2	1.15
P − 2	91.2	57.0		94.1	$\boxed{82.0}$	94.2	1.11
P -1 -0	69.4	43.4	Type II	124.9	$\boxed{52.7}$	104.4	1.32
P -1 -S	87.7	54.8		83.2	$\boxed{66.4}$	104.4	1.32
P -2 -C	62.5	39.1		58.7	$\boxed{51.1}$	104.4	1.22
P -2 -G	90.5	56.6		124.7	$\boxed{79.5}$	104.4	1.14
P -1 -N	99.3	62.1		83.9	$\boxed{81.4}$	107.2	1.22
P -1-86	75.8	47.4		82.7	$\boxed{59.3}$	65.2	1.28
P -1-60	60.8	38.0	Type III	79.9	72.9	$\boxed{39.0}$	1.56

*1) In sasc of P-2-C:reduction coefficient $\alpha = 1-(\ell_o / \ell_1) = 1-(36/161) = 0.776$
Qsu(cal.) = $7.56\times0.776=58.7$tf and Qpu(cal.)= $65.8\times0.776 = 51.1$tf

*2) Herein.Qsu(cal.) is the designed value when failure type-1 is shown. Qpu(cal.) is the designed value of type-2. and Qmu(cal.) is the designed value of type-3.

*3) Excepting the above mentioned test specimens, non-strengthening (original) specimen, RC-1and specimen RCS-1 installed of only steel rim frames are tested.

Table 6.1 Resisting types of RC frames strengthened with steel

Types of resistance	Open frame	Framed steel member	Connector
Type I Resisting type of strength and ductility	•Flexural failure of the columns and beams •Shear failure of the columns and beams	•Brace retrofitting: Yielding or flexural failure of braces •Panel retrofitting: Shear failure of panels or flexural failure of flanges	•No failure
Type II Resisting type of strength	•Punching shear failure of the column on the tension side and shear failure of the column on the compression side •Punching shear collapse of the beams	•No yielding or bending failure	•Sliding failure
Type III Resisting type of Ductility	•Tencile failure of the column on the tension side •Compressive collapse of the column on the compression side	•No yielding or bending failure	•No failure
Type IV Resisting type of strength	•Extremely brittle shear failure	•Brace retrofitting: Yielding or bending failure of braces •Panel retrofitting: Shear failure of panels or bending failure of flanges	•No failure

(Anotation) Type I is the flexural failure of strengthened RC frames

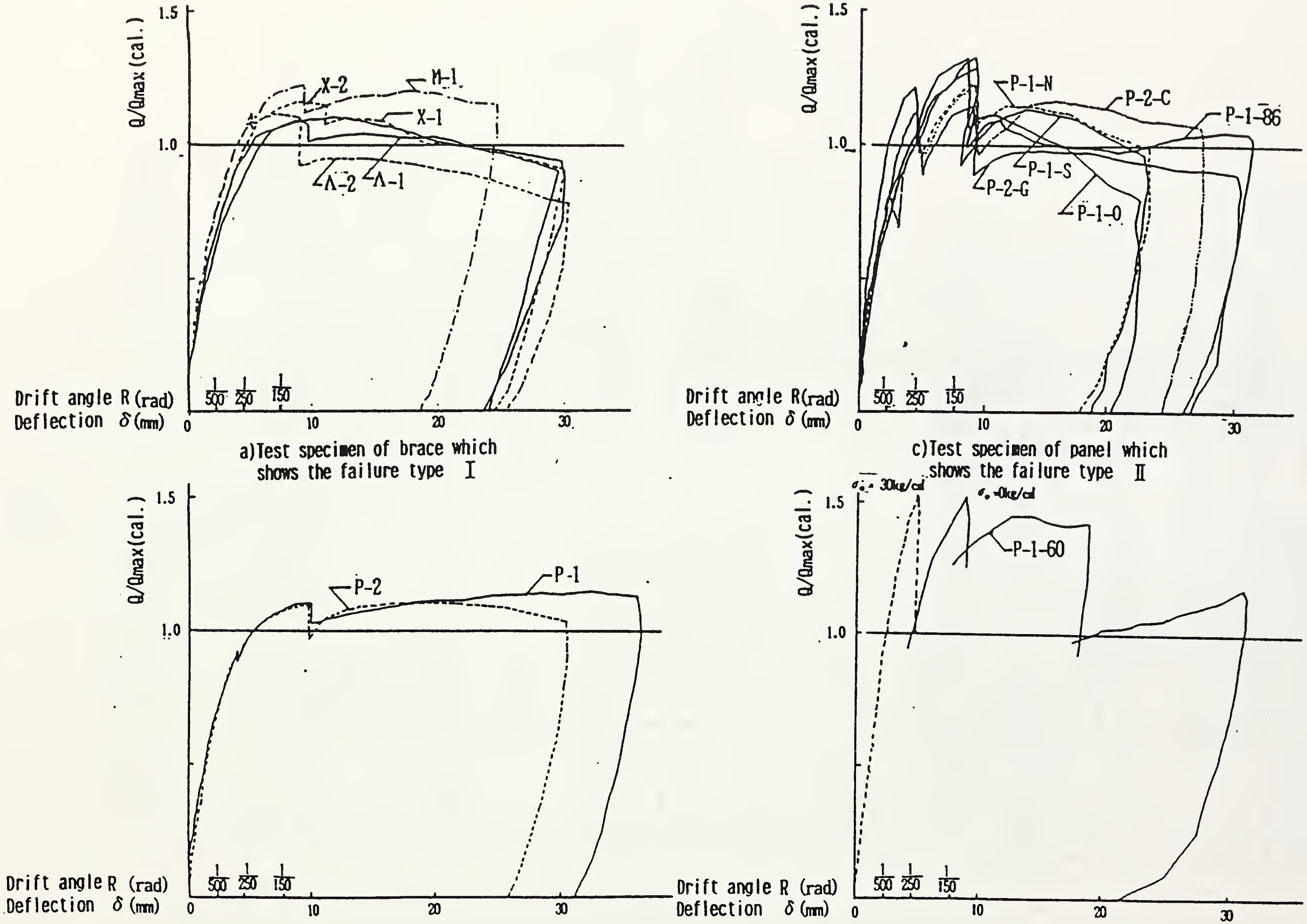
Table 6.2 Ductility indices of the frames strengthened with steel

Types of resistance	Failure types of the open frame	Ductility index F
Type I	•Flexural failure of column and/or beam •Shear failure of column and/or beam	2.0 ≦ F ≦ 3.0, and less than the value F of the existing RC frame 2.0
Type II	•Punchig shear failure of column or beam	1.5
Type III	•Bending failure type of the whole RC frame	2.0
Type IV	•Extremely brittle shear failure of column	1.27
Type V	•Overturning failure of the foundation	3.0

Table 6.3 Ductility index of RC frames strengthened with rimless frames

Types of resistance	Failure types of the open frame	Ductility index F
Type I	•Bending columns or beam yield type columns •Shear failure of column and/or beam	1.27 ≦ F ≦ 2.0, and less than the value F of the open frame 1.0
Type II	•Punchig shear failure of column or beam	0.8
Type III	•Bending failure type of the whole RC frame	F ≦ 2.0
Type V	•Overturning failure of the foundation	3.0

Remark: In this case, Type IV is not permitted.



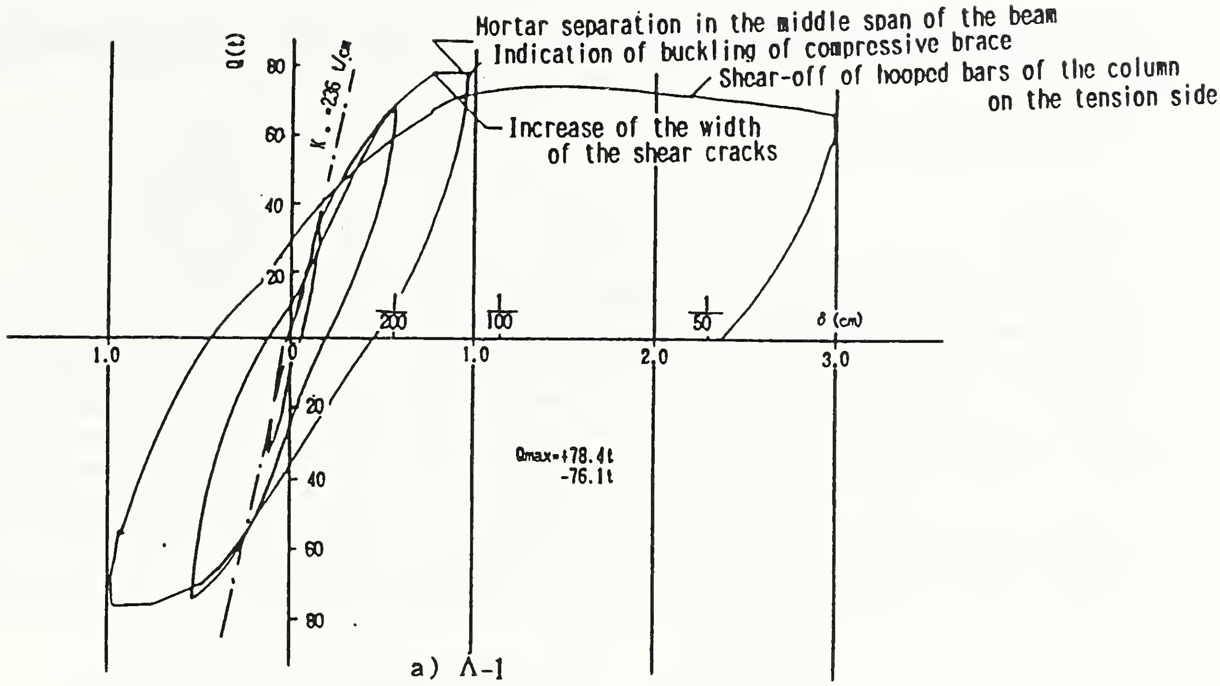
a)Test specimen of brace which shows the failure type I

b)Test specimen of panel which shows the failure type I

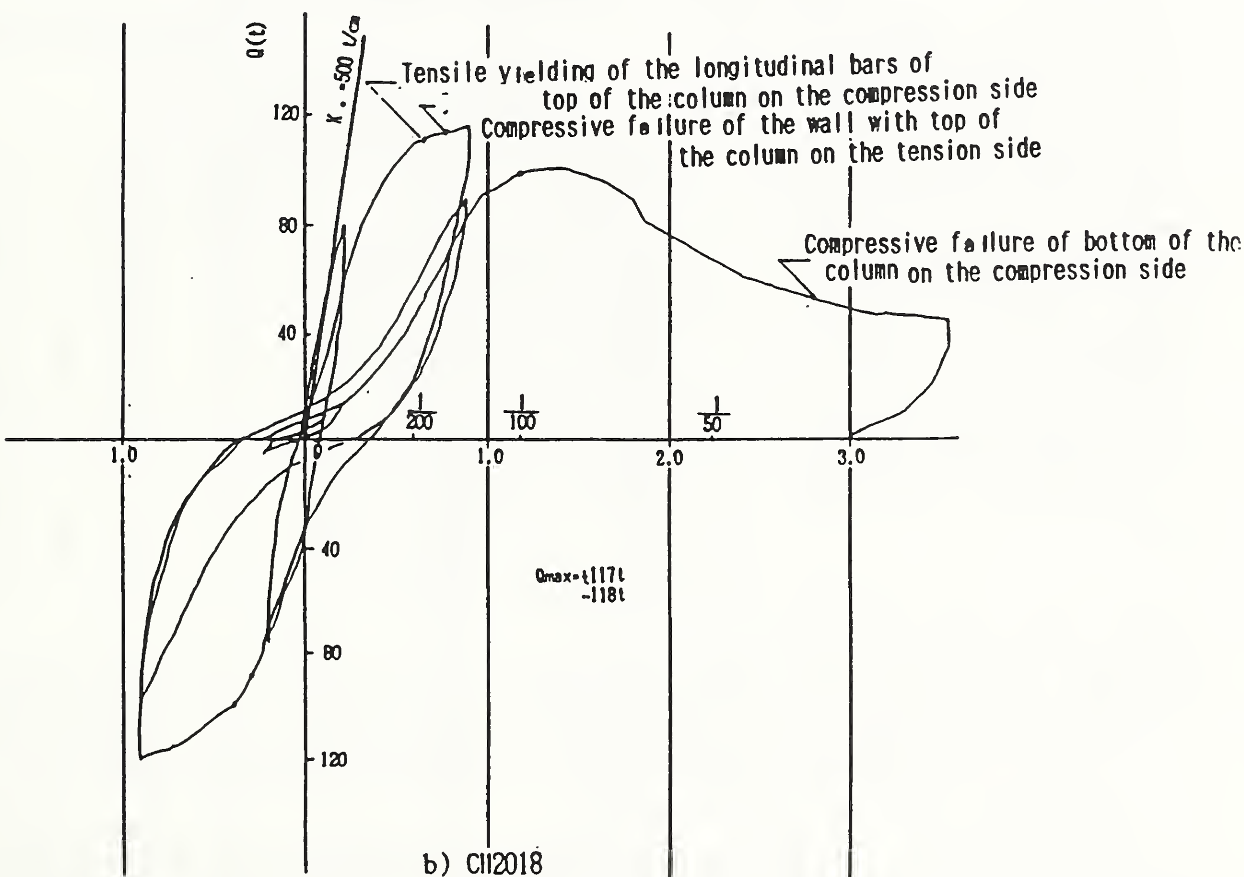
c)Test specimen of panel which shows the failure type II

d)Test specimen of panel which shows the failure type III

Fig. 7.1 The develop curve of framed steel braces and panel with rims



a) A-1



b) CH2018

Fig. 8.1 Comparison between the hystericistics characteristics of framed steel and that of post-cast RC walls

Method of Strengthing and Designing Shear Connectors Between Existing Reinforced Concrete Frame and Infilled Steel Brace and Panel

Yasutoshi YAMAMOTO* and Hiroyuki AOYAMA**

1. Abstract

There are a number of ways to strengthen an existing reinforced concrete building by integrating it with a framed steel brace and panel after removing non-structural materials, such as waist-high wall, drop wall and side wall, from the frame. It is, therefore, essential to design the connector so that it has satisfied strength. This paper will discuss indirect connectors that are resistant to stress and provide stable performance (ref.1,2 and 4). The inner circumference of the existing reinforced concrete frame is embedded with anchor bolts, while headed studs are welded to the outer steel frame at specified intervals. The studs and anchors are arranged alternately. High-strength and non-shrinkage mortar is added to integrate the existing reinforced concrete frame and the framed steel brace and panel. This integration with the framed steel brace and panel is referred to as strengthening. Here, the indirect connector means this integration of the existing reinforced concrete frame and the framed steel brace and panel. The process of determining the limit state shear strength on the basis of direct shear tests of the mortar joint and the details of its structure will be described.

2. Direct Shear Test Results

2.1 Specimens

Table 1 lists the specimens used for testing. The specimens are cutout models of mortar joints. Typical examples are BP-120S-O, -C and -T shown in Figure 1. Thirty-three specimens were tested: 23 full-scale specimens and 10 that were reduced by one-third. The cross-section of the mortar was designed to be 200mm thick in the full-scale specimens, while the thickness of the mortar was 80mm in the reduced-scale specimens to match the framed steel brace width. The reinforced concrete surface interfacing with the mortar was chipped to a depth of 5 mm to increase friction.

Chemical anchors of JIS SD30 specifications were used. The full-scale specimens used D19 anchors, while the reduced-scale specimens incorporated D10 anchors. Headed studs of 19ø and 16ø were used for the full-scale specimens and those of 9ø for the reduced-scale specimens. The compressive strength and Young's modulus of the infilled mortar were set with ample safety margins at 250 kgf/cm² and 210 tf/cm², and those of the concrete were set at 180 kgf/cm² and 210 tf/cm², respectively.

Table 1. Specimens of direct shear test

Note: o: Specimens with studs of 2-16ø
*: Specimens with studs of 1-19ø

No.	Specimen mark	Size	h (cm)	Headed studs arrangement	d_s (mm)	l (cm)	l/d_s	L (cm)	p_j (%)
*1	BP-120S-0		19.6	S	19	17.0	8.95	14.4	1.18
2	BP-120S-C		19.6	S	19	17.0	8.95	14.4	1.18
3	BP-120S-T		19.6	S	19	17.0	8.95	14.4	1.18
*4	BP-180S-0		19.6	S	19	17.0	8.95	14.4	0.79
o5	BP-120D-0		19.6	D	16	17.0	10.63	14.4	1.68
o6	BP-120DR-0	full	19.6	D	16	17.0	10.63	14.4	1.68
o7	BP-180D-0		19.6	D	16	17.0	10.63	14.4	1.12
8	CP-120A-0		9.6	S	D19	9.6	5.05	(0)	1.20
*9	HP-120S-0		19.6	S	19	17.0	8.95	14.4	1.18
10	CP-120S-0		9.6	S	19	7.0	3.68	4.4	1.18
11	CP-180S-0		9.6	S	19	7.0	3.68	4.4	0.79
12	BM-60S-0		6.87	S	9	6.0	6.67	5.2	1.33
13	BM-60S-C		6.87	S	9	6.0	6.67	5.2	1.33
14	BM-60S-T		6.87	S	9	6.0	6.67	5.2	1.33
15	BM-90S-0	reduced	6.87	S	9	6.0	6.67	5.2	0.89
16	TM-60S-0		6.87	S	9	6.0	6.67	5.3	1.33
17	TM-60S-C		6.87	S	9	6.0	6.67	5.3	1.33
18	TM-60S-T		6.87	S	9	6.0	6.67	5.3	1.33
19	TM-90S-0		6.87	S	9	6.0	6.67	5.3	0.89
*20	CP-180S-0-4		12.0	S	19	9.0	4.74	6.0	0.79
o21	CP-120D-0-4		12.0	D	16	8.0	5.00	4.0	1.68
o22	CP-180D-0-4		12.0	D	16	8.0	5.00	4.0	1.12
o23	CP-240D-0-4		12.0	D	16	8.0	5.00	4.0	0.84
*24	CP-120S-0-6		16.0	S	19	13.0	6.84	10.0	1.18
*25	CP-180S-0-6	full	16.0	S	19	13.0	6.84	10.0	0.79
o26	CP-120D-0-6		16.0	D	16	11.0	6.88	6.0	1.68
o27	CP-180D-0-6		16.0	D	16	11.0	6.88	6.0	1.12
o28	CP-240D-0-6		16.0	D	16	11.0	6.88	6.0	0.84
29	CP-180D-T-6		16.0	D	16	11.0	6.88	6.0	1.12
30	CP-180D-C-6		16.0	D	16	11.0	6.88	6.0	1.12
o31	CP-180Do-0-6		16.0	D	16	11.0	6.88	6.0	1.12
32	TM-90S-0	reduced	7.0	S	9	6.0	6.67	5.0	0.89
33	TM-60S-0		7.0	S	9	6.0	6.67	5.0	1.33

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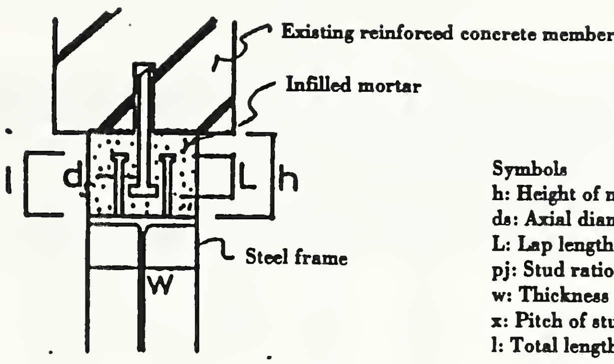


Symbols
h: Height of mortar section
ds: Axial diameter of studs
L: Lap length
pj: Stud ratio (sectional area of one pair of studs/w · x)
w: Thickness of mortar section
x: Pitch of studs
l: Total length of studs

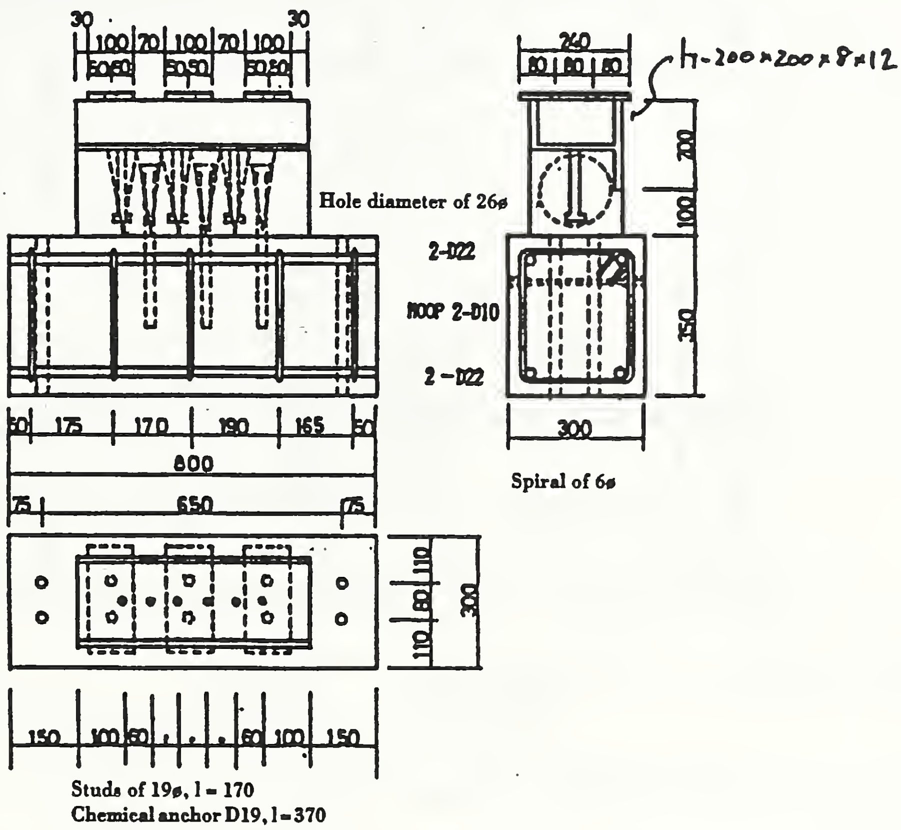


Figure 1. Details of specimens of BP-120S-0, -C and -T

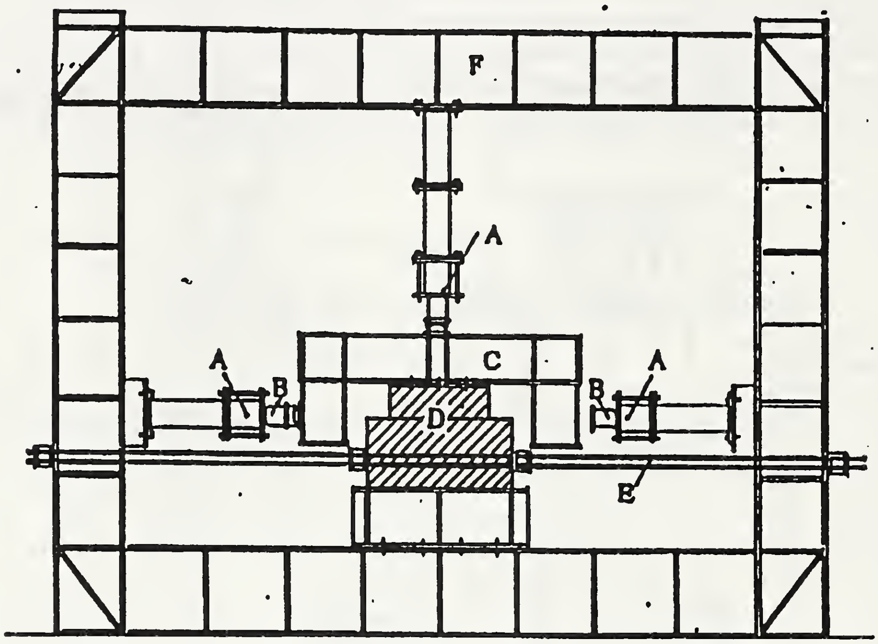
Arrangements of the chemical anchors and headed studs were determined on the basis of Fisher's formula. The shear connectors were arranged so that their mean shear stress at the failure stage τmax would be 20-30 kgf/cm² for the full-scale specimens and 30-50 kgf/cm² for the reduced-scale specimens. Axial pressure (perpendicular to the direction of the shear connector) was varied in the test to determine the relation between axial load and shear strength.

The first letter of the specimen mark corresponds to the shape of the steel framed section (see Table 1): B is for the bench type, C for the channel type, and H and T for original framed forms. The second letters, P and M, stand for full-scale and reduced-scale specimens, respectively. Numerals in the second segment of the specimen mark indicate the pitch of the chemical anchor bolts and studs, while the suffix letters S and D indicate single and double line arrangements of the studs, respectively. The letter A indicates the use of chemical anchor indirect connector. The letter R indicates the use of a ladder hoop for expanding crack prevention, while the lack of an R indicates the use of a spiral hoop. The third segment designates the status of axial pressure. An O indicates no axial pressure, while C indicates that compressive pressure was applied. A T means that tensile force was applied. Only specimen No.8 was a direct connector using D19 chemical anchors.

2.2 Loading Frame

Figure 2 illustrates the loading set-up for the direct shear test. Lateral load was applied so that the interface of the reinforced concrete element and the infilled mortar would be purely shorn. Therefore, a bending moment was applied, in addition to the shear load, to the border of the upper steel frame and the mortar section.

Positive and negative lateral loads in the first cycle were increased to a drift angle of 1/200 of the mortar element and up to about 1/100 in the second cycle. The positive load was increasingly applied in the third cycle until destruction was clearly observed.



A: 50 tf oil ram
B: 30 tf load cell
C: Loading frame
D: Specimen
E: PC steel rod
F: Reaction frame

Figure 2. Loading set-up for direct shear test

2.3 Results

Table 2, which summarizes the direct shear test results, shows the maximum shear load (Qmax), number of studs on specimens (n), shear strength per stud (q_s), mean shear strength (q_s/a_s) obtained by dividing q_s with a sectional area of stud (a_s), slip displacement (δu) at maximum shear strength, drift angle of the element R($=\delta u/h$) at maximum shear strength, axial pressure σ_0 and mode at destruction. The axial pressure was positive for compression and negative for pulling. The maximum load was barely detected on specimen No. 10.

The shear strengths of 16 specimens marked with * and o (ten specimens with studs of 2-16ø and six specimens with studs of 1-19ø) were as follows:
The average shear strength of a stud (q_s) was 5.45 tf per stud (= 2.71 a_s tf/cm²) in the case of 16ø, and 8.04 tf per stud (= 2.83 a_s tf/cm²) in the case of 19ø. Efficiency was better when 19ø studs were used in a single line.

Table 2. Results of shear tests

No.	Qmax (tf)	n	q_s (tf)	q_s/a_s (t/cm²)	δμ (mm)	Rμ (10^{-3}rad)	σNo (kg/cm²)	Failure mode
1	23.8	3	7.93	2.79	4.53	2.31	0	Stud
2	30.5	3	10.17	3.58	4.91	2.51	+10.0	Stud
3	16.5	3	5.50	1.94	11.62	5.93	-4.8	Stud
4	23.8	3	7.93	2.79	5.47	2.79	0	Mortar
5	25.6	6	4.27	2.12	3.60	1.84	0	Mortar
6	27.7	6	4.62	2.30	2.03	1.04	0	Mortar
7	28.5	6	4.75	2.36	2.99	1.53	0	Stud
8	16.5	3	5.50	(—)	6.03	6.28	0	Stud
9	23.3	3	7.77	2.74	2.35	1.20	0	Stud
10	(13.0)	3	(4.33)	(1.52)	(—)	(—)	0	Stud
11	(14.9)	3	(4.97)	(1.75)	2.13	2.22	0	Stud
12	10.6	4	2.65	4.14	4.98	7.25	0	Stud
13	13.0	4	3.25	5.08	7.31	10.64	+10.0	Stud
14	9.4	4	2.35	3.67	6.11	8.89	-4.8	Stud
15	10.8	4	2.70	4.22	2.00	2.91	0	Stud,Mortar
16	8.9	4	2.23	3.48	8.81	12.82	0	Stud
17	13.1	4	3.28	5.13	7.71	11.22	+52.0	Stud
18	7.2	4	1.80	2.81	7.21	10.49	-4.8	Stud
19	9.1	4	2.28	3.56	10.82	15.75	0	Stud
20	8.66	1	8.66	3.05	2.76	2.30	0	Stud
21	10.86	2	5.43	2.70	2.99	2.49	0	Mortar
22	11.04	2	5.52	2.75	4.08	3.40	0	Stud
23	13.72	2	6.86	3.41	4.89	4.08	0	Stud
24	8.91	1	8.91	3.14	(3.01)	(1.88)	0	Stud
25	6.97	1	6.97	2.45	8.80	5.50	0	Stud
26	11.97	2	5.98	2.98	2.95	1.84	0	Mortar
27	10.00	2	5.00	2.49	2.90	1.81	0	Stud
28	12.76	2	6.38	3.17	4.73	2.96	0	Stud
29	6.46	2	3.23	1.61	5.74	3.59	-1.28	Mortar
30	20.08	2	10.04	5.00	4.10	2.56	+74.2	Mortar
31	11.41	2	5.70	2.84	2.01	1.26	0	Mortar
32	1.96	1	1.96	3.06	2.30	3.29	0	Stud
33	1.90	1	1.90	2.97	6.04	8.63	0	Stud

2.4 Comparison with Other Stress Calculation Formulae

i) Applying the formula for calculating shear strength of the embedded anchor bolt according to the previous aseismic retrofitting guide (ref.4),
$qs = \min(0.4\, a_s \sqrt{E_m \times F_m},\ \sigma max \times a_s/\sqrt{3}) = \min(2.90\, a_s, 2.37\, a_s) = 2.37\, a_s$.
Accordingly, shear strength depends on the properties of the materials used for the stud.

ii) Utilizing the values recommended by Klingner et al,
$qs = \min(0.5\, \phi_c \times a_s \sqrt{E_m \times F_m},\ 0.75\, \phi_s \times a_s \times \sigma max)$
$= \min(0.235\, a_s \sqrt{E_m \times F_m},\ 0.675\, a_s \times \sigma max) = \min(2.35\, a_s, 2.77\, a_s) = 2.35\, a_s$.
Here the shear strength is determined by the properties of the mortar. øc and øs are capacity reduction factors of the concrete and stud, respectively. We adopted $\phi_c = 0.65$ and $\phi_s = 0.9$.

For the calculations of i) and ii), the mechanical properties of materials were assumed to be as follows:
Mortar: compressive strength $\sigma_M = 250$ kgf/cm² and Young's modulus $E_M = 210$ tf/cm².
Stud: yield strength $\sigma y = 3.0$ tf/cm², maximum strength $\sigma max = 4.1$ tf/cm².
Chemical anchor bolt: yield strength $\sigma y = 3.5$ tf/cm², maximum strength $\sigma max = 5.0$ tf/cm².

iii) Determination of permissible stress intensity
The strengths of the mortar joint calculated by the previous aseismic retrofitting design guide (ref.4) and the values recommended by Klingner et al,are in a close range from $q_s = 2.35 - 2.37\, a_s$, although the former value is determined by the tensile strength of the stud while the latter depends on the properties of the mortar. However, the mean shear strength (q_s) obtained from the results of the direct shear test is different from the two above-mentioned values and close to the respective high-end values of 2.90 a_s of the design guide and 2.77 a_s of the Klingner values. Further, the test results of the mortar joint show that slip displacement occurred at the boundary of the steel frame and the mortar, to reach destruction at the final stage. It is reasonable to consider that the shear strength is more dependent on the tensile strength of the stud than the shear strength of the mortar. We have

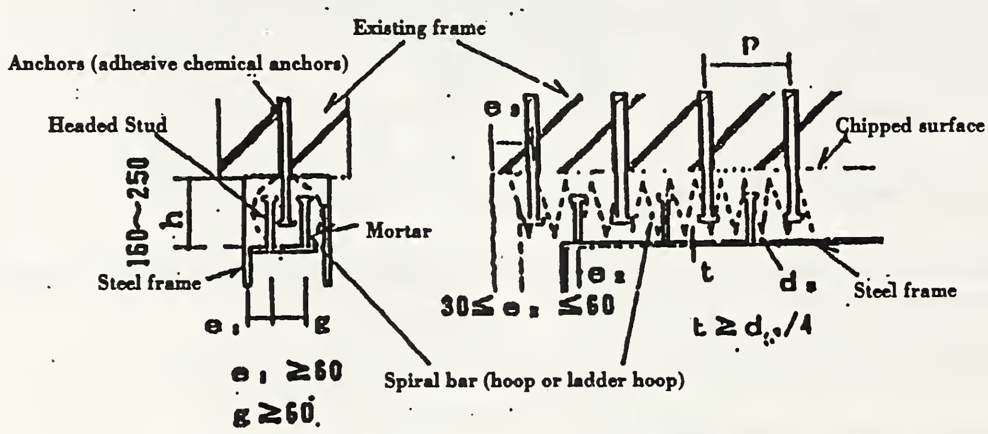
modified the capacity reduction factor of the values recommended by Klingner et al, to $\phi = 0.85$ to make the value 5% more safety bound and determined the allowable shear load of the stud as $q_a = 0.75 \times 0.85 \times 4.1\ a_s = 0.64\ \sigma max \times a_s = 2.61\ a_s$ (tf/cm²).
As a result of the modification, the permissible shear load per stud (q_a) is 5.24 tf for the 16mm diameter stud and 7.41 tf for the 19mm diameter stud.

iv) Comparison of the test results and permissible design load
Comparison of the permissible shear load values for design with the test results revealed that six of ten values for the 16ø specimens and five of six values for the 19ø specimens were safety-bound (Table 2).

2.5 Reasons for Concluding Equal Shear Strength for 2-16ø Studs and 1-D19 Chemical Anchors

Only 16ø and 19ø studs (comparable to SR24) and 1-D19 chemical anchors (comparable to SD35) were tested. The tensile force of the studs used for testing was 4.38-5.15 tf/cm², while that of the chemical anchor was 5.53-5.56 tf/cm². When the 2-16ø studs and the 1-D19 chemical anchors were used in the shear connector arrangement, the tensile force of the 1-D19 anchors was smaller than that of the 2-16ø studs. It is seemeds unreasonable to determine the strength of the mortar joint on the basis of the yielding strength of the stud, as this would have resulted in a danger-bound value being adopted. However, no chemical anchors were destroyed before the studs were broken in the direct shear test of the mortar joint with the above-mentioned arrangements.
According to the hypothesis of shear friction, the coefficient of friction between the mortar and the steel frame is $\mu = 1.0$. As the coefficient of friction of the interface between the chipped surface of the reinforced concrete and the new concrete can be assumed to be $\mu = 1.4$ (refer to 7), it is clear why the chemical anchors did not yield before the studs in the direct shear tesf. Accordingly, we adopted the arrangement of 1-D19 chemical anchors of SD35 along with 2-16ø studs of SR24.

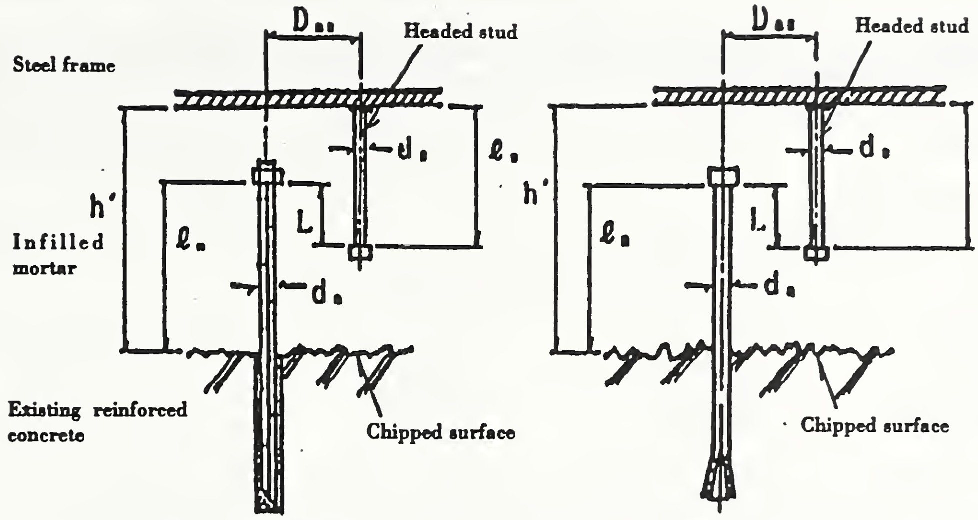
3. Details of Structure



e_1: Edge clearance of the headed stud and the steel frame
e_2: End distance of the headed stud and the steel frame
g: Gauge of the headed stud

Figure 3. Indirect mortar joint with steel frame for strengthening

Based on the test results mentioned in section 2, details of the structure, which were adopted in the draft of the Design Guide for Retrofitting, are described below. For details of the structure of the indirect mortar joint with framed steel brace, the example in Figure 3 is recommended. Performance of this joint was confirmed through testing, and a number of actual work examples are available. The details include items of the steel frame, anchor bolts, headed studs, strength of infilled mortar, spiral hoop, hoop and ladder hoop for crack prevention.



Adhesive (chemical) anchor

Mechanical anchor

Ds: Diameter of the stud
Da: Diameter of the anchor
h': Height of the infilled mortar
ln: Fixing length of the anchor
L: Lap length of the stud and anchor

Figure 4.

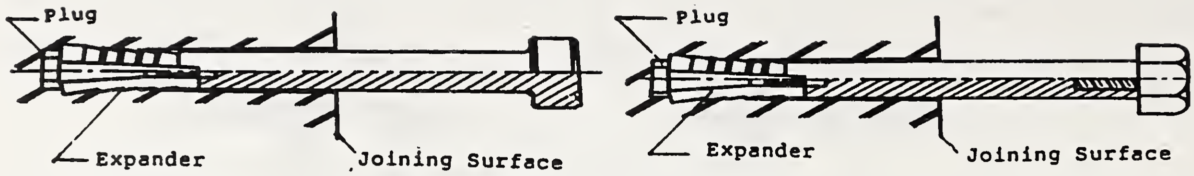
3.1 Steel frame

The average clearance of the steel frame is set at 160-250mm from the chipped surface of the existing reinforced concrete, and the mortar is filled into this space. The clearance is required for construction work and for fixing the anchors and headed studs.
The plate thickness of the steel frame must be >= $d_s/4$ (d_s: axial diameter of the stud) to assure good welding of the stud and the frame.

3.2 Anchors

As a rule, the anchors should be arranged on all the circumferences of posts and beams to equally distribute stress as much as possible. If, for some reason, anchors cannot be embedded into the posts, extra care should be taken when the tensile load is placed on the anchors embedded in the beams.
A full-scale test of an indirect mortar joint with steel frame for strengthening was carried out in only one case for an adhesive (chemical) anchor of D19 (with embedded depth of 9 d) (ref.1,2,3). However, the use of a mechanical anchor was judged possible as (1) the shear force is mainly applied to the anchors when they are distributed at all circumferences of the posts and beams, (2) the shear strength is calculated for safety-bound values based on a number of direct shear tests using different anchors of different diameters and (3) the shear strength was determined by the studs in the indirect mortar joint with steel frame (ref.3).

It is recommended, however, to observe following precautions.
i) Anchors for the mortar joint for strengthening should be embedded at all circumferences of the posts and beams after the contact surface has been chipped. Either chemical or mechanical anchor can be used, but avoid using the two together.
ii) Use only anchors with axial diameters larger than 16mm.
iii) The effective embedded depth of the chemical and mechanical anchors should be more than 7 d_a.
iv) The fixing depth of the anchors (from the neck of the anchor to the surface of the existing concrete) in the mortar section (ln) should be 6 d_a or more (d_a: axial diameter of the anchor).
v) Headed mechanical anchors should be used (as illustrated in Fig.5).
vi) The pitch of the anchors should be less than 250mm.



a) unit type (recently developed) b) bolt and nut type

Figure 5. Mechanical anchor to strengthen steel frame

3.3 Headed studs

Headed studs of JIS B1198 specifications should be used. As the full-scale test was carried out only with studs of 16mm and 19mm in diameter, additional testing will be required for studs of different diameters.
i) The pitch of the headed studs should be less than 250mm. It is advisable to use the same pitch as that of the anchors to make construction work easier.
ii) Use headed studs with an axial diameter of 16mm or 19mm.
iii) The length from the neck of the stud to the steel frame surface (ls) should be more than 6 d_a, the same as in the case of the anchors.
iv) The arrangement should be in double lines when 16ø studs are used and in single line when 19ø studs are employed.
v) The following size recommendations should be observed.
Edge clearance of the headed stud and the steel frame e_1: longer than 60mm
End distance of the headed stud and the steel frame e_2: longer than 30mm and shorter than 60mm
Gauge of the headed stud g: greater than 60mm
As welding work may be difficult at the edge of the steel frame, care should be taken when arranging the studs.

