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Proceedings of the 4th U.S.–Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems

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Proceedings of the 4th U.S.–Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems

Held at the Biltmore Hotel
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Edited by

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TABLE OF CONTENTS

FOREWORD

The future of lifeline earthquake engineering depends, in large part, upon our ability to develop and communicate new concepts and ideas for design, construction, operation, and post-event restoration of lifelines. For the past several decades, the governments of Japan and the United States have shared the belief that the exchange of information and resources is integral in improving the seismic performance of lifeline systems in both countries. Both countries have supported opportunities to meet in technical meetings to discuss the development of technologies to mitigate seismic effects on lifeline systems. One ongoing activity has been to meet under the auspices of the U.S.-Japan Natural Resources (UJNR) program.

To date, there have been three previous workshops organized by the Panel on Wind and Seismic Effects of the UJNR program with specific focus on the seismic performance of lifelines:

- U.S.-Japan Workshop on Lifeline Systems, Washington, D.C, 1984
- Second U.S.-Japan Workshop on Seismic Behavior of Buried Pipelines and Telecommuncation Systems, Tsukuba, Japan, 1984
- Third U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Tsukuba, Japan, 1989

The following proceedings document the results of the Fourth U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems. This workshop was jointly sponsored by the National Science Foundation (NSF) of the United States, and by the Public Works Research Institute (PWRI) of Japan. Additional financial support for this workshop was provided by the National Institute of Standards and Technology (NIST) and the Metropolitan Water District of Southern California. The workshop was held at the Biltmore Hotel in Los Angeles, California, between August 19-21, 1991, and was organized by Task Committee F, "Disaster Prevention Methods for Lifeline Systems," UJNR Panel on Wind and Seismic Effects. Mr. Ronald T. Eguchi of Dames & Moore, Mr. Robert Dikkers of NIST, Professor Henry Lagorio and Dr. S.C. Liu of NSF, and Drs. Y. Sasaki and K. Kawashima of PWRI led the organization of this workshop.

The theme of this workshop focused on "Future Directions for Research, Application and Design of Lifeline Systems." Developing this theme required an examination of past research activities, an assessment of current research efforts, and recommendations for future directions for seismic vulnerability assessment and design, and emergency response and planning. It is intended that collectively, these papers will provide the basis for establishing priorities for future joint activities between both countries.

In this workshop, a broad range of technical topics were discussed. These included:

- Effects of Soils on Lifeline Components;
- Seismic Design and Retrofit of Lifeline Facilities;
- Dynamic Response and Analysis of Lifeline Facilities;
- Repair and Rehabilitation of Lifeline Systems;
- System Reliability Methods for Lifeline Systems;
- Post-Earthquake Damage Detection Procedures;
- Socio-Economic and Environmental Impact of Lifeline System Failures;
- Emergency and Disaster Response Management of Lifeline Systems.

In total, thirty papers were presented in the two days of plenary sessions; 16 papers from Japan and 14 papers from the U.S.

In addition, the workshop was highlighted by:

- Active Question and Answer periods after each session;

- A full-day field trip to the Ralph's Grocery Company Automated Warehouse Distribution Center, the Metropolitan Water District of Southern California's Joseph Jensen Filtration Plant and several CALTRAN's facilities; and

- Resolutions fully supported by the participants of both countries that identify future research priorities and cooperative activities between the two countries.

The materials in this proceedings are presented as follows: (1) all technical papers that were presented during the workshop are provided in their order of presentation; (2) resolutions that were unanimously adopted by the participants during the closing session of the workshop, and (3) appendices that include the workshop program, list of participants, and photos to commemorate the workshop.

ACKNOWLEDGMENTS

The organizers of the workshop would like to thank the National Science Foundation and the Public Works Research Institute for sponsoring the workshop and its activities. In addition, the organizers would like to acknowledge the Metropolitan Water District of Southern California for sponsoring dinner activities, and the National Institute of Standards and Technology for its support in the printing of this document.

At the National Science Foundation, we would like to thank Professor Henry J. Lagorio and Dr. S.C. Liu for their guidance and support in this effort.

We extend our sincere appreciation and thanks to the members of Dames & Moore who helped to organize the workshop and who assisted in the actual meeting of the workshop participants. In particular, we would like to acknowledge the efforts of Ms. Julee Mon of Dames & Moore who worked many extended hours to insure the success of the workshop. Special thanks are also extended to Ms. Susan D. Pelmulder, Ms. Mary Jean Koontz, and Ms. Carmela Valerio, all members of Dames & Moore.

Ronald T. Eguchi
Dames & Moore

Kazuhiko Kawashima
Public Works Research Institute

Workshop Organizers

I. SOIL EFFECTS

"Performance of Water Supply Pipelines in Liquified Soil"
K.A. Porter, C.R. Scawthorn, D.G. Honegger, T.D. O'Rourke, F. Blackburn

"A Proposal of Practical Earthquake Response Analysis Method of Cylindrical Tunnels in Soft Ground"
Y. Shiba, S. Okamoto

"Visual Information System for Seismic Ground Hazard Zoning"
K. Tokida, H. Matsumoto, Y. Sasaki

PERFORMANCE OF WATER SUPPLY PIPELINES IN LIQUEFIED SOIL

K.A. Porter, C. Scawthorn, D.G. Honegger[1], T.D. O'Rourke[2], F. Blackburn[3]

ABSTRACT

Reliable performance of underground water supply pipe in earthquakes is a matter of critical importance, to support immediate firefighting and assure potable water supply. Existing approaches to estimating water pipe breakage, based primarily on estimated MMI, may underestimate major pipe damage concentrated in regions of large liquefaction-induced permanent ground deformations. A recent study (*San Francisco Liquefaction Study*) necessitated estimation of pipe breakage in areas of potential large permanent ground deformations associated with liquefaction and lateral spreading. Data from the 1906 San Francisco and 1989 Loma Prieta earthquakes were used to correlate pipe break as a function of permanent ground deformation. The results tend to agree with previous studies from the 1971 San Fernando and several Japanese earthquakes. The major finding is that normalized pipe break rate is nonlinear with permanent ground displacements. Relatively small displacements cause significant initial pipe breakage; at larger displacements additional breaks occur, but at a relatively smaller rate.

1 INTRODUCTION

The vulnerability of buried water supply pipe due to earthquake is of critical significance, both for post-earthquake fire as well as for continued potable water supply. It has long been recognized that, while earthquakes cause pipe breakage over wide areas, damage is especially concentrated in areas where liquefaction and lateral spreading have produced large permanent ground deformations (Schussler, 1906). A number of studies have examined water supply piping in earthquakes, but most have relied on gross regional performance, producing correlations with seismic intensity measures such as Modified Mercalli Intensity (MMI), or peak ground acceleration (PGA). While these measures may be adequate for overall estimates of regional performance, they tend to underestimate damage concentrated in regions subject to liquefaction and lateral spreading. Other studies have developed analysis techniques that provide failure criteria for pipe subjected to simplified models of ground displacements, well specified with regard to location and dimensions of ground deformation (eg, Eguchi, 1983a; O'Rourke et al, 1985). These

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methods generally cannot yet be used for water system vulnerability assessments, since currently feasible geotechnical engineering methods only permit estimation of area-wide trends of ground settlement or lateral spreading. A need therefore exists, where area-wide quantified trends of ground settlement or lateral spreading is feasible, but vulnerability measures of underground piping subjected to these deformations are lacking.

In an attempt to fill this need, this study addresses the estimation of damage to buried water pipe subjected to estimated (ie, quantified, although imprecisely) permanent ground displacements. This work grew out of a recent study ("San Francisco Liquefaction Study," Harding Lawson et al 1991) which necessitated the development of relationships for the estimation of water pipe damage in regions of potential ground failure. Specifically, this paper describes the development of correlations between gross water pipe break rate and general magnitude of permanent ground displacement, regardless of pipe orientation.

2 LITERATURE REVIEW

This section briefly reviews key literature on water pipe seismic vulnerability, as a function of seismic intensity, fault offset, ground movement and joint type.

2.1 WATER PIPE DAMAGE CORRELATED WITH GROUND SHAKING INTENSITY

Kubo and Isoyama (1980), as well as Katayama, Kubo, and Sato (1975) provided damage rates based on gross regional performance of pipe subjected primarily to JMA V (MMI VII-VIII) ground shaking. In the 1978 Miyagi-Ken-Oki Earthquake, and in the 1923 Kanto and 1968 Tokachi-oki earthquakes, failure ratios in JMA V ground shaking ranged between 0.012 and 0.379 failures per 1000 feet of pipe, depending on material. Cast iron pipe failure ratios ranged between 0.17 and 0.40 breaks per 1000 feet of pipe, not including "joint loosenings," whose definition Katayama et al could only conjecture.

Eguchi (1983b) summarized pipe break rate versus MMI for several United States earthquakes, developing a bilinear semilogarithmic curve for cast iron pipe subject to ground shaking effects only (Figure 1). He found that damage rates increase rapidly with MMI, and then more slowly as MMI exceeds VIII.

2.2 WATER PIPE DAMAGE CORRELATED WITH FAULT OFFSET

Duryea et al (1907) describe damage to cast and riveted iron pipeline crossing fault offsets in the 1906 earthquake. They indicate that cast and riveted iron pipe fractured wherever it crossed a fault offset of several feet. Photographs of the damage indicate that riveted iron pipe fractured at riveted joints, through shear of the rivets.

Eguchi (1981) presents damage to buried cast iron water pipe near faulting during the 1971 San Fernando Earthquake. He correlated damage with fault displacement and distance from the predominant line of fault rupture (Figure 2). Ground shaking effects accounted for relatively little breakage.

Eguchi also evaluated the effect of joint type on break rate in the Sylmar fault area. He found that breaks rates were similar for pipes with caulked joints and pipes with rubber gasket joints, and that the predominant modes of failure for pipe in these areas were joint failure, pipe ruptures, and splits. Cement-caulked joints had a tendency to shatter while pipes with rubber gasket couplings tended to pull apart.

2.3 PIPE DAMAGE CORRELATED WITH LIQUEFACTION-INDUCED GROUND MOVEMENT

Several authors provide data or correlations between pipe breakage and degree of permanent ground deformation. O'Rourke et al (1989) describe ground movement as well as pipe breakage in San Francisco's Sullivan Marsh and Mission Creek in the 1906 San Francisco earthquake. Pipe breakage data is taken from Schussler (1906), who provided a map showing water main breaks identified in the three months following the earthquake (Figure 3). Using photographs and other records, O'Rourke mapped ground movements in the Embarcadero and South of Market (e.g., Figure 4).

In later work, O'Rourke et al (1990) provided ground settlement contours in the Marina District in the 1989 Loma Prieta earthquake, and overlaid pipe breakage records on the settlement map (Figure 5). The data from 1906 San Francisco and 1989 Loma Prieta earthquakes are used in the present paper to develop break rate relations for water pipe in liquefied soil.

Hamada (1989) described Noshiro City gas pipe damage that occurred in liquefied soils in the 1983 Nihonkai-Chubu earthquake. Damage to cast iron gas pipe reached approximately 21 breaks per 1000 feet of pipe, where displacements were on the order of 2 feet.

Eguchi (1983b) provides a constant figure, 1.8 break per 1000 feet for cast iron pipe subjected to landslide, and 1.0 breaks per 1000 feet for cast iron pipe subjected to liquefaction and lurching, regardless of displacement.

2.4 AFFECT OF JOINT TYPE ON PIPE DAMAGEABILITY

Shaoping et al (1983) studied the contribution of joint damage to overall pipe damage. They point out that joints are the weakest points in segmented pipe, and that lead filled joints were superior to cement filled joints, since the latter are brittle and cannot sustain even small deformation. In the Tangshan Earthquake, where break rates in the mezoseismal area reached 10.0 per km (3.0 per 1000 ft), the rate of damaged joints was 79.6%. In Haicheng, approximately 60% of breaks were associated with joints.

3 PIPE TYPES AND BREAK DATA

3.1 SAN FRANCISCO WATER SYSTEMS AND PIPE TYPES

San Francisco possesses three water supply systems: two specifically for firefighting use and one for both fire fighting use and municipal use (potable water). The two firefighting systems are the truck-borne Portable Water Supply System (PWSS), and the underground Auxiliary Water Supply System (AWSS). Both are owned and operated by the San Francisco Fire Department (SFFD). The Municipal Water Supply System (MWSS) is owned and operated by the San Francisco Water Department (SFWD).

The two systems incorporate a variety of pipe materials and construction. The pipe can be roughly classified into 7 types: bell and spigot cast iron, double-spigot cast iron with heavy walls, ductile iron with rigid joints, ductile iron with flexible joints, riveted steel, gas welded steel, and arc welded steel.

Bell and Spigot Cast Iron Pipe

The majority of MWSS water mains are of this type. Until the 1930's, pipe was vertical pit cast iron, 12 feet long, typically exhibiting brittle mechanical properties with tensile strengths on the order of 15 ksi (Ahmed, 1990). Joints were sealed with an oakum gasket and packed with lead caulk. Restraint was typically provided at dead ends and branch installations. After 1930, centrifugally cast iron came into wide use in water systems. Also brittle, this material nonetheless exhibits strengths on the order of 35 ksi. The advantage of stronger material may be offset to some degree by the stiffer joint construction to which SFWD switched at around the same time. Around the 1930s, SFWD began to seal joints with a rubber gasket and dry mortar caulk.

Virtually all the water mains affected in the Marina District by the October 17, 1989 Loma Prieta earthquake were of the pre-1930 construction, as were all mains 24 inches in diameter or smaller in the 1906 San Francisco earthquake.

Double Spigot, Heavy Wall Cast Iron Pipe

This is the type of pipe used for the AWSS. Four characteristics tend to make AWSS cast iron pipe stronger and more flexible than MWSS cast iron pipe: double spigot joints, heavy walls, extra restraint, and fewer lateral connections. Each of these characteristics might be expected to reduce AWSS pipe vulnerability relative to cast iron water pipe described above.

Double spigot joints use cast iron or cast steel sleeves, sealed with lead and oakum, as shown in Figure 6. This type of construction allows twice as much joint rotation as does bell and spigot construction with a similar lead/oakum seal. The Schedule H pipe used in the AWSS has walls typically 60% thicker than those of the MWSS cast iron pipe. Since the AWSS operates at higher pressure than MWSS, extra restraint is employed. Rods connect as many as 10 pipe segments at turns, tee joints, hills and other points of likely stress, as opposed to the restraint of one joint in MWSS tees. Finally, since AWSS contains no service connections (it is a dedicated firefighting only system), laterals occur only at hydrants and branches. Laterals provide restraint and may act as concentrated load points on a water main, potentially resulting in additional pipe damage.

Ductile Iron Pipe with Rigid Joints

After about 1960, ductile iron was used for MWSS pipe of 16 inch or smaller diameter, with geometry and construction similar to the cast iron it replaced, but cast centrifugally in 18 foot lengths. SFWD continued to use gasket and grout joints in new ductile iron pipe construction until 1989.

Ductile Iron Pipe with Flexible Joints

Since 1989, SFWD pipe joints have employed U.S. Pipe's Field Lok Gasket, an elastomeric gasket fitted with mechanical teeth that provide longitudinal restraint (Figure 7). The gasket requires no packing, thereby allowing significant rotational flexibility.

Riveted and Welded Steel

Pre-1930 MWSS pipe larger than 24 inch diameter is of riveted iron or steel construction. Longitudinal joints were shop riveted; circumferential joints were riveted in the field. Gas welding came into use around 1930; during the 1940s, arc welding found widespread use. After about 1960, pipe larger than 20 inch diameter was of welded steel. Joints were bell and spigot with fillet welds at the lap joint. Pipe less than 24 or 30 inches in diameter received a single fillet weld on the outside. Pipe 30 or 36 inches in diameter or larger were welded inside and outside. Weld leg size was equal to pipe thickness.

3.2 1906 PIPE DAMAGE AND GROUND MOVEMENTS

Records of water main damage within San Francisco were not kept during the few months following the 1906 earthquake. The only source of data is a map supplied by Schussler (1906), who recorded 300 main breaks and over 23,000 service breaks by July 1906. Broken mains of all diameters from 2 or 3 inches up are indicated in Schussler's map. As Schussler noted, it is likely that this record is incomplete; many streets were still covered with debris at the time the map was prepared, and no doubt more pipe damage was uncovered as the restoration process continued. Pipe breakage data from the 1906 earthquake therefore must be employed with caution.

Where no breaks or occasional breaks appear on the map, no conclusion can be drawn, since breaks may have remained buried beneath rubble. More confidence may be placed in the completeness of the record for individual blocks that had numerous breaks. One may conclude that on these blocks, breaks were found by restoring service pressure over that block. Restoration of service pressure would have required identification and repair of most or all the breaks.

3.3 1989 PIPE DAMAGE AND GROUND MOVEMENTS

The 1989 Loma Prieta earthquake resulted in over 160 MWSS pipe breaks, approximately 3/4 of which were located in the Marina District. Breaks within the Marina were concentrated in the regions of hydraulic fill created circa 1906-1917. Locations of water main and service breaks are shown in Figure 8, which is based on SFWD records. In the next section, these will be correlated with liquefaction-induced ground settlements. Lateral spreading as large as 7 inches over 100 feet is known to have occurred in the Marina in 1989 (Harding Lawson Associates et al, 1991), but lateral movements have not been mapped.

4 ANALYSIS

4.1 BELL AND SPIGOT CAST IRON PIPE

In order to develop estimates of pipe breakage resulting from area-wide permanent ground deformation, break rates in San Francisco water mains was correlated with amount of ground settlement observed in the Marina in 1989, and lateral spreading observed in Sullivan Marsh and Mission Creek areas in 1906. Marina data was used primarily to develop break rate estimates at low permanent ground displacements; the 1906 data included displacements up to 6 feet.

Because records of settlement are available in the Marina, and records of lateral movement are available in the Sullivan Marsh and Mission Creek, our analysis equates vertical settlement with permanent lateral ground displacement. This was done despite it being generally known that lateral displacements of the same order of magnitude as vertical displacements are known to have occurred in the Marina (good lateral displacement data however are lacking). As a result, our analysis may underestimate total displacements in the Marina.

Low Displacement/1989 Marina Data Analysis

To develop a break rate relationship for low displacements, breaks within the area of the Marina that experienced settlement were aggregated to the nearest street intersection. The number of breaks at each intersection was then divided by the average length of pipe associated with an intersection. No differentiation was made for pipe diameter or pipe orientation relative to ground movement. The resulting break rate and settlement data per intersection were then used to correlate breaks per 1000 linear feet of pipe with permanent ground displacement (ie, settlement).

Large Displacement/1906 Data Analysis

Ground movements as large as nine feet occurred in several parts of San Francisco in the 1906 earthquake, causing widespread damage to pipe of the same construction as existed in the Marina in 1989. Five locations were selected and data taken from Schussler's map and O'Rourke's ground movement records of the 1906 earthquake (O'Rourke et al, 1990). Data were only drawn from locations where both the damage record and ground movement record appear to be complete. It should be noted that the assumptions regarding completeness of the pipe break record (described above) may tend to bias damage data toward atypically high break rates. The potential for this bias cannot be clarified without more complete information on 1906 pipe damage.

Regressions of the data were performed on both sets of data, and a combined set. The best fit was found as follows: (a) the 1989 data was treated as one set, and a regression was performed to fit a line with zero intercept over the domain 0 to 5 inches of displacement; (b) the 1906 data were treated as a separate set, and a regression was performed to produce a best fit line over the domain 5 inches to 9 feet; (c) a bilinear break rate curve resulted, as shown in Figure 9. A null hypothesis test indicates that a correlations exists with at least 95% confidence (Crow et al, 1960). **These correlations and resulting curve apply only to bell and spigot cast iron pipe with lead and oakum joints.**

4.2 VULNERABILITY OF OTHER PIPE TYPES

Detailed damage data is unavailable for the other six classes of pipe. To develop break rates for these other classes, the break rate for bell and spigot cast iron was multiplied by a "relative vulnerability" factor *R*. Relative vulnerabilities were determined based on a review of the literature, as well as engineering judgment.

4.3 DOUBLE SPIGOT, HEAVY WALL CAST IRON PIPE

Relative vulnerability *R* for this class of pipe was arrived at by multiplying factors to account for joint rotational capacity (J), wall thickness (W), extra restraint (B for bolts), and laterals (L):

$$R = J * W * B * L$$

The use of double spigot joints doubles joint rotation capacity, so J was assigned a value of 0.5. Since AWSS pipe walls are 8/5 as thick as those of bell and spigot MWSS cast iron, W was assigned a value of 0.625. It is difficult to assess that benefit of added restraint in AWSS pipe. The section properties of restrained joints are only slightly greater than those of unrestrained joints, so bolting is considered to add little overall pipe strength. B was therefore assigned a value of 1.0. L was assigned considering the damage associated with service connections in Marina District MWSS pipe in 1989. Approximately 25% of main breaks in the Marina were associated with services. Since the number of laterals in AWSS pipe is negligible next to that of MWSS, L was assigned a value of 0.75. *R* is therefore approximately 0.25.

4.4 DUCTILE IRON PIPE WITH RIGID JOINTS

This type of construction was used in San Francisco between about 1960 and 1989. It was assumed to be similar to pipe ductile iron pipe damaged in the in the Miyagi Prefecture in the 1978 Miyagi-Ken-Oki earthquake. The failure ratios reported by Kubo and Isoyama for ductile iron and cast iron water pipe were 0.04 and 0.17 failures per km, respectively. The ratio of these figures equates with *R*. For simplicity, *R* has been assigned a value of 0.25.

4.5 DUCTILE IRON PIPE WITH FLEXIBLE JOINTS

Hard data were unavailable on the relative performance of cement caulked joints as compared with lead/oakum joints. The ultimate compressive strength of the two materials is probably comparable, although the strains at ultimate vary greatly. Lead flow plastically while cement mortar will chip out as rotation occurs (ie, brittle failure). It is estimated that Field Lok Gaskets allow joints in bell and spigot ductile iron pipe to rotate approximately twice as much as similar pipe with mortar-caulked joints. Therefore, a value of 0.125 has been assigned to *R*.

4.6 GAS WELDED STEEL PIPE

Hamada (1983) compared cast iron pipe breakage with steel gas pipe breakage in the 1983 Nihonkai-Chubu earthquake, and concluded that the rate of damage to cast iron pipe was two to three times that in the case of steel pipe. Eguchi describes gas-welded steel pipe in faulting regions as 30% as vulnerable as cast iron. In landslide, the factor is 61%, and in liquefaction and lurching, 70%. Based on these observations, $R = 0.50$ was assigned.

4.7 RIVETED STEEL PIPE

Little data is readily available describing the performance of riveted steel pipe. Judging from photos presented by Duryea, it appears that the failure mode of riveted pipe is shear of the rivets. It is unlikely that significant local deformation can occur around rivet holes because these are most likely fabricated from iron boiler plate. We consider that riveted joints were designed to develop pipe body yield stresses with a marginal safety factor. It was reasoned that riveted steel pipe would perform similarly to pipe construction that immediately succeeded it (ie, gas-welded steel pipe) and therefore the same $R = 0.50$ was assigned.

4.8 ARC WELDED STEEL PIPE

Eguchi describes Grades A and B arc-welded steel gas pipe in fault rupture areas as 5.8% as vulnerable as cast iron. In landslide, the factor is 11%, and in liquefaction and lurching, 15%. The material properties of welded steel water pipe are fairly similar to those of contemporary gas pipe. Using these data, arc-welded steel water pipe was assigned a relative vulnerability $R = 0.125$.

5 DISCUSSION

Break rate functions developed using the methods discussed here are presented in Figure 10. These relationships indicate a trend of break rate increasing from 0 to 6 breaks per 1000 feet of pipe as permanent ground displacement increases from 0 to 6 feet. The trend and order of magnitude of damage tend to agree with Eguchi's data on pipe damage near fault rupture in 1971 San Fernando. In that earthquake, damage ranged from 0.4 to 5 breaks per 1000 feet near fault offsets as fault displacement increased from 3 to 98 inches. The figures developed for the present study are somewhat larger than Eguchi's figures for pipe damage in regions of liquefaction and landslide.

Hamada's data on the 1983 Nihonkai-Chubo earthquake indicate break rates in cast iron gas pipe reach as high as 21 breaks per 1000 feet, but it may be supposed that the higher magnitudes could result from stiffer joint construction. The order of magnitude is also supported by the Tangshan earthquake data presented by Shaoping et al.

The high initial break rate agrees with Miyajima's conclusion that pipe damage can be caused even by an average ground strain of less than 1 percent. Eguchi's relationship between break rate and MMI shows a similar trend, with the logarithm of damage increasing at a slower rate as MMI exceeds VIII.

6 CONCLUDING REMARKS

Damage data from the 1906 San Francisco and 1989 Loma Prieta earthquakes have been used to estimate break rates for cast iron water pipe subjected to permanent ground displacement. A bilinear curve was fitted to the data. The first segment (0 to 5 inches displacement) was based on water pipe damage in the Marina District in the 1989 Loma Prieta earthquake. The second segment (5 inches to 6 feet) was based on records of damage to similar pipes damaged in the 1906 San Francisco earthquake.

Damage rates for other types of water pipe were developed by applying a relative damageability factor, R, to the function developed for cast iron water pipe. The factors were developed for each type of water pipe used by the City of San Francisco for feeder and distribution water mains. Where possible, the factors were based on data presented in the literature. Where such data were unavailable, engineering judgment was employed to estimate relative damageability. As a result, the damage functions for several different types of San Francisco water pipe are similar bilinear curves differing by a factor *R*.

The major observation is that normalized pipe break rate is nonlinear with permanent ground displacements. Relatively small displacements produce initial pipe breakage; at larger displacements, break rates increase at a smaller rate. A possible explanation for the nonlinearity is that damage initiates at low magnitudes of permanent ground displacement, breaking the original pipe network into shorter segments that are relatively free to move with the surrounding soil. Relatively larger displacements are required to cause further breaks in the remaining intact network segments.

Data presented in the literature tend to verify the trend of relatively small displacements causing significant initial pipe breakage, while larger displacements result in additional breaks, but at a relatively smaller rate. The magnitude of break rates developed here tend to lie within the range of break rates recorded for gas and water pipe in other earthquakes under similar conditions.

Additional research is needed on this topic, including for example: (a) careful collection of post-earthquake survey measurements of vertical and lateral displacements in areas of liquefaction and/or high pipe breakage, and (b) small-scale laboratory experimental investigation of pipe break as a function of displacement.

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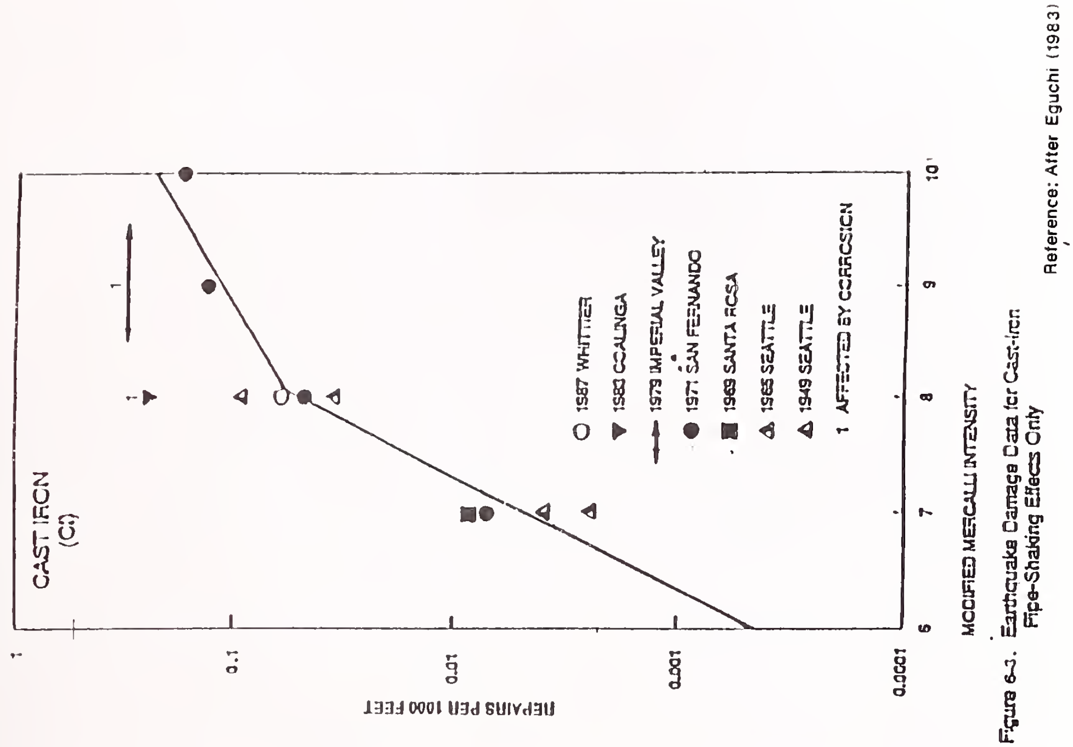

Figure 1: Damage data for shaking effects only
(after Eguchi 1983, appended by Harding Lawson et al, 1991)

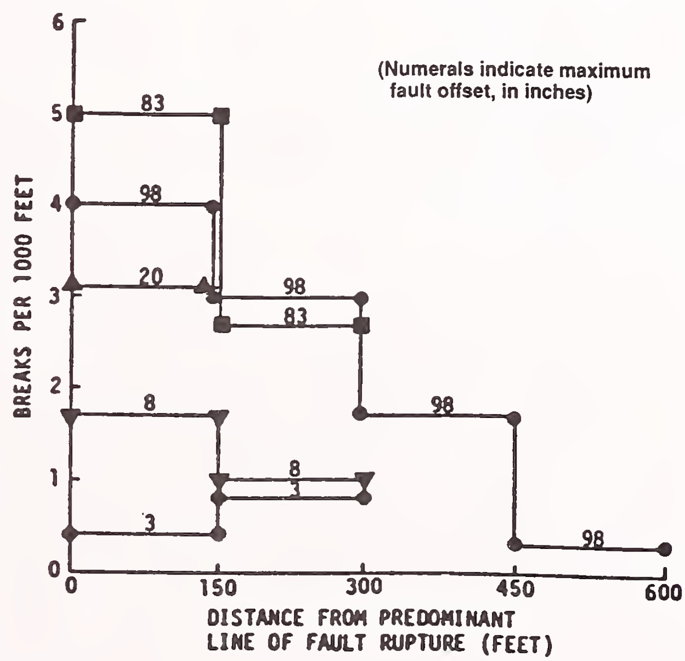

Figure 2: Effect of fault rupture
(Eguchi, 1983)

Figure 3: 1906 water main breaks
(Schussler, 1906)

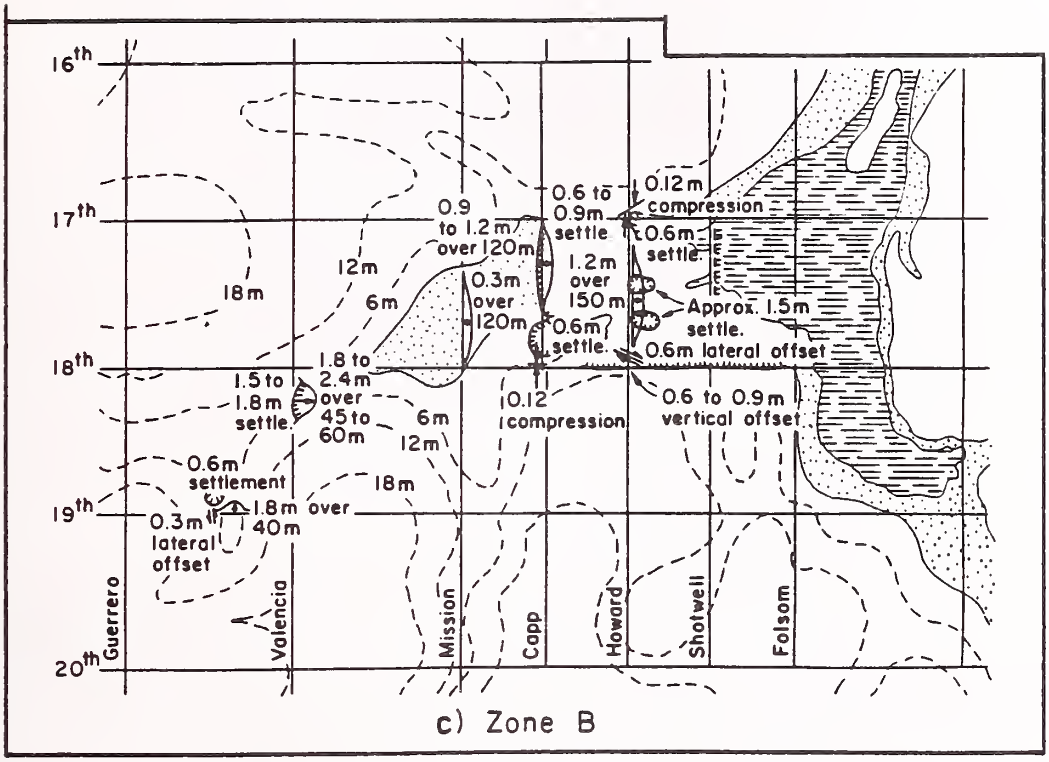

Figure 4: 1906 ground movements in Mission Creek (O'Rourke, 1989)

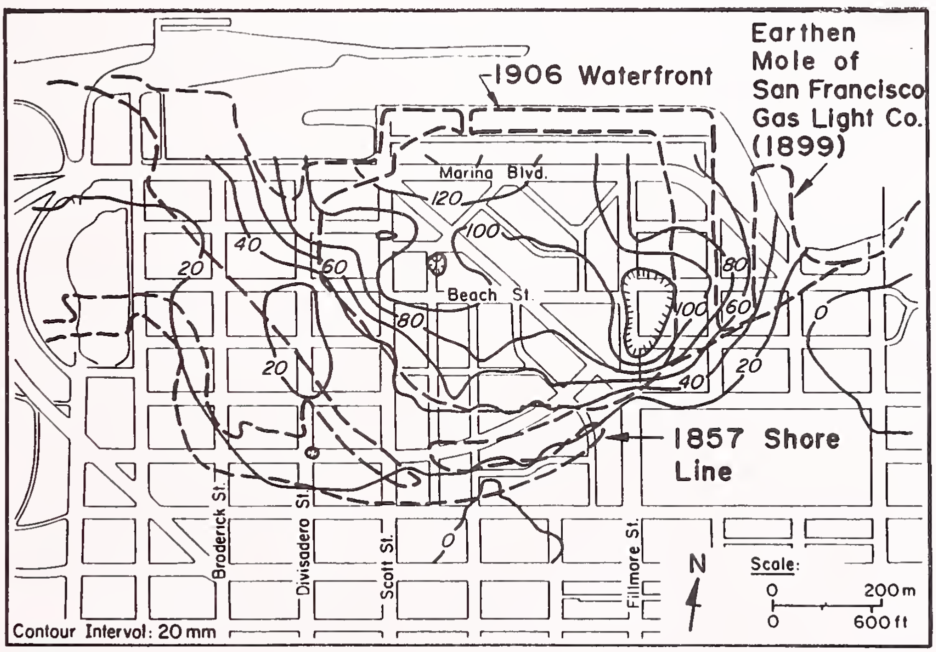

Figure 5: 1989 Marina settlement contours (O'Rourke, 1990)

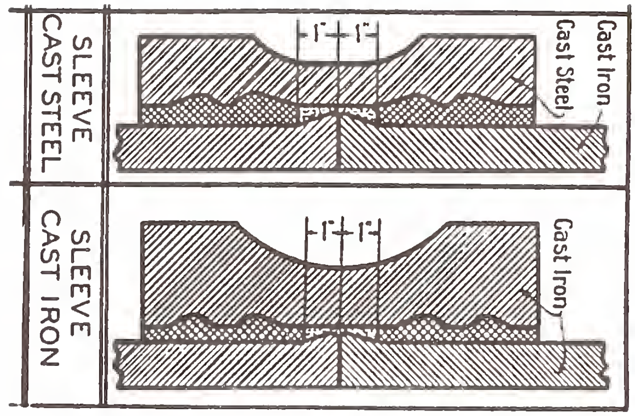

Figure 6: Double spigot joint

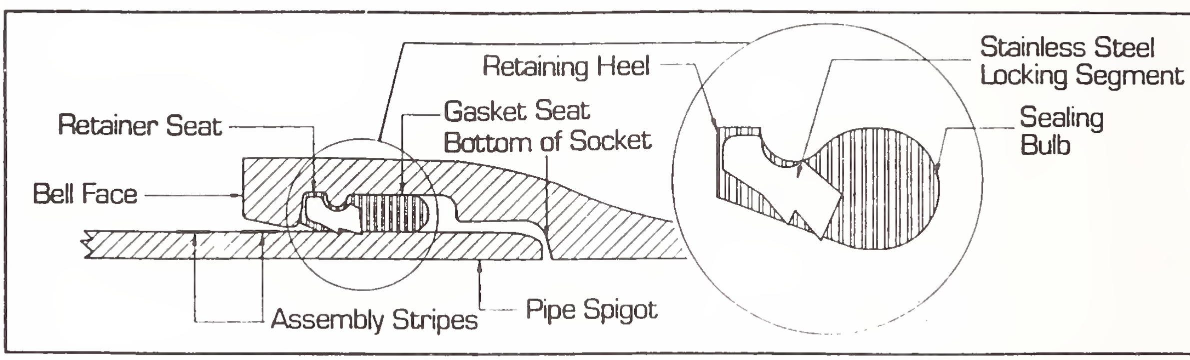

Figure 7: Field Lok Gasket
(U.S. Pipe)

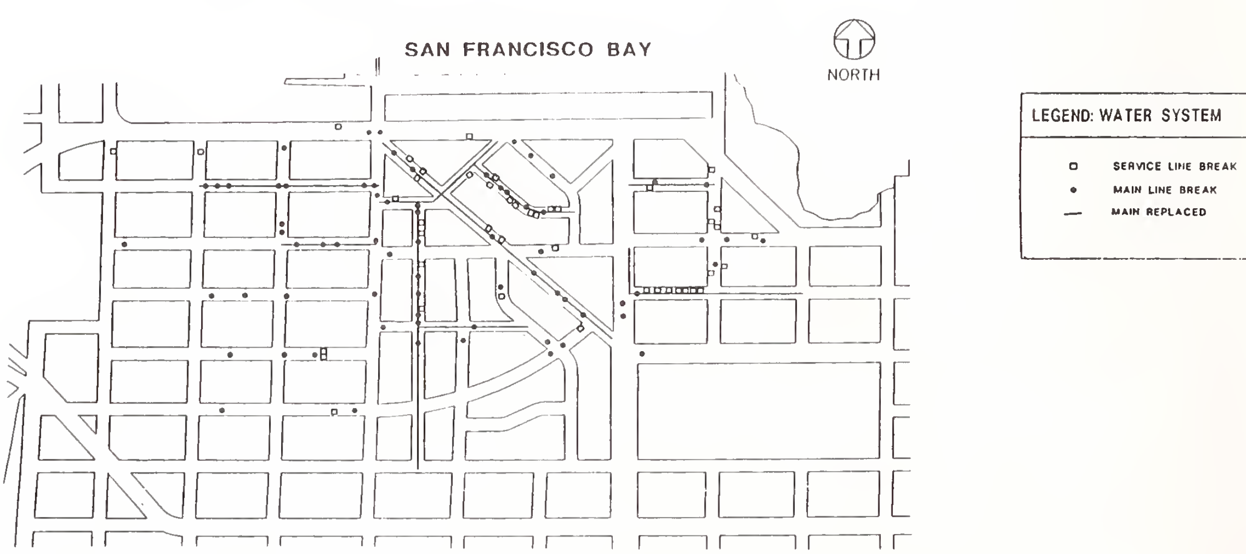

Figure 8: 1989 Marina water system breaks

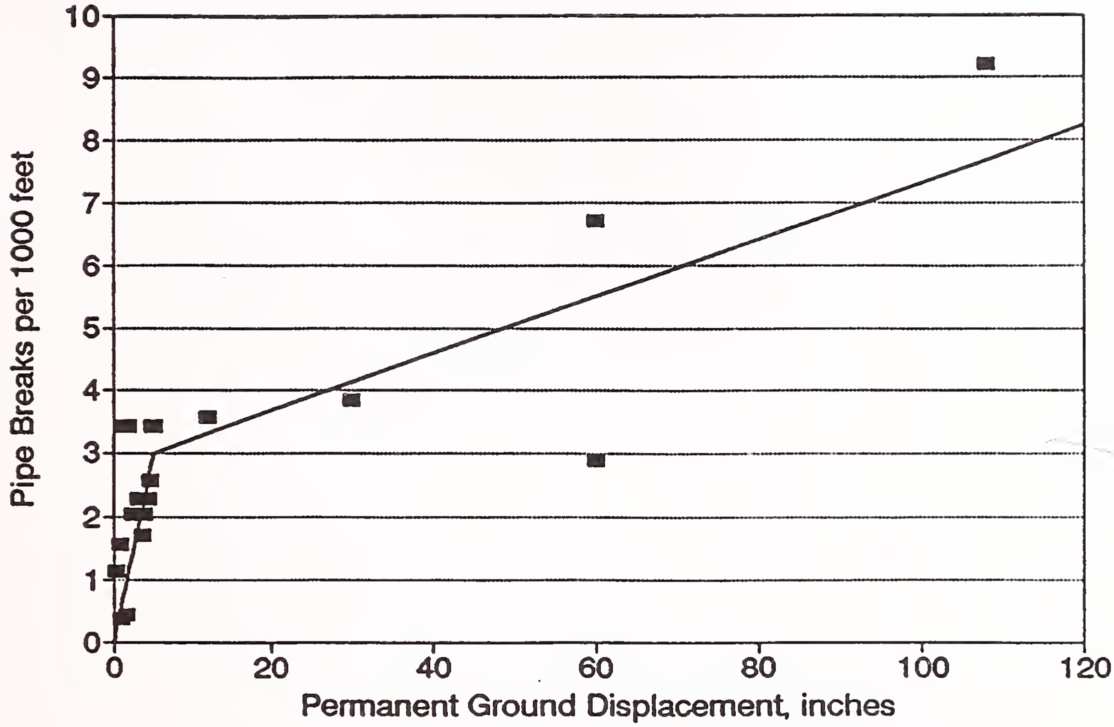

Figure 9: 1906 and 1989 cast iron water pipe damage

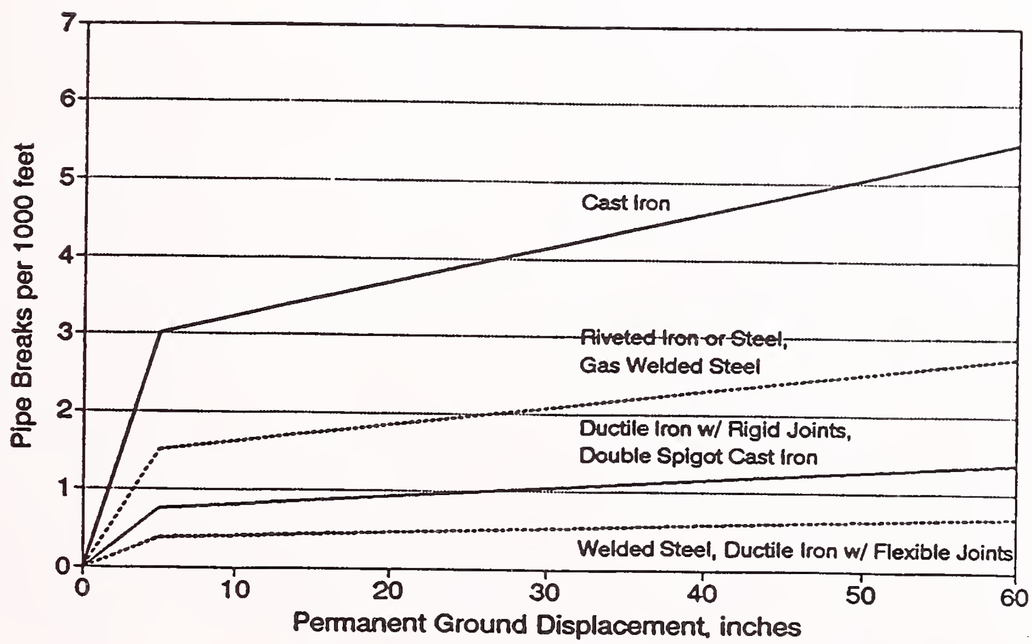

Figure 10: Estimated water supply pipe damage

A PROPOSAL OF PRACTICAL EARTHQUAKE RESPONSE ANALYSIS METHOD OF CYLINDRICAL TUNNELS IN SOFT GROUND

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ABSTRACT

This paper proposes a simple method for the earthquake response analysis of the cross section of cylindrical tunnels in homogeneous soft ground. In this method, the tunnel lining is modeled as a simple ring-on-elastic-support and is subjected to two kinds of seismic forces, i.e. the ground displacements and the ground shear stresses which are estimated from the free-field earthquake response. The seismic stresses induced in the tunnel lining are thus described by mathematical formulae that can be calculated easily by means of a personal computer. Accuracy of the method is almost comparable to that of FEM. For designers convenience, approximate formulae for estimating roughly the seismic stresses are also proposed in this paper.

INTRODUCTION

In recent years, most urban tunnels for lifeline facilities and transportation systems in Japan have been constructed by shield tunneling method, which forms a cylindrical tunnel fabricating segmented lining members in a tunnel boring machine. Because of effective use of underground space, the diameter and depth of these tunnels tend to be large and deep.

Until now, the influence of earthquakes on tunnels in soft ground has been discussed mainly with regard to the longitudinal direction. However, unlike buried pipes with small diameters such as gas pipelines, it is also necessary to examine the seismic stability of the cross section of tunnels with large diameters, which are supposed to be largely deformed by shear strain in soft ground during earthquakes. In these days, FEM (Finite Element Method) or the Seismic Deformation Method, which uses a beams-and-springs model of tunnel-ground systems, are employed as earthquake response analysis methods for cross sections of very large and important tunnels. However, since these analysis methods are time and labor consuming, and moreover, since tunnels are thought to be safe from earthquakes, usually the earthquake response analysis of cross sections is not executed in the design process for ordinary shield tunnels. It is desirable to develop a rational and yet simple method of analysis. In this paper, a simple method to analyze the earthquake response of cross sections of cylindrical tunnels is proposed.

METHOD OF EARTHQUAKE RESPONSE ANALYSIS

Object of Analysis

The object of earthquake response analysis in this paper is the cross section of a cylindrical tunnel as the one shown in Fig.1. The tunnel has radius R and its center is located at depth H. The surrounding ground consists of two layers: a soft surface layer and a rigid base layer. The surface layer, with thickness Hg, is assumed to be homogeneous.

Model of Tunnel-Ground System

Fig. 2 shows the general concept of the proposed method. The tunnel lining is modeled as an uniform ring with unit length along the tunnel axis. In the case of a shield tunnel, the tunnel lining, which is segmented in the circumferential direction, can also be modeled as a uniform ring with an equivalent rigidity taking the joint effect into consideration.

The interaction spring, which represents interaction effect between the tunnel lining and the surrounding ground, is distributed continuously around the ring. The interaction spring in the present model is not of the Winkler type but of the coupled type, which mutually acts on the radial and tangential directions of the ring; the characteristics of the interaction spring are detailed later. The model of the entire tunnel-ground system is therefore a ring-on-elastic-support.

Seismic Forces

From the results of many earthquake observations on underground structures and shake table tests on their scale models, it has been concluded that seismic stresses induced in underground structures are governed principally by surrounding ground strain, i.e. differential ground displacement. Based on this idea, the Seismic Deformation Method, which is the most widely used earthquake response analysis method for underground structures, calculates seismic stresses and deformations by statically subjecting the structure model, supported with interaction springs, to ground displacement. According to the Seismic Deformation Method,

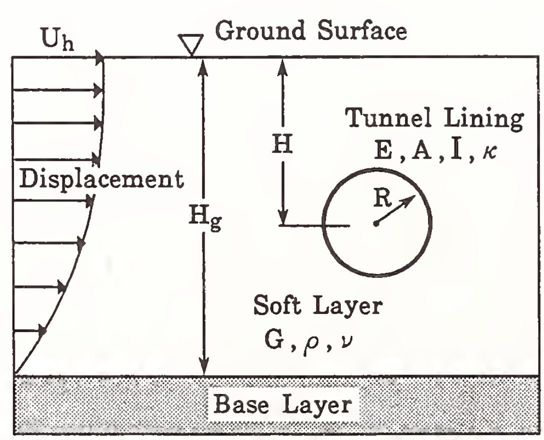

Fig. 1 Object of Earthquake Response Analysis

seismic stresses in a cylindrical tunnel lining can be obtained by statically displacing the end of the interaction spring of a ring-on-elastic-support model to the position of the ground displacement as determined from the free-field earthquake response.

However, the Substructure Method [Ref. 3] which is an efficient method for soil-structure interaction analysis, suggests theoretical seismic forces for underground structures as follows:

(1) In case of using free-field motion, seismic forces should be the product of the displacement and the interaction spring constant, and the ground stress along the position of the tunnel-ground interface.

(2) In case of using scattered-wave motion (i.e. motion of the free surface along the position of the tunnel-ground interface when considering that the tunnel has been removed), the seismic force should only be the product of the displacement and the interaction spring constant.

It is obvious that although the Seismic Deformation Method uses free-field motion, it does not take ground stress into consideration.

The present method takes both ground displacement and stress into account as seismic forces. As shown in Fig.2, these seismic forces are determined from earthquake response displacement and shear stress along the tunnel-ground interface in the free field. Ground displacement acts on the ring through the interaction spring, and stress acts directly on the ring.

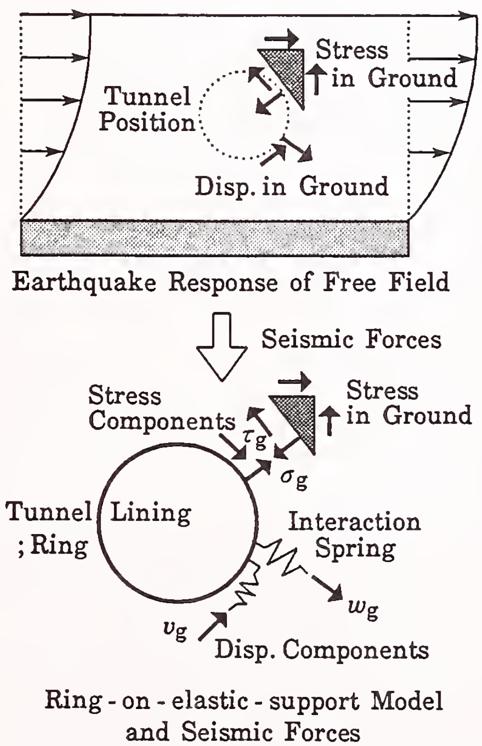

Fig. 2 Concept of the Proposed Method

Assumptions and Restrictions

In addition to the assumptions included in the modeling process of the tunnel-ground system, the following assumptions are used to formulate the tunnel behavior during earthquakes :

(1) The theory of curved beam is used for the analytical description of the ring behavior.

(2) The soft surface layer is supposed to oscillate in the 1st mode

(3) The influence of inertia force of the tunnel lining on the tunnel stresses is supposed to be negligible.

(4) The nonlinear characteristics of lining and soil are not taken into consideration.

(5) The depth of tunnel crown should be much larger than the diameter of the tunnel, and the tunnel invert should not be close to the rigid base layer.

FORMULATION OF TUNNEL BEHAVIOR

Notations

For the description of the ring behavior during an earthquake, the following notation is used.

H_g : Thickness of the surface layer.

G : Shear modulus of the surface layer.

ρ : Mass density of the surface layer.

ν : Poisson's ratio of the surface layer.

U_h : Displacement amplitude at the ground surface.

H : Depth of the tunnel center.

R : Radius of the tunnel.

E : Young's modulus of the lining.

A : Sectional area of the curved beam (per unit length along the tunnel axis).

I : Moment of inertia of the curved beam (per unit length along the tunnel axis).

κ : Section modulus of curved beam defined as Eq.(1).

$$\kappa = \frac{1}{A \cdot R} \cdot \int_A \frac{y^2}{R - y} dA \tag{1}$$

where y expresses the distance from the centroid of the cross section.

The following notation is shown in Fig.3.

ϕ : Angle from crown of the ring.

$w(\phi), v(\phi)$: Displacement in radial and tangential directions of the ring; subscript g of w and v denotes displacement in the free-field.

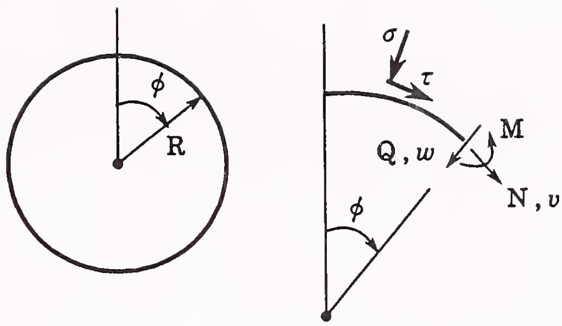

Fig. 3 Coordinate System of Ring

$M(\phi), N(\phi), Q(\phi)$: Bending moment, normal force and shearing force in the ring per unit length along the tunnel axis, respectively.

$\sigma(\phi), \tau(\phi)$: Seismic forces in radial and tangential directions acting on the ring. Subscript g expresses the ground stress and subscript i expresses the seismic force as the product of the displacements and the interaction spring constants.

The positive directions of $\phi, w, v, M, N, Q, \sigma, \tau$ are also shown in Fig.3.

Governing Equations

The following equations are derived as the governing equations of the ring from equilibrium and stress-strain equations.

$$M = -E \cdot A \cdot \kappa \cdot \left(\frac{d^2 w}{d\phi^2} + w\right) \tag{2.a}$$

$$N = -\frac{E \cdot A}{R} \cdot \left\{\kappa \cdot \frac{d^2 w}{d\phi^2} + (1+\kappa) \cdot w - \frac{dv}{d\phi}\right\} \tag{2.b}$$

$$Q = \frac{1}{R} \cdot \frac{dM}{d\phi} \tag{2.c}$$

$$N = -\frac{dQ}{d\phi} - R \cdot \sigma_g - R \cdot \sigma_i \tag{2.d}$$

$$Q = \frac{dN}{d\phi} + R \cdot \tau_g + R \cdot \tau_i \tag{2.e}$$

Modeling of Interaction Spring

Modeling of the interaction spring is very important to obtain the displacements and stresses in the ring using Eqs.(2.a)~(2.e). Currently there is not an appropriate model of the interaction spring. Based on the theory of elasticity, a coupled type interaction spring model is proposed here.

Consider the cylindrical cavern in the homogeneous infinite media when the tunnel has been removed as shown in Fig.4. The stress-displacement equations for the interaction spring model are derived from the relation between the displacements along the cavern free surface and the reactions of the homogeneous media. Suppose that the radial and tangential displacements along the cavern surface are expressed as a Fourier series as follows,

$$w(\phi) = \sum_{n=1}^{\infty} w_n \cdot \sin(n \cdot \phi) \tag{3.a}$$

$$v(\phi) = \sum_{n=0}^{\infty} v_n \cdot \cos(n \cdot \phi) \tag{3.b}$$

where the symmetric condition of the displacement distribution in the cavern surface has been considered, and w_n and v_n express Fourier components of radial and tangential diplacements in the nth Fourier mode. The reactions of the homogeneous infinite media are derived using the theory of Airy's stress functions. Eqs.(4.a) and (4.b) are the results using the restriction that the horizontal stress component in the first Fourier mode derived by reactions σ_i and τ_i is equal to zero,

$$\sigma_i(\phi) = \sum_{n=1}^{\infty} (K_n \cdot w_n + K'_n \cdot v_n) \cdot \sin(n \cdot \phi) \tag{4.a}$$

$$\tau_i(\phi) = \sum_{n=0}^{\infty} (K_n \cdot v_n + K'_n \cdot w_n) \cdot \cos(n \cdot \phi) \tag{4.b}$$

K_n and K'_n correspond to the interaction spring constants which are defined for each Fourier mode of displacements along the cavern surface as follows,

$$K_n = \frac{2G}{R} \times \begin{cases} 1 & : n = 0 \\ 2 & : n = 1 \\ \dfrac{2n + 1 - 2\nu(n+1)}{3 - 4\nu} & : n \geq 2 \end{cases} \tag{5.a}$$

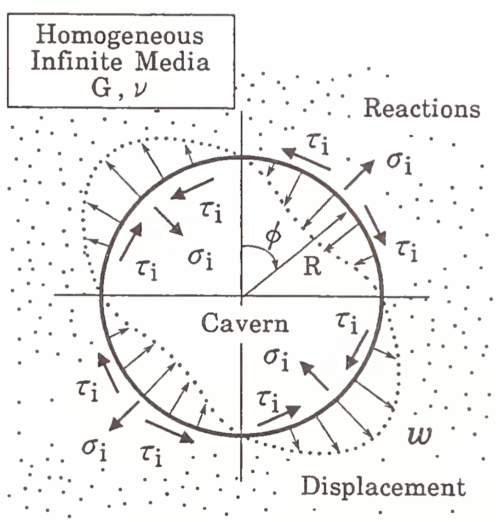

Fig.4 Tunnel-Ground Interaction

(The second Fourier Mode)

$$K'_n = \frac{2G}{R} \times \begin{cases} 0 & : n = 0, 1 \\ \dfrac{n + 2 - 2\nu(n+1)}{3 - 4\nu} & : n \geq 2 \end{cases} \tag{5.b}$$

In the Seismic Deformation Method, the Winkler's springs are generally used as the interaction springs. However the springs proposed here (i.e. coupled type springs) have two remarkable different characteristics compared with the Winkler's springs:

(1) Spring constants K_n and K'_n are defined for each Fourier mode of displacement.

(2) By enforcing radial displacement on cavern surface, not only radial reactions but also tangential reactions occur in the homogeneous infinite media. Also, by enforcing tangential displacement, the radial and tangential reactions occur.

In case that K'_n is equal to zero, mutual effects of radial and tangential springs are ignored. Thus, in this condition interaction springs are equivalent to the Winkler's springs. The results using the Winkler's springs are described later.

Formulation of Deformations and Stresses in Tunnel Lining

The displacement and shear stress distribution in the free field for the fundamental oscillation mode as shown in Fig.5 are given by Eqs.(6) and (7),

$$U(z) = U_h \cdot \cos(\frac{\pi \cdot z}{2H_g}) \tag{6}$$

$$q(z) = G \cdot \frac{dU(z)}{dz} = \frac{\pi \cdot G \cdot U_h}{2H_g} \cdot sin(\frac{\pi \cdot z}{2H_g}) \tag{7}$$

The radial and tangential components of the ground displacement and the ground shear stress at the position of the tunnel are derived as a function of the angle ϕ from Eqs.(6) and (7) as shown in Eqs.(8.a)~(8.d).