Table 3. Height to the neck of the headed studs and lap length

Clearance h'	anchors <required ln> ln	headed studs <required ls> ls [tabulated length]	lap length L
160	D16 or 16ø<96> 110	16ø double <96> 110 [120]	60
200	D19 or 19ø <114> D22 or 22ø <132> 140	16ø double <96> 19ø double <114> 19ø single <114> 140 [150]	80
250	D19 or 19ø <114> D22 or 22ø <132> 190	19ø double <114> 19ø single <114> 190 [200]	130

Tabulated length is (ls + 10)mm.

3.4 Lap length of the anchor and stud, and their distance

The lap length of the anchor and headed stud should be longer than 1/2 of the distance to the neck of both.
The lap length (lapped length of fixed parts of the anchor and stud: $L = l_a + l_s - h'$) should satisfy the following equation (Fig. 4):
$L >= \max(l_a/2, l_s/2)$
The value for ln is determined by assuming the values of l_s and h'.
* When $l_s <= (2/3)h'$, $l_a >= 2(h' - l_s)$.
* When $l_s >=(2/3)h'$, $l_a >= h' - l_s/2$.

The clearances between the anchor and the steel frame, and between the stud and the existing reinforced concrete should be around 50mm for the sake of construction work. A relation of about $l_s = l_a$ is desirable.
The distance between the neighboring anchor and stud D_{as} should be less than twice the lap length. When an unusually large tensile load is applied to the anchor and stud, $D_{as} <= L$ is required.
Table 3 provides an example. By assuming the clearance between the existing reinforced concrete and steel brace to be 160mm, 200mm and 250mm, and determining diameters and fixing lengths of the anchors and headed studs, the required fixing length in the mortar section and lap length are calculated.

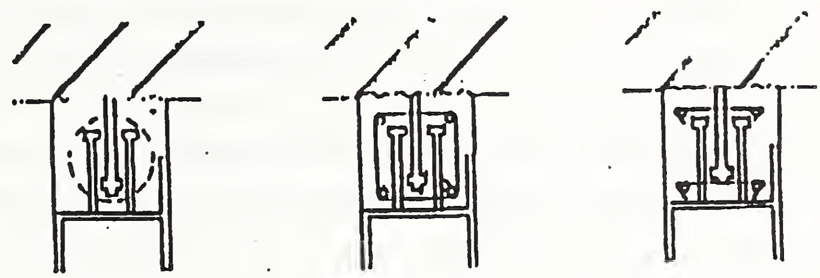
3.5 Infilled mortar

As a rule, non-shrink mortar is grouted into the space between the existing reinforced concrete and framed steel brace. This eliminates any void below the beams and assures the shear strength of the mortar joint.

The following properties of the infilled mortar are recommended.

(1) The compressive strength of the mortar should be greater than 300 kg/cm^2.
(2) There should be good adhesion to the existing reinforced concrete and steel frame surface.
(3) No volume tric expansion should occur after hardening, although slight expansion is needed.
(4) There should be minimal bleeding or sand separation.
(5) There should be good fluidity and ease of work.

3.6 Spiral hoop, rectangular hoop and ladder hoop



Spiral bar Rectangularhoop bar Ladder hoop bar

Figure 6 Reinforcement for preventing crack expansion

Insert the spiral hoop, rectaugular hoop and ladder hoop into the mortar to distribute cracks in the mortar (Fig. 6). When a large stress is applied to the mortar joint for strengthning, the spiral hoop and rectaugular hoop are particularly effective in restraining the mortar and transferring stress to the anchors and studs. The reinforcing bar ratio p_s in the mortar should be larger than 0.4%. The formula is $p_s = a_s/h' \times x_s$ (x_s: pitch of the reinforcing bars, a_s: sectional area of one set of the reinforcing bars, h' = height of the infilled mortar).

Conclusion

We have reported the results of tests on indirect mortar joints used to strenghten existing reinforced concerete frame, and a design guide (draft) for strengthening based on these test results. We stress that the mortar joint is the most critical element in strengthening existing reinforced concrete frame with the use of a framed steel brace and panel. Therefore, the design of such joints requires extra care and attention.

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Ultimate Shear Strength of Reinforced Concrete Members with Extremely Small Shear Span Ratios

by

Yasutoshi YAMAMOTO[1]

ABSTRACT

In generally, it is known that in inverse proportion to the ratio of shear span to depth of reinforced concrete (herein after refered to as RC) member, ultimate shear strength increases, however, ductility in it's state decreases.

When a great deal of concentrated load is added to the RC member of which shear span ratio is less than one, failure of sllipage often occurres in close vicinity of the end of RC member. Here, this failure is called punching shear failure in common with slab's case, and the maximum shear resistace is called punching shear strength. This paper describes determination of the punching shear strength to be adopted in the revised "Design Guideline for Aseismic Retrofitting of Existing Reinforced Concrete Buildings".[5)]

KEYWORDS: Brace; Panel; Punching shear; Retrofit; Shear strength; Strengthening.

INTRODUCTION

When existing RC frames are planned to strengthen with steel walls or post-cast RC walls, these walls show a tendency to shear the vicinity of ends of the RC beams and/or columns, because of the lack of jointing strength between the existing part and post-cast part.[6)] In this case, punching shear failure tends to appear at the RC beam and/or column. Therefore, it is very important and urgent to find out the ultimate punching shear strength to evaluate an aseismic strengthened RC buildings. It is considered that this punching shear strength depends upon the magnitude of basic shear strength (equal to ultimate shear strength under direct shear load) and influence factor varied with shear span ratio. Comparing with shear test results, the value of ultimate shear strength in RC members with shear span ratio less than one, are discussed as follows.

BASIC SHEAR STRENGTH

Here, Basic Shear Strength " τ_0 " means the ultimate shear strength in RC

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sectional area, obtained from direct(pure) shear test like a push-off test (see Fig.1). A. H. Mattock et al. have described the failure mechanism of concrete section arranged reinforcing bar perpendicularly.[1] And, they conclude that shear in the state of ultimate loading is finally transfered by concrete compression and by tension of reinforcement as the truss action . That is, volumetric expansion of aggregates in concrete by means of their interaction leads to the elongation of perpendicular reinforcement, and the reinforcement resists against the slippage at the shear face by friction. According to the shear friction hypothesis, shear resistance equals to shear friction caused by compression among aggregates. [2] This shear friction is finally proportional to the following horizontal tention force (σ_x).

$$\sigma_x = p_s \times f_y \quad \cdots\cdots\cdots (1)$$

where, p_s :ratio of total sectional area of reinforcing bars perpendicular to shear concrete section ($= a_s /(t \cdot D)$)

t :thickness of concrete member

D :depth of concrete member

f_y :yield strength of re-bar

This shear friction hypothesis is assumed to be applicable for punching shear failure appeared in short spanned RC members. And the failure envelope curve, proposed by Mattock, is used in determining the ultimate strength of RC members under direct shear. Relatioship between basic shear strength (τ_o) and compressive stress (σ) at the shear section is determined according to Fig.2. Substantially, the parameters affecting τ_o are considered as sectional area of reinforcing bars, it's yield strength, and concrete compressive strength(σ_B). Therefore, total stress acting on the shear face (σ) is equal to ($p_s \times f_y + \sigma_o$, here, σ_o : axially added constant compressive stress).

To decide the Mattock's envelope curve, next steps are set forward.

1) Line OA on the Cartesian coordinates in Fig. 2, is drawn as the constant value $\tau / \sigma_y = [Q/(t \times D)]/[Q/(t \times w)] = w/D = 1.0$, in case of push-off specimens.

2) Circles which place a center at any point on x-axis, and which contact on the Zia's envelope curve inside, are drawn successively. In Fig. 2, only a circle is illustrated for example.

3) Point A' is decided as the intersection point of a circle, and line OA is drawn as a 45 degree line downward to the right, and Line O_1A' which connects the center of circle (pointA_1) and point A' , is extended to the opposite circumference of a circle, in order to decide point B'.

4) Finally, Mattock's envelope curve regarded to τ vs. σ relationship, is

illustrated as sequences of intersection point B' for many circles. Thus, by means of Mattock's envelope curve, we can arbitrarily obtain the ultimate shear strength(σ) in the case of non shear span ratio. However, this process for the ultimate shear strength, that is, the basic shear strength(τ_o), is too much bothersome for design purposes. So we should try to derive an approximate value.

APPROXIMATE BASIC SHEAR STRENGTH

To simply obtain the basic shear strength, an approximate method substituting the two lines for Mattock's envelope curve is developed as follows.

1) Expressions for Zia's envelope curve

To decide Zia's envelope curve in Fig.2, in the first place, two circles ① and ② must be drawn. Circle① is the one that has a diameter equal to safety-bound cocrete compressive strength (σ_c =0.85 σ_B). Circle ② is another one that has a diameter equal to safety-bound cocrete tensile strength (σ_t = 1.6 $\sqrt{\sigma_B}$). Then, expressions of circule① and line ① contacting at slope 37 degree with circule ① are expressed as follows:

$$(\sigma - \sigma_c/2)^2 + \tau^2 = (\sigma_c/2)^2 \quad \text{for circle ①} \quad \cdots\cdots (2)$$

$$\tau = \sigma \tan 37^\circ + a \quad \text{for line ①} \quad \cdots\cdots (3)$$

Here, value (a) is the distance from origin to point of intersection where straight line ① being contact with circle ① at slope 37 degree crosses y-axis.

Substituting τ in eq.(2) with τ in eq.(3), we get a next expression.

$$(\sigma - \sigma_c/2)^2 + (\sigma \tan 37^\circ + a)^2 = (\sigma_c/2)^2 \quad \cdots\cdots (4)$$

As circle ① should contact to the straight line ①, therefore, eq. (4) should have multiple root regarding to σ. From discriminant of eq. (4), distance (a) will be obtained.

$$a = \left(\sqrt{(\tan 37^\circ)^2 + 1} - \tan 37^\circ\right)\sigma_c/2 = 0.249\sigma_c \quad \cdots\cdots (5)$$

where, $\sigma_c \geqq 0$

Similar relation is obtained for circle ② and straight line②.

$$(\sigma + \sigma_t/2)^2 + \tau^2 = (\sigma_t/2)^2 \quad \text{for circle ②} \quad \cdots\cdots (6)$$

$$\tau = \sigma \tan\alpha^\circ + a \quad \text{for line ②} \quad \cdots\cdots (7)$$

where, $\sigma_t \leqq 0$, and α is a gradient of line② which contacts a circle ② and passes through point (a) on y-axis.

As the results, next equation concerning multiple root is obtained.

$$(2a \times \tan\alpha + \sigma_t{}^2 - 4a^2((\tan\alpha)^2 + 1) = 0 \quad \cdots\cdots (8)$$

Finally, gradient of line② is to be determined.

$$\alpha = \tan^{-1}(a/\sigma_t - \sigma_t/4a) \quad \cdots\cdots (9)$$

Therefore, Zia's envelope curve will be drawn by eq.(3) to eq.(8).

2) Approximate Mattock's envelope curve[3)]

It is necessary to rapidly predict the basic shear strength as simple as possible. Then, provided that the shape of push-off specimens has constant w/D (= 1.0), it will be capable to approximately represent Mattock's envelope curve as the two straight lines(see, Fig.3). These lines are to be drawn, if we can obtain the primary four points as follws;

• Point 1(see, Fig.4): When a particular circle possesses center at 0_1, and contacts Zia's envelope curve at point (a) on y-axis, opposite-side point B' against point A' on the circumstance is defined as point 1.

• Point 2(see, Fig.5): When a particular circle contacts to the line① , furthermore, passes through origin (0), that is, stress circle for the pure compressive strength contacts the line① , point B' is coresponding to point 2. Therefore, coordinates (σ, τ) of point 2 are decided by following expression:

$$\sigma = \tau = \sigma_c / 2 \quad \cdots\cdots (10)$$

• Point 3(see, Fig.6): When a particular circle possessing the center at 0_1, goes across distance (a) on y-axsis, point 3 is corespons to the B'. The circle is drawn according to next expression.

$$(\sigma - a \times \tan \alpha^\circ)^2 + \tau^2 = a^2(1 + (\tan\alpha^\circ)^2) \quad \cdots\cdots (11)$$

Point A' in Fig.6 is obtained substituting $-\sigma$ for τ in eq.(11). Therefore,

$$\left.\begin{aligned} \sigma &= a(\tan \alpha^\circ + \sqrt{(\tan\alpha^\circ)^2 + 2})/2 \\ \tau &= -a(\tan \alpha^\circ + \sqrt{(\tan\alpha^\circ)^2 + 2})/2 \end{aligned}\right] \quad \cdots\cdots (12)$$

Point B' is a symmetric point against point A'. Therefore, the coordinate of point 3 is

$$\left.\begin{aligned} \sigma &= a(3\tan\alpha^\circ - \sqrt{(\tan\alpha^\circ)^2 + 2})/2 \\ \tau &= -a(\tan \alpha^\circ + \sqrt{(\tan\alpha^\circ)^2 + 2})/2 \end{aligned}\right] \quad \cdots\cdots (13)$$

• Point 4(see, Fig.7): Among the stress circles possessing the center 0_1 on x-axis, besides contacting line ②, a particular stress circle would be chosen which point B' on the circumference of a circle is to be on the y-axis. For this purpose, angle $\angle 0B'0_1$ should be 1/2 ($\pi - \angle A'0B'$). The circle satisfied this condition is expressed as follows;

$$\tau^2 + (\sigma - \xi)^2 = 5\xi^2 \quad \cdots\cdots (14)$$

where, ξ indicate a length 00_1

Since the circle of eq. (14) must contact the line ②, substitution τ in eq.(7) for τ in eq. (14) results in following expression.

$$(\sigma \times \tan \alpha^\circ + a)^2 + (\sigma - \xi)^2 = 5\xi^2 \quad \cdots\cdots (15)$$

As eq. (15) should have multipule root, discriminant is as follows;

$$(a \times \tan\alpha - \xi)^2 + ((\tan\alpha)^2 + 1)(a^2 - 4\xi^2) = 0 \quad \cdots\cdots (16)$$

Solving eq.(16) in relation to ξ, next equation will be obtained.

$$\xi = a \{\tan\alpha + \sqrt{5((\tan\alpha)^2 + 1)}\} / (5 + 4(\tan\alpha)^2) \quad \cdots\cdots (17)$$

Therefore, coordinate of point 4 is as follows;

$$\left.\begin{aligned} \sigma &= 0 \\ \tau &= 2\xi = 2a \{\tan\alpha + \sqrt{5((\tan\alpha)^2 + 1)}\} / (5 + 4(\tan\alpha)^2 \end{aligned}\right] \cdots (18)$$

Approximate envelope curves ① and ② in Fig.3, are obtained by connecting above mentioned four points 1 to 4. Line(τ_{01}) is the line ① tied point 1 to point 2, and line(τ_{02}) is the line ② tied points 3 to 4, respectively.

As a result, those equations are represented as follows;

$$\tau_{01} = 0.254\sigma_B + 0.493\,\sigma$$

$$\tau_{02} = [\ \{16b^3 + 16b - 4\sqrt{5(b^2-1)}\} / \{(3b - \sqrt{b^2+2})(5 + 4b^2)\} - 1\ ]\,\sigma + \{2a(b + \sqrt{5(b^2+1)}\} / (5 + 4b^2) \quad \cdots\cdots (19)$$

where, $b = \tan\alpha$

Thus, Mattock's envelope curve are to be expressed by eq.(19). However, the second expression(τ_{02}) in eq.(19) are too complex to adopt as approximate one. Therefore, in place of eq.(19), next eqation was prepared.

$$\left.\begin{aligned} \tau_{01} &= 0.22\sigma_B + 0.49\sigma \text{ (kgf/cm}^2\text{), for } (0.33\sigma_B - 28) \\ \tau_{02} &= 10.0 + 0.1\sigma_B + 0.85\sigma \text{ (kgf/ cm}^2\text{), for } 0 < \sigma \leqq 0.66\sigma_B \end{aligned}\right] \cdots\cdots (20)$$

This eqations also can be illustrated as Fig. 8.

3) Influence factor due to changes of shear span ratio

In preceding section, we discussed the shear strength of RC members regarded as non shear span ratio. In this section, how to predict the influence factor affecting on shear strength for any state of shear span ratio (a/D), would be described. Here, influence factor is defined as the value of shear strength obtained from RC specimens having (a/D) more than zero devided by the value of eq. (20) as basic shear strength. Fig. 9 shows the values of the strength decreasing tendency, coresponding to shear span ratio, obtained from testing results. On condition of cosidering decline of shear strength under repeating load, influence factor in fig. 9 is modified by multiplying ϕ as 0.8 . Data were totally 106 specimens, contained 22 specimens for push-off type as to direct shear test, and 58 specimens for beam and column, and 26 specimens for shear wall, having shear span ratio less than one. From Fig. 9, clearly, we can recognize that shear strengh is reversely proportional to the shear span ratio.

As the results, average and minimum influence factors with regard to the shear span ratio are expressed by next equations, respectively.

$$\left.\begin{aligned} k_{av(erage)} &= 0.58/(0.76 + a/D) \\ k_{min(imum)} &= 0.34/(0.52 + a/D) \end{aligned}\right] \quad \cdots\cdots (21)$$

where, $a/D \leq 1.0$

Though the conditons are treated as neglisible as regards shear reinfocement like hoop and stirrup, equation (21) is cosidered effective shear estimation for extremely short spaned members.

4) Decision of punching shear strength

In preceding sections, we have described how to decide both basic shear strength and influence factor. In the result, the punching shear strength(Q_p) for RC member will be obtained from next expression within the safty-bound.

$$Q_p = k_{min} \times \tau_0 \times b_e \times D \quad \cdots\cdots (22)$$

where, τ_0 : basic shear strength either lower value in eq.(20)

be : effective width of RC member

D : depth of RC member

Flow chart in Fig. 10 shows a process to obtain the punching shear strength.

VERIFICATION BY TESTS

Outline of testings, and the comparison between test results and expression derived from eq. (22) will be described in following article.

1) Specimens and loading

Tests were carried out in laboratory to veryfy the above mentioned punching shear strength(Q_p). Test specimens were produced by ten in type of cantilever (see Fig. 10). The root of every specimens was fixed to rigid concrete foundation. Constatly applied axial pressure (σ_0) was changed from -20 to 40 kgf/㎠ to observe the differences of punching shear behavior. Specimen's marks are shown in Table 1. First letter of the specimen mark corresponds to the positive or negative direction of axial pressure: C for compression, and T for tension. Second letter stands for quantities of axial pressure (σ_0) in unit (kgf/ ㎠). Third letter is the number of specimens. Specimen's dimensions of concrete piece were 200mm width, 250mm depth, and 400mm length, respectively.

Longitudinal reinforcement was 6-D16 (p_g =2.4%) for specimens axially applied compression, and 6-D19 (p_g =3.4%) for axially tensioned specimens. Compressive strength of cocrete was 211kgf/ ㎠ in average except for 235kg/ ㎠ in specimen T-20-2. Tensile yield points of re-bars were 3570 kgf/ ㎠ for D19, 3750 kgf/ ㎠ for D16, and 2340kgf/ ㎠ for 4ϕ, respectively.

For all specimens, lateral reinforcement was arranged 2-4 ϕ 120ctc(pw=0.1%). Oneway repeating shear load Q was applied at the position of shear span equal to 110 mm from the top of foundation(ratio a/D= 0.44). The load was applied up to the occurrence of shear crack in the first cycle, and up to the outstanding failure in the second cycle.

2) Test Results

The latter of Table 1. lists the characteristics of test results. There are flexural cracking load (Q_{BC}) and it's deflection(δ_{BC}), shear cracking load (Q_{SC}) and it's deflection (δ_{SC}), moreover, maximum load (Q_U) and it's deflection (δ_U) respectively. As decreasing of rigidity after flexural and shear cracking was not outstanding, each load-deflection relationship drew smooth curves. However, after experiencing the maximum load, there were remarkable decreasing of strength and cracks along the longitudinal reinforcing bars, especially, conspicuous concrete slippage in vicinity of the top of foundation.

3) Discussions of Test results

i) Relation between shear strength and constant axial pressure

The relationship between shear stress divided by concrete compressive strength (τ/σ_B) and constant axial pressure (σ_0) are plotted in Fig. 12. Each value of τ/σ_B is plotted concerning the times of characteristic shear and flexural cracks, and ultimate strength. Each regression line is drawn in the figure. On the whole, Each line has a trend of increasing with axial pressure. Especially, the value of τ/σ_B at the shear crack is strongly affected by axial pressure. Most important value of τ_u/σ_B at the ultimate shear strength is expressed as follows;

$$\tau_0/\sigma_B = 0.3 + 1.2 \times 10^{-3} \quad \sigma_0 \qquad \text{for } a/D = 0.44 \quad \cdots\cdots\cdots\cdots (23)$$

ii) Comparison the test results with some expressions

The expressions of this proporsal (eqs. 20, 21 and 22), and those of previous retrofitting guideline[5] and ACI(318-83) [4], are presented in Fig. 13. Test results show that the expressions in our proporsal have the nearly same gradient regarding to the axial pressure, and safty allowance for both average and minimum punching shear strength.

CONCLUSION

The value of punching shear strength was derived from the multiplying basic shear strength by influence factor. The basic shear strength was obtained from the simplified Mattock's envelope curve and the influence factor was from many test results regarding beam, column and shear wall. Comparing with the test results of RC members having shear span ratio less than one, ultimate punching shear strength(Q_p) using average influence factor(k_{av}) is sufficiently lower than tested one. Therefore, it is evident that our proposed expression used minimum influence factor(k_{min}) is safety-bound for punching shear strength, and is effective to the retrofitting design.

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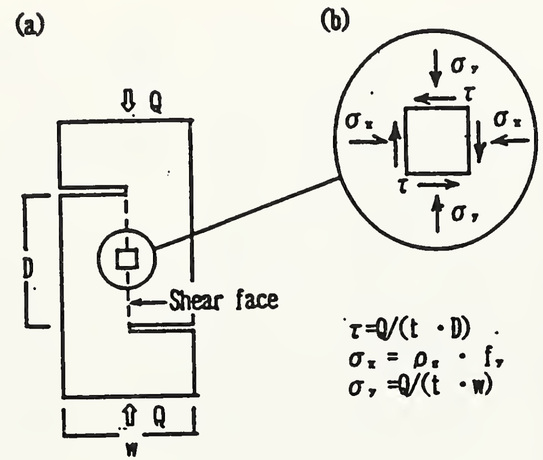


Fig.1 Push-off type specimen and stresses at shear face

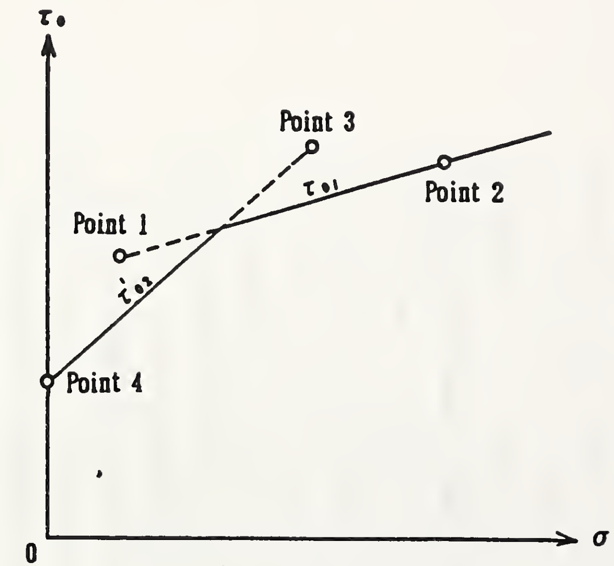


Fig.3 Mattock's approximate envelope curve

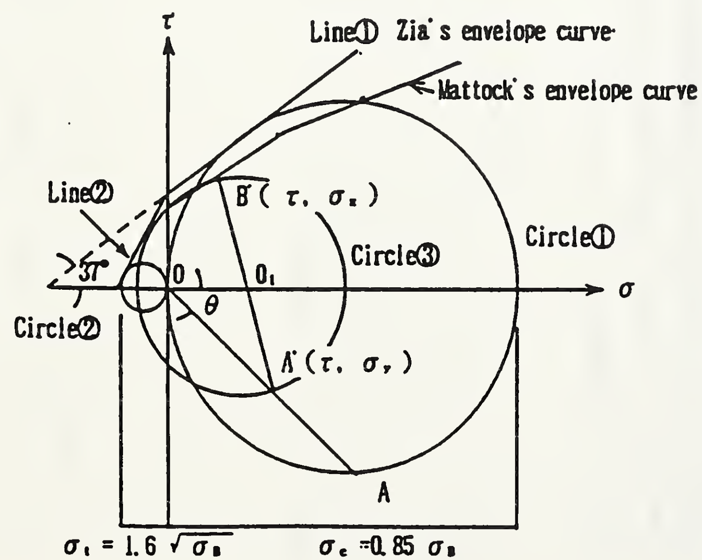


Fig.2 Zia's and Mattock's envelope curves

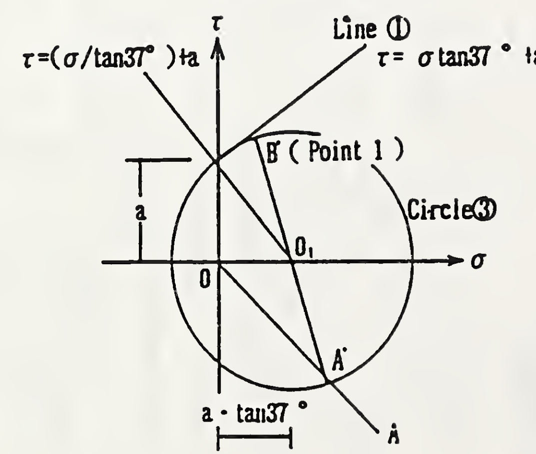


Fig.4 Position of Point 1

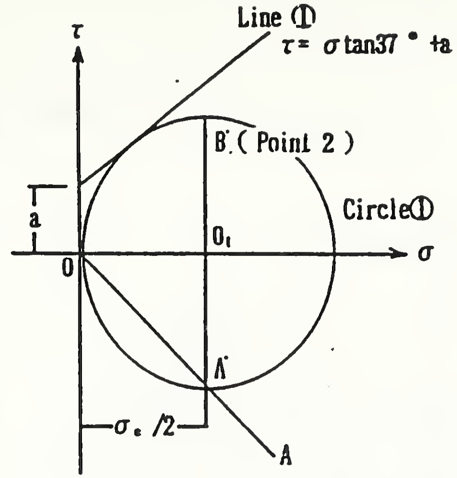


Fig. 5 Position of Point 2

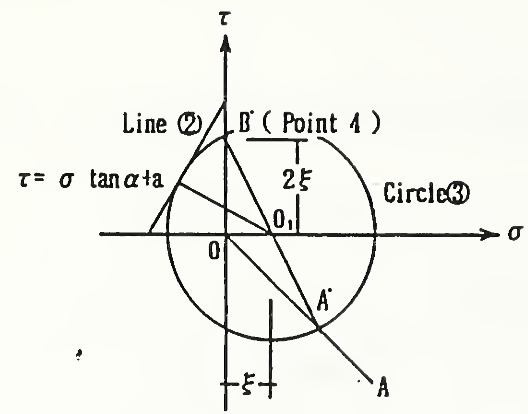


Fig. 7 Position of Point 4

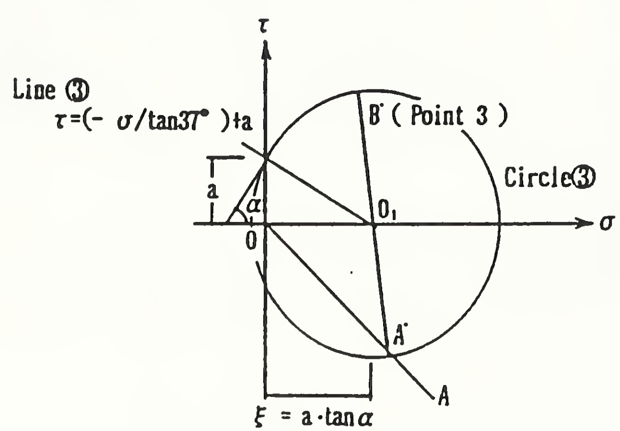


Fig. 6 Position of Point 3

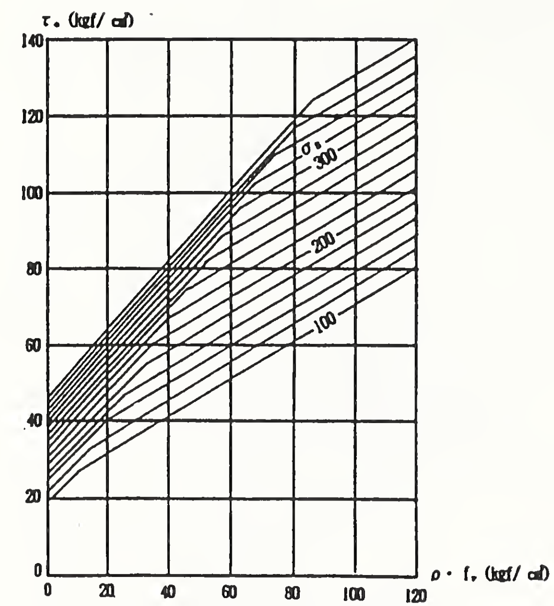


Fig. 8 Groups of Mattock's approximate euvelope curve

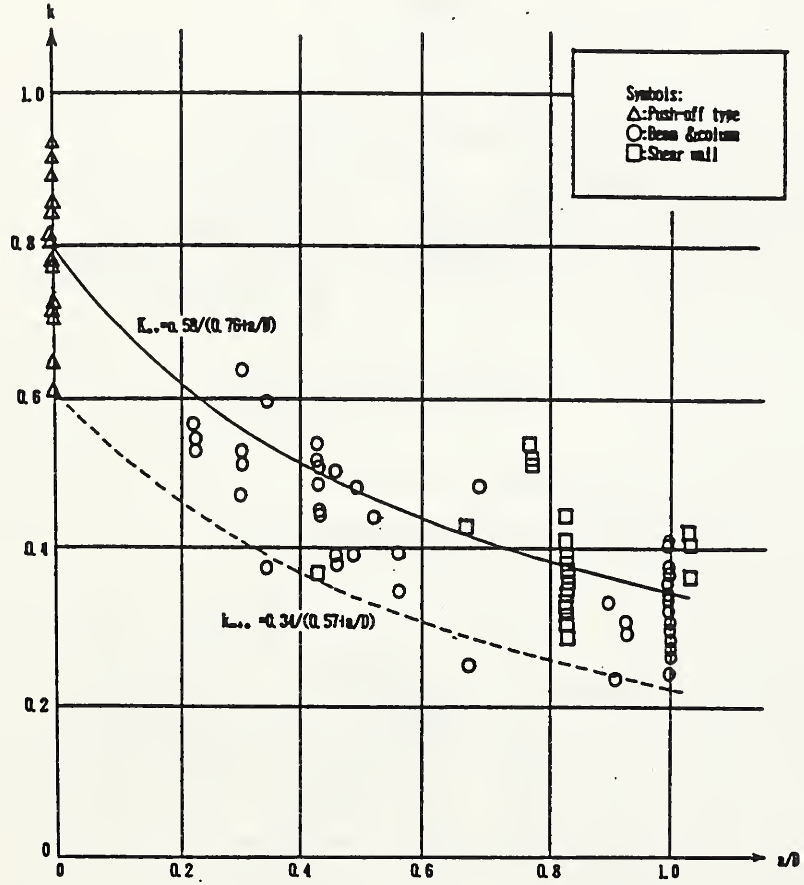


Fig. 9 Relation between k and a/D (a/D ≤1.0)

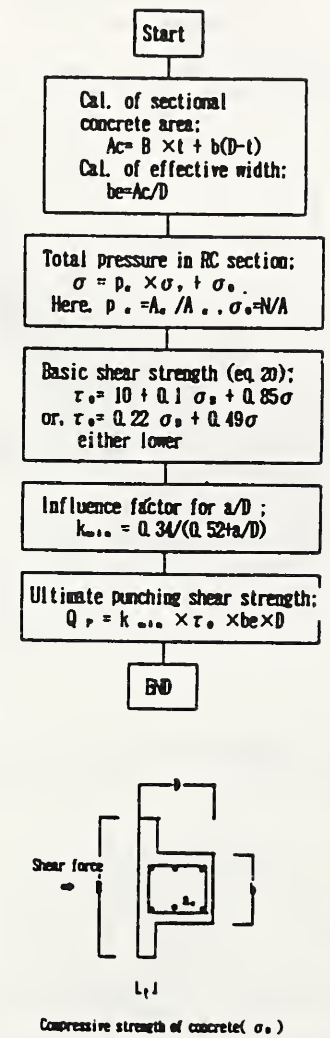
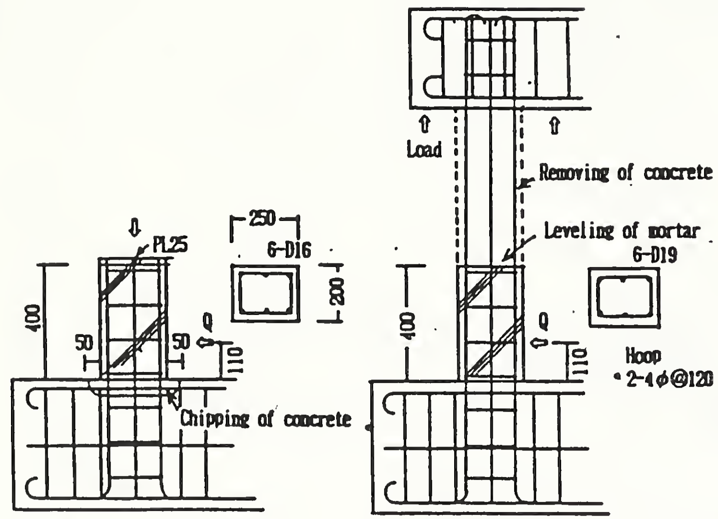


Fig. 10 Process to obtain the punching shear strength(Q)



a) Specimens in case of $\sigma_0 \geq 0$ b) Specimens in case of $\sigma_0 < 0$

Fig. 11 Shape and dimension of specimens

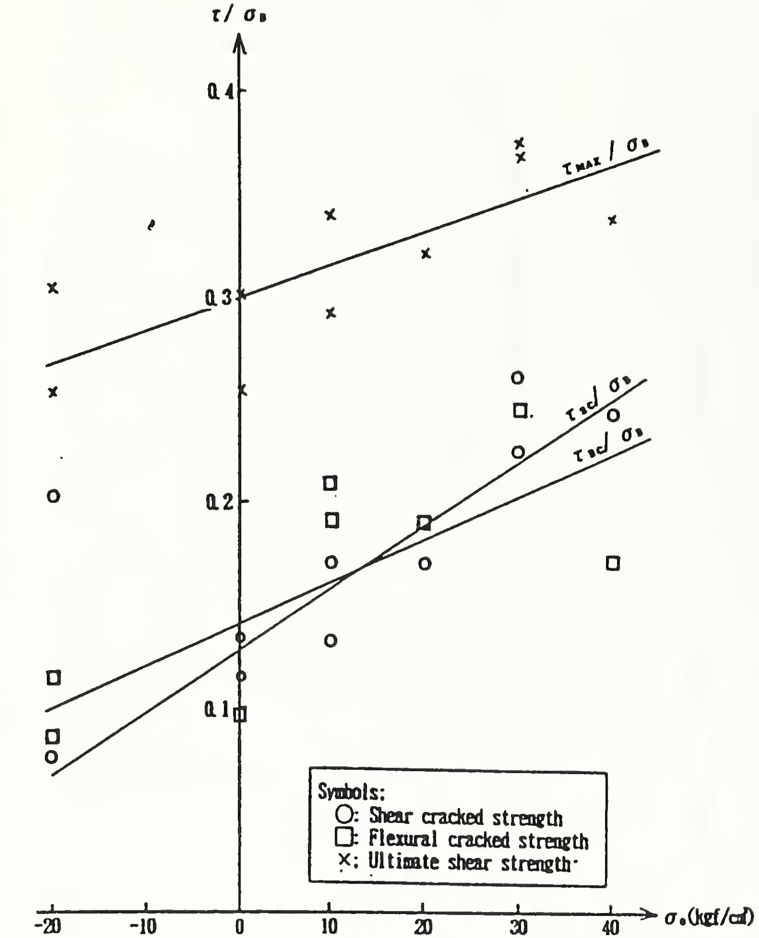


Fig. 12 Relation between shear stress and axial force

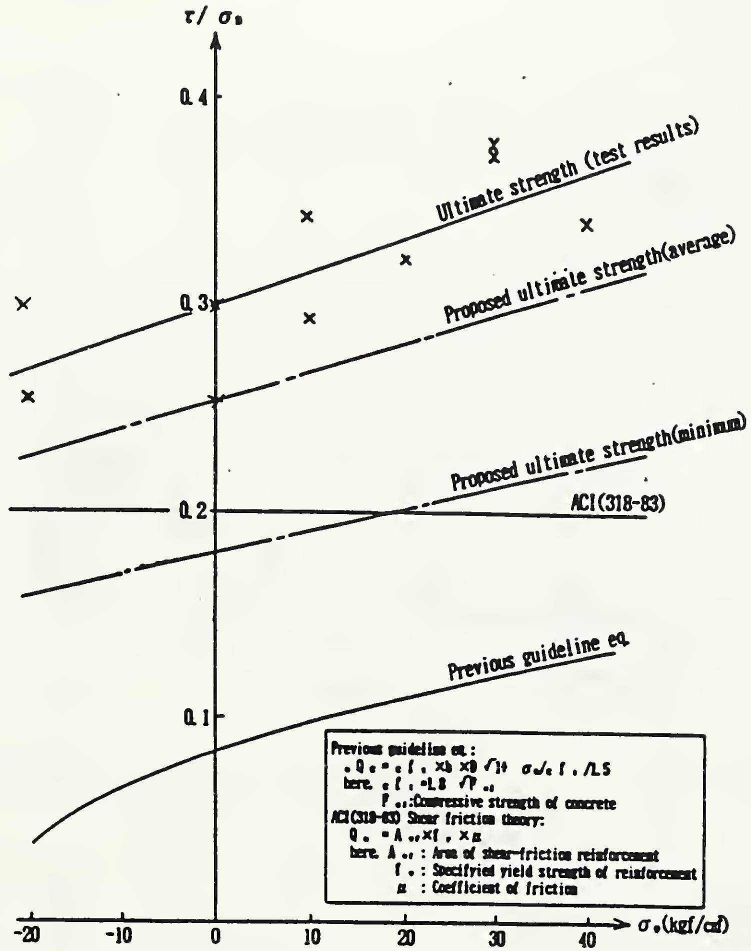


Fig. 13 Relation between shear stress and axial force

A Study on Shear Strength of Post-Installed Anchors

Tomoaki AKIYAMA[1)], Masaya HIROSAWA[2)], Yasushi SHIMIZU[3)], Taichi KATAGIRI[4)]

1. Outline

Recently, as seismic strengthening and retrofitting and repair work have become more prevalent, more post-installed anchors (hereinafter referred to as anchors) are being used in superstructures. Since anchors used for these purposes require a large bearing capacity, the trend is toward using larger size anchors than the ones which have been used for purposes other than seismic strengthening. Given the above circumstances, there have been many experiments conducted on anchors, and most of them have focused on the pull-out strength. As for shear tests, as indicated in Figure 1, they have used the method of the direct shear test, and one-way loading was applied to many of the anchors with single-type slender anchors of 19mm or less.