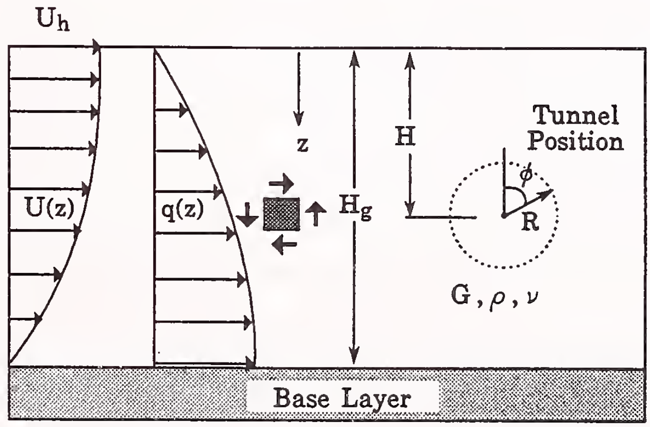

Fig. 5 Displacement and Stress in Free Field under 1st Mode Vibration

$$w_g(\phi) = -U_h \cdot \cos\left\{\frac{\pi \cdot (H - R \cdot \cos\phi)}{2H_g}\right\} \cdot \sin\phi \tag{8.a}$$
$$v_g(\phi) = U_h \cdot \cos\left\{\frac{\pi \cdot (H - R \cdot \cos\phi)}{2H_g}\right\} \cdot \cos\phi \tag{8.b}$$
$$\sigma_g(\phi) = -\frac{\pi \cdot G \cdot U_h}{2H_g} \cdot \sin\left\{\frac{\pi \cdot (H - R \cdot \cos\phi)}{2H_g}\right\} \cdot \sin 2\phi \tag{8.c}$$
$$\tau_g(\phi) = \frac{\pi \cdot G \cdot U_h}{2H_g} \cdot \sin\left\{\frac{\pi \cdot (H - R \cdot \cos\phi)}{2H_g}\right\} \cdot \cos 2\phi \tag{8.d}$$

Deriving the seismic forces σ_i and τ_i, the ground displacements w_n and v_n in Eqs.(4.a) and (4.b) are replaced by the difference between the ground displacements and the displacements of the ring, $(w_{gn} - w_n)$ and $(v_{gn} - v_n)$, respectively. Here w_{gn} and v_{gn} express the results of Fourier series expansion which include the first kind Bessel functions of w_g and v_g in Eqs.(8.a) and (8.b). By substituting σ_i and τ_i, and the Fourier series expansion of σ_g and τ_g into Eqs.(2.a)~(2.e), the radial displacement of the nth Fourier mode of the ring, w_n, can be expressed as follows,

$$\frac{w_n}{U_h} = \frac{F_n}{n^6 - (2 - \kappa\beta_n)n^4 + (1 + \beta_n - 2\kappa\beta_n)n^2 - 2\beta'_n n + (1 + \kappa)\beta_n + \kappa(\beta_n^2 - \beta_n'^2)} \tag{9.a}$$

Where,

$$F_n = P_n \cdot \Big\{ \{n^2 + n + \kappa \cdot (\beta_n + \beta'_n)\} \cdot \{(\beta_n - \beta'_n) \cdot J_{n-1} + \beta_G \cdot (\frac{\pi \cdot R}{2H_g}) \cdot J_{n-2}\}$$
$$+ \{n^2 - n + \kappa \cdot (\beta_n - \beta'_n)\} \cdot \{(\beta_n + \beta'_n) \cdot J_{n+1} - \beta_G \cdot (\frac{\pi \cdot R}{2H_g}) \cdot J_{n+2}\} \Big\} \tag{9.b}$$

$$P_n = \begin{cases} (-1)^{(n+1)/2} \cdot \cos(\frac{\pi \cdot H}{2H_g}) & :n \text{ is odd} \\ (-1)^{n/2} \cdot \sin(\frac{\pi \cdot H}{2H_g}) & :n \text{ is even} \end{cases} \tag{9.c}$$

$$\beta_G = \frac{R \cdot G}{E \cdot A \cdot \kappa} \qquad \beta_n = \frac{R^2 \cdot K_n}{E \cdot A \cdot \kappa} \qquad \beta'_n = \frac{R^2 \cdot K'_n}{E \cdot A \cdot \kappa} \tag{9.d}$$

In the above equations, J_n denotes the nth mode of the first kind Bessel functions $J_n(\pi \cdot R/2H_g)$. Substituting w_n into Eqs.(2.a)~(2.e), the displacements and stresses in the ring are derived as follows,

$$\frac{w(\phi)}{U_h} = \sum_{n=1}^{\infty} \frac{w_n}{U_h} \cdot \sin(n \cdot \phi) \tag{10.a}$$

$$\frac{v(\phi)}{U_h} = \left\{J_1 - \frac{\beta_G}{\beta_0}(\frac{\pi \cdot R}{2H_g}) \cdot J_2\right\} \cdot \sin(\frac{\pi \cdot H}{2H_g}) + \sum_{n=1}^{\infty} \frac{1}{n + \kappa \cdot \beta'_n} \cdot$$
$$\Big\{ - \{\kappa \cdot (n^2 - 1)^2 + 1 + \kappa \cdot \beta_n\} \cdot \frac{w_n}{U_h} + \kappa \cdot P_n \cdot$$
$$\{\beta_n \cdot (J_{n+1} + J_{n-1}) + \beta'_n \cdot (J_{n+1} - J_{n-1}) - \beta_G \cdot (\frac{\pi \cdot R}{2H_g})$$
$$\cdot (J_{n+2} - J_{n-2})\} \Big\} \cdot \cos(n \cdot \phi) \tag{10.b}$$

$$\frac{M(\phi)}{E \cdot A \cdot \kappa \cdot U_h} = \sum_{n=2}^{\infty} (n^2 - 1) \cdot \frac{w_n}{U_h} \cdot \sin(n \cdot \phi) \tag{10.c}$$

$$\frac{R \cdot Q(\phi)}{E \cdot A \cdot \kappa \cdot U_h} = \sum_{n=2}^{\infty} n \cdot (n^2 - 1) \cdot \frac{w_n}{U_h} \cdot \cos(n \cdot \phi) \tag{10.d}$$

$$\begin{aligned}\frac{R \cdot N(\phi)}{E \cdot A \cdot \kappa \cdot U_h} = \sum_{n=1}^{\infty} \frac{1}{n + \kappa \cdot \beta_n'} \cdot \Big\{ & \{(n^3 + \kappa \cdot \beta_n') \cdot (n^2 - 1) + n \cdot \beta_n - \beta_n'\} \cdot \frac{w_n}{U_h} \\ & - n \cdot P_n \cdot \{\beta_n \cdot (J_{n+1} + J_{n-1}) + \beta_n' \cdot (J_{n+1} - J_{n-1}) \\ & - \beta_G \cdot (\frac{\pi \cdot R}{2H_g}) \cdot (J_{n+2} - J_{n-2})\} \Big\} \cdot \sin(n \cdot \phi)\end{aligned} \tag{10.e}$$

Approximate Formulae for Calculation of Seismic Stresses

Eqs.(10.c)~(10.e) are the accurate solutions of the stresses in the tunnel. The inclusion of Bessel functions in these solutions makes it a little difficult to calculate the stresses in the tunnel. Therefore approximate formulae, that are easier to compute than the accurate solutions, are proposed by simplifying Eqs.(10.c)~(10.e) mathematically.

The series expansion of the first kind Bessel functions is given by,

$$J_k(x) = (\frac{x}{2})^k \cdot \sum_{n=0}^{\infty} (-1)^n \cdot (\frac{x}{2})^{2n} \cdot \frac{1}{n! \cdot (n+k)!} \tag{11.a}$$

where $x = \pi \cdot R/2H_g$ (11.b)

Generally the depth of the tunnel crown is much larger than the diameter of the tunnel. Thus x in Eq.(11.b) is negligible compared to one and the terms of $(x/2)^2, (x/2)^3 ...$ in Eq.(11.a) are neglected. After this approximation, only J_0 and J_1 in Bessel functions are effective in Eqs.(10.c)~(10.e),

$$J_0(\frac{\pi \cdot R}{2H_g}) \approx 1 \tag{12.a}$$

$$J_1(\frac{\pi \cdot R}{2H_g}) \approx \frac{1}{2} \cdot (\frac{\pi \cdot R}{2H_g}) \tag{12.b}$$

$$J_k(\frac{\pi \cdot R}{2H_g}) \approx 0 \ : \ k \geq 2 \tag{12.c}$$

In Eq.(13) κ is defined as an approximate formula using radius R, area A and moment of inertia I of the ring.

$$\kappa \approx \frac{I}{A \cdot R^2} \tag{13}$$

In case that the term including κ is negligible compared with the other terms, it can also be neglected.

After these mathematical approximation, only the terms of the 2nd Fourier mode in Eqs.(10.c)~ (10.e) are effective for calculating the seismic stresses in the ring. Thus the maximum bending moment and normal force occur at the diagonal direction of the tunnel, and the maximum shearing force occurs at the crown and invert of the tunnel. Eqs.(14.a)~(14.c) express the maximum seismic stresses in the tunnel lining as given by the approximate formulation.

$$M_{max} = \frac{3\pi \cdot E \cdot I}{2R \cdot H_g} \cdot U_h \cdot \sin(\frac{\pi \cdot H}{2H_g}) \cdot C \tag{14.a}$$

$$Q_{max} = \frac{3\pi \cdot E \cdot I}{R^2 \cdot H_g} \cdot U_h \cdot \sin(\frac{\pi \cdot H}{2H_g}) \cdot C \tag{14.b}$$

$$N_{max} = \frac{3\pi \cdot E \cdot I}{R^2 \cdot H_g} \cdot U_h \cdot \sin(\frac{\pi \cdot H}{2H_g}) \cdot (1 + \frac{G \cdot R^3}{6E \cdot I}) \cdot C \tag{14.c}$$

$$C = \frac{4(1-\nu) \cdot G \cdot R^3}{(3-2\nu) \cdot G \cdot R^3 + 6(3-4\nu) \cdot E \cdot I} \tag{14.d}$$

Eqs.(14.a)~(14.c) correspond to the seismic stresses in the ring for the case in which the ground shear strain is uniform (i.e. displacement distribution of the free field is linear). The value of the uniform strain is equal to the shear strain at the position of the center of the ring when the ground is oscillating in the fundamental mode.

VERIFICATION OF THE PRESENT METHOD BY FEM

Analysis Cases

In order to verify the present method, comparative analyses with the FEM(Super FLUSH) have been carried out. Four cases, as shown in Table 1, have been analyzed. Case 1 is the fundamental case. In case 2, the tunnel is located close to the base layer. In case 3, the diameter of the tunnel is changed to a small one. And in case 4, the surface layer is made more rigid.

In FEM, only the left half of the area was analyzed considering the symmetry of the system, and the tunnel lining was modeled by 36 beam elements. The material properties of the tunnel lining are listed in Table 2. Fig. 6 shows the FEM meshes. The input earthquake for each case is a harmonic wave with natural frequency equal to that of the 1st oscillation mode of the surface layer, and with amplitude of 100 Gals at base layer outcrop. In the present method, the displacement amplitude at the ground surface, Uh, as shown in Fig.7, was determined from the results obtained by FEM (see also Table 3).

Table 1 Analysis Cases

No.	Tunnel			Surface Layer					
	Diameter 2R (m)	Thickness h (m)	Depth H (m)	Thickness H_g (m)	Density ρ (t/m³)	Velocity V_S (m/s)	Poisson's Ratio ν	Damping Factor	Natural Frequency (Hz)
1	16	0.8	25	50	1.6	120	0.45	0.02	0.6
2	16	0.8	40	50	1.6	120	0.45	0.02	0.6
3	6	0.3	25	50	1.6	120	0.45	0.02	0.6
4	16	0.8	25	50	1.8	240	0.40	0.02	1.2

Table 2 Material Properties

Properties	Tunnel Lining	Base Layer
Mass Density (t/m³)	2.6	2.0
Young's Modulus (GPa)	34.3	0.86
S-wave Velocity (m/s)	2370	400
Poisson's Ratio	0.17	0.35
Damping Factor	0.02	0.02

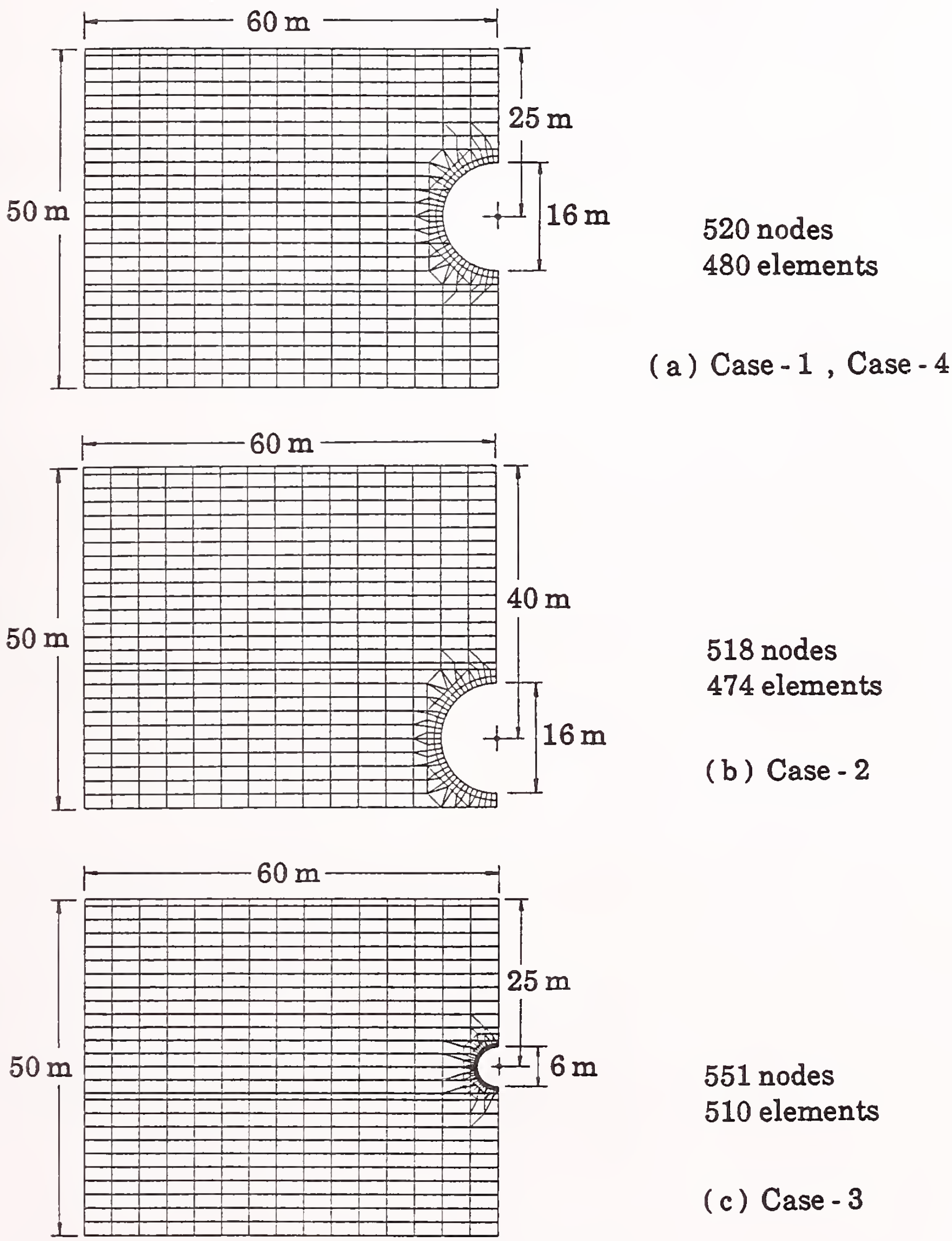

Fig. 6 FEM Meshes

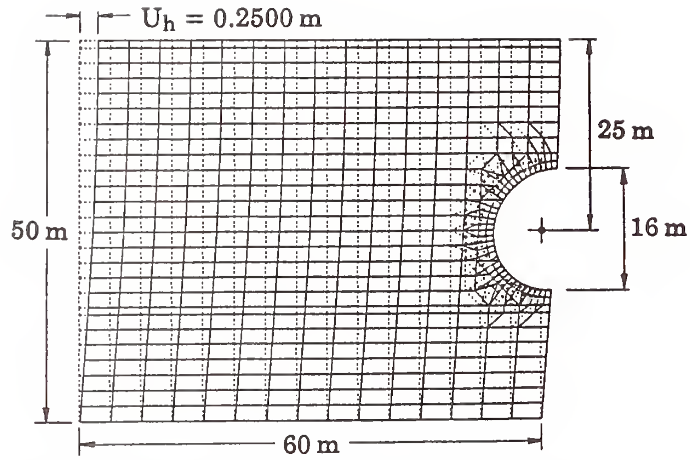

Fig. 7 An Example of Displacement Diagram by FEM (Case 1)

Results

Fig.8 illustrates the comparison of stress distributions induced in the tunnel lining obtained by the proposed method and by the FEM. The maximum stresses are summarized in Table 3.

From Fig.8 and Table 3, the following can be noted:

(1) Except in Case 2, both the maximum values and the distribution patterns of seismic stresses calculated by the proposed method show good agreement with those obtained by FEM. The difference in maximum values between the two methods is of a few percents.

(2) In Case 2, in which the tunnel is located close to base layer, the shearing force at a region in the neighborhood of the tunnel invert differs with that obtained by FEM. The reason for the difference is supposed to be that the interaction spring in the proposed method is modeled uniformly along the tunnel ring, while kinematic interaction effects, in fact, vary with distance between the tunnel and the surface/bottom boundary of the ground layer. Therefore, when using the present method, it is necessary to pay attention to the location of the tunnel.

(3) In case a Winkler type interaction spring is used for the analytical model, as illustrated with broken lines in Fig.8, normal force in all cases, and other stresses in Case 4, were underestimated. This indicates that interaction forces which mutually act on radial and tangential direction of the ring have considerable influence on seismic stresses, and that the coupled type interaction spring used in the proposed method is quite appropriate.

(4) Approximate formulae underestimate seismic stresses. According to the detailed examination of the approximate formulae, they underestimate seismic stresses at most 20% of those by exact solutions (10.c)~(10.e) [Ref. 1, 2]. Although the approximate formulae have limits of accuracy, they are of great use for making rough estimates.

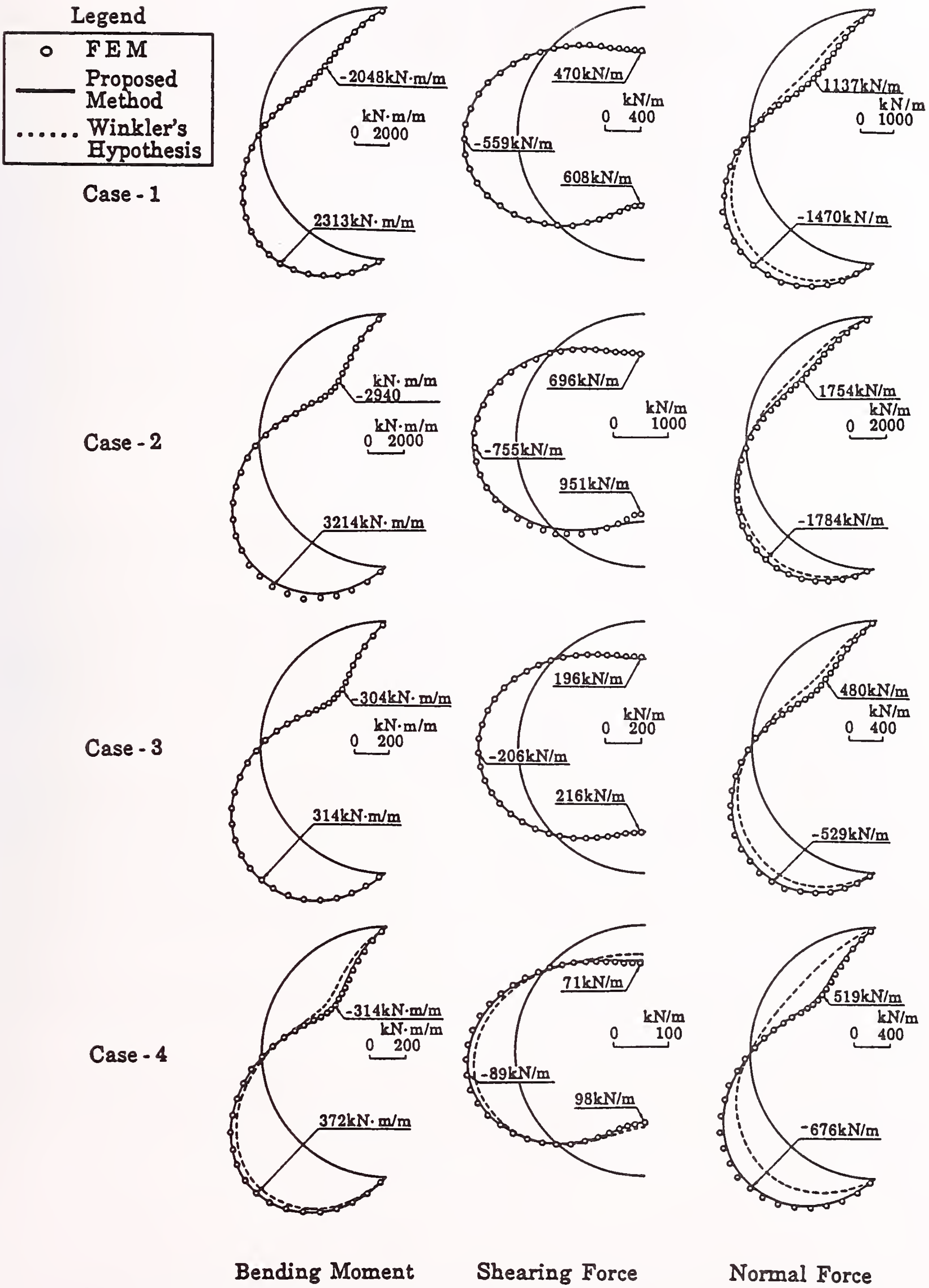

Fig. 8 Comparison between the Present Method and FEM

Table 3 Comparison of Tunnel Stresses

Case No.	Ground Response		Tunnel Stresses								
	Acc. (Gal)	Disp. U_h (cm)	Max. Bending Moment (kN·m/m)			Max. Shearing Force (kN/m)			Max. Normal Force (kN/m)		
			FEM	Proposed	Approx.	FEM	Proposed	Approx.	FEM	Proposed	Approx.
1	367	25.00	2313	2365 (1.02)	2240 (0.97)	608	624 (1.03)	560 (0.92)	1470	1458 (0.99)	1312 (0.89)
2	371	25.37	3214	3091 (0.96)	3058 (0.95)	951	784 (0.82)	764 (0.80)	1784	1838 (1.03)	1791 (1.00)
3	368	25.18	314	324 (1.03)	318 (1.01)	216	221 (1.02)	212 (0.98)	529	516 (0.98)	496 (0.94)
4	178	2.77	372	363 (0.98)	333 (0.90)	98	97 (0.99)	83 (0.85)	676	643 (0.95)	587 (0.87)

( ) : Ratio to FEM

CONCLUSIONS

A simple method for earthquake response analysis of cross sections of cylindrical tunnels in homogeneous soft ground has been developed. The general idea and peculiarity of this method can be summarized as follows :

(1) The tunnel-ground system is modeled simply as a ring-on-elastic-support.

(2) The interaction spring as elastic support in the model is not of the Winkler-type but of the coupled type, which mutually acts on the radial and tangential direction of the ring. In case that the Winkler-type ground spring is employed in the model, seismic stresses, especially normal force, in the tunnel are underestimated.

(3) The seismic forces for the model are the displacement and shear stress evaluated from the free-field earthquake response along the position of the tunnel-ground interface, whereas only the displacement is taken into account in the Seismic Deformation Method, which is the most widely used earthquake response analysis method for underground structures.

(4) The deformation and stresses in the tunnel are thus described by mathematical formulae using a series of Bessel functions, that can be calculated easily by means of a personal computer.

(5) Accuracy of the method is almost comparable to that of FEM under the condition that the depth of the tunnel crown is much larger than the diameter of the tunnel and that the tunnel is not too close to the rigid base.

Besides the above-mentioned exact solution of seismic stresses, approximate formulae are also presented. Although the approximate formulae underestimate the stresses by at most 20% of the exact values, they are useful for making rough estimates.

FUTURE DIRECTIONS

As a result of this study, a method to examine the seismic stability of cross sections of cylindrical tunnels was developed for a considerably idealized tunnel-ground system.

In order to establish a seismic design method of cross sections of large scale sheild tunnels, it is necessary to advance the study to a more practical stage considering the following:

(1) In this study, the surface layer of the tunnel-ground system is assumed to be elastic and homogeneous. In practice, however, behavior of the ground during large earthquakes shows non-linear characteristics due to material non-linearity of the soil. Furthermore, when a shield tunnel with large diameter and large depth at urban site is planned, the case of the cross section of the tunnel lying across a boundary line between soft Alluvial and hard Diluvial stratum is often encountered. It is therefore necessary to improve the proposed method into a more practical one as far as it is preserved simple.

(2) In addition to improving the earthquake response analysis method, we should examine the characteristics of seismic stresses induced in the tunnel lining in comparison with static stresses due to earth and water pressures, and discuss the total stability of the tunnel.

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VISUAL INFORMATION SYSTEM FOR SEISMIC GROUND HAZARD ZONING

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ABSTRACT

Because most lifeline facilities are installed underground and have the potential to be damaged by ground movement during earthquakes, seismic ground hazard zoning is very effective for understanding and estimating the damage of underground structures over a large region.

Seismic zonation in the past has been generally done for only a certain magnitude of seismic intensity of an area. It usually does not consider different magnitudes of earthquakes and locations of epicenters. A system which is capable of assessing the ground hazard against any earthquake which might affect the area is very useful for making a rapid inspection and for implementing urgent restoration measures against damage to structures. This is because information about seismic ground motion and ground hazard is essential to understanding the damage to buried structures.

In this paper, the dynamic visual information system for seismic zonation of ground hazard which is being developed at the Public Works Research Institute (PWRI) is introduced.

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INTRODUCTION

Because damaging earthquakes affect structures over a wide area when they hit urban regions, it generally takes a considerable amount of time and labor to understand the damaged state of facilities immediately after the occurrence of large earthquakes. Correct information on damage should be gathered quickly to take action as soon sa possible for restoration just after an earthquake. In addition, the rapid collection of useful information from various information transferred in a confused state is necessary. The type and cause of the damage need to be estimated and to given to the people who work on damage restoration. For this purpose, a visual display of the distribution of the seismic intensity in an area, even though it might be an estimated one, and the result of assessed ground hazard is thought to be a very effective aid to rapid restoration work just after an earthquake.

Although the underground structures for lifeline systems such as water pipes, gas pipes, and so forth may be severely damaged during earthquakes, it is difficult to directly find the damage because the structures are buried in the ground. It is necessary and effective to estimate the damage of underground structures indirectly with use of ground hazard zoning because the structures are easily affected by the dynamic behavior of ground. Because the causes of damage to underground structures are seismic ground motion, soil liquefaction and so on, the zonation of ground motion and ground hazard is effective for evaluating the hazard of lifeline systems extensively.

In this paper, the seismic ground hazard information system, which is being developed to estimate the ground hazard and damage of underground structures quickly and correctly with use of a computer system, is introduced. The system is being developed as a pilot-system to investigate the usefulness of visualization methods in sieving correct information effectively out of various information which is reported just after the event. This is done by using a digitized map which has various information such as ground and structure condition, ground hazard, structural damage and location of structures.

SEISMIC INFORMATION RELATED TO GROUND HAZARD

The purposes of establishing the information system of seismic ground motion and hazard are twofold (see **Table. 1**):

1. To make possible the urgent and rapid inspection and restoration of earthquake hazards just after an earthquake. Then, action can be taken based on the actual or rationally estimated distribution of seismic ground motion, and actual or rationally estimated ground hazard and actual damage of structures.
2. To be utilized for strengthening the structures against future earthquakes based on the rationally estimated distribution of seismic ground motion, ground hazard and damage of structures.

Although there are several kinds of indices of earthquake ground motion and hazard types, the ones of interest in this study are those related to the earthquake-induced damage of structures, especially underground structures. They are as follows:

1) Seismic intensity of ground motion.
2) Soil liquefaction.
3) Lateral ground spread induced by soil liquefaction.

The flow of seismic information of ground motion, ground hazard and structure damage during the earthquakes can be summarized as shown in **Fig. 1**. The topics related to the seismic information in this study can be classified as follows:

1) Fundamental data of earthquake and induced-ground motion and hazard.
2) Estimated data of seismic ground motion and hazard.
3) Quantitative estimation methods of seismic ground motion and hazard.
4) Data base needed for seismic zoning of ground hazard.
5) Actual data of earthquake-induced damage of (underground) structures.
6) Quantitative estimation methods of earthquake-induced damage of (underground) structures.
7) Data base needed for estimating damage of structures by ground hazard.

In this paper, the research results on the above subjects of 1) to 4) will mainly be shown.

Fundamental Data of Earthquake and Induced-Ground Motion and Hazard

The information related to seismic ground motion and hazard which can be recorded and collected just after an earthquake can be considered as follows.

1) Epicentral location of actual or estimated earthquake.
2) Magnitude of actual or estimated earthquake.
3) Seismic ground motion recorded by seismographs.
4) Ground hazard found at the site.

Because the location and Richter-Scale Magnitude of an earthquake are reported in Japan by the Japan Meteorological Agency (JMA) just after an earthquake, these fundamental data of an earthquake can be obtained easily through mass media. When the earthquake observation system is improved as an on-line system, the seismic ground motion records can be obtained without delay just after an earthquake and used for actual seismic zoning of ground hazard. Furthermore, the ground hazard which is estimated by indirect information can be corrected with use of the actual hazard which is observed at the site and reported after the earthquake.

Estimated Data of Seismic Ground Motion and Hazard

Although the fundamental data of an earthquake and the recorded ground motion are very useful, they can not necessarily yield enough information on the ground motion at every site of interest. Thus it is necessary to supplement the fundamental information with analyzed data for urgent and rapid inspection and restoration of earthquake-induced damage of structures. The information related to ground hazard which is needed and useful to supplement the fundamental information for restoration just after an earthquake can be classified as follows:

1) Seismic ground motion estimated at a site of interest in an area and its distribution.
2) Soil liquefaction potential at a site of interest in an area and its distribution.
3) Potential magnitude of lateral ground spread by soil liquefaction and its distribution.

Quantitative Estimation Methods of Seismic Ground Motion and Hazard

Reasonable methods for analyzing the original data from of an earthquake and the ground condition have been investigated and established. The following methods are necessary for estimating the secondary information quantitatively.

1) Method for estimating the attenuation of seismic ground motion.
2) Method for estimating the soil liquefaction potential.
3) Method for estimating the lateral ground flow induced by soil liquefaction.

Data Base Needed for Seismic Ground Hazard Zoning

It is necessary to establish a data base to indicate and analyze the original information before an earthquake occurs. The following data base should be established for seismic ground hazard zoning:

1) Mapping data which draw the fundamental information such as the coastline, boundaries of cities, locations of highways and rivers and so on.
2) Ground group information to estimate the seismic ground motion and soil liquefaction potential.
3) Soil profile, geology and topographical ground condition to estimate the soil liquefaction potential.
4) Contours of elevation to estimate the lateral ground spread by soil liquefaction.

METHODS TO ESTIMATE QUANTITATIVE GROUND HAZARD

Attenuation of Seismic Ground Motion

The maximum acceleration of the ground surface can be estimated with the use of equation (1) according to the ground group at the site of interest [Ref. 1] .

$$\alpha_{max} = \begin{bmatrix} 87.4\times 10^{0.216M} \\ 232.5\times 10^{0.313M} \\ 403.8\times 10^{0.265M} \end{bmatrix} \times (\Delta + 30)^{-1.218} \quad \begin{matrix} \text{[Ground Group 1]} \\ \text{[Ground Group 2]} \\ \text{[Ground Group 3]} \end{matrix} \qquad (1)$$

where α_{max} is maximum acceleration of the ground surface (gal), M is the Richter-Scale Magnitude of an earthquake and Δ is the epicentral distance (km). The Richter-Scale Magnitude is reported by JMA just after an earthquake and the epicentral distance can be calculated from the location of an earthquake. The Ground Groups 1, 2 and 3 are rock or diluvial ground, alluvial ground, and soft alluvial ground or reclaimed ground, respectively [Ref. 2] .

The Seismic Intensity on the JMA scale is also estimated based on the maximum acceleration of the ground surface calculated with the use of equation (1) as follows.

α_{max} (gal)	~ 0.8 ~	2.5 ~	8.0 ~	25 ~	80 ~	250 ~	400 ~	
Seismic Intensity	0	1	2	3	4	5	6	7

Soil Liquefaction Potential

The sandy layers at a site of interest with the following conditions can be considered to be vulnerable to liquefaction during earthquakes, and their liquefaction potential should be examined [Ref. 3] :

1) The layer is a saturated alluvial sandy layer.
2) The layer is within 20 m from the ground surface.
3) The ground water level is within 10 m from the ground surface.
4) The D_{50}-value is between 0.02 mm and 2.0 mm for grain size accumulation curve.

As for sandy soil layers judged to be vulnerable to liquefaction, liquefaction potential can be examined based on the liquefaction resistance factor F_L defined by the following equation (2) [Refs. 2 and 3] . Soil layers with a liquefaction resistance factor F_L less than 1.0 are judged to liquefy during the earthquake considered.

$$F_L = R / L \tag{2}$$

where F_L is the liquefaction resistance factor, R is the resistance of soil elements to dynamic loads and L is the dynamic load applied to the soil elements. R and L are estimated simply with use of the following equations:

$$R = R_1 + R_2 + R_3 \tag{3}$$

$$R_1 = 0.0882\sqrt{N}/(\sigma_v' + 0.7) \tag{4}$$

$$R_2 = \begin{cases} 0.19 & (0.02\text{mm} \leq D_{50} \leq 0.05\text{mm}) \\ 0.225\log_{10}(0.35/D_{50}) & (0.05\text{mm} < D_{50} \leq 0.6\text{ mm}) \\ -0.05 & (0.6\text{ mm} < D_{50} \leq 2.0\text{ mm}) \end{cases} \tag{5}$$

$$R_3 = \begin{cases} 0.0 & (0\ \% \leq F_c \leq 40\ \%) \\ 0.004F_c - 0.16 & (40\ \% < F_c \leq 100\ \%) \end{cases} \tag{6}$$

$$L = r_d \cdot k_s \cdot (\sigma_v / \sigma_v') \tag{7}$$

$$r_d = 1.0 - 0.015z \tag{8}$$

$$k_s = C_Z \cdot C_G \cdot C_I \cdot k_{s0} \tag{9}$$

where R_1 is the resistance based on relative density, R_2 is the resistance based on D_{50}-values, R_3 is the resistance based on fine contents, N is the blow count by standard penetration test, D_{50} is the mean grain size (mm), F_c is the fines content (%), z is the depth from the actual ground surface (m), k_s is the seismic coefficient for evaluating liquefaction, C_Z is the seismic zone factor, C_G is the ground condition factor, C_I is the importance factor, k_0 is the standard seismic coefficient (= 0.15), σ_v is the total overburden (kgf/cm^2) and σ_v' is the effective overburden pressure under static conditions (kgf/cm^2).

In the system in this study, the seismic coefficient (K_s) is assumed to be estimated by equation (10) instead of equation (9).

$$K_s = \alpha_{max}/g \tag{10}$$

where α_{max} is the maximum acceleration of the ground surface estimated by equation (1) and g is the gravity acceleration (= 980cm/sec^2).

In this study, the liquefaction potential at the site of interest is defined with two indices : the total thickness of liquefiable layers with the F_L-Value less than 1.0 (H_{A3}) and the topographical ground condition. Based on experience in past earthquakes, the topographical ground conditions can be used as one of the indices to roughly estimate the liquefaction potential. The topographical ground conditions are classified into three groups as shown in **Table 2** [Ref. 4] . The liquefaction potential of a site of interest can be classified into one of three groups as shown in **Table 3.**

Group-A : Site/Zone with high liquefaction potential.
Group-B : Site/Zone with low liquefaction potential.
Group-C : Site/Zone with negligible liquefaction potential.

Method to Estimate the Lateral Spread of Ground by Soil Liquefaction

The lateral ground spread (flow/movement) induced by soil liquefaction has been investigated by the Public Works Research Institute [Refs. 5 and 6] and is being included as one ground hazard in the seismic information system. Because the simplified method to estimate lateral ground spread quantitatively is now under investigation, equation (11), which has been proposed empirically in past earthquakes, has been tentatively plugged into the pilot-system in this study [Ref. 7] .

$$D = 0.75\sqrt{H^3}\sqrt{\theta} \tag{11}$$

where D is the maximum lateral displacement of the ground surface (m), H is the thickness of liquefied layers (m) and θ is the maximum slope of the ground surface or of the lower boundary of the liquefied layer (%).

VISUALIZATION OF GROUND HAZARD

Information about the seismic ground motion, ground hazard and the damage of structures should be summarized simply for the urgent and rapid inspection and restoration of works after an earthquake disaster. Because graphically displayed information with the use of a computer system is very understandable, the visualization of actual and analysed information with a computer system is very helpful for people who work on earthquake disaster remediation. In this section, the visual information system, which has been developed as a pilot-system to investigate the methods to display information visually over the past few years at the Public Works Research Institute, is introduced using one case study. The above-mentioned methods to estimate the seismic ground motion and ground hazard are applied in the pilot-system.

Case Study Model Area for the Visual Information System

The case study model area for the visual information pilot-system is the Southern Area of Kanto-District around Tokyo-Bay in Japan. The map of the model area can be displayed on the computer-monitor with three kinds of scale according to their purpose as follows (each map is overlaid with grid lines which form rectangular cells):

Map A : This map indicates the wide sized area (of 158 km × 130 km) with 14 × 14 = 196 cells each of which is same size as the entire Map B.
Map B : This map indicates the middle sized area (of 11.3 km × 9.25 km) with 20× 20 = 400 cells each of which is same size as the entire Map C.
Map C : This map indicates the narrow sized area of 566 m × 466 m scale.

It is possible to select a cell of interest in Map A and B, and to display it as Map B and C respectively.

Example of Display in the Visual Information System

Typical examples of the display in the visual information pilot-system are shown in **Photos. 1 to 9.**

The hardware for the visual information pilot-system is composed of a memory apparatus, a display apparatus, a keyboard with a mouse, a printer and a mainframe computer with a map displayed as shown in **Photo. 1.** The system can be worked with a key or a mouse by selecting the menus shown on the display or inputting the necessary data.

The earthquake conditions, such as the epicentral location (longitude and latitude) and Magnitude (M) (which are reported by JMA just after an earthquake or estimated as a future earthquake), are necessary to be input as shown in **Photo. 2.** Furthermore, the seismic intensity reported at typical cities where JMA offices are located can also be input.

The maximum acceleration and the converted seismic intensity on JMA scale can be estimated with the use of the earthquake conditions above-mentioned by equation (1) and can be displayed on Maps A, B and C. **Photo. 3** shows the display for Map A. Based on this visual information on the distribution of seismic ground motion, it is possible to estimate the location and size of heavily damaged areas for rapid and appropriate action just after an earthquake. Furthermore, it is possible to easily understand which areas have a high potential for ground hazard problems in future earthquakes included in the earthquake disaster prevention plan.

The data base of ground groups and topographical ground conditions above-mentioned in equation (1) and **Table 2**, respectively, should be established in advance in order to be able to estimate the seismic ground motion by equation (1) and the total liquefaction potential defined in **Table 3**. **Photo. 4** shows the zoning of ground groups for each cell in Map B. The ground group has been determined with the use of typical boring data for each cell based on Seismic Design Specifications [Ref.3] . **Photo. 5** also shows the zoning of topographical ground conditions for each cell in Map B. The topographical ground condition has been determined by the average condition of the topography at each cell.

The liquefaction potential defined in **Table 3** can be displayed with three kinds of colored grids added to the objective cell. As shown in **Photo. 6**, the cell of Group-A with high liquefaction potential is displayed with a red grid and the one of Group-B with low liquefaction potential is with a green grid. Furthermore, the Group-C with negligible liquefaction potential is colorless. Based on these visual information, it is possible to estimate the area with high liquefaction potential easily and widely for both rapid and appropriate action just after an earthquake and for future plans against earthquake disaster.

The lateral ground spread estimated by equation (11) can be displayed with red-colored arrows for each cell as show in **Photo. 7**. The magnitude of ground spread can be shown with the length of the arrow and the direction of it is shown with the direction of the arrow. The thickness of the liquefied layer (H) at each cell in equation (11) is determined from the results of calculations of liquefaction resistance factor (F_L) defined by equation (2) with typical boring data. On the other hand, the maximum slope of the ground surface (θ) at each cell is defined as the maximum slope between the cell of interest and the eight cells around the cell of interest which is calculated from the average height at the center of each cell.

The detailed location of the structures of interest can be displayed in Map C as shown in **Photo. 8**. The epicentral distance, ground group and total liquefaction potential of Map C can also be displayed. When the data base of structural conditions for each structure is introduced in advance, the conditions of structures can be extracted from the data base and the damage conditions of structures reported after earthquakes can also be input in Map C. The damaged structures can be visualized to a larger scale in Map B as shown in **Photo. 9**. This scale is very useful for comprehending the distribution of damaged structures.

FUTURE VIEW

1. Because lifeline systems are located over a large region under the ground, seismic zoning is very effective for both seismic inspection and restoration action just after earthquakes and for disaster prevention programs for future earthquakes.
2. Although seismic zoning of ground motions and ground hazards has been described in this paper, further investigation is needed to establish a simple but rigorous method to quantitatively estimate the magnitude of liquefaction induced lateral spread of the ground in the future.
3. Because various damage of underground structures such as lifeline facilities is caused by various ground hazards, it is necessary to investigate the relation between these ground hazards and the damage level of lifeline facilities in order to create useful seismic zoning.
4. The visual information system with the use of a computer system is very useful for people who work on earthquake disaster prevention so that they can understand the seismic information and data rapidly and adequately.
5. For establishing the useful zoning of seismic ground hazard and induced-damage of structures, fundamental data on the ground and structures is needed. Estimates of the ground hazard and damage of structures should be established in advance.

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Table 1 Application of seismic ground motion and hazard during earthquakes.

- 1) Urgent and Rapid Inspection and Restoration Just After Earthquakes.
  - Based on
    - (1) Actual Distribution of Seismic Ground Motion.
    - (2) Estimated Distribution of Seismic Ground Motion and Hazard.
    - (3) Estimated Damage of Structures, Especially Underground Structures Induced by Ground Motion and Hazard.
- 2) Planning of Earthquake Disaster Prevention in Future Earthquakes.
  - Based on
    - (1) Estimated Distribution of Seismic Ground Motion and Hazard.
    - (2) Estimated Damage of Structures, Especially Underground Structures Induced by Ground Motion and Hazard.

Table 2 Relation between topographical ground condition and liquefaction potential.

Liquefaction Potential	Geographical Conditions
(A) Highly Possible to be Liquefied	Existing Channel, Existing River Bed, Old Channel, Old River Bed, Lowland between Sand Hills, Reclamation of Old Water Surface (Reclaimed Area of Seashore, Lake-shore, Paddy Field, etc.)
(B) Possible to be Liquefied	Hinter Wet Land, Natural Bank, Sand Hill, Sandbar, Sandy Beach, Other than the above (A) and below (B)
(C) Not Easily to be Liquefied	Plateau, Hill, Fand Land

Table 3 Simplified method for estimating liquefaction potential.

Topographical Ground Condition	Thickness of Liquefiable Layer (m)		
	$0 \leqq H_{AS} \leqq 2$	$2 \leqq H_{AS} \leqq 5$	$5 \leqq H_{AS} \leqq 20$
I	B	A	A
II	C	B	A
III	C	C	B

(Note) **A** :Site/Zone with High Liquefaction Potential
B :Site/Zone with Liquefaction Potential
C :Site/Zone with Negligible Liquefaction Potential

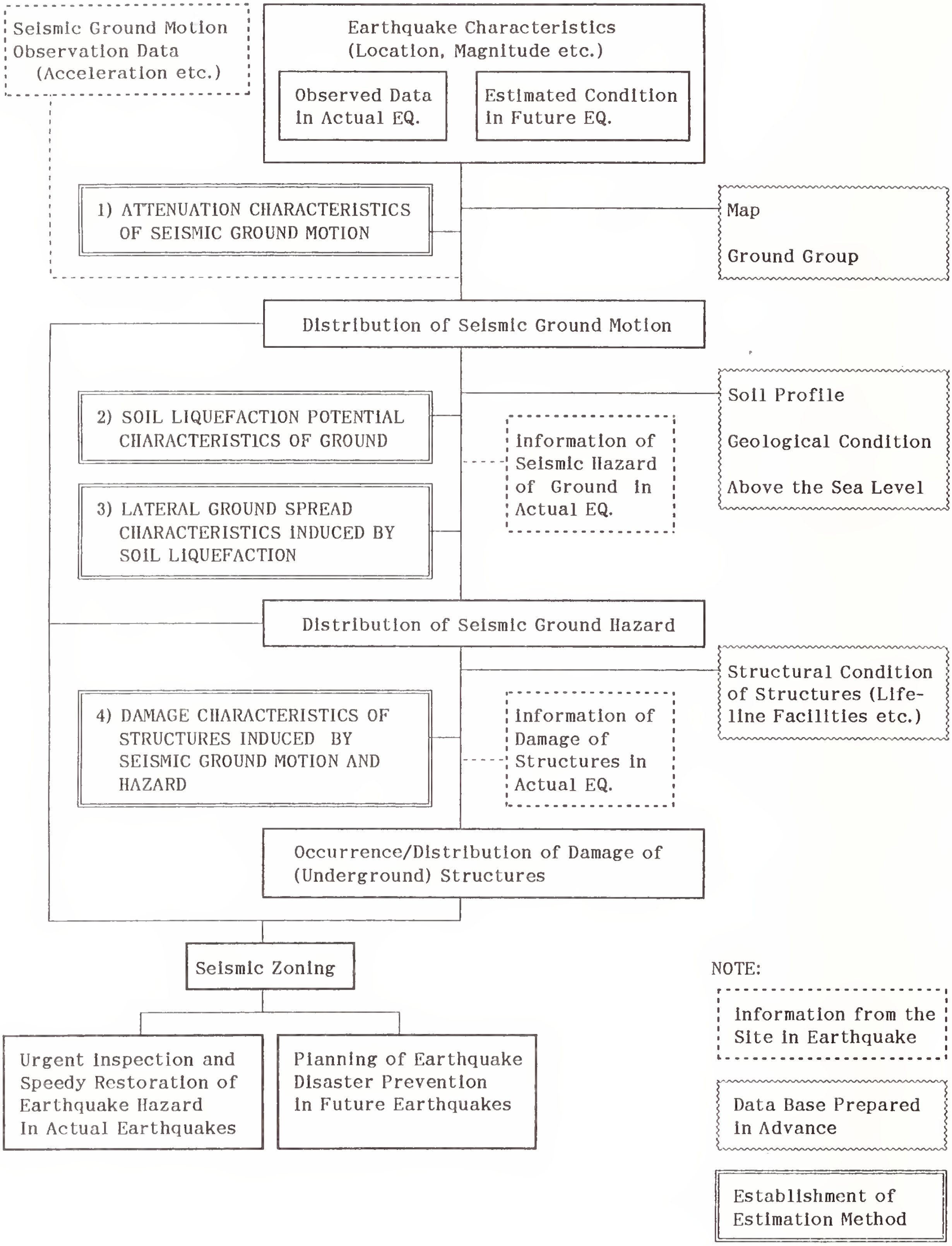

Fig. 1 Flow of information related to seismic ground motion/hazard and induced-damage of structures.

Photo. 1 External appearance of visual information pilot-system.

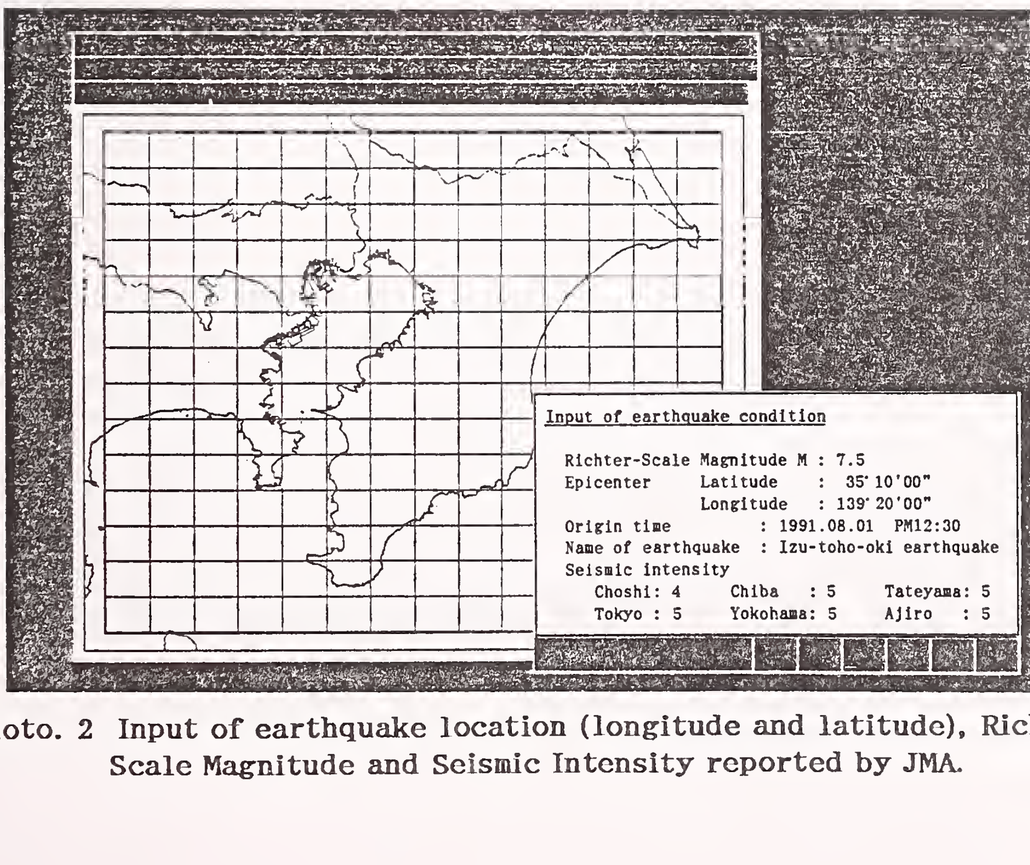

Photo. 2 Input of earthquake location (longitude and latitude), Richter-Scale Magnitude and Seismic Intensity reported by JMA.

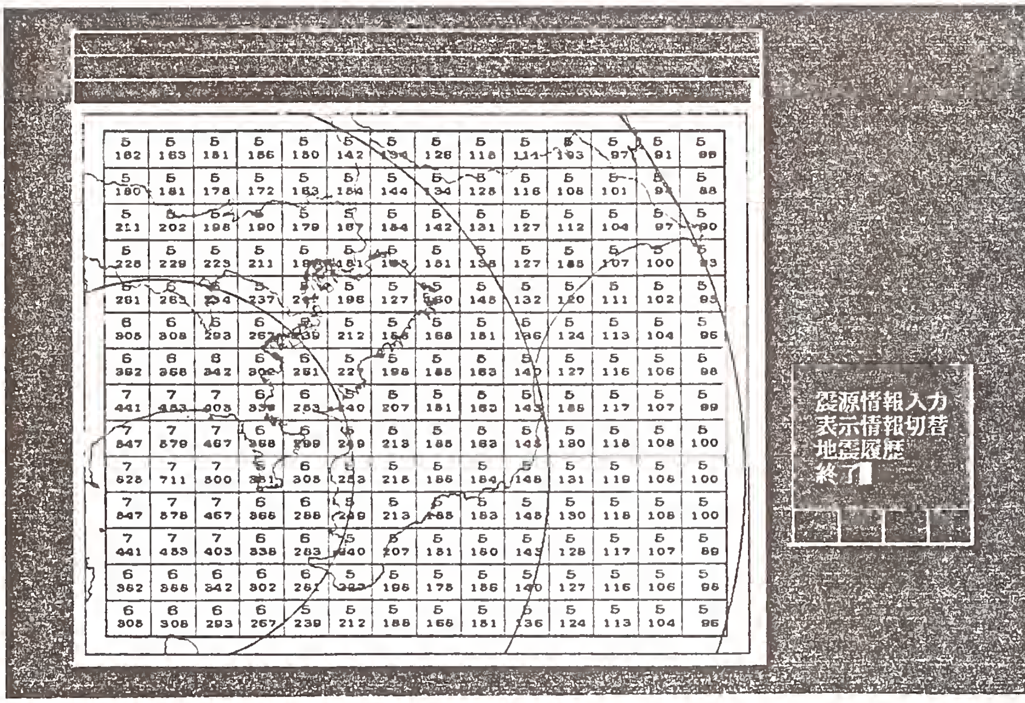

Photo. 3 Display of maximum ground surface acceleration and Seismic Intensity estimated in Map A (Southern-Area of Kanto-District : 158km×130km, 196 cells).

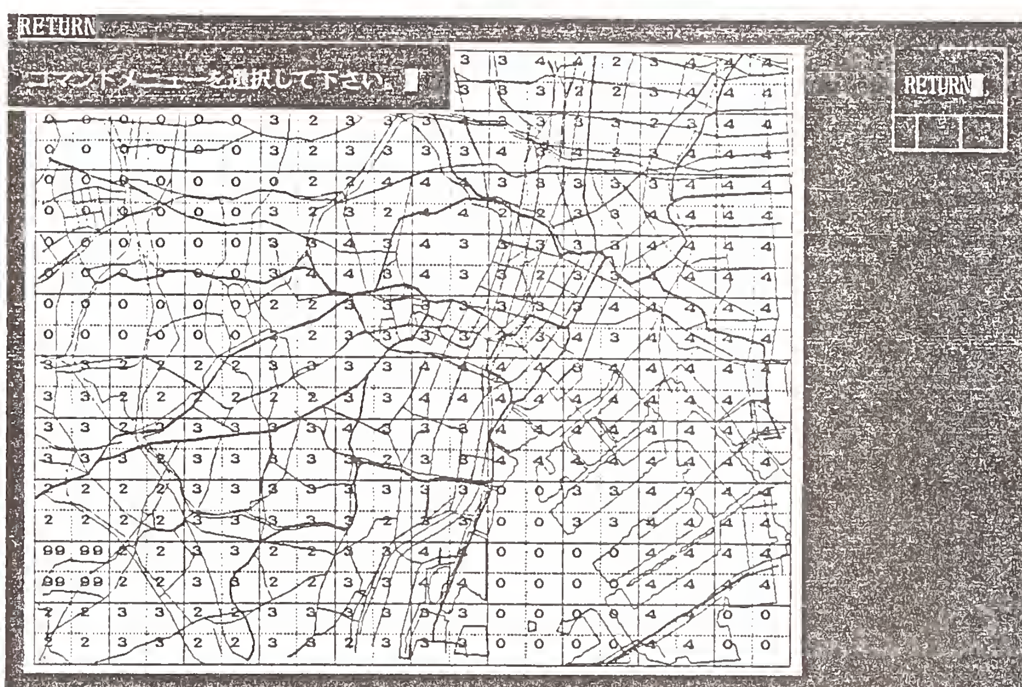

Photo. 4 Display of Map B on the ground groups estimated from actual ground data to estimate maximum ground surface acceleration.

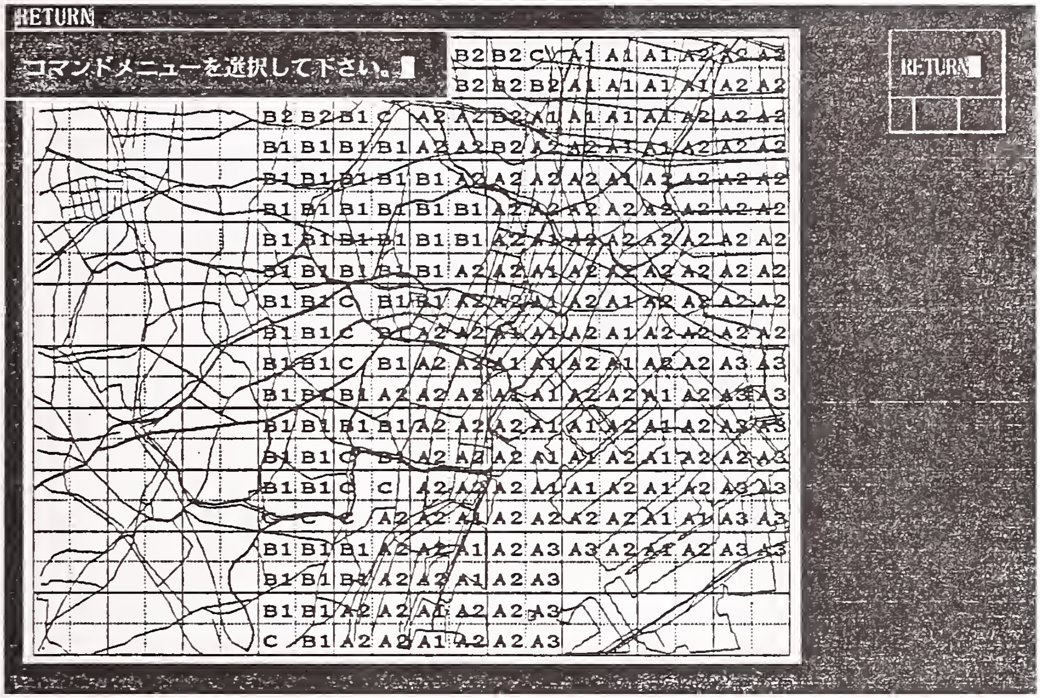

Photo. 5 Display of Map B on the topographical ground conditions to estimate liquefaction potential.