In terms of retrofit for the interface between the existing frame and the added member, conventionally, the post-cast concrete or mortar is bonded to the chipped existing concrete surface, and the shear strength of the anchor, especially slender ones, used for the interface depends heavily on the shear-friction effect of the concrete surface. This has been proved by a shear test[1)] on slender anchors where the maximum shear strength was recorded at the point of the maximum shear friction of the concrete surfaces. The load-deflection characteristics of the above test is illustrated conceptually in Figure 2-A , and the maximum strength is recorded at points with relatively small shear deflection and the deterioration after the maximum shear strength is very rapid. On the other hand, with large-size anchors, the shear friction does not have too much effect on the maximum shear strength, and so the maxmum shear strength is expected to be recorded at a point where shear deflection is large. Figure 2-B illustrates the conceptual load-deflection characteristics of the latter case.

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Here, we have conducted a unique direct shear test on large-size anchors (mainly D25, D22 chemical anchors) in order to verify the relationships among shear strength, deflection and failure mode, and to examine the effects of loading methods, anchor sizes, anchor embedment depths, and the number of anchors loaded simultaneously on these static characteristics.

The result of the above shear test was analyzed by the conventional shear capacity estimation formulas, and it was proved that the formulas needed some alterations according to the different ratio of the embedment depth to the anchor size. Here, some of the new shear capacity estimation formulas are suggested based on our experiments on 28 expanded metal anchors and 48 chemical anchors, as well as on the results from the previous experiments, and considering the limit state deflection for seismic strengthening.

2. Shear Test with Large-Size Anchors

2.1 Test Specimens

As indicated in Table 1, 45 specimens were tested, among which were chemical anchor specimens mainly made from epoxy acrylic resin (12 group-type specimens and 24 single-type specimens) and expanded metal anchor specimens with a head (9 single-type specimens). These anchors were bonded to the existing large concrete blocks illustrated in Figure 3, and examined. Figure 4 indicates different methods of fixing anchors on the existing large concrete blocks. The anchors were installed 175 mm inside the edge of the concrete blocks in order to simulate anchors installed in a beam of a building. As illustrated in Figure 5, the headed part to be simulating added member was surrounded by a steal plate and arranged split-proof bars (D6), and then non-shrink mortar was injected with pressure in the surrounded area. For the purpose of avoiding friction at the shear surface between the non-shrink mortar and the concrete blocks, fluoridized film was applied in two layers so the shear force is transmitted only to the anchors. Table 2 shows the properties of material of anchor re-bars, and Table 3 shows the compressive strength test result using a test piece of the non-shrink mortar and the concrete core(ϕ100) taken from the existing concrete blocks.

2.2 Experiment Procedures

A center-hole-type oil jack with the capacity of 70 tons was attached to a reaction steel frame and a loading beam were used for loading (Figure 6).

The loading was applied as the center axis of the oil jack passed through directly shear surface between the concrete block and the anchor, thus conveying only the shear force to the anchor.

Two types of loading, namely one-way loading and cyclic loading were applied. For the cyclic loading, five times load cycles were applied to the level of two thirds of $0.7 \cdot {}_s\sigma_y \cdot {}_{sc}a$ (${}_s\sigma_y$: the design yield point strength of the anchor re-bar based on the Building Standard Law in Japan, 3. 5 x 1.1 = 3.85 t/cm², ${}_{sc}a$: the sectional area of the shear of the anchor re-bar), and after that, the anchor specimen was pushed out toward the positive side. Vertical and horizontal deflections were measured at the time of loading, and as for the horizontal deflection, the measurement was conducted 2.5cm above the shear plane of the anchor at each stage of the loading as illustrated in Figure 5.

2.3 Test Results and Analyses

The maximum shear strength and other data are indicated in Table 4, and the typical load-displacement curves are shown in Figures 7 and 8. As indicated in these tables and figures, each specimen showed quite different failure mode and load-displacement characteristics. Especially, the horizontal displacements at the time of the maximum shear stregth were very big, and some reached 4cm or more. As illustrated in Figures 7 and 8, many of the specimens showed the result that the load was relatively small when the horizontal deflection was on the increase, and it gradually became bigger as the deflection was further increased. Table 4 shows the maximum shear strength and related shear strength when the horizontal deflection reached 0.75cm as the limit state deflection. The deflection of 0.75cm was calculated assuming that the anchor was used for the purpose of seismic strengthening of a building, and that the story drift angle was about 1/200 when the RC wall added to a 300cm-high building reached the maximum shear strength (300 1/200 1/2=0.75cm). The failure modes were classified according to the each failure state specified in Table 5. As a result, the typical failures modes were pulled out anchor bolts, shear failure in bolts, expantion failures, and split failures. As for the split failures among the failure modes in the tables, their shear strength is determined by the failure of the mortar, and so it does not reflect the shear strength of the anchor itself. Furthermore, the shear strength(τ_{max}) of each specimen which failed in each mode was; shear failure in bolts > expantion failures (expanded metal anchors) > pulled out anchor bolts (split failures).

Figure 9 illustrates the relationship between the diameter of the anchor and the average shear stress at the maximum strength(τ_{max}). Figure 10 shows the relationship between the failure mode and the τ_{max}. The following points were discussed using the above 2 Figures and so on.

(1) Shear Strength and Cyclic Loading

As illustrated in Figure 9, the τ_{max} in the cyclic loading was generally about 25% lower than that in the one-way loading in the case of D22 chemical anchors 5da(da=anchor diameter) embedment. For other types of specimens, cyclic loading had almost no effect on the τ_{max}. (See Figure 7)

(2) Shear Strength and Anchor Diameter

In the case of chemical anchors, among single-type specimens embedded 5da which tested under one-way loading, D22 showed a larger τ_{max} than the one of D25. On the other hand, specimens embedded 7.5da which tested under cyclic loading showed a little reverse result. Other specimens including group-type specimens did not indicate any influence of different diameters, D22 and D25, on the τ_{max}. In the case of expanded metal anchors, 19ϕ specimen showed the τ_{max} somewhat larger than the τ_{max} of 22ϕ and 25ϕ, but between 22ϕ and 25ϕ, the diameters did not seem to influence the τ_{max} so much.

(3) Shear Strength and Anchor Type

As indicated in Figure 9, chemical anchors (7.5da embedment) and expanded metal anchors (5da embedment) were compared, and the τ_{max} of the chemical anchors was, generally speaking, 1.6 to 1.8 times bigger than the one of the expanded metal anchors in the cases of both 22mm and 25mm diameters. As Figure 8, showing the typical load-displacement curves of each specimen, indicates, the chemical anchors recorded a bigger horizontal deflection at the time of failure than the expanded metal anchors. Thus, in the failure mode, the chemical anchors endured until the bolts shear off, while many of the expanded metal anchors had expantion failures before the bolts shear off.

On the other hand, in the case where all the specimens were embedded 5da, among 22mm-diameter specimens, chemical anchors and expanded metal anchors showed little difference in terms of the τ_{max}, while among 25mm diameter specimens, the τ_{max} of the expanded metal anchors was about 1.1 times bigger than the one of the chemical anchors. In the failure mode of this case, many of the expanded metal anchors suffered expansion failures, and the bolts were pulled out for all the chemical anchors. (See Figure 10)

(4) Number of Anchors Installed

In the case of 5da embedded chemical anchors, neither D22 nor D25 showed difference in the τ_{max} between single-type specimens and group-type specimens. The failure mode of all the specimens here were pulled out bolts. On the other hand, in the case of 7.5da embedded chemical anchors, both D22 and D25 single-type specimens showed the τ_{max} of about 5.2t/㎠ , while the group-type specimens showed the τ_{max} of about 2.6t/㎠ , and the difference was remarkable. This seems to be mainly caused by the difference in the failure modes, such as in the case of single-type specimens, anchor shear failures, and in the case of group-type specimens, split failures. (See Figure 10)

(5) Depth of Embedment

When test results were compared in terms of the depth of embedment, in the case of single-type specimens of D25 chemical anchors, the τ_{max} of the 7.5da embedded specimens was generally about 2.1 times larger than the τ_{max} of 5da embedded ones. Also in the case of D22, the τ_{max} of the 7.5da ones was 1.5 times and 2 times larger than the one of 5da embeded specimens, respectively in the cases of one-way loading and cyclic loading. Figure 11 illustrates the relationship between the maximum shear strength (τ_{max}) and the depth of embedment (1/da, 1:anchor embedment length), and this figure expressly indicates that the ratio between the anchor diameter and the depth of embedment greatly influence the value of the τ_{max}.

As for the failure mode, while all the 7.5da embeded specimens had bolts shear off, the 5da ones all suffered pulled out anchors. Thus, the failure mode difference between the cases of 7.5da and 5da embeded specimens proved influential to the value of the τ_{max}.(See Figure 10)

The relationships between the shear strength at the time of the limit state deflection (τ_a) and the diameter of anchors in terms of different anchor types and the embedment depth ratio (1/da) is shown in Figure 12. As illustrated there, the values of $\tau_a(=QAs/a_s)$ were compared when δ was about 0.75cm, in the case of anchors with the diameter of about 19-25mm, the ratio of anchors' depths of embedment to the diameter did not have very much influence on the shear stress; $\tau a = 1.8 \sim 2.2 t/cm^2$.

2.4 Analyses Using the Conventional Shear Strength Fomula

Test results were analysed based on the following formulas, which have been used for shear designing of anchors in Japan:

<u>Formula in Seismic Retrofiting Design</u>

Smaller of either

$$\left.\begin{array}{l} Qd1 = a_s \cdot \sigma_{max} / \sqrt{3} \quad \text{or} \\ Qd2 = 0.4 \cdot a_s \cdot \sqrt{\sigma_B \cdot Ec} \end{array}\right. \tag{1}$$

<u>Formula in Design Recommendation for Composit Constructions</u>

Mechanical Anchor Bolt

$$q_a = 0.75 \cdot \phi_{s3} \cdot (0.5 \cdot a_s \cdot \sqrt{\sigma_B \cdot Ec} \tag{2}$$

Reinforced Anchor Bolt (Chemical Anchor Bolt)

$$q_a = \phi_{s2} \cdot (0.7 \cdot {}_s\sigma_y \cdot a_s) \tag{3}$$

where

σ_{max} : Tensile strength of anchor bolts (kg/cm^2)

a_s : Sectional shear plane area of an anchor bolt(cm^2)

σ_B : Concrete compressive strength for the existing RC structure (kg/cm^2)

Ec : Concrete Young's Modules of the existing RC structure (kg/cm^2)

ϕ_{s2}, ϕ_{s3}: Reduction Coefficient for short-term loading $\phi_{s2}=1.0$, $\phi_{s3}=0.6$

${}_s\sigma_y$: Yield point strength of anchor bolts (kg/cm^2)

In Figure 13, the ordinate indicates the loads at the time of the limit state deflection (QAS) defined in 2.3 herein, and the abscissa indicates the sectional area of an anchor, and the data were plotted together with the above formulas for shear strength. As shown in the figure, as for Formulas (1) and (3), the QAS's of all the specimens were less than the calculated value, while QAS's of all the expanded metal anchors exceeded the calculated values by Formula (2), which is used for the designing of expanded metal anchors. As for the chemical anchors, all the D25 specimens embedded 7.5da and D22 specimens embedded 5da showed larger QAS's than the one according to Formula (2). On the other hand, the chemical anchors of D22 and 7.5da embedment showed rather unstable QAS's, and two specimens had QAS lower than the one according to Formula (2). Also, many of the group-type specimens showed lower values than the ones calculated by Formula (2). Figure 14 indicates the relationship between the different design formulas for shear strength and the maximum shear strength (Qmax). As shown in the figure, the Qmax's of all the chemical anchor specimens embedded 7.5da exceeded the one calculated using Formula (1). In addition, Qmax's of all the expanded metal anchor specimens as well as the ones mentioned above exceeded the values calculated by Formula (3). Furthermore, as for Formula (2), all the specimens showed a higher QAS than the calculated ones.

3. Various Factors Influencing Shear Strength

Among the conventional shear tests of anchors, the test data of direct shear tests of concrete-concrete (or mortar) interface indicated in Section2 and Figure 1e) were analysed in terms of the following conditions: in the case of expanded metal anchors and chemical anchors, the effective embedment length, le (1-da), was 4da (da:anchor diameter) or longer, and the edge distance(ls) was no less than 2.5da. The data satisfying the above conditions were 28 among 93 for expanded metal anchors and 48 for chemical anchors. In the case of chemical anchors, the conditions were le 6.5, and ls 2.5, and the data fulfilling them were 30 among the total of 48.

(1) Consideration of $\sqrt{\sigma_B \cdot E_c}$ in the Fisher's Formula[4)]

A study was carried out concerning $\sqrt{\sigma_B \cdot E_c}$ in the Fisher's Formula, on which shear strength formulas have been based in order to meet the standards in Japan. Figure 15 a) shows the relationship between $\sqrt{\sigma_B \cdot E_c}$ in the Fisher's Formula in the case of expanded metal anchors and the shear strength data obtained from the tests (Qm). As indicated in the figure, although the data are rather unstable, there certainly is a correlation between $\sqrt{\sigma_B \cdot E_c}$ and Qm, (correlation coefficient: $\rho=0.80$). Figure 15 b) shows the relationship between $\sqrt{\sigma_B \cdot E_c}$ and Qm in terms of chemical anchors. The figure indicates that the instability of data values is even greater than in the case of expanded metal anchors, and the correlation is not so strong, and the correlation coefficient was $\rho=0.57$. This seems to be because in the case of chemical ancors with le≧6.5da, the failure mode at the time of the ultimate shear strength is determined by the shear off of an anchor. Therefore, the relationship between the Qm other than in the case of chemical anchors with $le \geqq 6.5da$ and $\sqrt{\sigma_B \cdot E_c}$ was examined, and the result showed that the correlation coefficient was $\rho=0.71$, and relatively large. What this indicates is that when the maximum shear force is determined by the anchor shear off, Qm does not necessarily correlate with $\sqrt{\sigma_B \cdot E_c}$.

(2) Effect of Sectional Area of Anchor Bolt (a_s)

Figure 16 a) shows the relationship between the sectional area of anchors(a_s) and the value of the maximum shear strength obtained from the shear test (Qm) in terms of expanded metal anchors. Using the regression formula based on such data, the shear strength was calculated, and the anchors with the a_s of 2.8cm^2 and 4.91cm^2 showed values higher than the calculated ones, while anchors with 3.9cm^2 area often showed lower values than the calculated ones. Here, the correlation between a_s and Qm had a coefficient of $\rho=0.68$, which is relatively large. The anchor material used had the yield point strength, σ_y =5,000 ~ 6,000kg/cm^2 (σ_{max}=5,400 ~ 6,200kg/cm^2). Figure 16 b) shows the relationship between the a_s and Qm concerning chemical anchors. In this relationship, the correlation coefficient, ρ, was 0.87, and considerably higher than that of the expanded metal anchors. The reason for this is considered that the shear strength is determined by the shear off in anchors or states close to that.

(3) Influence of Effective Embedment Lengths(le)

Figure 17 shows the relationship between the effective embedment lengths(le) and the values of the maximum shear strength obtained from the shear test(Qm) in terms of expanded metal anchors. The effective embedment lengths were 2.9cm to 10cm, and many had the embedment length of 5.2cm. As a result of the regression analysis, there was a strong correlation between the values of le and Qm. Figure 17 b) indicates the le-Qm relationship in the case of chemical anchors. Compared with the ones of le=about 7.0cm, although chemical anchors with le=14.3~16.3cm showed more fluctuation in terms of Qm, the correlatioin is still strong with the coefficient of $\rho=0.85$, as in the case of a_s. In addition, among specimens with le of around 7cm and 14.3-16.3cm, there are ones whose shear strength does not depend upon le but is determined by the failure of mortar (group-type specimen, $le \geqq 6.5da$ embedded), and so the relationship between Qm and le was examined excluding such data. As a result, the correlatioin is strong with the coefficient of

ρ=0.94 , as in the case of expanded metal anchors. However, le often is proportional to the anchor diameter(da), and the correlation can be interpreted as based on the anchor diameter or the sectional area of the anchor.

4. Proposal for Shear Strength Formula

Considering the values obtained through the existing shear strength formulas and the data of the test on large-size anchors mentioned previously, the factors, such as the sectional area of anchors(a_s), and the effective embedment lengths(le) influence the shear strength. The following shear strength formulas for seismic strengthening anchors used for braces and retrofit walls were suggested taking into account the above facts, the deflection mode, and the ratio of the effective embedment length to the anchor diameter.

Expanded Metal Anchors when $4da \leqq le < 7da$

Smaller of either

$$Q_{a1}=0.7 \cdot {}_m\sigma_y \cdot {}_sa_e \quad \text{or}$$
$$Q_{a2}=0.3 \cdot \sqrt{\sigma_B \cdot E_c} \cdot {}_sa_e \qquad (4)$$

where, $\tau(Q_a/{}_sa_e)$ is 2500kg/cm² or below

Chemical Anchors and Expanded Metal Anchors when $7da \leqq le$

Smaller of either

$$Q_{a1}=0.7 \cdot {}_m\sigma_y \cdot {}_sa_e \quad \text{or}$$
$$Q_{a2}=0.4 \cdot \sqrt{\sigma_B \cdot E_c} \cdot {}_sa_e \qquad (5)$$

where, $\tau(Q_a/{}_sa_e)$ is 3000kg/cm² or below

${}_m\sigma_y$: Yield point strength of anchors (kg/cm²)

${}_sa_e$: anchor's sectional area at the shear plane (cm²)

E_c : Young's modules of the existing concretes (kg/cm²)

σ_B : compressive strength of the existing concretes(kg/cm²)

Due to the difference in the anchor's resistance mechanism against pull-outs, (expanded metal anchors: wedge effect at the extended part, chemical anchors: adhesiveness around the anchor itself), recently in Japan, expanded metal anchors with the embedment length of 5da are used, and chemical anchors 8da. As described in Section 2, considering the fact that the bearing capacity is greately influenced by the ratio of the embedment length to the anchor diameter, the separate shear strength formulas, (4) and (5), were suggersted according to the difference in the embedment lengths.

Among the above formulas, the Qa1 formula is based on the yield point strength of the steel while the Qd1 formula in Formula (1) was based on the tensile strength of the anchor steel. On the other hand, when the horizontal deflections were taken into account, as specified in Section 2, the shear deflections(δ), especially in the case of large-size anchors, were as long as 4 to 5 cm at the time the anchors reached the ultimate shear force. On the other hand, in the case of infilled walls, there is a report that the shear gap between the existing beam and the infilled wall is about 2cm at the time of the maximum shear force. Therefore, taking the above information in consideration, some limitations were applied to the shear strength formulas; when $4da \leqq le < 7da$(expanded metal anchor), $\tau \leqq 2500kg/cm^2$, and when $7da \leqq le$(chemical anchor and expanded metal anchor), $\tau \leqq 3000kg/cm^2$. (See Figure 19)

As described in Section 3, the embedment lengths have a considerable influence upon the shear strength, and the shear strength is proportional to the effective embedment length until the latter reaches a certain length. Also as Section 2 indicates, the experiment using adhesive anchors with le=6.5da did not have any pulled out anchors, and the shear force was increased. Taking the above into consideration, Formula (5) was suggested for expanded metal anchors when $le \geqq 7.0da$. In addition, in the case of expanded metal anchors, since their l is usually set for 5da, shear tests are mostly conducted with le=4da, and there is very limited data with regard to the cases where $4da < le < 7da$. Therefore, here, Formula (4) was suggested for the cases where $4da \leqq le < 7da$, regarding these effective embedment lengths as safety.

Figure 18 a) shows the relationship between the values based on Formula (4) and the values of the maximum shear strength derived from the shear test(Qm). Figure 18 b) shows the relationship between the values based on Formula (5) and the values of the ultimate shear strength derived from the shear test(Qm). As indicated in these figures, the test data were estimated 95% safer than the values calculated by Formulas (4) and (5).

5. Summary

(1) As for large-size anchors, when the shear test was conducted with variable factors of anchor diameters, loading methods, and the number of anchors, the following was found:

1) The loads and deflection showed a relatively small elastic limit, and at the time of the maximum shear force, the horizontal deflections were 15～60mm.

2) Among the variable factors, the most influential factor on the maximum average shear stress(τmax) was the length of embedment, and the other factors did not have so much influence. In other words, as the anchor embedment length increases, generally the failure mode changes from pulled-out bolts - expansion failures (in the case of expanded metal anchors) - to shear out in bolts, but the elastic limit strength is not changed so much.

a) As for D22 chemical anchor specimens embedded 5da, the shear strength deteriorates as they go through repeated loading, but other types of specimens show almost no difference in terms of shear strength deterioration caused by different loading methods.

b) In the case of chemical anchors, the maximum average shear stress (τ max) is not so much influenced by the diameter difference as small as D22 and D25.

c) As for the τmax's of chemical anchors and expanded metal anchors both embedded 5da, when the diameter is 25mm, expanded metal anchors show numerical values generally about 1.1 times bigger than the one of chemical anchors, but in the case of 22mm diameter, they show almost no difference.

d) When the embedment length of the chemical anchors is 5da, there is almost no difference between the τmax's of single-type and group-type specimens.

e) In the case of single-type specimens, the influence of the embedment length on the τmax is especially large.

(2) With regard to anchor designing, when the embedment length is secured at or above 7.5da at the time of the maximum shear strength, the failure mode will be shear off in bolts, and any estimation formulas will be able to estimate safe values. However, since the shear slip deflections at the time of the maximum shear strength are very large, the shear strength must be

estimated taking this fact into account. This is why for certain maximum deflection, QAS, Formula (2) gives the average values, while relatively many test data on the group-type specimens are lower than them.

(3) Based on the shear test result summarized in the above (1), after studying the conventional bearing capacity estimation formulas as well as several factors influencing the shear strength, and also considering the anchor's shear deflection and the effective embedment lengths, new shear strength formulas, (4) and (5) were suggested. The previous test data were analyzed using these formulas and it was verified to almost 95% estimate safe values. Furthermore, with regard to the newly estimated shear strength using the new formulas, the horizontal deflections are 1.0～1.5cm, and kept below 2.0cm.

References

1) Shimizu, Y., Endo, T. : Shear Strength of Post-Installed Anchors, Technical Report Collection of KANTO Branch of AIJ, 1986.

2) The Japan Building Disaster Prevention Association : Guideline for Repair and Retrofit Design of Existing Reinforced Concrete Buildings, Mar., 1977.

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4) Fisher, J.W., et al. : Shear Strength of Stud Connectors in Light-Weight and Normal-Weight Concrete, AISC, Engineering Journal, April, 1971.

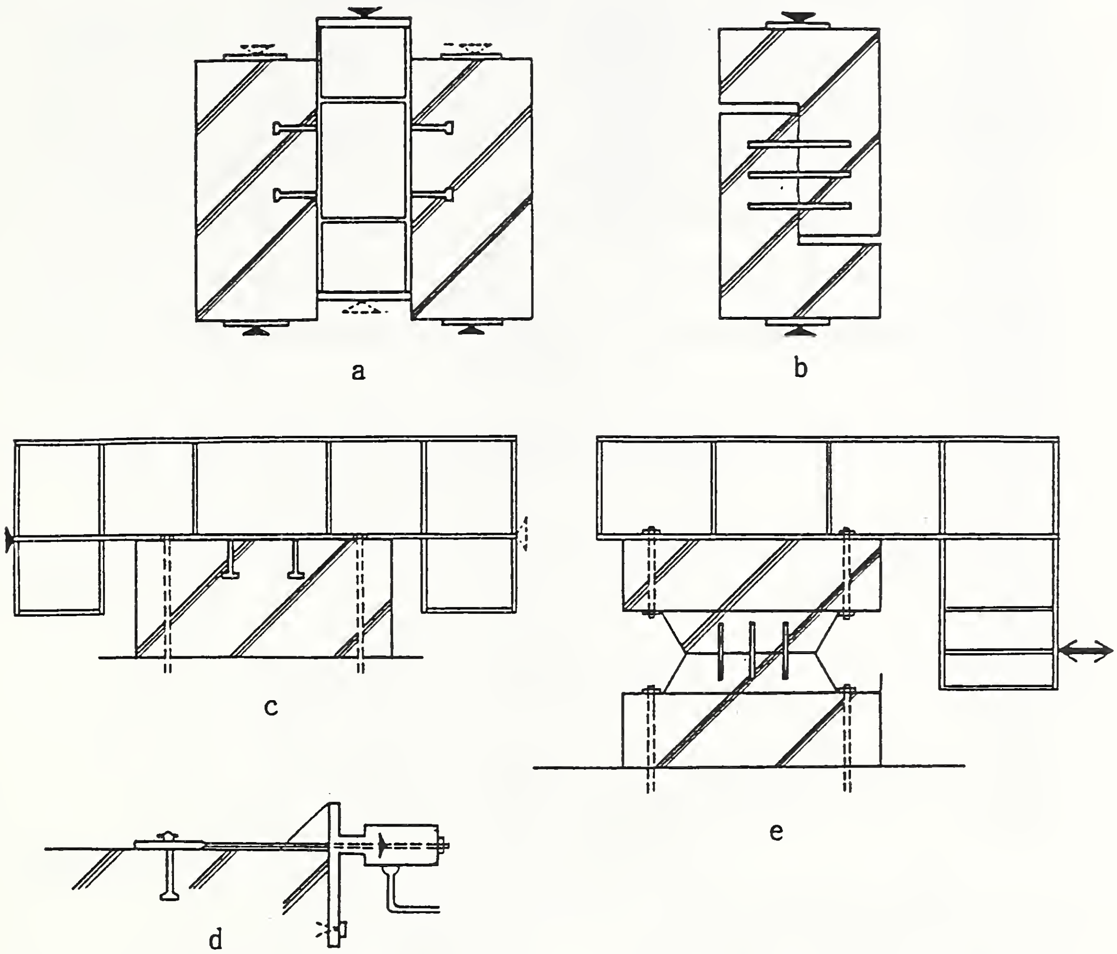


Fig. 1 Method of direct shear tests

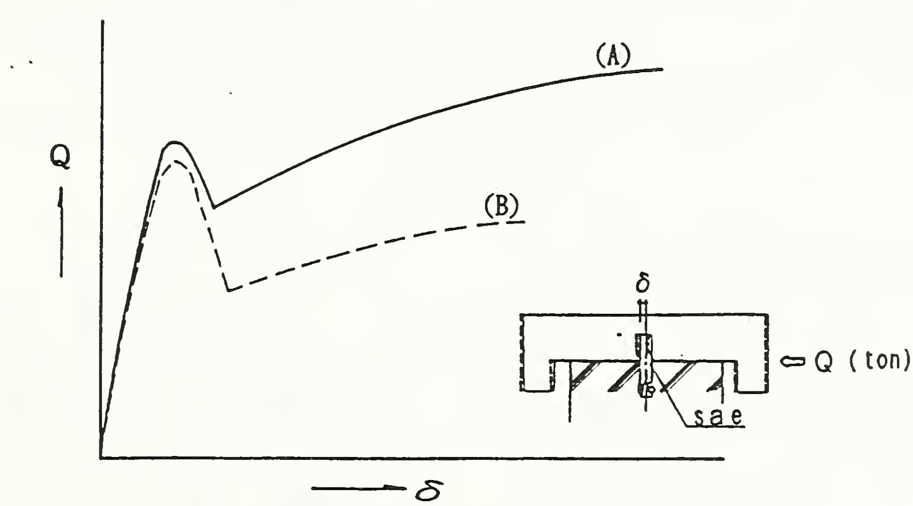
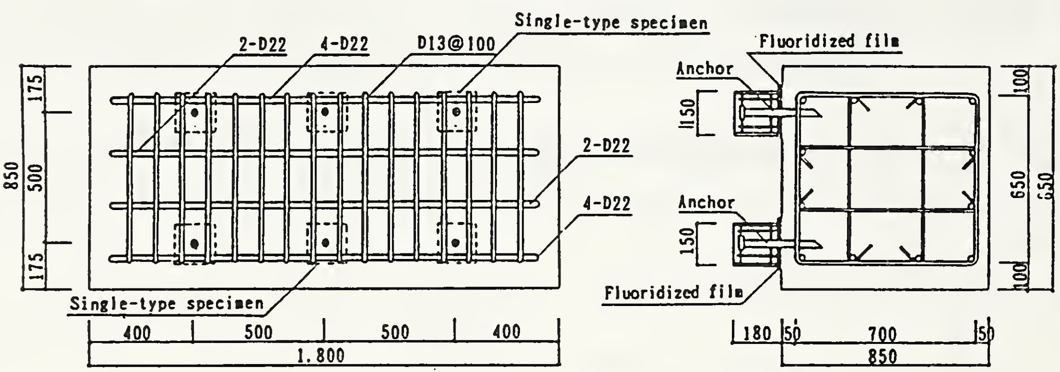


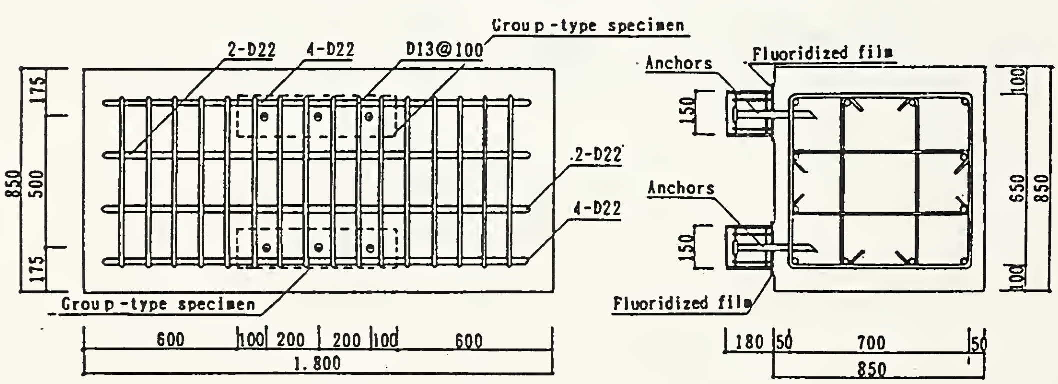
Fig. 2 Concept of load-deflection characterisfics

Table1 A list of test anchors

Specimen	Number of anchor	Diameter (mm)	Embedment depth	Loading	Kind of anchor
1A-1	3	25	7.5da	Cyclic	Chemical
1A-2			5da		
1B-1		22	7.5da		
1B-2			5da		
2A-1	3	25	7.5da	Cyclic	Chemical
2A-2			5da		
2B-1		22	7.5da		
2B-2			5da		
3A-1	3	25	7.5da	Cyclic	Chemical
3A-2			5da		
3B-1		22	7.5da		
3B-2			5da		
4A-1	1	25	7.5da	One way	Chemical
4A-2				Cyclic	
4A-3				One way	
4A-4			5da		
4A-5				Cyclic	
4A-6				One way	
4B-1	1	22	7.5da	One way	Chemical
4B-2				Cyclic	
4B-3				One way	
4B-4			5da		
4B-5				Cyclic	
4B-6				One way	
5A-1	1	25	7.5da	One way	Chemical
5A-2				Cyclic	
5A-3			5da	One way	Metal
5A-4					Chemical
5A-5				Cyclic	
5A-6				One way	Metal
5B-1	1	22	5da	One way	Metal
5B-2			7.5da	Cyclic	Chemical
5B-3				One way	
5B-4			5da		Metal
5B-5				Cyclic	Chemical
5B-6				One way	
6A-1	1	25	5da	One way	Metal
6A-2			7.5da	Cyclic	Chemical
6A-3		19	5da	One way	Metal
6A-4					
6A-5		25		Cyclic	Chemical
6A-6		19		One way	Metal
6B-1	1	22	5da	One way	Metal
6B-2			7.5da	Cyclic	Chemical
6B-5			5da		



Concrete block for single-tipe specimen



Concrete block for group-tipe specimen

unit:mm

Fig. 3 Test specimen details

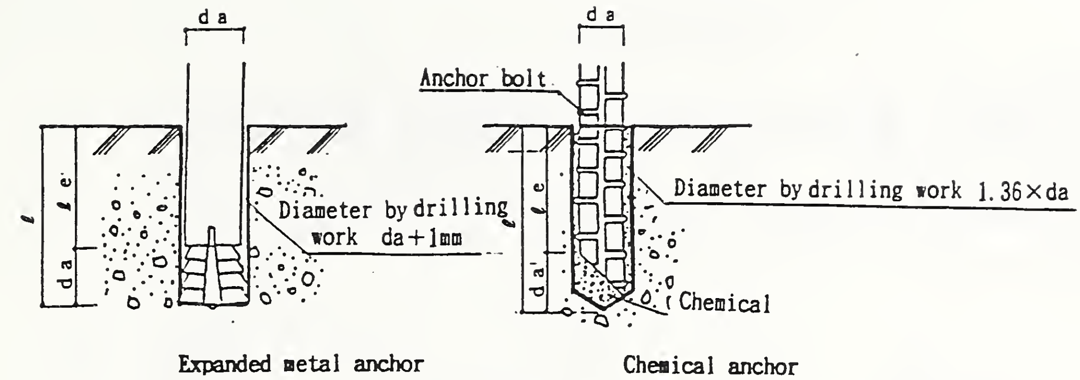


Fig. 4 Embedment method of expanded metal anchor and chemical anchor

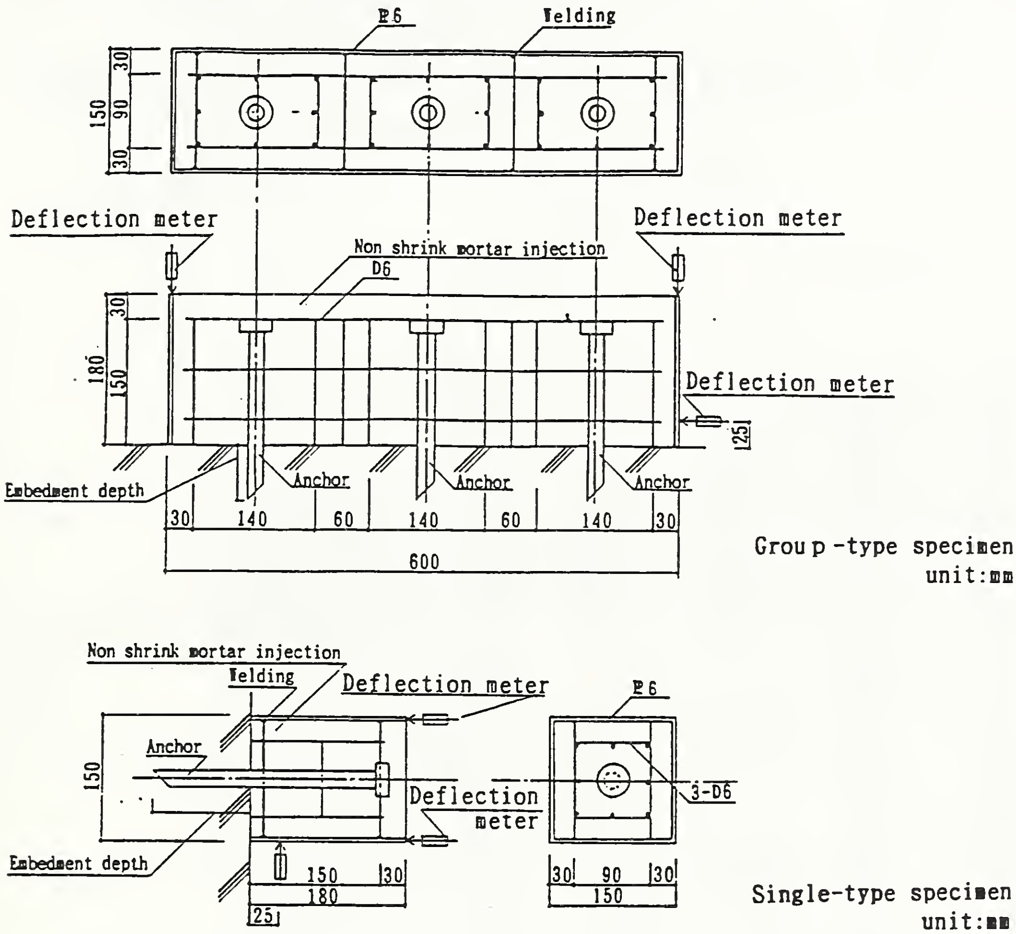


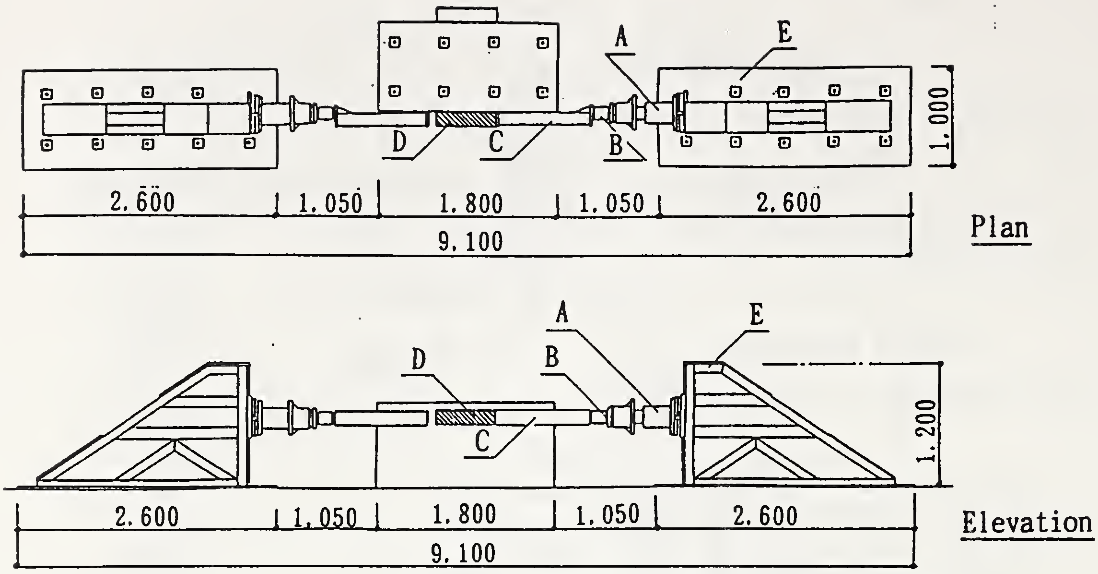
Fig. 5 Test anchor details

Table 2 The properties of material of anchor re-bars

Rein-forcement	Nat	Sectional area (cm^2)	Yield strength (Kg/cm^2)	Tensile strength (Kg/cm^2)
D22	M22	3.87	3695	5613
D25	M24	5.07	3769	5693

Table 3 The properties of material of concrete and mortar

Specimen	Unit weight (t/m^3)	Compressive strength (Kgf/cm^2)	Young's modulus ($\times 10^5$ Kgf/cm^2)
Existing concrete	2.31	337	2.11
Non-shrink mortar	2.12	380	2.52



A:Oil jack (Center hole type) D:Specimen
B:Load cell E:Reaction frame
C:Loading beam

unit:mm

Fig. 6 Loading apparatus for direct shear test

Table 4 Test results

Specimen	Qmax(ton)	δmax (mm)	Q_{AS}(ton)	Q_{AS}/Qmax	Failure mode
1A-1	38.60	—	26.0	0.67	(Split failure)
1A-2	35.37	15.74	29.6	0.84	
1B-1	33.76	19.48	21.0	0.62	
1B-2	27.30	26.69	21.0	0.77	Pull-out of anchor
2A-1	39.69	57.34	27.0	0.68	(Split failure)
2A-2	29.63	36.21	27.0	0.91	Pull-out of anchor
2B-1	25.20	30.52	21.0	0.83	(Split failure)
2B-2	34.25	30.60	21.0	0.61	Pull-out of anchor
3A-1	37.77	17.11	28.4	0.75	(Split failure)
3A-2	39.99	27.03	29.6	0.74	Pull-out of anchor
3B-1	42.44	28.32	20.8	0.49	(Split failure)
3B-2	22.38	16.58	21.0	0.94	Pull-out of anchor
4A-1	26.43	—	10.2	0.39	Shear off in anchor
4A-2	25.92	—	9.0	0.35	
4A-3	27.13	—	10.6	0.39	
4A-4	12.23	20.07	9.3	0.76	Pull-out of anchor
4A-5	12.06	23.53	9.0	0.75	
4A-6	12.20	15.56	11.2	0.92	
4B-1	22.05	—	6.4	0.29	Shear off in anchor
4B-2	20.91	—	7.0	0.33	
4B-3	20.58	—	9.1	0.44	
4B-4	12.60	26.11	8.7	0.69	Pull-out of anchor
4B-5	10.99	25.42	7.4	0.67	
4B-6	12.60	22.13	8.8	0.70	
5A-1	19.00	34.60	11.3	0.59	Shear off in anchor
5A-2	26.67	50.60	11.7	0.44	
5A-3	16.15	39.79	11.3	0.70	Expantion failure
5A-4	13.60	—	9.5	0.70	Pull-out of anchor
5A-5	12.65	40.71	8.2	0.65	
5A-6	15.20	—	10.2	0.67	
5B-1	11.9	41.15	7.2	0.60	Expantion failure
5B-2	17.03	45.59	7.0	0.41	Shear off in anchor
5B-3	18.60	—	4.9	0.26	
5B-4	13.46	25.47	7.6	0.56	Expantion failure
5B-5	10.20	41.11	6.2	0.61	Pull-out of anchor
5B-6	11.78	20.57	7.8	0.66	
6A-1	14.79	—	9.6	0.65	Expantion failure
6A-2	29.60	—	10.8	0.36	Shear off in anchor
6A-3	11.20	49.65	6.2	0.55	Expantion failure
6A-4	9.96	—	6.2	0.62	Pull-out of anchor
6A-5	13.00	39.26	9.0	0.69	
6A-6	9.26	47.02	6.0	0.65	
6B-1	11.97	50.58	7.4	0.62	Expantion failure
6B-2	20.28	61.73	8.4	0.41	Shear off in anchor
6B-5	9.96	—	7.0	0.70	Pull-out of anchor

Note : Split failure does not indicate the shear shrength of anchor.

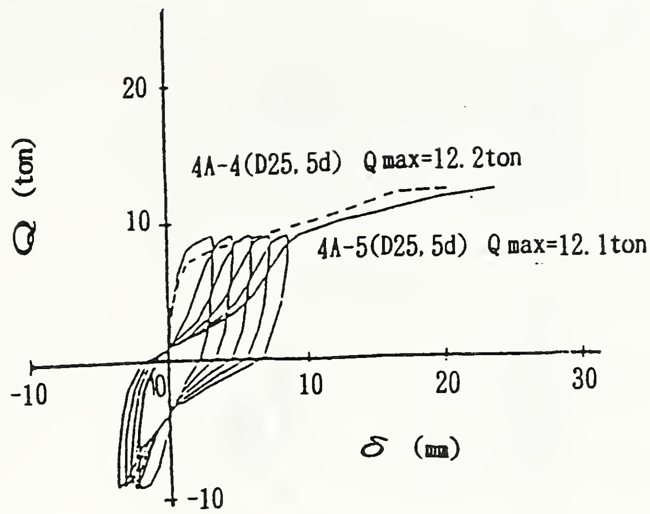


Fig. 7 Load-deflection curves(effect of one-way loading and cyclic loading)

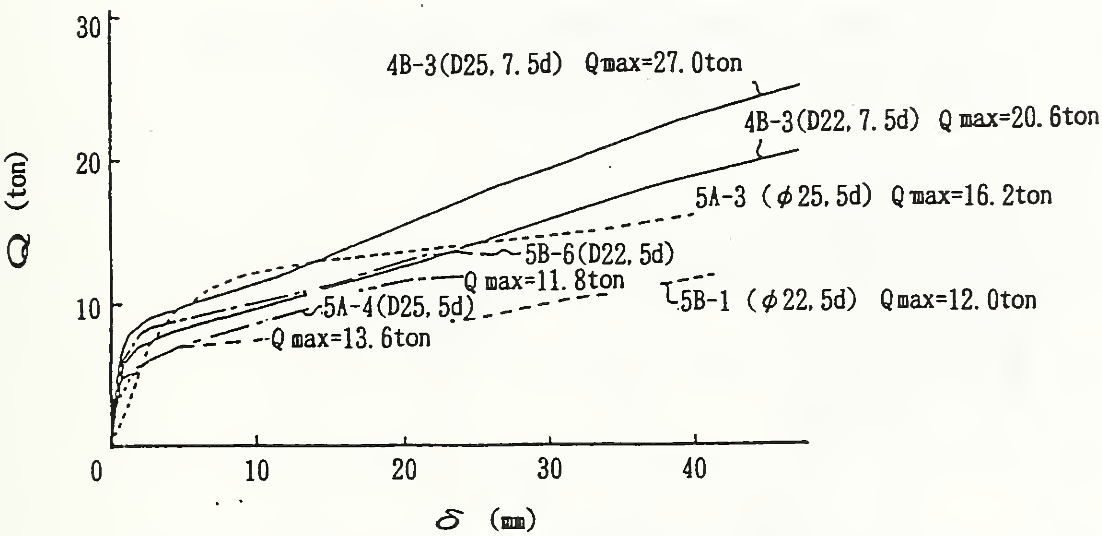


Fig. 8 Load-deflection curves for typical specimens

Table 5 State of failure modes

Failure mode	Failure state	Single-type chemical anchor		Group-type chemical anchor		Expanded metal anchor
		5d	7.5d	5d	7.5d	
Split failure	Non shurink mortal are cracked along the split-proof bar.	—	—	1	6	—
Shear-off in bolt	Anchor are out off due to shearing force.	1	11	—	—	—
Pull-out anchor bolt	Anchor are pulled out from concrete block.	11	—	5	—	3
Expansion failure	Expansive part in metal anchor are broken, so anchor are pulled out.	—	—	—	—	6

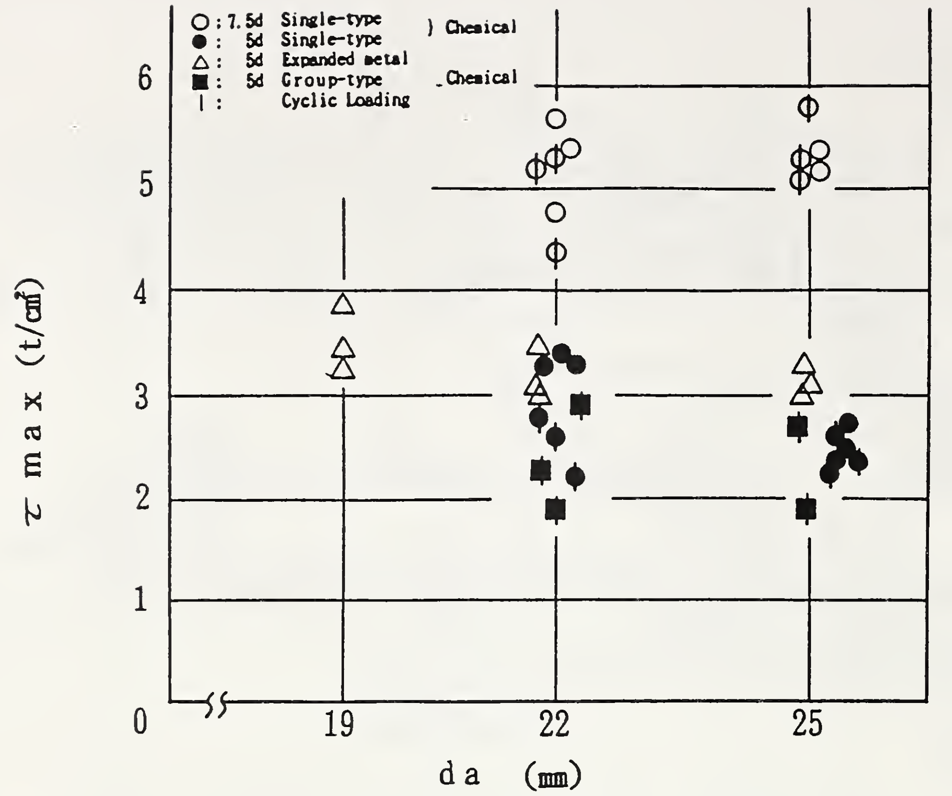


Fig. 9 Relationship between diameter of anchor and average shear stress at maximum strength (τ max)

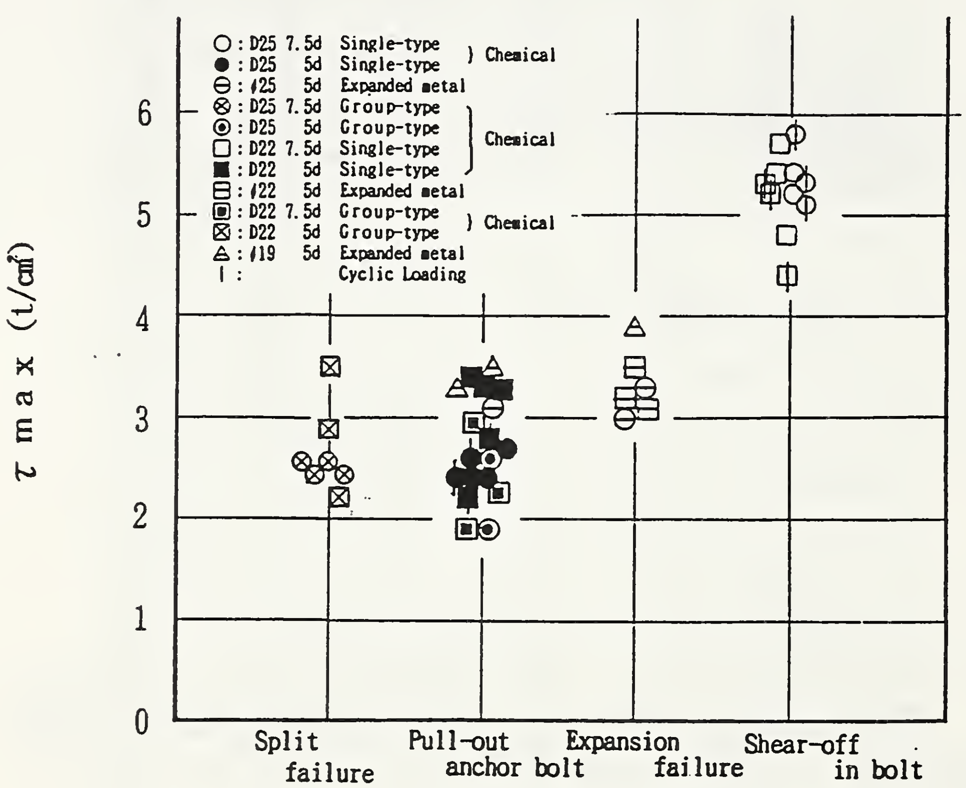


Fig. 10 Relationship between failure mode and average shear stress at maximum strength (τ max)

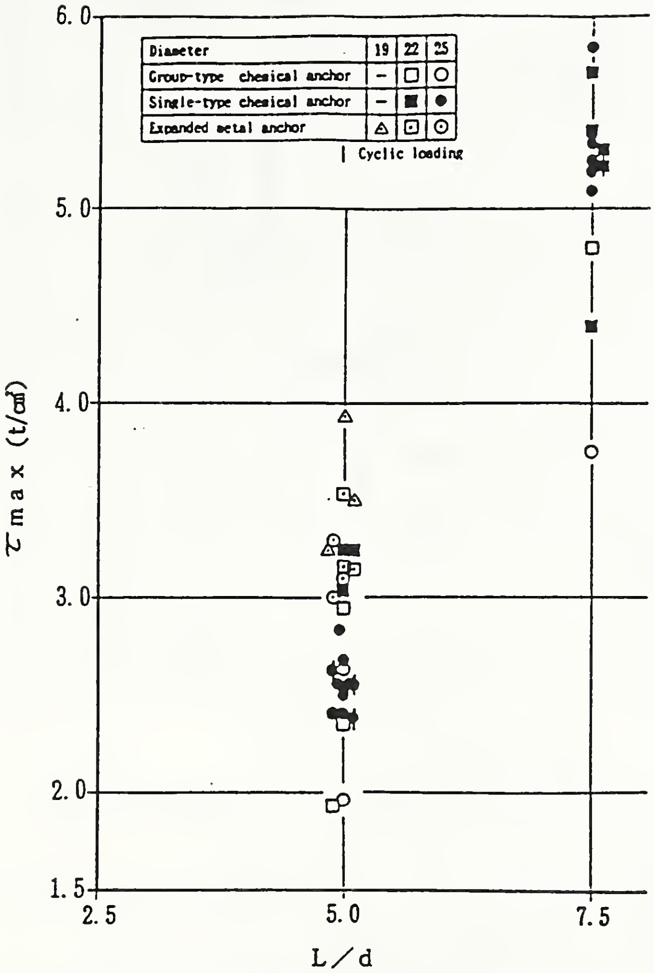


Fig. 11 Relationship between maximum shear strength (τ max) and depth of embedment (ℓ /da)

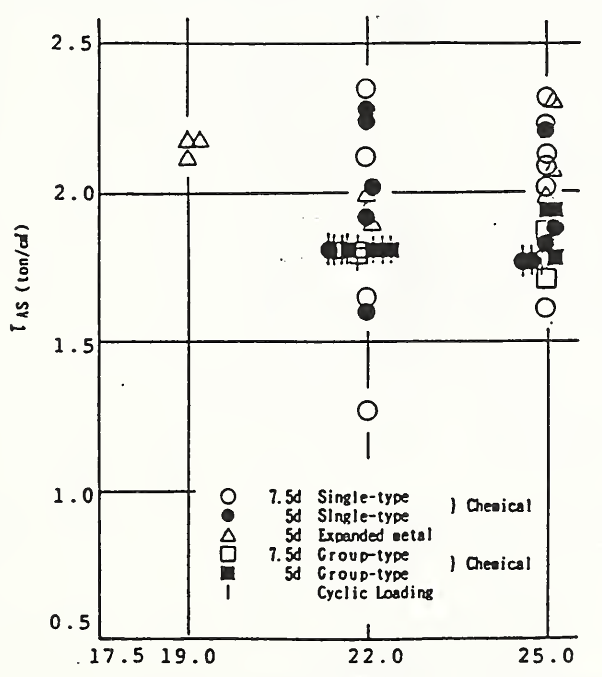


Fig. 12 τ_{AS} at limit state deflection versus anchor diameter(da)

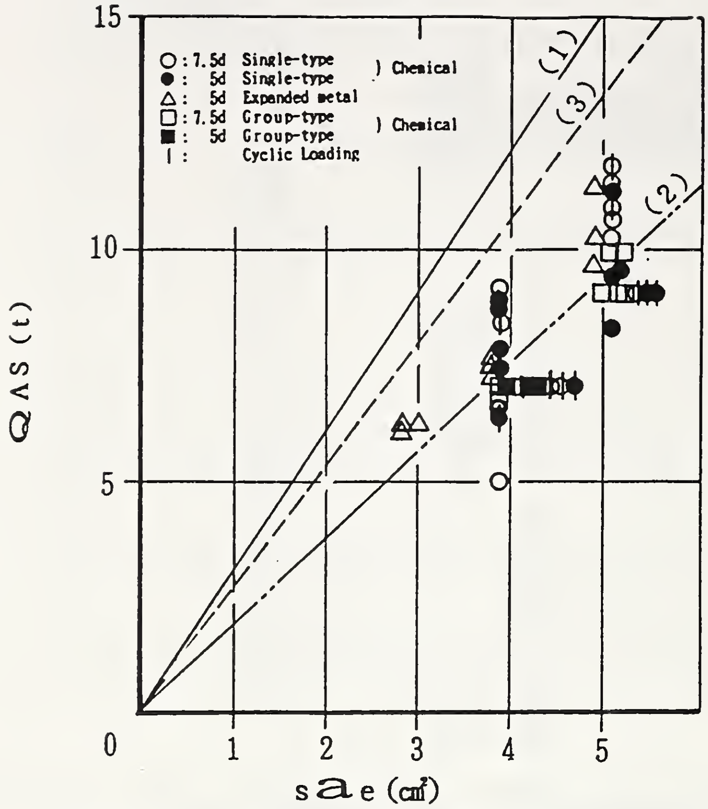


Fig. 13 Each designing formulas versus shear strength (Q_{AS}) at limit state deflection

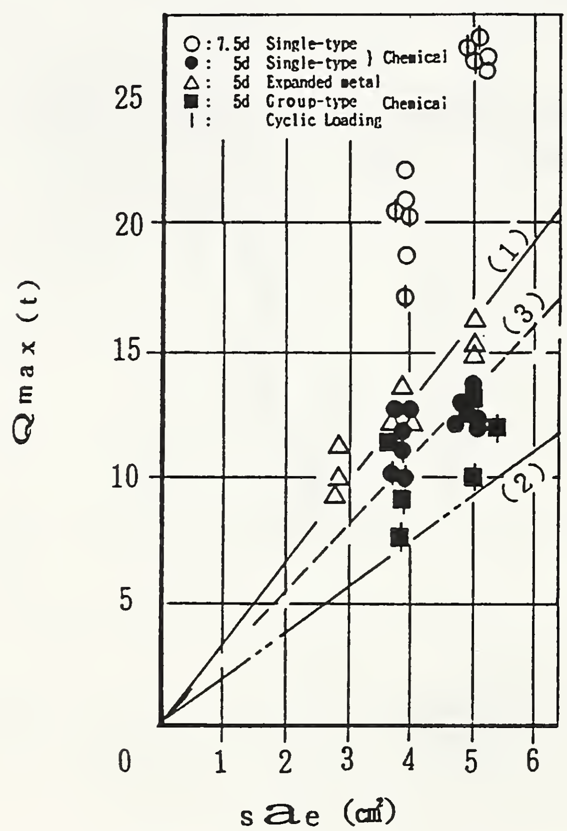
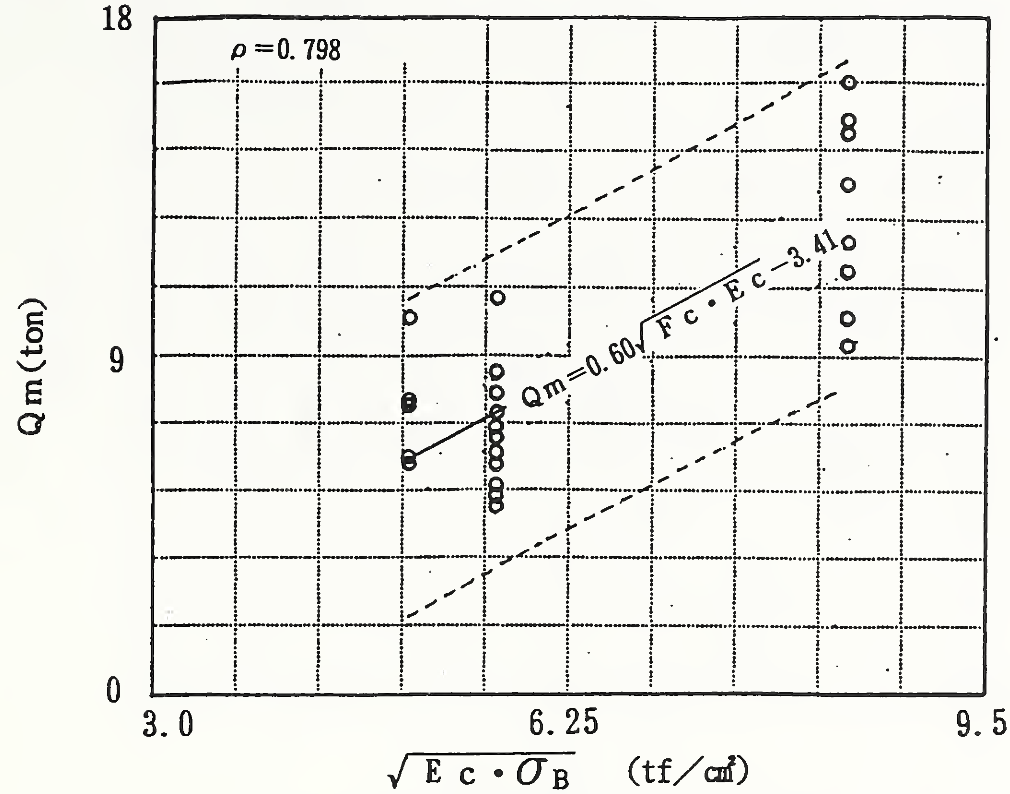
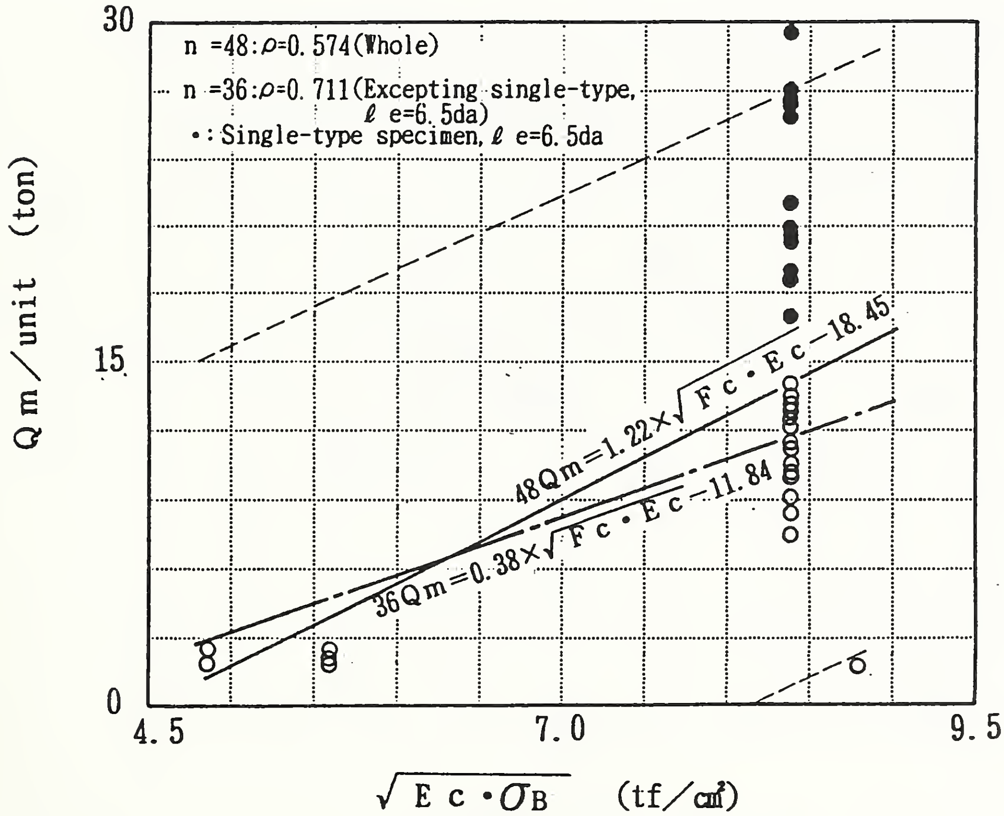


Fig. 14 Each designing formulas versus maximum shear strength (Q_{max})

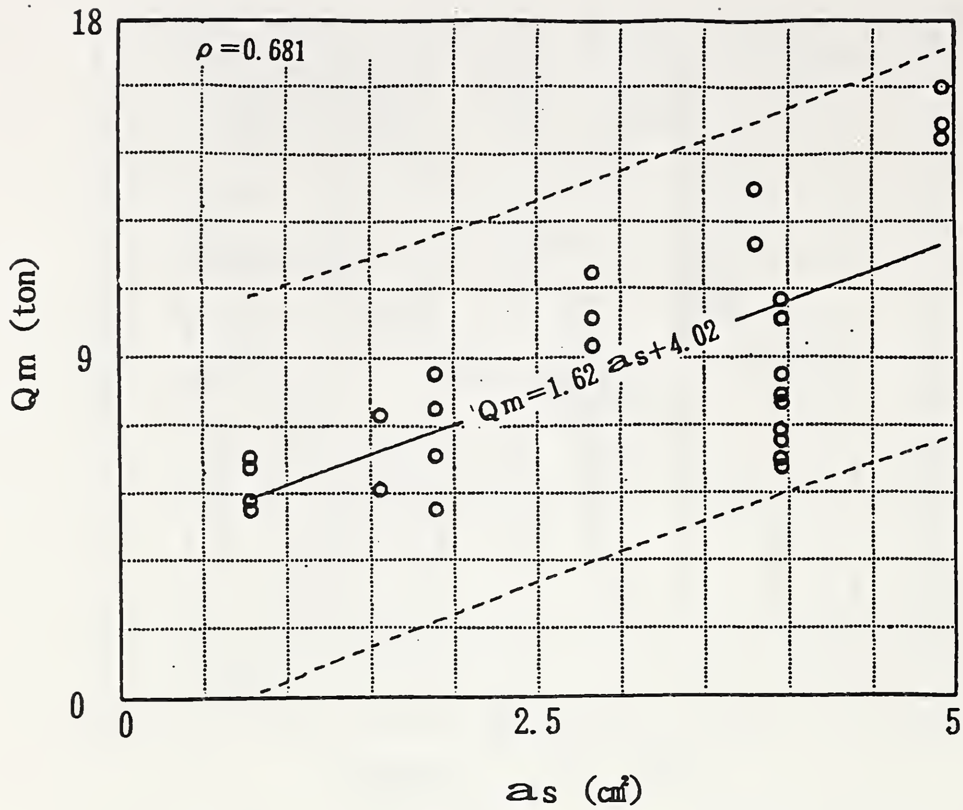


a) Expanded metal anchor

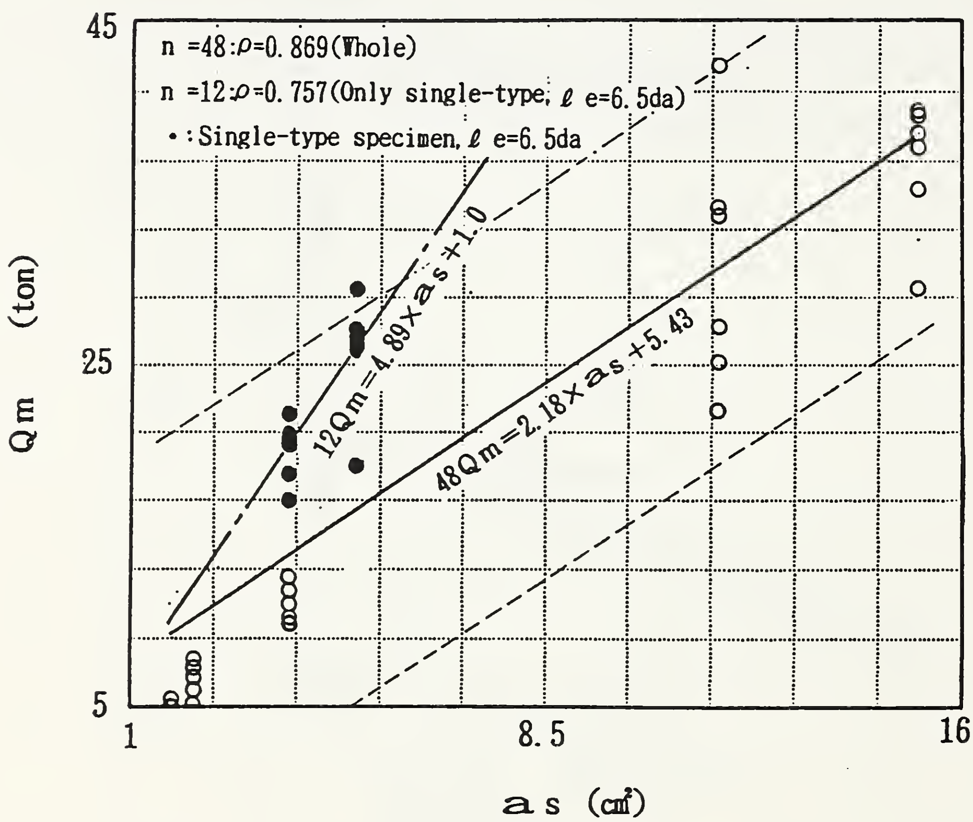


b) Chemical anchor

Fig. 15 $\sqrt{Ec \cdot \sigma_B}$ in the Fisher ' s Formula versus values of maximum shear strength on test (Qm)

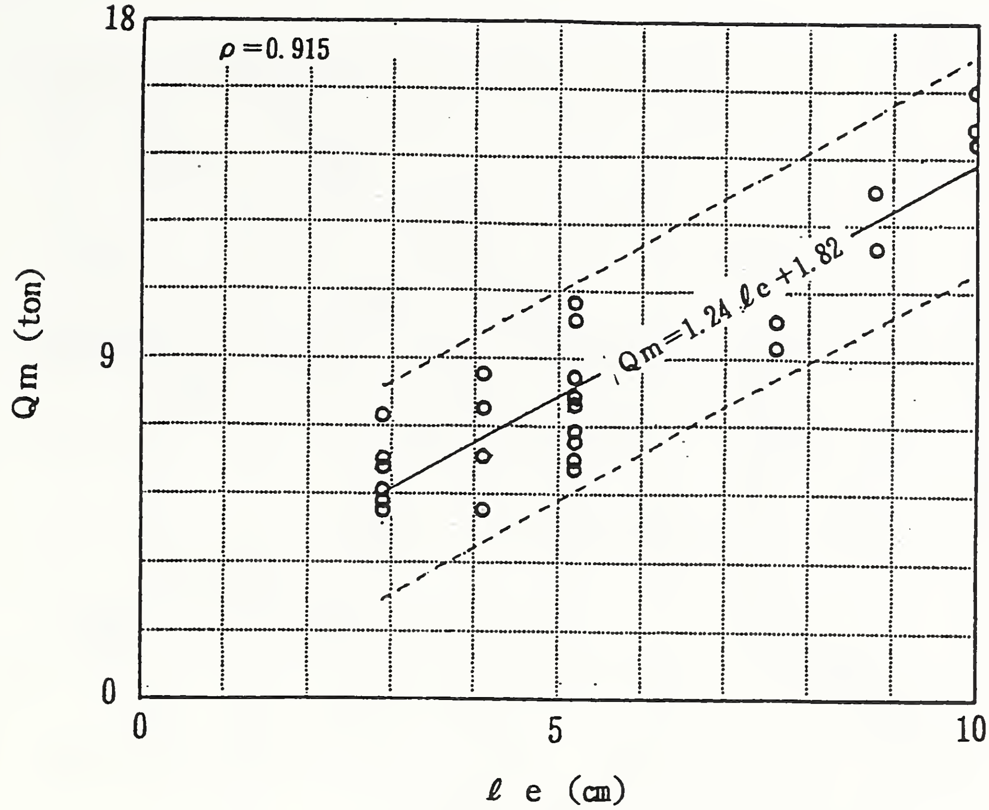


a) Expanded metal anchor

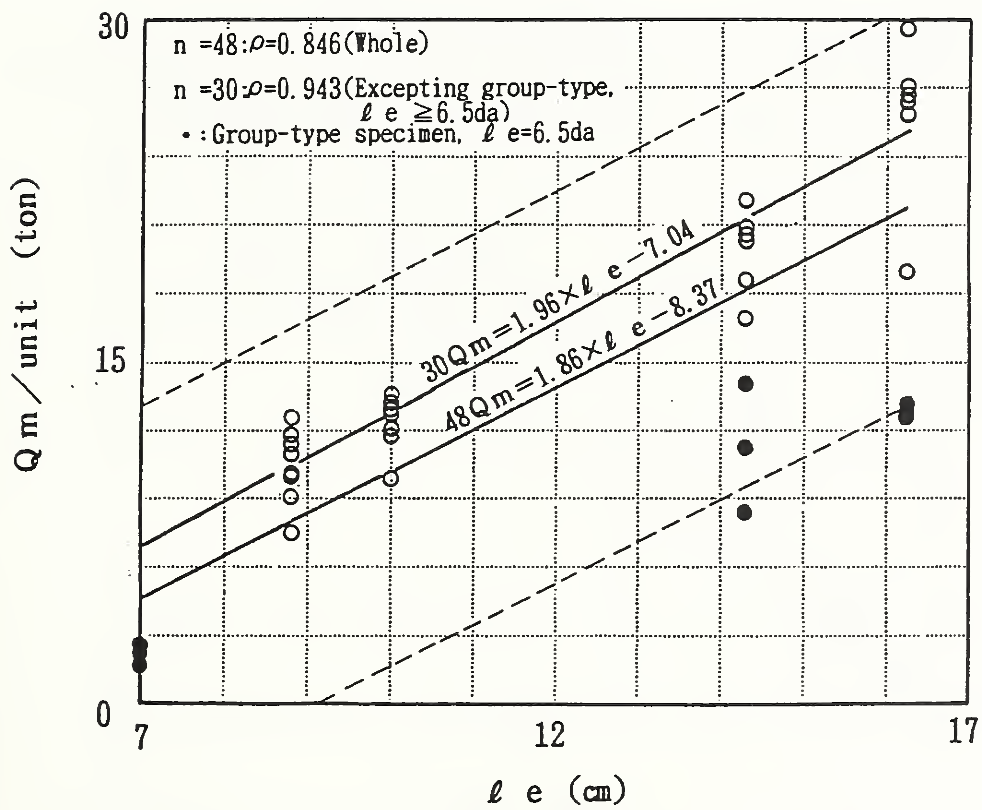


b) Chemical anchor

Fig. 16 Sectional area of anchor re-bars (a s) versus values of maximum shear strength on test (Q m)

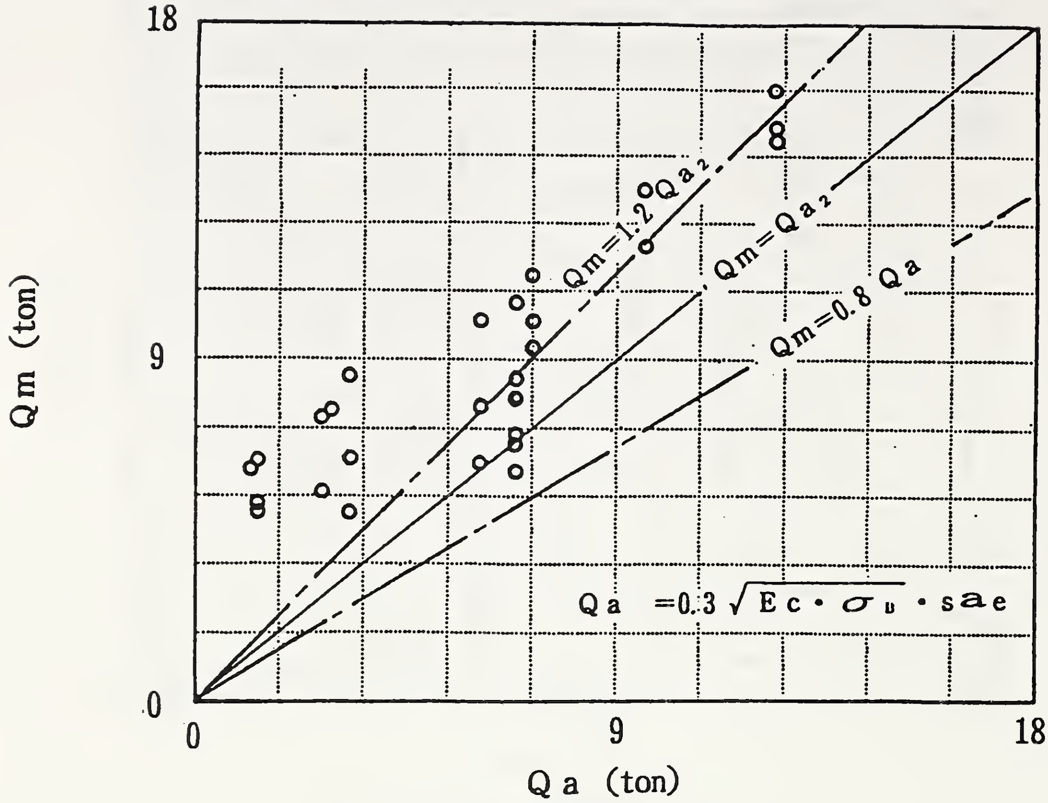


a) Expanded metal anchor

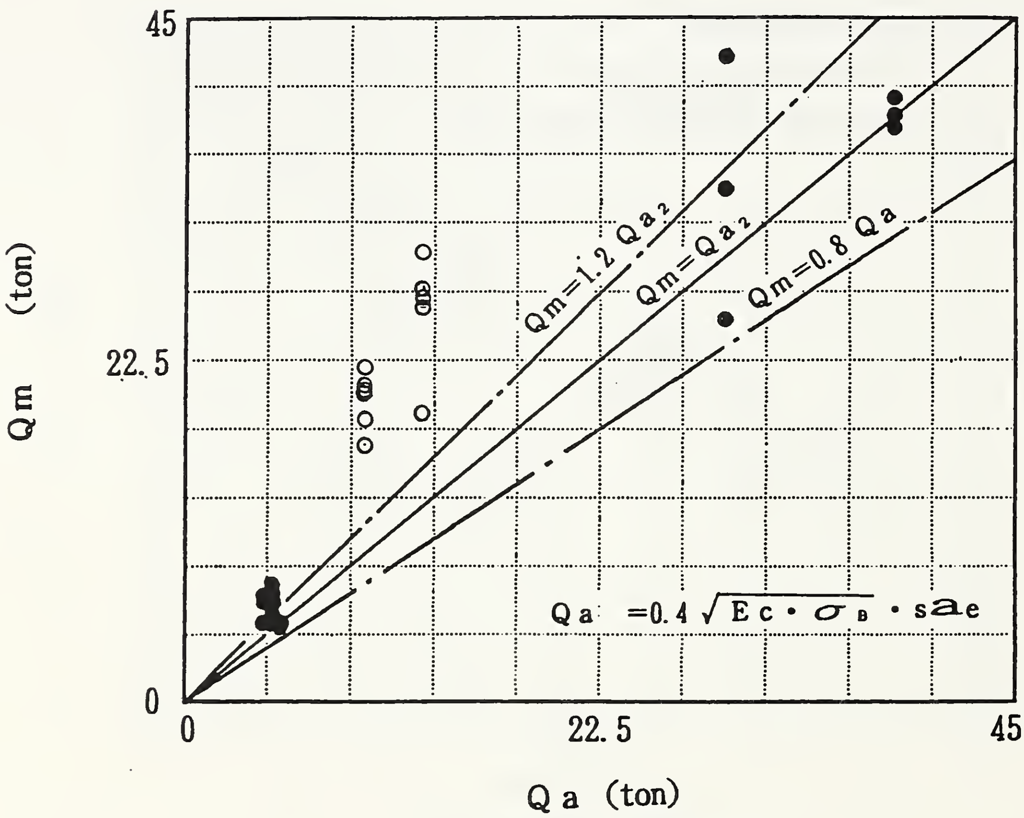


b) Chemical anchor

Fig. 17 Effective embedment length (ℓ e)versus values of maximum shear strength on test (Qm)



a) Expanded metal anchor



b) Chemical anchor

Fig. 18 Values on shear strength formula (Qa) versus values of maximum shear strength on test (Qm)

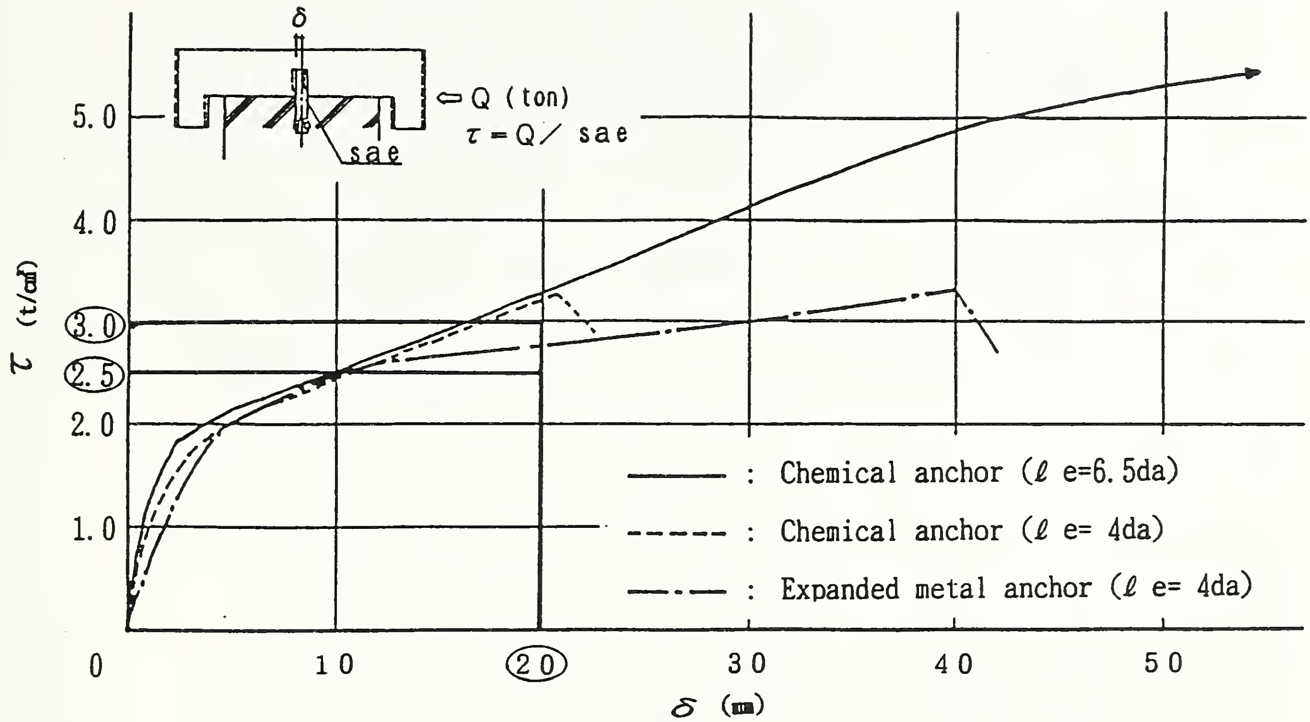


Fig. 19 Typical $\tau - \delta$ curves obtained from direct shear test of anchors and idea for upper limit values of τ

SEISMIC STRENGTHENING OF AN EXISTING REINFORCED CONCRETE BUILDING

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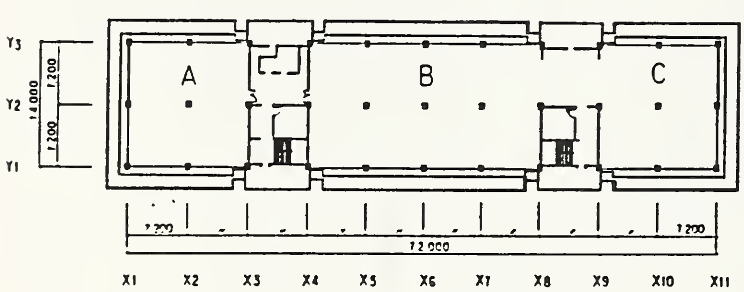
1. Introduction

This report describes seismic strengthening of a R/C office building in Tokyo.
The building is a waffle flat slab structure constructed about ten yeas ago. Recently remarkable cracks are observed and heavy shakes are noticed even in small erthquakes. The office workers begin to feel fear and, therefore, seismic capacity eveluation and sesmic strengthening are required.