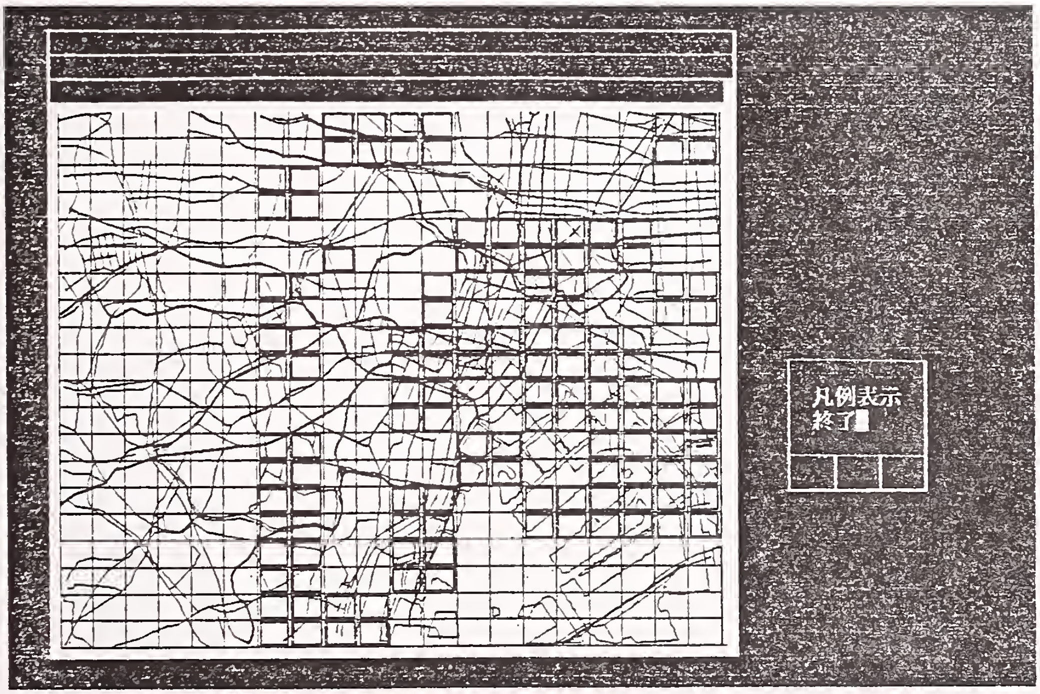

Photo. 6 Display of Map B on the liquefaction potential estimated for each cell (Group-A : Red grid added, Group-B: Green grid added, Group-C : Not colored grid).

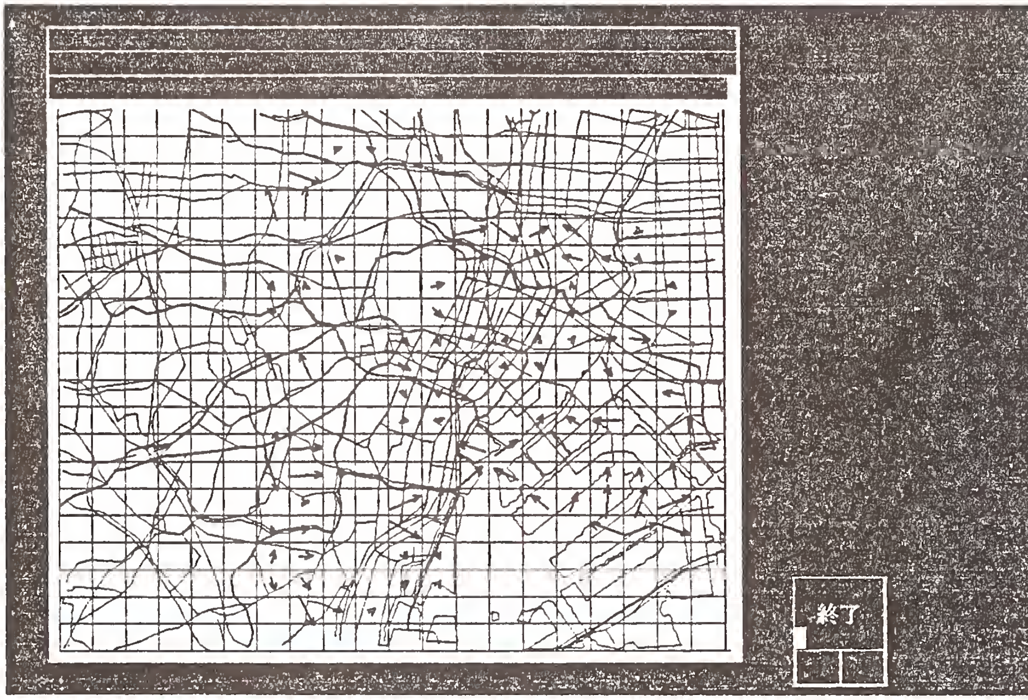

Photo. 7 Display of Map B of the magnitude and direction of maximum lateral spread of ground for each cell.

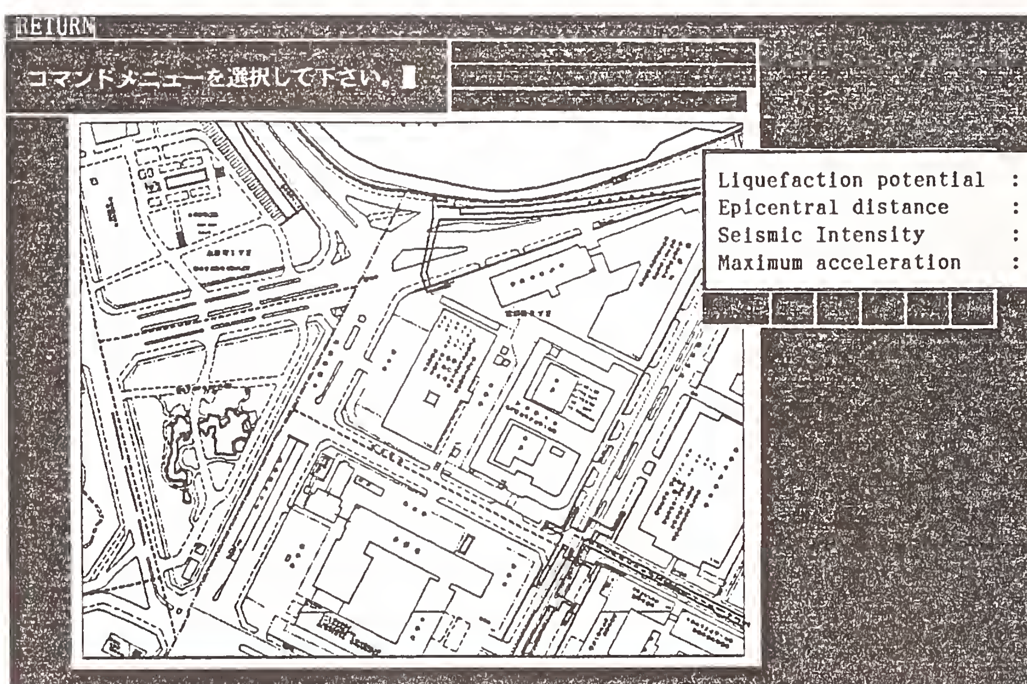

Photo. 8 Display of Map C of the location of structures and liquefaction potential.

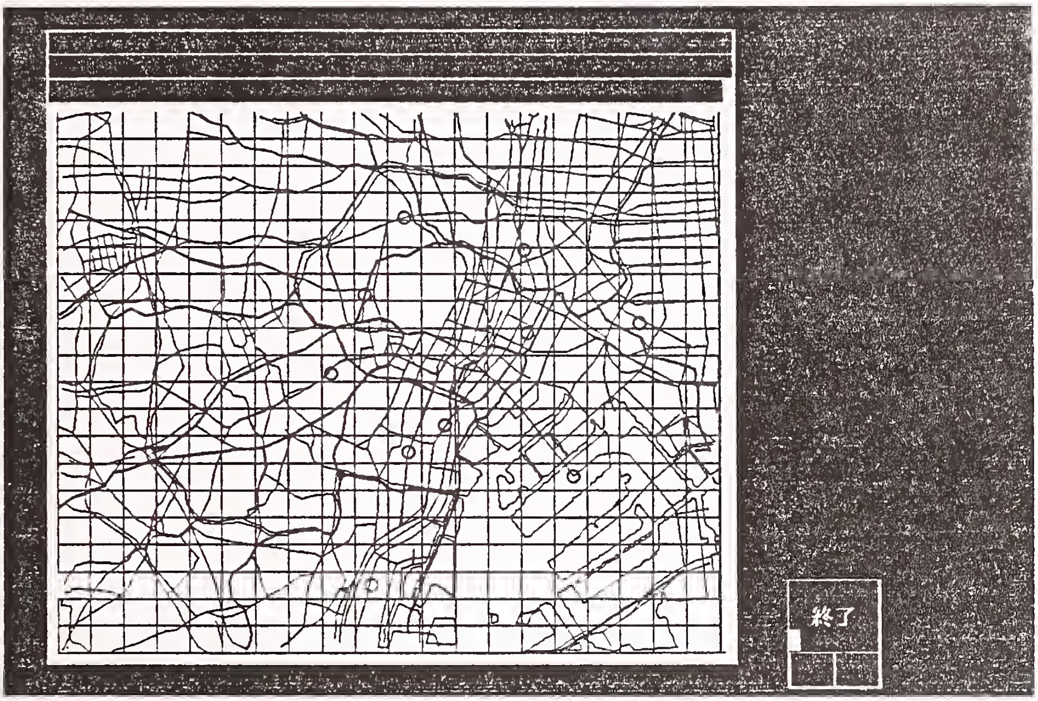

Photo. 9 Display of Map B on the location of damaged structures reported from sites.

II. SEISMIC DESIGN AND ANALYSIS: TRANSPORTATION

SEISMIC RISK IDENTIFICATION & PRIORITIZATION IN THE CALTRANS SEISMIC RETROFIT PROGRAM

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ABSTRACT

The procedure used by the California Department of Transportation in the State's Seismic Retrofit Program to identify and prioritize the bridges most vulnerable to collapse due to large earthquakes is presented in this paper. A risk analysis is developed in which expert judgements replace a typically large statistical database. The result is a risk analysis based on expert knowledge gained from past earthquakes and bridge characteristics which can be applied quickly to a decision making process. Brief comments on project classification and projected project dates are included for completeness. Proposed future objectives and modifications to the procedure are discussed.

INTRODUCTION

The 1971 San Fernando earthquake exposed a number of deficiencies in the bridge design specifications of that time. The bridge design specifications were modified to correct the deficiencies and a seismic retrofit program was started to address the various deficiencies of the structures in place. The initial objective of the seismic retrofit program was to insure continuity at all of the bridge superstructure joints to prevent drop-type failures. Using 55 million dollars of state funds approximately 1,300 bridges were retrofitted. Typical methods used were to add restraining cables or rods at piers and hinges and shear keys at abutments and bearings. In 1987, shortly after the Whittier earthquake, 64 million dollars of state funds were made available for additional retrofitting. These funds were to be directed toward the retrofit of structures with single column bents, as they were perceived to be less redundant and therefore at higher risk. This effort was underway when the Bay Area of California was rocked by the Loma Prieta earthquake on October 17, 1989.

CALTRANS has received specific direction in response to the Loma Prieta Earthquake. Following the Loma Prieta Earthquake, Senate Bill No. 36 {1} was passed which appropriated 80 million dollars immediately, and potentially a significantly greater amount for future seismic retrofit projects. This bill identified Caltrans as the lead agency for inspection and retrofit if necessary for all publically (i.e., local and state) owned bridges throughout the state, except for those bridges not on the state highway system in the counties of Los Angeles and Santa Clara, in which cases the respective counties are designated as the lead agencies. The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake {2} stated in a list of recommended actions, ".... specific goals of this policy shall be that all transportation structures be seismically safe and that important transportation structures maintain their function after earthquakes." With this direction and funding, CALTRANS has been charged with the task of providing safety to the travelling public through necessary retrofit modifications to any transportation structures which may not be adequate during a large earthquake.

CALTRANS is responding to these directions. Figure 1 illustrates the strategy adopted within the Division of Structures to identify and prioritize, group into projects, and submit to design bridge structures to be retrofitted.

There are some 23,000 (approximately 11,000 state and 12,000 city or county) bridges in the state of California (see Figure 2) that CALTRANS is responsible for in some capacity threatened by over 200 faults (see Figure 3). It was and still is economically unrealistic to suggest every structure be immediately retrofitted to withstand large magnitude earthquakes without some damage. A retrofit philosophy was adopted at the start of the seismic retrofit program which offers reasonable direction.

It is CALTRANS philosophy to first retrofit those structures which are at greatest risk and are the most vital. The ultimate goal is to see that all of the bridges in the state are capable of surviving large earthquakes. Some damage is inevitable but collapse is believed to be preventable with proper retrofitting. The exception to this is in the case of lifeline structures. If undue stress or hardship may be place on a community due to a structure being temporarily out of service the structure should be made to withstand even the maximum level earthquake and remain in service.

IDENTIFICATION & PRIORITIZATION

Identification of bridges likely to sustain damage during an earthquake is an essential first step in a retrofit program. What can be classified as a level one risk analysis was employed as the framework of the process which led eventually to a consensus list of prioritized bridges.

A conventional risk analysis determines a probability of failure or survival. This probability is derived from a relationship between the load and resistance sides of design equations. Not only is an approximate value for the absolute risk determined, but relative risks can be obtained by comparing determined risks of a number of structures. Such analyses generally require vast collections of data to defined statistical distributions for all or at least the most important elements of some form of analysis, design, and/or decision equations. The acquisition of this information can be extremely costly if obtainable at all. Basically, what is typically done is to execute an analysis, evaluate both sides of the relevant design equation, and define and evaluate a failure or survival function. All of the calculations are carried out taking into account the statistical distribution of every variable throughout the entire procedure.

To avoid such a large questionable investment in resources and to obtain result which could be applied quickly as part of the seismic retrofit program, an alternative was recognized and developed. What can be called a level one risk analysis procedure was used. A similar risk evaluation to identify and prioritize bridges for retrofitting was used in the single column retrofit program {3}. The difference between a conventional and level one risk analysis is that in a level one risk analysis expert judgements take the place of data supported statistical distributions.

The level one risk analysis procedure employed can be summarized in the following steps:

1) Survey the available expert database to identify and weight high risk structural and transportation characteristics,

2) Define preweight scoring schemes,

3) Calculate bedrock accelerations at all bridge sites,

4) Identify high risk soil sites which possess the capacity to liquify or substantially amplify bedrock accelerations, and

5) Prioritize bridges by summing weighted bridge structural and transportation characteristics.

Step 1 was performed by surveying experts in the fields of bridge design, maintenance, and construction; and geotechnical and geological sciences. Typically, bridge structural and transportation characteristics with high correlations to past earthquake bridge damage would be used to identify variable components of a risk evaluation. Due to the absence of a substantial database of maximum credible California earthquakes and their effects on California bridge structures and transportation systems, an alternative database was identified and tapped. Expert judgements by professionals in the field of bridge engineering were used to derive and calibrate a risk algorithm. The survey requested which characteristics would have high correlations to bridge damage or cost to public transportation and what their relative correlations would be. This panel of experts represented hundreds of years of bridge experience. The result of this step is illustrated in Figure 4.

Preweight scoring schemes were developed in step 2. These were developed using engineering judgement considering what data were available, their form, and engineering/mechanical relationships between the particular characteristic and typical structural or transportation system responses. Each preweight score is a number between 0.0 and 1.0. A number close to 0.0 reflects a relatively low risk and a number close to 1.0 reflects a relatively high risk. A typical preweight scoring scheme is presented in Figure 5.

Step 3 was carried out by consulting the California Division of Mines and Geology. A team of seismologists and engineers identified seismic faults believed to be the sources of future significant events. Selection criteria included location, geologic age, time of last displacement (late quarternary and younger), and length of fault (10 km minimum). Each fault was evaluated for style, length, dip, and area of rupture surface in order to estimate potential earthquake magnitude. Fault locations were digitized. An appropriate attenuation model was developed by Mualchin of the California Division of Mines and Geology to be used throughout the state. It is a weighted average of several published models.

These two efforts combined to produce a method for determining the maximum credible peak bedrock acceleration at the site of each state and locally owned bridge in California. This is achieved by attenuating maximum credible bedrock accelerations from the closest fault to the bridge site. This is greatly simplified with Map Sheet #45 by Mualchin of the California Division of Mines and Geology {4}. This map is effectively a map of maximum credible peak bedrock accelerations throughout the state of California.

Top CALTRANS engineering geologists from throughout California collaborated to complete step 4. A knowledge base constructed by years of studying and working with the geologic strata of California was thus made available to the Division of Structures. The team of engineering geologists, working with 26 geologic maps, identified high risk soil sites which possess the potential to liquify and/or substantially amplify bedrock accelerations. The identified high risk soil sites were then digitized for computer use.

Step 4 is the process of combining the previous efforts via a logical, dependable, and repeatable algorithm which can be computerized. This was performed with a graphical interface system by ULTIMAP at CALTRANS. A final single risk number for each structure was calculated by summing the products of risk algorithm weights and each structure's preweight scores, producing a numerical measure of relative risk between 0.0 and 1.0. An example calculation is included in the appendix of this paper. An indepth description of each component and each preweight scoring scheme is presented in another paper by Maroney {5}. Figure 6. illustrated the distributed of assigned risk numbers to state and local bridges.

DATA COLLECTION

Acquisition of the bridge structural and transportation data was a formidable task. The CALTRANS Structures Maintenance database was relied upon heavily. The database is a federally mandated library of records which describe a variety of structural, transportation, and economic bridge data. However, any single source of information cannot be relied upon alone. Parallel information seeking efforts were initiated to gather the best available information on all state and locally owned bridges. These additional efforts include a solicited Seismic Retrofit Inventory (SRI) survey of locally owned bridges and a state wide General Plan (GP) review of all bridges, and a collection of special knowledge on selected structures which engineers state wide are identifying as threatened.

The SRI forms requested local agencies to field survey their structures and return to CALTRANS data which would assist in evaluating their potential need for retrofit. An SRI form is included in the appendix of this paper. All fifty-eight counties and over 300 cities have completed approximately 12,000 SRI forms in 18 months. Approximately 100 forms are outstanding and 700 forms have been received that do not match any existing records at the writing of this paper. CALTRANS processed, corrected if necessary, and entered the returned form's information into a database. This information is serving as a vital element in the retrofit program.

The GP review utilized engineers examining the general plan for every structure CALTRANS has plans for in its archives. The goals of this effort were twofold. The first was to remove all structural types from the retrofit program which have proven to be not susceptible to catastrophic failure due to earthquakes. These kinds of structures include: modern bridges designed since 1980 without outrigging knee joints, flat slab, timber, single-span monolithic, typical two-span monolithic, and well-seated single-span bridges. This effort reduced the number of bridges which required a more detailed review or analysis. The second reason for the GP review was to serve as a check for bridge identification and database quality. This proved valuable in cases in which bridge structures had been replaced, renamed, or renumbered. Included in the appendix are the GP seismic review data sheet and the detailed seismic review data sheet. It should be noted that the data sheets were modified as new conditions were encountered which suggested modification. Approximately 9000 state and 4000 locally owned bridge GPs have been reviewed. Over 2000 detailed seismic reviews have been performed.

Special knowledge pertaining to bridge structures throughout the state has proven valuable. The Division of Structures maintains an open door policy for engineers and professionals to contribute to the effort in identifying high risk structures through special knowledge of structures or site characteristics which they have gained through professional experience with a bridge or site in the state of California.

In order to respect previously committed time schedules, the bridge plan review effort has two levels of review. They are the GP review discussed earlier and a detailed seismic review which is providing an opportunity to remove a large number of bridges from the retrofit program by investigating the structural plans and details. Such details would include support widths, column reinforcement, footing reinforcement, bearing type, etc.... The detailed review will also allow reviewers to take advantage of additional knowledge gained from recent retrofit structural analyses. This review is taking place before and following the assignment of the risk values.

It is recognized that certain structures on the state transportation system can be identified as having unusually high levels of risk associated with them. Examples include: structures with rigid outrigger bents, structures with leased airspace below, structures spanning faults, and structures on routes which can be categorized as lifeline arteries. These structures are being identified by numerous methods and will be appropriately analyzed, and if necessary retrofitted.

PROJECT CLASSIFICATION

Identified bridges are being grouped into projects. The total inventory of bridges is separated first into three groups respecting the three lead agency jurisdictions and then further by the results of the GP review effort. Respective lists were provided to the state, and Santa Clara and Los Angeles counties with seismic risk factors associated with all bridges. Methods employed to oversee the retrofitting of the bridge structures at the project level in the counties is the responsibility of the respective lead agencies. Each of the bridge structures which fall under CALTRANS jurisdiction not eliminated by the GP review will be assigned a seismic risk factor. The risk factors will be used to prioritize the bridges.

Project grouping considers ownership, number of bridges, estimated costs, and geographic locations. All bridges in any single project are owned by the same city or county. The maximum number of bridges in any one project is 20. (It is estimated that 50% of these will usually be eliminated in a detailed final screening. Maximum estimated costs per project do not exceed $4 million. Reasonable geographic limitations are used to group bridge sites into projects.

Each project is prioritized. Projects are assigned a Seismic Project Priority Number (SPPN). The SPPN is the average of each bridge;s risk factor in a project weighted by each respective bridge's estimated retrofit construction cost. It is estimated that project prioritization will be completed in 1991.

FUTURE OBJECTIVES

The seismic retrofit program is currently underway and is based in part on an identification and prioritization program which employed the best available resources of the time and good engineering judgement. These resources can be improved and the results enhanced through data collection and relatively simple research. Some of the areas targeted for refinement include:

1) improvement of the dataset,

2) hazard mapping,

3) documentation of structural sensitivity by class,

4) identification of lifeline refinement,

5) calibration procedure refinement,

6) transformation of risk model from additive model to multiplicative model,

7) higher level risk analysis, and

8) sensitivity study of model weights.

Any decisions or judgements made based on data are dependent on the quality and quantity of the supporting data. Though the dataset available to CALTRANS is large it can and should be enriched.

Fault data will be enhanced by the processing of data which will facilitate the assignment of additional fault characteristics such as mechanism, depth, orientation, projection, and probable frequency content and return periods. With a more comprehensive fault dataset Caltrans will be poised to develop, evaluate, and thereby consider probability of seismic events if necessary. With such data organized, maps illustrating maximum credible and probable event responses are a natural by-product.

Processing and refinement of soil data will be a high priority. CALTRANS maintains boring logs and driving records for nearly every state bridge. These logs contain information which could be used to predict soil and structure response to an earthquake. This data is not presently accessible by automated means. However, an effort is underway to facilitate electronic access to such date.

Structural data could be enhanced by including superstructure seat widths and locations, design accelerations and Z factors, column heights and/or stiffness variations, etc... Transportation data could be enhanced by including local input to identify lifeline corridors which must be kept open in the event of a major earthquake (e.g., a bridge on the sole route leading to a community hospital).

The present risk model was calibrated from the response of a survey sent to qualified experts. Though this surveyed was an effective and efficient method of calibration, some iterative and personal interaction would certainly improve the calibration scheme. A conference is planned to provide technical interaction through sharing of ideas and perspectives toward refinement of the risk model calibration.

The risk model is to be transformed from an additive to a multiplicative model. That is, from the form of

$$R=\sum_{i=1}^{n}\left[(weight_i)(pre\text{-}weightscore_i)\right]$$

to the form of

$$R=\prod_{i=1}^{m}\left[(macroweight_i)\left(\sum_{j=1}^{n}(microweight_j)(pre\text{-}weightscore_j)\right)_i\right]$$

This will allow structures which have high risk structural and transportation characteristics to be excused more easily (i.e., via automation) from the retrofit program if their seismic hazard is low. It should be noted this will also required very precise fault information.

Higher level risk analyses could be implemented. The present analysis produces a measure of relative risk. If resources are made available, future analysis could produce measures of absolute risk or probabilities of damage or impact to the state or local communities.

Judgements made in risk evaluation will be investigated through a sensitivity study. The algorithm component weights will be varied and the variation in the resulting distribution will be analyzed. There exists an infinite number of combinations for the weighting scheme. It is anticipated that structures built before 1980 will be evaluated respecting only hazard, then respecting only structural characteristics, and then only transportation characteristics.

SUMMARY

Past earthquakes have exposed potentially threatening deficiencies in California bridges. CALTRANS recognized this situation as early as 1971 and immediately initiated a retrofit program. However, modest state funding of the program limited its scope to high potential areas. The Loma Prieta Earthquake brought attention to the program. And with legislation, and recommendations from the Governor's board of inquiry, CALTRANS is being directed to accelerate the seismic retrofit program. Important initial steps to the strengthened program are to identify and prioritize bridge structures which possess a probability of server damage in a maximum credible earthquake. This is currently underway at CALTRANS using what can be termed a level one risk analysis. The risk analysis employs an expert knowledgebase and good engineering judgement to produce a prioritized list of bridges which will be grouped into projects and released for design and construction. The risk analysis program is an ongoing effort to identify potentially catastrophic and functional failures in the California transportation system. As additional and more accurate incorporated, the program's evaluations will mature in accuracy and precision.

FIGURES

SEISMIC RETROFIT PROJECT FLOWCHART

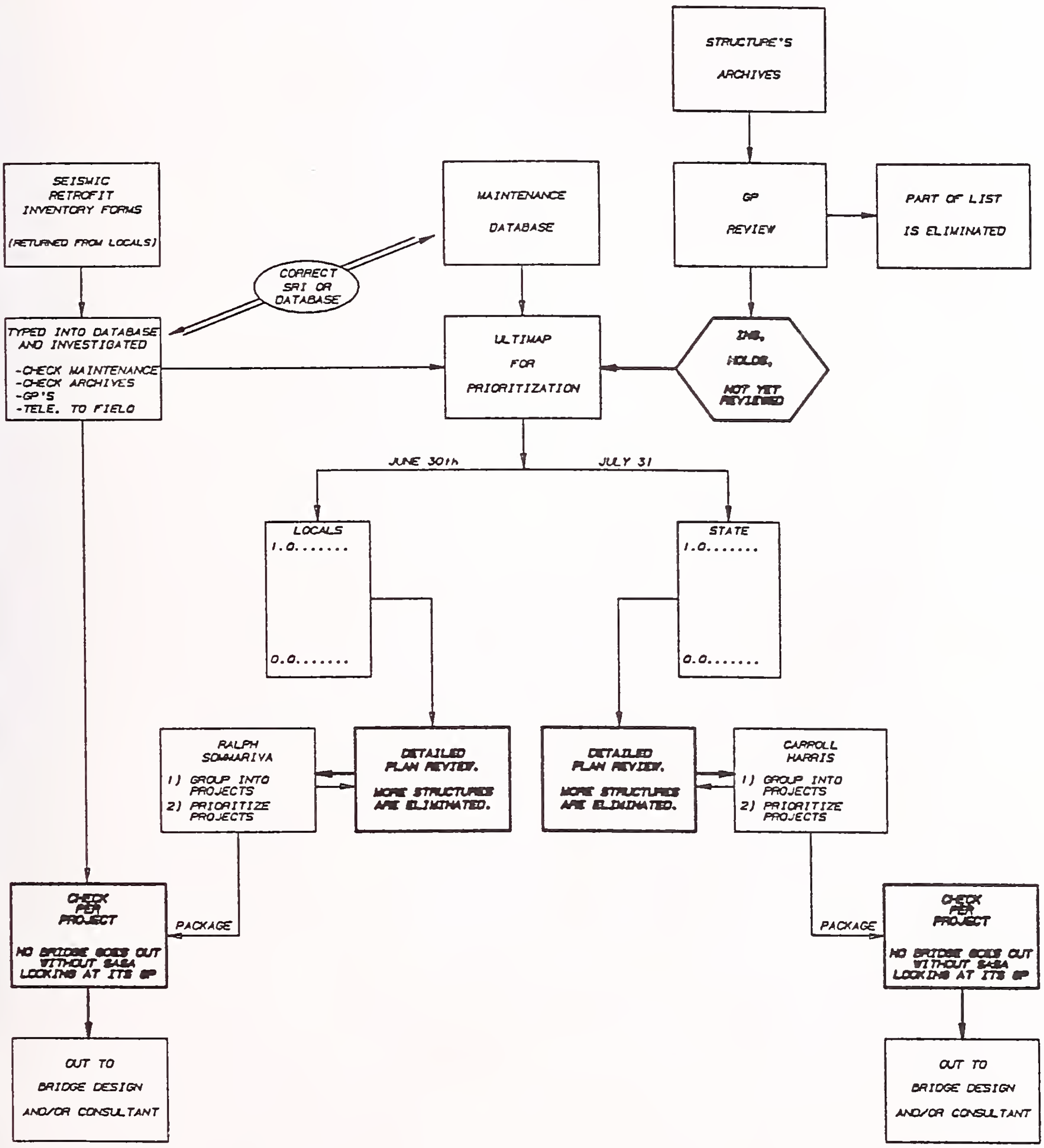


Figure 1. Program Flowchart

PHASE II RETROFIT

Figure 2. State of California Bridges

PHASE II RETROFIT

Figure 3. State of California Seismic Faults

RISK ALGORITHM

$$\boxed{R = \sum[(wt)\bullet(pre\text{-}wt)]}$$

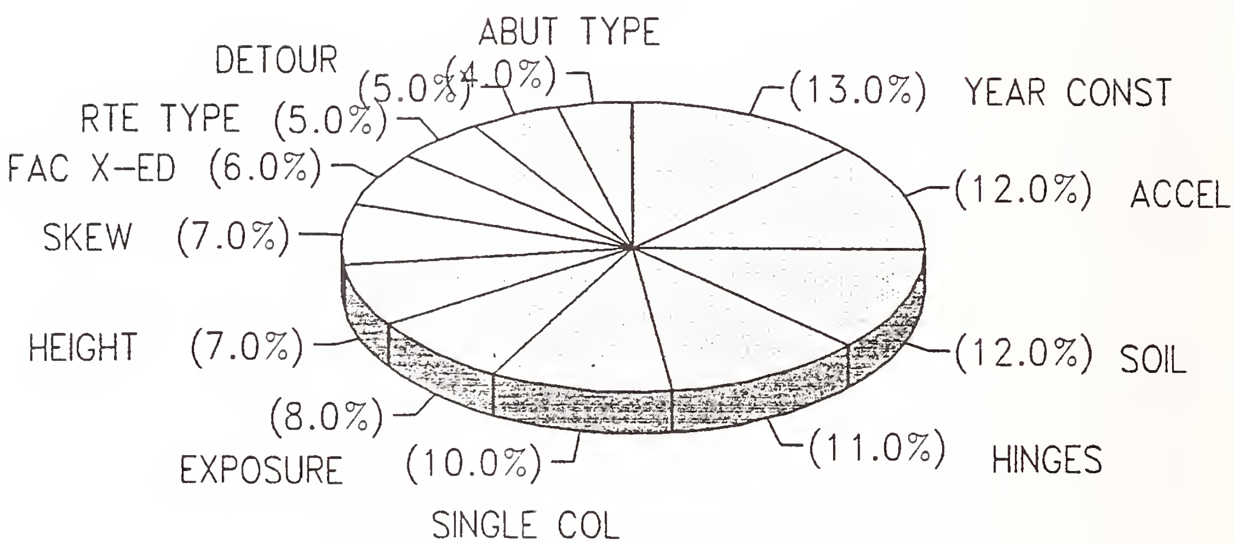


Figure 4. Prioritization Weights

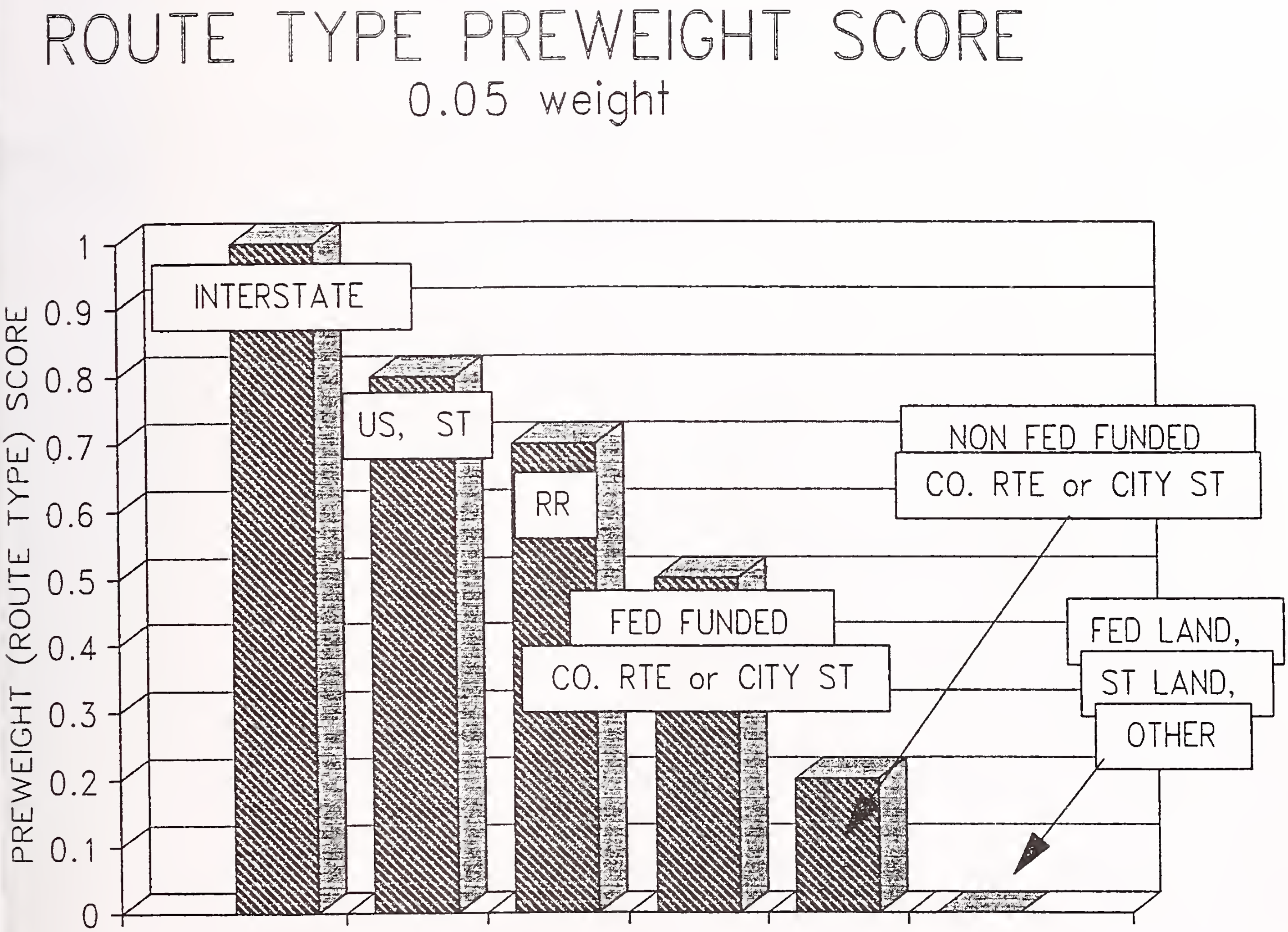


Figure 5. Example Preweight Function

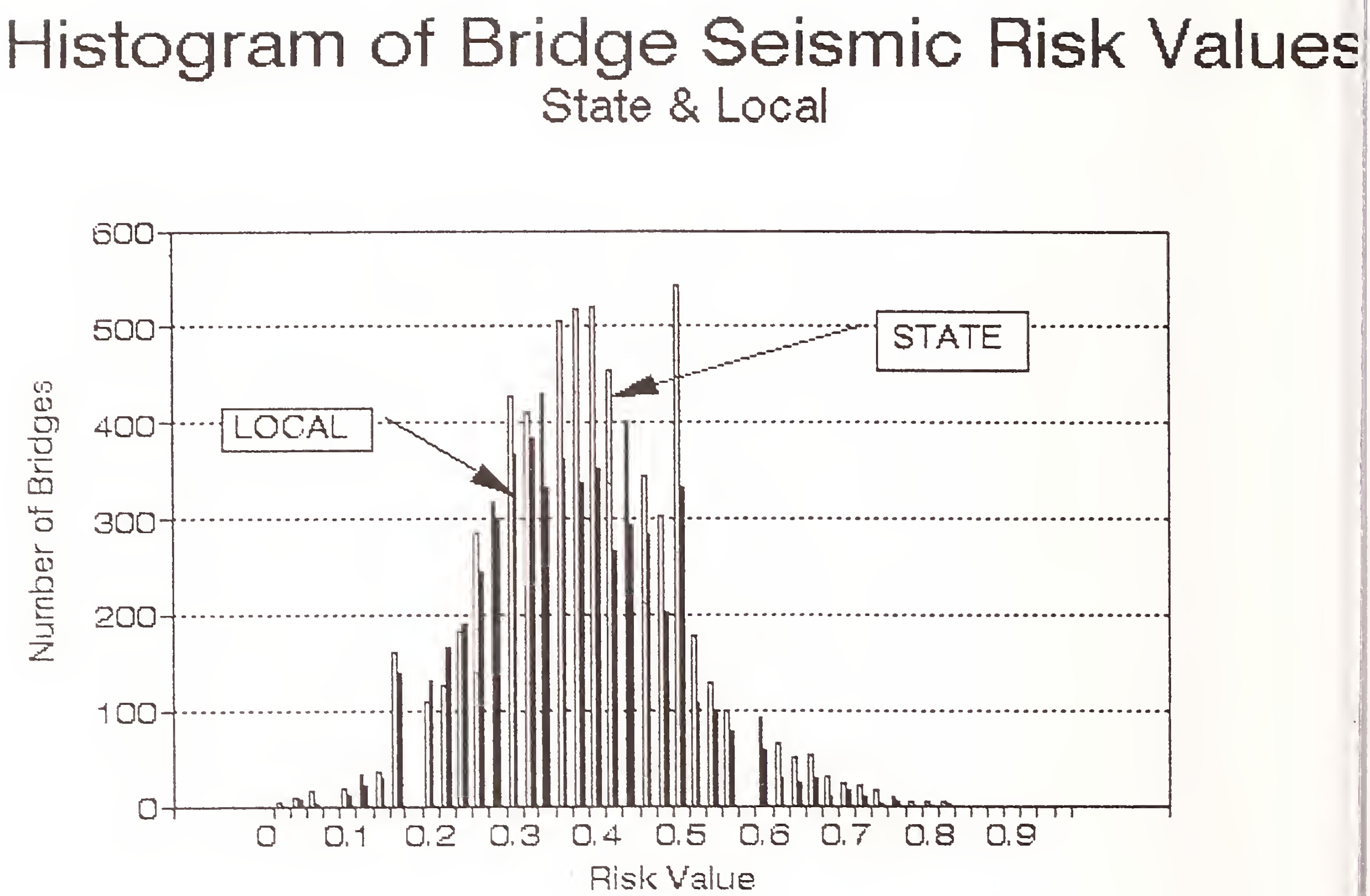
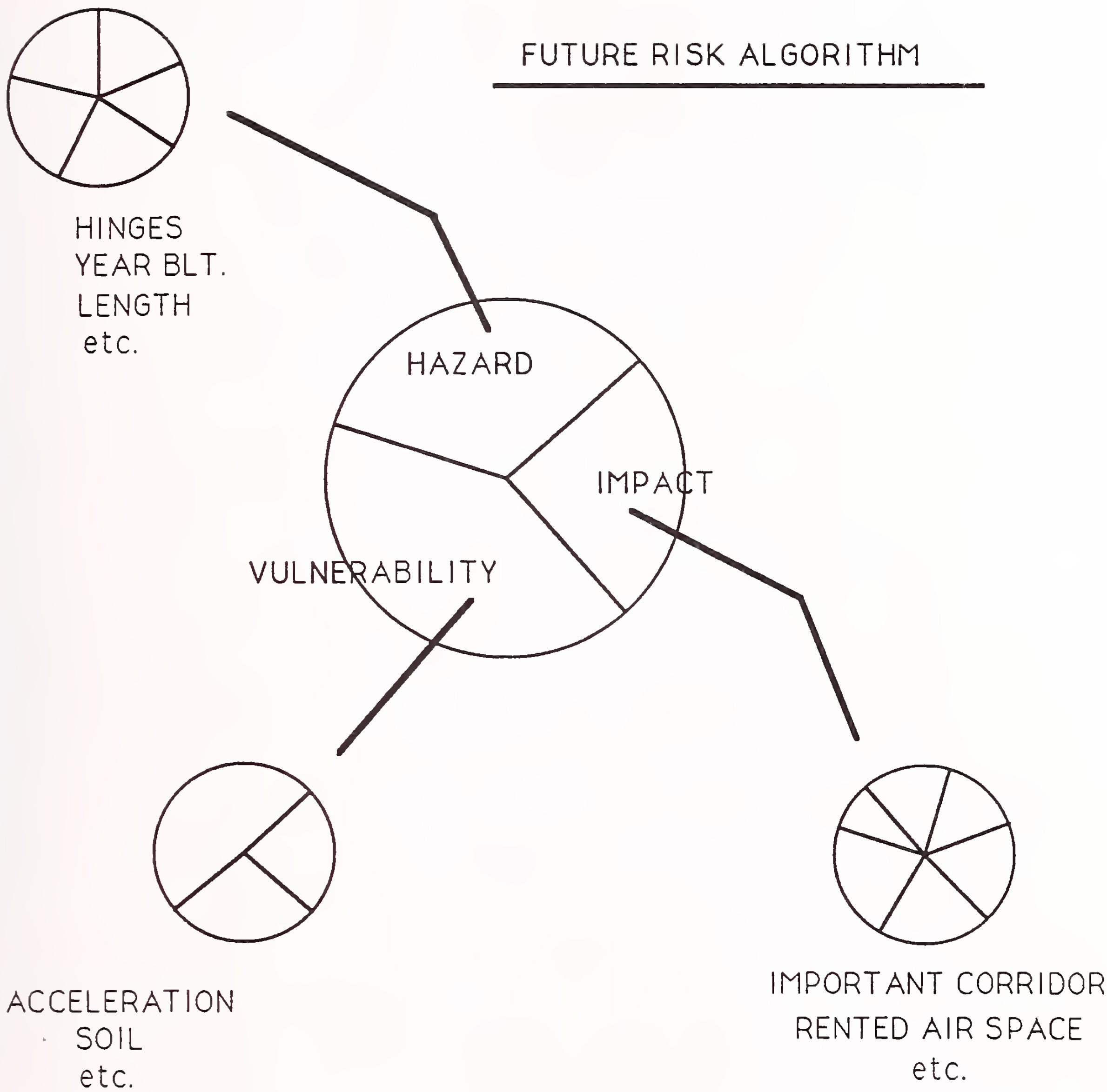


Figure 6. Prioritized Bridge Distribution



$$\prod_{i}^{m}\left[\left(\text{macro weight}_i\right)\left(\sum_{j}^{n}\left(\text{micro weight}_j\right)\left(\text{pre-weight score}_j\right)\right)_i\right]$$

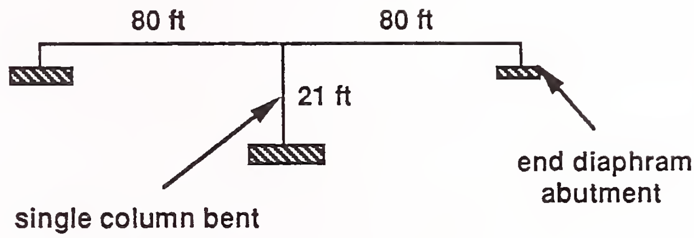
Figure 7. Probable Form of Future Risk Algorithm

APPENDIX

RISK ALGORITHM EXAMPLE

RISK RATING EXAMPLE:

GIVEN :



-- YEAR DESIGNED	1955
-- PEAK BEDROCK ACCELERATION	0.45g
-- HIGH RISK SOIL SITE	not in a high risk soil zone
-- # OF HINGES	0
-- COLUMNS / BENT	1
-- ADT ON BRIDGE	20,000 $\frac{\text{veh}}{\text{day}}$
-- HEIGHT	21 ft (SRI form calls out 20 - 30 ft)
-- LENGTH	160 ft
-- SKEW	15 degrees
-- FACILITY CROSSED	U.S. route
-- ROUTE TYPE	county route
-- DETOUR LENGTH	10 miles
-- ABUTMENT TYPE	end-diaphragm

COMPONENT		
-- YEAR DESIGNED	0.13 * (1.0)	0.13
-- PEAK BEDROCK ACCEL.	0.12 * ($\frac{.45g}{.7g}$)	0.077
-- HIGH RISK SOIL SITE	0.12 * (0)	0.0
-- # OF HINGES	0.11 * (0)	0.0
-- COLUMN/BENT	0.10 * (1.0)	0.1
-- TRAFFIC EXPOSURE	0.08 * (20000 $\frac{veh}{day}$ * 160 ft = 3.2* 10^6 ==> 0.0317) see TRAFFIC EXPOSURE PREWEIGHT CURVE	0.0025
-- HEIGHT	0.07 * (0.995) if height available use HEIGHT PREWEIGHT CURVE if height not available use length to estimate height In this case SRI form is the best available data, and an average for the 20-30 ft range is used.	0.07
-- SKEW	0.07 * (0.025) see SKEW PREWEIGHT CURVE	0.0018
-- FACILITIES X-ED	0.06 * (0.8) U.S. route crossed	0.048
-- ROUTE TYPE	0.05 * (0.5) county route on bridge	0.025
-- DETOUR LENGTH	0.05 * (0.1) see DETOUR PREWEIGHT CURVE	0.005
-- ABUTMENT TYPE	0.04 * (0) monolithic	0.0

RISK RATING ==> $\sum =$ 0.46

RISK ALGORITHM

YEAR CONSTRUCTED	0.13 *	(0.0 --> yr >71; 0.5 --> yr<=45; 1.0-->45<yr<=71)
PEAK ROCK ACC.	0.12 *	(MCE acc, normalized to 0.7g)
SOIL AT SITE	0.12 *	(0.0 --> low risk site; 1.0 --> high risk site)
# OF HINGES	0.11 *	(0.0 --> 0; 0.5 --> 1; 1.0 --> 2 or more)
COLUMNS PER BENT	0.10 *	(0.5 --> multi-col; 1 --> single col)
TRAFFIC EXPOSURE (length & ADT on deck)	0.08 *	(neg. parabola, normalized to $2*10^8$ ADT*LENGTH))
HEIGHT (length)	0.07 *	((LOCAL) neg. cubic, normalized to 30) ((STATE) 0.0 --> 0-300; 0.5 --> 300-600; 1.0 --> >600)
SKEW	0.07 *	(pos. parabola, normalized to 90)
FACILITY CROSSED	0.06 *	(same as RTE TYPE, STREAM = 0.8)
ROUTE TYPE (on structure)	0.05 *	(INTERSTATE --> 1.0; U.S. ROUTE --> 0.8; STATE ROUTE --> 0.8; RAILROAD --> 0.7; FED. FUNDED CO. ROUTE OR CITY ST. --> 0.5; NON-FED. FUNDED CO. ROUTE OR CITY ST. --> 0.2; FED LAND, STATE LAND, & UNDEFINED --> 0.0)
LENGTH OF DETOUR	0.05 *	(linear, normalized to 100)
ABUT. TYPE	0.04 *	(0.0 --> monolithic; 1.0 --> nonmonolithic)

THE SUM OF THESE WILL BE BETWEEN 0.0 AND 1.0

** all preweights are between 0.0 and 1.0

AN OLD BRIDGE GETS A SEISMIC FACELIFT

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ABSTRACT

Elastomeric bearings are used to retrofit 28 supports for a 50-year-old 1010-foot (308 m) long bridge and pipe. The structure is an essential part of The Metropolitan Water District's feeder system to the Southern California area. Use of the bearings insures that the structure will behave elastically and safely during the controlling maximum expectable event, a magnitude 7.0 earthquake originating on the San Jacinto fault 11 miles (18 km) distant. The bearings were economical and easily installed with minimal shut down time.

INTRODUCTION

The Santa Ana bridge carries the Upper Feeder pipeline, which here is a 116 in. (259 cm) inside-diameter steel cylinder. The 1,010 ft. (308 M) bridge consists of two types of structures, one being three, 180 ft. (55 m) long steel trusses, and the other, a series of concrete piers on 50 ft. (15.3 m) centers with the pipe spanning between them (See Figs. 1 and 2).

The pipeline is part of the 700 miles (1130 kms) of distribution pipeline used by the Metropolitan Water District of Southern California (MWD) to import water from the Colorado River and northern California. MWD feeds over 2 million acre-feet (2.5 billion cu. meters) of water a year to almost half of southern California's 15 million people.

Immediately following the area's 1971 earthquake in San Fernando, MWD began intensively examining its system to identify segments that might not survive a major earthquake. The bridge was found to be one such segment.

A pseudo-static analysis indicated that the bridge's trusses and pipe-support pedestals, as well as some individual truss members and connections, could be highly overstressed during a major earthquake. So MWD hired Lindvall, Richter & Associates (LRA) to do a more detailed seismic stability analysis.

ISOLATION BEARINGS

As a result of LRA's analysis, the bridge has been retrofitted with isolation bearings at all truss piers and pipe piers (See Figs. 3, 5, and 6). The bearings are rectangular blocks of natural, ozone-resistant rubber with cylindrical cores of pure lead. Inside each bearing, closely spaced horizontal sheet steel lamination (shims), bonded to the rubber during fabrication (vulcanization by heat), provide vertical stiffness to the bearing by inhibiting side bulging (See Fig. 3).

Under small lateral loads and movements, the lead provides most of the lateral stiffness and behaves elastically. However, under sufficiently large loads the lead yields, and the bearings' lateral stiffness is then provided by the rubber alone. At the Santa Ana River Crossing, the bearings were sized so the lateral force on each bearing is about 15% of the vertical load when the pipe is full of water. This lateral force is about twice the maximum windload the pipe will experience. The lead core begins to yield when the bearing has displaced 5/8 in. (16 mm) laterally.

This thinking led to bearings from 12 in. to 20 in. (305 mm to 508 mm) square in plan, and from 9 to 11 1/2 in. (229 mm to 292 mm) high. Their lead cores have diameters from 4 1/2 to 7 in. (114 mm to 178 mm)

Lateral properties of the bearings came from design guidelines and test data supplied by bearing supplier Dynamic Isolation Systems, Inc., of Berkeley, Calif.

COMPUTER MODEL

The Santa Ana bridge and its isolation bearings were modelled by feeding truss and beam elements into the SAPV program developed at the University of California, Berkeley, and modified at the University of Southern California, Los Angeles, and the ADENA program developed at MIT.

The program performed linear and nonlinear analyses, respectively. In the latter, the bearings were modelled as kinematic hardening bilinear truss elements in the horizontal directions, as linear truss elements in the vertical.

The mathematical model representing the bridge and motions it will undergo is highly sophisticated. Each member of the three trusses was represented explicitly, and where stability or secondary bending effects were important, the pieces of each member were modelled individually.

The pipe skin was represented in the mathematical model as a cylindrical Vierendeel truss. The water was represented by assuming the Vierendeel truss was internally supported by a beam running down the pipe axis, with radial spokes at the terminus of each finite element in the model.

Shear and flexural behavior of the pipe were represented by modifying the Vierendeel truss skin to eliminate the effects of the axial area of the longitudinal elements. Inertia and shear properties in the plane of these skin elements were selected to match the theoretically calculated bending and distortion behavior of the steel pipe. Longitudinal properties were represented entirely by the centralized axial beams. The expansion joint in the pipe near the bridge's mid-span was modelled by deleting one of the centralized axial beams.

LOADING CONDITIONS

The bridge was analyzed for gravity, earthquake, thermal, hydraulic and wind loads. Because of the inertial effects of water in the pipe and the proximity of major faults, wind loads are subordinate to the earthquake loads. Hydraulic loads, from a 464 ft. head, produce a hoop tension on the pipe equal to 49% of yield.

The thermal load conditions give rise to rather complex behavior. The pipe is fixed along its axis at the two bridge abutments; an expansion joint makes the pipe discontinuous near bridge midspan. Further, each truss is fixed along its axis to one of its supporting piers. To take gravity loads, the pipe is supported by rocker supports along the bridge trusses. Lateral stability of the pipe is assured by hinged ties affixed to the trusses. Spaced at 36 ft. (11 m) on center, these vertical and lateral supports permit differential thermal longitudinal motion of pipe and truss.

Seismicity at the bridge is dominated by the San Andreas and San Jacinto faults (See Fig. 4). The former is capable of producing the greater magnitude event, but the latter is closer, 11 miles (17.7 km) away vs. 17 (27.4 km). That, and the fact that the more recent significant seismic activity has been along it, makes the San Jacinto Fault dominant.

A maximum expectable earthquake on the San Jacinto fault was defined as a Richter magnitude 7.0 event. For this event the peak ground acceleration at the bridge is 0.35 g, and the duration of strong ground shaking is about 20 sec. To respond satisfactorily, the supporting bearings must maintain a stability factor of safety of at least two.

A maximum credible event on the San Jacinto fault would have a Richter magnitude of 7.5. This event would produce a peak ground acceleration of 0.40 g at the bridge, with a strong shaking duration of 30 sec. For this event the bridge and pipe structure was permitted to sustain some damage, but it must remain in service with no risk of collapse.

Ground motions were constructed on the computer to represent both maximum credible events. These motions were synthesized from the records obtained during the 1952 earthquake in Kern County, Calif., at the Taft Lincoln School. The modified records have response spectra that compare well with those from the 1971 San Fernando and 1979 Imperial Valley earthquakes. Three uncorrelated components, two horizontal and one vertical, were constructed to represent both the maximum expectable and maximum credible events.

DYNAMIC ANALYSIS

The ADENA program was used to analyze the bridge-pipeline's behavior with and without the new bearings. A step-by-step Newmark integration procedure was deployed, with constant average accelerations in each 0.02 sec time-step interval. Energy dissipation was introduced into the model by using mass proportional damping. The proportionality constant for the damping matrix was selected so that modal damping close to 5% of critical was realized throughout the linearalized period range of interest.

This damping value of 5% is appropriate for most steel structures subject to strong ground shaking.

The maximum expectable earthquake produced a wholly satisfactory response in the bridge and pipe. Only the elastomeric bearings experienced yielding, and by doing so they limited the extreme lateral force experienced by the bridge to less than 20% of its weight. In contrast, for the pre-retrofit condition, the potential lateral force was equal to the structure's full weight, which could have resulted in significant failures or even collapse of the bridge.

Two factors gave rise to the structure's much improved seismic response. The use of soft, yielding lateral supports shifted the fundamental period of the structure from about 0.6 sec to about 1.0-1.5 sec. This in turn reduced accelerations and lateral forces felt by the bridge and pipeline. Secondly, and more important, the bearings increased the rate of energy dissipation to a value equal to that for a 40% damped oscillator.

The earthquake lateral force felt by the structure was reduced 80%, and extreme displacements were modestly reduced. Most of the displacements were in the bearings themselves, with the supported bridge and pipe moving together much as a single lobe over its 1,010 f. (308 m) length. The extreme lateral displacement was 4.3 in. (109 mm).

RETROFIT STOPS

After completing the installation of the elastomeric bearings at the trusses, the first phase of construction, the trusses were monitored for longitudinal movement due to thermal expansion and contraction. The initial assumption that the trusses would thermally expand and contract longitudinally in place proved to be incorrect. Due to horizontal forces imposed on the bearings from the pipe rocker supports, which canted when the warm steel pipe was filled with cold water and contracted, the bearings distorted and moved the south truss north 2-1/2" over several days. Because of the location of the pipe expansion joint (See Fig.2), the north and center trusses moved south 1" and 1-1/2" respectively in the same time frame. The movement of the trusses was stopped by welding temporary shear plates to the top and bottom bearing plates of both bearings at one end of each truss.

Returning the trusses to their original positions was accomplished by removing the temporary shear plates and installing sandwiched plates, that could be locked using high strength friction bolts, on all of the truss bearings. The trusses were "walked" back to their original positions by alternately locking and unlocking the plates in the early morning and late afternoon to take advantage of the warm days and cold nights. When the trusses returned to their original positions, one end of each truss was locked.

A permanent stop consisting of a beam spanning along the width of the concrete pier, between the elastomeric bearings, with a hinged center support was installed on the north end of the north and center trusses and the south end of the south truss (See Fig. 7). The ends of the beam were welded to the top bearing plates. The hinged support at the center of the beam was anchor bolted to the concrete pier. Longitudinal distortion of the bearings due to rocker-induced loads is restrained, yet lateral distortion of the bearings during a seismic event will be uninhibited.

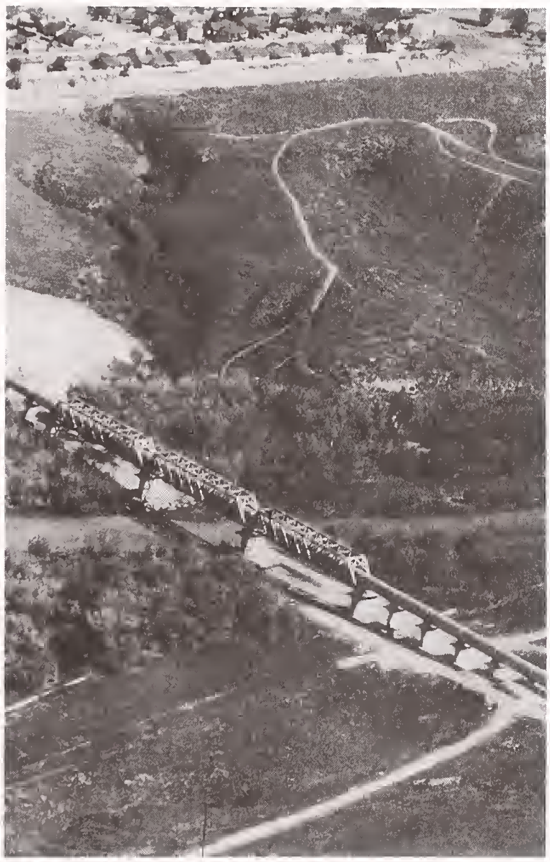
The second phase of construction consisted of replacing the rigid pedestal supports with elastomeric bearings at the pipe piers. It was decided to design stops to limit the longitudinal distortion of the elastomeric bearings supporting the pipe (See Fig. 8) at the piers due to rocker-induced loads caused by thermal expansion and contraction of the pipe. Like the stops installed on the trusses, these stops will limit longitudinal bearing distortion, yet permit lateral distortion of the bearings during a major earthquake.

CONCLUSION

The use of elastomeric bearings in lieu of rigid pedestals to support the Santa Ana River Crossing has increased the period of the structure and the rate of energy dissipation. Consequently, the design earthquake lateral forces will be reduced to 20% of what the structure would have experienced prior to modification. Use of these bearings, in combination with stops that prevent creep and rocker induced loads from distorting the new bearings longitudinally, has mitigated the risk of significant damage to the Santa Ana River Crossing, due to a major earthquake, while permitting the structure to function as originally designed.

POSSIBLE FUTURE RESEARCH

Since the Santa Ana River Crossing is seismically instrumented, the ground, Pier No. 8, and the truss acceleration, velocity, and displacement will be known after a major earthquake. This data could be used to compare the actual response of the structure with design predicted response.