2. Outline of building

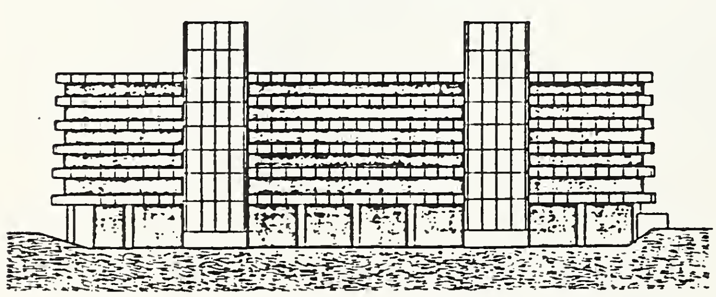
The building is a R/C building with one basement and six stories, and is used as an office building. The floor are separated into 3 office units; A, B and C, by two structural core systems.
Framed erthquake resistant walls are used for the cores.
To the exterior frame side facing to a main road, there is a lobby space which has big glass sashes through the basement and the first floor. It makes a part of outer wall.



(a) Typical Floor Plan

Table 1 outline of the building

use	office		
scale	building area	1493.1 m²	
	total floor area	10332.3 m²	
	floors	on the ground	6
		basement	1
	building height	GL+19.96 m	
structure	reinforced concrete		
	waffle flat slab structure		
basement foundation	piles constructed on the spot		



(b) West Elevation

(c) North Elevation

Fig.1 Plan and Elevation of Building

3. Evaluation of seismic capacity

The evaluation was based upon "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Building" issued by Japan Building Disaster Prevention Association.

The second evaluation method consider only the strength and ductility of each vertical members assuming the slab and beam systems are stiff and strong enough.

In the 3rd evaluation, the strength and ductility of the floor and beam systems and the failure mechanism of the whole frame are also considered.

The basic structual index (E_0) is calculated from the strength index of the ultimate strength (C) and the ductility index depending upon the failure mechanism (F).

The final judgement is modified by the time index (T) and the structural design index (Sd).

The seismic capacity index (Is) is calculated by the following fomula.

$$Is = E_0 \times Sd \times T \qquad (1)$$

4 Material strength and Moment capacity of flat slab

i)The strengths of the members used for the eveluation capacity are shown in the table 2.

Table 2 Material strength

concrete	Fc-180	σ_B =180 kg/cm²	note
reinforce-ment	SD30 SR24	σ_y =3000+500=3500 kg/cm² σ_y =3000 kg/cm²	D16～D25 9ϕ ～ 13ϕ

ii)For the estimation of the moment capacity of flat slab, two cases are considered.

----- the case where bending yield occures on the column face

----- the case where bending yield occures on the drop panel edge

The smaller value of these two is adopted as the moment capacity.

$$M_u = \min[(M_u+M_s+M_t),(M'_f+M_s+M_t)]$$

$$M_f=0.9\cdot a_0\cdot\sigma_y\cdot d\cdot\frac{C_1+d_1}{X} \qquad M'_f=M_f+Q\cdot\Delta L_1$$

$$M_s=\tau_u(C_2+d_2)\cdot d_2(C_1+d_2) \qquad Q=\frac{(M_{f1}+M_{f2})}{L_0}$$

$$M_t=\tau_{tu}\frac{d_2^2}{2}\{(C_2+d_2)-\frac{d_2}{3}\}\cdot 2 \qquad M_{f1}=0.9\cdot a_t\cdot\sigma_y\cdot d_2$$

here,

a_t:cross sectional area of one main bars

x_t:distance between main bars

a_t:the total cross sectional area of the reinforcing bars in the column strip zone

$\tau_u = 1.06\sqrt{F_c}$

$\tau_{tu}=6\tau_u$

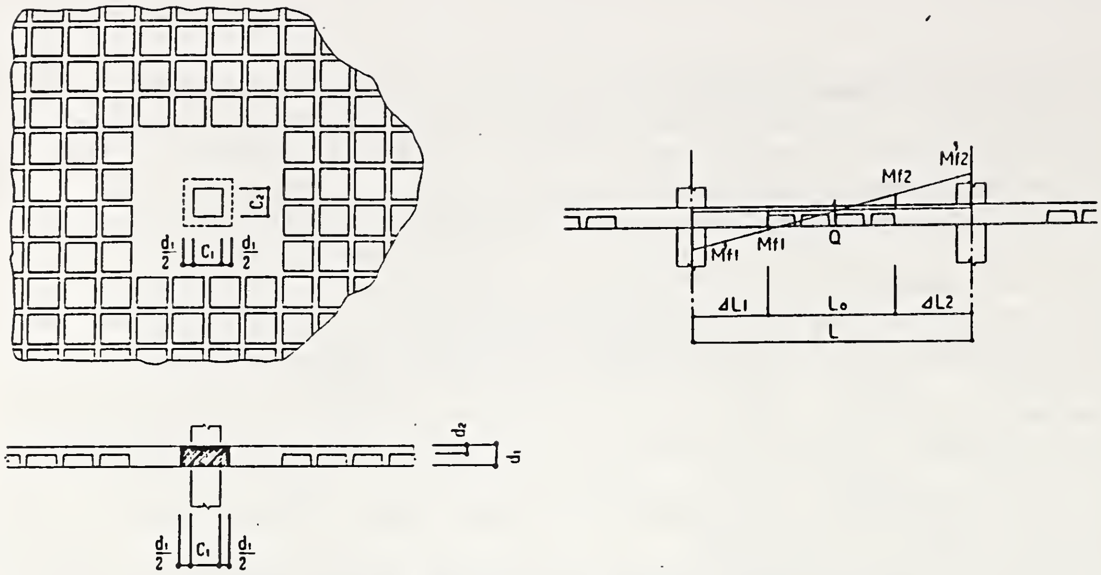


Fig.2 Layout, Panel Moments, and Critical Sections-waffle Slab with Solid Head.

5. Result of the evaluation

Since the ultimate strength of slab system is controlled by the bending yield at the drop panel edge, the same F-value as the bending beam dominating type was adopted(F=3.0). The seismic wall in the longitudinal direction was judged as a rotating wall,holding comparatively high ductility. The estimated seismic capacity index E_0 was 0.55 to the longitudinal direction.

The core wall to the span direction formed by shear walls was judged as brittle members and the result was almost E_0=0.35 .

The microtremor measurement showed that the fundamental period was 0.54 sec.,which also proved that the stiffness was rather small compared with the ordinary R/C building in Japan.

6. Strengthening Design

(i) For the increase of strength and stiffness as well as the improvement of ductility,it was proposed to install a multi-story steel braced frame which would cause less weight increasing.

(ii) The instalation of shear walls strengthened by steel plates was recommended to increase the ductility.

Based upon the principles above mentioned,the following strengthening were carried out.

X direction;

V-shaped steel braced frames were installed on Y1 and Y3 frames. However,at the lobby space of the first floor,X-shaped braced frames were installed.

Y direction;

V-shaped braced frames were installed into the walls at the both ends of the building.

The seismic wall of the structural core was strengthened by attaching steel plates.

Columns;

Corner and center columns were covered by steel plates to increase the ductility.

7. Effect of the strengthening

The result of the evaluation of the seismic capacity after the strengthening is shown in Fig.3. Sufficient seismic capacity which exceeds the target value is obtained.

The improvement of the seismic capacity depends both upon the increase of strength and the ductility as shown in Fig.4.

The E_0 value to the X direction is in the range of 0.76-0.83 ,which is about 1.3-1.5 times of the original.

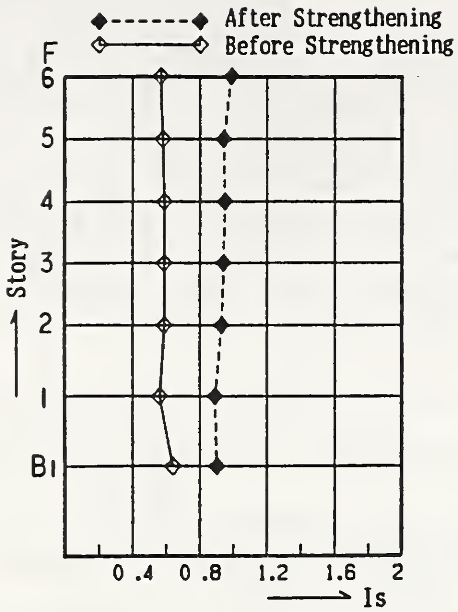
The same as the above,the E_0 value in the Y direction is in the range of 0.69-0.71 ,which is almost double of the original.

8. Microtremor measurement

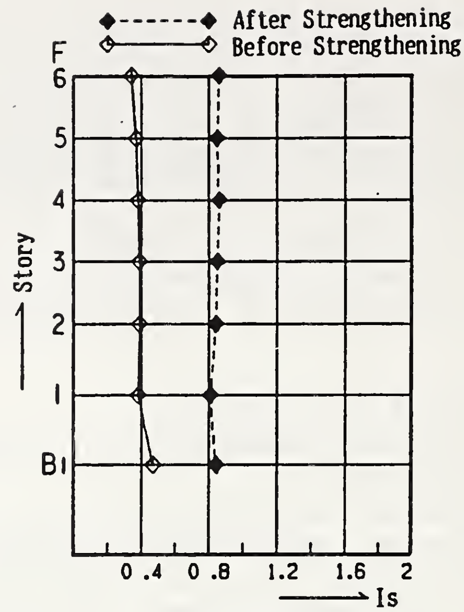
Microtremor measurements are scheduled before, during and after the strengthening.

Since the strengthening work has not been completed, the measurement was carried out where the strengthening of the basement and the first floor were completed.

The fundamental natural period of 0.44-0.51 sec. was obsearved which had been 0.54 sec. before the strengthening. To verify the increase of the stiffness, the measurent will be carried out after the completion of the strengthening.

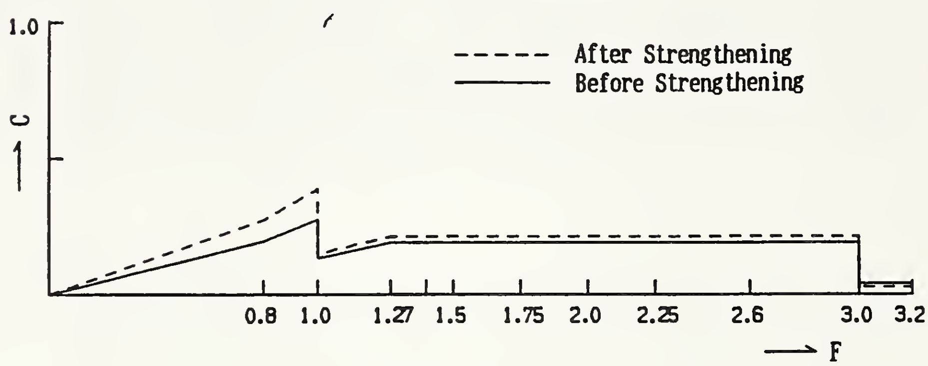


(a) X direction

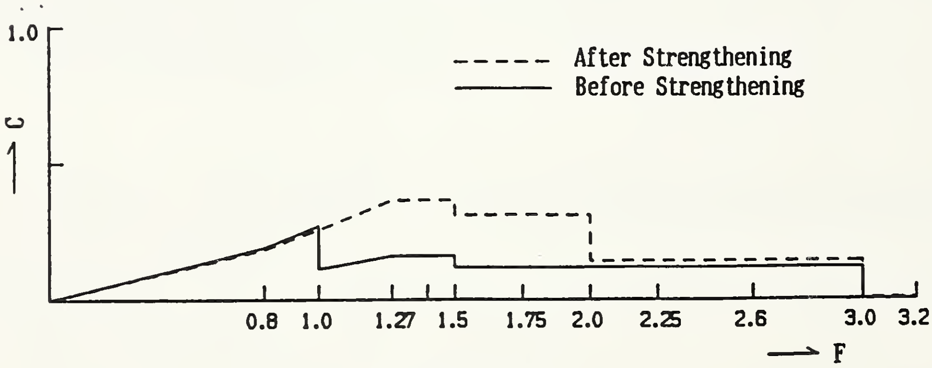


(b) Y direction

Fig.3 Distribution of Seismic Structure Index (Is) Before and After Strengthening



(a) X direction



(b) Y direction

Fig.4 First Floor C-F Value Diagram

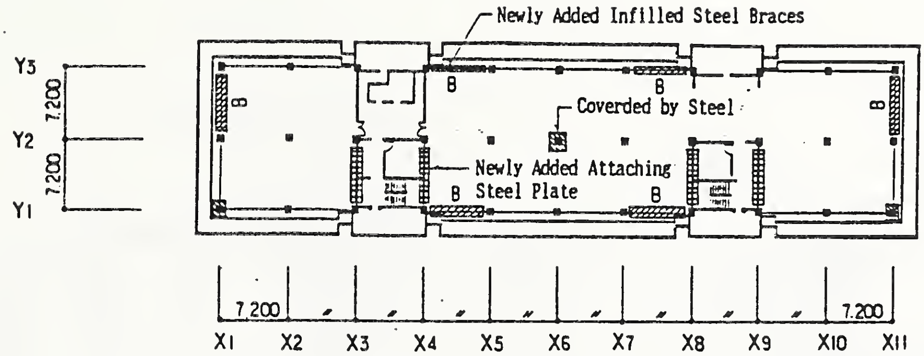
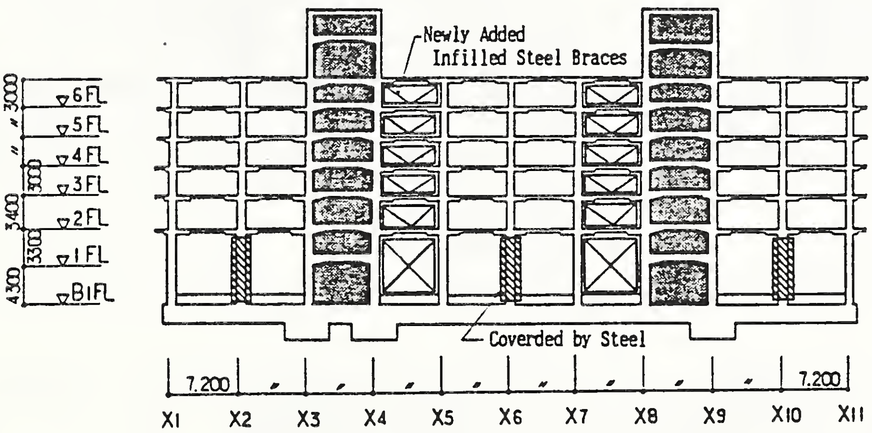
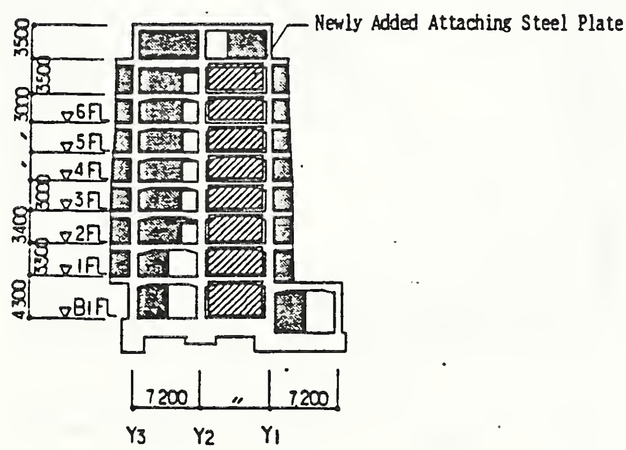


Fig.5 Typical Floor Plan After Strengthening



After Strengthening

Fig.6 Elevation of Strengthening for Longtudenal Direction



After Strengthening

Fig.7 Elevation of Strengthening for Span Direction

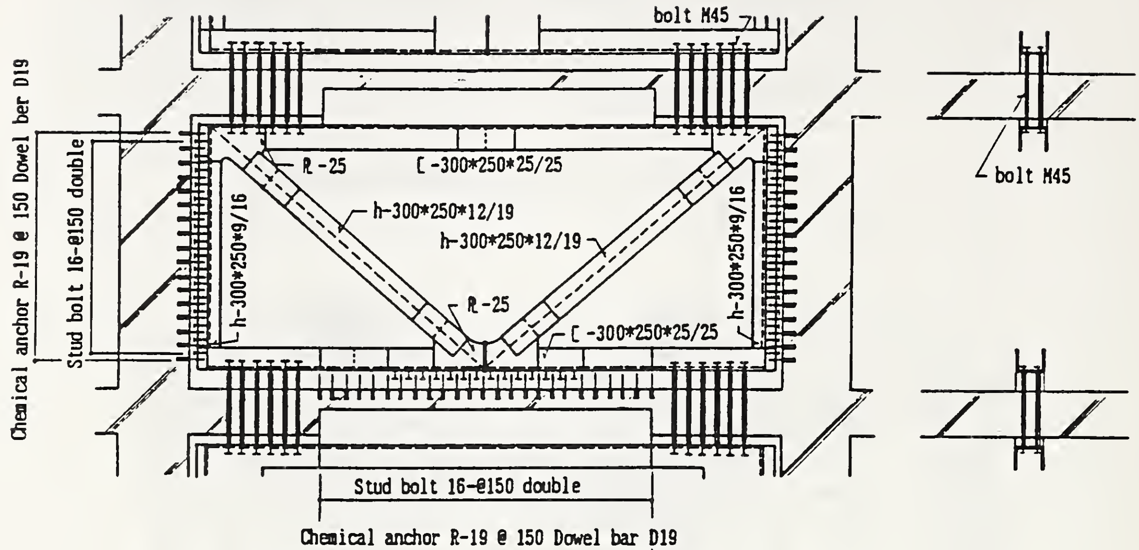


Fig.8 Newly Added Steel Brace(Typical Type)

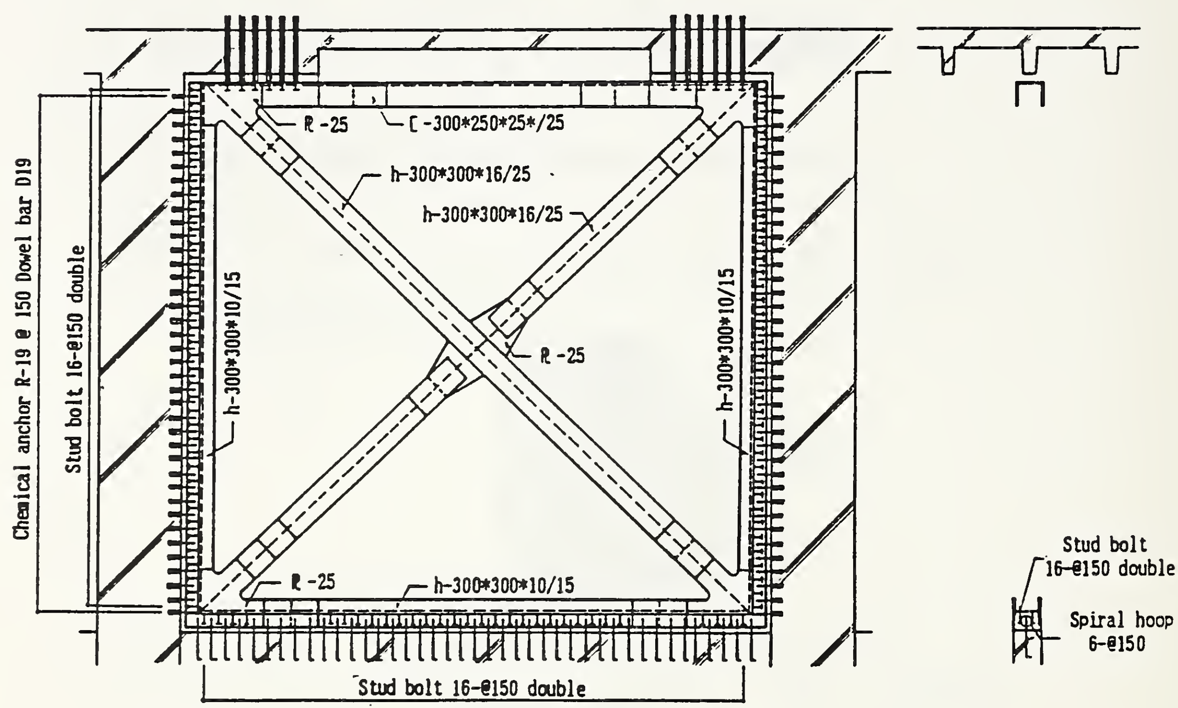


Fig.9 Newly Added Steel Brace for Basement(FrameY3)

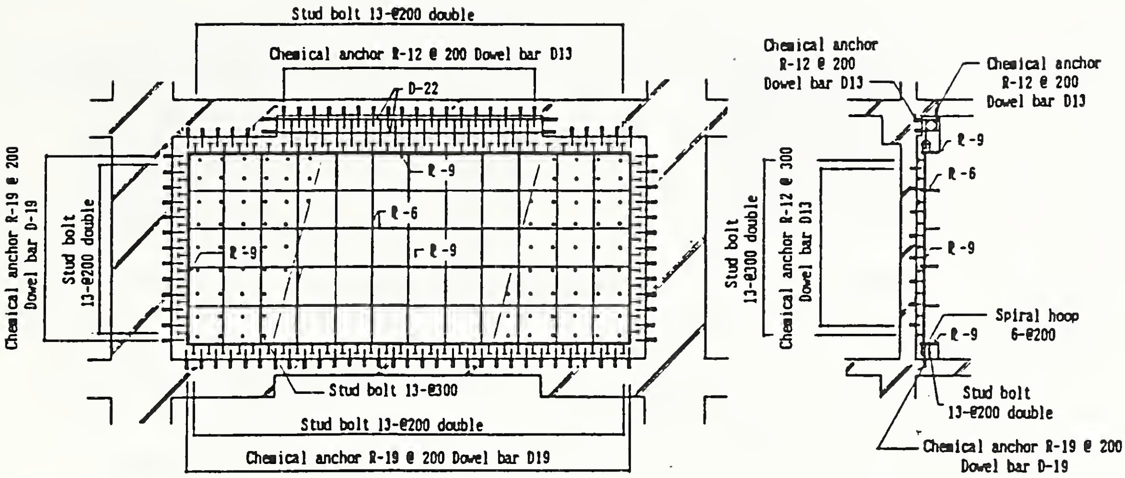


Fig.10 Details of Core Walls Attached by Steel Plate

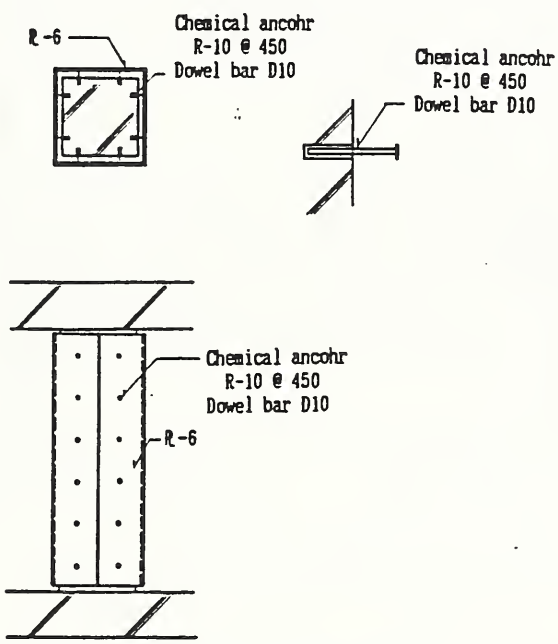
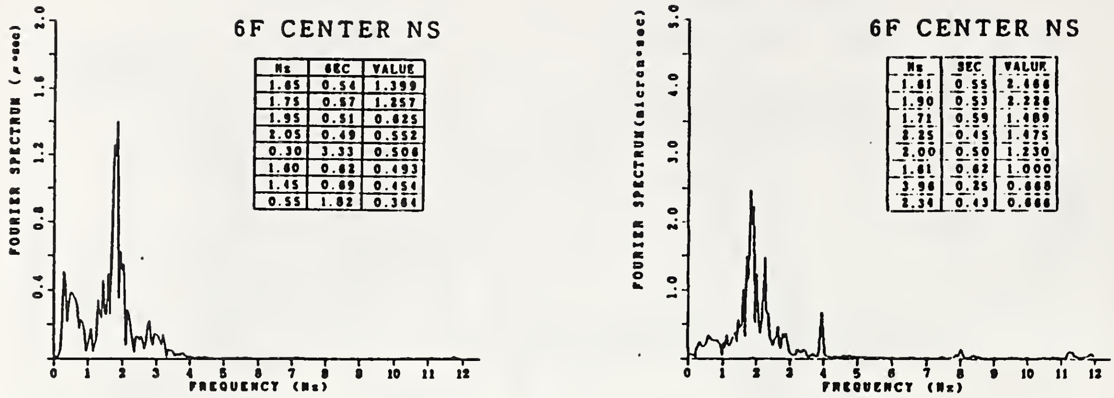
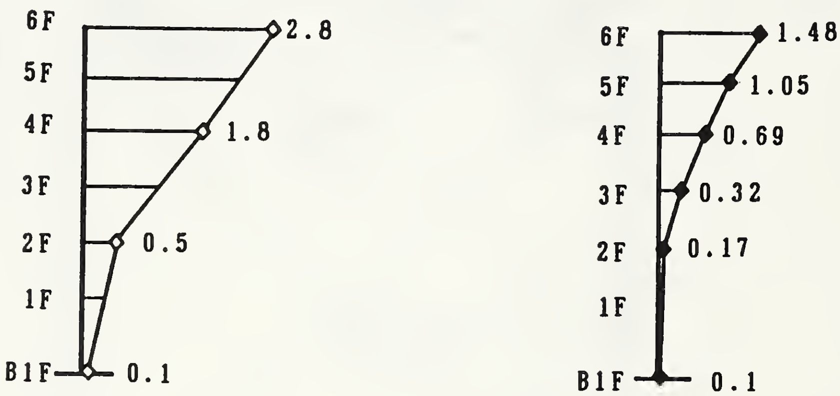


Fig.11 Details of Columns Covered by Steel Plate



(a) Before Strengthening (b) During Strengthening

Fig.12 Fourier Spectrum



(a) Before Strengthening (b) During Strengthening

Fig.13 Vibration Mode

Recent Research Results in Strengthening Methods by Steel Brace and Panel on Existing Reinforced Concrete Frames

by

Yasushi SHIMIZU* and Yasutoshi YAMAMOTO**

ABSTRACT

Various methods have been proposed to retrofit by steel members for existing reinforced concrete buildings. Collected data on recent research results of seismic retrofit of existing reinforced concrete buildings by steel members in JAPAN are introduced in this paper. Failure mode, kinds of connector, strength and ductility, etc. of twenty-four specimens are shown.

The observations indicate considerable increase in strength and ductility of the strengthened frames by steel members, and more or less the lateral load carrying capacities of specimens came near their calculated strength.

1. INTRODUCTION

When insufficient seismic safety of buildings comes into question as the results of the application of "The CRITERION on the EVALUATION of SEISMIC SAFETY of EXISTING REINFORCED CONCRETE BUILDINGS"[1], appropriate strengthening methods may be required for improving the earthquake resistant characteristics of the buildings.

In the recent tendency, strengthening methods by steel members are adopted well for these cases, in JAPAN.

This report reviews recent research results in an attempt to establish the current state-of-the-art in our knowledge of strengthening methods by steel brace and panel in JAPAN.

Collected data on test specimens of seismic retrofit of existing concrete frames in JAPAN are introduced in this paper. Very few laboratory works, however, has been done on the behaviors of strengthened frames by steel brace and panel.

Twenty-four specimens are shown, and seismic performances before and after retrofit are discussed.

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2. THE TYPICAL PAST RESEARCH IN JAPAN

The past experimental researches on strengthening methods by steel brace and panel are listed in Table 1. Twenty-four specimens strengthened by steel brace and panel, types of steel, connectors of joint, slenderness ratios of steel, maximum loads of strengthened frame, ultimate deflections and failure types etc. are shown in this Table.

The earliest research was originally completed in 1978 by S.SUGANO etc., and from 1979 to 1980 Y.SHIMIZU etc. reported the research paper. These specimens have direct shear connector, and after this, all specimens have indirect shear connector which was originally proposed by prof. Y.YAMAMOTO etc.

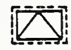
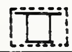
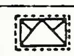
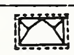
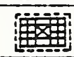
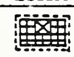
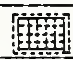
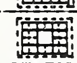
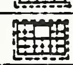
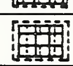
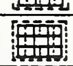
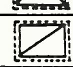
The kinds of the shear connector are shown in Figure 1. When we interpret these metods in a general sense, these can be classified into the following two classes, the direct shear connector and the indirect shear connector.

The examples of steel brace strengthening methods are shown in Figure 2.

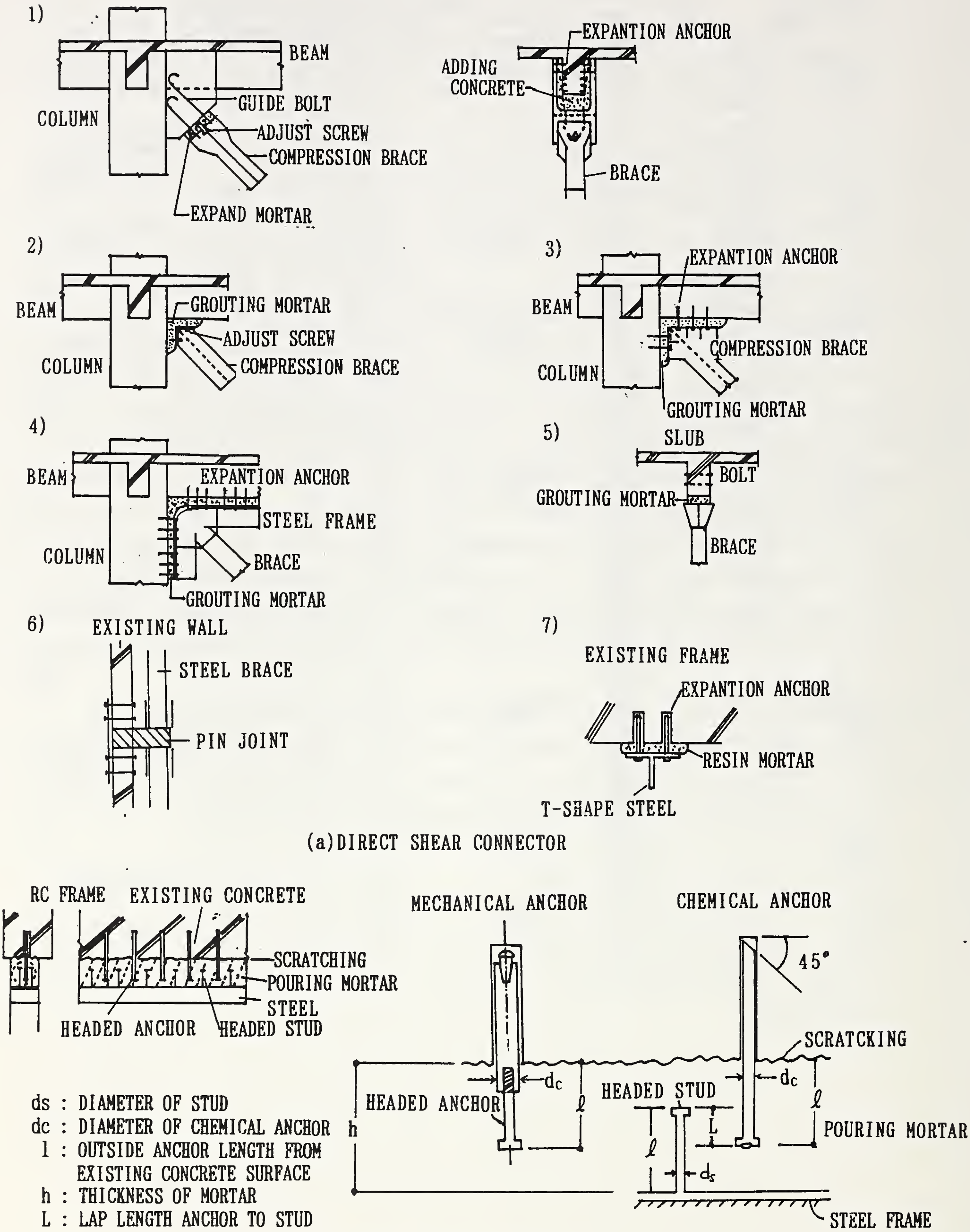
Usually, after removal of the wing wall in existing reinforced concrete frame, post-installed anchors are settled. Subsequently a steel frame with a brace or a panel are placed, and non-shrink mortar are filled up between them.

The shear connector between existing reinforced concrete frame to steel brace and panel is so important for strengthening methods by steel members. If shear slipping have occured in shear connector, proper aseismic performance is difficult to achieve for strengthening frame by steel members.

Table 1. LIST OF PAST RESEARCH

AUTHOR・PUBLICATION YEAR	SPECIMEN					TEST RESULT			
		No. SYMBOL	SHAPE	CONNECTOR	SLENDERNESS RATIO	MAXIMUM LOAD P(t)	MAXIMUM LOAD Pi/Po *1	*2 ULTIMATE DEFORMATION	FAILURE MODE
YAMAGUCHI, SUGANO, FUJIMURA, etc. 1978	W−S (PANEL)	1 □		MECHANICAL ANCHOR	(31.3)	55.0	4.30	8.81	SHEAR FAILURE IN COLUMN
	B−C (BRACE)	2 ○		——	39.4	45.1	3.52	15.0↑*3	SLIP IN BOTTOM OF COLUMN
	B−T (BRACE)	3 ○		H.T.B.	191	48.1	3.76	15.0↑	・BUCKLING IN BRACE ・SHEAR FAILURE IN COLUMN
HIGASHI, ENDO, SHIMIZU, etc. 1979	78−No.7 (BRACE)	4 ○		MECHANICAL ANCHOR	71.7	26.1	2.35	35.4	SHEAR FAILURE IN COLUMN
	78−No.8 (FRAME)	5 △			——	26.2	2.36	40.0↑	
	78−No.9 (TRUSS)	6 ▽			——	18.6	1.68	20.0	
HIGASHI, ENDO, SHIMIZU, etc. 1980	No.6-35B (BRACE)	7 ○		MECHANICAL ANCHOR	54.0	7.95	4.08	42.2↑	YIELDING IN COLUMN
	No.7-35F (FRAME)	8 △			——	6.09	3.12	52.0↑	
YAMAMOTO, KIYOTA, AOYAMA 1983	X−1 (BRACE)	9 ●		・CHEMICAL ANCHOR ・STUD	53.8	90.4	5.91	25.9	BUCKLING IN BRACE
	X−2 (BRACE)	10 ●			52.6	74.2	4.85	25.9	
	Λ−1 (BRACE)	11 ●			35.2	78.1	5.10	25.9↑	
	Λ−2 (BRACE)	12 ●			33.6	59.7	3.90	21.6	
	M−1 (BRACE)	13 ●			37.0	79.3	5.18	21.6↑	
	P−1 (PANEL)	14 ■			(56.1)	94.1	6.15	31.5↑	YIELDING IN PANEL
	P−2 (PANEL)	15 ■			(56.1)	91.2	5.96	26.6↑	
AOYAMA, YAMAMOTO, KIYOTA 1984	P-1-0	16		・CHEMICAL ANCHOR ・STUD		69.4	4.39	7.5	SHEAR FAILURE IN EXTREMELY SHORT COLUMN
	P-1-S	17				87.7	5.55	8.03	
	P-2-C	18				62.5	3.96	7.86	
	P-2-G	19				90.5	5.73	7.86	
	P-1-N	20				99.3	(6.28)	8.38	
YAMAMOTO, AOYAMA 1985	P-1-86	21		・CHEMICAL ANCHOR ・STUD		75.8	(4.80)	8.03	
	P-1-60	22				60.5	(3.83)	7.69	FLEXURAL FAILURE IN FRAME
GOTO, NKK 1989	(BLACE)	23 A−2		・CHEMICAL ANCHOR ・STUD		99.8	5.94	7.52	SHEAR FAILURE IN COLUMN
	(BLACE)	24 A−3				71.8	4.27	7.29	

*1 Pi/Po : comparison with pure frame
*2 (×10^{-3}rad.)
*3 ↑: more than



(a)DIRECT SHEAR CONNECTOR

(b) INDIRECT SHEAR CONNECTOR

Figure.1 KIND OF SHEAR CONNECTOR

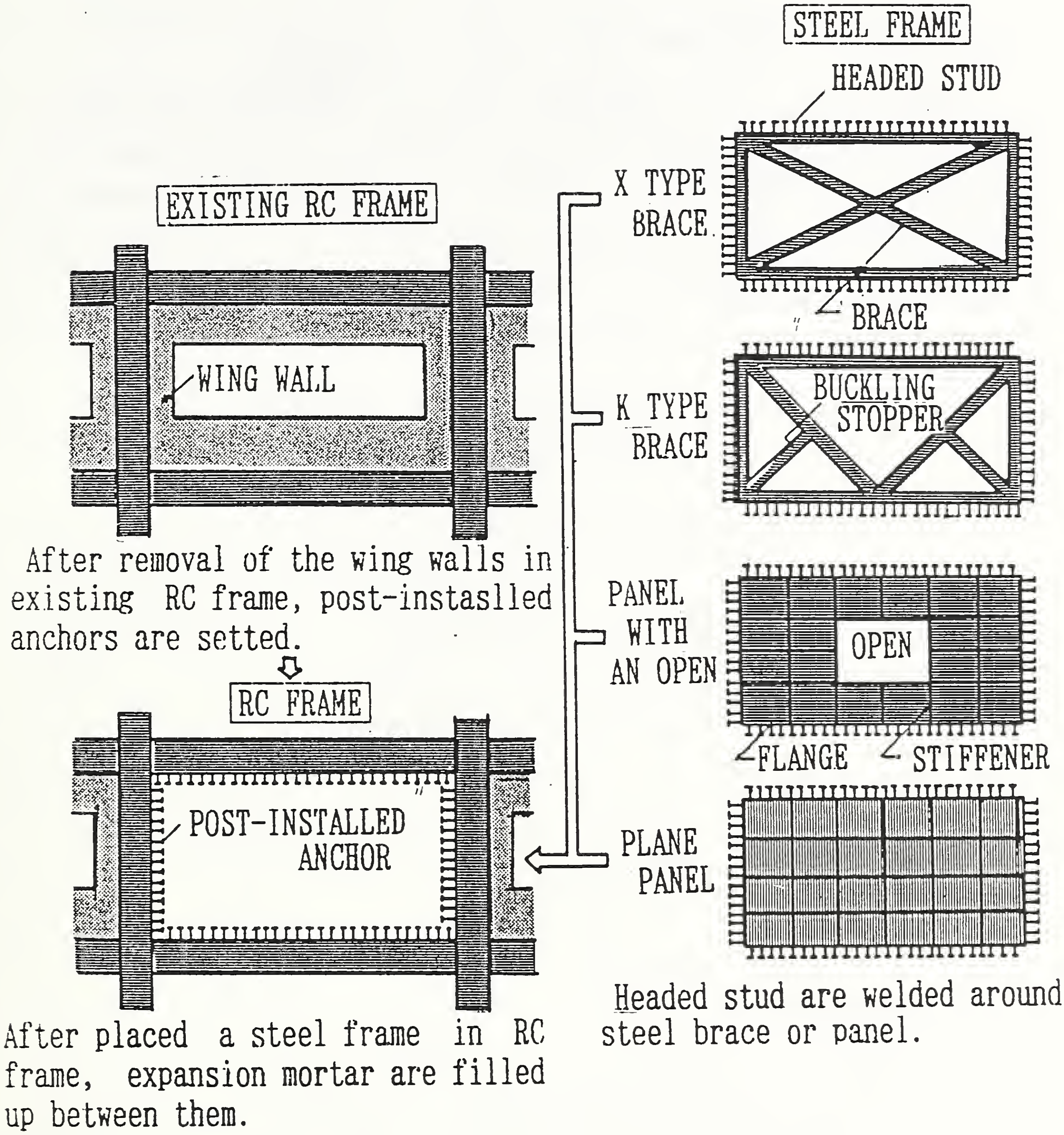


Figure 2. EXAMPLES OF STEEL BRACE STRENGTHENING METHOD

3. THE EFFECTS OF STRENGTHENING

Table 2 shows the failure types of frames strengthened by steel brace and panel. As the experimental studies progressed, the classification of the various failure types have arranged in four types. These failure types are shear failure in reinforced concrete frame (type 1), shear failure of extremly short member of reinforced concrete frame which we call punching shear failure (type 2), flexural failure in retrofitted reinforced concrete frame by steel brace and panel (type 3), and extremely brittle failure in reinforced concrete column (type 4).