AERIAL VIEW OF SANTA ANA RIVER CROSSING

FIGURE 1

SANTA ANA RIVER CROSSING
SCHEMATIC DIAGRAM

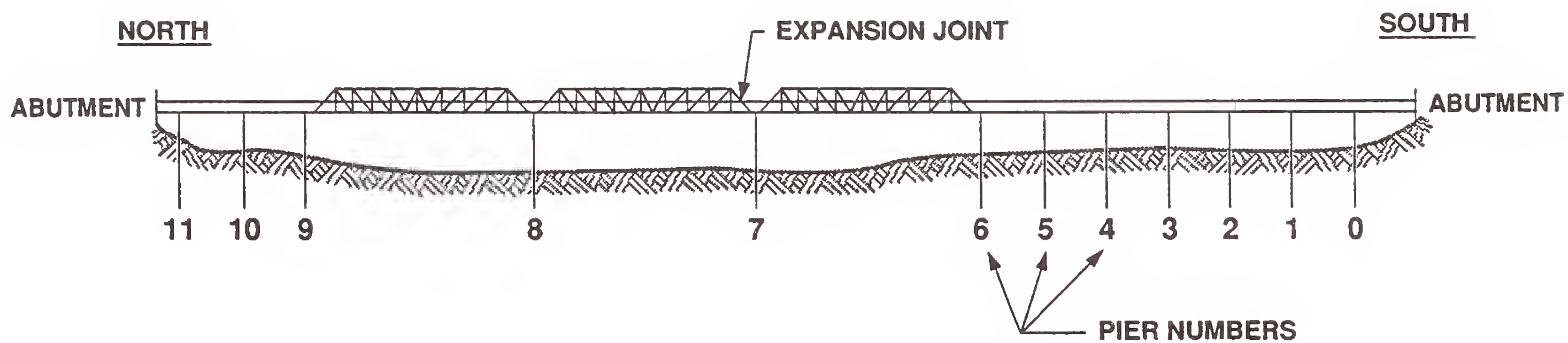


FIGURE 2

ELASTOMERIC BEARING - SCHEMATIC VIEW

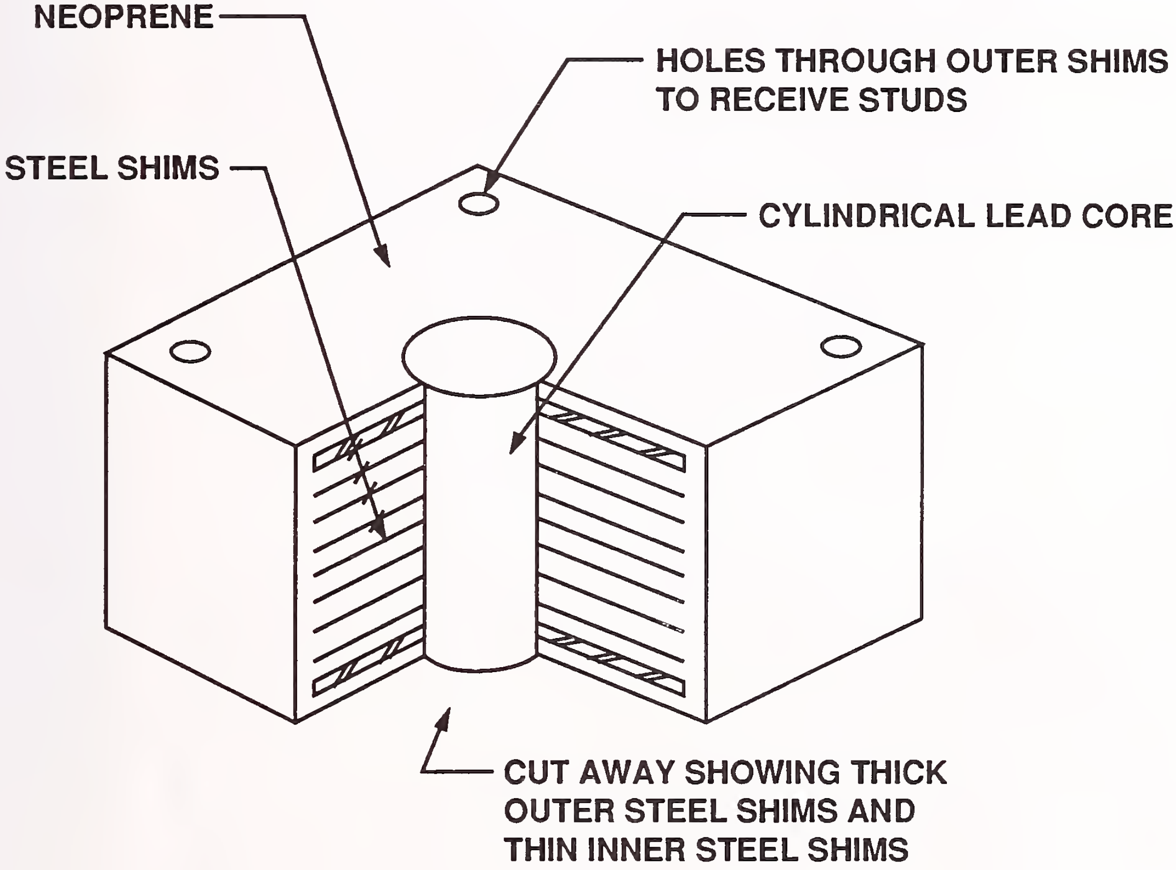
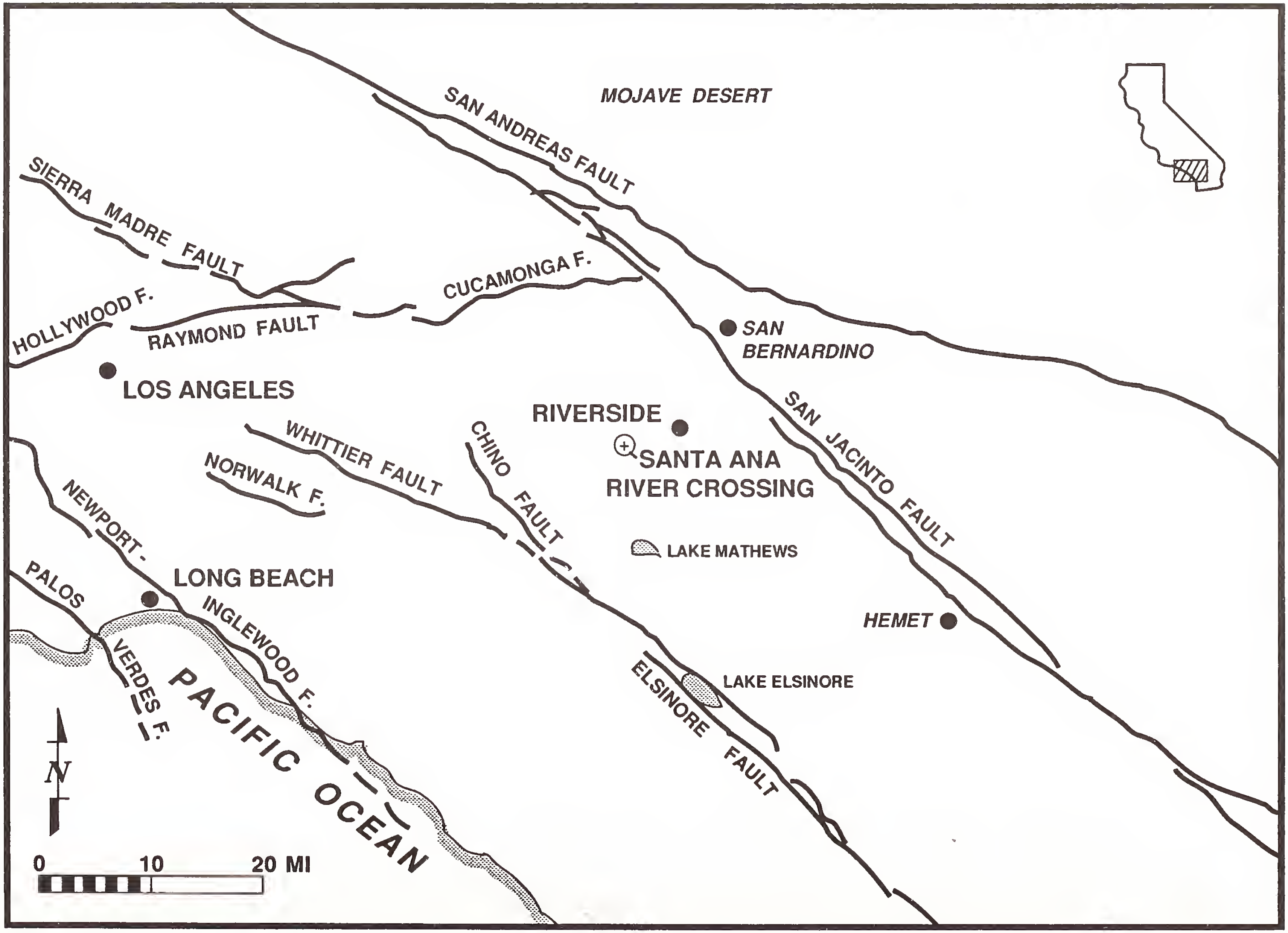


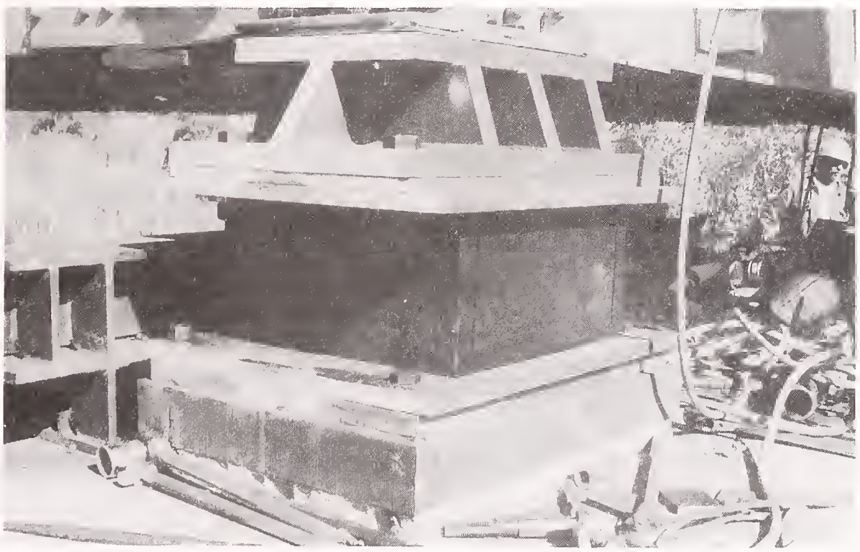
FIGURE 3



Map showing known active faults in Southern California and Location of Santa Ana River Bridge.

FIGURE 4

ORIGINAL SUPPORT PEDESTAL - FIGURE 5



RETROFIT ELASTOMERIC BEARING - FIGURE 6

TRUSS RETROFIT STOP - FIGURE 7

PIPE SUPPORT BEARING AND BEARING STOP - FIGURE 8

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SEISMIC INSPECTION AND SEISMIC STRENGTHENING OF HIGHWAY BRIDGES IN JAPAN

BY

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ABSTRACT

This paper presents a state of practice of seismic inspection and seismic strengthening of highway bridges in Japan. Description is firstly given to the history of seismic inspection and seismic strengthening of highway bridges in Japan. An inspection method of existing highway bridges based on a statistical analysis of past seismic damage is presented. Practical seismic strengthening methods for vulnerable highway bridges are described. Finally, current research efforts for developing seismic inspection and seismic strengthening methods of reinforced concrete bridge piers with termination of main reinforcement at mid-height are presented.

INTRODUCTION

Highways in Japan consist of Expressways (3,721 km), National Highways (46,661 km), Prefectural Roads (128,202 km), and Municipal Roads (925,138 km). Along the highways and roads, excluding the Municipal Roads, there are about 60,000 bridges with span lengths (deck length between two adjacent substructures) of 15 m or longer.

Although the number of bridges constructed per year depends on the year and span length, it is about 6,000 for concrete bridges and 2,000 for steel bridges with a length of 15 m or longer.

Located along the Pacific Seismic Belt, Japan is one of the most seismically disastrous countries in the world and has often suffered significant damage from large earthquakes. **Fig. 1** shows the largest magnitude of the earthquakes which occurred in the past[1]. It is recognized that the earthquakes with magnitude over 8 have occurred with rather short recurrent period in and around Japan in the past. It should be noted that seismicity is especially high along the Pacific coast. Cities large in population and industrial products such as Tokyo, Osaka and Nagoya are located in this region.

Table 1 shows the highway bridges which suffered damage in the past earthquakes since the Kanto Earthquake of 1923. It should be noted that although there were many bridges which suffered damage due to earthquakes, the number of bridges which fell down was only 15.

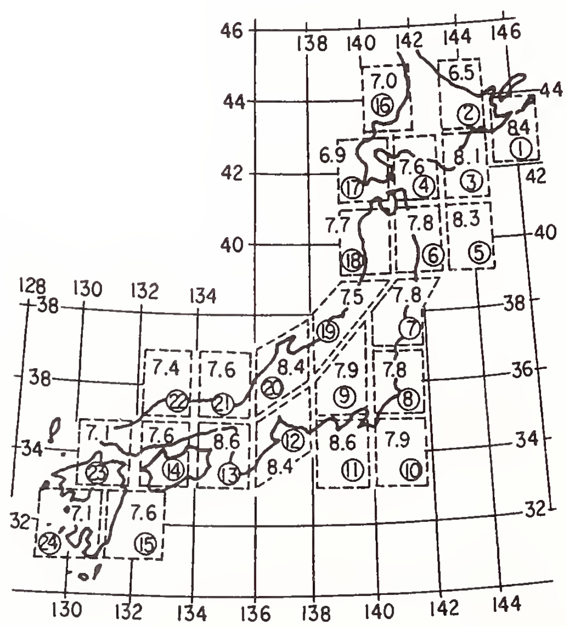


Fig. 1 Largest Magnitude of Earthquakes in the Past

Based on surveys of the damaged bridges, it is pointed out that the three major factors which contributed to the damage of bridges are[2] :

a) weakness of substructures
b) weakness of bearing supports, and
c) weakness of surrounding subsoils.

From such factors, the following types of damages were most often developed in the past:

a) substructure : tilting, settlement, sliding, cracks, and overturning
b) superstructure : movement, buckling and cracks near the supports, and falling of girder
c) bearing supports : failure of supports, and pull-out or rupture of anchor bolts

Although these kinds of damage are the ones commonly observed in the past earthquakes, the damage types have been changing as shown in **Table 2** in accordance with the progress of seismic design methods and improvements in construction practice[3].

Table 1 Damage of Highway Bridges in the Past since Kanto Earthquake of 1923

DATE	EARTHQUAKE	MAGNITUDE	NUMBER OF BRIDGES DAMAGED	NUMBER OF BRIDGES WHICH FELL DOWN
1923. 9. 1	KANTO	7.9	1,785	6
1946.12.21	NANKAI	8.1	346	1
1948. 6.28	FUKUI	7.3	243	4
1949.12.26	IMAICHI	6.4	1	0
1952. 3. 4	TOKACHI-OKI	8.1	128	0
1962. 4.30	MIYAGI-KEN-HOKUBU	6.5	187	0
1964. 6.16	NIIGATA	7.5	98	3
1968. 2.21	EBINO	6.1	10	0
1968. 5.16	TOKACHI-OKI	7.9	101	0
1978. 1.14	IZU-OHSHIMA	7.0	7	0
1978. 6.12	MIYAGI-KEN-OKI	7.4	95	1
1982. 3.21	URAKAWA-OKI	7.1	5	0
1983. 5.26	NIHON-KAI-CHUBU	7.7	176	0
1984. 9.14	NAGANO-KEN-SEIBU	6.8	14	0
TOTAL			3,191	15

Table 2 Change of Damage Types

Year	Major Earthquokes	Change of Major Seismic Domoge	Seismic Design Method	Seismic Inspection ond Strengthening
1920	1923 Kanto Earthquake (M7.9)	Failure of Superstructure due to Tilting/Movement of Foundation	1926 Initiotion of Seismic Design (Details of Road Structures)	
1930		Failure of Concrete around Fixed Bearing	1939 Introduction of Standord Seismic Coefficient (Design Specifications of Steel Highway Bridges)	
1940	1946 Nankai Earthquake (M8.1) 1948 Fukui Earthquake (M7.3)	Damage due to Liquefaction		
1950	1952 Tokachi-oki Earthquake (M8.1)	Failure of RC Piers, ond Bearing	1956 Seismic Coefficient depending on Zone and Ground Condition (Design Specifications of Steel Highway Bridges)	
1960	1964 Niigata Earthquake (M7.5)			
1970	1978 Miyagi-ken-oki Earthquake (M7.4)		1971 • Seismic Coefficient depending on Zone, Ground Conditions, Importance and Structural Response • Introduction of Evoluotion Method for Liquefoction (Specificotions for Seismic Design)	1971 Seismic Inspection 1976 Seismic Inspection 1979 Seismic Inspection
1980	1982 Urakowo-oki Earthquake (M7.1) 1983 Nihon-kai-chubu Earthquake (M7.7)		1980 • Part V Seismic Design, Specifications for Design of Highway Bridges • Introduction of New Evaluation Method for Liquefactions	1986 Seismic Inspection
1990			1990 Part V Seismic Design, Specifications for Design of Highwoy Bridges	

This paper presents pre-earthquake measures for mitigating earthquake hazards of highway bridges in Japan. Development of seismic inspection and seismic strengthening methods for existing highway bridges are presented.

HISTORY OF SEISMIC INSPECTION AND SEISMIC STRENGTHENING OF HIGHWAY BRIDGES IN JAPAN

The nationwide seismic inspection of highway bridges with span length longer than 15 m was made in 1971, 1976, 1979 and 1986 by the Ministry of Construction. Overcrossings in city area are also included in "bridges". **Table 3** shows the inspected items of the past seismic inspections of highway bridges. The first inspection in 1971 and second inspection in 1976 were to detect deteriorated highway bridges susceptible to falling-off of superstructures during earthquakes. The third and fourth seismic inspection in 1979 and 1986 was to clarify structural resistance of highway bridges against falling-off of superstructures during destructive earthquakes.

Table 3 Inspection Items Considered in the Past Seismic Inspection of Highway Bridges in Japan

Year	Inspection Items
1971	· Deterioration · Bearing Seat Length S for Bridges supported by Bent Piles
1976	· Deterioration of Substructures, Bearing Supports and Girders/Slabs · Bearing Seat Length S and Devices for Preventing Falling-off of Superstructure
1979	· Deterioration of Substructures and Bearing Supports · Devices for Preventing Falling-off of Superstructure · Effect of Soil Liquefaction · Bearing Capacity of Soils and Piles, and Strength of RC Piers · Vulnerable Foundations (Bent Piles and RC Frame on Two Independent Caisson Foundation)
1986	· Deterioration of Substructures, Bearing Supports and Concrete Girders · Devices for Preventing Falling-off of Superstructure · Effect of Soil Liquefaction · Strength of RC Piers (Bottom of Piers and Termination Zone of Main Reinforcement) · Bearing Capacity of Piles · Vulnerable Foundations (Bent Piles and RC Frame on Two Independent Caisson Foundation)

Note) Deterioration includes cracks of concrete members, excessive inclination, slip and subsidence of foundations, and scouring of surrounding soils of foundation.

For those bridges which were judged vulnerable to destructive damages during earthquakes, seismic strengthening works have been made. **Table 4** shows the number of highway bridges inspected and the budget spent for the seismic strengthening works. For those bridges consisting of several spans for crossing rivers and valleys, a whole group of bridges is counted as "a bridge" in **Table 4**. Therefore actual number of bridges is much larger than the number presented in **Table 4**. Because bridges inspected in the past also require the latest inspection, the number of bridges inspected and strengthened includes duplication. In the latest 1986 inspection, about 40,000 bridges were inspected and about 11,700 bridges were found to require seismic strengthening. Most of the bridges require installation of the device for preventing superstructure from falling-off from the substructures as shown in **Fig. 2**[6]. In the past four seismic strengthening programs by 1989 fiscal year, 23,466 bridges were strengthened. As of 1989, about 300 billion Yen was spent for the seismic strengthening works including bridges.

Table 4 Number of Highway Bridges Inspected by the Past Seismic Inspection in Japan[4) 5)]

Year	Inspected	Require Strengthening	Strengthened
1971	17,970	3,200	1,441
1976	24,504	7,110	2,742
1979	34,712	16,011	13,580
1986	40,370	11,764	5,703

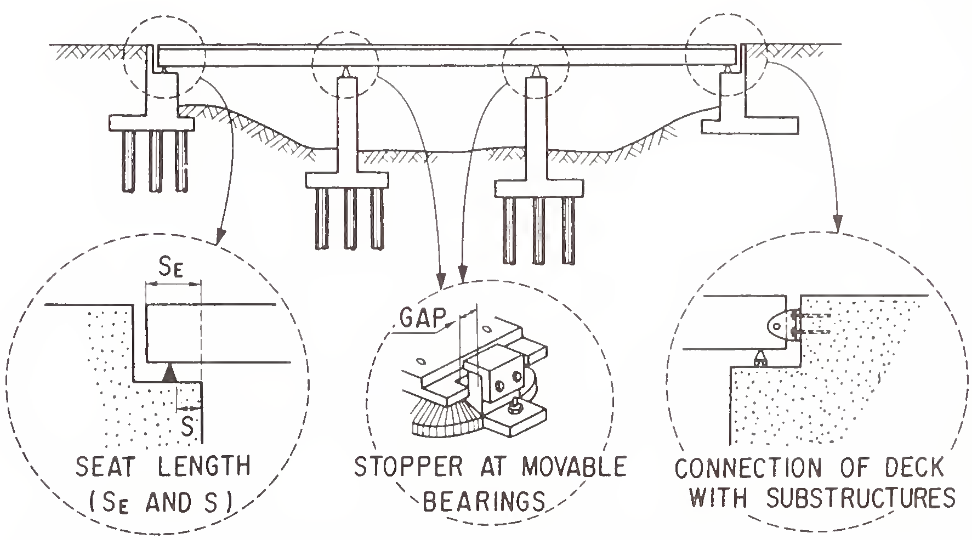


Fig. 2 Devices for Preventing Superstructure from Falling

VULNERABILITY INSPECTION OF HIGHWAY BRIDGES

Seismic inspection methods to detect highway bridges vulnerable to earthquakes have been developed and amended several times to reflect progress of bridge earthquake engineering and lessons learned from the past seismic damage. The most important requirement for the inspection method was to be able to assess the vulnerability of a number of highway bridges at site without complex calculations. The latest seismic inspection method[7] which was referenced in the 1986 seismic inspection was formulated on the statistical analyses of 124 bridges damaged in the past earthquakes[8].

In the statistical analyses, factors which are likely to affect the seismic vulnerability of highway bridges were firstly studied with use of the Type II quantification analysis. The rank of damage degree was classified into high vulnerability (Rank A), moderate vulnerability (Rank B) and low vulnerability (Rank C) as shown in **Table 5**. From the analysis of the past seismic damage, 15 items, as shown in **Table 6**, were selected as the factors likely to affect seismic vulnerability. The 15 items consists of four principal factors, i.e., intensity of earthquake ground motions, properties of superstructure and substructures, devices for preventing falling-off of superstructure from substructures, and ground condition. Each item was further divided into several categories.

Table 5 Definition of Seismic Vulnerability

Rank	Vulnerability of Seismic Damage	Rank of Damage Degree
Rank A — High Vulnerability	Possibility for suffering damage or damage degree is high	5 Falling-off of Superstructure or 4 Extensive Damage
Rank B — Moderate Vulnerability	Possibility for suffering damage or damage degree is moderate	3 Moderate Damage
Rank C — Low Vulnerability	Possibility for suffering damage or damage degree is low	2 Slight Damage or 1 Minor Damage or 0 No Damage

Predicted damage rank y_i was assumed to have a form of

$$y_i = \sum_j \sum_k \delta_{ijk} \cdot x_{jk} \qquad (1)$$

in which x_{jk} represents a weighting factor for the k-th category of the j-th item, and δ_{ijk} represents a variable corresponding to the category k in the item j of the i-th bridge. The variable δ_{ijk} was so defined that it takes a value of 1 if the characteristics of the i-th bridge correspond to the category k in the item j, and is 0 otherwise. The weighting factor x_{jk} was determined so as to minimize the sum of square of the difference between the predicted damage rank and the actual damage rank.

Table 6 Items and Categories which Affect Seismic Vulnerability of Highway Bridges

Item	Category	Seismic Damage Rank: 0 No Damage	1 Minor Damage	2 Slight Damage	3 Moderate Damage	4 Extensive Damage	5 Falling-off of Superstructure	Results of Statistical Analysis: Normalized Score	Range	Partial Correlation Coefficient
①Intensity of Peak Ground Acceleration a_{max} [cm/sec²]	$a_{max} < 200$	1 (5%)			1 (5%)			0.169	1.219	0.343
	$200 \leq a_{max} < 300$	13 (68%)	5 (42%)	36 (65%)	8 (40%)	1 (13%)		−0.283		
	$300 \leq a_{max} < 400$	5 (26%)	5 (42%)	18 (33%)	10 (50%)	6 (75%)	4 (40%)	0.176		
	$400 \leq a_{max} < 500$		2 (17%)	1 (2%)	1 (5%)	1 (13%)	2 (20%)	0.936		
	$500 \leq a_{max}$						4 (40%)	0.616		
②Design Specifications	1926 or 1939	1 (5%)		6 (11%)	6 (30%)	5 (63%)	7 (70%)	0.517	0.740	0.319
	1956 or 1964	17 (89%)	6 (50%)	41 (75%)	11 (55%)	3 (38%)	3 (30%)	−0.223		
	1971 or 1980	1 (5%)	6(50%)	8 (15%)	3 (15%)			0.284		
③Type of Superstructure	Gerber or Simply Supported Girder (2 or More Spans)	12 (63%)	7(68%)	40 (73%)	17 (85%)	8 (100%)	10 (100%)	0.081	0.390	0.161
	One-span Simply Supported Girder, or Countinuous Girder with 2-spans or More	5(26%)	3 (23%)	11 (20%)	2 (10%)			−0.232		
	Arch, Frame, One-span Continuous Girder, Cable-stayed Bridge, Suspension Bridge	2 (11%)	2 (17%)	4 (7%)	1 (5%)			−0.308		
④Shape of Superstructure	Skewed or Curved	3 (16%)	1 (8%)	4 (. 7%)				−0.396	0.424	0.122
	Straight	16 (84%)	11 (92%)	51 (93%)	20 (100%)	8 (100%)	10 (100%)	0.027		
⑤Materials of Superstructure	RC or PC	7 (37%)	5 (42%)	22 (40%)	7 (35%)	1 (13%)	6 (60%)	−0.141	0.233	0.131
	Steel	12 (63%)	7 (58%)	33 (60%)	13 (65%)	7 (88%)	4 (40%)	0.092		
⑥Slope in Bridge Axis	6% or Steeper			1 (2%)		1 (13%)	1 (10%)	0.919	0.941	0.161
	Less than 6%	19 (100%)	12 (100%)	54 (98%)	20 (100%)	7 (88%)	9 (90%)	−0.023		
⑦Device to Prevent Falling-off of Superstructure	None	1 (5%)	1(8%)	20 (36%)	8 (40%)	6 (75%)	10 (100%)	0.459	1.106	0.358
	One Device	16 (83%)	6 (50%)	28 (51%)	11 (55%)	2 (25%)		−0.181		
	Two Devices or More	2 (11%)	5 (42%)	7 (13%)	1 (5%)			−0.647		
⑧Type of Substructure	Single-line Bent Pile	1 (5%)		5 (9%)	2 (10%)		3 (30%)	0.292	0.411	0.178
	Reinforced Concrete Frame	1 (5%)	1 (8%)	9 (16%)	8 (40%)	4 (50%)	4 (40%)	0.259		
	Others	17 (89%)	11 (92%)	41 (75%)	10 (50%)	4 (50%)	3 (30%)	−0.119		
⑨Height of Pier H	$10m \leq H$	4 (21%)	5 (42%)	12 (22%)	8 (40%)	2 (25%)	3 (30%)	0.172	0.284	0.125
	$5m \leq H < 10m$	7 (37%)	3 (25%)	29 (53%)	7 (35%)	6 (75%)	5 (50%)	−0.038		
	$H < 5m$	8 (42%)	4 (33%)	14 (25%)	5 (25%)		2 (20%)	−0.112		
⑩Ground Condition	Extremely Soft in Group 4	1 (5%)			1 (5%)	2 (25%)		0.523	0.983	0.273
	Group 4	5 (26%)		2 (4%)	11 (35%)	4 (50%)	7 (70%)	0.224		
	Group 3	3 (16%)	8 (67%)	33 (60%)	6 (30%)	1 (13%)	3 (30%)	0.040		
	Group 2	1 (5%)	1 (8%)	8 (15%)	1 (5%)			0.112		
	Group 1	9 (47%)	3 (23%)	12 (22%)	1 (5%)	1 (13%)		−0.461		
⑪Irregularity of Supporting Ground	Irregular		1 (8%)	1 (2%)	4(20%)	1 (13%)		0.307	0.420	0.104
	Almost Uniform	19 (100%)	11 (92%)	54 (98%)	16 (80%)	7 (88%)	10 (100%)	−0.024		
⑫Effect of Soil Liquefaction	Liquefiable			4 (7%)	2 (10%)	2 (25%)	6 (60%)	0.724	0.816	0.255
	Non-liquefiable	19 (100%)	12 (100%)	51 (93%)	18 (90%)	6 (75%)	4 (40%)	−0.092		
⑬Effect of Scouring	Recognized				1 (5%)			0.312	0.315	0.033
	None	19 (100%)	12 (100%)	55 (100%)	19 (95%)	8 (100%)	10 (100%)	−0.003		
⑭Materials of Substructure	Plane Concrete in Accordance with 1926 Specs. or 1939 specs.				1 (5%)		2 (20%)	0.995	1.020	0.167
	RC, PC, Steel or Unreinforced Concrete in Accordance with Specs. in 1956 or Later	19 (100%)	12 (100%)	55 (100%)	19 (95%)	8 (100%)	8 (80%)	−0.025		
⑮Type of Substructure	Timber, Brick, Masonry, Other Old Unknown Materials		1 (8%)	1 (2%)	1 (5%)	1 (13%)	4 (40%)	0.383	0.697	0.171
	RC Piles, Pedestal Piles or Pier Supported by Two Independent Caissons	2 (11%)	1 (8%)	9 (16%)	4 (20%)	2 (25%)	2 (20%)	−0.314		
	Foundations Designed by Specs. in 1971 or Later	17 (89%)	10 (83%)	45 (82%)	15 (75%)	5 (63%)	4 (40%)	0.034		

From the weighting factors presented in **Table 6**, the effects of each item were found as shown in **Table 7**. The following considerations were subsequently included in the statistical analysis to formulate an inspection method.

a) Because the objectives of the inspection method are to assess the seismic vulnerability of highway bridges subjected to ground shaking of JMA Intensity V or larger (peak ground acceleration larger than 0.25 g), the Intensity of Peak Ground Acceleration a_{max} is dropped from the evaluation item assuming that a_{max} is larger than 0.25 g.

b) Evaluation of the strength of reinforced concrete piers at the mid-height where main reinforcement is terminated was introduced.

c) Because collapse, such as falling-off of superstructure, generally occurred because of excessive relative movements between the superstructure and the substructures, and failure of substructures due to inadequate strength, the evaluation of the seismic vulnerability for both the relative movement and for the strength of substructures should be made.

d) Even if there are some unsatisfactory conditions in the evaluation, final evaluation may not consider them to be critical if the remaining factors are in good evaluation. Consequently, it was decided that those bridges with at least one "critical" or "safe" condition are to be considered to have either high or low seismic vulnerability. Below are some examples of evaluation of such specific types of structure :

- Those designed in accordance with the 1980 Specifications shall be classified into Rank C (safe) unless appreciable deterioration is detected.
- Those constructed by timber, brick, masonry, or old unknown materials shall be classified into Rank A (vulnerable).
- Those supported by single-line bent piles foundation which are constructed on loose alluvial sandy layer vulnerable to liquefaction or very loose clayey deposits shall be classified into Rank A (vulnerable).
- Single-span simply supported girder bridges with the span length less than 15 m shall be classified into Rank C (safe).

The procedure of seismic inspection and an inspection sheet are shown in **Fig. 3** and **Table 8**, respectively[7)8)]. The evaluation of the seismic vulnerability is made in accordance with the points which reflect the vulnerability to excessive relative movement between superstructure and substructures (point X), and the strength of substructures (point Y) as shown in **Table 9**.

Table 7 Factors which Affect Seismic Vulnerability of Highway Bridges

Items	Seismic Vulnerability
① Design Specifications	Those designed in accordance with 1926 or 1939 Specifications have higher vulnerability
② Type of Superstructure	● Gerber or simply supported girders with 2 or more spans have higher vulnerability ● Arch, frame, continuous girders, cable-stayed bridges or suspension bridges have lower vulnerability
③ Shape of Superstructure	Skewed or curved bridges do not necessarily have higher vulnerability than straight bridges
④ Materisls of Superstructure	Reinforced concrete bridges or prestressed concrete bridges have lower vulnerability than steel bridges although the difference is small
⑤ Slope in Bridge Axis	Bridges with slope in bridge axis have higher vulnerability
⑥ Device for Preventing Falling-off of Superstructure	Bridges with devices for preventing falling-off of superstructure have lower vulnerability
⑦ Type of Substructure	Bridges supported by single-line bent piles or by reinforced concrete frame placed on two separate caisson foundations have higher vulnerability
⑧ Height of Piers	Bridges supported by higher piers have higher vulnerability
⑨ Ground Condition	Bridges constructed on soft soil have higher vulnerability
⑩ Effect of Soil Liquefaction	Bridges constructed on sandy soil layers susceptible to liquefaction have higher vulnerability
⑪ Irregularity of Supporting Soil Condition	Bridges constructed on soils with irregularity of supporting conditions have higher vulnerability
⑫ Effect of Scouring	Bridges where the surface soils are scoured have higher vulnerability
⑬ Materials of Substructures	Bridges supported by plane-concrete substructures designed in accordance with 1926 or 1939 specifications have higher vulnerability
⑭ Type of Foundation	Bridges supported by timber, brick, masonry or other old unknown type substructures have higher vulnerability
⑮ Intensity of Ground Motion	Bridges subjected to higher intensity of ground acceleration have higher vulnerability. In particular, vulnerability becomes quite high when the bridges are subjected to peak ground acceleration larger than 400 gal (0.4g)

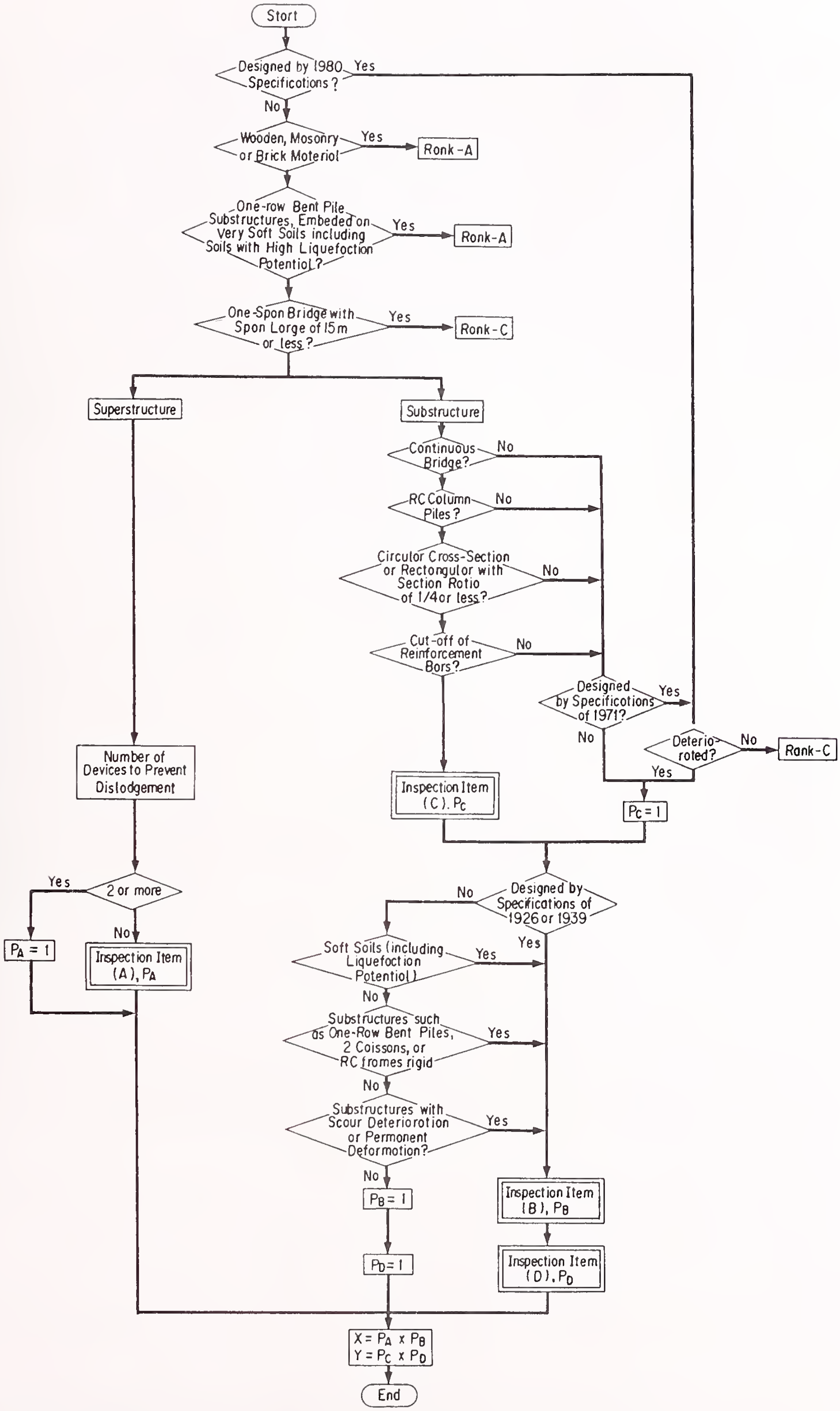


Fig. 3 Procedure of Seismic Inspection for Highway Bridges

Table 8 Inspection Sheet for Seismic Vulnerability of Highway Bridges

Point of Inspection		Factors of Inspection	Evaluation
Inspection for Vulnerability to Develop Excessive Deformation	Inspection Format (A) Inspection for Deformation of Superstructure	① Design Specifications	4.0: 1926 Specs. or 1939 Specs. / 2.0: 1956 Specs. or 1964 Specs. / 1.0: 1971 Specs. or 1980 Specs.
		② Superstructure Type	3.0: Gerber Girder or Simply-supported Girders with Two Spans or More / 1.5: Simply-supported Girder or Continuous Girders Consisting of Two Spans or More / 1.0: Arch, Flame, Continuous Girder (One Span), Cable-stayed Bridge, Suspension Bridge
		③ Shape of Superstructure	1.2: Skewed or Curved Bridge / 1.0: Straight Bridge
		④ Materials of Superstructure	1.2: Rc or PC / 1.0: Steel
		⑤ Gradient	1.2: 6% or Steeper / 1.0: Less Than 6%
		⑥ Falling-off Prevention Device	2.0: None / 1.0: One Device
		P_A = ① × ② × ③ × ④ × ⑤ × ⑥	P_A =
	Inspection Format (B) Inspection for Deformation of Substructure	⑦ Type of Substructure	2.0: Single-line Bent Pile Foundation / 1.0: Others
		⑧ Height of Pier H	2.0: $H \geqq 10m$ / 1.5: $5 \leqq H < 10m$ / 1.0: $H < 5m$
		⑨ Ground Condition	5.0: Extremely Soft in Group 4 / 2.5: Group 4 / 2.0: Group 3 / 1.2: Group 2 / 1.0: Group 1
		⑩ Effects of Liquefaction	2.0: Liquefiable / 1.0: Non-liquefiable
		⑪ Supporting Ground Condition	1.2: Irregular / 1.0: Almost Uniform
		⑫ Scouring	1.5: Recognized / 1.0: None
		P_B = ⑦ × ⑧ × ⑨ × ⑩ × ⑪ × ⑫	P_B =
Inspection for Vulnerability to Develop Failure Due to Inadequate Strength of Substructure	Inspection Format (C) Inspection for Strength of RC Pier at Termination of Reinforcement	⑬ Shear Span Ratio (h/D)	2.0: $1 < h/D < 4$ / 1.0: $h/D \geqq 4$ / 0.5: $h/D \leqq 1$
		⑭ Tension Cracks in Flexure at Terminated Point of Main Reinforcement	2.0: Cracks Wil Occur / 1.0: Cracks Will Possibly Occur / 0.3: Cracks will Not Occur
		⑮ Safety Factor for Yield Strength at Terminated Section of Main Reinforcement — (15-1) S_{ln}	3.0: $S_{ln} \leqq 1.1$ / 2.0: $1.1 < S_{ln} < 1.5$ / 0.5: $S_{ln} \geqq 1.5$
		⑮ — (15-2) S_{mn}	3.0: $S_{mn} \leqq 1.1$ / 2.0: $1.1 < S_{mn} \leqq 1.3$ / 1.0: $1.3 < S_{mn} < 1.5$ / 0.5: $S \geqq 1.5$
		⑯ Shear Stress σ (tf/m^2)	3.0: $\sigma \geqq 45$ / 2.0: $30 \leqq \sigma < 45$ / 1.0: $15 \leqq \sigma < 30$ / 0.5: $\sigma < 15$
		P_C = ⑬ × ⑭ × (15-1) × (15-2) × ⑯	P_C =
	Inspection Format (D) Inspection for Strength of Substructure	⑰ Failure of Fixed Supports and Proximity	5.0: Extensive Failure / 2.0: Small Failure / 1.0: None
		⑱ Extraordinary Damage of Pier	5.0: Extensive Damage / 2.0: Small Damage / 1.0: None
		⑲ Materials of Substructure	2.0: Plane Concrete Older Than 1926 Excluding Gravity-type Abutment / 1.0: Others
		⑳ Construction method of Foundation	2.0: Timer Pile, Masonry, Brick, Other Old Construction Methods / 1.5: RC Piles, Pedestal Piles, Pier Supported by Two Independent Caissons / 1.0: Foundation Designed by 1971 Specs. and Other Later Specs.
		㉑ Foundation Type	1.5: RC Flame Supported by Two Independent Caisson Foundations / 1.0: Others
		⑫ Extraordinary Failure of Foundation	2.0: Recognized / 1.0: None
		P_D = ⑰ × ⑱ × ⑲ × ⑳ × ㉑ × ㉒	P_D =
Evaluation of Deformation and Strength			$X = P_A \times P_B =$ and $Y = P_C \times P_D =$

Table 9 Evaluation of Seismic Vulnerability

Rank of Seismic Vulnerability	Evaluation Points		
	X	Y	
		$P_C = 1.0$	$P_C \neq 1.0$
A—Vulnerable	$X \geqq 60$	$Y \geqq 10$	$Y \geqq 100$
B—Moderate	$20 \leqq X < 60$	$5 \leqq Y < 10$	$50 \leqq Y < 100$
C—Safe	$X < 20$	$Y < 5$	$Y < 50$

Note) Out of two ranks obtaind from X-point and Y-point, higher rank (A is the highest) should be taken as the final ranking of the bridge inspected. For obtaining evaluation points X and Y, refer to Table 8.

Table 10 shows the feasibility of seismic strengthening against the 16 factors which would affect the seismic vulnerability of highway bridges (refer to **Table 7**). Among the 16 factors, ⑥ Devices for Preventing Falling-off of the Superstructure, ⑦ Type of Substructure, ⑩ Effect of Soil Liquefaction, ⑫ Effect of Scouring, ⑬ Materials of Substructures, ⑭ Types of Foundation, and ⑯ Effect of Termination of Main Reinforcement at Mid-height are the factors for which countermeasures for strengthening the bridge are feasible. Countermeasures against for the remaining 9 factors can not be made unless the whole bridge be replaced.

Table 10 Feasibility of Seismic Strengthening for 16 Factors which Affect Seismic Vulnerability of Highway Bridges

Items	Feasibility of Seismic Strengthening	Principles of Countermeasures
① Design Specifications	×	————
② Type of Superstructure	×	————
③ Shape of Superstructure	×	————
④ Materials of Superstructure	×	————
⑤ Slope in Bridge Axis	×	————
⑥ Device for Preventing Falling-off of Superstructure	○	Installation of Devices
⑦ Type of Substructure	○	Strengthening of Substructures
⑧ Height of Piers	×	————
⑨ Ground Condition	×	————
⑩ Effect of Soil Liquefaction	○	Strengthening of Fundations or Strengthening of Surrounding Soils
⑪ Irregularity of Supporting Soil Condition	×	————
⑫ Effect of Scouring	○	Treatment for Prevention of Scouring, or Strengthening of Foundations
⑬ Materials of Substructures	○	Strengthening of Substructures
⑭ Type of Foundation	○	Strengthening of Foundations
⑮ Intensity of Ground Motion	×	————
⑯ Effect of Termination of Main Reinforcement at Mid-height	○	Strengthening of Substructures

Note ○ Items for which seismic strengthening is feasible
× Items for which seismic strengthening is not feasible

Fig. 4 shows how the countermeasures can be made for the above described 7 factors. Installation of devices for preventing falling-off of superstructure, strengthening of foundations, and strengthening of piers and abutments are the main measures of seismic strengthening.

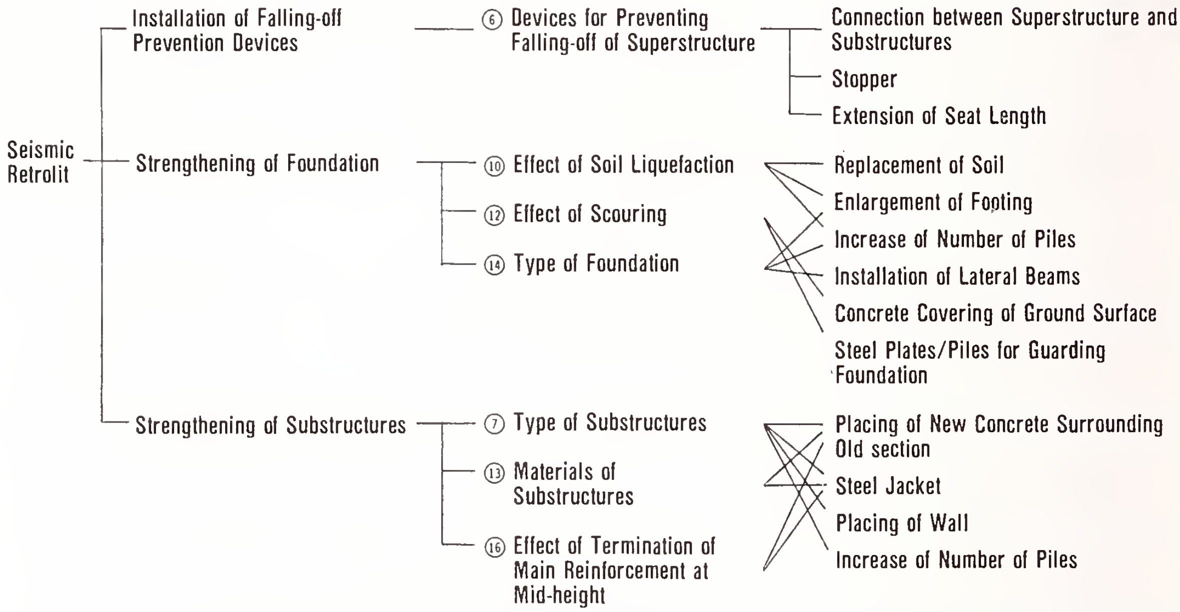


Fig. 4 Selection of Seismic Strengthening Measures for Highway Bridges

Figs. 5, 6 and 7 show the methods for selecting the installation of the stoppers at movable supports, falling-off prevention devices, and elongation of seat length S_E and bearing seat length S as measures of seismic strengthening for existing bridges. The methods are based on a number of practices in the past[9].

Figs. 8 and 9 show measures for strengthening foundations and strengthening substructures for existing bridges, respectively.

The seismic inspection and strengthening methods of transportation facilities including highway bridges were compiled and published from the Japan Road Association in the form of the "Guide Specifications for Earthquake Hazard Mitigation for Transportation Facilities – Pre-Earthquake Countermeasures –" in 1987[9].

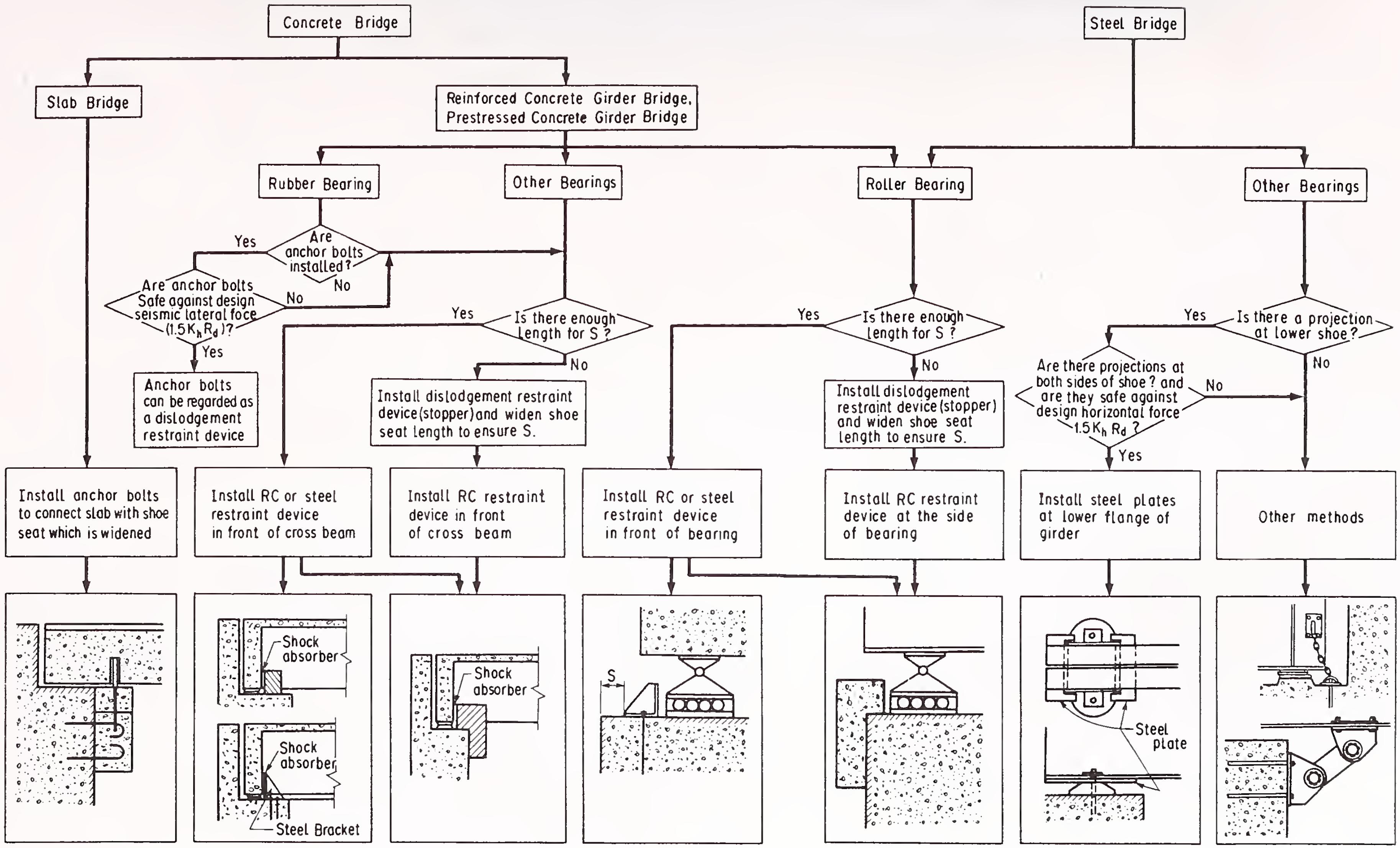


Fig. 5 Installation of Stopper at Movable Bearings for Existing Bridges

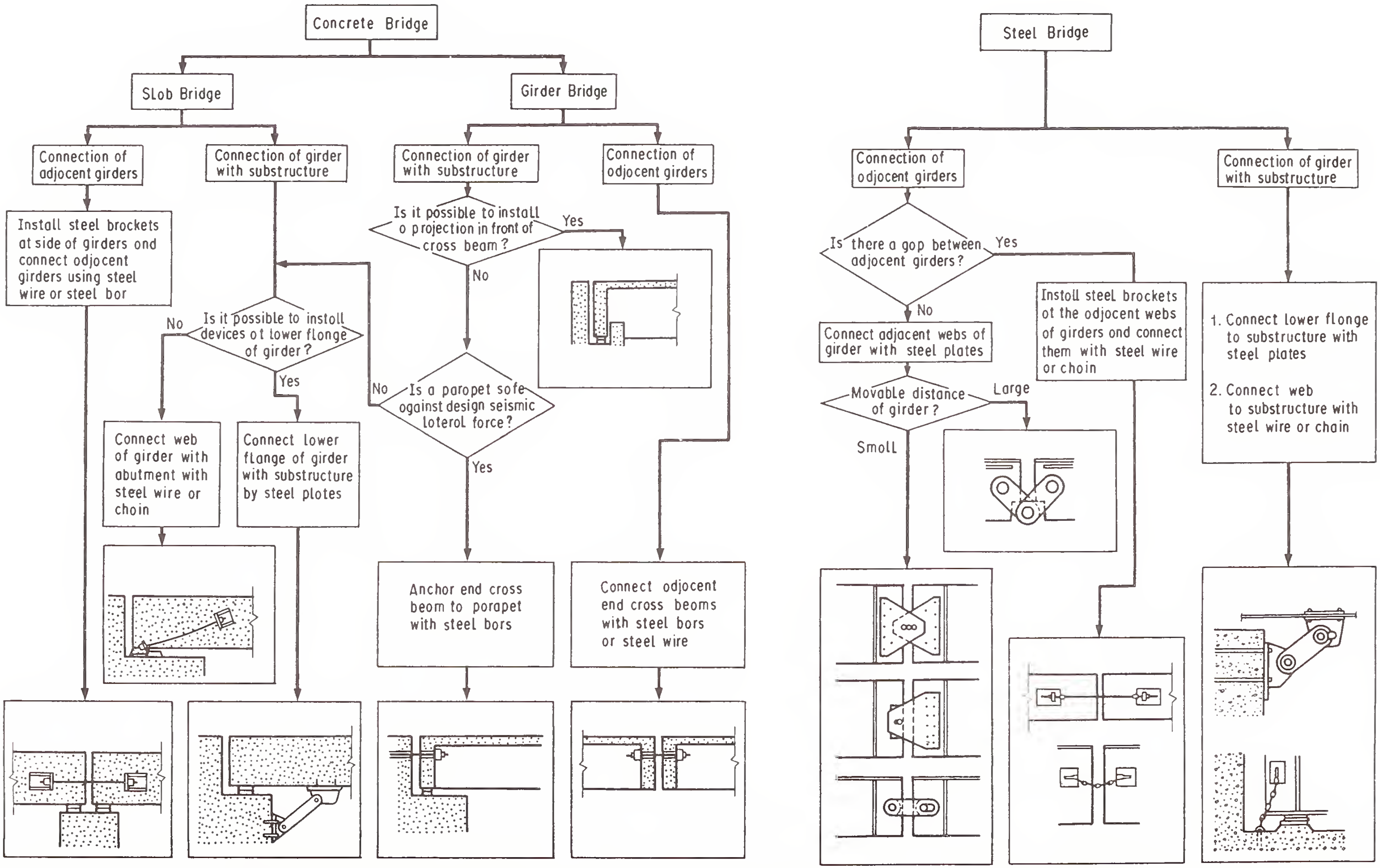


Fig. 6 Installation of Devices Connecting Deck and Substructure, and Device Connecting Two Adjacent Decks for Existing Highway Bridges

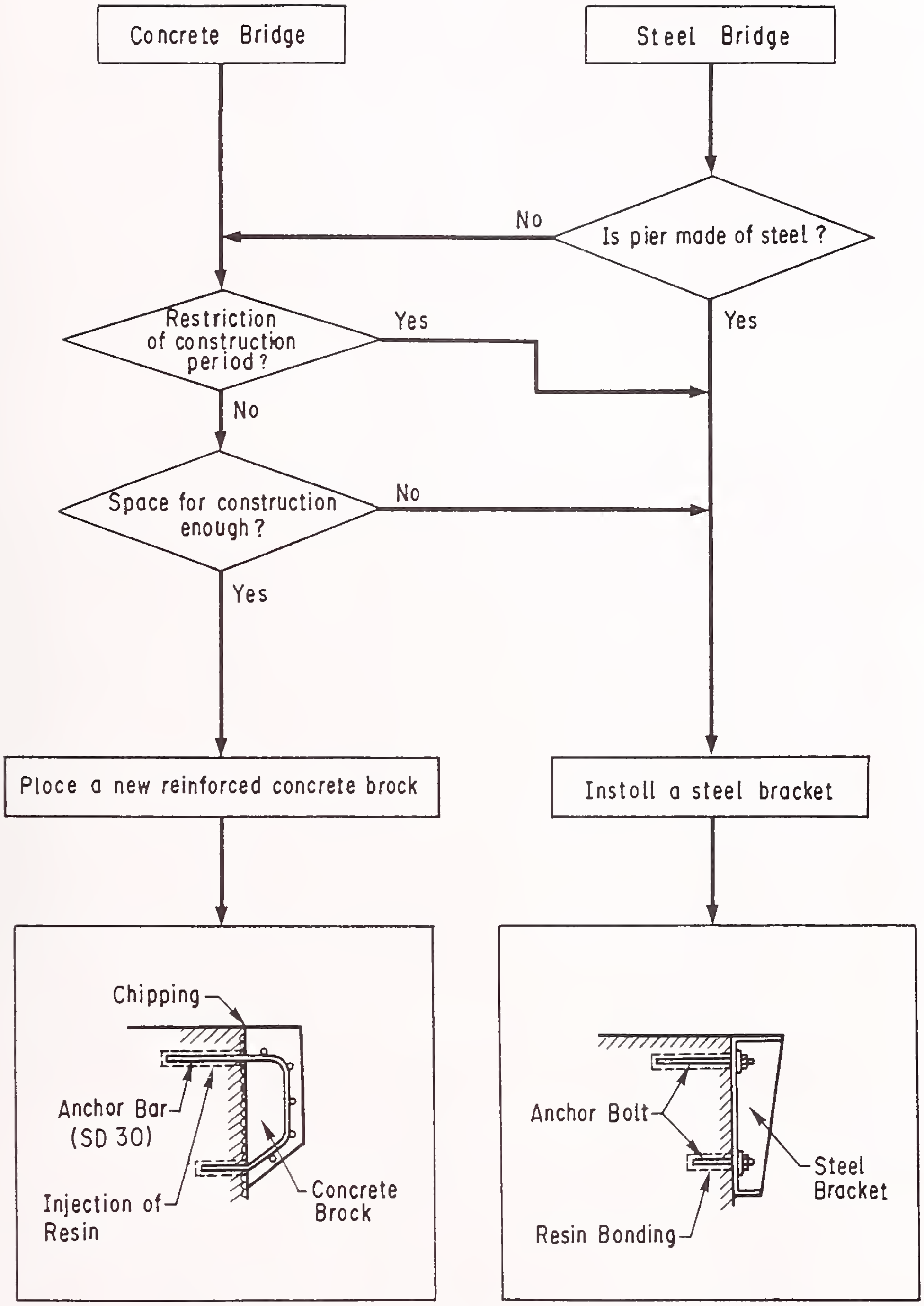


Fig. 7 Elongation of Seat Length S_E and Bearing Seat Length S for Existing Highway Bridges