Type 1 can be expected for the increase of strength and ductility. We anticipate the strength for the type 2, and increase of the ductility are anticipated for type 3.

The comparison of the experimental results Q_{max} with the calculated results Q_{wu} are shown in Figure 3. The maximum loads are calculated by following formulas[5].

CALCULATED EQUATION FOR Q_{wu}

STEEL BRACE

$$Q_{wu} = N_u \cdot \cos\theta$$

$$N_u = N_t \quad \cdots\cdots \quad (\lambda \geqq 200/\sqrt{F_c})$$

$$= 2N_c \quad \cdots\cdots \quad (200/\sqrt{F_c} > \lambda > 50/\sqrt{F_c})$$

$$= 2N_t \quad \cdots\cdots \quad (50/\sqrt{F_c} \geqq \lambda)$$

$$N_c = \{1 - 0.4(\lambda/\Lambda)^2\}\ F \cdot {}_BA \quad \cdots\cdots \quad (\lambda \leqq \Lambda)$$

$$= \{0.6F/(\lambda/\Lambda)^2\}\ {}_BA \quad \cdots\cdots \quad (\lambda > \Lambda)$$

$$N_t = F \cdot {}_BA$$

$$\Lambda = \sqrt{\{\pi^2 \cdot E/(0.6F)\}}$$

STEEL PANEL

$$Q_{wu} = t_w \cdot l_w \cdot F/\sqrt{3}$$

in which

λ : SLENDERNESS RATIO、 Λ : CRITICAL SLENDERNESS RATIO、

E : MODULUS OF ELASTICITY OF STEEL、 F : YIELD STRENGTH OF STEEL、

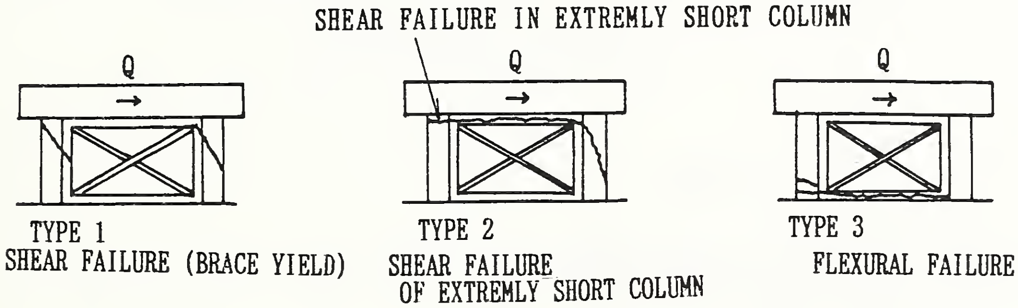
${}_BA$: SECTIONAL AREA OF STEEL BRACE、 t_w : THICKNESS OF STEEL PANEL、

l_w : HORIZONTAL LENGTH OF STEEL PANEL

The calculating values comparatively agrees with that by test in every specimens, but the data of the specimens with direct joint are widely scattered.

Table 2. THE SHEAR TRANSFER MECHANISMS OF STRENGTHENING METHODS BY STEEL BRECE AND PANEL

TYPE OF PROOFING	EXISTING RC TYPE	ADDING STEEL FRAME	JOINT
TYPE 1 STRENGTH-DUCTLITY TYPE	·FLEXURAL FAILURE IN COLUMN OR BEAM ·SHEAR FAILURE IN COLUMN OR BEAM	·BRACE ··· YIELD OR BUCKLING ·PANEL ··· PANEL SHEAR YIELD OR FLANGE YIELD	SOUNDNESS
TYPE 2 STRENGTH TYPE	SHEAR FAILURE IN EXTREMELY SHORT MEMBER	SOUNDNESS	SHEAR SLIPPING
TYPE 3 DUCTILITY TYPE	FLEXURAL FAILURE	SOUNDNESS	SOUNDNESS
TYPE 4 STRENGTH TYPE	EXTREMELY BRITTLE FAILURE IN COLUMN	·BRACE ··· YIELD OR BUCKLING ·PANEL ··· PANEL SHEAR YIELD OR FLANGE YIELD	SOUNDNESS



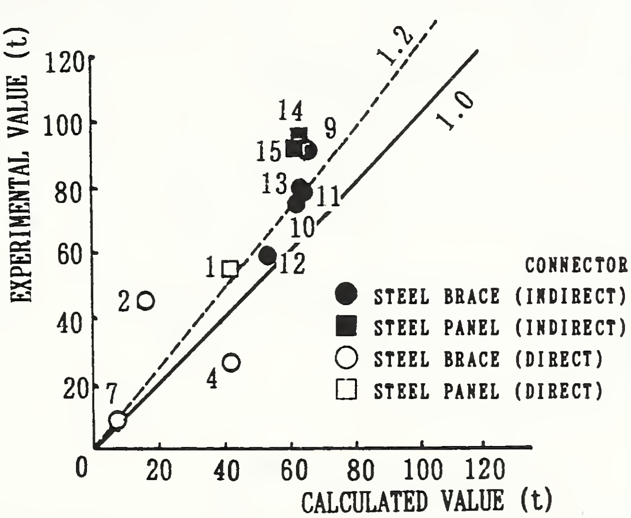


Figure 3. MAXIMUM STRENGTH Qmax OF STRENGTHENED BY STEEL BRACE

4. DUCTILITY OF STRENGTHEND FRAMES

The envelope curves of the reinforced concrete frames retrofitted by steel brace and panel with indirect connector are shown in Figure 4, and the ductility factors μ of reinforced concrete frame strengthened by steel members are shown in Figure 5. For the comparison, the data of post casted shear walls with an opening and reinforced concrete pure frame are shown in this Figure.

The method of indirect shear connector greatly affected the ductility.

For the reinforced concrete frames strengthened by steel brace and panel, the large ductility factor (F-index) can be expected.

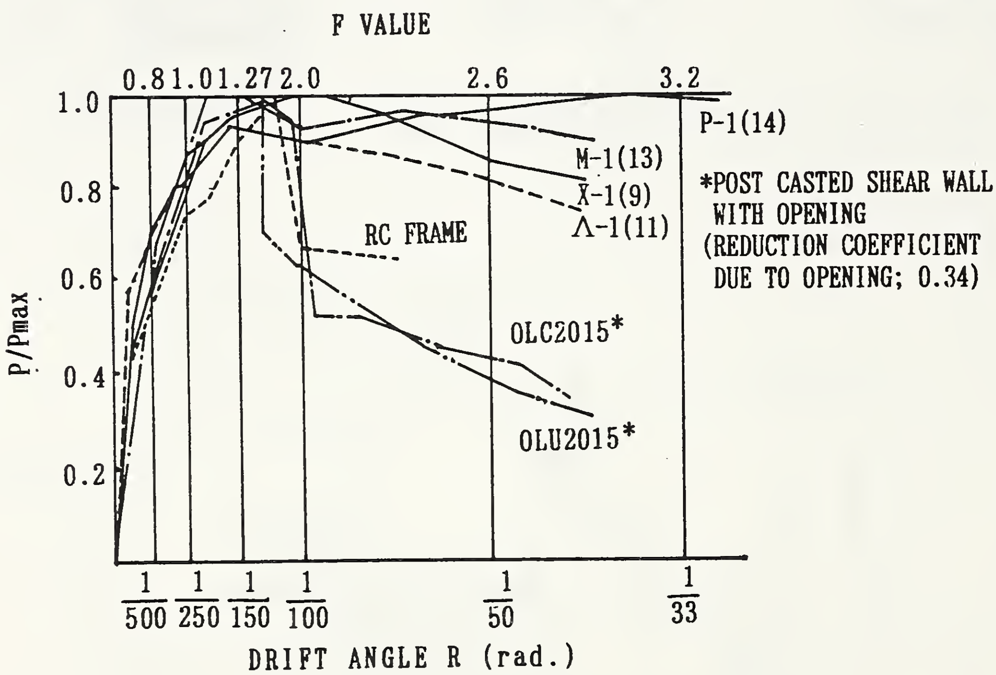


Figure 4. COMPARISON OF SKELETON CURVES

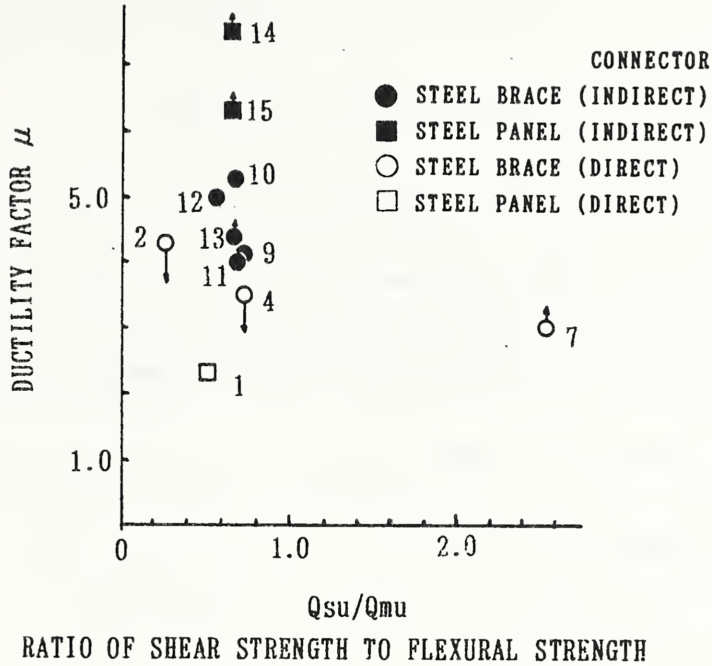


Figure 5. DUCTILITY FACTOR μ OF FRAMES STRENGTHENED BY STEEL BRACE AND PANEL

5. CONCLUSION

The results of the recent research on the effect of the strengthening methods by steel brace and panel, are summarized as follows;

1) The strengthening methods by steel brace and panel are very usefull, but in case of the direct shear connector this methods may well become difficult in specification.

2) The methods of indirect shear connector greatly affected the strength and ductility, and are usefull enough to increase the aseismic capacity of brittle frames.

3) The measured maximum loads of the tests fairly agreed with calculated values.

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Experimental Study on Mortar Joints between Reinforced Concrete Frame and Steel Shear Brace

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1. Introduction

Because the mortar joint is so important in construction, utmost care should be taken in its design when strengthening a concrete building with steel shear braces. If failure mode is considered in designing the mortar joint, the normal ductibility expected from a steel shear brace frame is difficult to obtain and proper aseismic performance is difficult to achieve. A chemical anchor bolt has been widely applied to the mortar joint for retro-bitting, and a number of experimental and research reports have described desirable bar arrangements.

The purpose of this report is to help provide greater freedom of design by using a mechanical anchor bolt instead of the usual chemical one.

We have devised a mortar joint that adopts a new type of headed mechanical anchor bolt (hereafter referred to as "anchor bolt"), and conducted shear strength tests. Fourteen mortar joint specimens with two rinds of anchor bolt diameters, embedment depths and fixing statuses (with or without tip expansion) were produced for testing.

2. Specimens

The specimens are shown in Figure 1. The specimens were produced on the assumption that they would be used in the strengthening of existing buildings by applying steel shear frames to beams of the buildings to improve aseismic performance. The specimens were of actual size and shape.

The anchor bolt was headed for use in a narrow mortar joint space 160mm in depth (Figure 2a).

Table 1 lists the specimens with their control numbers and the relative variables of the anchor bolt.

Table 1 Specimens and variable factors of mechanical anchor bolt

Specimen symbol	Diameter Da(mm)	Embedded depth dHd	Fixing status	No. of specimens
195A	19	5D	expanded	3
195Y	19	5D	straight	1
198A	19	8D	expanded	2
198Y	19	8D	straight	1
225A	22	5D	expanded	2
225Y	22	5D	straight	1
228A	22	8D	expanded	3
228Y	22	8D	straight	1

Major variables of the specimens were anchor bolt diameter (Da), embedment depth in concrete block (dHd) and fixing status, i.e., with or without tip expansion. Seven specimens had anchor bolts of 19mm in diameter, the ordinary bolt diameter for a framed steel brace used in reinforcement, while the other seven specimens had anchor bolts of 22mm

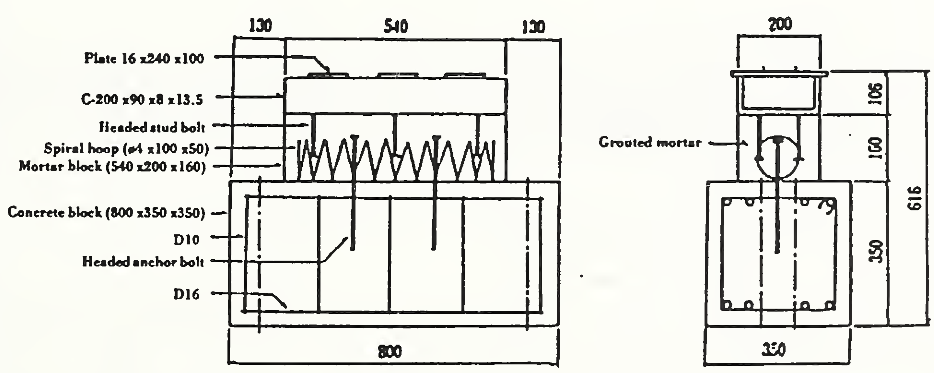

Figure 1 Mortar joint

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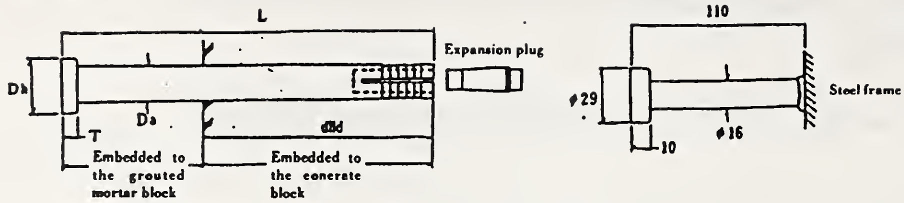
*2 Department of Architecture, Faculty of Engineering, Shibaura Institute of Technology
*3 Technical High School, Tokyo Institute of Technology
*4 Tokyo Soil Research Co., Ltd.



(a) Headed mechanical anchor bolt (b) Stud connector

Figure 2

in diameter. The embedment depth of the anchor bolts was set at 8 Da or 5 Da to examine the effects of depth, although 8 Da is most common.

To examine the effect of bolt fixing, two fixing methods were used to qualitatively analyze the relationship between anchor bolt fixing and shear strength. One type of bolt was firmly fixed with its tip end expanded (skirted) by percussion, while the other remained straight, i.e., it was not processed or altered.

The depth of embedment in the mortar block was 110mm for all specimens. Accordingly, the 19mm diameter anchor bolts had a depth of approximately 5.8 Da, while the 22mm diameter anchor bolts had a depth of 5 Da.

All headed stud bolts were 16mm in diameter (Figure 2b).

Table 2 Size of anchor bolt in mm

Description	Da	L	Dh	T	Hd	dHd
195	19	210	32	10	20	95
198	19	270	32	10	20	152
225	22	230	35	10	23	110
228	22	290	35	10	23	176

Da: outer diameter of anchor bolt
L: total length of anchor bolt
Dh: outer diameter of head of anchor bolt
T: length of head of anchor bolt
Hd: diameter of hole in mm
dHd: embedded depth in mm

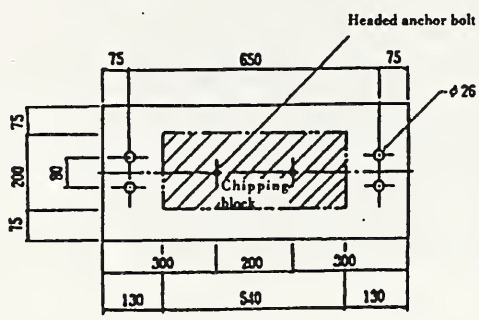


Figure 3 Concrete block, chipping block and anchor bolt position

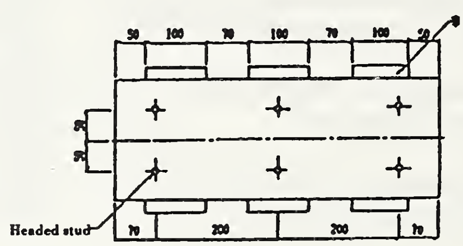


Figure 4 Steel frame and stud bolt welding position

The specimens were produced in the following sequence. Twenty days after concreting a block, two anchor bolts were embedded in each specimen (Figure 3). The concrete surface then was scratched. The steel frame, mounted with stud bolts (Figure 4), was tentatively set 16cm from the concrete block. Spiral hoops of 4mm in diameter (100ø @50) were arranged to prevent cracks, and expand mortar was grouted.

A steel frame (C-200 x90 x8 x13.5) with studs welded in two banks 20cm apart was used.

3. Mechanical properties of materials

The test results for each material composing the specimen were as follows:

Concrete material samples were produced at the same time the specimens were concreted, then aged for two or four weeks. Similarly, samples of mortar material were made when the specimens were mortared 20 days after concreting, then aged for one, two or four weeks.

Table 3-1 Test results of component materials: concrete and expanded mortar

Aging period (days)	Concrete		Mortar	
	Compressive strength kg/cm²	Young's modulus kg/cm²	Strength kg/cm²	Young's modulus kg/cm²
7	-	-	399	1.88×10^5
14	210	2.20×10^5	450	2.11×10^5
28	230	2.14×10^5	486	2.19×10^5

Table 3-2 Tensile strength test results of component materials: headed mechanical anchor bolt

Description	Diameter cm	Sectional area cm^2	Yielding point kg/cm^2	Tensile strength kg/cm^2	Stretch %
195/198	1.4	1.5	4200	4363	36.2
25/228	1.4	1.5	4742	4903	20.8

Note: The axis of anchor bolt for the experiment was processed to be compatible with the JIS Z 2202 No.4 testing specimen.

Table 3-3 Tensile strength test results of materials: Stud bolt

Materials	Diameter in mm	Sectional area in cm^2	Tensile strength kg/cm^2	Stretch %
16.0	15.75	1.9	4470	34.6

Table 3-4 Tensile strength test results of materials: Spiral hoop

Diameter mm	Sectional area cm^2	Tensile strength kg/cm^2
4.0	0.1	7400

Note: The spiral hoop of 25cm long, free from scratch, was chosen for testing after the experiments have been completed.

The axial part of the anchor bolts used for testing was processed to conform to JIS Z 2202-4 specifications and underwent tensile strength testing. The headed stud bolt was made of material compatible with SWRH16A as defined by JIS G 3507 specifications.

4. Loading device and methods of measurement

Figure 5 depicts the load device, while a displacement measuring unit is illustrated in Figure 6.

Load was applied to the specimen fixed on the steel load frame. The point to which load was applied was at the center of the mortar joint. Static load was applied horizontally from alternate ends. No axial load was applied to the mortar block. In a programmed load schedule, the first load was used for load control, and positive and negative shear force of P14 ton (=13.0 $kg/cm^{2)}$ was repeatedly applied. Next, the load was utilized for deformation control. Each loading of 81mm and 2mm lateral displacements was applied once, then load was increased to the breaking point.

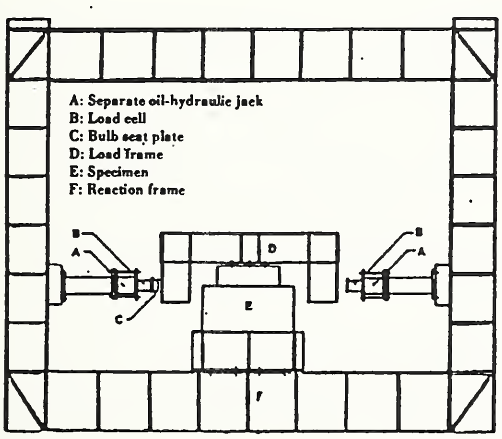


Figure 5 Loading frame for direct shear test

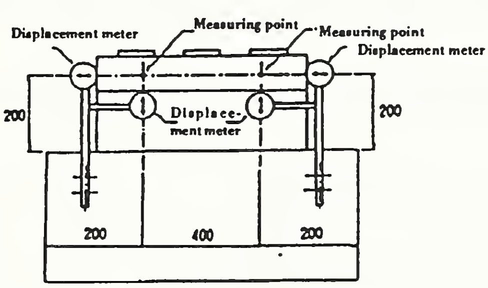


Figure 6 Displacement measuring position

Displacement was measured by a displacement meter to a precision level of 1/200 mm. The meter was used to measure the displacement of mortar and concrete, as well as the vertical lift-up of the mortar block on both sides of the specimen. Distortion of the anchor bolt was measured at the interface of the mortar and concrete.

5. Results

Results are shown in Table 4. The table shows, per cycle, the maximum load and accompanying deformation, maximum resistance (Pm) and accompanying deformation, and their values at breaking. Figure 7 indicates the final cracking pattern of the representative specimens, while Figure 8 presents results on load-deformation.

Vertical cracks developed relatively early along side the stud bolt located at the end farthest from the point of pressure. Sheared cracks developed diagonally at 45-degree angles from the root of the stud when additional pressure was applied. As the pressure increased, the sheared cracks became wider, eventually reaching the breaking stage.

Table 4 Specimens and test results

Specimens	P1 Ton	δmm	P2 Ton	δmm	P3 Ton	δmm	Pm Ton	δmm
195A-1	10.00	0.04	16.02	0.10	10.17	0.57	16.02	0.10
195A-2	13.97	0.12	16.57	0.64	18.45	2.06	18.45	2.06
195A-3	13.97	0.13	16.95	0.97	14.00	1.84	16.95	0.97
198A-4	13.97	0.06	17.75	0.64	13.37	1.45	17.75	0.64
198A-5	14.02	0.52	16.80	1.10	14.00	1.54	16.80	1.10
225A-6	14.00	0.10	14.35	0.21	7.07	1.15	14.35	0.21
225A-7	14.02	0.38	14.85	0.96	13.35	1.94	14.85	0.96
228A-8	15.05	0.11	17.80	2.08	4.10	1.76	17.80	2.08
228A-9	14.00	0.18	13.97	0.17	16.52	0.32	17.55	0.80
228A-10	14.02	0.11	16.95	0.19	15.50	0.65	16.95	0.19
195Y-11	9.97	0.03	15.82	0.22	16.20	1.01	16.20	1.01
198Y-12	13.97	0.70	14.00	1.03	18.52	1.45	18.52	1.45
225Y-13	13.25	0.38	4.50	1.17	10.60	3.28	13.25	0.38
228Y-14	13.97	0.92	14.00	1.43	15.02	2.06	15.02	2.06

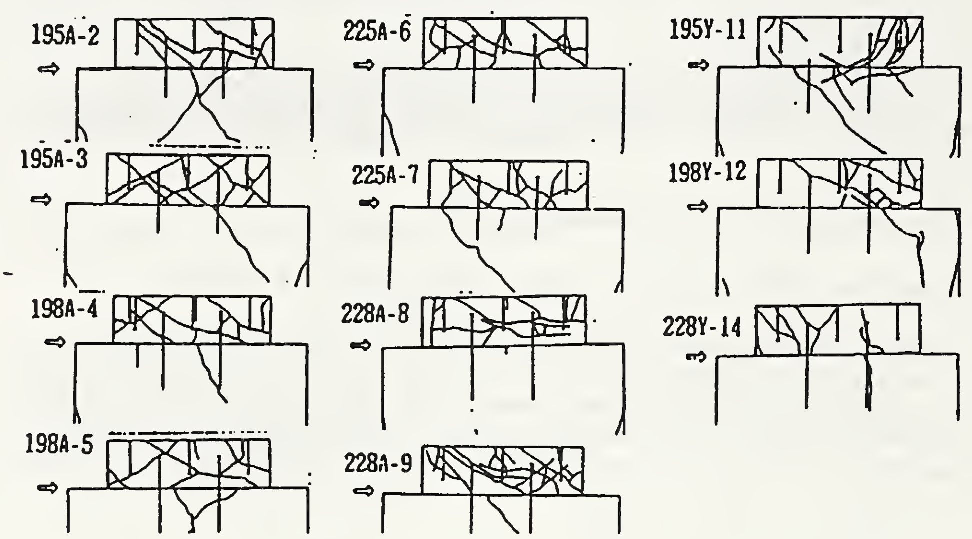


Figure 7 Breaking (failure) status at end
Arrow indicates the direction of pressurization

A lift-up of the mortar block was noted on the specimens with the straight anchor bolts (without end expansion).

As all specimens showed sheared breaking of the mortar block at the end stage, there was little difference in maximum load according to anchor bolt diameter, embedment depth or fixing status.

Table 5 Pull-out tests of anchor bolt

Specimens	Depth mm	Pmax Ton	δ mm	Remarks
TS1921-1	95	8.3	8.3	concrete breaks
TS1921-2	95	5.3	13.9	concrete breaks
TS1921-3	95	5.9	16.4	concrete breaks
TS1921-4	152	6.9	9.1	anchor tip breaks
TS1921-5	152			measurement failure
TS1921-6	152	7.0	5.9	anchor tip breaks
TS2225-7	110	8.8	3.4	concrete breaks
TS2225-8	110	7.2	10.7	concrete breaks
TS2225-9	110	7.2	10.7	concrete breaks
TS2225-10	176	9.3	20.5	concrete breaks
TS2225-11	176	8.9	20.7	concrete breaks
TS2225-12	176	8.2	21.9	concrete breaks

Figure 8 Load displacement curve

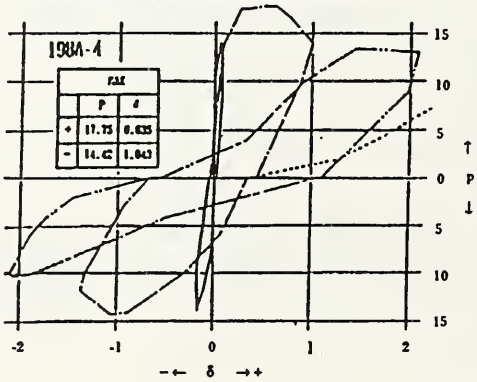


Figure 9 shows a per-unit comparison of the shear load of chemical and mechanical anchor bolts. The chemical anchor bolt samples used for comparison were D19 (2 or 3 bolts were embedded); studs were 16mm or 19mm in diameter (2, 3 or 6 studs were welded). The chemical anchor bolts were arranged in a single line, while the studs were welded in a dual array. Thirteen chemical specimens were selected as samples for comparison, and embedded in the mortar to a depth of 16cm or 19.6cm, similar to the mechanical anchor bolt specimens.

The chemical bolt group could be divided into two categories: above and below a shear load of 6 tons. Differences were considered to be caused by individual differences among samples. Shear load per unit showed similar or equal distribution among the eight chemical anchor bolts above the 6 ton shear load and the 14 mechanical anchor bolt specimens.

Table 5 presents the pull-out strength, a vital factor in evaluating the performance of a mechanical anchor bolt. Mechanical anchor bolts of the same shape were used for the shear test, but they were threaded and not headed.

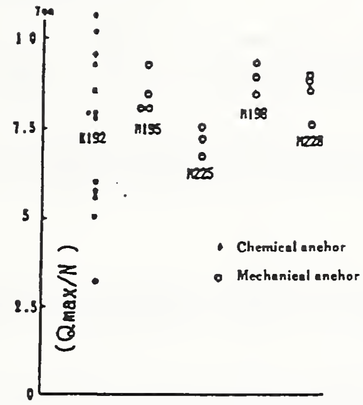


Figure 9 Comparison of shear load of chemical and mechanical anchor bolts per unit

The pull-out test was conducted on the side of the concrete specimen used for the shear test, but no obstacles were present during this testing. The mechanical tensile strength was 5444 kg/cm^2 for the 19mm diameter bolt and 6046 kg/cm^2 for the 22mm diameter bolt.

6. Discussion

From the various formulae available, the following two equations, in which strength of the concrete block is a decisive factor, were adopted here to compute the shear load of the joint using the mechanical anchor bolts. The values from the experiments and the calculations are presented in Table 6.

Design guide for retrofitting [3]

$$Q_{a1} = 0.4\,{}_{s}a_{e}\sqrt{Ec \times Fc} \quad \text{.......................(1)}$$

Composite structure design guide [4]

$$Q_{e} = 0.75\,\phi s_{2}\,(0.5\,{}_{s}a_{s}\sqrt{Ec \times Fc}) \quad \text{........... (2)}$$

where Q_{a1}, Q_{e}: allowable shear capacity per anchor bolt, ${}_{s}a_{e}$: cross-sectional area of the anchor bolt, Fc, Ec: compressive strength of the existing concrete , and Young's modulus, ϕ_{s2}: capacity reduction factor for short-term loading = 0.6.

Table 6 Comparison of experimental and calculated values

Specimens	Experiment value	Calculated by formula (1)	Exp./ Calc.(1)	Calculated by formula (2)	Exp./ Calc.(2)
195A-1	16.02	15.72	1.02	8.84	1.81
195A-2	18.45	15.72	1.17	8.84	2.09
195A-3	16.95	15.72	1.08	8.84	1.82
198A-4	17.75	15.72	1.13	8.84	1.92
198A-5	16.80	15.72	1.07	8.84	2.01
225A-6	14.35	21.33	0.67	12.00	1.20
225A-7	14.85	21.33	0.70	12.00	1.24
228A-8	17.80	21.33	0.83	12.00	1.48
228A-9	17.55	21.33	0.82	12.00	1.46
228A-10	16.95	21.33	0.79	12.00	1.41
195Y-11	16.20	15.72	1.03	8.84	1.83
198Y-11	18.52	15.72	1.18	8.84	2.10
225Y-13	13.25	21.33	0.62	12.00	1.10
228Y-14	15.02	21.33	0.70	12.00	1.25

The capacity reduction factor for short-term loading ϕ_{s2} was taken into consideration in equasion 2, causing all the calculations for the load of specimens to be lower than the experimental results. With equasion 1, the loads were calculated as lower than the experimental results for the 19mm diameter bolt, while the values calculated for the 22mm diameter bolts were higher than the experimental values. This is due to the fact that the experimental values were less influenced by anchor bolt diameter, while bolt diameter was assumed to have a direct correlation to the computed values. When applying equasion 2 to bolts with large diameters, certain considerations with regard to breaking mode should be taken into account.

7. Summary

Fourteen joint specimens with mechanical headed anchor bolts were produced with variations in anchor diameter and embedment depth, and with or without tip end expansion. A direct shear test was conducted to examine the influence of shear load on the specimens. The following results were obtained.

(1) All specimens eventually showed shear failure at their expanded mortar block. Thus, there was little difference in maximum load according to differences in anchor bolt diameter and embedment depth, as well as in fixing status.

(2) All values computed for the specimens by the composite structure design guide equasion, with short-term loading taken into consideration, provided safety-bound values.

(3) The design guide for improvements provided safety-bound values for the 19mm diameter bolt, but danger-bound values for all 22mm diameter bolts. Consequently, when applying the equasion to anchor bolts with a large diameter, the breaking mode should be taken into consideration.

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APPLICATIONS OF RETROFIT METHOD WITH CARBON FIBER FOR EXISTING REINFORCED CONCRETE STRUCTURES

by

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ABSTRACT

Some of existing reinforced concrete structures do not have sufficient seismic capacity, thereby various retrofitting techniques are proposed. However, such techniques are not always adequate on the increase in weight and the maintenence of the function. In order to overcome these problems, the authors have developed a new method with carbon fiber, which is one of high strength and light weight materials and recently developed as a 'high-tech' material.

In this paper, basic properties and application techniques of carbon fiber itself are introduced ; and the following applications of this retrofitting method are presented ; (1) ductility retrofit for columns in buildings, (2) strength retrofit for chimneys, and (3) ductility and strength retrofit for bridge columns of expressways.

KEY WORDS bridge column ; carbon fiber ; chimney ; column ; earthquake ; retrofit

1. INTRODUCTION

Some of existing reinforced concrete structures have been found from the next two not to possess sufficient seismic capacity ;

(1) large damage during recent earthquake shocks
(2) estimation according to the new codes or methods [1,2] which were established after such earthquakes.

Thereby a lot of retrofitting methods, as shown in the followings, have been proposed and some of them were applied for the actual retrofit projects in Japan [3].

1) a postcast reinforced concrete shear wall infilling to the existing reinforced concrete frame,
2) a braced steel frame installed and connected to the exisiting reinforced concrete frame,
3) a encasing steel rectangular tube grouted with non-shrinkage mortar around the existing reinforced concrete column or chimney.

These methods can improve strength, ductilty, or both of strength and ductility of structures, however there are some problems ;

1) the postcast walls and the braced steel frames often disturb the function which needs large open space.
2) the postcast walls and the encasing steel tubes (for only chimneys) are accompanied with significant increase in weight.
3) the braced steel frames and the encasing tubes need high-quality construction for satisfying structural performance evaluated in retrofitting design.

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than 2 km.

The minimum unit of such type carbon fiber is called "monofilament" (see Fig.2), and is a very fine fiber. A practical unit is called "strand" and consists of 1000 to 12000 monofilaments. Strand is usually impregnated with epoxy resin as discribed later (section 2.3). Another practical products of carbon fiber are generally manufactured from strand, like UD tape (Uni-Directional Tape). UD tape is a sheet-like product ; in the tape, carbon fiber strands are unidirectionally arranged

We use the strand impregnated with resin for transverse reinforcement of a structure and the UD tape for longitudinal reinforcement (see Fig.1).

2.3 Stress-Strain Relationship and Impregnation with Resin

The properties of HP grade carbon fiber are shown in Table.1, compared with steel. Tensile strength, Young's modulus, weight density, and durability are strong points for structural material, but elongation is a weak point. From the idealized stress-strain relationship of these two materials (see Fig.3), it is found that in carbon fiber, there is no yield plataue and no hysteretic energy dissipation, which can be highly expected of steel. We should take the following consideration into using carbon fiber ;

1) Use of carbon fiber is limited to the part or the reinforcement where care to only strength is significant.
2) Carbon fiber subjected to stress concentration may easily rupture since stress redistribution is impossible due to its brittle manner. Some techniques reducing stress concentration should be employed.
3) Carbon fiber is weak for a sharp edge, like a knife, and hence some work of arrangement on the concrete surface to be retrofitted is necessary (discussed in 2.5).

According to the second consideration, impregnation into carbon fiber strand or UD tape with resin is usually carried out as one of techniques for reducing severe stress concentration on monofilament. Non-impregnated carbon fiber strand is very weak but impregnated carbon fiber strand is as strong as monofilament (see Fig.4). We can consider the strength of the impregnated strand as sum of the strengths of the monofilaments, and also we define the area of the strand as the net area of carbon fiber, excluding the area of epoxy resin. The definition of the area of UD tape is the same as the impregnated strand.

Another technique reducing stress concentration, "unbond" substrate treatment, was introduced in the previous report [4].

2.4 Curing of Resin

This retrofit technique uses the next nature ; carbon fiber strand and tape are very flexible and easy for handling before the impregnating resin is hardened. We perform at first both impregnation into carbon fiber and winding or glueing carbon fiber onto the concrete surface, and then we start the curing of resin on the concrete surface. It couold be considered that the impregnation to carbon fiber strand could be carried out after winding. From a study of this procedure, however, it was recognized that the impregnating scatter was large.

In ordinary factories treating carbon fiber, airplane or sports goods ones, the impregnating resin is cured in high temperatures over 100 ℃ for more than 2 hours (see Table.2). In our retofit work of existing concrete

In order to overcome above problems, and focusing on the first and second problems, the anthors have developed a new retrofitting method with carbon fiber, which is stronger and lighter than steel. As shown in Fig.1, this new method is that ; onto the concrete surface of the existing member, winding carbon fiber strand spirally, as like spiral transverse reinforcement, and/or glueing carbon fiber tape along the direction of the member axis, instead of longitudinal reinforcement. Carbon fiber strand and tape are very flexible ; and hence they can tightly stick to the concrete surface and strongly confine the retrofitted concrete. This method with carbon fiber is similar to the steel tube encasing method but is superior on increase in weight since grout mortar is not used.

In this paper, applications of this method with carbon fiber, i.e. retrofitting of columns in buildings, chimneys, and bridge columns, are introduced, and investigation for the applications is discribed, including the various studies on carbon fiber itself. Among such applications, retrofitting of building columns was already reported [4].

2. CARBON FIBER

2.1 Outline

Ordinary users of carbon fiber have been (1) aircraft manufacturers and (2) sports goods manufacturers. Since their market size is not large enough to satisfy the manufacturers of carbon fiber, the building construction and/or civil engineering field, in which a huge amount of materials are consumed, is considered as one of new market [5].

The cost of carbon fiber is very expensive (approximately 100 times of that of steel per unit weight) and will not be recognized to drastically fall down if using the current producing technique. In retrofit work, however, the cost of material occupys little area of the total cost since the used amount of material is very small and the labor for retrofit work is considerable large. The percentage of the labor cost and/or the temporary work is much larger ; carbon fiber can be used in retrofit work from the point of view of an ecconomical side. Moreover, in usual retrofit work, the most important problems are ; (1) the improvement of structural performance and (2) the keeping of the function demanded to the structure. The cost of retrofit work is a secondary matter.

Apart from retrofitting itself, the discussion of newly constructing with carbon fiber is introduced for a moment. The cost of carbon fiber prevents from using it for a newly general construction though some engineers begin to recognize the remarkable performance of carbon fiber. The researchers in Japan investigate some methods in which the function of carbon fiber should be more focused on [6], for example ;

(1) off-and/or on-shore structures using high durability of carbon fiber
(2) prestressing tendons using high strength and modulus of carbon fiber

2.2 Products

Crbon fiber is composed of more than 90% carbon, and in the fiber, groups of carbon atoms are continuously connected in the direction of the fiber. Carbon fiber can be classified into many grades of mechanical properties and into two types of fiber length. In this paper, only HP (High Performance) grade and continuous type carbon fiber is discussed ; where, HP grade ; tensile strength is approximately 300 kgf/mm^2 and Young's modulus is roughly 24 tf/mm^2, and continuous type ; fiber length is not limited and generally more structures, however, such curing cannot be adopted at all. We have developed the resin of which curing is completed in ordinary temperatures, 10 to 40 ℃, for 14 to 4 days (7 days at 20℃), named LTC (Low Temperature Curing) type resin.