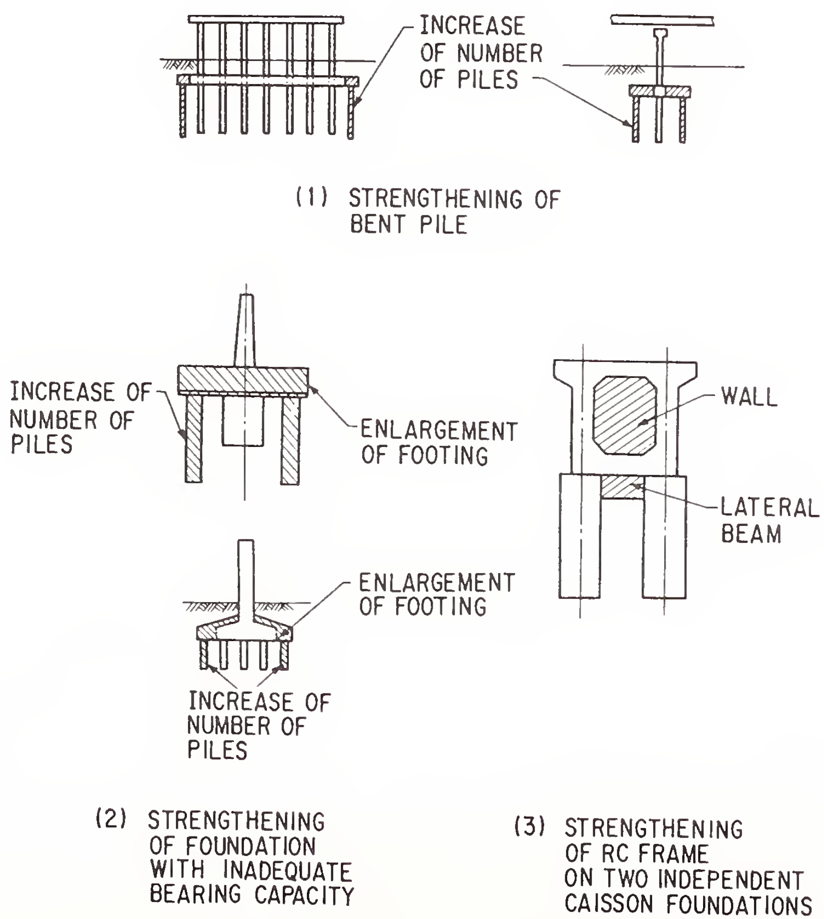


Fig. 8 Strengthening of Foundations

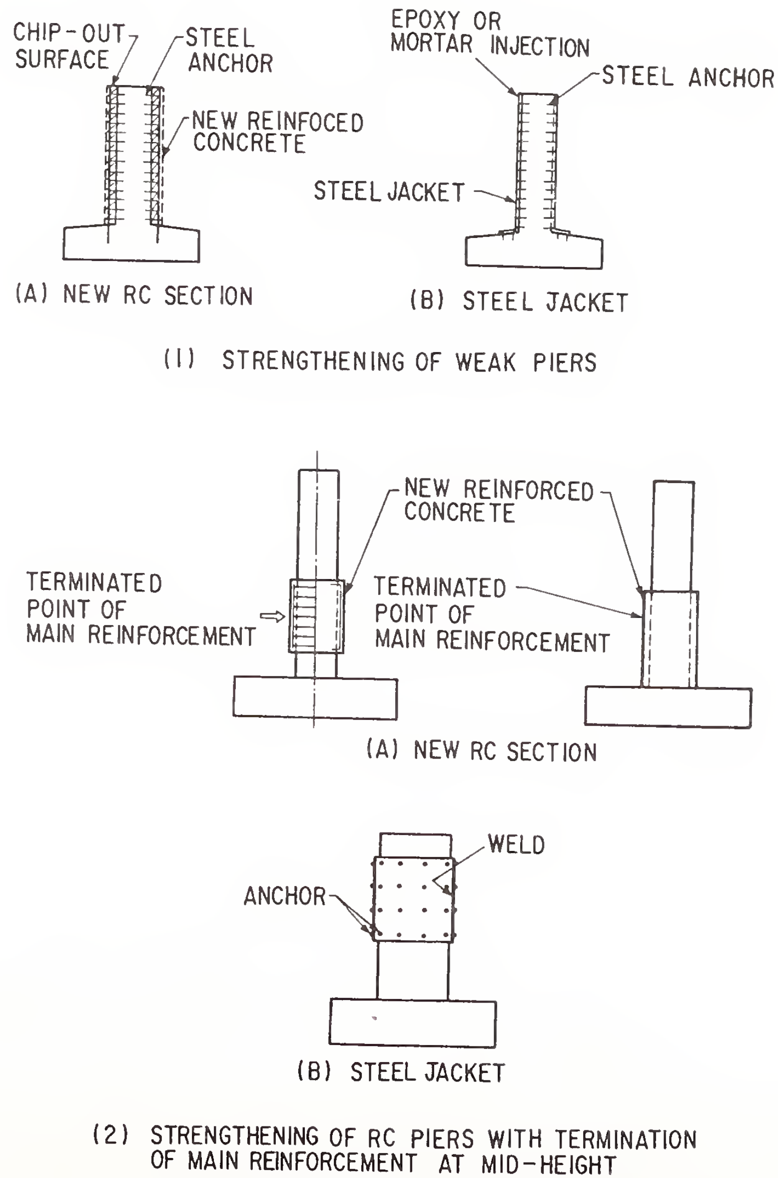


Fig. 9 Strengthening of Piers

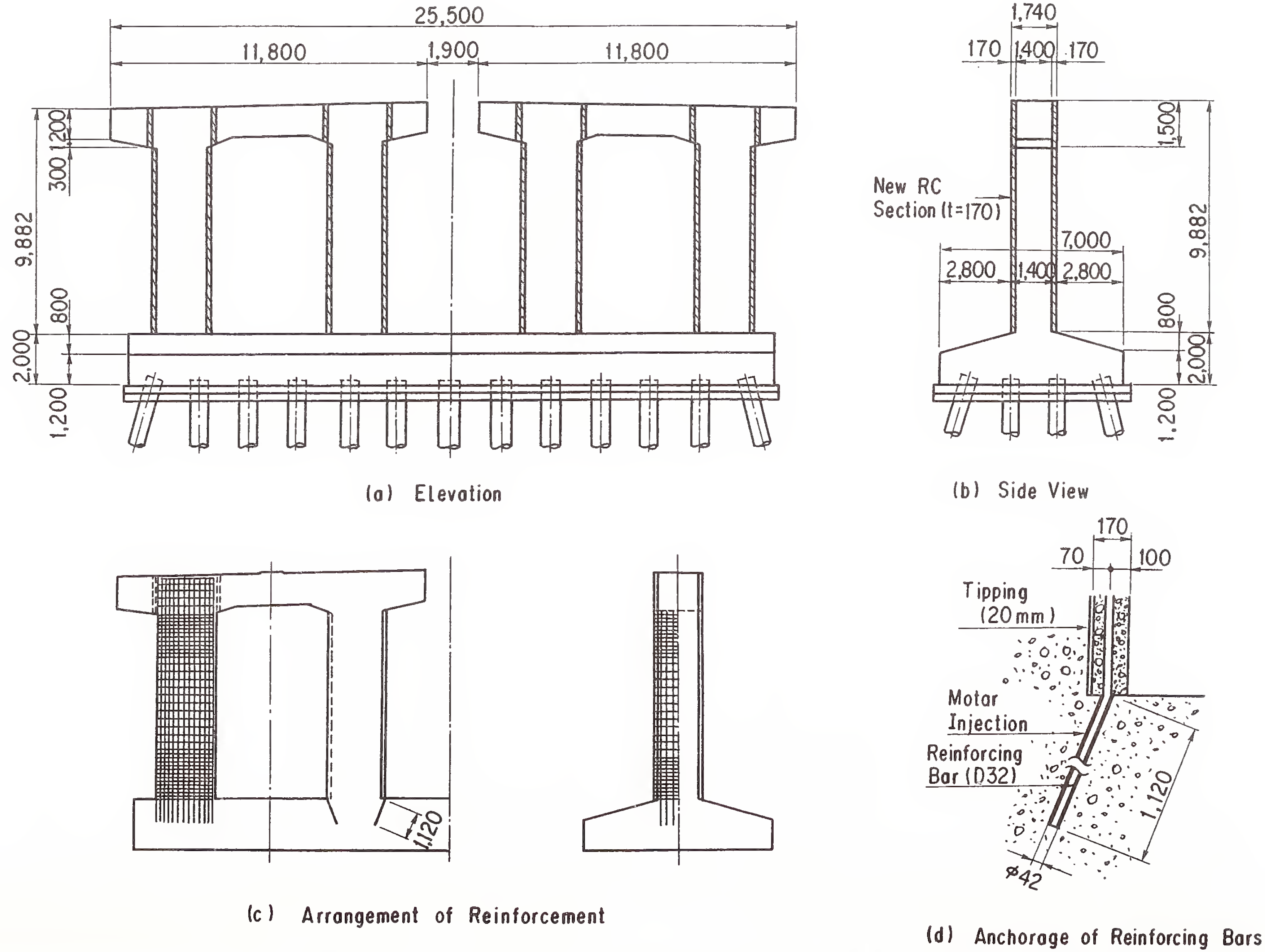
EXAMPLE OF SEISMIC STRENGTHENING

Fig. 10 shows an example of seismic strengthening of reinforced concrete frame supporting a bridge, constructed in 1968, on the Tomei Expressway of Japan Highway Public Corporation[10,11]. The frame was strengthened so that the seismic safety be ensured against the Tokai Earthquake which is anticipated to occur with the magnitude over 8 in the Tokai area. The piers were strengthened by placing new reinforced concrete with thickness of 17 cm. Reinforcing bars were anchored into the footing. They were bent at the base of piers with anchorage length of 112 cm (35 times the diameter of reinforcing bars) being provided to prevent pulling-out from the footing. **Photo 1** shows the completion of the strengthening work.

Photo 1 Seismic Strengthening of RC Frame by Placing New Reinforced Concrete

Fig. 11 shows an example of seismic strengthening of foundation constructed in 1952. To increase bearing capacity of the foundation, number of piles was increased by enlarging the foundation[12].

Fig. 12 shows an example of seismic strengthening of bent piles which was constructed in 1959. Because bent pile has small stiffness to inertia force in longitudinal direction, it was judged vulnerable. A unique strengthening method was adopted in this bridge[12]. Instead of strengthening each bent pile, only two abutments at both ends of the deck were strengthened by constructing new cast-in-place concrete piles, and the top of the bent piles was tied to a steel beam which was newly placed on both sides of the deck.



(a) Elevation

(b) Side View

(c) Arrangement of Reinforcement

(d) Anchorage of Reinforcing Bars

Fig. 10 Seismic Strengthening of RC Frame by Placing New Reinforced Concrete

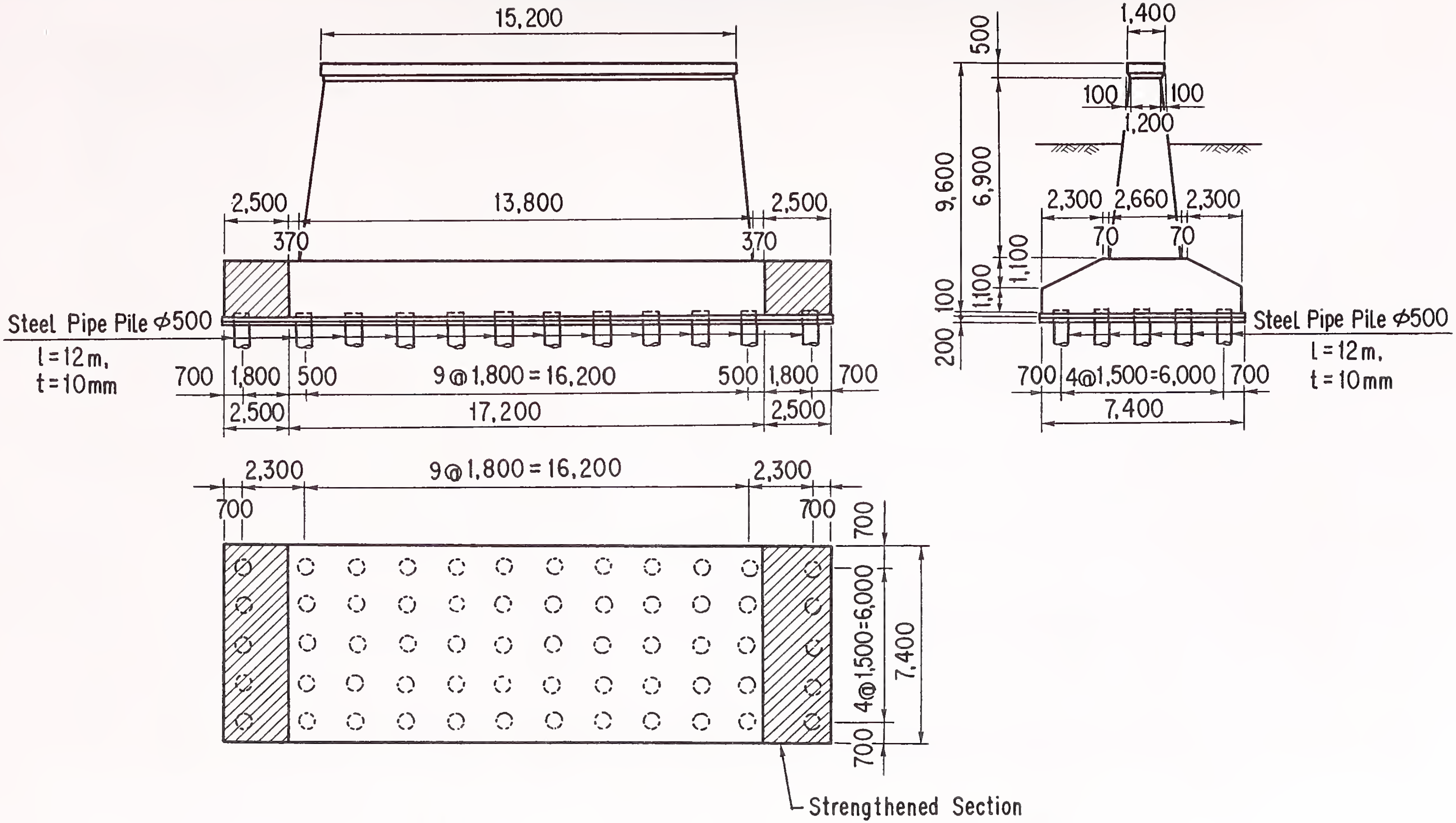


Fig. 11 Seismic Strengthening of Foundation by Increasing Number of Piles

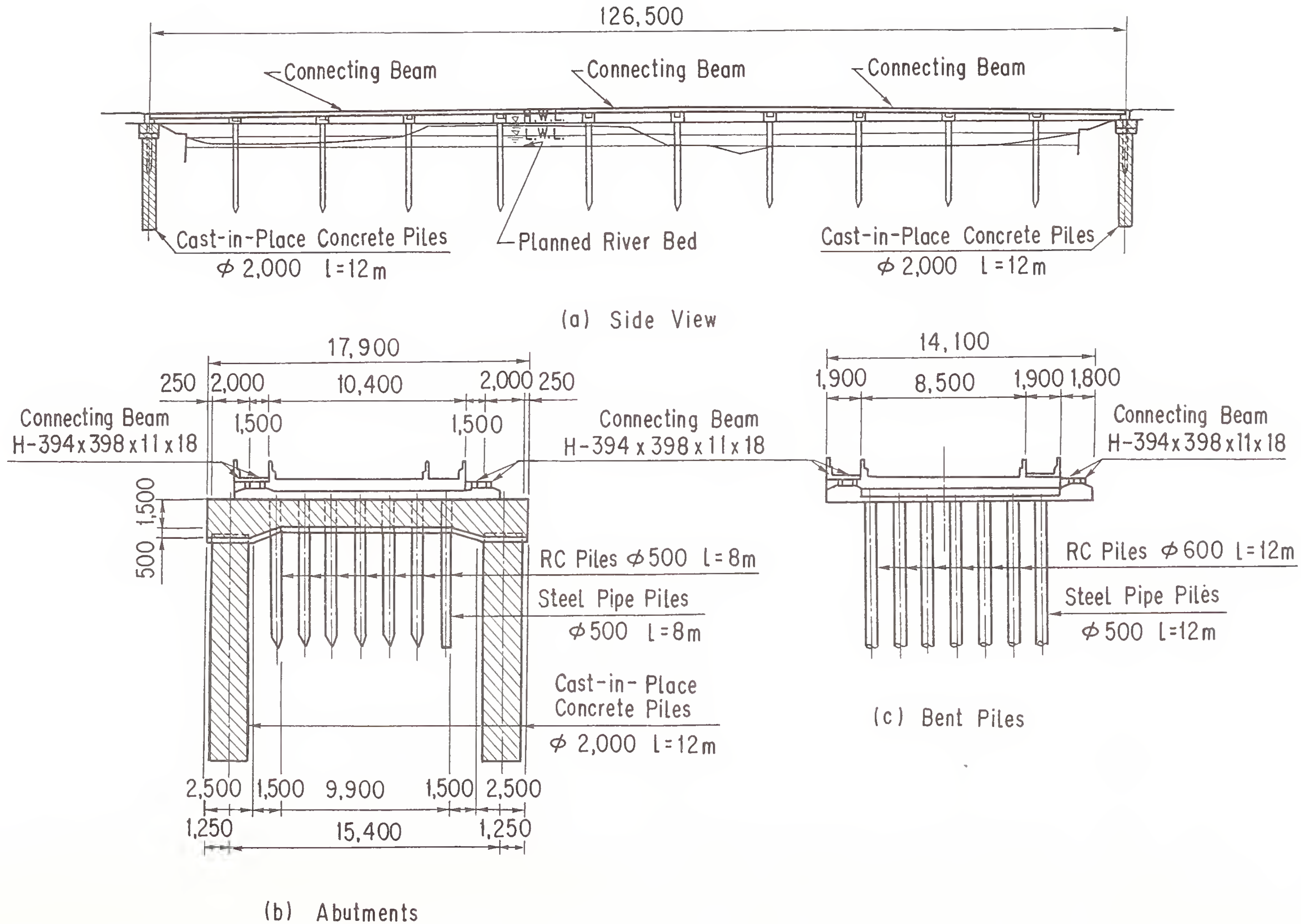


Fig. 12 Seismic Strengthening of Bent Piles on Liquefiable Soils by Connecting Beams

RESEARCH FOR DEVELOPING SEISMIC STRENGTHENING METHODS

Various research efforts have been paid to develop rational seismic inspection and strengthening method. Because current efforts are being concentrated to develop an inspection and strengthening method of reinforced concrete piers with termination of main reinforcement at mid-height, research progress for the termination zone is introduced.

The importance for this type of damage became apparent when a bridge was critically damaged during the Urakawa-oki Earthquake in 1982[13]. The Shizunai bridge which is of five-span continuous girders and reinforced concrete piers suffered destructive damage at the piers as shown in **Photo 2.**

Significant shear failure was found at mid-height of reinforced concrete piers, where some of main reinforcement were terminated. **Photo 3** shows the similar damage during the 1978 Miyagi-ken-oki Earthquake[14]. Although the damage was not so serious as the Shizunai Bridge, it was found that this type of failure occurred widely in the past earthquakes. Because wall type piers were commonly constructed in the past, shear failure developed by the inadequate anchorage length of main reinforcement did not become so pronounced in the past. Single columns with smaller concrete sectional area are likely to be adopted in recent years, and it is considered important to investigate failure mechanism of this type of damage for developing seismic inspection and strengthening method.

Photo 2 Damage of Shizunai Bridge during Urakawa-oki Earthquake in 1982

Photo 3 Damage of Sendai Bridge during Miyagi-ken-oki Earthquake in 1978

Before the Design Specifications of Highway Bridges was revised in 1980, anchorage length of main reinforcement terminated at tension zone was only 20 - 30 times a diameter of reinforcing bars. It was increased in 1980 to effective width of the piers plus 20 times the diameter or the termination has to be made upper than the point where the stress of the reinforcing bar becomes half of the allowable stress.

For investigating failure mechanism, a series of loading test were made at the Public Works Research Institute as shown in **Photos 4** and **5**[15]. **Fig. 13** shows a method proposed based on the tests at PWRI for identifying the vulnerability at the termination zone. The inspection is made by two parameters, i.e., failure mode factor S_y and safety factor at termination zone F_y^T, which are defined as

$$S_y = F_y^T / F_y^B \quad (2)$$
$$F_y^T = M_y^T / M^T \quad (3)$$
$$F_y^B = M_y^B / M^B \quad (4)$$

where

S_y : failure mode factor

F_y^T, F_y^B : safety factor representing strength of pier in terms of bending moment at termination point and base, respectively

M^T, M^B : bending moment developed at termination point and base, respectively, when pier is subjected to a lateral inertia force during earthquakes (tf·m)

M_y^T, M_y^B : yielding bending moment at termination point and base, respectively (tf·m)

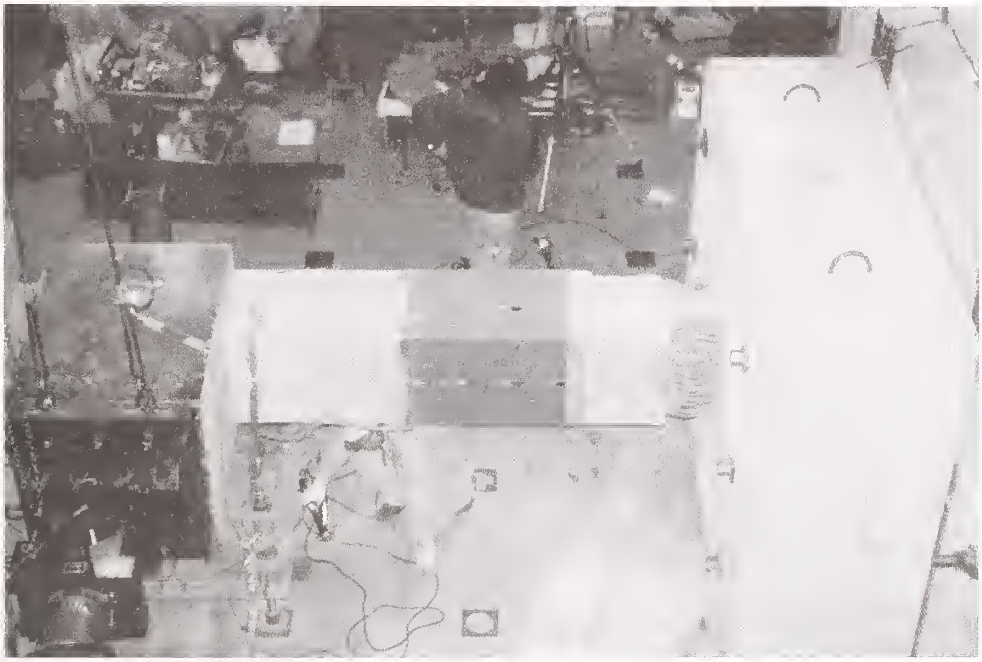


Photo 4 Loading Tests of RC Pier with Termination of Main Reinforcement at Mid-Height (Square Section)

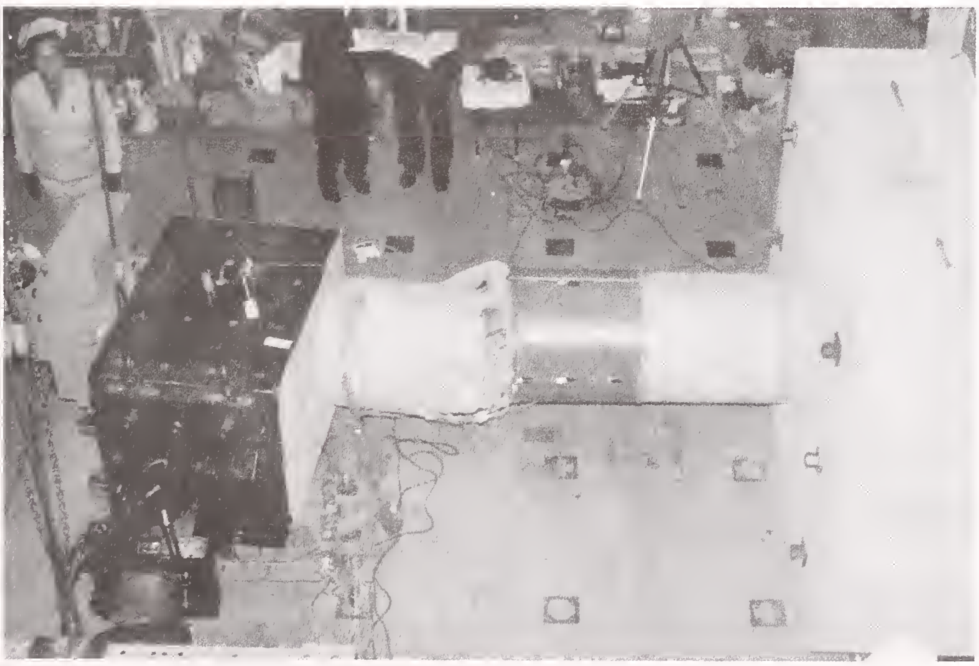


Photo 5 Loading Tests of RC Pier with Termination of Main Reinforcement at Mid-Height (Circular Hollow Section)

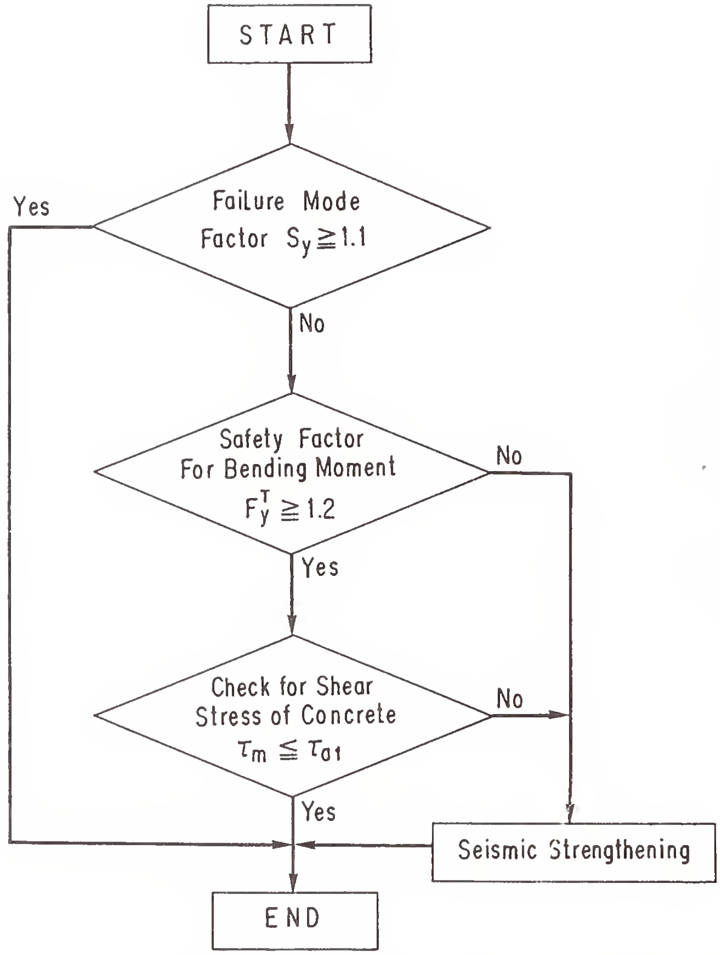


Fig. 13 Seismic Inspection for Reinforced Concrete Piers with Termination of Main Reinforcement at Mid-Height

In Eqs. (2) and (3), the failure mode factor S_y is used to identify where failure is likely developed firstly at either termination zone or base, and safety factor F_y^T represents redundancy to failure at the termination zone. With use of S_y and F_y^T, the decision is made in accordance with **Table 11** by following a flow chart as shown in **Fig. 13**. Check of shear stress is also made as shown in **Fig. 13**.

Fig. 14 shows the effectiveness to identify where the failure was developed by means of the failure mode factors S_y. Failure is likely to be developed at the termination zone when $S_y \leqq 0.9$, while, flexural failure at the base tends to be developed when S_y is larger than or equal to 1.1 as shown in **Table 11.**

Fig. 15 shows how the safety factor F_y^T is effective to assess the damage degree. The damage degree is defined as the failure which is possibly developed at the loading stage of 5 times the yielding displacement ($5\delta_y$). From **Fig. 15,** it is clear that significant failure including rupture of main reinforcements tends to be developed when $F_y^T < 1.2$.

Table 11 Estimation of Seismic Vulnerability on the Strength of Termination Zone by Failure Mode Factor S_y and Safety Factor F_y^T

(a) Estimation of Failure Mode

Failure Mode Factor Sy	Failure Mode
$Sy \geqq 1.1$	Flexural Failure at Base
$Sy < 1.1$	Vulnerable to Failure at Termination Zone

(b) Estimation of Damage Degree (In Case of $Sy < 1.1$)

Safety Factor F_y^T	Damage Degree at Termination Zone at the Loading Displacement of $5\delta y$
$F_y^T < 1.2$	Rupture and Serious Buckling of Main Reinforcement
$F_y^T \geqq 1.2$	Diagonal Cracks, Spalling-off of Cover Concrete, Slight Buckling of Main Reinforcement

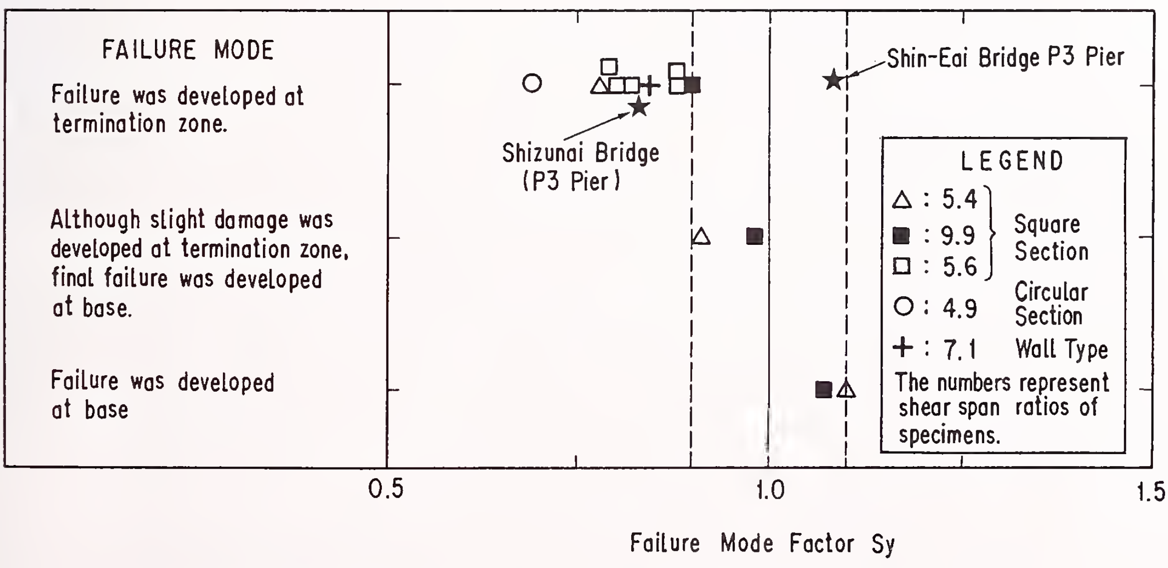


Fig. 14 Estimation of Failure Mode by S_y

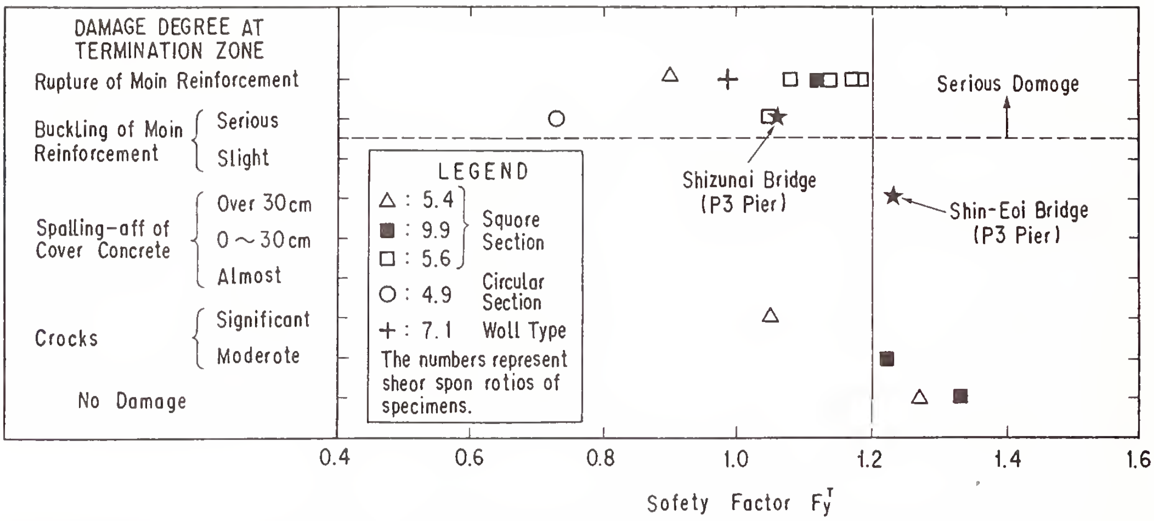


Fig. 15 Relation between Safety Factor F_y^T and Damage Degree at the Loading Displacement of $5\delta_0$

Seismic strengthening methods for the termination zone are being developed at various organizations[16)-21)]. There are several possible ways for strengthening the termination zone. One is to place new reinforced concrete section around the existing section. However, in this method, increase of the mass of the pier for requires additional increase of the foundation, and noise and air pollution during tipping of cover concrete make it difficult to be adopted in urban area. To avoid such difficulties strengthening by means of a steel jacket wrapped around the existing pier seems promising. Another wrapping by means of carbon fiber is also being studied[18)].

A series of dynamic loading tests of specimens strengthened by a steel jacket are being made at the Public Works Research Institute jointly with the Metropolitan Expressway Public Corporation and Hanshin Expressway Public Corporation[15)-17)]. Parameters considered in the tests are length and thickness of steel jacket, and injection material between steel jacket and concrete. Tests are being made for circular, square and wall type piers.

According to the past tests, steel jackets were effective to strengthen piers vulnerable to shear failure at the termination zone. The minimum length of the steel jacket may be 1.5 to 2.0 times the pier width D. Epoxy resin seems better than concrete mortar as an injection material, because separation of steel jacket from concrete surface is more restricted.

Based on the test results, about 20 piers each were strengthened at the Metropolitan Expressway Public Corporation[16)] and Hanshin Expressway Public Corporation[17)]. **Figs. 16** and **17**, and **Photos 6** and **7** show seismic strengthening of reinforced concrete piers. The piers were covered fully from the top to the bottom by steel jackets with thickness of 6 mm and epoxy resin was used injected.

Photo 6 Seismic Strengthening of RC Piers on Metropolitan Expressway by Steel Jackets

Photo 7 Seismic Strengthening of RC Piers on Hanshin Expressway by Steel Jackets

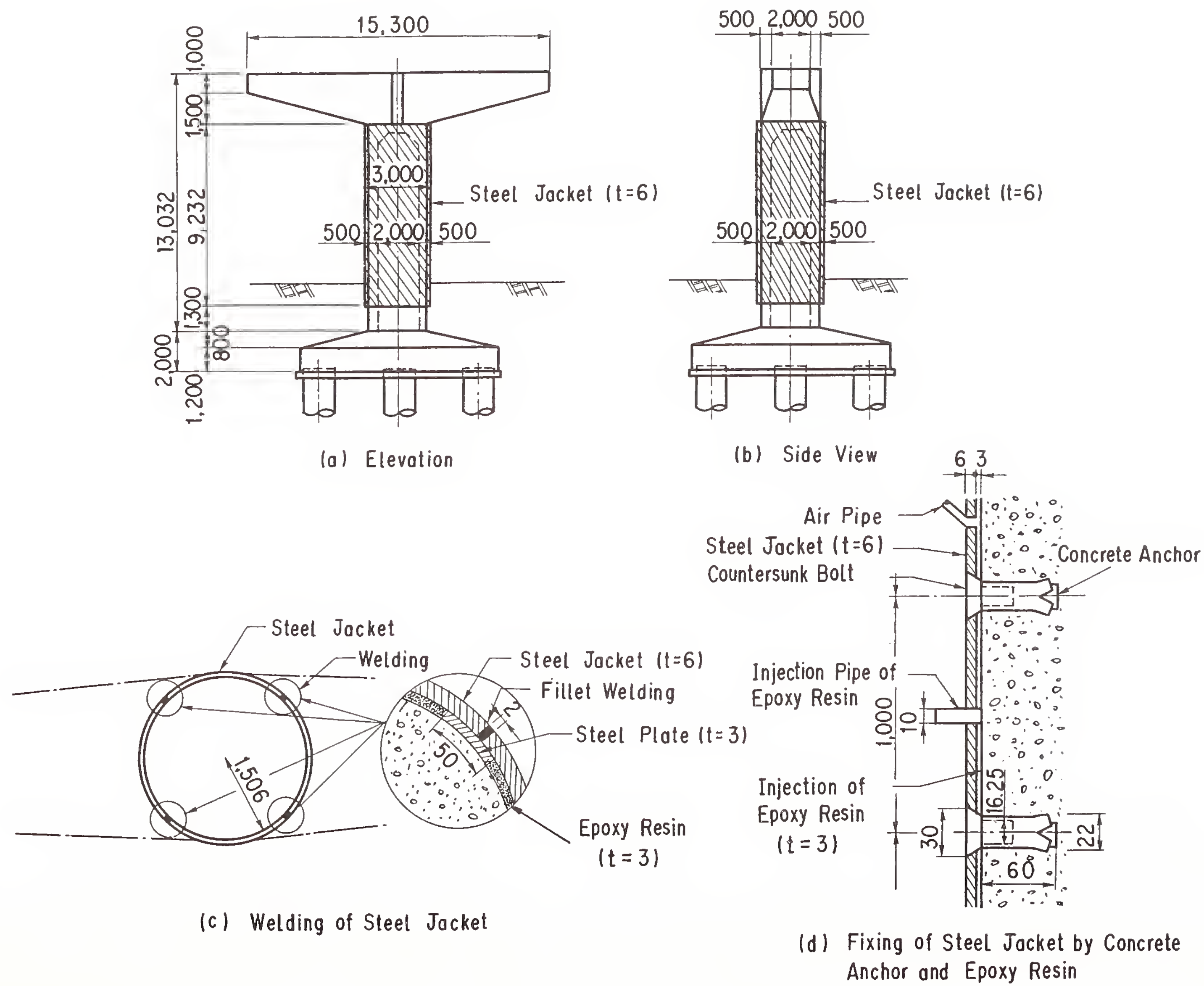
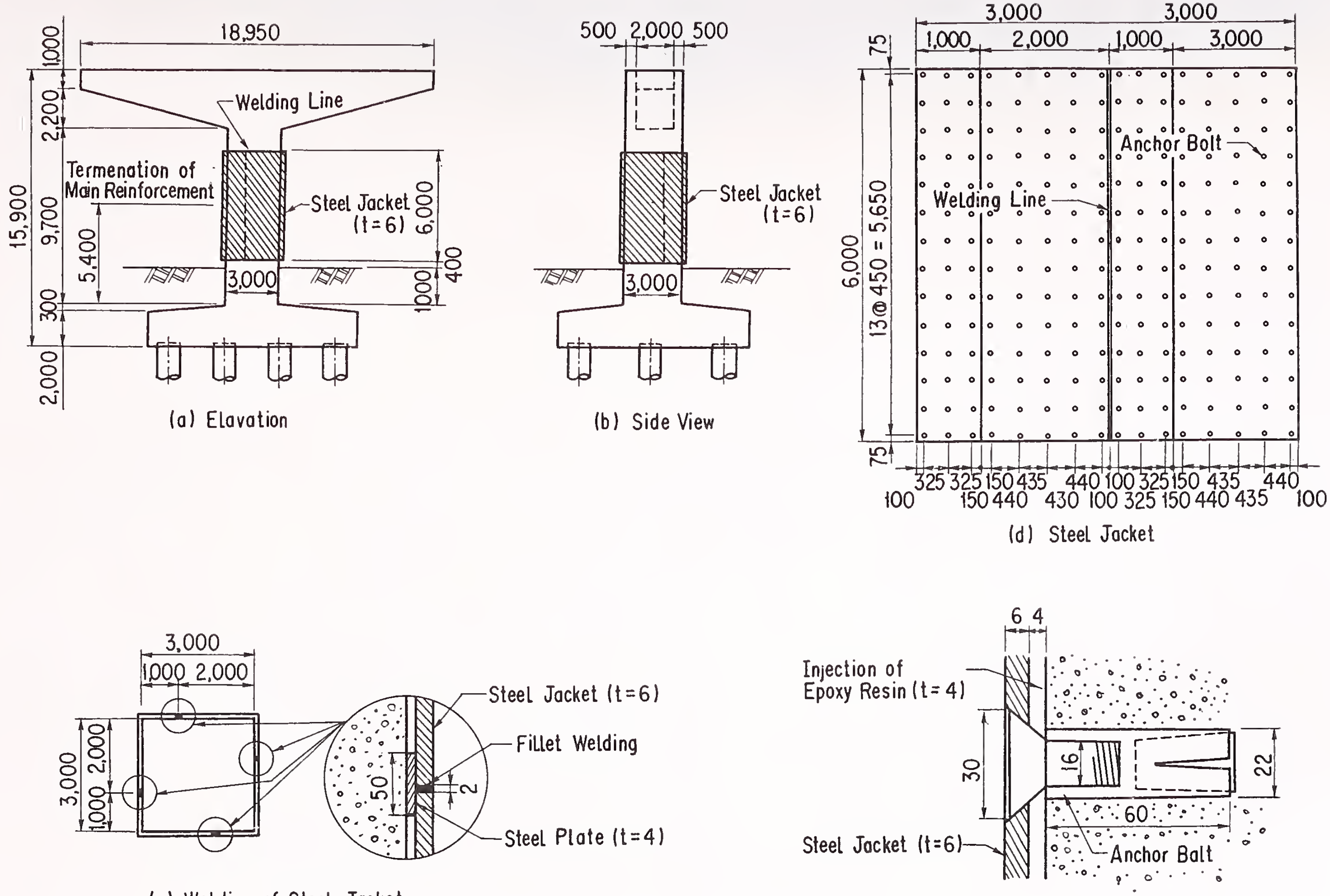


Fig. 16 Seismic Strengthening of RC Piers on Metropolitan Expressway by Steel Jackets



(a) Elavation

(b) Side View

(d) Steel Jacket

(c) Welding of Steel Jacket

(e) Fixing of Steel Jacket by Concrete Anchor and Epoxy Resin

Fig. 17 Seismic Strengthening of RC Piers on Hanshin Expressway by Steel Jackets

CONCLUDING REMARKS

Because seismic design for highway bridges was initiated in 1926, and because revision and amending of seismic design codes have been made based on bitter experiences learned from a number of seismic damage in the past, seismic performance of highway bridges designed by the current seismic design code is considered adequate. However, there are a number of existing bridges vulnerable to seismic effects, which were designed by the past design practices.

By the great efforts in the past seismic strengthening works, installation of falling-off prevention device is now almost being completed for important bridges, but seismic strengthening of foundations and piers have been made for few bridges. Therefore it is now facing to a time to initiate seismic strengthening of foundations and piers. For this purpose, it is absolutely important to develop a reliable seismic inspection method of existing highway bridges. Further research efforts for improving reliability and accuracy of seismic inspection method is required as well as development for seismic strengthening method.

FUTURE DIRECTION

It is required to develop various subjects for seismic inspection and seismic strengthening including the following issues :

a) Level of Seismic Strengthening
There is no wonder to require strengthening for those bridges vulnerable to developing critical failures such as falling-down during an earthquake. However, from the view of importance of highway bridges as a vital component of transportation facilities, a certain type of bridges requires to keep their function without causing damage even after a destructive earthquake. On the other hand, there may be bridges which were constructed in old time and are now in use for a few amount of traffic. Depending on importance, impact causing indirect damage, difficulty for repair and service time left before replacement, an appropriate evaluation method for deciding priority of seismic strengthening and for deciding the target level required for strengthening is important.

b) Improvement of Reliability of Seismic Inspection
It is required to have more reliable evaluation on the seismic performance of bridges for spending a big amount of budget such as for seismic strengthening of foundations. The point required in the seismic inspection is not to know whether the design of a highway bridges is appropriate in accordance with the current design specifications, but to know whether the bridge could perform in a way expected in accordance with the target of seismic strengthening during an anticipated earthquake. Much more improvement on the structural

performance is required.

c) Development of Seismic Strengthening Method for Vulnerable Highway Bridges
Because there are several possible modes which lead to critical failure of highway bridges, development of seismic strengthening method is required for them. These includes the strengthening method for bent pile foundation, reinforced concrete frame on two independent caisson foundations and plain concrete piers. Strengthening method of reinforced concrete piers with termination of main reinforcement with inadequate anchorage length is also important. Strengthening methods by applying new materials such as carbon fiber and fiber reinforced plastic, and new technology such as connection of adjacent girders or slabs, non-destructive inspection method and devices for preventing falling-off of superstructure with energy absorbing capability are also to be developed.

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RECENT ADVANCES IN SEISMIC DESIGN AND RETROFIT OF CALIFORNIA BRIDGES

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ABSTRACT

This paper describes the damage and lessons learned from the most recent major earthquakes, the resulting bridge seismic design and detailing changes and the seismic retrofit program implemented by the California Department of Transportation after the 1971 San Fernando Earthquake and accelerated after the 1989 Loma Prieta earthquake. The goal of the program is to reinforce most of the bridges designed and constructed prior to acquiring the current levels of knowledge of seismic loading and structure response (about 1978) to improve their structural ductility and resistance to the major factors which contributed to damage and collapse in the San Fernando and Loma Prieta events. The program has been executed in two phases. The initial phase, completed in 1988 at a total cost of $54 million, provided reinforcement to superstructures of 1260 bridges by connecting all narrow expansion joint seats with hinge restrainers and anchoring girders and other superstructure elements to the substructure. The $3.4 billion single and multiple column retrofit phase, now under active implementation, is designed to reinforce substructure elements and create ductile members by increasing confinement. A Risk Analysis procedure was developed to prioritize the bridges for seismic upgrading so that highest risk bridges are retrofitted first. Legislation was recently enacted to require the Department of Transportation to also take the lead role in seismically retrofitting all locally owned bridges in the state. Research has been conducted, and is currently underway, at the University of California, San Diego to test and confirm the validity of several proposed design solutions for seismic retrofitting of existing bridge single column bent substructure elements. Since the Loma Prieta earthquake additional research is being conducted at the University of California at Berkeley, San Diego, Irvine and Davis to develop and test retrofit techniques for multiple column bents and double level structures, including abutment and footing details. Some full scale testing was completed on a standing portion of the Cypress Street viaduct in Oakland to determine its fundamental response characteristics and to test some techniques which may be applied to the double deck structures in San Francisco and on complex structures with outrigger supports and other non standard support configurations throughout the state.

BACKGROUND

The 1971 San Fernando earthquake caused substantial damage to recent bridge construction and exposed a number of deficiencies in the bridge design specifications of that time. These deficiencies have the potential to impact dramatically on transportation lifelines and the traveling public today. Bridge design specifications were immediately modified to correct the identified deficiencies on

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newly designed bridges. Structures in-place however, have served to be a substantially more challenging problem. Research was undertaken both in the United States and overseas to improve analytical techniques, and to provide basic data on the strengths and deformation characteristics of lateral load resisting systems for bridges. Damage to long multiple level bridges from the October, 1989 Loma Prieta earthquake showed the need to more carefully consider longitudinal resisting systems because earthquake forces cannot be carried into abutments and approach embankments as they can on shorter bridges.

The initial phase of the Bridge Seismic Retrofit Program involved installation of hinge and joint restrainers to prevent deck joints from separating. This was the major cause of bridge collapse during the San Fernando earthquake and was judged by Caltrans engineers to be the highest risk to the traveling public. Included in this phase was the installation of devices to fasten the superstructure elements to the substructure to resist vertical accelerations and also to prevent superstructure elements from falling off their supports. This phase was essentially completed in 1989 after approximately 1,260 bridges on the State Highway System had been retrofitted at a cost of over 55 million dollars.

While the hinge and joint restrainers performed well, shear failure of columns on the I-605/I-5 separation bridge in the moderate Whittier Earthquake of October 1, 1987 reemphasized the inadequacies of pre-1971 column designs. Even though there was no collapse, the extensive damage resulted in plans for basic research into practical methods of retrofitting bridge columns on the existing pre-1971 bridges. The research program was initiated in early 1987 and is currently being conducted at the University of California at San Diego. Funding levels for implementation were increased four fold after the Whittier earthquake.

The Loma Prieta earthquake of October 17, 1989 again proved the reliability of hinge and joint restrainers but the tragic loss of life at the Cypress Street Viaduct on I-880 in Oakland emphasized the necessity to immediately accelerate the column retrofit phase with a higher funding level for both research and implementation. Other structures in the earthquake affected counties performed well, suffering the expected column damage without collapse. With the exception of an outrigger column-cap confinement detail, those bridges using the post-1971 design specifications and confinement detail changes performed well. Research and analysis subsequent to the Loma Prieta event have shown conclusively that a column pedestal detail unique to the Cypress structure was the main cause of the total collapse. The effect of the response of deep soft soils in the structure foundations also proved to be a contributing factor which must be analyzed and included in future design procedures, especially for long, tall structures with relatively high periods of vibration.

PRE AND POST 1971 COLUMN DESIGN

Bridge columns designed before the 1971 San Fernando earthquake typically contain very little transverse reinforcement. A common detail for both circular and rectangular columns consisted of #4 (12.7 mm diam.) transverse peripheral hoops at 12 inch to 18 inch

(300 mm to 450 mm) centers, regardless of column size and area of main reinforcement. Also, it was common practice to extend short lengths of dowel or tails on the footing reinforcing steel out of the footing and lap splice the main column reinforcing steel cage at that point. As a consequence of these details, the ultimate curvature capable of being developed within the potential plastic region is limited by the strain at which the cover concrete starts to spall. The result is flexural failure resulting from inadequate ductility capacity, or shear failure due to lack of adequate shear reinforcement. Tests conducted at UC San Diego confirm this theory. Several bridges suffered column shear failures due to the elastic design philosophy under which they were designed prior to 1971.

Columns designed since 1971 contain a slight increase in the main column reinforcing steel and a major increase in confinement steel over the pre-1971 designs. All new columns, regardless of geometric shape, are reinforced with one or a series of spiral wound circular cages. The typical transverse reinforcement detail now consists of #6 (19 mm diam.) hoops or spiral at three inch (76 mm) pitch. This provides approximately eight times the confinement reinforcing steel in columns than what was used in the pre 1972 non-ductile designs. All main column reinforcing is continuous into the footings and superstructure. Splices are mostly welded or mechanical, both in the main and transverse reinforcing. Transverse reinforcing steel is designed to produce a ductile column by confining the plastic hinge areas at the top and bottom of columns.

RETROFIT PHILOSOPHY

There are some 24,000 highway structures for which the State of California is responsible in some capacity. It is economically unrealistic to suggest that every structure be immediately retrofitted to withstand maximum magnitude earthquakes without some damage. The retrofit philosophy adopted at the start of the Restrainer Retrofit Phase of the program offers reasonable direction to the remaining phases of the Retrofit Program.

It was and still is Caltrans philosophy to first retrofit those structures which pose the greatest risk to the public and are the most vital to the transportation system. The ultimate goal is to see that all of the bridges in the state will be capable of surviving maximum credible earthquakes without collapse. Some damage is inevitable but, with proper retrofitting, it is believed that collapse is preventable and, further, it is believed that damage can be held to a minimum, to the extent that the critical elements of the transportation system can remain open and functioning during a civil disaster or during repair. As we get into the actual analysis and design it is becoming apparent that retrofitting many older structures to this standard may not be cost effective. It is highly probable that we may have to accept some period of closure on many structures to effect repairs. It is also apparent that some structures will be replaced rather than retrofitted simply as an economic decision. Time and further analysis will bring this problem to the decision makers. As a direct result of the one month loss of the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake, it has been recommended that major transportation structures be designed for higher elastic seismic force levels and longer shaking periods to reduce the damage to non structural type.

To accomplish this goal a new "importance factor" will be introduced into the design and retrofit criteria. This represents a major change in the seismic design criteria for bridges.

PRIORITIZATION PROCEDURE

Identification of bridges likely to sustain damage during an earthquake was an essential first step in the Bridge Seismic Retrofit Program. Damage analysis after the San Fernando earthquake led the caltrans bridge engineers to the conclusion that unconnected joints at hinges, bents and abutments posed the highest threat to collapse. It was also determined that single column supported bents posed the next highest threat. These judgements were documented as policy in 1973 and have been confirmed by performance of both retrofitted and non retrofitted bridges in several subsequent earthquakes. The overwhelming evidence from Loma Prieta supports the priority of retrofit that had been established in 1973 after analysis of bridge damage caused by the San Fernando earthquake.

After the hinge restrainer phase of the retrofit program was completed, the department began to prioritize the single and multiple column supported structures for sequence of upgrading. What can be classified as a level one risk analysis was employed as the framework of the process which led to a consensus list of risk prioritized bridges.

A conventional risk analysis produces a probability of failure or survival. This probability is derived from a relationship between the load and resistance sides of a design equation. Not only is an approximate value for the absolute risk determined, but relative risks can be obtained by comparing determined risks of a number of structures. Such analyses generally require collections of large quantities of data to define statistical distributions for all or at least the most important elements of some form of analysis, design and/or decision equations. The acquisition of this information can be costly if obtainable at all. Basically, what is done is to execute an analysis, evaluate both sides of the relevant design equation, and define and evaluate a failure or survival function. All of the calculations are carried out taking into account the statistical distribution of every equation component designated as a variable throughout the entire procedure. To avoid such a large time consuming investment in resources and to obtain results which could be applied quickly as part of the Single Column Phase of the Retrofit Program, an alternative was recognized.

What can be called a level one risk analysis procedure was used. The difference between a conventional and level one risk analysis is that in a level one analysis judgements take the place of massive data supported statistical distributions.

This level one risk analysis has been revised several times and will be continually improved with time and more information. The bridges, especially those in the mid-range of the risk list, will be reprioritized and the risk lists revised. Meanwhile, we were able to identify the most critical or highest risk bridges and get retrofit contracts underway. With the prediction of activity on nearby faults within three to seven years, Caltrans did not have the luxury of time to conduct a conventional risk analysis. The level one risk analysis procedure we used can be summarized in the following steps:

1. **Identify major faults with high event probabilities (priority one faults).**

This step was carried out by consulting the California Division of Mines and Geology and recent US Geological Survey studies. A team of seismologists and engineers identified seismic faults believed to be the sources of future significant events. Selection criteria included location, geologic age, time of last displacement (late quarternary and younger), and length of fault (10 km min.). Each fault recognized in step 1 was evaluated for style, length, dip and area of faulting in order to estimate potential earthquake magnitude. Known faults were placed in one of three categories; minor (ignored for the purposes of this project), priority two (mapped and evaluated but unused for this project), or priority one (mapped, evaluated, and recognized as immediately threatening). Despite the fact that we use Maximum Credible events for analysis of a typical bridge, we have completed a form of probabilistic approach in the above procedure. On major structures a comprehensive hazard analysis, using the probabilistic approach is conducted. After identification of single column supported bridges close to priority one faults the single column program was initiated. After the Loma Prieta earthquake it was decided that all remaining structures were to be evaluated equally, regardless of whether they were affected by priority one or priority two faults.

2. **Develop attenuation relationships at faults identified in step 1.**

An average attenuation model was developed by Mualchin of the California Division of Mines and Geology to be used throughout the state. It is the average of several published models. Mualchin, a well known seismologist, is now a member of the Caltrans Office of Earthquake Engineering

3. **Define the minimum ground acceleration capable of causing severe damage to bridge structures.**

The critical (i.e., damage causing) level of ground acceleration was determined by performing nonlinear analyses on a typical highly susceptible structure (single column connector ramp) under varying maximum ground acceleration loads. The lowest maximum ground acceleration that demanded the columns to yield (provide a ductility ratio of 1.3) was defined as the critical level of ground acceleration. That level of ground acceleration was 0.5g. Loma Prieta proved this assumption to be wrong and adjustments have been made in current evaluations. Based on input from our Geologists and the type of material over the bedrock, an acceleration level as low as 0.25g could be critical.

4. **Identify all the bridges within high risk zones defined by the attenuation model of step 2 and the critical acceleration boundary of step 3.**

The shortest distance from every bridge in California to the two closest priority one faults was calculated. Each distance was compared to the distance from each respective level of magnitude fault to a 0.5g decremented acceleration boundary. If the distance from the fault to the bridge was less than the distance from the fault to the 0.5g boundary, the bridge was determined to lie in the high risk zone and was added to the screening list for prioritization. The prioritization procedure is described below.

The Caltrans Division of Structures has developed a computerized data base which has the coordinates of all 24,000 State, County and City bridges stored. We can produce a map of the entire state or any portion of the state showing the bridges, the major faults and an overlay of the combinations. These maps can be viewed on the computer screen or printed for use by designers in screening to identify high risk bridges. The procedure is quite simple, using the computer data base.

1. Locate all Highway Bridges on the State System

2. Locate all Earthquake Faults

3. Determine those structures that are in a high risk zone.

5. Prioritize the threatened bridges by summing weighted bridge structural and transportation characteristic scores.

This step constitutes the process used to prioritize the bridges within the high risk zones to establish the order of bridges to be investigated for retrofitting. It is in this step that a risk value is assigned to each bridge. A specifically selected subset of bridge structural and transportation characteristics of seismically threatened bridges was drawn from the California Department of Transportation structures computer database. Those characteristics were:

Ground Acceleration
Route Type-Major or Minor
Average Daily Traffic(ADT)
Column Design-Single or Multiple Bents
Confinement Details of Column(relates to age)
Length of Bridge
Skew of Bridge
Availability of Detour

After evaluating the results of the 1989 Loma Prieta earthquake, Caltrans engineers modified the Risk Analysis Algorithm by adjusting the weights of the original characteristics and adding the following new characteristics to the list. A total of 18 different characteristics are now evaluated. One of the most important new factors is the soil type at the site; this is due to the significant influence of deep soft soils on structure performance during the Loma Prieta event. The addition of exposure and type of facility crossed recognize the higher damage and loss effects of long structure failure and those on high volume traffic routes.

Soil Type
Hinges, Type and Number
Exposure(Combination of Length, Height & ADT)
Abutment Type
Type of Facility Crossed

All of the components listed above were considered in the Caltrans risk evaluation. A team of experts representing hundreds of years of experience in the fields of bridge design, construction and maintenance engineering and geotechnical and geological sciences were employed to identify and weight appropriate risk components. Normalized preweight characteristic scores from 0.0 to 1.0 were assigned based on the information stored in the database for each bridge. Scores close to 1.0 represent "high risk structural" characteristics or "high cost of loss" transportation characteristics. The preweight scores were multiplied by prioritization weights. Post-weight scores were summed to produce the assigned prioritization risk value.

In summary, an evaluated risk number is calculated respecting source, distance, local geologic site conditions, bridge structural components and what is at risk in addition to the bridge.

Determined risk values are not to be considered exact. Due to the approximations inherent in the judgements adopted, the risks are no more accurate than the judgements themselves. The exact risk is not important. Prioritization list qualification is determined by fault proximity and empirical attenuation data and not so much judgement. Therefore, a relatively high level of confidence is associated with the risk ranking and identified bridges on the initial list of threatened bridges. Relative risk is then used to establish the order of bridges to be investigated in detail for possible need of retrofit by the designers. The risk analysis offers consistency in applying the judgements adopted to all bridges in the state.

A number of assumptions were made in the process of developing the prioritized list of seismically threatened bridges. This is typical of most engineering projects. These assumptions are based on what is believed to be the best engineering judgement available. It seems reasonable to pursue verification of these assumptions some time in the future to insure that we have not missed anything. Two steps seem obvious: (1) monitoring the results of the design engineer's retrofit analyses, and (2) executing a higher level risk analysis where necessary and better data are available.

Important features of the prioritization procedure are the ease and minimal cost with which it was carried out and the database, highlighting bridge characteristics, to identify structures in need of retrofit. This database will serve as part of the statistical support for the current risk analysis and of any future conventional risk analysis. The additional accuracy inherent in a higher order risk analysis will serve to verify previous assumptions, provide very good approximations of actual structural risk, and develop or evaluate postulated scenarios for emergency responses. In the interest of getting the job accomplished the shorter procedure has been used and the entire 24,000 bridge inventory has been risk analyzed. More detailed analyses can now be utilized to fine tune the list but it is reasonable to analyze only selected structures for this purpose. The original lists of 12,000 local and 12,000 state bridges were reduced to 4000 local and 7000 state bridges which require more detailed review and, in many cases, dynamic analysis. In the next step a manual screening process is being used which includes review of "AS-Built" plans by three engineers to identify bridges with column and footing details that appear to need upgrading. Some structures were eliminated based on details, location and other judgement factors. In the final analysis, our designers and field maintenance and construction engineers are being asked to just look at bridges with which they are familar and alert us to any they question. No risk algorithm will replace common sense and judgement, however, it is an excellent method of screening large numbers of bridges to begin the process.

DESIGN

The California Department of Transportation implemented the single column phase in 1986 and the multiple column phase (covering all remaining bridges) of the Bridge Seismic Retrofit Program in 1990. In the current program entire structures may be subject to modification to reduce the likelihood of catastrophic failure during a large earthquake. The main goal of the program is to prevent collapse, but a secondary goal is to increase serviceability by reducing damage to a level that can be repaired without closing the structure to traffic. This is a goal that may have to be modified as cost-benefit considerations are evaluated. Special attention is being focused on overall structure response. Two key items are recognized as being primary in achieving this goal: (1) providing continuity in superstructures at joints to prevent supported elements from collapsing, and (2) increasing ductility at certain locations throughout the structure and specifically in columns and column-superstructure and column-footing joints. The key to the column retrofit phase is ductility provided by the supplemental external confinement and improvements to foundations and abutments. Currently, that external confinement consists of steel shells designed to resist the column shear forces and provide confinement of main column reinforcing at the plastic hinge location. Other techniques for wrapping with cables or fiber reinforcing are also being tested. Foundations and abutments are being reinforced with additional piles, confinement reinforcement, soil anchors, pile shaft retainers, bolster walls and other details to improve seismic resistance.

Design engineers have been assigned the final task of verifying or discrediting the prioritized bridges' need for retrofitting and then, if necessary, developing retrofit contract plans. Verification of the need for retrofit is necessary due to the possibility of prioritized bridges already being capable of withstanding the maximum credible earthquake. This can only be determined by additional analysis and will be the case when judgements made in the prioritization process prove to be too conservative. Emphasis is being placed on evaluation of the total bridge during this design phase. In most cases a dynamic analysis is necessary before the final decision to retrofit or not can be made. Results from current research and analysis indicate that we have been too conservative in the strength assessment of existing non-ductile structures. It is probable that application of higher allowable ductility factors in the analysis of these older structures will reduce the total number that must be retrofitted or that the amount of retrofitting on each structure can be reduced.

A number of structure modification schemes are under consideration in the bridge seismic retrofit program (since the Loma Prieta earthquake the Single and Multiple Column phases of the program have been combined with the old superstructure phase into the single, "Bridge Seismic Retrofit Program" with no phases). Restrainer and superstructure retrofit techniques which have proven to be successful during recent earthquakes will continue to be used to effectively force superstructures to act more like single units. The problems associated with preventing the type of substructure failures seen at San Fernando, Whittier and Loma Prieta are complicated. If all columns are made to carry earthquake loads, then so must the footings and the pile groups. This is not an acceptable solution economically. Some columns may be allowed to pin at a point of contraflexure under dynamic loading while selected retrofitted columns and the abutments absorb the seismic energy and continue to hold the damaged bridge up, preventing bridge collapse. Substructure modifications are currently being evaluated by researchers at the University of California, Berkeley and San Diego. The conclusions drawn from their work will be used to standardize retrofit design schemes. All retrofit designs are subjected to intense scrutiny by a group of experienced design supervisors, seismologists, geologists and dynamic analysis experts in a "strategy meeting" where the project designer must justify design assumptions and retrofit schemes. These sessions often require several hours and may result in major redesign to achieve an acceptable solution. In addition, Seismic Safety Peer Review Panels of acknowledged experts in structural engineering and seismology are utilized to review the design criteria, assumptions and final plans on major, complicated structures.

RETROFIT DETAILS

Some typical superstructure retrofit methods used to date have been to add restraining cables or rods at piers and hinges and add shear keys at abutments. In some cases new, longer hinge and abutment bearing seats had to be installed. Where this was not practical heavy duty pipe hinge extenders were installed to resist both horizontal and vertical seismic forces. Additionally, these hinge extenders carry the supported portion of a bridge in the unlikely event it were to move off the narrow hinge bearing seat. The continuing Legislatively mandated Bridge Seismic Retrofit Program will include this type of retrofitting on all state and other publicly owned bridges (i.e. County, City, Transit Systems, Other State agencies).

Column ductility will be increased by the installation of external confining jackets to provide the concrete confinement now provided internally in new design details by the spiral reinforcing cages. These external jackets are primarily structural steel but we have tested and will begin using fiberglass wrapped jackets and possibly prestressing strand if construction details for wrapping around small diameter members can be developed. On single column supported bents the columns and footings must both be retrofitted to provide resistance to overturning from lateral forces. Most footings on the older structures must be rebuilt with the addition of top reinforcing steel mats and additional piles or soil anchors to provide the required additional overturning resistance. In many cases "super bents" must be constructed in areas that are accessible and out of the way of traffic. These bents are designed to take most of the lateral forces and preclude or reduce the need for retrofit on columns near traffic lanes.

Work is beginning on the multiple column structure retrofit as research results become available. On most of the multiple column supported structures hinges can be allowed at the top of footing and this precludes the need for footing reinforcement. Most of the details are similar to those used on single column supports. The total program will consist of retrofitting approximately 4500 bridges on the state system and 1500 bridges on the local city and county systems. Priority of funding and implementation will be given to the 750 bridges on the state system and 180 bridges on the local system which must be under contract by December 31, 1993 to meet the current Seismic Safety Criteria. The remaining bridges will be retrofitted primarily to reduce future damage and will be programmed over the last half of the decade. During this period we will go back and look at all original phase I bridges again to insure that none are missed by the initial screening processes.

RESEARCH AND PROOF TESTING

Work at the University of California at San Diego was funded in 1987 and consisted of half scale model testing of the various single column bent retrofit techniques. Theoretical calculations and research work previously conducted in New Zealand by Doctor Nigel Priestley showed that enclosing the columns in steel casings could significantly increase their shear strength and ductility by providing the additional confinement at the hinge areas. A series of tests have been completed on round columns with outstanding results. Based on this work, the first contracts for bridge column retrofit were advertised in January, 1990 and work is underway in the Los Angeles area on more than 100 bridges. By the end of 1991 we will have approximately 277 bridges under contract for retrofit statewide. A second series of tests was begun in February, 1990 on rectangular single column bents and the results are being implemented now. Both series of tests include models of the prototype columns with the pre-1971 reinforcing details without retrofitting, retrofitted columns using the steel shell confinement and a post damage retrofitted column using the steel shell to determine whether a non-retrofitted damaged column can be salvaged after an earthquake. Typical displacement ductility factors on retrofitted undamaged columns are 6 to 8. On the post damage retrofitted column a ductility factor of 2 was achieved. Even though displacement ductility factors of 6 to 8 have been common in these first tests, our analysis procedure is based on moment ductility demand no greater than 4. Tests have recently been conducted on columns retrofitted by wrapping pre-stressing strand and fiber reinforced sheet wrapping similar to the technique used on large concrete tanks and industrial smoke stacks. Initial results on the fiber wrapped technique are encouraging and contracts will be advertised to test the constructibility and cost effectiveness of this procedure. Additional tests are scheduled to be conducted on these and other retrofit techniques and specific details.

Work has recently been funded at both UC San Diego and UC Berkeley to build and test half scale models of column cap outrigger bent joints and will be available this summer (1991). The most complex model test of the series will be conducted during the summer (1991) on half scale models of the edge beam-column connections proposed for strengthening the double deck viaducts in San Francisco. One of the major problems which will be addressed in this model construction is constructibility of the complex joints.

Work at UC Berkeley is also funded and was started in November, 1990 to test retrofit techniques on other types of multiple column bents. These substructure configurations are more complex and difficult to retrofit but they have not demonstrated as high a risk as do single column supports. During the aftermath of the Cypress Street Viaduct cleanup efforts a three span segment of the standing portion of the viaduct was instrumented and tested by the University of California researchers to determine its fundamental period. Column jacketing retrofit techniques proposed for use on the double deck viaducts in San Francisco were tested to prove their theoretically calculated value and actual reliability in increasing column ductility and shear capacity. This was a unique opportunity to test full scale structure frames to yield, apply several retrofitting techniques and retest the upgraded structure. The results have been published by the University and offer some degree of confidence for use in very specific applications. Lateral load tests of several new types of foundation piles are also scheduled, especially in the San Francisco area where soft soils pose a problem in developing uplift capacity to resist overturning moments. Lateral load capacity tests of large diameter piles up to seven feet will also be conducted at the sites of new construction.

SUMMARY

The procedure used by the California Department of Transportation to identify and prioritize seismically threatened bridges to be investigated for possible retrofit has been presented. The original decision to retrofit deck joints first and columns later was based on experience from the 1971 San Fernando earthquake. Subsequent earthquakes, including the recent Loma Prieta event, have proved the validity of this decision. Several hundred bridges with only the deck joint restrainers in place have performed well during these earthquakes. The level one risk analysis used to prioritize the structures in the single column phase was discussed in which decisions were made based on reasonable judgement instead of massive statistical data. The level one risk analysis offers a procedure to consistently apply knowledge gained from past earthquakes and known characteristics of bridges throughout the state which can be carried out quickly without developing a large, more sophisticated statistical database. Steps will be taken to verify assumptions made in the risk analysis in an attempt to improve confidence in this analysis. A modification of this risk analysis procedure will be used to prioritize the more complex bridges remaining in the program as the candidate list is screened. Research on column retrofit techniques will be continued to refine and improve those techniques. Research on the effects of soft foundation materials and the effects of variable foundation material response on long structures will also be continued.

Finally, the legislative direction and funding is being made available to accelerate the California Bridge Seismic Retrofit program for all publicly owned bridges which require upgrading to meet modern Seismic Safety Standards.

III. SEISMIC DESIGN & ANALYSIS: PIPELINES & TUNNELS, PART ONE

"Repair and Rehabilitation of Buried Water and Sewer Lifelines"
L.R.L. Wang, H. Kennedy

"Investigations on External Force Evaluation in the Seismic Deformation Method"
N. Takahashi, M. Takeuchi, K. Irokawa

"Estimation of System Reliability for Lifeline Standards and Example Using the City of Everett, Washington Lifelines"
D. Ballantyne

"Dynamic Response of Twin Circular Tunnels during Earthquakes"
T. Okumura, N. Takewaki, K. Shimizu, K. Fukutake

"Development of Seismic Design and Construction Standards for Lifelines"
R. Dikkers

REPAIR AND REHABILITATION OF BURIED WATER AND SEWER LIFELINES

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ABSTRACT

Buried water and sewer pipelines have been damaged heavily by recent earthquakes that included the 1985 Mexico City earthquake, the 1987 Whittier, California earthquake and the most recent Loma Prieta earthquake of October 17, 1989. Existing buried pipelines are unsafe even under moderate earthquakes, because most existing water and sewer pipelines have been built without seismic considerations.

The paper describes an ongoing research on repair and rehabilitation of buried water and sewer pipelines for seismic resistance. The project is sponsored by the National Science Foundation and co-sponsored by EBAA Iron Inc., an industry that contributes to the testing program of pipe joints and participates in the overall study. The objectives of the research are (1) to review/synthesize current methods for inspection/detection of damaged/deteriorated pipelines;(2) to document/develop cost-effective methodologies for repair and restoration of damage pipelines and to (3) develop/recommend guidelines/ strategies for effective rehabilitation and strengthening of vulnerable pipelines.

INTRODUCTION

Earthquakes are perhaps the most devastating and destructive in nature. They are unpredictable at the present time and can be violent. Due to the increase of population and the concentration of inhabitants in urban areas, the need to protect human lives and to mitigate the economic loss become increasingly urgent.

Buried lifelines which include water, sewer, gas and oil pipelines have been damaged heavily by earthquakes including the 1985 Mexico City earthquake[1], the 1987 Whittier, California earthquake[2] and the most recent Loma Prieta earthquake[3] of October 17, 1989. Existing buried lifelines are unsafe even under moderate earthquakes.

Because of the importance of lifelines to the health, supply, and safety of the public, lifeline earthquake engineering has drawn the attention of the engineering profession in recent years. Since there is still no codified seismic provision to design buried lifelines, most water and sewer pipelines, including relatively new ones, have been built without adequate earthquake protection. Damages to water and sewer pipelines during earthquakes are unavoidable and economic loss can be substantial. Effective repair methods under emergency situations and for a permanent basis must be developed. Due to the fact that the replacement of all existing water and sewer pipelines is very expensive, if not impossible, the development of effective rehabilitation techniques and strategies is urgently needed.

BACKGROUND

In general there are four causes of seismic hazards to buried water and sewer pipelines, namely:

a) soil straining induced by seismic ground shaking, (i.e. wave propagation effect);
b) differential ground movement/rupture along fault zone, lateral soil spreading or landslides;
c) soil liquefaction induced by ground shaking;
d) earthquake induced internal hydrodynamic surge pressure in water pipes.

Over all, the most frequent failure modes as reported by several investigators [4, 5] were pulled-out of joints and/or crushing of pipe bodies due to seismic shaking in the longitudinal direction; shear and/or bending failures (cracks or fractures) due to soil lateral spreading/ground movement in the transverse direction; longitudinal cracks and bursting holes due to internal pressure and uplift/sinking due to soil liquefaction.

One of the most important elements for repair and rehabilitation of buried water and sewer pipelines would be the pipeline joints/couplings or junctions. However seismic resistant characteristics and energy absorption capability of commonly used joints, bends and junctions are not well understood.

This project also intends to carry out experimental evaluations (statically and dynamically) of some typical (commonly used and newly developed) pipe joints, bends and junctions that can be used effectively for repair and rehabilitation of buried pipelines.

ISSUES CONCERNING REPAIRS AND RETROFIT

Repairs of lifelines may be classified into two categories, a) emergency or temporary repairs and b) permanent or complete repairs. Various temporary and permanent repair methodologies should be developed and transferred to the users for implementation. In fact some repair methods for leaks due to weathering or aging in the normal maintenance routine may be applicable for earthquake damage repairs. This study will conduct a survey of all existing repair methods for pipeline damages. They will be analyzed and synthesized on a comparative basis of their seismic resistant performance, availability, simplicity of operation, durability, and cost.

The retrofitting of existing lines for seismic resistance is urgent needed in seismic zones. However, the replacement of existing water or sewer pipelines is very expensive, if not impossible. The difficulty lies in the facts that a) most water and sewer lines are buried under congested streets and b) many lines are in need of retrofitting. The opening of a street for repair at an isolated location is very difficult and expensive, the opening of all streets for replacing all existing lines at one time is out of the question. At this time, no effective systematic retrofitting method has been developed. Guidelines for small utilities to retrofit their existing facilities do not exist. Two types of retrofit issues need to be considered: a) low-cost modifications that would be applied by a utility for its everyday maintenance program and b) moderate cost retrofit projects as permanent solutions.

OBJECT and SCOPE

The overall objective of this project is to provide an effective repairs guidance and rehabilitation strategy for seismic resistance of buried lifelines.

The scope of this paper covers, but is not limited to, the following items:

- Assessment of Pipeline damage behavior
- Assessment of available methods for inspection/detection of damaged water and sewer pipelines
- Assessment of available methods for repair and restoration of damaged pipelines
- Assessment of available methods for retrofitting/strengthening of vulnerable existing systems
- Testing and evaluation of currently available and/or newly developed pipe joints, bends and junction for repair and retrofit purposes
- Developing repair/rehabilitation guidelines and strategies

These tasks will be studied in detail during the course of the investigation. Some preliminary findings are given below:

ASSESSMENT OF PIPELINE DAMAGE BEHAVIOR

In order to develop effective repair/retrofit methodologies for buried pipelines against earthquakes, one must understand the seismic damage behavior of these pipelines. The types of damage of buried pipelines (water and sewer) from past experiences in the United States[2, 3, 5], Japan[4] and other parts of the world[1,5] can be summarized as follows.

a) Pipeline as a whole
.Waving of center line
.Uplift

b) Pipe Body
.Circumferential cracks
.Longitudinal cracks

.Settlement
.Buckling (beam or shell)

.Breaking of pipe body
.Soil deposit into pipe

c) Joint/Junction
.Shear break
.Bending opening
.Pulling off wall
.Rubber ring displacement
.Mortar seal breaking away
.Loosening and leakage

d) Manhole (Sewer)
.Breakage of top cup
.Breakage of inclined wall, vertical and base wall
.Breakage of mortar joint
.Breakage of pipe connection
.Breakage at intersection

With the experiences learned from earthquakes from the United States, Japan and China, the following general conclusions can be made.

- Rigid joints (such as lead caulked joints) failed more than flexible joints (such as rubber gasket joints)
- With ordinary push-on rubber gasket joints, the joint is weaker than the pipe body itself with respect to longitudinal ground motions.
- Pipe failed more in weaker soil.
- More failures occurred at the connection between manhole or heavy structure and pipe.

Notice that the damage behavior of buried pipelines listed above may be caused by the seismic shaking, large fault movements/ground displacements, soil liquefactions, internal pressure or combination (old pipelines are affected by erosion also). Thus, the repair/rehabilitation of buried pipelines should consider both external (ground/soil) environments and the internal (pipe body/joints or junctions) design parameters of the pipelines.

One should also note that damage to sewer pipelines may continue to appear for several years after the earthquake because the initiation of cracks due to an earthquake may not have been or can not be detected immediately after the earthquake. The assessment of the effectiveness of current methods of inspection/detection of damage of buried pipelines, particularly sewer lines will be an important task.

ASSESSMENT OF INSPECTION/DETECTION AND REPAIR/RESTORATION METHODS

In order to develop effective repair methods and rehabilitation strategies this task is to conduct a thorough survey and analysis of existing methods for inspection/detection, repairs/restoration and retrofitting/strengthening of buried pipelines, including those from the United States, Japan and perhaps other countries when possible.

Inspection/Detection of Damage

In general, inspection and detection of damage may be done in two ways. The first step is an emergency survey/quick inspection for the purposes of quick repair. Then, a thorough damage survey and inspection is needed for systematic restoration of the system. Please note that the damage of water system is easily recognizable because of water flow and pressure drops. However, the survey and inspection of damages of sewer pipeline/manhole need special tools and are laborious. It is important to prepare in advance for the possible types of damage caused by an earthquake.

Following discussions apply mainly to sewer systems, while some to water systems.

Check Points for Emergency Survey/Quick Inspection

The purpose of the emergency survey/quick inspection is to prevent the propagation of a minor damage to a major disaster. The main effort is to limit the effect of the damaged component to the surrounding facilities. The emergency survey/inspection should include main lines, distribution lines, and treatment and disposal plants. Check points for emergency survey/quick inspection are suggested as follows:

.Whether there are unusual or abnormal sign of operations in the pumping stations and/or disposal facilities;
.Whether there are unusual phenomena in manholes and the surrounding area of the pipeline;
.Whether there are leaks from water pipelines or from water storage tanks;
.Whether there are inflows of dangerous material (gas, oil, sandy soil, etc.) into the conduits or manholes;
.Whether there are damages of conduits, manholes, etc.;
.Whether there is any deterioration of pumping capability;
.Whether there is any outflow of sewerage from the manhole.

Methods of Emergency Survey/Quick Inspection

The emergency survey/quick inspection is to obtain information for the purpose of quick repair and restoration of the system.

One method of the emergency survey/inspection is by sight and the other is by instrument. It is also important to observe and record the road condition, manhole condition, and their surrounding environments which may show signs of damage of water or sewer systems. It is also important to inspect the treatment plants and pumping stations. For emergency survey, it is very important to keep a record of inspection supplemented with photography. Sometimes, it is useful to use a helicopter for a quick survey.

Methods of Thorough Survey for Complete Restoration

The damage survey should include inspection of cracks (width, length and depth) of pipe body, damage to joints (breakage or separation), misalignment of pipe axis (vertical and horizontal) and settlement or floating of pipeline or manholes.

The thorough survey/inspection method for buried pipelines can be classified into two types, namely: direct and indirect methods. When possible, it is desirable to use the direct method to find out the exact location and intensity of the damage. The direct method is to examine the damage point by eye or by a remote control camera. Some available direct survey/inspection methods [6] are listed below:

.Actual observation by eye
.Laser beam to check misalignment of pipes (see Figure 1)
.Radar
.Robotics inspection by a video, special water-proof camera, or rolling TV camera with a motor (See Figure 2)

In general, for larger diameter pipes (greater than 150 cm) and manholes, direct visual observations and/or direct measurements by various survey instruments will be preferable. For small diameter-pipes (less than 100 cm), survey by remote control camera, TV camera and/or video, laser beam etc. would be used for these survey/inspection methods. Safety precaution should be made to protect the safety of the workers and instruments during the survey and repair process.

The indirect survey/inspection method is used when the direct method is not available for some reasons or when it is difficult to assess the damage by the direct method. The principle of the indirect method is to observe the flow condition by using some type of instrument. Some of the available indirect survey/inspection method [6] for sewer lines are listed below:

.Smoke test (see Figure 3)
.Added water test for manhole damage
.Air pressure test for pipe and/or joint damage (see Figure 4)
.Stopping water test
.Flow-rate test
.Water quality test
.Relative leakage test
.Infiltration in sewers by pumping water between manholes

In addition, when necessary the cleaning of the pipelines can serve as survey/inspection of damage. One may note that these tests are to measure the status of leakage. Unfortunately these tests can not locate exactly the spot of damage or degree of damage. However, the results can be as a guide to select repair method(s).

ASSESSMENT OF REPAIR & RESTORATION METHODS

Emergency Repair/Restoration Measures

During and just after the earthquake, it is difficult sometimes to carry out all emergency measures because of insufficient manpower and material. Therefore, it is necessary to set the priority of the regions or the tasks that need emergency measures, such as survey/inspection, repair and restoration. The emergency measures should be carried out for the areas where urgency needs exist.

When it is decided that emergency repair and restoration should be carried out, the following measures should be considered according to the intensity or the effects of the damage.

.Stop leakage from pipes/conduits using repair clamps/sleeves
.Drain excess water or waste water using portable pumps
.Set-up temporary conduits or pipes
.Dredge sand/soil in conduit pipe and/or manholes
.Repair gaps between manholes and roads
.Fence the rupture places on damaged roads
.Set-up signs warning of road settlement and/or ruptures
.Fill sinking holes with sand
.Set-up traffic control for the dangerous regions

The best repair technique is one that puts the system back into proper functioning order in the shortest amount of time. For emergency repair, it is to stop leakage from pipes/conduits as quickly as possible. Currently, some known methods (note that some methods are applicable to sewer pipes only) of repair for leakage are as follows:

- Caulking
- Injection of epoxy
- Sealing
- Grouting chemically
- Rubber band on outside surface of pipe
- Rubber band on inside surface of pipe
- Water stopping flexible joint
- Insertion of a new pipe inside damgaged one
- Others

For this assessment program, a clear and concise questionnaire of repair/maintenance practices will be developed and be sent to water and sewer utilities in seismic regions in US, China and Japan. The questionnaires for repair and rehabilitation will include materials/components used, time and labor for installation, strength and durability if available, and cost, etc.

All methods of emergency repair will be analyzed and/or compared on the basis of their seismic resistance performance, availability, practicability, durability and cost. Some methods of repair can only be applied to some type of pipelines. They will be identified and classified for easy usage. Advantages or disadvantages of each repair method will be discussed.

Some of these methods are related to the behavioral damage environments such as soil conditions, ground water level, buried depth or construction methods, comparison of their functional operations of the repairs will be evaluated.

Permanent Repair and Restoration Methods

The purpose of permanent repair and restoration is to rebuild water and sewer system to its original form. As to complete or permanent repairs/restorations of damaged pipelines, some methods are used to repair/restore a damaged pipeline to its original strength while other methods are used to correct misalignment which is important particularly to sewer pipelines.

a) Some methods of repair/restoration to its original strength are given below:

- Replacement of damaged pipes with new pipes
- Replacement of damaged pipe joint with a new joint/junction
- Reuse of old pipes
- Adding concrete around damaged pipe
- Injection of epoxy
- Caulking
- Encasement of pipe
- Others

b) Some methods of correcting misalignment of sewer pipeline are:

- Leveling Techniques
- Injection of epoxy
- Grouting with cement
- Replacement with new pipe
- Others

For a complete coverage on various cases, practices in California, Japan and China will be surveyed. Those methods for permanent repair/restoration will be analyzed, and compared on the basis of their seismic performance, availability at the site, applicability, durability, simplicity of operation and cost.

REHABILITATION STRATEGY

As discussed earlier, the replacement of all existing lines is very expensive, if not impossible. Other important facts are that 1) so many uncertainties, such as exact soil properties, seismic intensities, etc. are involved in a retrofitting project and 2) no effective retrofitting method has been yet developed. Therefore, the replacement of all existing buried pipelines is not recommended by most experts in the lifeline earthquake engineering field.

Old pipelines made-up of cast iron, clay, asbestos or plain concrete material with cement- or lead-caulk joints are very brittle and corrosive. They are most vulnerable to seismic hazards. Criteria or strategies to retrofit or replace them must be developed.

The low-cost retrofitting strategy at the present time is to upgrade the existing system during routine maintenance or disaster repair works. Following items are to be considered.

- Replace current brittle pipes with more ductile material pipes
- Replace current rigid joints with more flexible and/or restrained joints
- Replace current pipes that have been weakened by corrosion
- Repair cracks with strong epoxy
- Insertion of a new pipe inside the damaged one

For an important project such as a single large pipeline at a critical fault zone, retrofitting, repair and/or replacement of pipelines may be necessary. Some suggested methods of strengthening the environment are as follows:

- Add drainage around existing pipes or add anchorage to pipeline
- Inject chemical (or epoxy) or cement into potential soil liquefaction region
- Drive piles at junction between pipeline and interconnected structures
- Densify soils surrounding the pipeline
- Install flexible joints/junctions at strategic locations
- Insetrion of new pipe inside the damaged one

It is noted that the above suggested rehabilitation methods for seismic resistance of buried pipeline structures are by no means completed. Further survey from the U.S., Japanese and Chinese practices would be necessary.

In this task, it is necessary to perform technical and economic analyses on various rehabilitation methods in order to determine the cost-effectiveness of a particular method. The in-depth study on the cost-benefit of new materials including composite materials and new flexible joints should be conducted. This project also studies the necessity and cost-benefit to strengthen the surrounding environment.

TESTING AND EVALUATION PROGRAM AT EBAA

Joint resistance characteristic plays a major role in the seismic resistance of buried pipelines. To mitigate the earthquake damage to buried pipelines, flexibility rather than stiffness of the pipeline system is most important. Pipelines need to be designed to move with the ground rather than to resist the ground movement from the earthquake. Flexible expansion pipe joint system can be effectively used to reduce the risk of pipe failure due to sudden ground movement during earthquakes as can be seen in Figures 5 to 6.

Currently, conventional joints and seals have not been thoroughly studied for performance during an earthquake. A further study of empirical data and analysis results would improve the characterization of stresses and deformations that can occur as a result of ground shaking, ground distortion at joints, bends, tees, and fittings.

The project will conduct static and dynamic testings of several commonly used joints/bends/junctions to determine their resistance (strength) and flexibility (expansion/contraction or rotation capability) characteristics, which are crucial to seismic performance of buried pipelines.

The project will perform at least but not limited to the following tests and evaluations:

- Evaluation of the performance of existing pipe joint systems with reference to pull-out (axial expansion and contraction) capabilities.
- Evaluation of the performance of existing pipe joint systems with reference to angular deflection and rotation.
- Evaluation of the performance of devices used to prevent pull-out.
- Categorization and cataloging of existing joint systems by type according to pull-out, restraint, and deflection performance.
- Development and recommendation of a classification system tying cataloged joint types to seismicity zones or known seismic hazard levels.
- Recommendations for areas of improved seismic design for pipe joints and fittings.
- Retrofit solutions for areas of existing lifeline seismic problems as pertains to pipe joints, valves, and fittings.

Similar to a Japanese work [7], a classification of joints for seismic resistance will be developed. The seismic resistant classification of joints/junctions extended from the Japanese work will consist of three criteria, namely flexibility (extension/contraction or rotational angle), strength (maximum axial force or bending movement resistant) and energy dissipation capabilities. EBAA Iron Co. program (an industrial participation/contribution) will provide the flexibility and strength while ODU program described below provide energy dissipation information of some commonly used joints/junctions in the United States.

To create a statistical data base, each test will be repeated at least three times. Three different commonly used diameters (i.e. 6", 12" and 18") of pipe joints will be tested in order to establish the performance correlation with respect to the size of the pipe.

TESTING PROGRAM AT ODU

This task is the extension of EBAA's task to evaluate and test the same pipe joints by cyclic and dynamic loads. The main objective of this part is to compare the static and dynamic resistant characteristics of these pipe joints and to study their energy absorption capabilities. The investigators have conducted a valuable investigation [8] on "Energy Dissipation and Resistant Characteristics of a Flexible Joint" (Figure 7). The tests have been performed at Old Dominion University using its MTS Dynamic Testing Machine. Dynamic stiffness and equivalent damping have been obtained by the semi-empirical means.

For earthquake engineering applications, the hysteretic characteristics of pipe joints will be tested using the MTS Dynamic Testing System at ODU in order to determine the energy dissipation capabilities of these joints. Low frequency cyclic loads varying from 0 Hz (static load) to 3 Hz at and increment of 0.5 Hz will be applied. An electronic data acquisition system will be used to record the load-displacement (load cell - LVDT) curves (hysteresis curves). The area under the hysteresis, which can be automatically calculated by the data acquisition system, will be established as a measure of energy dissipation capacity of the joint.

Experiences from earthquakes in the United States, Japan, and China suggest that new types of flexible-restrained joints would help improve the seismic performance of segmented and jointed pipelines by allowing larger displacement and absorption of more energy during seismic shaking, liquefaction, and fault movement.

Upon completion of the testing programs at EBAA and ODU, joint classification in terms of seismic resistance will be developed for repair and rehabilitation purposes. The guidelines to select proper joint/junction will depend on the seismic environment (shaking, liquefactions, fault movements, etc.), site condition (soil, geology, etc.) and pipe material used.

SYNTHESIS AND GUIDELINE DEVELOPMENT

Upon the completion of previous tasks effective repair methodologies and effective retrofitting strategies will be developed. There is no unique solution because of the variation of conditions about pipes, joints, surrounding environments, seismicities etc. Nevertheless guidelines for effective repairs and rehabilitation strategies should be provided to utilities in order to reduce economical losses due to disastrous earthquakes.

Development of Repair Guidelines

The project is to cover nearly all the methods of repair for buried water and sewer pipelines along with their economical, and technical information on these methods. Specifically the technical information about the seismic performance of joints, bends, junctions and other items related to repair and/or retrofit of buried water/sewer pipelines will be analyzed and synthesized.

The comparison would be based on several indicators, namely: a) seismic performance, b) cost of installation, c) time required to complete the repair, d) labor/man-power needed, e) durability, f) availability, g) expected life, h) environmental impact, and i) other factors.

It is expected that upon the completion of the task, clear and concise guidelines for selecting repair methods under various conditions will be established and simple but cost-effective repair method(s) will be recommended for future practice.

Development of Rehabilitation Strategies

Effective permanent repairs can be considered as acceptable rehabilitation means. However, rehabilitation of buried lifelines needs more than just repairs of damage. To develop effective strategies for a rehabilitation program, one needs to know the whole field of lifeline earthquake engineering, the damage history and economical loss due to the disastrous earthquakes, the limitations of the repair/retrofit methods, the damage survey and inspection techniques, the current operation and monitoring system etc.

One simple example is that old cast-iron pipelines built before 1930 may be too corroded or with insufficient service capacity to be considered for retrofitting. It may be more cost-effective to plan a new line instead of retrofitting the old network. Some other measures must be prepared before the new line is completed. Another example is that a relatively sound transmission line located in a region previously thought with low seismicity may be required to be retrofitted because of the consequence of potential disasters.

Listed previously are several low-cost retrofitting plans by replacing older brittle pipes with newer ductile pipes, older brittle joints with newer flexible joints etc. during normal maintenance routines. However, locations have higher seismic risks must not wait too long to retrofit the entire system because a disastrous earthquake may come at anytime and lifeline losses and consequence could be substantial. Effective retrofitting strategies must be developed and actions must begin right away. The development of effective rehabilitation program will include at least the following schemes:

a) Scheme for inspection and identification of weak components

b) Scheme for evaluation of the effects of the damage to system performance

c) Scheme for economic evaluation of the cost of upgrading/replacement of weak components or adding parallel, redundant pipelines, where applicable

d) Scheme for setting priorities for upgrading/replacement of weak components

This task of the project develops the procedures and guidelines on each of the schemes indicated above. The effective strategy would be to combine the normal maintenance along with committed rehabilitation funds to upgrade the entire system in shortest possible time. In this task, the guidelines to determine the period of time to complete the rehabilitation program will be established.

The effective repair methods will be used for rehabilitation purposes. Unless pipe material is deteriorated, upgrading of the system at strategic locations may only be necessary.

SUMMARY AND CONCLUSIONS

This paper describes an on-going research project on "Repair and Rehabilitation of Buried Water and Sewer Lifelines" at Old Dominion University, supported by the National Science Foundation supplemented by an industrial firm, EBAA Iron Inc. of Eastland, Texas has contributed to the project with their internal funds to conduct the static testing program on commonly used and newly developed pipe joints at their facilities and in-kind services to carry out other tasks of the project.

The project consists of three major tasks. The first task is conduct surveys and assessments of pipeline damage behavior, inspection/detection techniques, emergency repair/restoration measures and permanent/complete repair/restoration methods from practices in U.S., Japan and China. The aim of this task is primarily for repair/restoration of damaged pipelines.

The second major effort will concentrate on the testing of commonly used and newly developed pipe joints. The testing will include both static and dynamic behavior with reference to pull-out and angular/rotation capabilities, energy dissipation and resistant/restraint characteristics. It is to develop a classification system tying cataloged joint types to seismic zones. The aim of this task would be to search for proper pipe joints/fittings that will be suitable for retrofitting/strengthening existing vulnerable/deteriorated pipelines, repairing/restoring damaged pipelines and designing/constructing new pipelines against earthquake hazards.

The third major task would be to develop repair guidelines and rehabilitation strategy. Synthesis and analysis pipeline damage behavior, current repair practices, strength and flexibility of commonly used and newly developed pipe joints fitting along with other technical and economic considerations will be performed. Ranking of various method against various seismic condition (seismic shaking, large faulty displacement, liquefaction etc) will be established.

ACKNOWLEDGEMENTS

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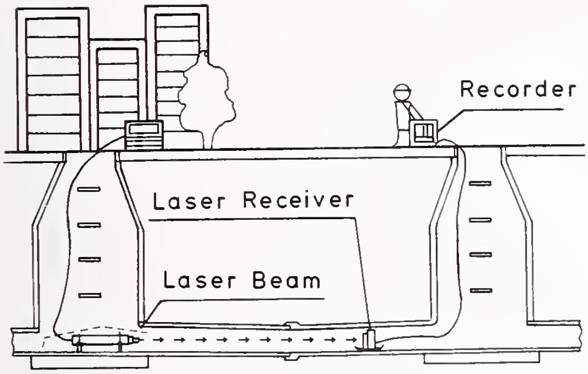

Fig.1 Checking Misalignment of Pipes by Laser Beam

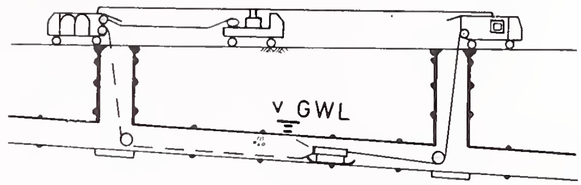

Fig.2 Inspection of Pipeline Damage by Remote TV Camera

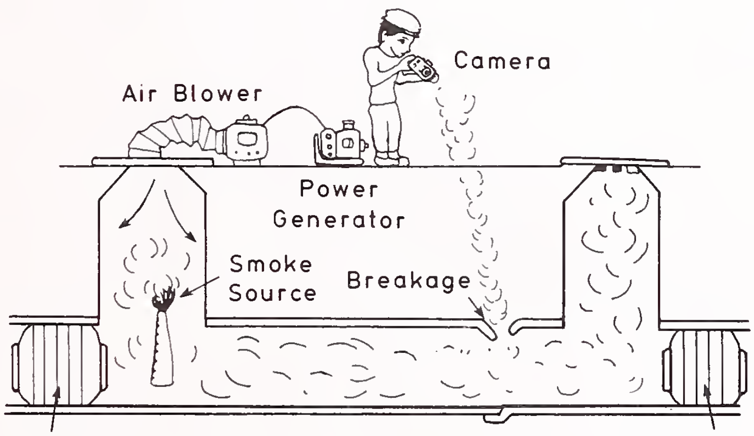

Fig.3 Smoke Test for Breakage of Pipe

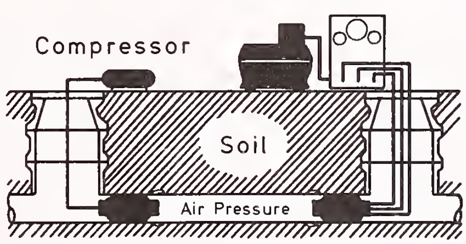

Fig.4 Leakage Test for Pipe and Joint by Air Pressure

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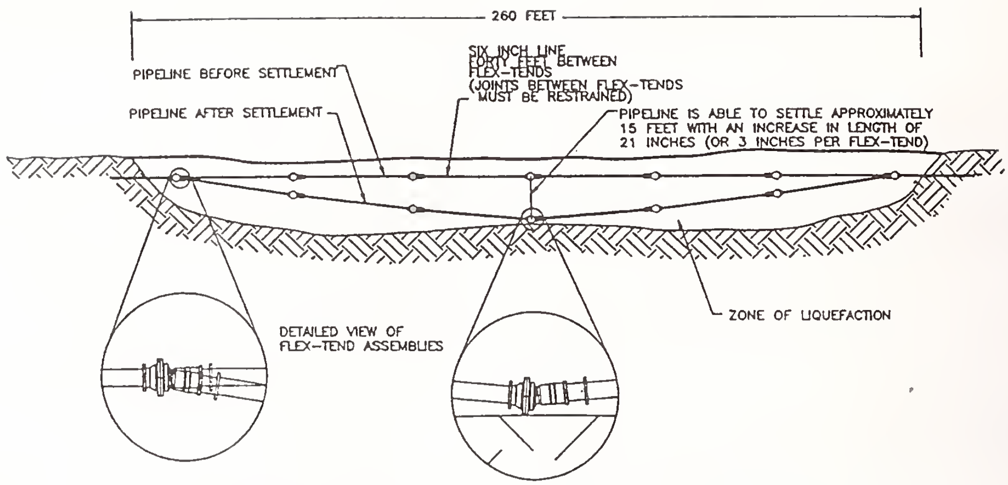

Fig.5 Pipeline with Flexible Joints in a Liquefaction Zone

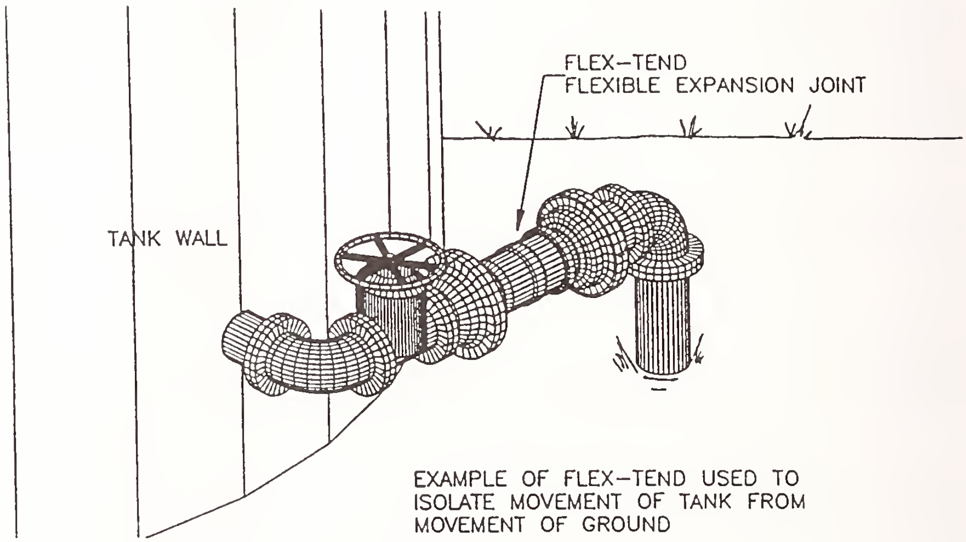

Fig.6 Flexible Joint for Pipe-Structure Connection

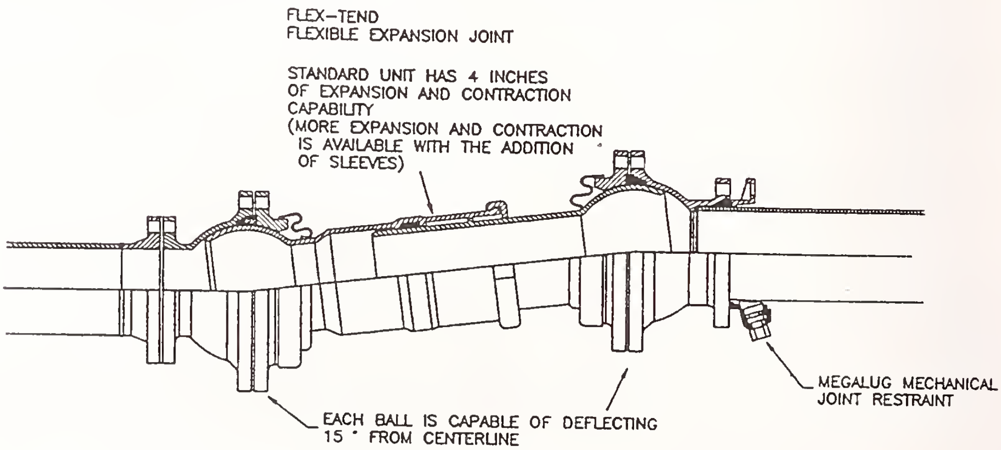

Fig.7 A Flexible Joint Cross Section

INVESTIGATIONS ON EXTERNAL FORCE EVALUATION IN SEISMIC DEFORMATION METHOD

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ABSTRACT

Investigations were made on the methods for evaluation of the external forces acting along the transverse direction of the structures in the seismic deformation method. Assuming rectangular cross-sections for the structures and uniform ground properties, two-dimensional FEM dynamic analysis was first implemented using the rigidity, mass and depth of the structures as parameters to ascertain the characteristics of external forces acting on the structures. Investigations were then made on the appropriate values for the external forces along the transverse direction for use in the seismic deformation method using the results of the two-dimensional FEM dynamic analysis as the norm. The results of these investigations may be summarized as follows.

1)The bending moment generated by the dynamic shear force acting on the external faces of the structure accounts for over 50% of the total bending moment.

2)The inertia force of the structure has little effect on the generated bending moment .

3)The calculation results in the seismic deformation method will agree fairly closely with the dynamic FEM analysis results, so long as the dynamic earth pressure acting on the side walls of the structure and the dynamic shear forces on the external faces of the structure are taken into account and the ground spring constants were evaluated appropriately.

INTRODUCTION

Investigations on the longitudinal forces on structures occupy the central role in seismic design of underground structures such as the lifelines and detailed investigations are not usually made on transverse forces. The increasing depths and cross-sections of underground structures in recent years accompanying the increasing intensity of land use, however, have given rise to a need to review the conventional methods for seismic design.

The investigations reported here were made with the purpose of establishing a method for evaluation of the external forces that should be made to act in the transverse direction in the seismic deformation method, the method most widely used in the seismic design of underground structures. The procedure adopted in the investigations was as follows.

1)The structures were assumed to have rectangular cross-sections and the ground properties assumed to be uniform. Two-dimensional FEM dynamic analysis was implemented using the rigidity, mass and depth of the structures as parameters for obtaining the vibration characteristics of the structures and the characteristics of the external forces acting on the structures.

2)Responses of the structures were calculated for various structural models and external forces acting on them by the seismic deformation method. The ground spring constant for the seismic deformation method is calculated from the relationship between the static external forces in the two-dimensional FEM and the ground displacement.

3)Comparison was made between the calculation results in the seismic deformation method and the FEM analysis results and considerations made on the appropriate external forces that should be made to act on the structures in the seismic deformation method.

CONDITIONS OF INVESTIGATIONS

(1) Analysis Conditions for FEM Dynamic Analysis
Large dynamic earth pressure will act on the sides of the structure , if its rigidity and mass are significantly different from those of the ground. If the deformation characteristics and mass of the structure are the same as those of the surrounding ground, the structure will show the same behaviour as the ground and the dynamic earth pressure acting on the walls of the structure will be minimal. In view of this, the analysis here was carried out by first establishing the two standard models of the "equivalent rigidity structure" and "equivalent mass structure." As shown in Fig.1, the equivalent rigidity structure is defined as that in which the displacement δ_s of the top of the structure when a load whose value per unit width is P is applied to the top of a rigid frame (height: H, width: L) simply supported at its base, equals the displacement δ_g under a similar load at the top of the ground element with the same volume as that of the soil removed to accommodate it. The equivalent mass structure, on the other hand, is defined as the structure with the same mass as that of the soil removed to accommodate it. The equivalent rigidity and mass are termed "G" and "M" in this report. The analysis model, physical properties of the ground and cross-sectional performance of the structure, and analysis cases are shown in Fig. 2, Table 1 and Table 2, respectively.

In the analysis model, the ground as uniform elastic ground with a rigid base was represented by plane strain elements, while the structure was represented by beam elements with 15 m square cross-sections. For the boundary conditions, the base of the model was made fixed, while the sides were supported by horizontal rollers. The analysis was conducted on one half of the symmetrical model. The standard model in the analysis cases had equivalent rigidity and mass and a depth of 15 m. The input waves

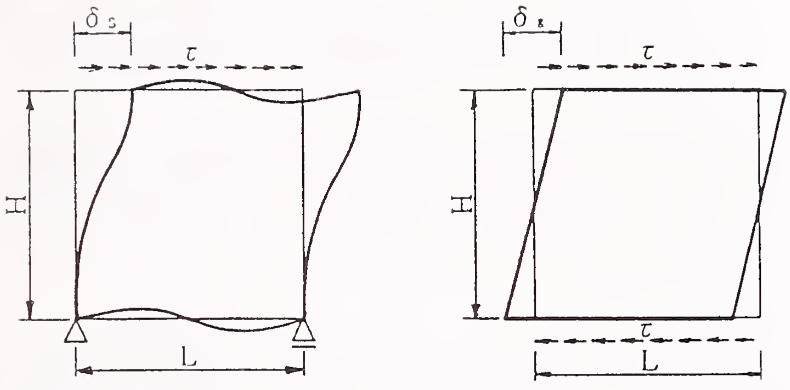


Fig. 1 Calculation Method for Equivalent Rigidity

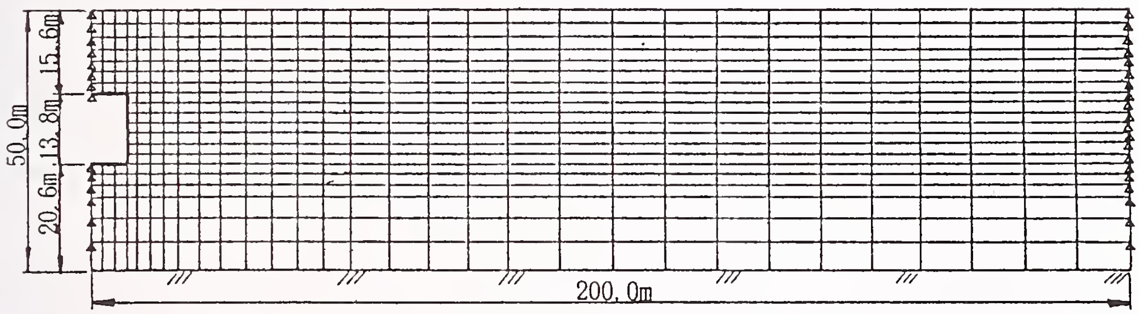


Fig. 2 Finite Element Idealization of Cross-Section for Numerical Analysis

Table 1 Material Properties (Ground and Structure)

GROUND	Shear wave velocity	V_s (m/sec)	200.0
	Shear modulus	G (tf/m²)	6530.0
	Poisson's ratio	ν	0.45
	Dumping constant	h	0.05
	Unit weight	γ (tf/m³)	1.60
STRUC-TURE	Sectional area	A (m²)	1.20
	Young's modulus	E^{*1} (tf/m²)	9.313×10^6
	Moment of inertia of a cross-section	I (m⁴)	0.144
	Damping constant	h	0.05
	Unit weight	γ^{*2} (tf/m³)	4.60

Note. *1 ; equivalent rigidity (G) , *2 ; equivalent mass (M)

Table 2 Parameters of Numerical Models

Rigidity	0.01*G	0.1*G	1.*G	10.*G	100.*G
Mass	0.0001*M	0.1*M	1.*M	2.*M	3.*M
Depth	0.0 m	-15.0 m			

Note. G; equivalent rigidity , M; equivalent mass were 20 sinusoidal waves (maximum acceleration amplitude: 100 gal) with a primary natural frequency (eigen value analysis result: 1 Hz) which is the prevalent mode in the ground.

(2) Calculation Conditions for Seismic Deformation Method
As shown in Fig. 3, the ground structure system was replaced by a plane skeleton structural model, with the structure being represented by beam elements and the ground by springs. The analysis cases are shown in Table 3.

Although the results obtained by the seismic deformation method are affected by the ground spring constants, there is at present no standard method for evaluation of the ground spring. In the investigations reported here, a uniformly distributed load was applied to the width which is equivalent to the width of each sides of the structure, using a FEM model of the ground system and the ground spring constant was calculated from the relationship between the load and displacement as shown in Fig. 4. The distribution of the normal spring along the side walls of the structure, and the shear spring along the walls is shown in Fig. 5. Average values for each face, however, were used in the calculations here because of the possibility of effects from the accuracy of element divisions and corners. The ground spring values used are listed in Table 4.

The external forces acting on the structure and the methods for calculating them were as follows.

(a) External Force due to Response Displacement of Free Field
As shown in Fig. 6(a), this external force is obtained by multiplying the displacement of the free field applied to the structure by the ground spring (normal and shear spring). The displacement here is the relative displacement of the free field in relation to the base of the structure .

(b) Dynamic Shear Force around Structure
As shown in Fig. 6(b), this seismic load is obtained by multiplying the dynamic shear stress of the free field at the depths at which the members of the structure are located by the unit surface area of each member .

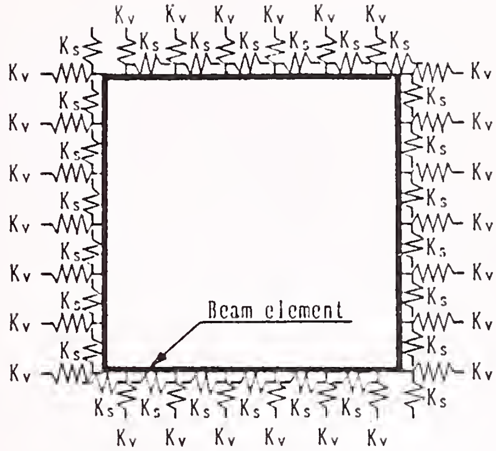
(c) Inertia Force of Structure
As shown in Fig. 6(c), this seismic load is obtained by multiplying the mass of each member of the structure by the horizontal acceleration of the free field at the depths at which each member is located .

The values for the external forces calculated according to the procedures described above are given in Table 5.