2.5 Strength of Carbon Fiber Strand on Beveled Corners [7]

When we retrofit an square (rectangular) column by transversely winding, carbon fiber on the corners may easily rupture, thereby some arrangement, beveling the concrete corners, is necessary (cf. section 2.3). In order to investigate influence of the beveling radius R on tensile strength of carbon fiber strand, the following tensile test is carried out.

Each test specimen, as shown in Fig.5, consists of two end plates and two carbon fiber strands impregnated with epoxy resin in the same way as ordinary construction. The test parameters are dimensions and shape of the end plates as follows ;

1) circle plate : varrying the radius R (R = 1,2,3, and 5 cm)
2) octagonal plate of which angles are sharp : corresponding to R = 0

The ordinary tensile test of carbon fiber strand is also conducted, employing straight ($R = \infty$) carbon fiber strands. The strengths corresponding to the radius R = 0,1,2,3,5, and ∞ can be consequently grasped.

The relationship between the radius R and tensile strength σ_{CF} is shown in Fig.6. When the radius is small, the strength is lower and the scatter of the test results is larger. On the other hand, the strengths of test specimens with R = 3 and 5 cm remain at approximately 5 % less than that of ordinary specimens ($R = \infty$), and the scatter is also small. Conclusion from these test results is that decrease in the tensile strength of carbon fiber on a corner can be ignored if the beveled radius of the corner is 3 cm or larger.

2.6 Influence of Temperature on Resin

Fire proof properties of the impregnated carbon fiber are controlled by the impregnating resin because carbon fiber is strong against high temperatures. Tensile tests of carbon fiber strand after 2 hour heating are conducted, varrying the maximum temperature. From the results as shown in Fig.7, we can recognize that the heating within 260 ℃ does not influence on the strength of carbon fiber strand. Although the study of fire proof design of this retrofit method has been continued and enough data have not been obtained, we may select a fire proof cover among mortar, gypsum board, or silicate board in the fire proof design so that the maximum temperature of carbon fiber on the column surface is less than 260 ℃ during a fire.

2.7 Concluding Remarks

In this chapter, the following points are discussed.

(1) Background of using carbon fiber in retrofitting
(2) 3 types of product, that is, monofilament, strand, and UD tape
(3) Stress-strain relationship of carbon fiber.
A weak point of carbon fiber is brittleness, including stress concentration, thereby the treatment (4) or the study (5) are important.
(4) Impregnation with epoxy resin and curing of the resin
The impregnation reduces stress concentration of monofilaments in strand.
(5) Strength of carbon fiber strand on beveled corners
If the radius of the beveled corner is 3 cm or larger, influence of corners on the strength of carbon fiber strand is negligible.
(6) Influence of temperature on resin

3. RETROFIT OF COLUMNS IN BUILDINGS

3.1 Introduction

Some of columns in existing reinforced concrete buildings are not ductile because ;

(1) trasnverse reinforcement is not sufficiently provided, and/or

(2) the clear span of such columns is shortened by the restraining of non-structural wall

Increasing transverse reinforcement by winding carbon fiber strand is one of retrofitting methods which improve the ductility of such brittle columns.

In the test of the previous paper [4], improvement of ductility was investigated and in this paper, improvement of shear strength through the above mentioned technique is experimentally studied, employing 15 beam type specimens. In the last of this chapter, construction procedure of this winding method is also introduced.

3.2 Shear Strength Test [7]

3.2.1 Outline

Among various factors influencing shear strength of a reinforced concrete member, test parameters of this experiment are determined as the next three ;

1) transverse reinforcement ratio p_{wf}

2) shear span ratio a/D

3) compressive strength of concrete F_c

In an ordinary existing column, there would be hoops and axial force, however these factors, which increase shear strength, would hide the effect of retrofitting carbon fiber. In this study, thereby these two factors are omitted, and a reinforced concrete beam without hoop in concrete is to be discussed. The first parameter, transverse reinforcement ratio p_{wf}, expresses the quantity of carbon fiber for retrofit. The list of specimens is shown in Table.3

For demensions of the specimen and loading method, we are referred to the previous work by Kokusho and Fukuhara, et al. [8], in which high strength steel was used for transverse reinforcement and shear strengths of reinforced concrete members were discussed. The specimens in this study, as shown in Fig.8, are reinforced concrete beams without hoop bar but most of specimens are strengthened with carbon fiber strand wound onto the concrete surface. Monotonic and antisymmetric loading is carried out (see Fig.9). The inflection point is located at the center of the clear span and the shear force induced within the clear span is constant. Material properties of the specimen are shown in Table.4.

3.2.2 Test Results

In all specimens, large diagonal cracks and bond cracks are observed during the test ; and in the retrofitted specimens, carbon fiber strands rupture at the final stage of the test. Failure patterns of the BWM series (a/D = 1.5, F_c = 214 kgf/cm^2, varrying only the amount of transverse reinforcement) are shown in Fig.10. We can find that the angle between shear crack and the member axis approaches from 20 to 45 deg as the amount of transverse reinforcement increases. Since the direction of the compressive stress flow in concrete approximately agrees with the direction of cracks, we can estimate the shear transfer mechanism as follows ;

(1) When the amount of transverse reinforcement is small, the shear force

is mainly transfered through the concrete strut from stub to stub.

(2) When the amount of transverse reinforcement is large, the shear force is mainly transfered by carbon fiber strands. The action of the concrete strut with the angle of 45 deg induces the reactions of the carbon fiber strands and the longitudinal reinforcement.

In the code recently proposed in Japan [9], the first mechanism is called "arch action" and the second "truss action".

Shear force vs. displacement relationship of the BL series (a/D = 2.0, F_c = 284 kgf/cm^2) and the BMW series (previously described) is shown in Fig.11. As for all test series, the maximum shear force rises as the amount of carbon fiber increases. The displacement at maximum shear force becomes also large and there is a case in which the drift angle R at maximum shear force exceeds 1/50. Longitudinal reinforcement does not show flexural yielding except for the specimens BM24, BM18, and BMW24, which are heavily strengthened with carbon fiber strand.

The strains of carbon fiber strand measured at some points on the central axis of the beam reach 0.8 to 1.0 percent at the final stage. The variation of the strains is not so severe because the level of shear stress is high and the shear cracks occur within the whole clear span. For these specimens, we can estimate that at the final stage, the stress of any carbon fiber strand is 2/3 of the maximum strength of carbon fiber, according to the stress-strain relationship (see Fig.3).

3.2.3 Evaluation of Maximum Strength

The relationship between maximum strength of these test series and transverse reinforcement ratio is shown in Fig.12. The calculated shear strengths, Qsu(1) by Eq.1 and Qsu(2) by Eq.2 (see Table.5), are presented in this figure. Our opinion is that it is possible to apply these equations for retrofitting with carbon fiber although the equations have been proposed for ordinary reinforced concrete members ; because the state of stress induced to the concrete does not change so much when carbon fiber strand is used for transverse reinforcement ; and because shear failure is determined by properties of concrete.

Qsu(1) is proposed in the new design code [9] and is one of theoretical estimations for shear strength of reinforced concrete members, expressed as sum of the contributions of the "arch action" and the "truss action". Qsu(2) is often employed in the current design procedure and is an emprical estimation proposed by Arakawa [10], indicating the mean value of the experimental shear strength. In these equation, p_w and σ_{wy} is evaluated as follows ;

(1) $p_w = p_{wf}$ and $p_{wf} = \sum a_f / b$

where, $\sum a_f$ = sum of area of carbon fiber strand within unit length

b = width of beam

(2) $\sigma_{wy} = \sigma_{wf}$ and $\sigma_{wf} = \sigma_{CF} \times (2/3)$

where, σ_{CF} = tensile strength of carbon fiber strand

(2/3) = ratio of the induced stress to tensile strength ; determined from the test results (cf. 3.2.2)

Though the original Eqs.1 and 2 have limitations on p_w and σ_{wy}, this study is conducted without being restrained by these limitations in order to grasp the efficiency of these equations when using high strength material.

Compared with the experimental value, the next tendencies of the calculated values are pointed out ;

(1) Evaluation by Qsu(2) is conservative while that by Qsu(1) is slightly

higher.

(2) When the amout of carbon fiber increases, the first tendency is more prominent. When the amount of carbon fiber is small ($p_{wf} \leq 0.06$ %), this tendency is sometimes reversed.

As for the trend of the Qsu(1), we can explain that the displacement at maximum strength is greater than the supposed displacement level at which shear failure occurs in ordinary reinforced concrete beams. That is, since the displacement is large when the amount of carbon fiber increases, the damage of concrete becomes severe ; and the effective strength coefficient ν of concrete and the direction cotϕ of truss action are too high, which are expressed by functions of displacement level and decrease for larger displacement.

3.2.4 Conclusion

The conclusions of this test on shear retrofitting effect with carbon fiber strand are that ;

(1) Shear strength can be increased by strengthening with carbon fiber strand.

(2) Shear strength after retrofit can be almost estimated by Eq.1 according to the new design code.

3.3 Procedure of Retrofit Work

The procedure of the retrofit work for columns in buildings is shown in Fig.13 ;

(1) Removal of the existing finishing and arrangement of the concrete surface
The existing finishing of a column for retrofit is removed in order to expose the structural concrete. The concrete surface is smoothly arranged and when a square (rectangular, etc.) sectioned column is dealt with, all corners are beveled according to the results of section 2.5.

(2) Winding carbon fiber strand
Carbon fiber strand is wound onto the arranged concrete surface through the winding machine, one of which is shown in Photo.1 ; diameter = 2.91 m ; and sticked to a column of 1 m square. This machine consists of the next 4 parts ;

1) inner ring : sticked to the column and supporting machine weight

2) outer ring : rotating around the inner ring (i.e. the retrofitted column) and carrying some sets of the boom and the impregnating unit (see later). The machine in Photo.1 carrys 4 sets of the boom and the unit in order to shorten the winding work time.

3) boom : descending/ascending proportionally to the rotation of the outer ring and feeding carbon fiber strand to the column from the top/bottom end of boom itself. Thus the strand can be wound with a constant pitch.

4) impregnating unit : impregnating with resin to the strand, applying small tension to the strand in order not to loosen, and feeding the strand to the top/bottom end of the boom

(3) finishing
The purposes of finishing are ;

1) protection of carbon fiber from human mischief

2) fire protection

3) architectural design

For fire protection, we choose an adequate finishing method, referred to the results of section 2.6.

3.4 Concluding Remarks

In this chapter, a retrofit method of existing reinforced concrete columns by winding carbon fiber strand is presented.

Our structural design, where the main work is to determine the amount of carbon fiber, is carried out as the following procedure ;

(1) provide carbon fiber strand so that the column will not fail in shear, estimating the shear strength by Eq.1 presented in section 3.2.3.

(2) provide carbon fiber strand so that the column will posess sufficient ductility against a supposing earthquake, estimating the ductility by results of the previous work [4].

Retrofit work is conducted as shown in section 3.3.

4. RETROFIT OF CHIMNEYS [11]

4.1 Introduction

Some of existing reinforced concrete chimneys in Japan have often damaged and sometimes broken at the height of 2/3 or more of the total height when a large earthquake attacked. This is because the previous design regulations (effective up to about 15 years ago) did not demand enough flexural strength in the top part of chimneys, thereby retrofit of longitudinal reinforcement should be performed for such chimneys in high risk regions. In a general chimney, dead load is nearly equal to weight of the structural part of the chimney ; and the retrofit work is done at the high and narrow place. Retrofitting material is required to posess (1) high strength and (2) light weight. Carbon fiber satisfys such demand, and we investigate the method of longitudinal reinforcement with carbon fiber.

In this cahpter, the followings are presented ;

1) illustration of retrofit method
2) test of structural performance
3) example of retrofit work

4.2 Retrofit Method for Existing Chimneys

Ordinary retrofit methods for exiting chimneys are ;

(1) cutting the upper part of a chimney and setting to this part a new stainless steel tube. If employing, operation of the chimney should be stopped.

(2) encasing a chimney with a steel tube and grouting mortar into the gap between the chimney and the tube. If employing, increase in weight cannot be ignored, thereby the retrofitting zone is enlarged.

The new method developed by the authors, as illustrated in Fig.14, is carried out as glueing carbon fiber UD tape onto the existing concrete surface. This method can overcome above difficulties and have various merits ;

(1) Operation of the chimney should not be stopped since only the outside of the chimney is retrofitted.

(2) The retrofitting zone is not enlarged since increased weight accompanied with retrofitting is negligible due to using light weight material.

(3) Durability of concrete is improved because the retrofitting carbon fiber sheets cover the concrete surface.

(4) Good durability is expected even under severe environment, like in the on-shore region.

Basic concept of this method consists of increasing longitudinal reinforcement through glueing carbon fiber UD tape on the outside of a chimney. In a general chimney, which is very tall compared with its diameter, the flexural strength is often critical rather than the shear strength. If shear strength and/or thermal stress of the hoop direction cannot be ignored, transverse reinforcing is performed.by winding carbon fiber strand.

Bond between concrete and carbon fiber is discussed here. If the bond is good, the strain, that is, the stress of carbon fiber is concentrated on a cracked portion but rupturing of carbon fiber easily occurs. For transverse reinforcement, if retrofitted enough, bond condition is not intrinsic [4], however for longitudinal reinforcement, bond is necessary. If bond is lost, elongation of longitudinal carbon fiber for carrying stress is much large since the carbon fiber is arranged along the member axis. The large elongation causes large deformation in the chimney and crushing of concrete. Thus we employ one of bond improvement techniques ; substrate treatment with epoxy primer.

A detail procedure for retrofitting of generally damaged chimneys is presented as follows ;

(1) Removal of a lightning conductor and a ladder
(2) Substrate treatment and/or arrangement of concrete surface
    The substrate treatment is to paint primer onto the concrete surface. This primer is one of epoxy resin ; and penetrates into concrete and helps adhesion between concrete and carbon fiber sheet.
(3) Glueing carbon fiber UD tape in the longitudinal direction
    Adhesive epoxy resin is painted on the concrete surface ; sheet type carbon fiber is glued along the axial direction on the whole surface of the retrofitting zone. If necessary, the procedure mentioned is repeated ; carbon fiber sheet is glued twice or more, until structural demand is satisfied.
(4) Winding carbon fiber strand in the hoop direction after hardening of the adhesive used in (3) step
(5) Resetting a lightning conductor and a ladder
(6) Painting according to the provisions for safety of airplanes

4.3 Test of Longitudinal Reinforcing for Circular Hollow Reinforced Concrete Beams

4.3.1 Outline

The specimens, modelled on an existing reinforced concrete chimney, are six circular hollow reinforced concrete beams, as shown in Fig.15. The test parameters are the next three (see Table.6) ;

(1) The amount of carbon fiber glued onto the outside surface
(2) The type of carbon fiber products glued onto the concrete ; UD tape and "cloth" are employed, where "cloth" is a cloth-like product which is longitudinally and transversely woven with carbon fiber strand.
(3) The amount of transverse carbon fiber strand wound onto the glued carbon fiber sheet

After glueing, specimens are strengthened at loading and supporting points with steel rings around the outside in order to prevent local failure. Within the concrete, 10 D3 bars (gross area = 0.72 cm^2) are arranged as longitudinal reinforcement and 3.2 ϕ spiral hoop bar (area = 0.080 cm^2) is arranged as transverse reinforcement. Material properties are shown in Table.7. Monotonic load is applied at two points of the specimen (see Photo.2) ; bending moment is constant between loading points.

4.3.2 Test results

A. Failure pattern

Failure pattern is shown in Photo.3. The spread of damage of each type specimen is the followings ;

(1) Non-retrofitted specimen (No.1)
After flexural cracking, increase in carrying load comes to stop. At the ultimete stage, crush of concrete causes reduction in carrying load.

(2) Retrofitted specimen with transverse carbon fiber (No.3 to 6)
Even after flexural cracking, increase in carrying load continues. At the ultimate stage, longitudinal carbon fiber sheets and reinforcing steel bars rupture and load carrying capacity is lost.

(3) Retrofitted specimen without transverse carbon fiber (No.2)
Damage is almost the same as the specimen No.3 to 6 up to the ultimate stage. Within constant bending moment zone, the carbon fiber sheet on the lower part of the specimen is finally peeled off from concrete. On the peeled zone, a lot of distributed flexural cracks are obseved. In the compressive side of the constant bending moment zone, expansion due to crushing of concrete is found.

B. Load-displacement relationship

The load-displacement relationship is shown in Fig.16.

(1) From Fig.16(a), in which influence of the amount of longitudinal carbon fiber sheet is expressed, it is found that when the amount of longitudinal carbon fiber is large,
1) maximum strength is much improved ;
2) displacement at maximum strength becomes larger ; and
3) tangent stiffness after cracking rises.
However,
4) initial stiffness is not inflenced by glueing carbon fiber.

(2) From Fig.16(b), effect of transverse carbon fiber on strength and displacement is not recognized.

(3) Fig.16(c) shows influence of carbon fiber products. Strength is little affected, however at the same carrying load, displacement of the "cloth" specimen is larger than that of the "UD tape" specimen.

C. Carrying load

Flexural cracking and maximum loads are shown in Table.8, compared with the analytical values. The analysis is carried out as the same way as the ordinary flexural analysis of a reinforced concrete section, employing the stress-strain relationships shown in Fig.17, where the maximum stress of carbon fiber is defined as the full tensile strength.

Experimental values of flexural cracking load are determined from load-strain relationship of carbon fiber sheet and longitudinal reinforcing bars because the concrete surface, surrounded by carbon fiber sheet, cannot be observed. Experimental values of flexural craking load are 1.24 to 1.75 times larger than analytical values, and are slightly improved by retrofitting.

The maximum carrying load of the non-retrofitted specimen No.1 is much smaller than the others ; and the load of the most retrofitted specimen No.6 is the highest. The experimental maximum load is 0.85 to 0.93 times of the analytical value. In order to express effectiveness of retrofitting with carbon fiber, the effective ratio α of carbon fiber is defined from experimental and analytical values as the next equation ;

$$\alpha = \{P(e) - P_c(e)\}/\{P(a) - P_c(a)\}$$

where, $P(e)$ = experimental maximum carrying load
$P(a)$ = analytical maximum carrying load

$P_c(e)$ = experimental ultimate carrying load of the specimen No.1
$P_c(a)$ = analytical ultimate carrying load of the specimen No.1

The value α is 0.82 to 0.88 and the average value is 0.85. We can estimate the flexural strength P of the reinforced concrete beam retrofitted by glueing carbon fiber sheet as follows ;

$$P = P_c + \alpha \cdot P_f$$

where, P_c = flexural carrying capacity of reinforced concrete beams
P_f = contribution to flexural capacity of carbon fiber, employing the full strength of carbon fiber

D. Load-strain relationship

Fig.18 shows strains of longitudinal carbon fiber and steel. In the compressive side (Fig.18(a)), carbon fiber contributes with concrete to carrying compressive stress until the load reaches 88% of the maximum. After that time, crush of concrete prevents compressive stress being transfered to the carbon fiber. In the tensile side (Fig.18(b)), strain of steel agrees with strain of carbon fiber up to yielding of steel. After that time, strain of steel becomes larger, influenced by cracking of concrete.

4.4 Example of Retrofit Work

A test on retrofit work was conducted, using a small chimney ; total height = 15 m, diameter at the bottom = 1385 mm, and diameter at the top = 930 mm. Retrofitting zone was from 9.0 m height over the ground to 13.5 m height ; length = 4.5 m and area = 12 m^2.

Work was carried out mainly using a high lifter, which is a truck having a boom and a scaffold at the top of the boom and shifting the scaffold through operation of the boom. For winding carbon fiber, the winding machine, as previouly described (cf. 3.3), was employed. It is noted that in the current work for (high rise) chimneys, a moving scaffold has been developed and employed ; on the scaffold, all retrofitting work, substrate treatment, glueing, winding, and painting, can be conducted. A winding machine of carbon fiber strand is equipped on the scaffold.

Carbon fiber sheet for longitudinal reinforcement was UD tape, in which content of carbon fiber was 175 g/m^2 (net thickness = 0.97 mm), and 2 plys of this UD tape were glued. For hoop reinforcement, carbon fiber strand (12000 monofilaments, net area = 0.46 mm^2) was wound at 5 mm pitch. Total weight of carbon fiber used for this work was approximately 5.5 kg.

Working procedure was as mentioned before (cf. 4.2 and see Photo.4). Time taken for each work was as follows ;

1) arrangement and substrate treatment : 3 days
2) glueing UD tape : 4 days
3) winding carbon fiber strand : 3 days
4) finishing (painting) : 3 days
5) total : 13 days

4.5 Concluding Remarks

A new method for retrofit of existing reinforced concrete chimneys by glueing carbon fiber UD tape onto the concrete surface has been developed. A bending test of circular hollow reinforced concrete beams and a test on practical retrofit work are carried out. It is found that this retrofitting technique has good performance on seismic capacity and practical work.

Major findings of the bending test are ;

(1) Maximum flexural carrying capacity can be increased by longitudinally glueing carbon fiber sheet and this effect is more prominent as the amount of glued carbon fiber increases.
(2) Displacement of the specimen with cloth type carbon fiber is larger than that with UD tape.
(3) Carbon fiber glued on the compressive side of the spcimen carrys compressive stress with concrete up to crushing of concrete. Carbon fiber glued on the tensile side carrys tensile stress with longitudinal reinforcement bars.

5. Retrofitting of Existing Bridge Column [12]

5.1 Introduction

Though existing bridge columns of expressways in Japan have not been subjected to heavy damage, it is pointed out that during an extreamly severe earthquake shock, some of these colums may be damaged so that the function of expressways cannot be maintained. Re-evaluations and structural experiments of a typical existing bridge column, which can be considered as a cantilever, have revealed that (see Fig.20 (a)) ;
(1) since about 1/2 of longitudinal reinforcing bars are cut off at 1/3 of the total height, flexural yielding occurs at the cut off point ;
(2) since existing transverse reinforcement is very small (pw = 0.04%, pw:transverse reinforcement ratio), high ductility cannot be expected though the shear span ratio is larger than that of columns in buildings.

Ordinary retrofitting methods are the next two ;
(1) Encasing with reinforced concrete
Around the existing column, longitudinal and transverse reinforcement bars are arranged and concrete is placed. The thickness of the additional part is 100 to 150 mm because increasing weight should be lessened.
(2) Encasing with steel tube
Around the existing column, steel tube is constructed by welding steel plates, and epoxy resin is pumped into the gap of roughly 3 mm thickness between the steel and the existing concrete.

In these two method, placing concrete or pumping resin is the most difficult work.

Applying the techniques mentioned before (cf. chapter 3. and 4.), the anthors have developed a new retrofitting method with carbon fiber (see Fig.19). In this chapter, retrofitting details are described.

5.2 Retrofitting Method

The newly developed method is an application of two techniques presented before, that is, flexural strengthening (cf. chapter 4) and ductility improvement (cf. chapter 3).
(1) flexural strengthening (see Fig.20(b))
By longitudinally glueing carbon fiber UD tape on the concrete surface near the height where the existing longitudinal reinforcement bars are cut off, sufficient bending carrying capacity at the cut off point is provided. Thus the flexural yielding at the bottom would occur before the flexural yielding at the cut off point ; and
1) strength is (slightly) improved ;
2) ductility is (slightly) improved because demanded plastic hinge rotation is reduced due to lengthened shear span.
(2) ductility improvement (see Fig.20(c))
By winding spirally carbon fiber onto the column surface of the bottom

and the flexurally strengthened zone, ductility is improved. The reasons are that ;

1) Though the existing transverse reinforcement is too small, good ductility of the plastic hinge region, i.e. the bottom of the column, is obtained by increasing transverse reinforcement.
2) Yielding of existing longitudinal reinforcement and crushing of concrete should be allowed to some extent at the cut off point, thereby confinement of concrete by transverse reinforcement is required in order to maintain stress transfer between concrete and carbon fiber UD tape.

The amount of carbon fiber UD tape and strand is determined according to the discussion mentioned before.

A retrofitting procedure is as follow ;

1) arrangement and substrate treatment of concrete surface
2) glueing UD tape near the cut off point
3) winding carbon fiber strand onto the bottom and the cut off point
4) finishing.

5.3 Conclusion and Vision

A retrofitting method for existing reinforced bridge columns of expressways is presented, i.e. ;

1) glueing carbon fiber UD tape : flexural strengthening of the cut off point of longitudinal bars
2) winding carbon fiber strand : transverse reinforcement of the cut off point and the bottom of the column

We are planning and conducting a test [12], employing specimens shown in Fig.21 and loading apparatus shown in Fig.22. We will obtain from this test,

1) effectiveness of this retrofitting method
2) the adequate area for glueing
3) the adeguate amount of transverse reinfocement

6. Conclusions

In this paper, carbon fiber itself is discussed first ;

1) outline (background, type of products, mechanical properties)
2) application (impregnation, cure, beveling surface, temperature)

Next, applications of retrofitting method with carbon fiber are presented.

1) column in buildings ; for improvement of ductility and shear strength ; by winding spirally carbon fiber strand onto the column surface.
2) chimney ; for improvement of strength ; by glueing carbon fiber UD tape on the outside surface ; if necessary, winding carbon fiber strand in the hoop direction.
3) bridge column ; for improvement of strength and ductility ; by glueing carbon fiber UD tape on the surface near the cut off point of existing longitudinal reinforcing bar, and by winding carbon fiber strand on the cut off point and the bottom of the column.

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In the list, the next abbreviations are used.

JCI = Japan Concrete Institute
AIJ = Architectural Institute of Japan
WCEE = World Conference on Earthquake Engineering
IAEE = International Association for Earthquake Engineering

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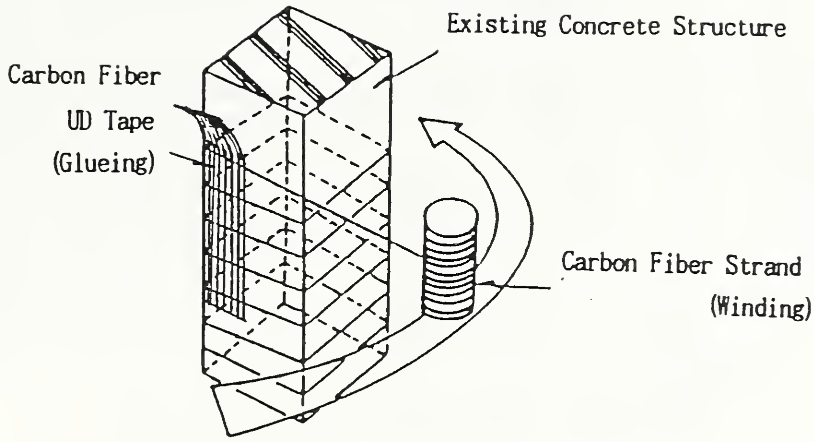


Fig. 1 Retrofit Method with Carbon Fiber

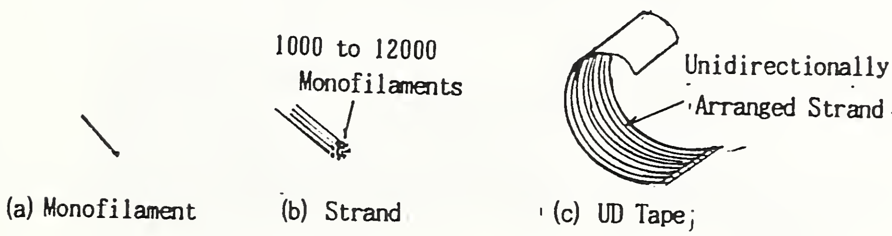


Fig. 2 Carbon Fiber Products

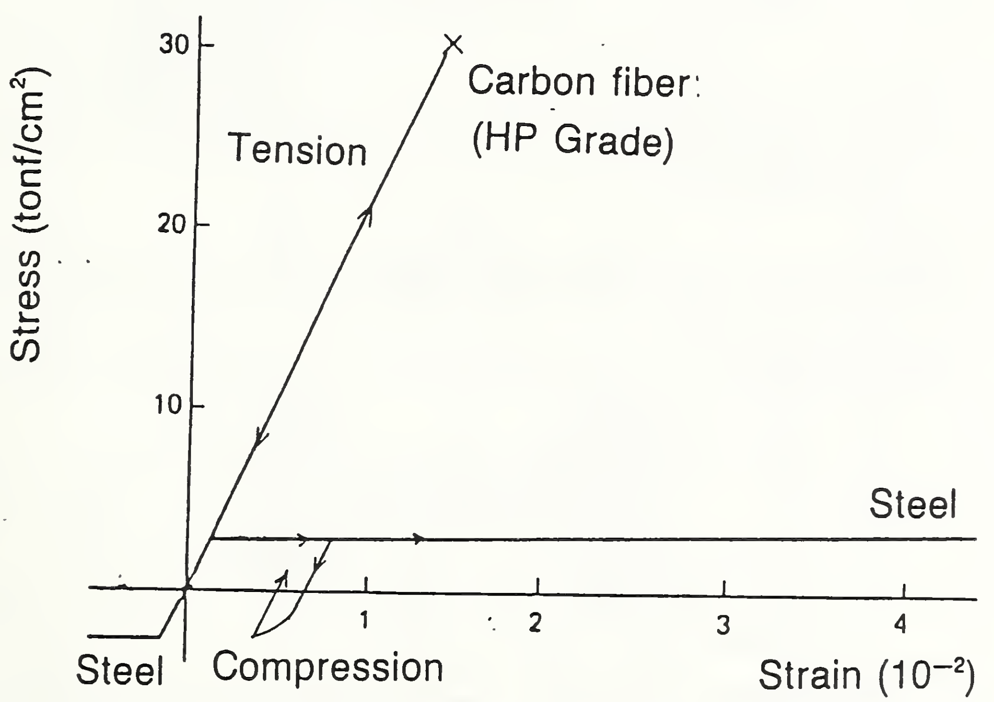


Fig. 3 Stress Strain Relationship

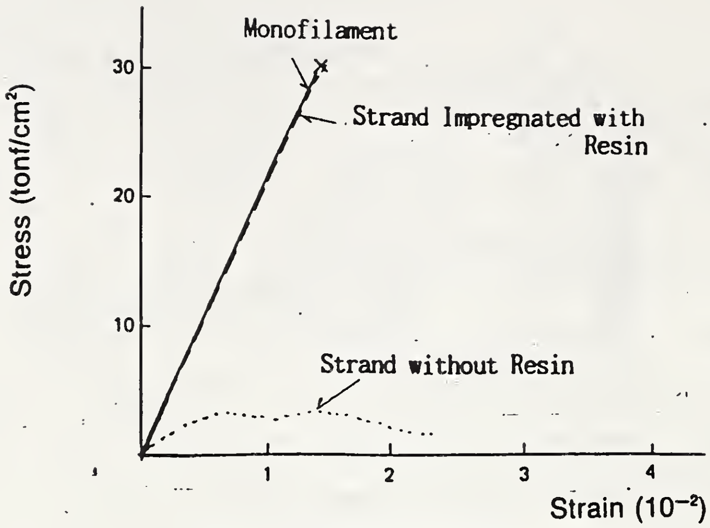


Fig. 4 Impregnated Effect

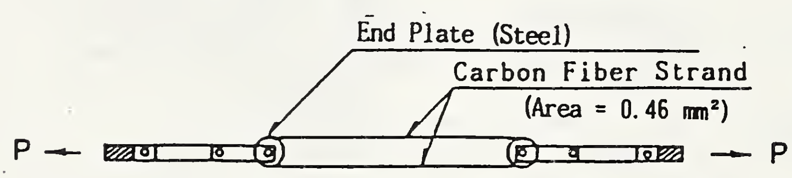


Fig. 5 Test Piece for Tensile Test

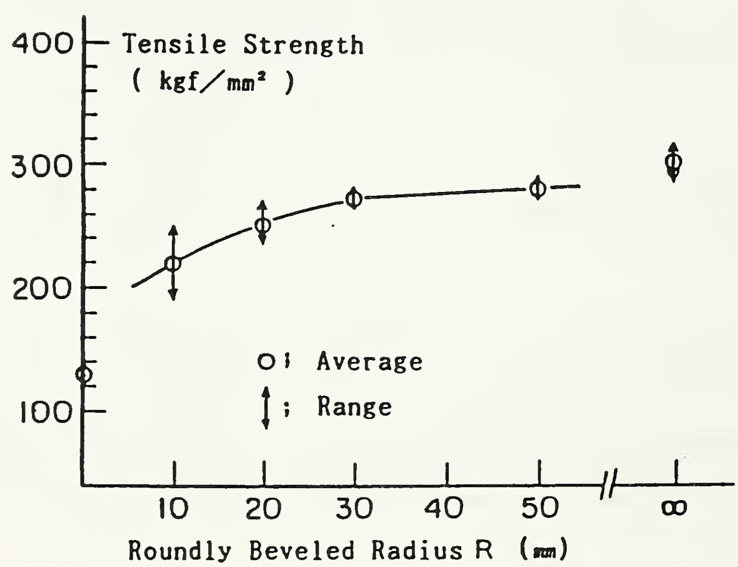


Fig. 6 Influence of Beveleing on Tensile Strength

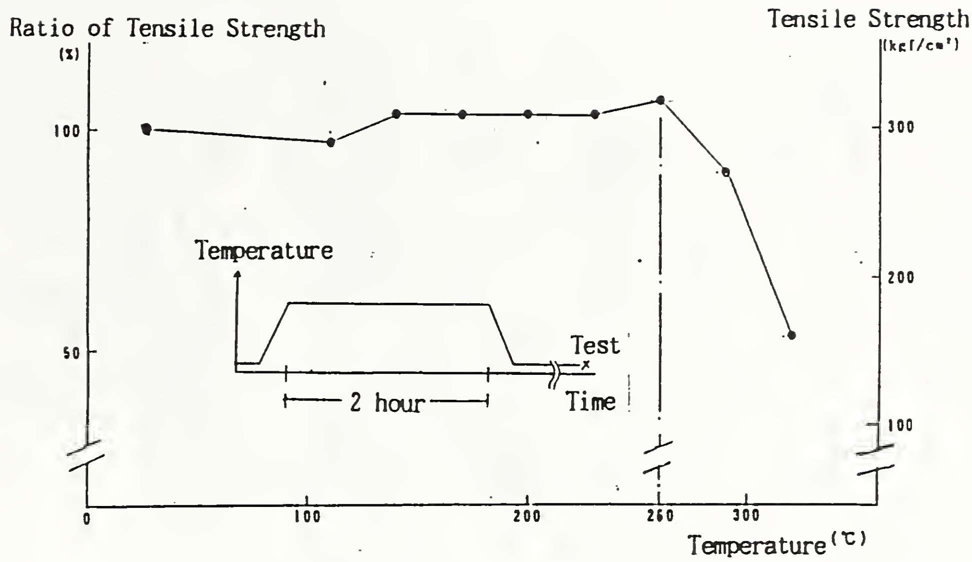


Fig. 7 Influence of Temperature on Tensile Strength

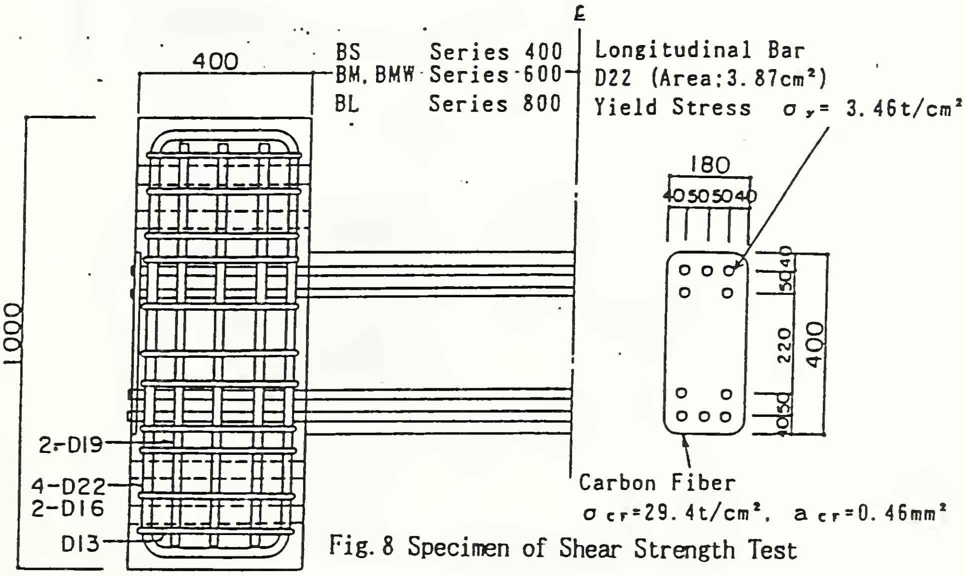


Fig. 8 Specimen of Shear Strength Test

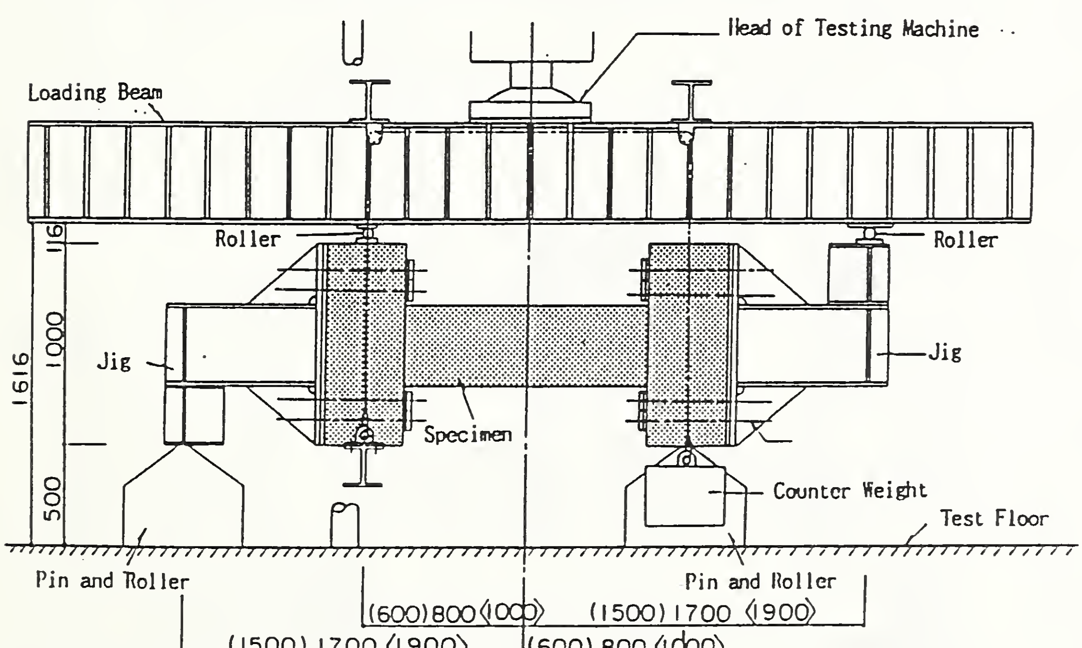


Fig. 9 Loading Apparatus of Shear Strength Test

no mark for the BM and BMW series
( ) for the BS series
⟨ ⟩ for the BL series

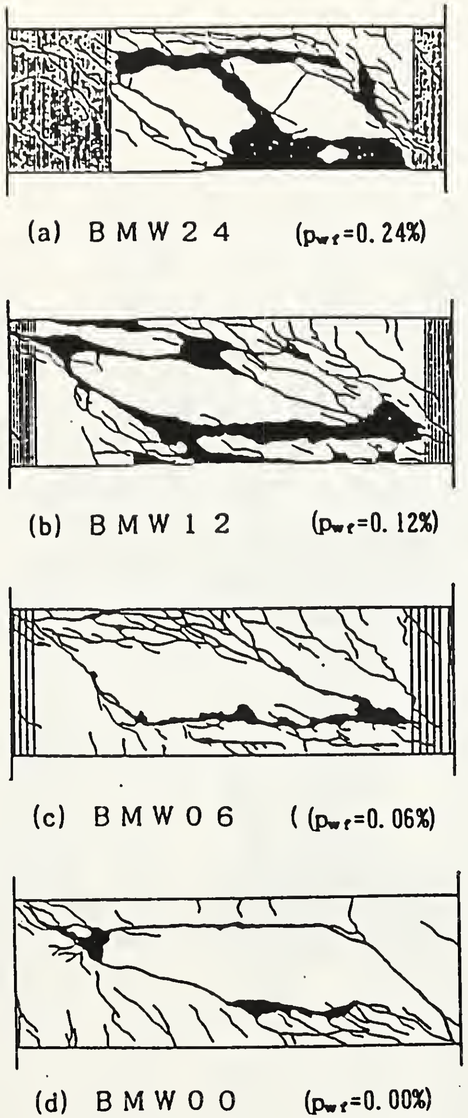


Fig. 10 Crack Patterns

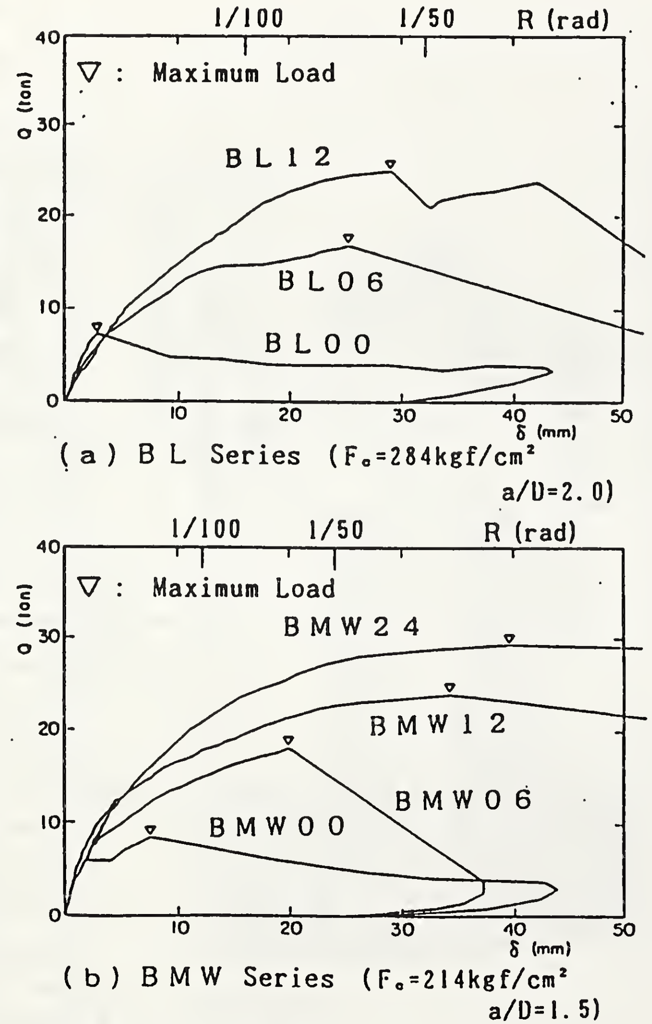
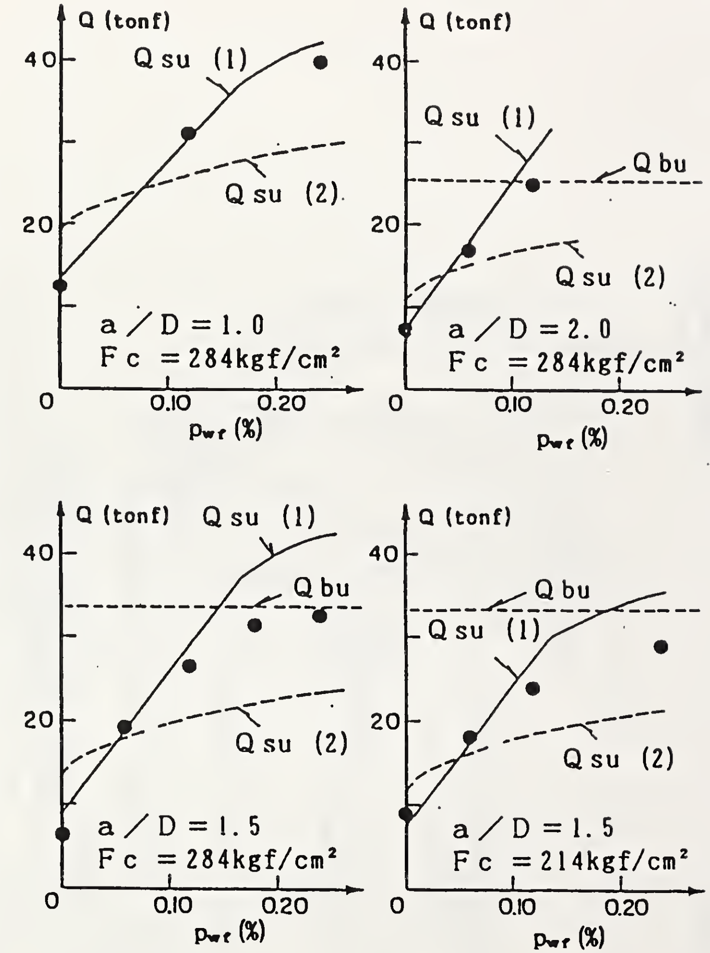


Fig. 11 Relationship of Shear Force vs. Displacement



● : Experimental Maximum Shear Force
Qbu : Calculated Bending Strength
Qsu(1) : Calculated Shear Strength according to Ref.9
Qsu(2) : Calculated Strength according to Ref.10

Fig. 12 Evaluation of Shear Strength

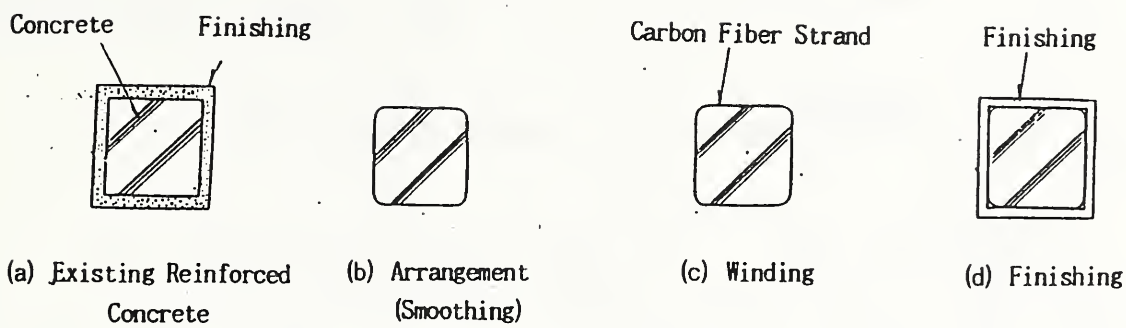


Fig. 13 Procudure of Retrofit Work

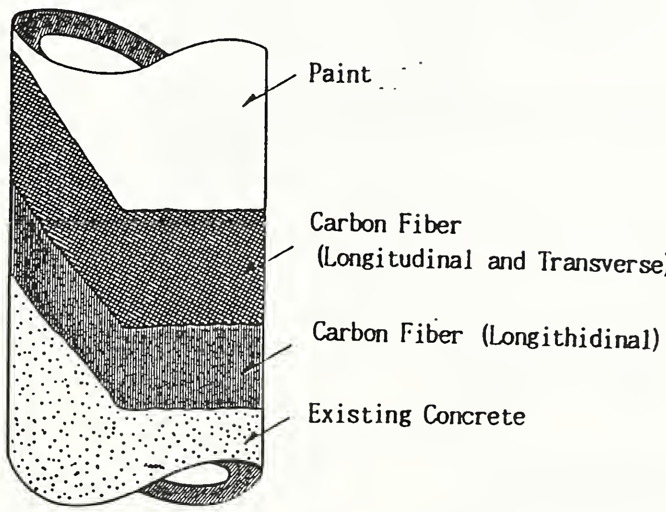


Fig. 14 Retrofit Method for Existing Chimneys

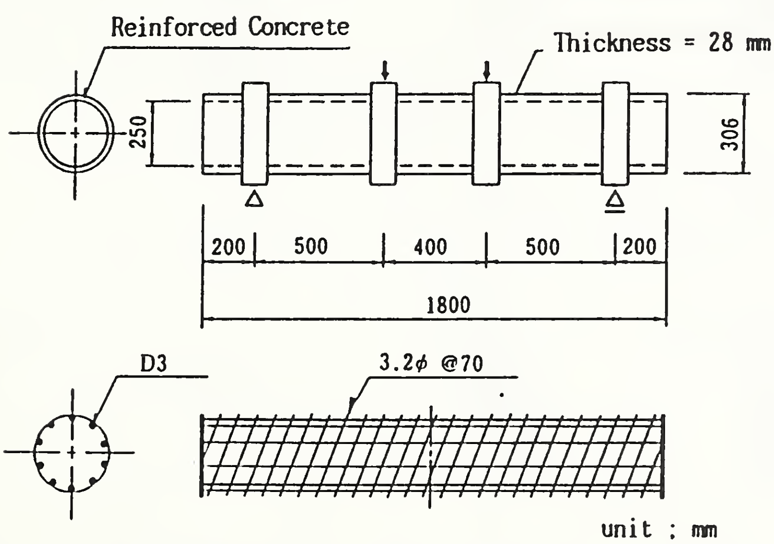
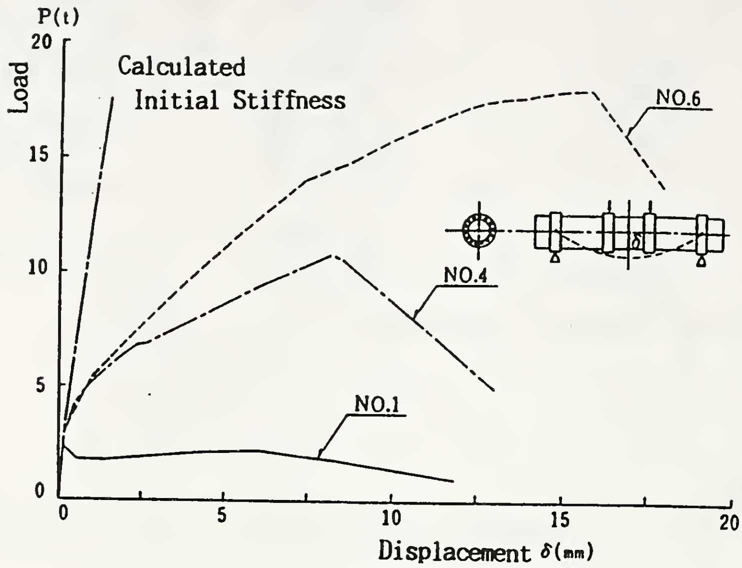
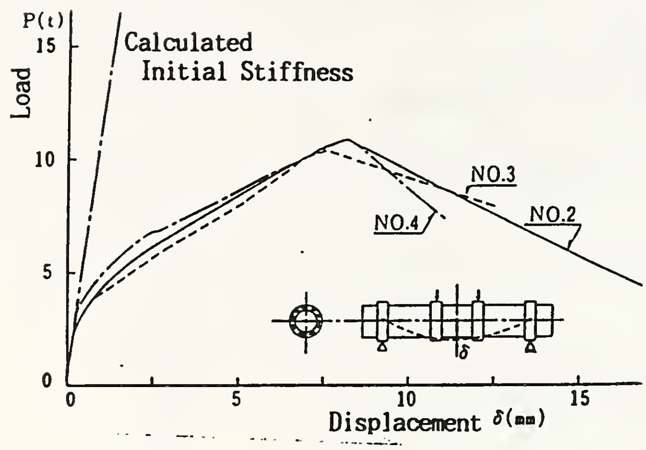


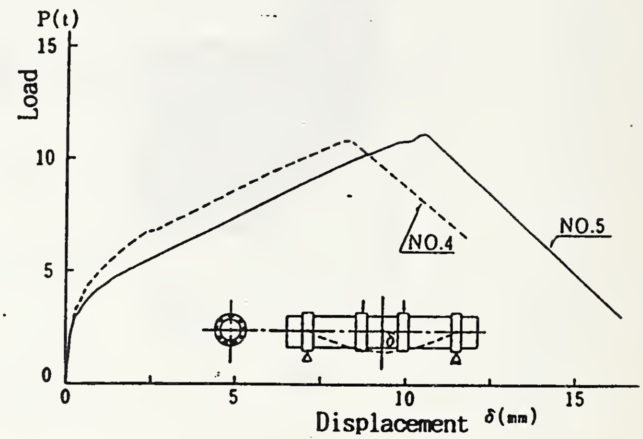
Fig. 15 Specimen of Bending Strength Test



(a) Influence of the Retrofitting Amount

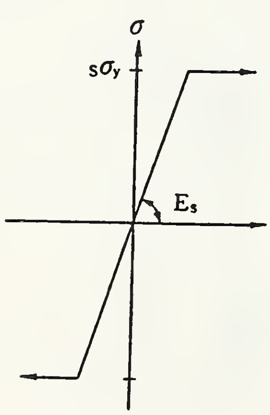


(b) Influence of Transverse Reinforcement

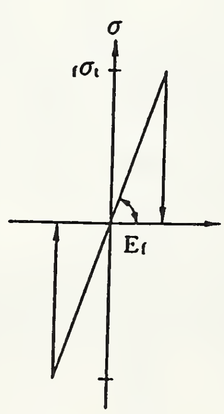


(c) Influence of Product Type

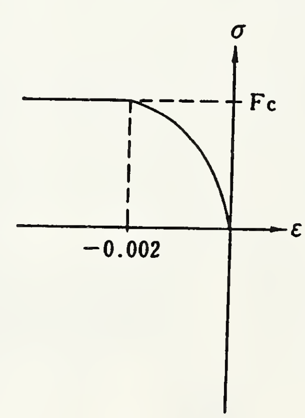
Fig. 16 Load-Displacement Relationship



(a) Steel Bar



(b) Carbon Fiber



(c) Concrete

Fig. 17 Assumption of Stress-Strain Relationship

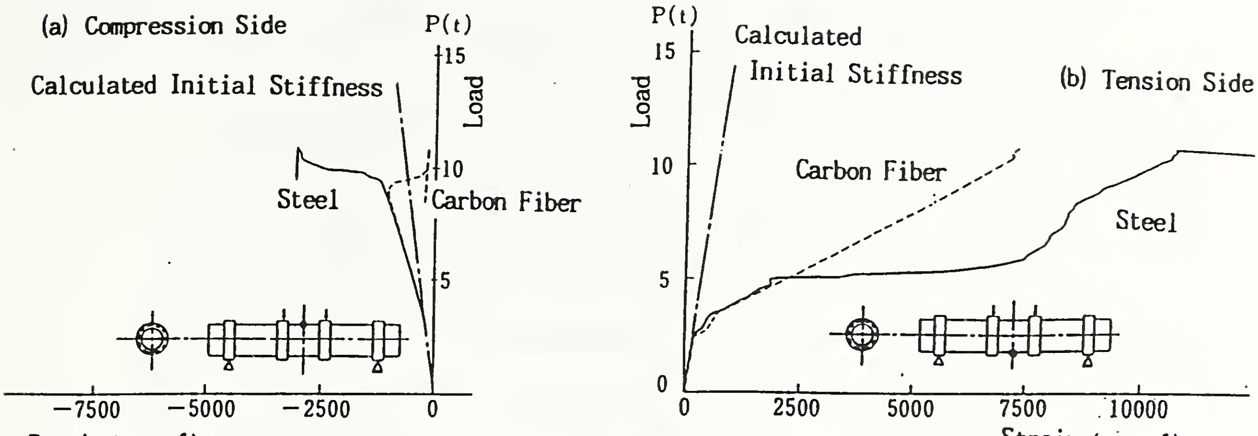


Fig. 18 Load-Strain Relationship (No. 2 Specimen)

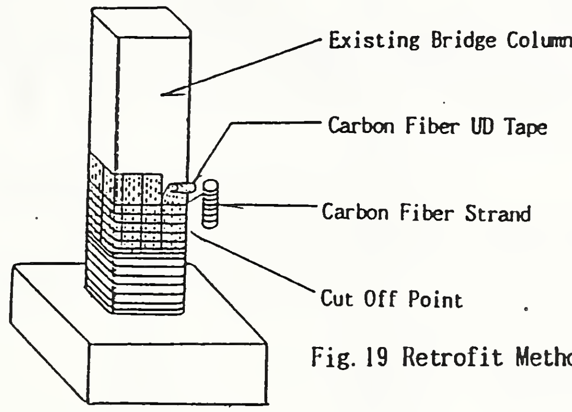


Fig. 19 Retrofit Method Bridge Column

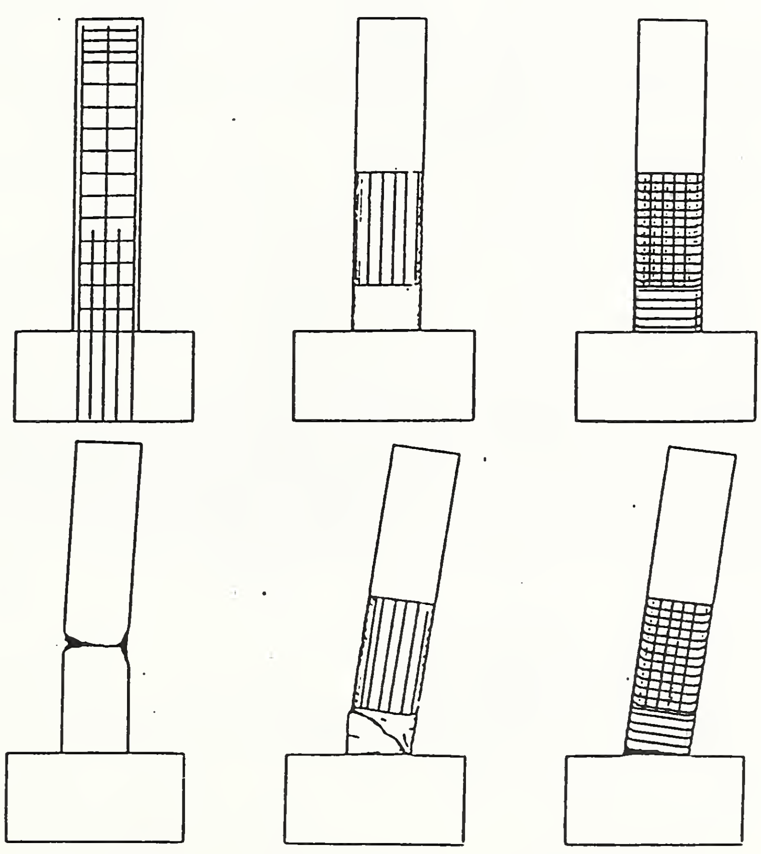


Fig. 20 Retrofitting and Failure Mode

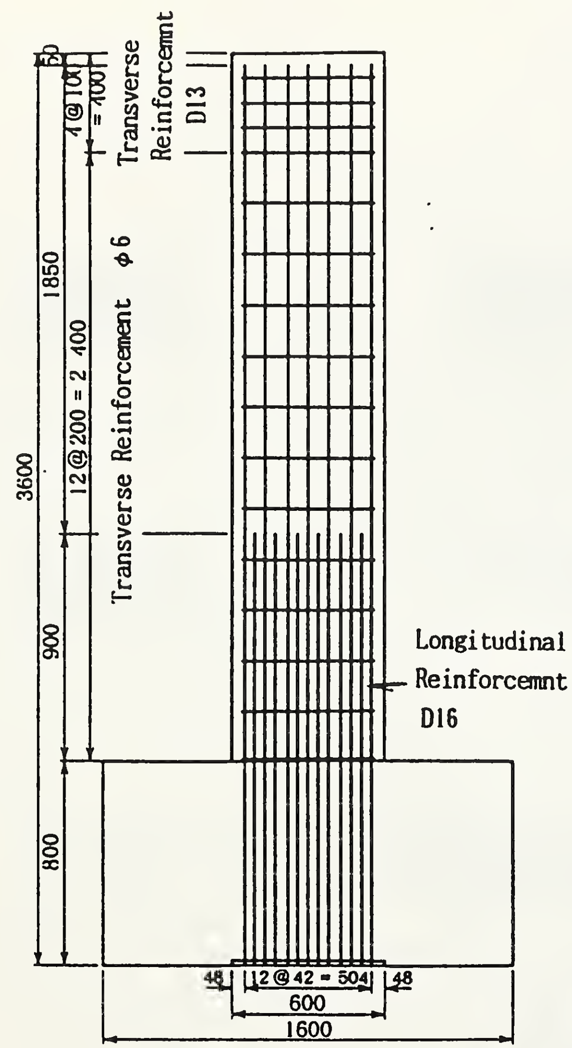


Fig. 21 Specimen of Bridge Column

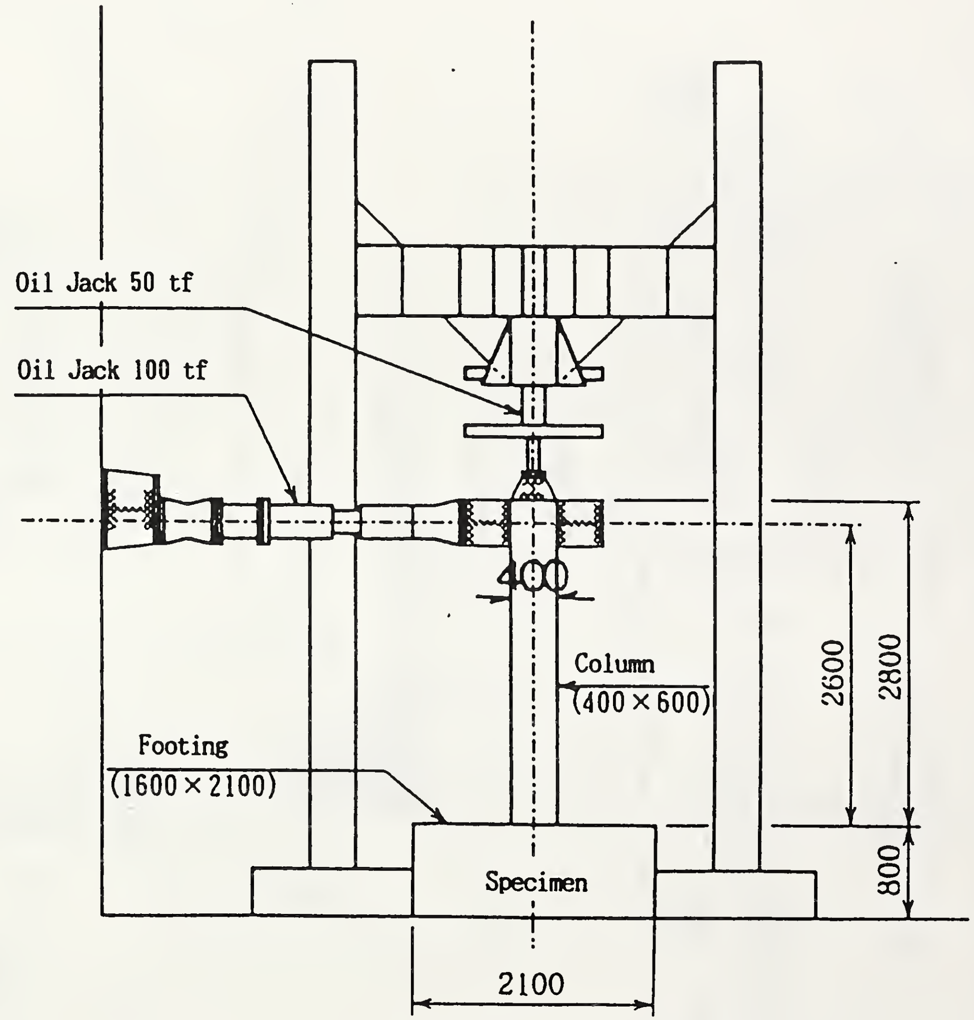


Fig. 22 Loading Apparatus of Bridge Column

Table. 1 Comparison of Material Properties

	CARBON FIBER	MILD STEEL
Strength (kgf/mm^2)	300	50
Yield Strength (kgf/mm^2)		30
Young's Modulus (tf/mm^2)	24	21
Weight Density (g/cm^3)	1.8	7.9
Elongation (%)	1.2	20
Durability	Good*	No Good**

* Stable against any chemical attack
Impregnated carbon fiber is almost stable because impregnating resin is usually strong for chemical attack.

** Rust easily occurs

Table. 2 Resin Classified by Curing

	Curing Temperature	Curing Time
Ordinary Resin	100 to 150 ℃	2 hours
LTC Resin	10 to 40 ℃	14 to 4 days
(Standard Condition)	20 ℃	7 days

Table. 3 Specimen and Test Result

Specimen	Parameter			Test Results			
	a/D	p_{wf} %	F_c $\frac{kgf}{cm^2}$	Q_{me} ton	τ_{me} $\frac{kgf}{cm^2}$	δ (Q_{me}) mm	R (Q_{me}) rad.
BS00	1.0	0.00	284	12.6	17.5	4.61	1/174
BS12		0.12		31.0	43.1	15.46	1/52
BS24		0.24		39.6	55.0	25.21	1/32
BM00	1.5	0.00	284	6.3	8.8	6.37	1/188
BM06		0.06		19.2	26.7	16.06	1/75
BM12		0.12		26.4	36.7	23.58	1/51
BM18		0.18		31.5	43.7	38.71	1/31
BM24		0.24		32.5	45.2	80.14	1/15
BL00	2.0	0.00	284	7.3	10.1	2.86	1/560
BL06		0.06		16.9	23.2	25.23	1/63
BL12		0.12		25.0	34.7	42.18	1/38
BMW00	1.5	0.00	214	8.8	12.2	7.55	1/159
BMW06		0.06		18.1	25.2	19.90	1/60
BMW12		0.12		23.9	33.2	34.32	1/35
BMW24		0.24		29.3	40.7	39.64	1/30

a : Shear Span
$a = L/2$
L : Clear Length
D : Depth
Fc : Compressive Strength of Concrete

p_{wf} : $2a_f / (b \cdot x)$
a_f : Area of Carbon Fiber Strand
b : Width
x : Spiral Pitch

Q_{me} : Experimental Maximum Shear Force
τ_{me} : Q_{me}/bD
δ (Q_{me}) : Displacement at $Q=Q_{me}$
R (Q_{me}) : δ (Q_{me}) $/L$

Table. 4 Material Properties of Shear Strength Test

Longitudinal Steel Bar (D22, Area = 3.87 cm^2)	Yield Strength Tensile Strength	= 3460 kgf/cm^2 = 5320 kgf/cm^2
Carbon Fiber Strand (Area = 0.0046 cm^2)	Tensile Strength Elastic Modulus	= 29.4 tf/cm^2 = 2360 tf/cm^2
Concrete BM, BS, and BL series BMW series	 Compressive Strength Compressive Strength	 = 284 kgf/cm^2 = 214 kgf/cm^2

Table. 5 Equations for Shear Strength Calculation

Qsu(1) ; Calculated Shear Strength according to Ref. 9

$$= b \cdot j_t \cdot p_w \cdot \sigma_{wy} \cdot \cot\phi + \tan\theta \cdot (1-\beta) \cdot b \cdot D \cdot \nu \cdot F_c / 2 \quad \cdots \text{Eq. 1}$$

j_t ; Distance between Tensile and Compressive Longitudinal Bar

$\cot\phi$; Direction of Concrete Strut for Truss Action

ν ; Effective Strength Coefficient of Concrete

Qsu(2) ; Calculated Strength according to Ref. 10

$$= \left\{ \frac{0.068 p_t^{0.23} (180+F_c)}{M/(Q \cdot d) + 0.12} + 2.7\sqrt{p_w \cdot \sigma_{wy}} \right\} \cdot b \cdot j \quad \cdots \text{Eq. 2}$$

p_t ; Percentage of Longitudinal Bar

j ; Distance between Tensile and Compressive Stress Center

$M/(Q \cdot d)$; Shear Span Ratio

Table. 6 Specimen and Parameters of Bending Test

Specimen	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Longitudinal Fiber Ratio (%) *	0.00	0.40				0.81
Transverse Fiber Ratio (%) *	0.00		0.33	0.77		
Type of Cabon Fiber Sheet		UD Tape			Cloth	UD Tape
Longitudinal Reinforcing Bar Ratio (%)	0.30					

* Longitudinal and transverse fiber ratio is defined as ordinary longitudnal and transverse reinforcement ratio, respectively.

Table. 7 Material Properties of Bending Test

	Yield Strength	Tensile Strength	Young's Modulus
Steel Bar	2.89	4.46	2180
Carbon Fiber		30.30	2450

	Compressive Strength	Young's Modulus
Concrete	0.17	169

unit ; tf/cm^2

Table. 8 Carrying Load

Specimen	Flexural Cracking			Maximum (Ultimate for No. 1)			
	(a) P^E (tf)	(b) P^A (tf)	(a)/(b)	(c) P^E (tf)	(d) P^A (tf)	(c)/(d)	α
No. 1	2.30	1.85	1.24	2.17	1.81	1.20	
No. 2	2.40	1.89	1.27	10.83	12.04	0.90	0.85
No. 3	2.51		1.33	10.35		0.86	0.80
No. 4	3.30		1.75	10.82		0.90	0.84
No. 5	2.56		1.35	11.15		0.93	0.88
No. 6	2.90	1.93	1.50	18.00	21.12	0.85	0.82

P^E and P^A : experimental and analytical value, respectively

α : effective ratio of carbon fiber

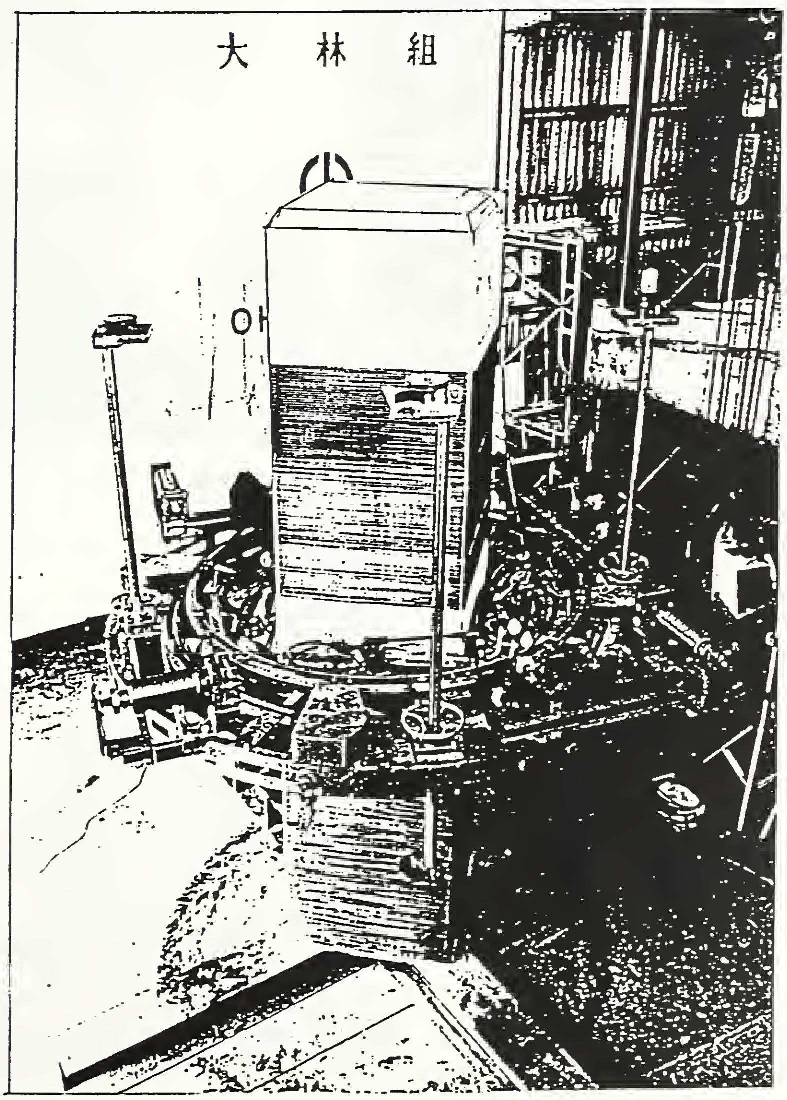


Photo. 1 Winding Machine

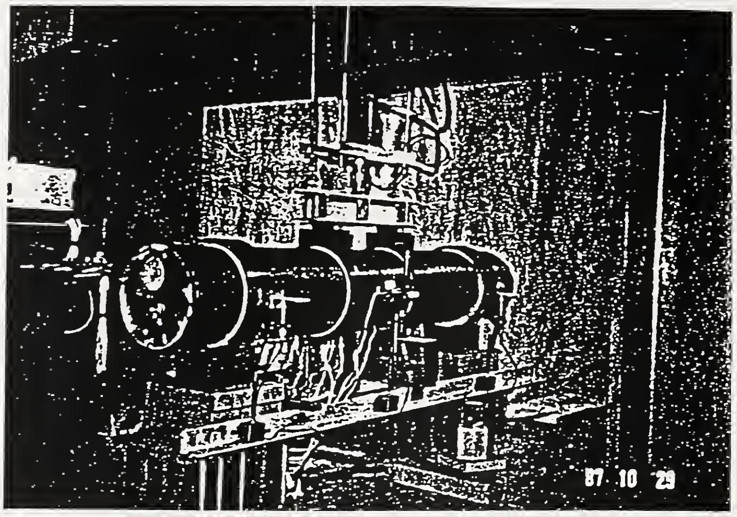


Photo. 2 Loading Apparatus

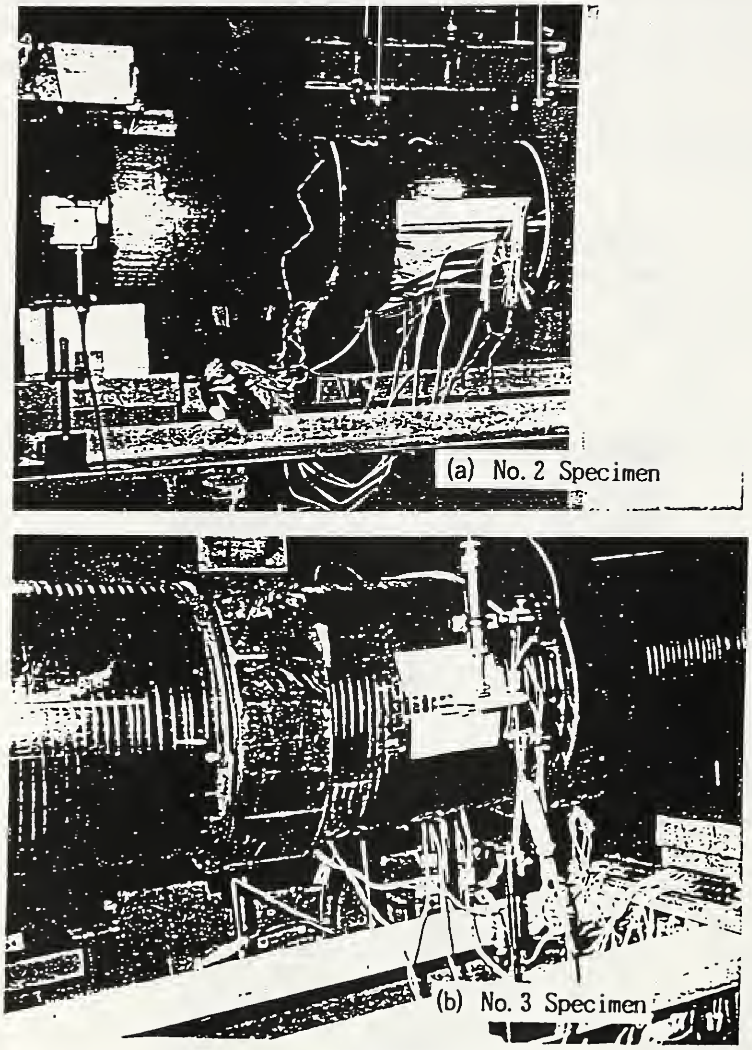
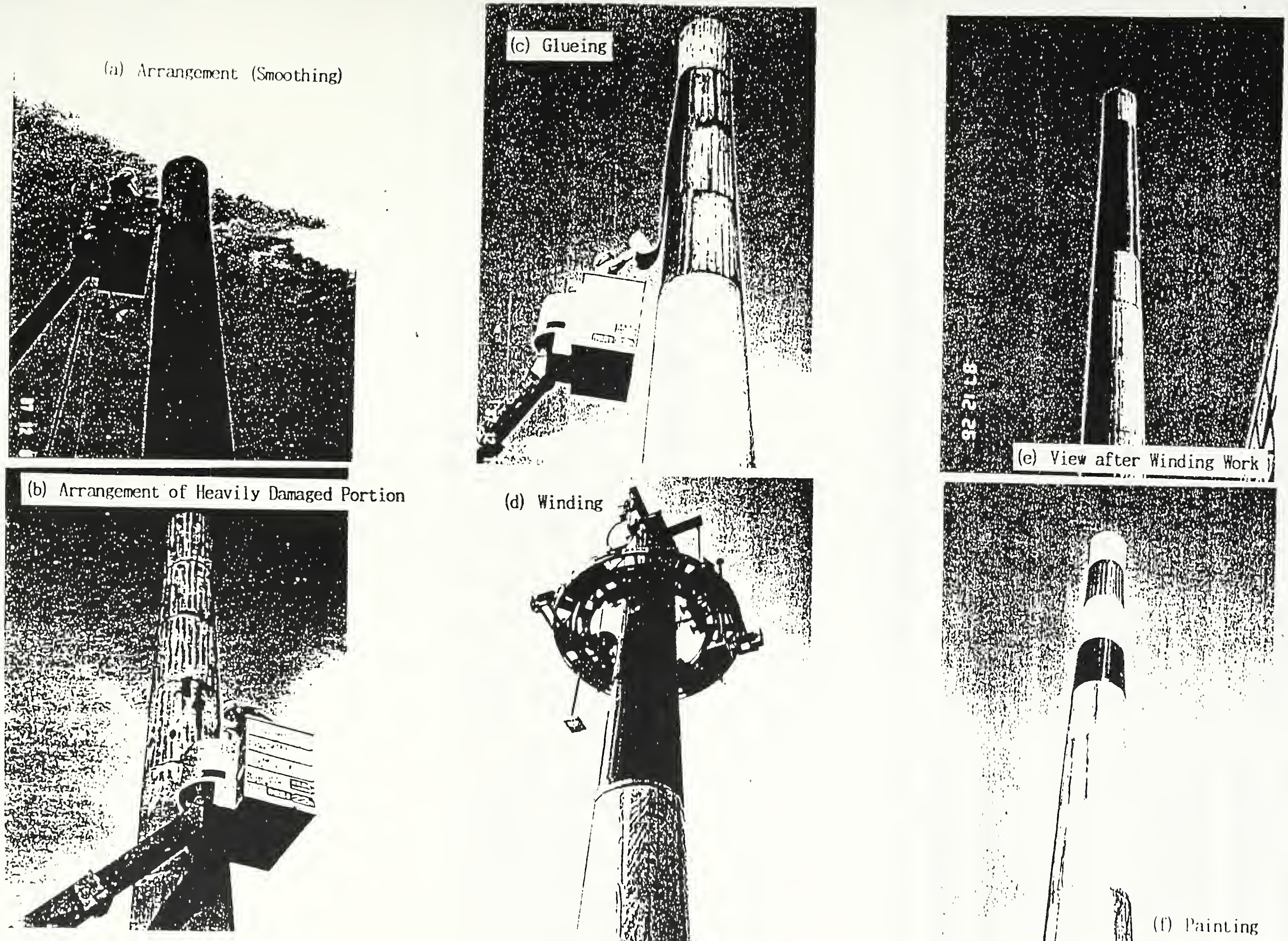


Photo. 3 Failure Pattern



(a) Arrangement (Smoothing)

(b) Arrangement of Heavily Damaged Portion

(c) Glueing

(d) Winding

(e) View after Winding Work

(f) Painting

Photo. 4 Retrofit Work for Chimneys

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11. ABSTRACT (A 200-WORD OR LESS FACTUAL SUMMARY OF MOST SIGNIFICANT INFORMATION. IF DOCUMENT INCLUDES A SIGNIFICANT BIBLIOGRAPHY OR LITERATURE SURVEY, MENTION IT HERE.)

This report is the Proceedings of an international workshop on Evaluation, Repair, and Retrofit of Structures. The workshop was a joint effort of Task Committees C and D, "Repair and Retrofit of Existing Structures" and "Evaluation of Structural Performance" respectively of the U.S.-JAPAN Panel on Wind and Seismic Effects. The workshop was hosted by the National Institute of Standards and Technology during May 12-14, 1990. The National Science Foundation provided the financial support. The subjects addressed included: evaluation of structures; performance of existing structures--their repair and strengthening; and research on techniques for repairing and retrofitting structures.

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evaluation; performance; repair; retrofit; strengthening; structures

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