ANALYSIS RESULTS

(1) FEM Dynamic Analysis Results

(a)Variation in Rigidity of Structure(Equivalent Mass Structure)

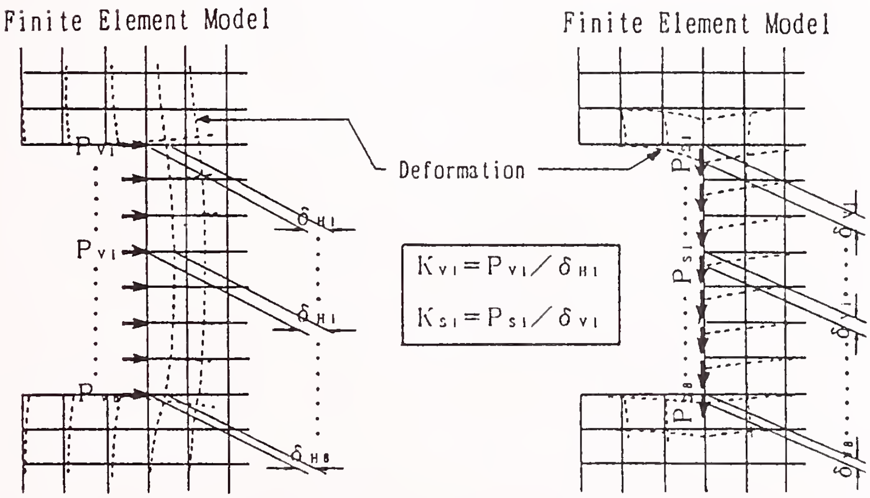


Kv ; Normal Spring Constant
Ks ; Shear Spring Constant

Fig. 3 Calculation Model (Seismic Deformation Method)

Table 3 Calculation Cases in Seismic Deformation Method

Rigidity	0.1*G	1. *G	10. *G
Mass	1. *M		
Depth	0.0 m	-15.0 m	



K_{VI} ; Normal Spring Constant
K_{SI} ; Shear Spring Constant
δ_{HI} ; Displacement in Horizontal Direction
δ_{VI} ; Displacement in Vertical Direction
P_{VI} ; Load in Horizontal Direction
P_{SI} ; Load in Vertical Direction

Fig. 4 Calculation Method for Ground Spring Constant

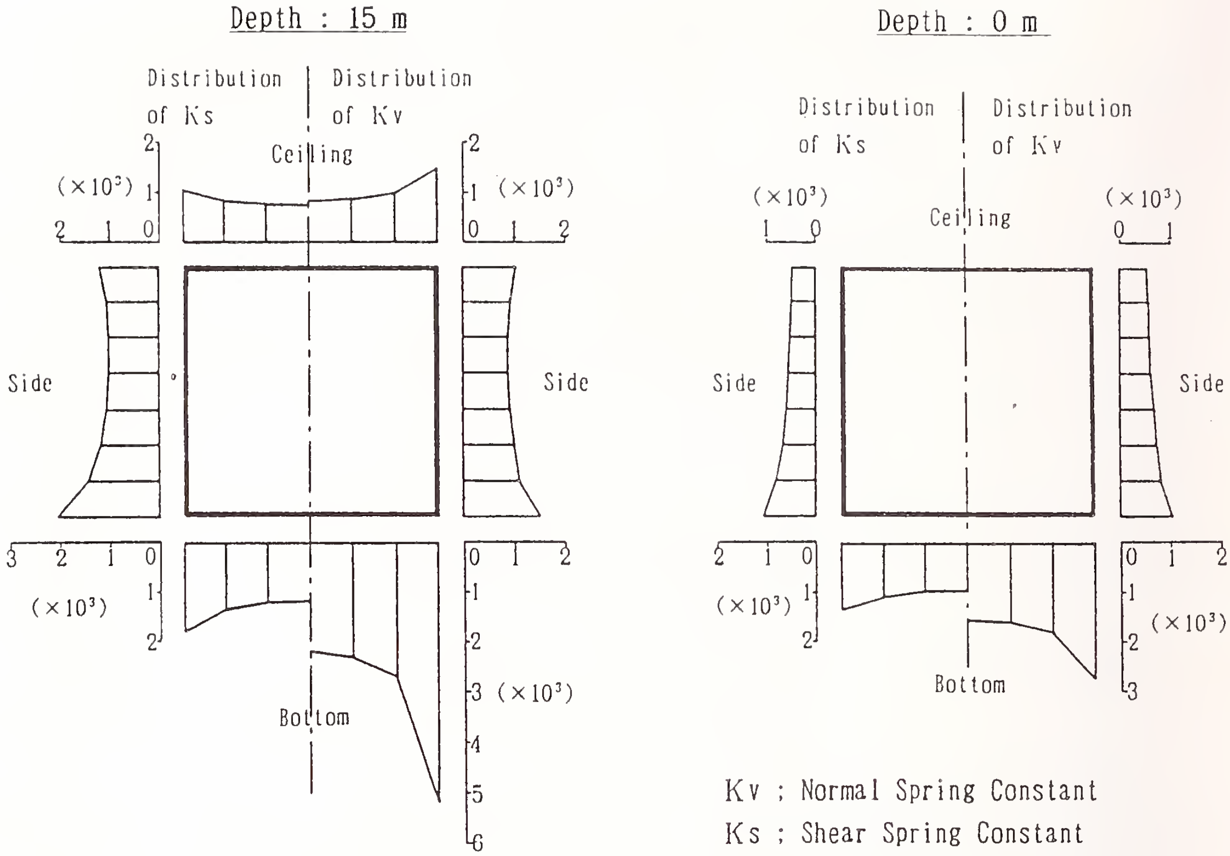
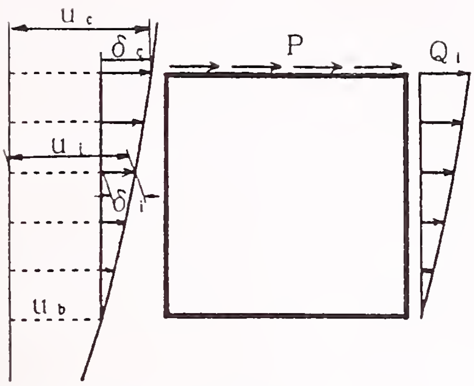


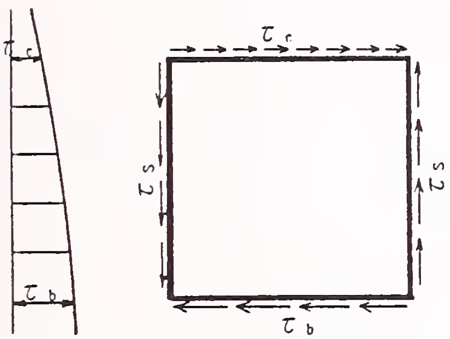
Fig. 5 Ground Spring Distribution (tf/m³)

Table 4 Coefficient of Ground Spring (tf/m³)

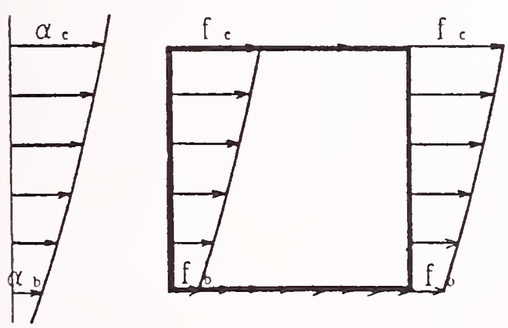
	Depth	Bottom	Side	Ceiling
Normal spring constant	0 m	1853	664	——
	15 m	2897	994	1009
Shear spring constant	0 m	1090	641	——
	15 m	1363	1224	861



(a) External Force due to Response Displacement of Free Field



(b) Dynamic Shear Force around Structure



(c) Inertia Force of Structure

$P = K_s \times \delta_c$

$Q_i = K_v \times \delta_i$

K_v ; Normal Spring Constant

K_s ; Shear Spring Constant

$\delta_i = u_i - u_b$

u_i ; Response Displacement of Free Field in Horizontal Direction

u_b ; Response Displacement of Free Field in Horizontal Direction on Bottom of Structure

δ_c ; Relative Displacement on Ceiling of Structure

τ_c ; Dynamic Shear Stress of Free Field on Ceiling

τ_b ; Dynamic Shear Stress of Free Field on Bottom

τ_s ; Averaged τ_c and τ_b

$\tau_s = (\tau_c + \tau_b) / 2$

f_i ; Inertia Force of Structure

$f_i = m_i \times \alpha_i$

α_i ; Response Acceleration of Ground

m_i ; Mass of Structures Distribution to each Nodal Point

α_c ; Response Acceleration of Grounds of Free Field on Ceiling

α_b ; Response Acceleration of Grounds of Free Field on Bottom

f_c ; Inertia Force of Structure on Ceiling

f_b ; Inertia Force of Structure on Bottom

Fig. 6 External Force in Seismic Deformation Method

Table 5 External Forces in Seismic Deformation Method

		0 m	15 m
External Force due to Response Displacement of Free Field	Side	180	656
	Ceiling	—	1057
Dynamic Shear Force around Structure	Bottom	383	742
	Side	190	582
	Ceiling	—	427
Inertia Force of Structure		378	297

The deformation of the structure at a depth of 15 m is shown in Fig. 7. Shown in Fig. 8 are the vertical response displacement distribution along the side walls of the structure, distribution of the relative displacement in relation to the free field and distribution of the lateral dynamic earth pressure (normal stress of the ground element in contact with the right-hand wall of structure).

It can be seen from Fig. 7 that the structure shows a similar behaviour to the ground. While the structure shows rotational displacement as a rigid body when the rigidity of the structure exceeds that of equivalent rigidity, shear and bending deformation prevails in the behaviour when it is smaller than the equivalent rigidity. Fig. 8 shows that the direction in which the dynamic earth pressure acts on the side walls of the structure is reversed at a certain point as the depth increases and at the equivalent rigidity as the rigidity increases.

The above indicates that the dynamic earth pressure acting on side walls of the structure is closely associated with the relative displacement between the structure and the free field and, when the rigidity of the structure is greater than the equivalent rigidity, this tends to result in increased displacement of the structure, while rigidity smaller than the equivalent rigidity will tend to reduce the deformation of the structure. This may be interpreted as meaning that the kinds of external forces that would encourage the structure to follow the displacement of the free field are found acting on the structure.

(b) Variation in Mass of Structure(Equivalent Rigidity Structure)
Shown in Fig. 9 are the vertical response displacement distribution along the side walls of a 15 m deep structure, distribution of the relative displacement in relation to the free field and distribution of the lateral dynamic earth pressure (normal stress of the ground element in contact with the right-hand wall of structure). Unlike in (a) above, there is no vertical variation here in the deformation amplitude characteristics and the variation in the parallel movement only is shown.

(c) Variation in Depth of Structure
The values corresponding to those shown in Fig. 8 and 9 for when the depth of the structure was altered to 0 m are given in Fig. 10 and 11. The response characteristics are basically the same at 0 m as at 15 m. The greater response acceleration of the structure at 0 m, however, means that the variation in the response with the variation in the mass of the structure cannot be neglected.

(d) Dynamic Shear Force on External Faces of Structure
The distribution of the dynamic shear stress acting on the external faces of the structure when the 15 m deep equivalent mass structure was given varying rigidities is shown in Fig. 12, and the dynamic shear forces discretely integrated along the top, side and bottom faces of the structure are shown in Fig. 13 according to parameters concerned.

The dynamic shear force acting on the external faces of the structure increases with the rigidity. This is for the same

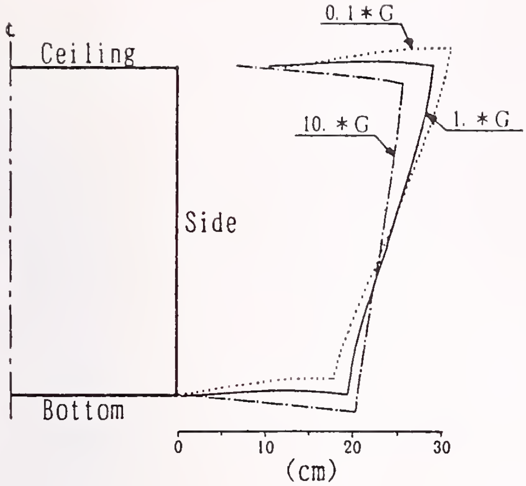


Fig. 7 Transition of Deformation Pattern with Change in Rigidity of Structure (Depth : 15 m)

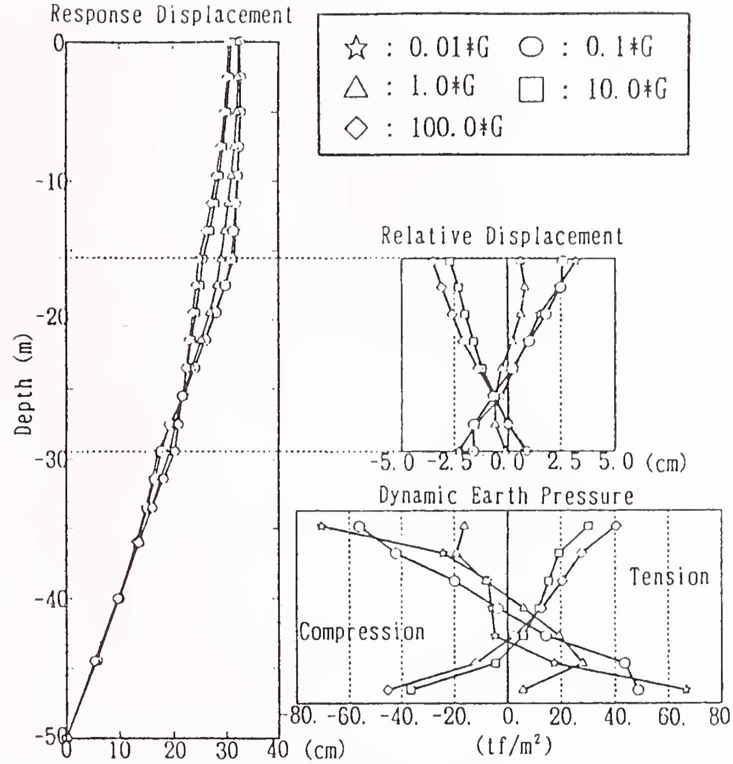


Fig. 8 Transition of Distribution Patterns of Response Displacement and Dynamic Earth Pressure Acting on Side Walls of Structure with Change in Rigidity of structure (Depth : 15 m)

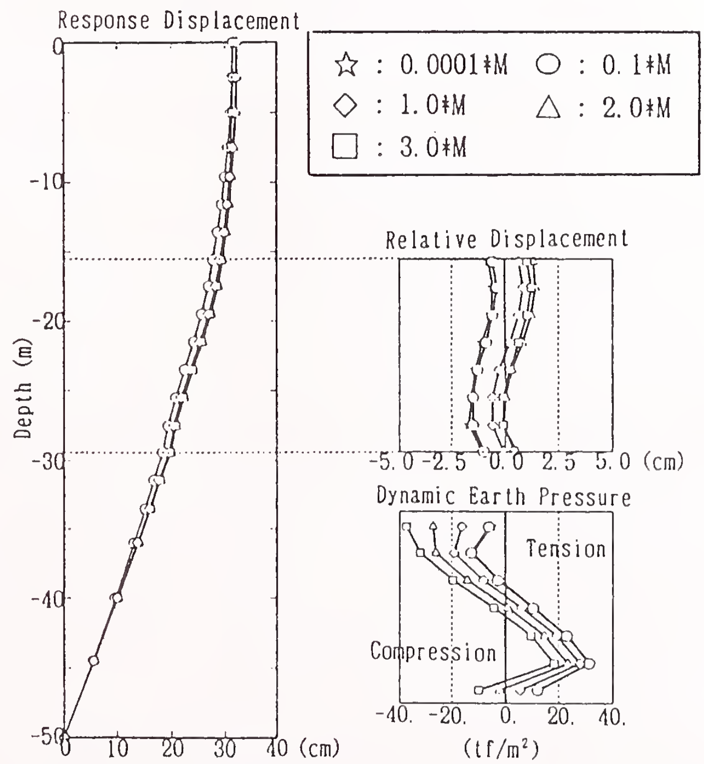


Fig. 9 Transition of Distribution Patterns of Response Displacement and Dynamic Earth Pressure Acting on Side Walls of Structure with Change in Bulk Specific Gravity of Structure (Depth : 15 m)

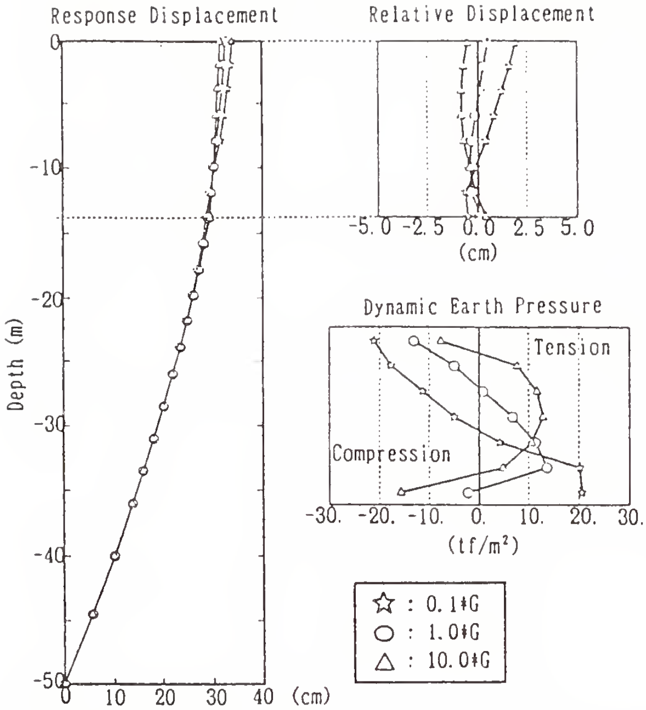


Fig. 10 Transition of Distribution Patterns of Response Displacement and Dynamic Earth Pressure Acting on Side Walls of Structure with Change in Rigidity of structure (Depth : 0 m)

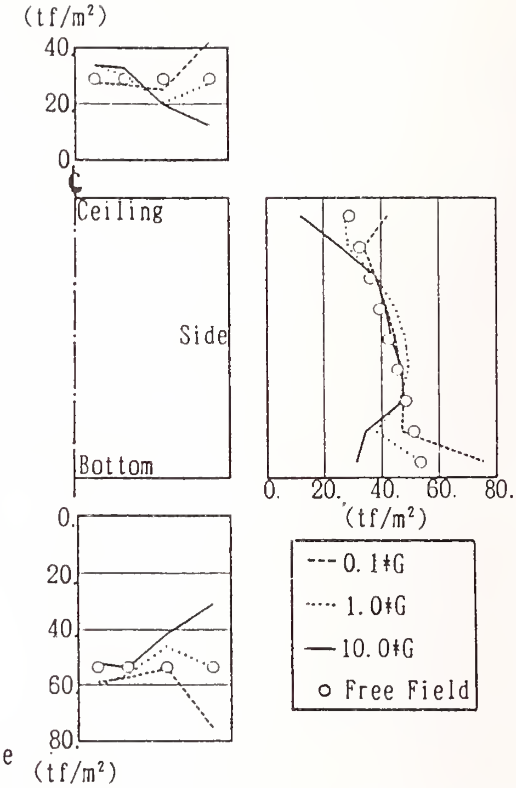


Fig. 12 Difference in Distribution Patterns of Dynamic Shear Stress around Structure according to Changes in Rigidity of Structure

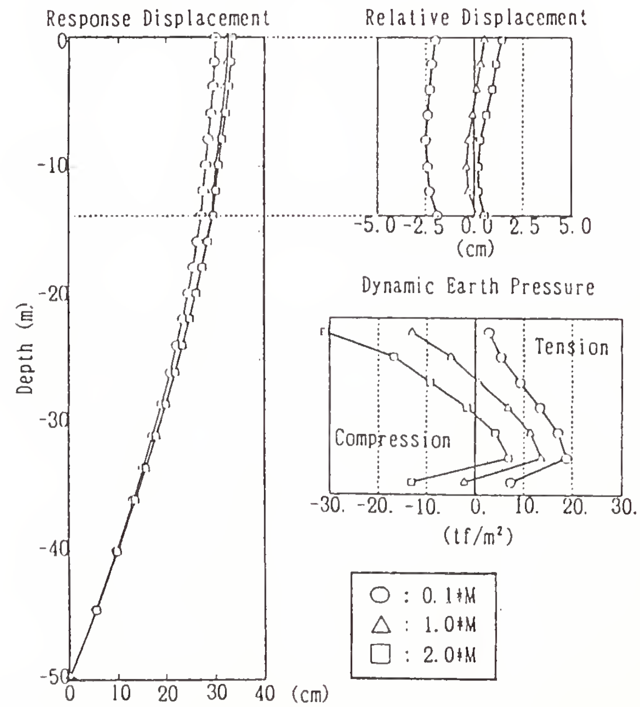


Fig. 11 Transition of Distribution Patterns of Response Displacement and Dynamic Earth Pressure Acting on Side Walls of Structure with Change in Bulk Specific Gravity of Structure (Depth : 0 m)

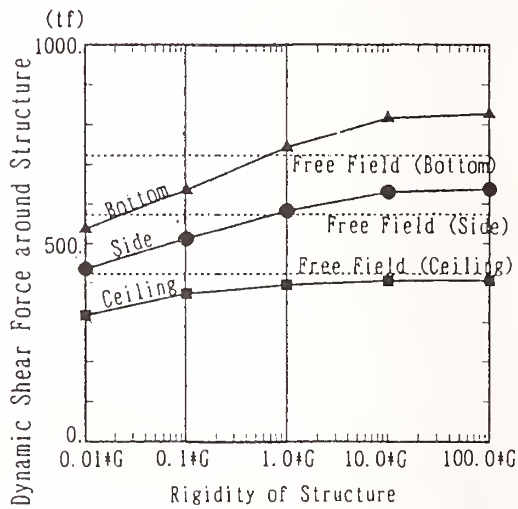


Fig. 13 Relationship between Rigidity of Structure and Dynamic Shear Force around Structure

reason as why the dynamic earth pressure acting on the side walls varies with the rigidity. As the rigidity increases, the deformation of the structure becomes smaller than that of the surrounding ground and the external forces will move in the direction of encouraging deformation in the structure.

(e) Bending Moment Generated in Structure
The bending moments generated on the top and side faces of the structure are shown in Fig. 14. As with the behavioural characteristics of the structure, the variation is greater with the variation in rigidity than with that in mass. The variation due to variation in mass, however, is greater at smaller depths.

(2) Comparison of Seismic Deformation Method Calculation Results and FEM Analysis Results

(a) Deformation of Structure
Comparison is made in Fig. 15 between the relative displacement of the side walls of the structure and the free field as obtained by the seismic deformation method and FEM dynamic analysis. When the rigidity of the structure is smaller than the equivalent rigidity, shear deformation and bending deformation predominates in the structure, while with greater rigidity than the equivalent rigidity, the structure will show a rotational behaviour. These response characteristics are the same as those obtained by FEM dynamic analysis and Fig. 15 also indicates the close agreement between the seismic deformation method calculation results and the FEM dynamic analysis results.

(b) Bending Moment Generated in Structure
Comparison is made in Fig. 16 between the bending moment on the top and side faces of the structure as obtained by the seismic deformation method and FEM dynamic analysis. The ratios of the bending moments under each external force obtained by seismic deformation method calculations to those obtained by FEM dynamic analysis are shown in Fig. 17. The following conclusions may be drawn from these fig..

1)Although the seismic deformation method calculations depend on the evaluation of the ground spring constants, the results obtained are generally close to those obtained by FEM dynamic analysis, the errors being within around 10%.

2)The bending moment due to dynamic shear forces acting on the external faces of the structure accounts for as much as 50% of the total bending moment, indicating the necessity of taking the dynamic shear force acting on the external faces in seismic deformation method calculations.

3)While the effect of the inertia force of the structure on the overall bending moment is generally small, it may account for as much as 20% in shallow structures and care needs to be taken over this point in calculations for structures at small depths.

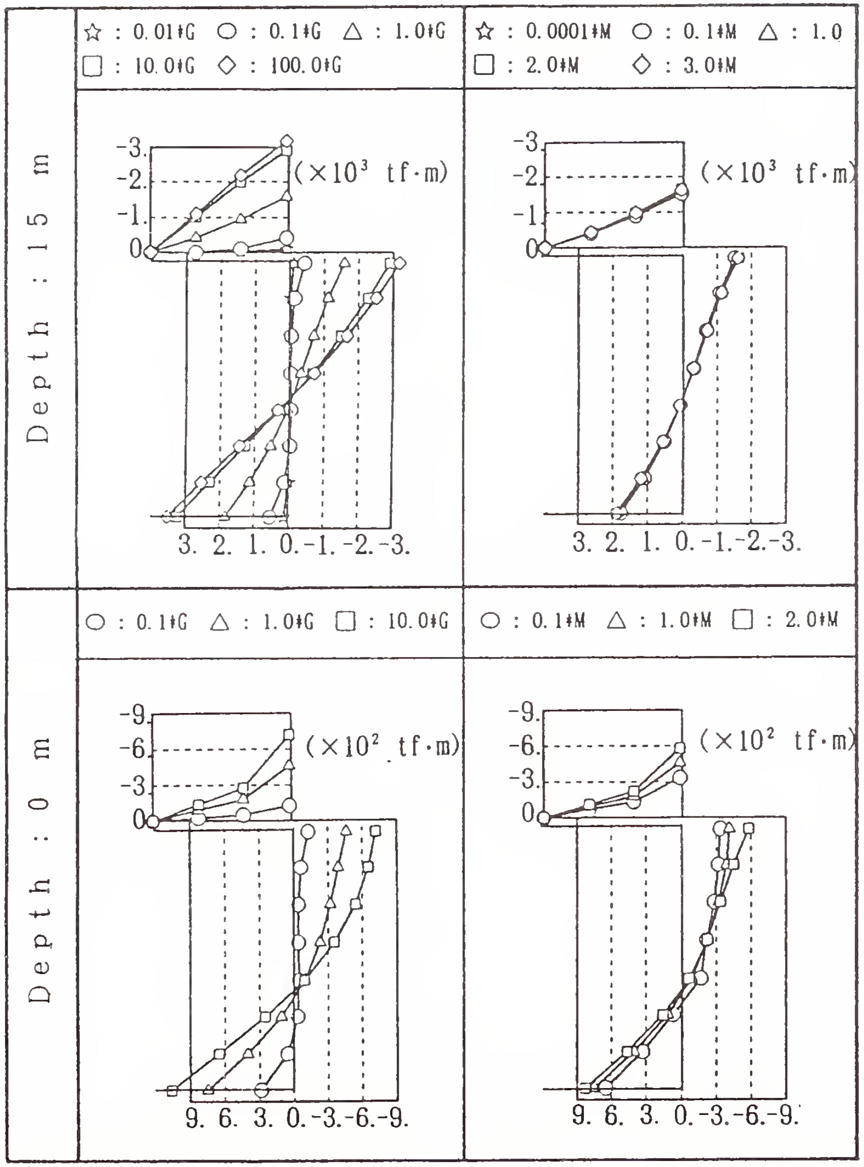


Fig. 14 Comparison of Distribution Pattern of Bending Moment on Ceiling and Side of Structure with FEM Results

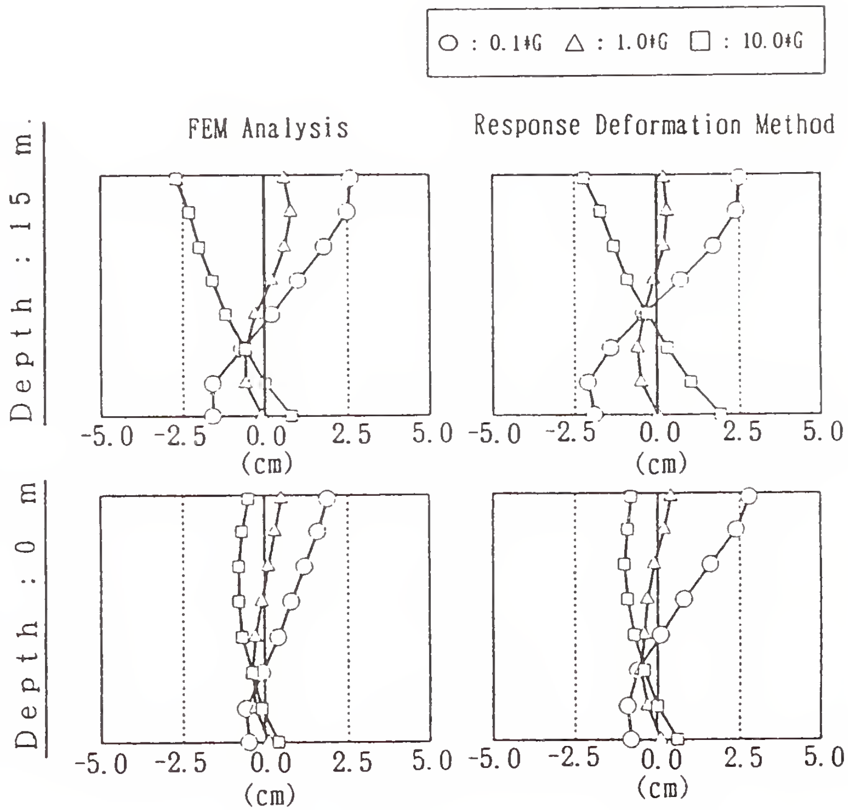


Fig. 15 Comparison of Distribution Pattern of Response Displacement and Relative Displacement along Side Walls of Structure with FEM Results

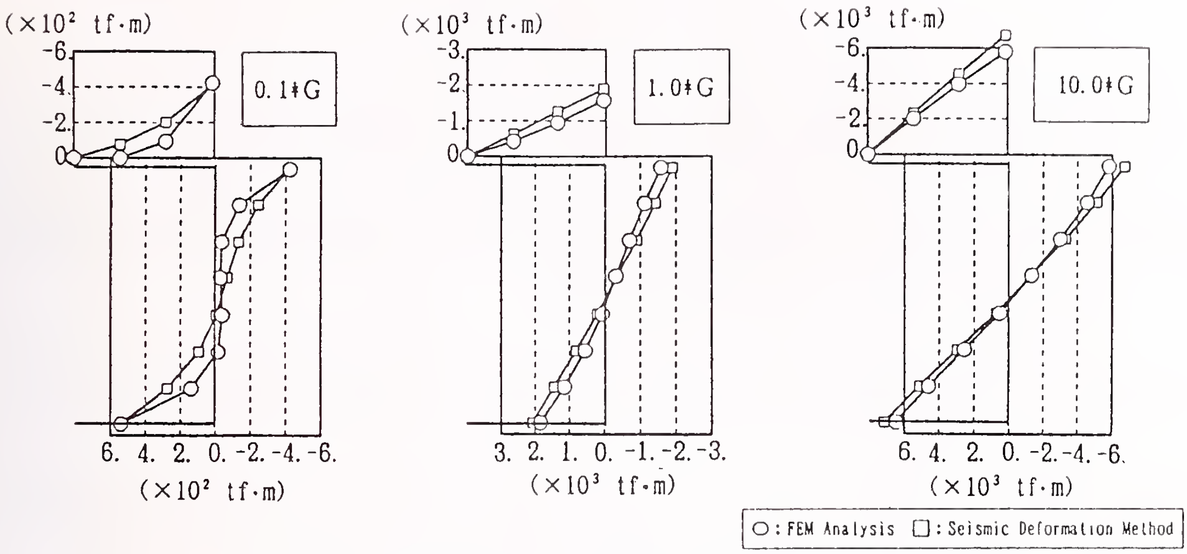


Fig. 16 Comparison of Distribution Pattern of Bending Moment with FEM Results

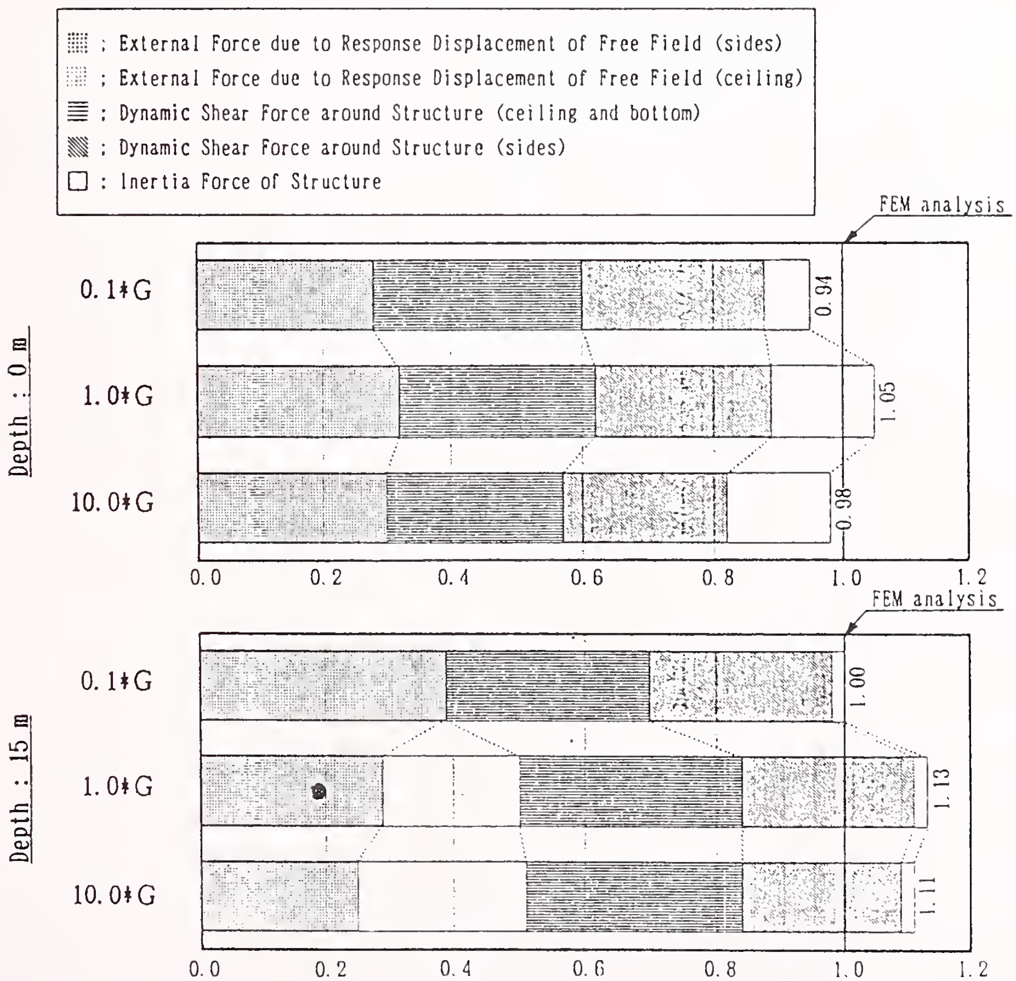


Fig. 17 Proportion of Each External Force in Total Bending Moment in Structure (Bottom Right Corner of Structure)

CONCLUSION

Two-dimensional FEM dynamic analysis was carried out using the rigidity, mass and depth of underground structures as parameters, for the purpose of investigating the basic transverse dynamic response characteristics and the results from this were used in making considerations on the validity of external force evaluation in the seismic deformation method. The following conclusions were drawn from these investigations.

1)The external forces acting on structures will vary with the rigidity of the structure in the direction that would encourage the structure to follow the behaviour of the free field.

2)While the variation in rigidity has a great influence on the response displacement and bending moment generated in the structure, the effect of the variation in mass is relatively small.

3)In seismic deformation method calculations, the bending moment due to the dynamic shear forces acting on the external faces of the structure accounts for over 50% of the total bending moment.

4)The results of seismic deformation method calculations will agree fairly closely with the FEM analysis results so long as the dynamic earth pressure acting on the side walls of the structure and the dynamic shear forces on the external faces of the structure are taken in to account.

POSTSCRIPT

The investigations reported here concerned with structures with square cross-sections in uniform ground. Investigations are being conducted at present on the validity of external force evaluation proposed here when applied to structures with horizontal and vertical sides of differing lengths.

Principal tasks for the future include the following.
1)Investigations on methods of calculating the ground spring constants in the seismic deformation method.
2)Investigations on limits of application of seismic deformation method with change in ground conditions and structure dimensions.
The study reported here was conducted as a part of the Comprehensive Research Project "Cooperative Research in Development of Seismic Design of Underground Structures".[1)]

REFERENCE

1)Public Works Research Institute, Advanced Construction Technology Centre et al., "Development of Seismic Design of Underground Structures, Cooperative Research Report of PWRI", 1990

ESTIMATION OF SYSTEM RELIABILITY FOR LIFELINE STANDARDS

and

EXAMPLE USING THE CITY OF EVERETT, WASHINGTON'S LIFELINES

Donald Ballantyne, P.E., Kennedy/Jenks Consultants

ABSTRACT

A standard to evaluate lifeline system reliability should be an element of a lifeline system earthquake code. That element is included in a proposed standard being planned by the National Institute of Standards and Technology, NIST, for water and sewer systems (Ballantyne, 1991a).

Earthquake loss estimates for six types of lifeline systems serving the City of Everett, Washington, were made for three earthquake scenarios (Ballantyne, 1991b). Lifeline systems included water, sewer, electric power, natural gas, telephone, and highways/bridges. System component damage was estimated by earthquake lifeline experts. Post-earthquake system functionality was appraised. Lifeline system operators evaluated the time to restore the system to be operable. Secondary losses, resulting from business interruption due to lack of lifeline service, were estimated.

The Everett study provides an example of the lifeline system evaluation process using current methodologies and damage estimating data. This paper discusses conclusions about the system reliability evaluation process that can be drawn from the Everett study. It proposes recommendations that should be included in the proposed NIST standard, and for future lifeline research.

INTRODUCTION

A standard to estimate lifeline system reliability should be a key element of seismic code for lifeline systems. Lifeline system reliability is the key component differentiating lifeline system standards from building standards. This consideration is of current interest because the United States Congress has given NIST the charge of developing a plan for the development of recommended standards for lifeline systems (Dikkers, 1991). One of the considerations in developing those standards is reaching consensus on an approach to assess post-earthquake lifeline system reliability.

A proposed standards approach should include five elements (Ballantyne, 1991a):

1. Policy Statement
2. System Evaluation
3. Component Design/Evaluation
4. Equipment and Material Standards
5. Emergency Planning and Response.

The focus of this paper is on Element 2, System Evaluation.

"Earthquake Loss Estimation of the City of Everett, Washington's Lifelines," the Everett study, provides an example of system evaluation for six types of lifeline systems for three earthquake events (Ballantyne, 1991b). Lifeline systems included water, sewer, power, natural gas, telephone, and highways/bridges.

The methodology applied in the Everett study provides an example of what a standard evaluation procedure could entail. Conclusions and recommendations are presented from consideration of the Everett study in light of the standards evaluation procedure.

EARTHQUAKE LIFELINE WATER SYSTEM STANDARDS

Five elements of the standard would provide a rational step-by-step approach to water system design to reduce earthquake imparted disfunction. Element 1, Policy Statement, would define expected levels of system operability. Element 2, System Evaluation, would provide a procedure to assess earthquake performance and resulting post-earthquake system operability. Element 3, Component Design/Evaluation, and Element 4, Material and Equipment Standards, which already exist for many water system components, would define procedures and criteria as described by their respective element names. Element 5 is a separate topic, and will not be discussed in this paper

Policy Statement

The policy statement would define post-earthquake system operating expectations. For example, for a "design" earthquake, fire protection capability could be required for areas with a real estate density exceeding some given level. Supply for drinking water could be required within three days, even if it was provided through temporary facilities. Full resumption of service would be required within seven days.

The policy statement would provide guidance on a schedule for meeting the policy requirements. It would be expected that those schedules would be based on risk. For example, California facilities would have a shorter time to reach compliance than those in New York State.

System Evaluation

The system evaluation standard would include a procedure and evaluation data to assess the system's expected performance for comparison against the policy statement. Both facility component loss estimates and repair times would be included.

For example, application of the standard approach could answer the question, "Will the system be capable of providing adequate fireflows to given areas following a particular earthquake event?" Another example would be, "Can the service be restored to a particular residential area within seven days following the defined major earthquake?"

The standard would establish standard component function loss estimating methodologies in the form of empirical fragility curves, structural calculations, or other approaches. Data from component design and material and equipment standards would be used when available. Approaches to establish system functionality using component functionality would be defined. Techniques to estimate system restoration times would be incorporated.

Component Design/Evaluation and, Material and Equipment Standards

Component design/evaluation and material and equipment standards would be used both to design and specify new/replacement facilities as well as conduct detailed assessments of the vulnerability of existing facilities.

Component design codes are found in such documents as the Uniform Building Code and the American Water Works Association's D100, Standard for Welded Steel Tanks for Water Storage. Additional standards can be developed where they do not exist, such as seismic resistant design of pipelines. Examples of material and equipment standards are for pipe, as developed by AWWA, and electrical equipment, as developed by NEMA. They can be supplemented for seismic considerations where needed.

DISCUSSION - SYSTEM EVALUATION ELEMENT

Need for System Evaluation Element

The interaction of system components, should differentiate lifeline earthquake system design standards from building design standards.

System evaluation is not required in building design, as there is no interrelationship between buildings, new or existing. As new buildings are constructed in accordance with seismic design codes, the overall vulnerability of real estate is decreased.

Lifeline system vulnerability is not necessarily reduced as new system components are constructed. Most lifeline systems already exist. Many have key components that will last for many years before replacement would be warranted based on non-seismic performance criteria. In many cases, such as cast iron pipe in liquefiable areas, those components are highly vulnerable to earthquake. Some pipe could last, as it has in the past, for centuries without replacement. In many cases, failure of those components would mean failure of the system as a whole. Therefore, system component earthquake standards may not result in the overall system achieving the criteria identified in the policy statement. Therefore, a system evaluation standard should be incorporated in the proposed NIST standard.

Another advantage of the system evaluation approach is its ability to identify systems that are inherently weak, at a system level. Simplistic examples would be identification of systems that had single water sources or transmission lines, on which the entire system depended.

If the system evaluation approach is not incorporated in lifeline standards, there would be a great concern that the vulnerability of lifeline systems could remain high for many years, even though new/replacement facilities were being designed in accordance with seismic resistant standards.

Intent of System Evaluation Provisions

The intent of a system evaluation element is to require consideration of seismic criteria in water system planning. Typically, water systems are modified on a continuing basis to:

- Accommodate growth by providing increased water supply
- Replace corroded pipelines with high maintenance requirements and/or reduced flow capacity
- Replace/upgrade treatment facilities required to meet more stringent water quality requirements

- Replace/upgrade pump stations and storage with high maintenance requirements or with those that are inefficient to operate.

It is the plan that by taking seismic vulnerability into consideration during the planning process for the considerations listed above, the additional cost specifically for seismic upgrade would be minimal.

Several examples of this intent can be provided. First, many water purveyors replace pipelines on a regular basis. The criteria for replacement typically includes maintenance history and flow reduction from corrosion. Seismic considerations could be added, promoting the replacement of, for example, cast iron pipe in liquefiable soil areas. Second, assume a water purveyor delivers all their water through a transmission line passing through a liquefiable area. Because of increasing demand, the purveyor requires a second transmission line. If earthquake system evaluation requirements were in place, an option to design a new pipeline sited outside the liquefiable area, and with a capacity to replace some of the original vulnerable pipeline's capacity, might be more favorable.

CITY OF EVERETT, WASHINGTON LIFELINES STUDY

Introduction

The Everett lifeline study evaluated system vulnerability and estimated earthquake losses to six lifelines systems serving the city. Donald Ballantyne was the principal investigator and Bill Heubach, project engineer (Ballantyne, 1991b).

The City of Everett, with a population of 70,000, is located about 20 miles north of Seattle. The city is quite self reliant with major employers including Boeing at their 747/767 assembly plant and a Scott Paper Company mill.

The city is on a peninsula bounded on the west by Possession Sound, a branch of Puget Sound and on the north and east by the Snohomish River. The area around the Snohomish River and, to a lesser degree, along Possession Sound is highly liquefiable.

Lifeline Systems

Each of the six systems was studied by lifeline experts who are some of the best in the country in their respective fields. Each expert worked closely with the respective system owners. The lifeline systems, owners, and experts are shown in Table 1.

Project Methodology

The project proceeded through the following steps:

1. Earthquake Hazard Definition
2. System Inventory/Replacement Cost Estimate
3. Data Review/Site Visit
4. Component Vulnerability Assessment/Loss Estimate
5. System Vulnerability/Functionality Assessment
6. Repair Time Estimate
7. Secondary Loss Estimate.

Earthquake Hazard Definition

Three post-earthquake events were defined and hazards mapped by C.B. Crouse of Dames & Moore and are shown in Table 2.

Two of the three earthquakes scenarios were very large, one in 400 years, events resulting in mapped areas of Modified Mercalli Intensity as high as IX. Liquefaction probability was also mapped. Earthquake intensity and liquefaction probability was provided to each of the lifeline experts.

System Inventory, Data Review/Site Visit

Each lifeline expert worked closely with the system owner in developing the system inventory, estimating direct losses (repair/replacement costs), and gathering design information. Repair costs were based on repair time estimates for each repair, labor, equipment, and material costs. Replacement costs were based on original construction costs (when available) escalated to current costs or comparing the costs to other facilities with a known value. Typically, a system owner representative joined the lifeline expert in site visits.

It should be noted that the study considered the entire water, sewer, and highway/bridge system, while only the local intertie (substation) and distribution systems were considered for electric power, telephone, and natural gas. This likely had an affect on the reported results.

Component Vulnerability Assessment/Loss Estimate, and System Vulnerability/Functionality Assessment

Each lifeline expert estimated the extent of damage to each system component based on fragility curves and/or engineering judgement. The extent of damage was estimated for both repair replacement and functionality. Damage assessment fragility curves and engineering judgement for each lifeline was based on each lifeline expert's experience in previous studies. In many cases, it was found that because of the linear nature of the systems that system outage could be estimated without the use of a system network analysis. The impact on the functionality of each system was then assessed with input from the system owner. Direct losses (repair/replacement) for the M8.25 Benioff Interplate Earthquake are shown on Table 3.

Repair Time Estimate

System owners were provided with estimated system component damage from each lifeline expert. System owners then estimated the time to repair their system based on expected available resources. Those system outage duration times are shown for the M8.25 Benioff Interplate Earthquake on Table 3.

One important consideration of the study was that damage information for each system was provided to all system owners simultaneously, in order for them to account for the impact of other system disfunction on the repair of their own systems. One example was the impact on access for repairs limited by bridges being out of service.

Secondary Loss Estimate

Business interruption (secondary) losses were estimated taking into account the gross city product, outage duration for each particular lifeline, and a economic dependency factor on the particular lifeline. Secondary losses from fire were not estimated in this project. Those business interruption losses are shown for the M8.25 Benioff Interplate Earthquake on Table 3.

Everett Study Conclusions

The study showed that business interruption losses were less than direct losses. However, these results were skewed by the high direct cost of bridge replacement. If bridges are excluded, the average ratio of business losses to direct losses is approximately 2 to 1. The business/direct loss ratios for each system are shown for the M8.25 Benioff Interplate Earthquake on Table 3.

The loss ratio ranges from (-0.1) to 76. This extreme range may be the result of several system-specific considerations. Lifeline systems are inherently different in their repair costs versus the cost of losing service. Bridge spans are extremely expensive to repair compared to repairing a pipeline break. Both failures may have the same order of magnitude impact on a business. The telephone system was evaluated as having extreme differences between the dollar loss and function loss fragility curves. Other lifelines had curves that more closely matched.

Another reason for the range of business to direct losses may be related to lack of standardization in loss estimation approaches.

The study shows that direct damage estimates do not adequately represent total losses expected as a result of earthquake lifeline failure.

CONCLUSIONS AND RECOMMENDATIONS

The Everett study demonstrates that it is feasible, using currently available methodologies and data, to evaluate lifeline system post-earthquake functional reliability. Estimation of functional reliability is the focus of Element 2, System Evaluation, in the proposed lifeline standard.

In addition to functional reliability, system outage duration, direct losses, as well as secondary, business interruption, losses can be estimated.

The effect of the interaction of system disfunction can be incorporated into the system disfunction and outage duration estimates, resulting in a more realistic model.

There is a significant variability among the lifelines evaluated between the estimated magnitude of direct losses and secondary losses. The inherent differences between different lifeline system's design and operation may account for some of these differences.

However, the methodologies and data available to develop Element 2, System Evaluation, with an acceptable level of reliability, lags compared to the other proposed standards elements.

Looking beyond the scope of the Everett study, in retrospect, there may be inconsistencies between the findings of that study and actual earthquakes. Even following the two large earthquakes scenarios proposed in Everett, system outages were as low as zero hours. There is some concern, that, for example, compared to the smaller Loma Prieta event, resulting system disfunction may be more severe for some of the lifelines studied.

Recommendations resulting from the consideration of the Everett study as an example of lifeline system evaluation are as follows:

- System evaluation should be included as an element in a lifeline earthquake standard.

- Better earthquake damage data from which to enhance fragility curve development should be developed on a continuing basis. This includes improved methods for quantification of earthquake hazards.

- A more consistent approach to loss estimation should be developed across the industry. This can be achieved in the standards development process.

- Better education of the system owners in understanding the effects of earthquakes on their lifeline systems should be undertaken.

ACKNOWLEDGEMENTS

The author would like to thank the National Institute of Standards and Technology working through the American Society of Civil Engineers Technical Council on Lifeline Earthquake Engineering for support in developing the NIST Draft Plan (Ballantyne, 1991a). Appreciation also goes to the United States Geological Survey for support in conducting the Everett study under award number 14-08-0001-G1804 (Ballantyne, 1991b).

A special acknowledgement goes to each of the Everett study teams presented in the text, including both the lifeline experts as well as lifeline system owner representative.

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Dikkers, Robert D., "Development of Seismic Design Standards for Lifelines" National Institute of Standards and Technology, presented at the U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, August 19-21, 1991, Los Angeles, CA.

TABLE 1

LIFELINES/OWNERS/LIFELINE EXPERTS

Lifeline	Owner	Lifeline Expert
Water	City of Everett	Donald Ballantyne, Kennedy/Jenks
Sewer	City of Everett	Donald Ballantyne, Kennedy/Jenks
Electric Power	Snohomish County PUD	Dennis Ostrom, S. California Edison
Telephone	GTE	Felix Wong, Weidlinger Associates
Natural Gas	Washington Natural Gas	Ron Eguchi, Dames & Moore
Highways/Bridges	WASHDOT	Stu Werner, Dames & Moore

TABLE 2

EARTHQUAKE EVENTS

Source Zone	Magnitude	Return Interval(yrs)	Hypocentral Distance(mi)	Focal Depth (mi)
Puget Trough	6.5	403	7.5	6
Benioff Interplate	7.0	58	32	25
Benioff Intraplate	8.25	452	46	9

TABLE 3

EVERETT STUDY RESULTS

Lifeline	Direct Losses (million)	Outage Duration (days)	Business Losses (million)	Business/ Direct Losses
Water	$4.8	6	$44.3	9.2
Sewer	6.2	6	44.3	7.1
Electric Power	0.3	0.1	0.5	1.7
Telephone	31.7	0	(-3.2)	(-0.1)
Natural Gas	0.2	2	15.1	76
Highway/Bridges	171	7	35.3	0.2
Total	$214.2		$136.3	

DYNAMIC RESPONSE OF TWIN CIRCULAR TUNNELS DURING EARTHQUAKES

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ABSTRACT

This paper presents the dynamic behavior of twin circular tunnels subjected to the vertically incident SV wave. Using the two dimensional finite element method, the interaction effects of the twin tunnels on the response of the lining are evaluated by comparing the results of the twin-tunnel and single-tunnel cases. The results show that the shearing force on the adjacent side of the lining increase due to the interaction between the two tunnels. The degree of the interaction is sensitive to the distance of the tunnels and the shear wave velocity of the soil surrounding the tunnels, although the depth of the tunnels does not affect the results. However, the interaction effects are discovered to be negligible when the distance between the two tunnels is greater than two times the diameter of the tunnel.

INTRODUCTION

Because of the difficulty in obtaining large and continuous lands for new construction projects in metropolitan areas such as Tokyo, subterranean spaces will be increasingly utilized in the near future for the construction of railroads, highways, etc.

Under the circumstances, "A Cooperative Research in Development of Seismic Design of Underground Structures" has been carried out by Public Research Institute, Advanced Construction Technology Center, and eight other institutions as a part of the Comprehensive Research Project entitled "Technology Research and Development Regarding Use of Underground Spaces." This study is a part of the cooperative research mentioned above.

Regarding the seismic resistant design of a tunnel, only the longitudinal direction has been examined in most cases. However, when the size of a tunnel becomes larger, the cross-sectional direction should also be examined since the displacement at the top relative to the bottom is greater than that of small tunnels, resulting in the larger resultant force of the lining. In addition, the dynamic behavior of twin tunnels has not been thoroughly studied, although such twin tunnels has been and will be constructed.

In this study, dynamic response analyses of twin tunnels are carried out by using the two-dimentional finite element method, and the dynamic interaction effects of the twin tunnels are examined. The results obtained in this study will provide useful information in the development of seismic design of underground structures.

SCOPE OF THIS STUDY

This study deals with the dynamic behavior of twin circular tunnels subjected to the vertically incident SV wave. For this purpose, the two-dimensional finite element method in the frequency domain is utilized to calculate the shearing force, axial force, and bending moment of the tunnel lining.

The interaction effect of one tunnel to the other is examined by comparing the numerical results (transfer functions, response time histories, and maximum response distribution along the lining) of the single tunnel with those of the twin tunnels with various distance, while other parameters such as shear wave velocity of the ground and depth of the tunnels are fixed.

From the comparison, the location where the interacton effect appears most noticeably is identified first. Next, the degree of increase in the shearing force, axial force and bending moment at these places are evaluated. Then, the relation between the degree of the interaction effect and the distance of the two tunnels is developed. The contribution of the shear wave velocity of the surrounding soil and the depth of the tunnels to the interaction effect is also studied.

The flow of this study is summarized in FIGURE 1.

ANALYSIS MODEL

FIGURE 2 depicts the summary of the analysis model used in this study. The finite element models have the viscous boundary at the bottom and the energy-transmitting boundary on the side to represent the half-space vertically and laterally. Beam elements are used to model the lining (concrete shield) of the tunnels.

Throughout this study, the diameter of the tunnels and the properties of the lining are fixed. The two tunnels are also assumed to be identical in size, depth and lining.

The following parameters are considered to be variables:
Distance between two tunnels (D): 5m, 10m, 20m and ∞(single)
Depth of tunnels (H): 20m, 35m and 50m
Shear wave velocity of soil (Vs): 100m/s, 200m/s and 400m/s

RESULTS AND DISCUSSIONS

Transfer Functions

First, the interaction effect of the two tunnels on the bending moment and axial and shearing forces of the tunnel lining is examined in the frequency domain.

FIGURES 3, 4 and 5 show the transfer functions of the bending moment, axial force, and shearing force, respectively, at several points on the lining of the (right) tunnel, and compares the results of the single and twin tunnels. These points on the lining are selected for each figure because the maximum forces appear at these points when a tunnel is subjected to the shearing deformation. These results are for the shear wave velocity of soil = 200m/s, and the depth of the tunnels = 50m.

It is seen in FIGURE 3 that the interaction effets on the bending moment appear especially on the side adjacent to the other tunnel. When the distance between the two tunnels is 5m, the bending moment on this side is approximately 20% larger in the very low frequency range than that of the single tunnel. However, the interaction effect is negligible on the farther side of the tunnel.

In FIGURE 4, the interaction effect on the axial force is not as remarkable. Even when the distance between the tunnels is 5m, the increase in the axial force is at most 10%.

On the contrary, as seen in FIGURE 5, the interaction effect on the shearing force is clear on the side adjacent to the other tunnel. When the two tunnels are 5m apart, the shearing force at $\theta=-90°$ is approximately 40% greater than the single tunnel. However, the degree of the interaction effect decreases as the distance between the two tunnels increases. When the distance is 20m, the result is almost identical to that of the single tunnel. It should be noted that the ratio of the amplitude of the twin tunnel to single tunnel is almost identical, at least in the low frequency range (0 to 4 Hz).

Time Histories of Response

FIGURE 6 compares the time histories of the shearing force at the two different points (θ=-90° and 90°) on the lining when the N-S component of 1940 El Centro record is used as the input earthquake ground motion. In each figure, the solid line represents the result of the twin tunnel with distance of 5m and the dotted line corresponds to the single tunnel.

At the point adjacent to the other tunnel(θ=-90°), the shearing force of the twin tunnel is magnified due to the interaction of the two tunnels. On the other hand, the difference is not noticeable at θ=90°. The increases in the maximum value of the time history are 39% at θ=-90° and only 2% at θ=90°.

Maximum Response Distribution

The distributions of maximum response on the lining are drawn in FIGURE 7 by plotting the maximum value of the time history at each of the 48 points (every 7.5°) on the lining.

The largest difference appears on the distribution of the maximum shearing force near θ=-90°. It agrees with the results shown in FIGURES 5 and 6. The distributions of the maximum axial and shearing forces of the twin tunnels shift slightly to the right on the figures. This suggests that the deformation of the tunnels is slightly slanted.

In order to measure the interaction effect quantitatively, the interaction coefficient is introduced in this study. The interaction coefficient is defined as the increase in the peak value of the maximum response distribution normalized by the value of single tunnel. In FIGURE 7, the coefficient is calculated as a/b. The range of the angle in which the coefficient is evaluated is shaded in the figure. In the case shown (D = 5m, Vs = 200m/s, and H = 50m), the interaction coefficients are 0.14, 0.08 and 0.41, respectively, for the bending moment, axial force and shearing force.

FIGURE 8 compares the distribution of the maximum responses of the twin tunnels for the different input ground motions. The solid and dotted lines correspond to the N-S component of 1940 El Centro record and N-S component of Hachinohe record of 1968 Tokachi-oki earthquake, respectively. Although the Hachinohe record causes the greater responses, the shapes of the distribution of responses are quite similar to each other, meaning that the difference in the frequency contents of input motion does not affect the degree of interaction. This fact is also supported by the transfer functions (FIGURES 3 to 5) where the ratio of twin tunnels to single tunnel is almost constant throughout the frequency range.

Relation between Interaction Effect and Distance

The interaction coefficient which was defined previously is calculated for various conditions, i.e., the shear wave velocity Vs= 100, 200 and 400m/s, and the depth H = 20, 35 and 50m. In all of the above cases, the distance between the two tunnels D is changed from 5 to 20m.

The interaction coefficients for the depth 20, 35 and 50m are shown in FIGURE 9. The shear wave velocity is fixed to 200m/s in the calculation. This figure clearly shows the way in which the interaction coefficients decreases as the distance between the tunnels increases. It is also apparent that the depth of the tunnels does not affect the degree of interaction.

FIGURE 10 shows the interaction coefficients for Vs = 100, 200 and 400m/s, where the depth is fixed to 50m. In contrast to FIGURE 9, the interaction coefficient is found to be very sensitive to the shear wave velocity of soil surrounding the tunnels. Given the same tunnel distance, the interaction coefficient is almost proportional to Vs, e.g., the coefficient for Vs = 400m/s is nearly twice as much as that for Vs = 200m/s, and almost four times that for Vs = 100m/s. However, even for the case with Vs = 400m/s, the interaction effect is negligible when the distance of the tunnels is 20m, i.e., twice the diameter of the tunnel.

CONCLUSIONS

In this study, the dynamic interaction effect of twin circular tunnels subjected to incident SV wave is examined numerically. Major results derived in this study may be summarized as follows:

(1) The interaction effect appears most clearly on the shearing force of the lining on the side adjacent to the other tunnel. On the contrary, the effect on the axial force is small.

(2) The degree of interaction is not sensitive to the frequency contents of input ground motion.

(3) The degree of interaction decreases exponentially as the distance between the tunnels increases. The effect is negligible when the distance is more than two times the diameter of the tunnel.

(4) The interaction coefficients defined in this study is almost proportional to the shear wave velocity of the soil

when other parameters are fixed. However, the difference in the depth does not change the coefficient.

FUTURE DIRECTIONS

This study is still on-going. The following subjects are to be studied in the future so that the results of this study are reflected in the development of seismic design of underground structures.

(1) The interaction coefficient may be expressed as a function of the distance between the tunnels (D), diameter of the tunnel (d) and shear wave velocity of soil (Vs). Additional parameter studies will be necessary to develop the form of the function.

(2) The interaction effect when the twin tunnels are constructed vertically or diagonally should also be examined.

ACKNOWLEDGMENT

This study is carried out as a part of the "Cooperative Research in Development of Seismic Design of Underground Structure."

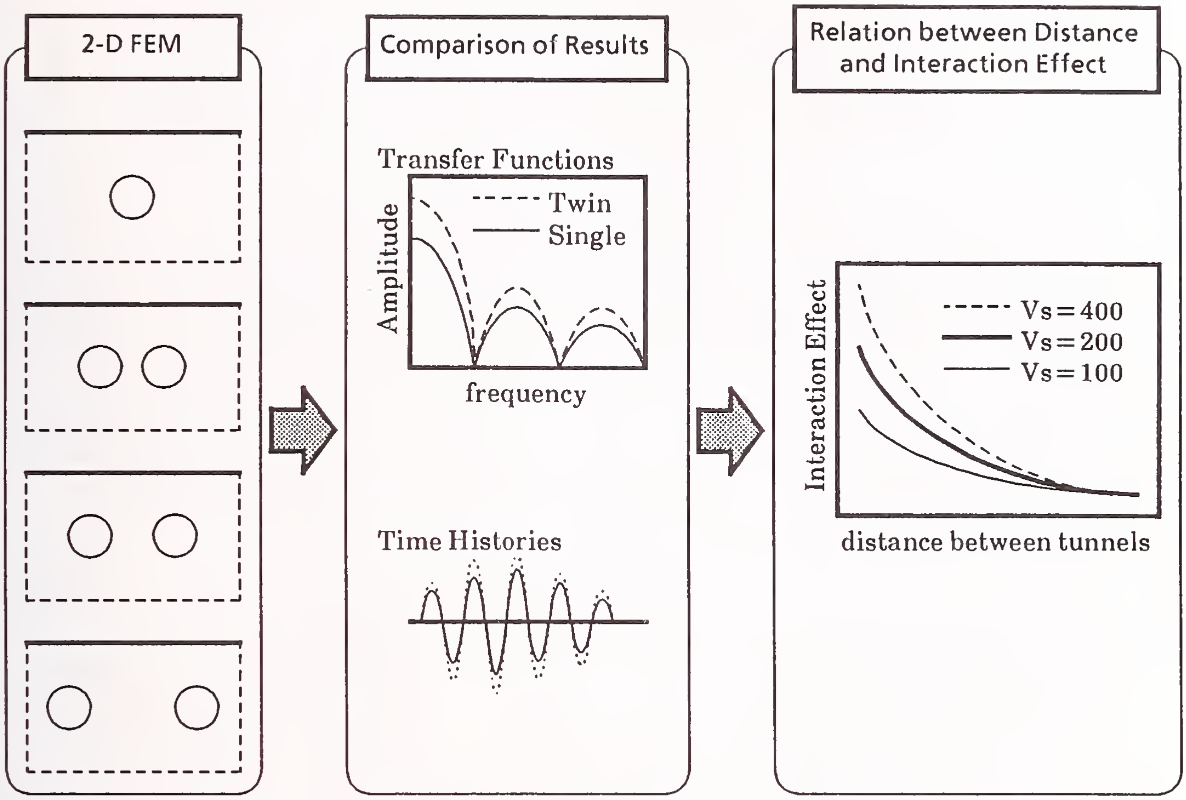

FIGURE 1 FLOW OF THIS STUDY

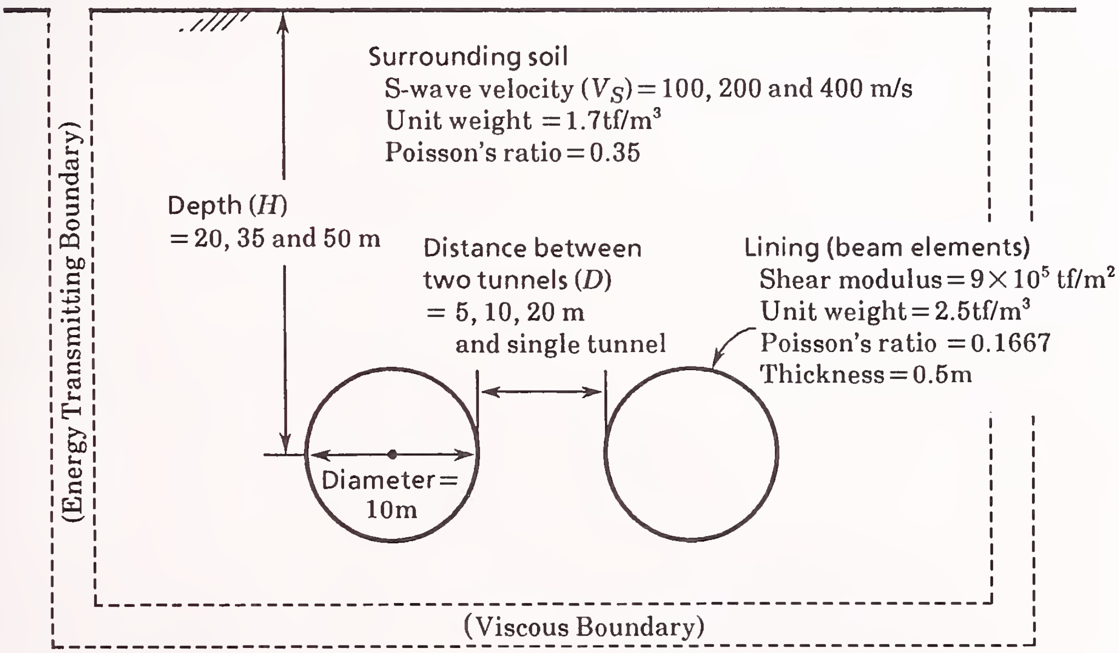

FIGURE 2 ANALYSIS MODEL

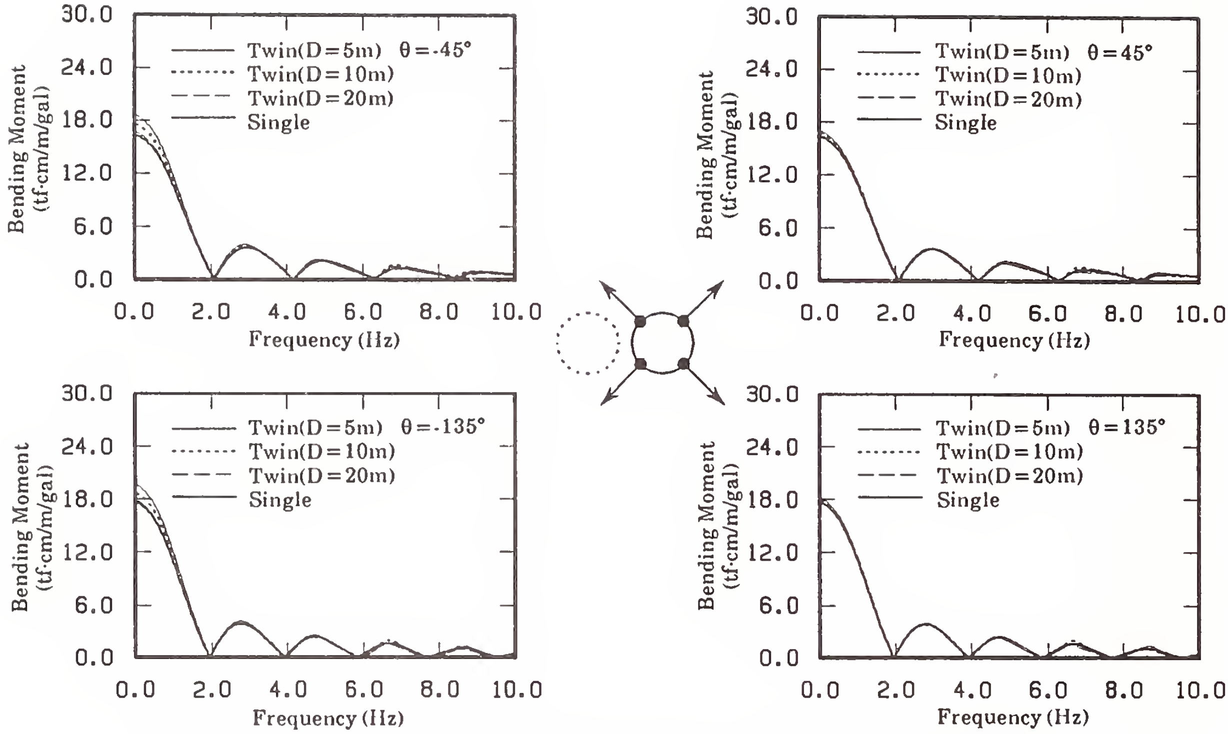

FIGURE 3 TRANSFER FUNCTIONS OF BENDING MOMENT (Vs=200m/s, depth=50m)

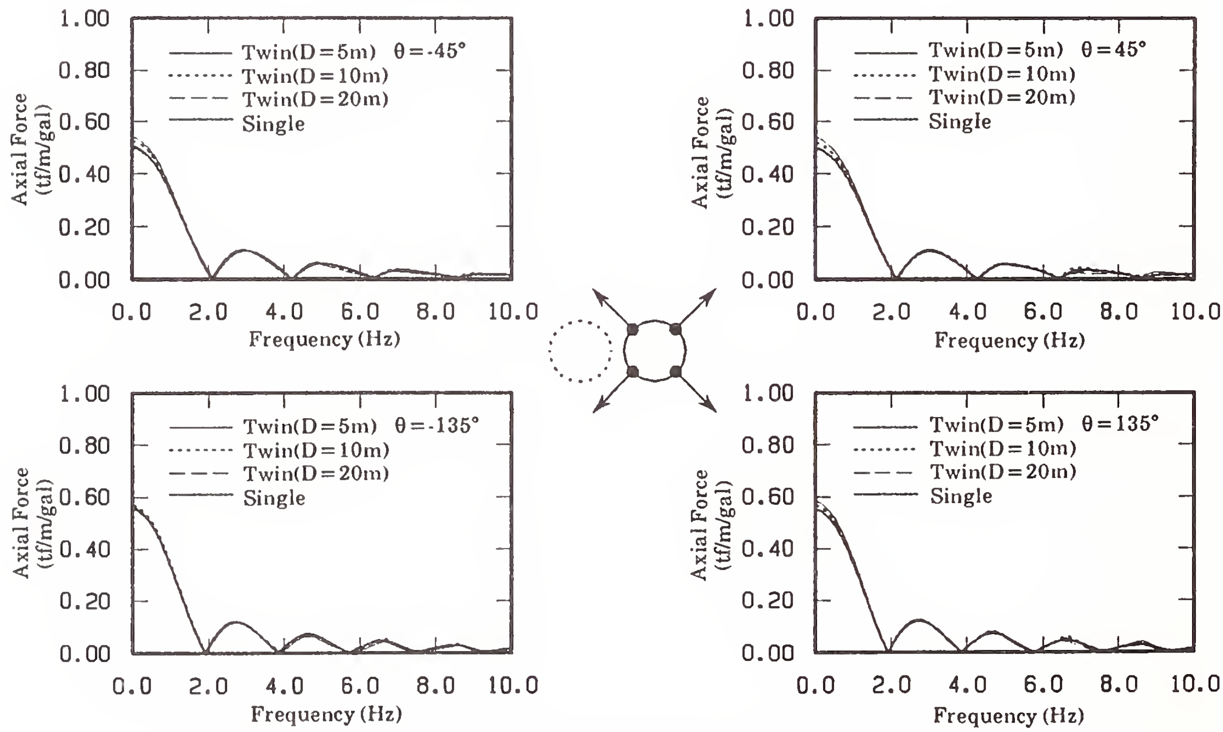

FIGURE 4 TRANSFER FUNCTIONS OF AXIAL FORCE (Vs=200m/s, depth=50m)

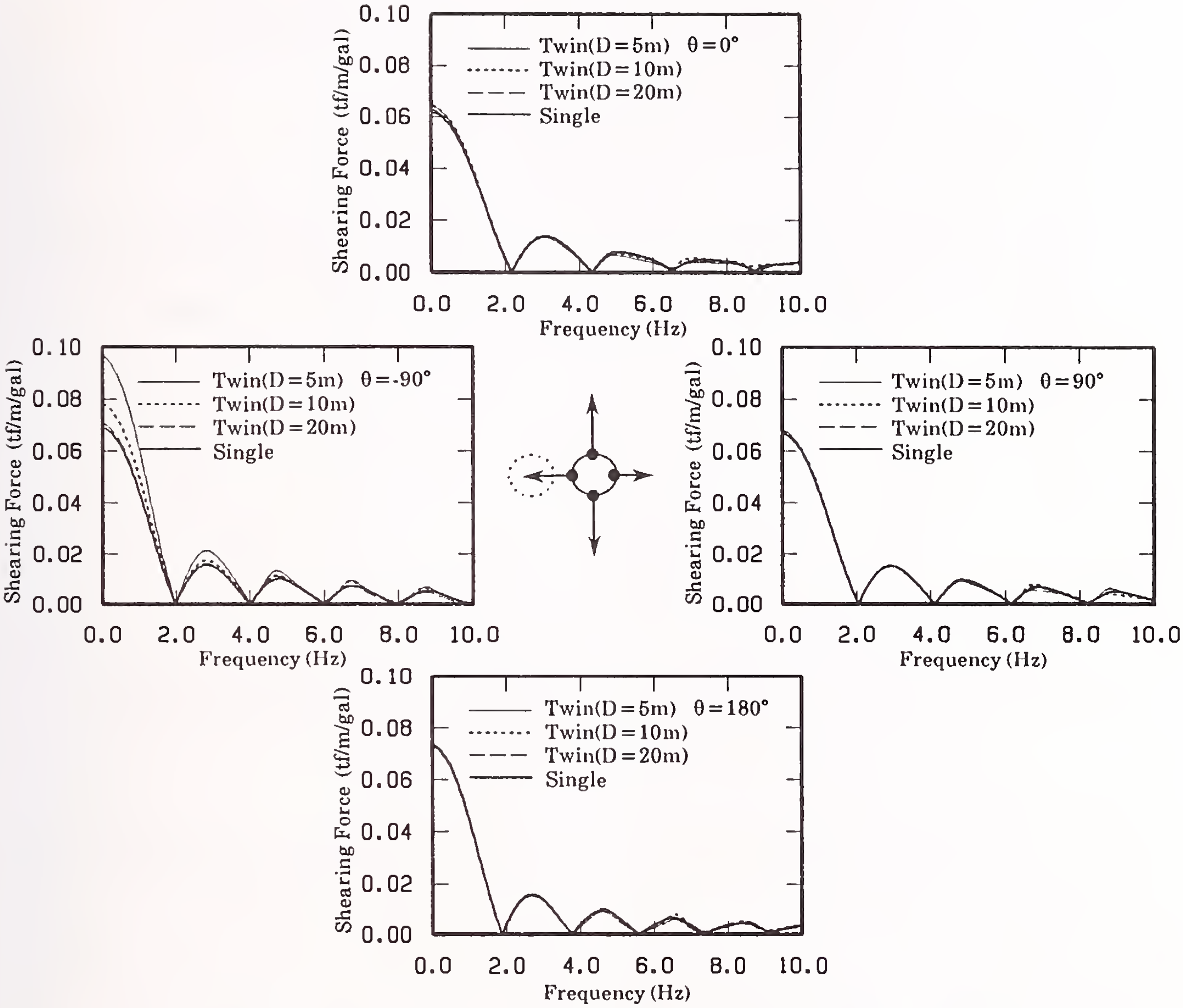

FIGURE 5 TRANSFER FUNCTIONS OF SHEARING FORCE
(Vs=200m/s, depth=50m)

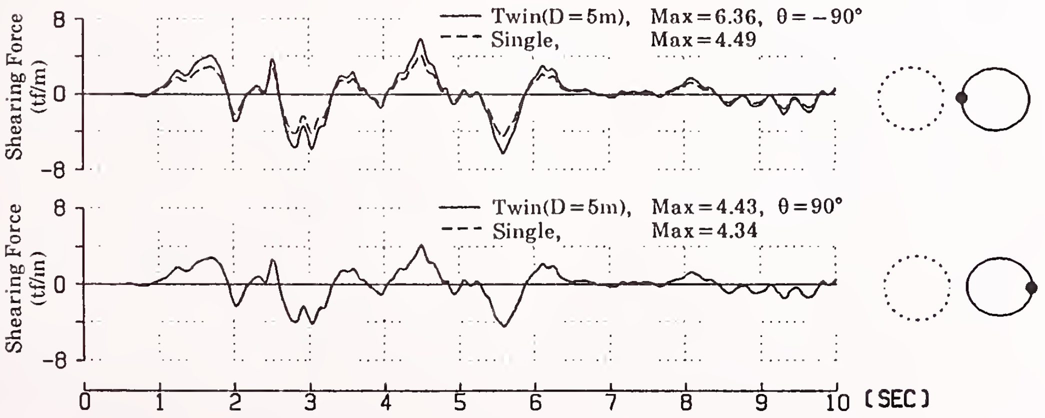

FIGURE 6 TIME HISTORIES OF SHEARING FORCE

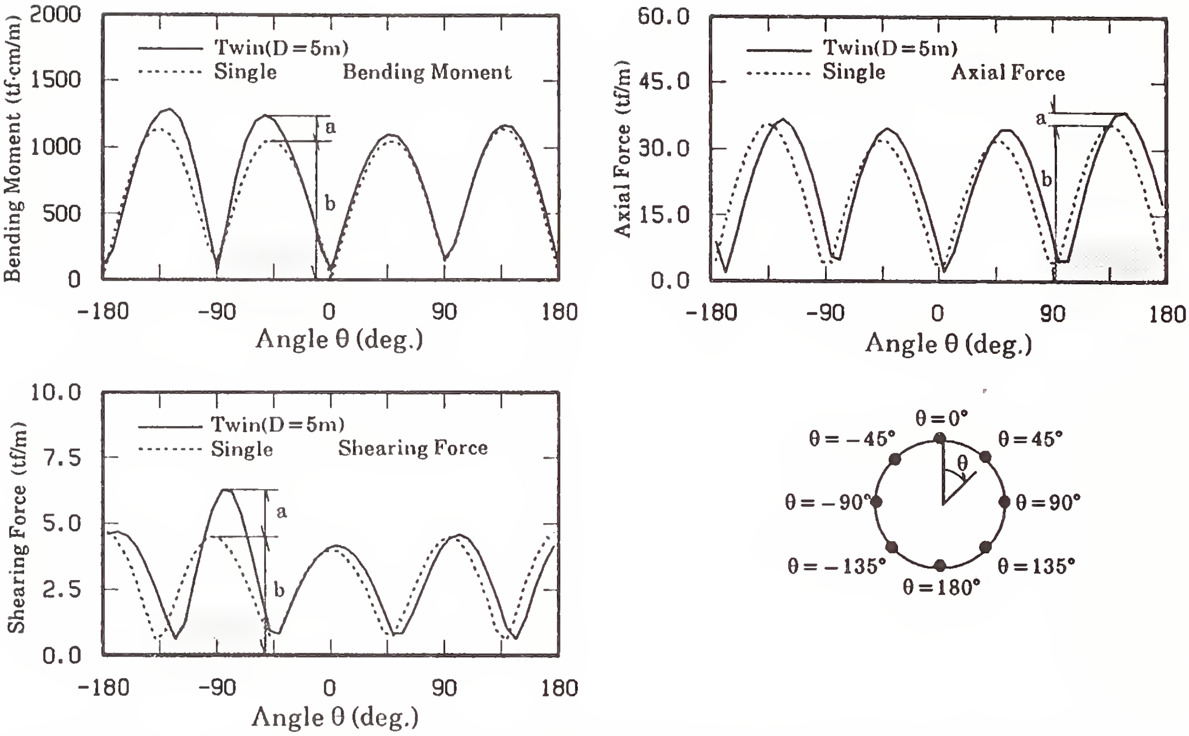

FIGURE 7 COMPARISON OF DISTRIBUTIONS OF MAXIMUM RESPONSE (SINGLE AND TWIN TUNNELS)

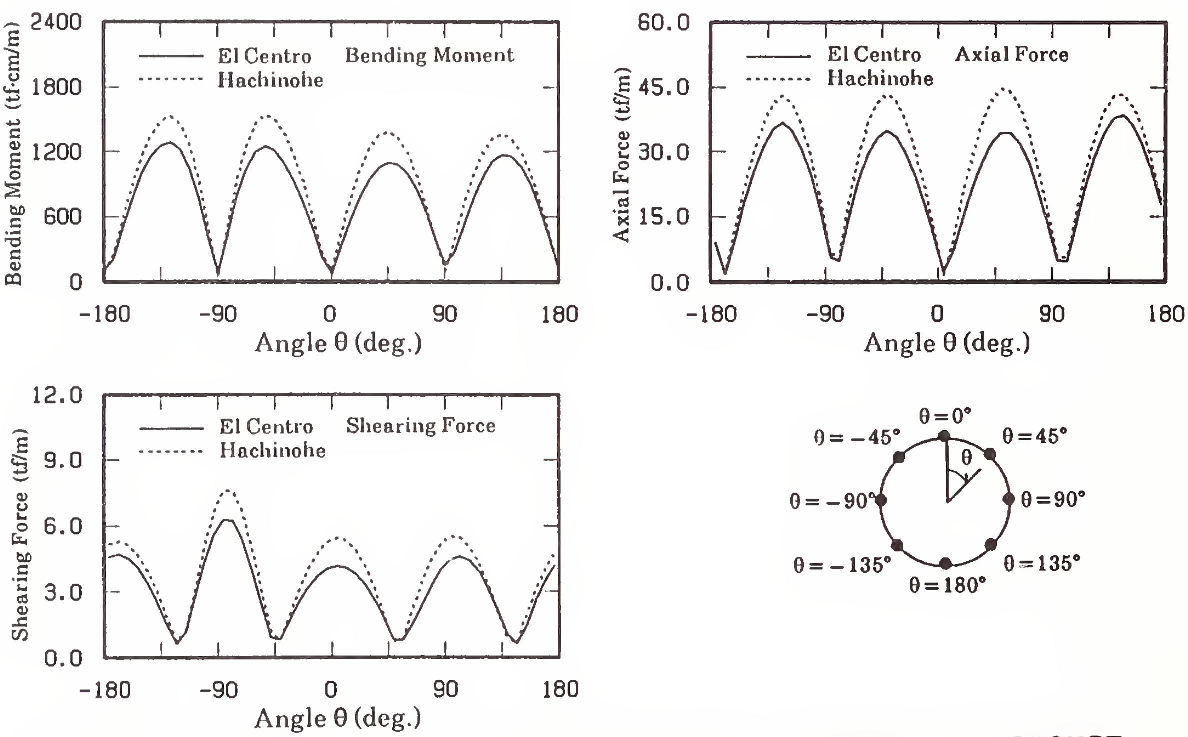

FIGURE 8 COMPARISON OF DISTRIBUTIONS OF MAXIMUM RESPONSE (EL CENTRO AND HACHINOHE RECORDS)

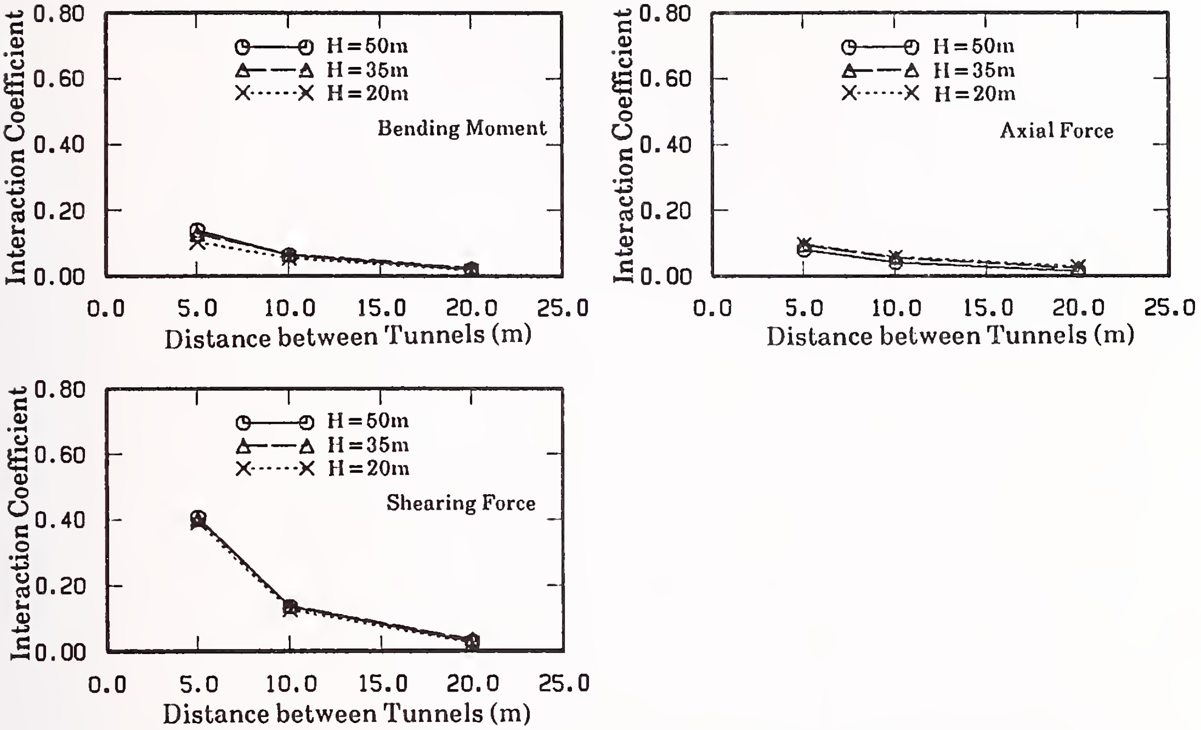

FIGURE 9 COMPARISON OF INTERACTION COEFFICIENTS FOR VARIOUS DEPTH

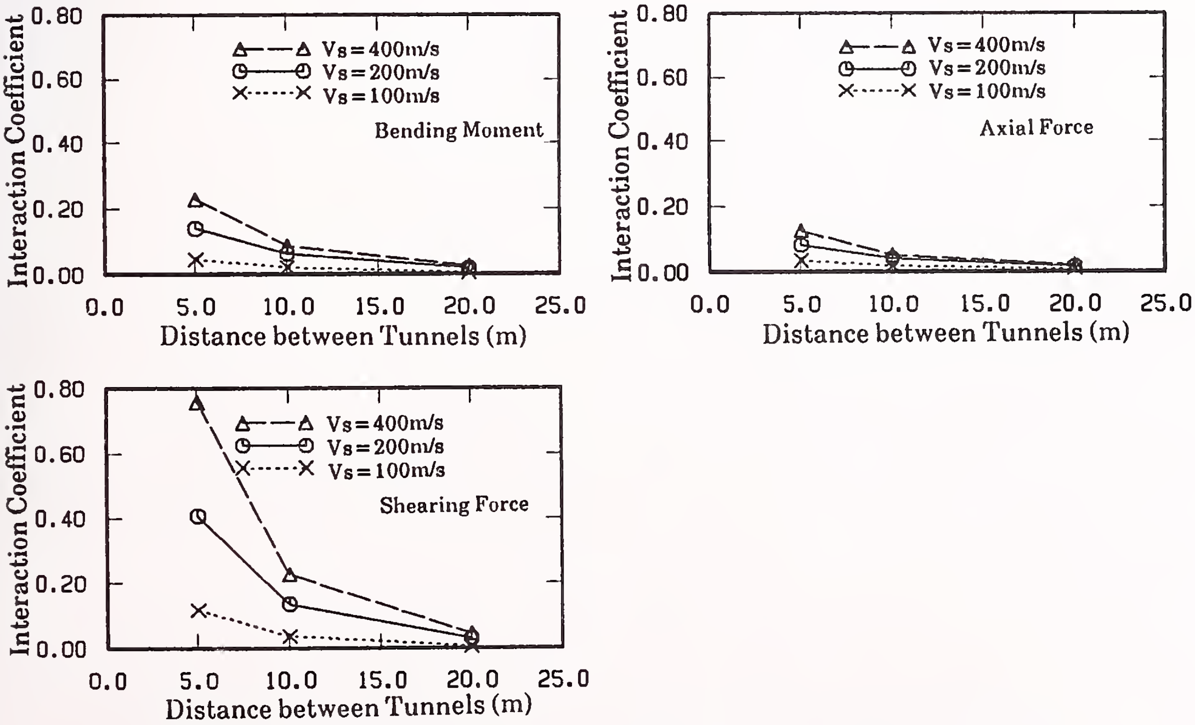

FIGURE 10 COMPARISON OF INTERACTION COEFFICIENTS FOR VARIOUS SHEAR WAVE VELOCITIES

DEVELOPMENT OF SEISMIC DESIGN AND CONSTRUCTION STANDARDS FOR LIFELINES

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ABSTRACT

Section 8(b) of the National Earthquake Hazards Reduction Program Reauthorization Act, which was approved on November 16, 1990, requires the Federal Emergency Management Agency (FEMA), in consultation with the National Institute of Standards and Technology (NIST), to submit to the U.S. Congress, not later than June 30, 1992, a plan for developing and adopting, in consultation with appropriate private sector organizations, design and construction standards for lifelines. This paper discusses the process, participants, and schedule being utilized to prepare the lifelines standards development plan. Lifeline systems being addressed are electrical power, gas and liquid fuel, telecommunications, transportation, and water and sewer. The plan will include the following information for the various lifeline systems: seismic vulnerability; current design and construction practices and standards; available knowledge to improve existing practices; recommended standards to be developed for new and existing construction; and recommended research to fill identified knowledge gaps.

INTRODUCTION

Earthquake Hazards Reduction Act of 1977

In October 1977, the U.S. Congress enacted Public Law 95-124, the Earthquake Hazards Reduction Act of 1977. Passage of this Act reflected a growing recognition that all 50 States are vulnerable to the earthquake hazard, and 39 States are subject to major or moderate seismic risk[1]. The purpose of the Act is to reduce the risks to life and property in the United States from future earthquakes through the establishment and maintenance of an effective earthquake hazard reduction program. The objectives listed in the Act for the National Earthquake Hazards Reduction Program (NEHRP) include:

- Conduct earthquake hazard-identification and vulnerability analysis;
- Develop seismic design and construction standards;
- Develop an earthquake prediction capability;
- Prepare plans for mitigation, preparedness, and response activities;
- Conduct fundamental and applied research into the causes and implications of earthquake hazards; and

- o Educate the public about earthquake hazards.

In regard to the second objective (i.e., development of seismic design and construction standards), a key accomplishment relating to building structures has been the publication, dissemination, and adoption of the principles contained in the NEHRP Recommended Provisions for Development of Seismic Regulations for New Buildings[2].

Abatement of Seismic Hazards to Lifelines: An Action Plan

In 1985, the Federal Emergency Management Agency (FEMA) asked the National Institute of Building Standards (NIBS), the parent organization of the Building Seismic Safety Council (BSSC), to prepare a plan to reduce seismic hazards to new and existing lifelines[3]. Under the direction of a BSSC Action Plan Committee, specialists in all lifeline categories and in legal/regulatory, political, social, economic, and seismic risk aspects of lifeline hazard mitigation were invited to prepare issue papers. Forty-two issue papers were circulated among peers and were the basis of discussions by the 65 participants at a workshop held November 5-7, 1986, in Denver, Colorado. The major product of the workshop was an action plan titled Abatement of Seismic Hazards to Lifelines: An Action Plan[4].

The Action Plan recommended actions in four areas: (1) public policy, legal and financial strategies; (2) information transfer and dissemination; (3) emergency planning; and (4) scientific and engineering knowledge. Of the almost $29 million required for the recommended activities, about $21 million was for scientific and engineering knowledge (e.g., improve geotechnical knowledge; increase knowledge of performance of specific components; develop improved equipment and material for use in seismic resistant construction; develop design criteria, codes and standards of practice for design, construction, and retrofitting of seismic resistant lifeline facilities; etc.).

In 1989 a NIBS ad hoc Panel on Lifelines recommended that FEMA undertake a nationally coordinated program to mitigate the effects of earthquakes and other natural hazards on lifelines[5]. Recommended activities included: awareness and education; vulnerability assessment; design criteria and standards; regulatory policy; and continuing guidance.

NEHRP Reauthorization Act of 1990

Section 8(b) of the National Earthquake Hazards Reduction Program Reauthorization Act, Public Law 101-614, which was approved on November 16, 1990, requires the Director of the Federal Emergency Management Agency (FEMA), in consultation with the National Institute of Standards and Technology (NIST), to submit to the U.S. Congress, not later than June 30, 1992, a plan, including precise timetables and budget estimates, for developing and adopting, in consultation with appropriate private sector organizations, design and construction standards for lifelines. The plan is also required to include recommendations of ways Federal regulatory authority could be used to expedite the implementation of such standards.

LIFELINES STANDARDS DEVELOPMENT PLAN

Need for Design and Construction Standards

Experiences in the 1971 San Fernando, California and the 1989 Loma Prieta, California earthquakes and other earthquakes show that the successful performance of lifeline systems (water and sewage, transportation, gas and liquid fuels, electrical power, and communications) is vital for prevention of severe human and economic losses[6]. Except for design standards for new highway bridges, dams, and nuclear reactor facilities, no nationally recognized standards are available in the United States for the design and construction of new lifelines or for the assessment and strengthening of existing lifelines.

The development of design and construction standards, as mentioned above, has been an important objective of the NEHRP since its inception in 1977. Although significant progress has been made in the development of improved seismic standards for new buildings, there is considerable research and work remaining to be done to achieve a comparable status with respect to lifelines. Recommendations which have been presented during the past 5 years for the development of standards for lifelines, along with the U.S. congressional mandate of 1990 for a lifelines standards development plan will establish a starting point from which similar progress can be made in lifeline systems.

As indicated by the NIBS ad hoc Panel on Lifelines, "design criteria and standards for lifeline hazard mitigation provide consistent minimum recommended levels of facility engineering design and construction practice. They identify natural hazard abatement techniques and practices for those responsible for all phases of lifeline design, construction, and operation and serve as the basis for model code provisions, which can be considered by local, state, and federal regulatory bodies for adoption into ordinances and regulations.[5]"

Plan Development Process and Scope

The overall lifelines standards plan development process presently being implemented has been established with the advice of a Steering Group organized by FEMA. The Group, chaired by Dr. Ronald Eguchi, Chairman, ASCE Technical Council on Lifeline Earthquake Engineering (ASCE TCLEE), includes representatives from FEMA, NIST, Department of Energy, Federal Energy Regulatory Commission, National Center for Earthquake Engineering Research (NCEER), and several members from various private sector organizations. During its first meeting in March, 1991, the Steering Group approved a strategy for the lifelines standards development process. The strategy included using lifelines experts to prepare and review draft plans for the development of design and construction standards for the various lifeline systems. The experts identified by the Steering Group and selected to author the draft plans are:

- Water & Sewer Systems -- Mr. Donald Ballantyne, Kennedy/Jenks/Chilton, Federal Way, Washington.

- Transportation Systems -- Dr. Ian Buckle, NCEER, Buffalo, New York.

- Gas and Liquid Fuel Systems -- Dr. Douglas Nyman, D.J. Nyman & Associates, Houston, Texas.

- Electrical Power Systems -- Dr. Anshel Schiff, Stanford University, Palo Alto, California.

- Telecommunication Systems -- Mr. Alex Tang, Northern Telecom Canada Ltd., Ontario, Canada.

- Federal Implementation & Other Issues -- Mr. Crane Miller, Attorney, Washington, DC.

In each of the above areas, five expert reviewers have also been identified and selected. These individuals will review the draft plans and participate in a discussion of the plans at a workshop to be held in Denver, Colorado, on September 25-27, 1991. Approximately 85% of the key individuals involved in the plan development process described above are active in the ASCE TCLEE.

In general, the plan will include the following information for the various lifeline system categories: seismic vulnerability; current design and construction practices and standards; available knowledge to improve existing practices; recommended standards to be developed for new and existing construction; and recommended research to fill identified knowledge gaps. It will also contain recommended timetables and budget estimates and recommendations of ways Federal regulatory authority could be used to expedite the implementation of such standards. The BSSC Action Plan[4] and NIBS ad hoc Panel report[5] are being used as background resource documents.

The current scheduled completion date for the final draft lifelines standards development plan is January 1, 1992. After this date, the draft plan will be submitted by FEMA to the Office of Management and Budget for review prior to its submission to the U.S. Congress not later than June 30, 1992.

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IV. SEISMIC DESIGN & ANALYSIS: PIPELINES & TUNNELS, PART TWO

"Data Base of Pipeline Failures, Loma Prieta Earthquake, October 17, 1989"
L. Lund

"Behavior of Flat Underground Structures during Earthquakes and Seismic Loads Acting on Them"
S. Mori, T. Ikeda, K. Matsushima, H. Tachibana

"Seismic Response of Super-Deep Vertical Shaft with Circular Cross-Section"
N. Ohbo, K. Hayashi, K. Ueno

"Seismic Design Method of Shield Tunnels with Axial Prestressing by Means of Rubber and PC Bar"
K. Matsubara, K. Urano

"Seismic Isolation for Underground Structures"
K. Ono, S. Shimamura, H. Kasai

DATA BASE OF PIPELINE FAILURES
LOMA PRIETA EARTHQUAKE
OCTOBER 17, 1989

Le Val Lund, P.E., M. ASCE
Civil Engineer, Los Angeles, CA

ABSTRACT

The American Society of Civil Engineers, Technical Council on Lifeline Earthquake Engineering was awarded a grant by the National Science Foundation (NSF) to develop a data base of pipeline failures from the Loma Prieta Earthquake, October 17, 1989. The purpose of the project is to preserve information on the location, pipe size, pipe material and type of pipeline failures for the engineering, geological and seismological community and as well as future seismic risk and lifeline researchers. The resulting data base and methodology will be available in a public depository, in disk format, for personal computers, to the engineering and scientific community at the cost of duplication.

INTRODUCTION

Lifelines

Lifelines are those services vital for people and the functioning of an urban and industrialized society. They are also necessary for emergency response and recovery of a community after a disaster such as an earthquake. Lifelines include power, communication, transportation, water, sewage, gas and liquid fuel systems.

Loma Prieta Earthquake

On Tuesday, 17 October 1989 at 5:04 p. m. (PDT) an earthquake, magnitude Ms 7.1, occurred along the San Andreas fault in the Santa Cruz Mountains of California. The epicenter was located 16 km (10 miles) northeast of Santa Cruz and 100 km (60 miles) south of San Francisco, California. This large; however of moderate magnitude, moderate event killed 62 people, injured 3,757 people, destroyed 367 businesses and left more than 12,000 homeless. The strong shaking lasted less than 15 seconds and it is estimated to have caused more than $7 billion in damage.

The Loma Prieta earthquake affected the performance of almost every type of lifeline system in the San Francisco and Monterey Bay areas. The most significant were in transportation, especially the loss of the Cypress Street Viaduct on Interstate 880 and the collapse of a linking span on the San Francisco-Oakland Bay Bridge. The more significant damage to pipelines occurred in the Marina and south of Market Street areas of San Francisco, areas along the the San Francisco Bay in Oakland and Alameda and areas along the San Lorenzo River in Santa Cruz. Underground pipelines were affected in unstable ground areas, especially subjected to liquefaction. In a few cases the inability of the these pipelines to function affected the emergency response and recovery activities.

American Society of Civil Engineers

The American Society of Civil Engineers (ASCE) - Technical Council on Lifeline Earthquake Engineering (TCLEE) participated in the Earthquake Engineering Research Institute (EERI) preliminary reconnaissance survey of the Loma Prieta earthquake, October 17, 1989. TCLEE members of the Earthquake Investigation Committee investigated the performance of transportation, power, gas, communications and water and sewage facilities and prepared a report on these lifelines which was published in the Earthquake Engineering Research Institute professional journal, Earthquake Spectra, Supplement to Volume 6, May 1990.

National Science Foundation

The American Society of Civil Engineers, Technical Council on Lifeline Earthquake Engineering was awarded a grant (BCS-9011325) by the National Science Foundation (NSF) to develop a data base of pipeline failures from the Loma Prieta Earthquake, October 17, 1989. The purpose of the project is to preserve information on the location, pipe size, pipe material and type of pipeline failures for the engineering, geological and seismological community and as well as future seismic risk and lifeline researchers. The resulting data base will be available in a public depository, in disk format for personal computers, to the engineering and scientific community at the cost of duplication.

OBJECTIVES

Functional seismic performance of pipelines is necessary to permit the emergency response and recovery of a community after an earthquake. Water supply is necessary for public fire fighting purposes, sewer pipelines are necessary to carry waste

away and prevent a situation of pollution which could cause illness or disease to the public and gas supplies are critical for heating and cooking for essential facilities such as hospitals. Gas and liquid fuels are essential for operation of emergency vehicles and equipment, and electric power generation to
provides electricity for emergency services. Also, the loss of lifeline services could result in a economic loss to business and industry.

The objectives of the project is to preserve data that may be lost, to place into a form useable by the research community and make available the data and methodology that may never be published.

SCOPE OF PROJECT

1. Establish advisory group to review objectives of the project and to determine the type of data to be preserved.

2. Collect the data on pipeline damage from agencies.

3. Convert location of pipeline damage into latitude and longitude coordinates.

4. Enter location and other statistical information into existing commercially available software into data base program adaptable for use on the personal computer.

5. Place data in disk form and methodology in a public depository available for use by future researchers.

Data Collection

In the process of contacting more than 35 water, sewage, gas and liquid fuel agencies it has become evident that a number of them have documented the location of damage to underground pipelines. Also some them have identified the size, pipe material, type of damage and other features of the damage. In contrast, there are a number of agencies who have concentrated on restoring the lifeline service and have minimized the documentation of the damage and mainly concentrating on the cost of repair. The cost information is necessary for completing the Federal Emergency Management Agency (FEMA), Damage Survey Report (DSR) which may permit reimbursement of the cost of repair for public agencies.

The Advisory Group recommended collecting the following basic information on the damaged pipelines:

Lifeline conveyed - Water, sewage, storm water, natural gas or liquid fuels.

Size - Diameter and thickness

Material - Steel, cast iron, ductile iron, plastic, copper, concrete (reinforced or unreinforced), concrete cylinder, etc.

Pipe or fitting - Pipe, valve, elbow, tee, reducer, etc.)

Joint type - Bell and spigot, welded (gas or electric arc), rubber gasket, bolted, mechanical coupling, butt strap, etc.

Age - Year of installation.

Type of damage - Tension, compression, bending, corrosion, etc.

Leak or Break - Leak - Lifeline continued to function with minimum loss of service. Break - Lifeline damaged with complete loss of function.

Other desirable information, but not always available.

Method of Repair - Repair clamp, welding, coupling, replace pipe or fitting, etc.

Water Table - Elevation or depth to water from ground surface.

Foundation Material - Description of bedding or trench material.

Coating and Lining - Cement mortar lining or coating, coal tar enamel lining or coating, multiwrap coating, etc.

Cathodically Protected - Yes or no.

Coordinate Conversion

Location of pipeline damage is plotted from the information provided by the lifeline agency on maps published by the United States Geological Survey (USGS) or equivalent maps which show latitude and longitude coordinates. USGS publishes maps both on 15 minute (1:24,000 scale) and 30 minute (1:100,000 scale) with latitude and longitude coordinates.

Computer Program

The computer program was developed using FoxBASE+ on the Macintosh computer. This program generates a source code that is compatible with dBASE. dBASE is available on both Macintosh and IBM or IBM compatible personal computers.

Problems

There is a significant difference in the amount of damage information available. Not all agencies keep detailed records of the type of damage
and method of repair. Most public agencies keep records on the cost of repair for possible reimbursement by FEMA. This reimbursement is not available to private agencies; however, they may be eligible for low interest loans if the earthquake impact area is declared a "disaster area" by a state and federal governments.

Some agencies either because they acquired old systems, or are newly created organization assembled from a group of smaller agencies or have no technical staff resources have limited information on their existing system.

In the Loma Prieta earthquake a flyer with a check sheet was distributed by the TCLEE Earthquake Investigation Committee (EIC) to a number of lifeline agencies after the event requesting they collect the information described above under data collection. The agencies were informed a TCLEE EIC representative would be contacting them in about 30 days to collect damage information for placing into a report on the performance of lifelines in the earthquake. In reality only a very few followed these suggestions.

In the future in domestic USA earthquakes the flyer and check sheet should be distributed; however, it should be recognized there may be very compliance.

DISSEMINATION

Provide computer program (floppy disk) and report on methodology for permanent storage in a public depository, such as, the National Information Service for Earthquake Engineering, University of California, Richmond, CA; National Earthquake Engineering Research Center, Buffalo, NY; Southern California Earthquake Center, University of Southern California, Los Angeles, CA; and other locations, and available to any researcher for the cost of duplication.

CONCLUSIONS

The data would be used by geotechnical researchers to determine damage as related to different geological conditions or seismicity. Seismic risk researchers would use the information for seismic risk analysis of lifeline systems. The would be used to improve the performance lifeline systems.

The methodology could be used on future earthquakes,collecting data in the same manner and the damage data into the same data base, so that accumulation of data can be placed in a data bank representing different magnitudes, geological and topographical conditions.

ACKNOWLEDGEMENTS

The author appreciates the representatives of the many lifeline agencies who took the time to provide the data and to Anschel Schiff the Coprincipal Investigator.

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Prepared for the Fourth U.S. - Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems, Los Angeles, CA, August 19 -21, 1991.

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BEHAVIOR OF FLAT UNDERGROUND STRUCTURES DURING EARTHQUAKES AND SEISMIC LOADS ACTING ON THEM

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ABSTRACT

Anticipating the future construction of underground urban facilities, such as underground roads and shopping centers, the authors have been conducting a study of seismic design methods for flat underground structures.

During an earthquake, all faces of an underground structure receive lateral and shear forces from the surrounding ground.

However, seismic load as defined under the seismic deformation method, a seismic design method for underground structures, is relative lateral displacement between the structure and the ground along the sides of the structure. Hence, other forces including shear force that should act laterally on the structure are ignored, and stresses occurring in the structure are underestimated.

Therefore, the same structure was analyzed using both a dynamic FEM model and a static frame model supported by Winkler springs. Then, seismic loads on underground structures that should be assumed were considered by comparing the results of those analyses.

As a result, it was confirmed that a major part of seismic load acting on a flat underground structure was shear force acting on the top and bottom slab of the structure, and that tendency became more pronounced as the width of the structure increased.

These results indicate that seismic loads assumed for the seismic design of a flat underground structure should include shear forces acting paralelly on faces of the structure in addition to lateral seismic loads on the sides of the structure as assumed by the seismic deformation method.

Introduction

In anticipation of the future construction of underground urban facilities, such as roads, streets, shopping centers, parking lots and factories, the authors have been conducting a study of seismic design methods for relatively flat and wide large-scale underground structures (hereafter called "flat underground structures").[1)-3)]

Seismic load as defined under the seismic deformation method, a today's seismic design method of underground structures, is a relative lateral displacement between the ground and the structure, due to ground deformation, acting on the sides of the structure. However, there will be not only normal components, but also shear components of loads acting on all faces (the sides and the top and bottom) of an underground structure. In a flat underground structure, in particular, shear force acting on the top and bottom of the structure is considered to constitute the major part of seismic load, because of the characteristic configuration of the structure. Hence, this study is aimed at (1) analyzing seismic load acting on flat underground structures, and (2) studying the applicability of the conventional seismic deformation method and a few improved methods.

In this paper, characteristics of the behaviors of a flat underground structure and the surrounding ground during earthquakes are discussed, and influences of the construction depth and the width-height ratio on various stresses occurring in the structure are investigated through two-dimensional dynamic FEM analysis[4)] and one-dimensional multiple reflection analysis[5)](hereafter called "1-D MR analysis"). Based on the responses of the ground and stresses in the structure obtained from the dynamic FEM analysis, the loads acting on the structure during earthquakes are divided into smaller elements. By comparing them with the results from static analysis of frames supported by soil springs, seismic loads that should be applied in the static seismic design of the flat underground structure are analyzed. All analyses are performed assuming the linearly of the materials.

Ground Conditions and Structure model

The following basic configurations were assumed in this study.

1. A 40 meter subsurface ground layer with a primary natural frequency of 1 Hz,
2. A horizontally stratified two-layer ground with an impedance ratio of about 1/3, and
3. A multiple box cellular structure consisting of a number of 5 meter high, 10 meter wide box shaped hollow modules arranged side by side.

These ground conditions are shown in Fig. 1. The influences of width/height ratios were then examined for structures with widths of 10, 30, 100, and 300 meters constructed at a depth of 10 meters to the foundation slab. Similarly influences of the construction depth were examined for a structure 100 meters wide constructed at depths of 5, 10, and 30 meters. It was assumed that the thickness of the walls and top and bottom slabs was 80cm, for all structures. The study of the width/height ratio is shown in Fig. 2.

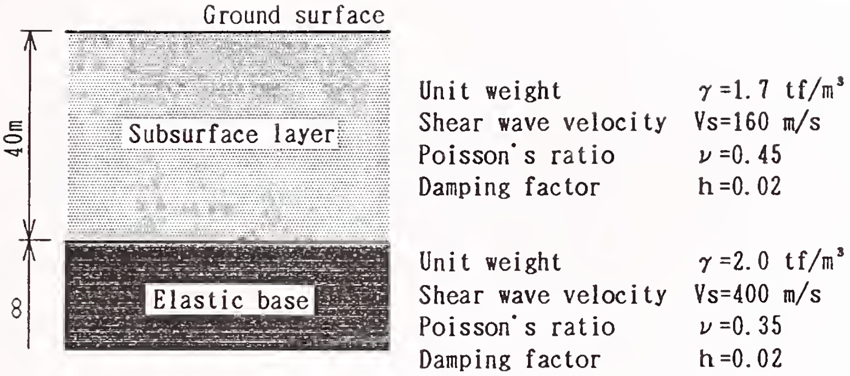


Fig.1 Model of ground

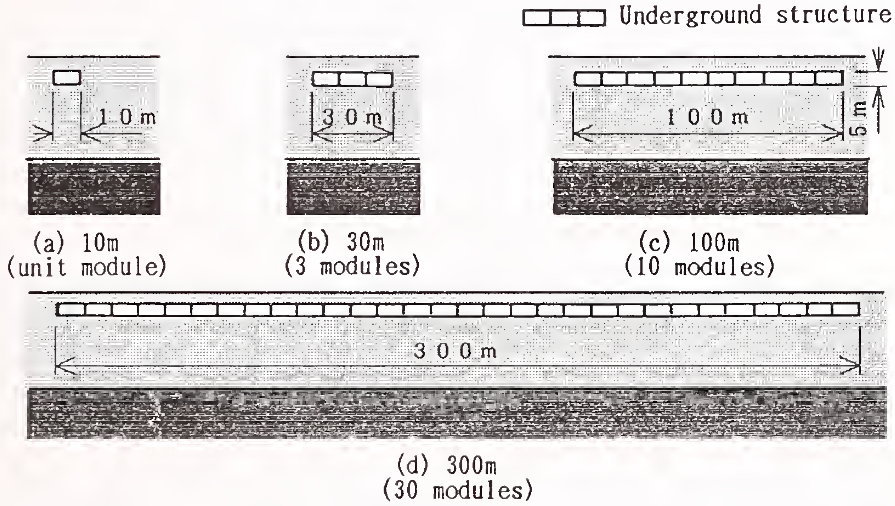


Fig.2 Models of underground structures of various width

Frequencies considered in the analyses were between 0 Hz and 8 Hz. Changes of the ground surrounding the structure under the influence of the structure were regarded as responses on the ground directly above the center of the structure (hereafter called "central ground surface"). Input seismic waves used in the analyses were El Centro 1940 NS components, Taft 1952 EW components, and Hachinohe 1968 NS components. It was also assumed that the input seismic waves SV waves as upward incident waves.

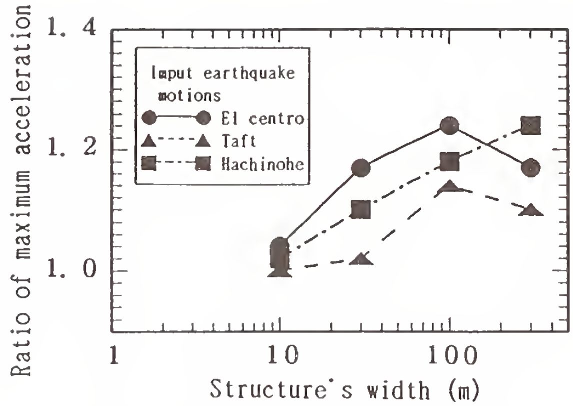
Influences of Width-Height Ratio and Construction Depth

The influences of width/height ratio

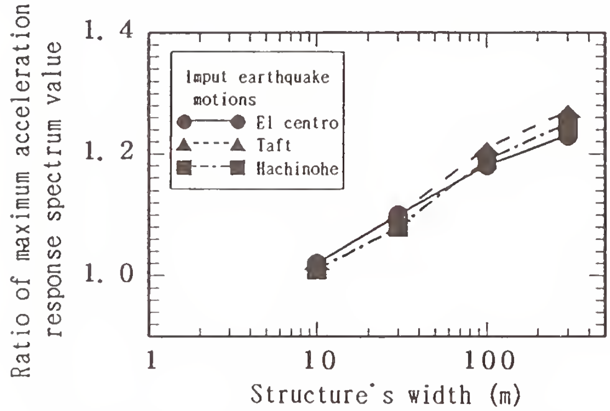
The influence of a width/height ratio were studied and results are summarized in Fig.3.

These figure indicates that all responses increase as the width increases, and that the changes in stresses are closer to the changes of maximum acceleration response spectrum values than to those of maximum acceleration values.

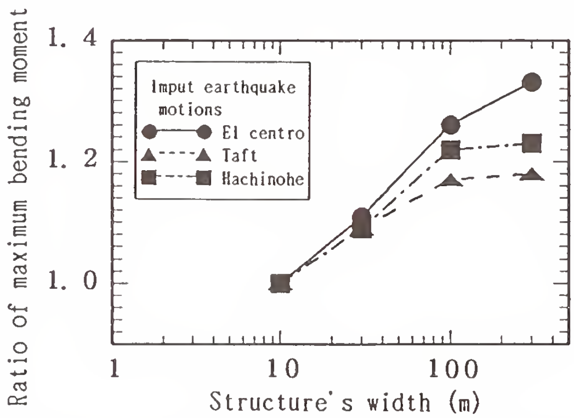
Fig. 4 shows changes of the mean value of the ratio of the acceleration response spectra of the central ground surface to those of the ground surface of the free field for the three waves, in relation to the width of the structure. Some changes can be observed in frequency ranges corresponding to the 2nd and 3rd modes for the subsurface ground. In particular, maximum responses on the central ground surface become predominant around the 2nd natural frequency range of the subsurface ground.



(a) Relationship bewteen structure's width
and ratio of maximum acceleration
(Central ground surface / Ground surface of the free field)



(b) Relationship bewteen structure's width and ratio
of maximum acceleration response sprctrum value
(Central ground surface / Groung surface of the free field)



(c) Relationship between structure's width and
ratio of maximum bending moment at the right-bottom
corner of structure to unit module structure

Fig.3 Relationship between structure's width and some dynamic responses

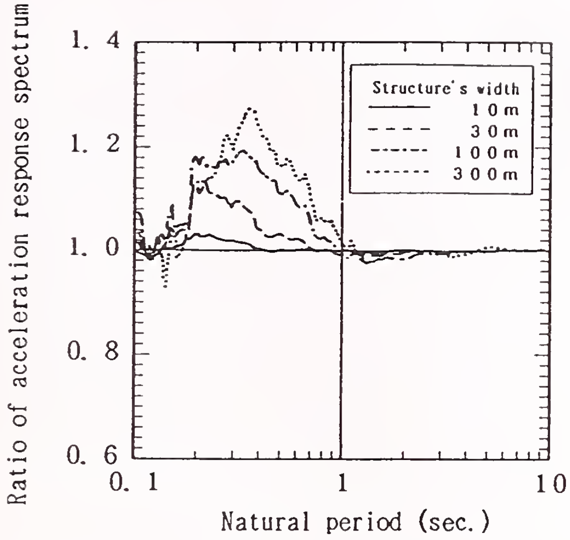


Fig.4 The influence of structure's width on acceleration spectrum(10m in depth) [Central ground surface / Ground surface of the free field]

Influences of the construction depth

Influences of the construction depth were investigated for the width of 100m. Fig. 5 shows similar changes of the mean value of the acceleration response spectrum ratio for the three waves, indicating considerable variations depending on construction depths. The fact that responses at the central ground surface become greater or smaller depending on construction depths suggests that the construction depth is an important factor.

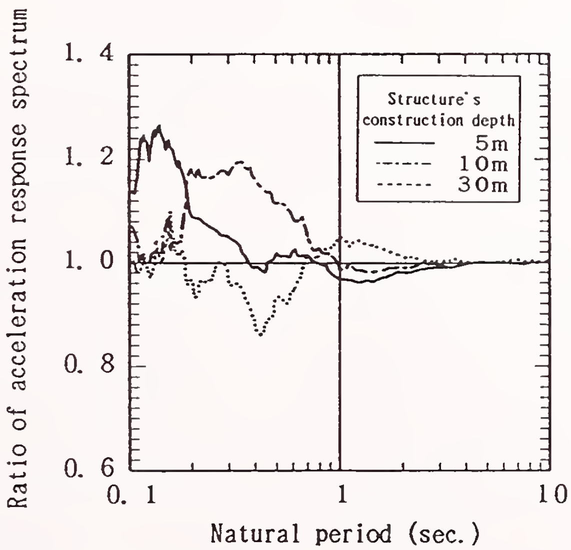


Fig.5 The influence of structure's construction depth on acceleration spectrum (100m in width) [Central ground surface / Ground surface of the free field]

Study of the Applicability of One-Dimensional Multiple Reflection Analysis Method and Influence of the Rigidity of Structure

Applicability of 1-D MR analysis

In order to determine the cause of the variation of responses at the central ground surface depending on the widths and depths of the structure, an assumed, 100m wide structure was replaced with a layer with the apparent weight and the apparent shear stiffness of the structure and was subjected to 1-D MR analysis. Fig. 6 shows the transfer functions for the central ground surface obtained from FEM analysis and 1-D MR analysis for the construction depth of 5m. The FEM model was meshed so that frequencies of up to 32Hz could be considered. In the FEM analysis, marked input loss effects[6)] due to ground confinement by side walls were observed at frequencies of 14Hz and above, but the results of these analyses agreed well at frequencies between 0Hz and 8Hz. This revealed that 1-D MR analysis is an effective method for examining the behaviors of flat underground structures and surrounding grounds during earthquakes.

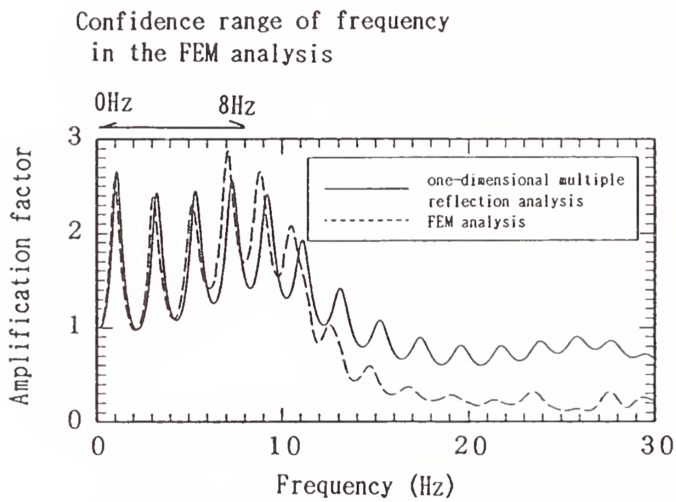


Fig.6 Comparison of transfer functions between one-dimensional multiple reflection analysis and two-dimensional FEM analysis

Influences of the rigidity of structure

Influences of the ratio of apparent impedances of the subsurface ground to those of the structure on the responses at the central ground surface were examined through 1-D MR analysis. Fig. 7 shows the ratios of the amplification factors of the transfer function for the central ground surface in relation to the ground surface of the free field, to the apparent impedance ratios of the structure to the ground. From this, it can be said that the above-mentioned amplification results from dynamic interaction due to the apparent shear stiffness of the structure of 0.62 and the impedance ratio of 0.55. This indicates that responses of the overburden soils can be reduced by increasing the thicknesses of elements of the structure and raising the apparent impedance of the structure to a level higher than that of the subsurface ground.

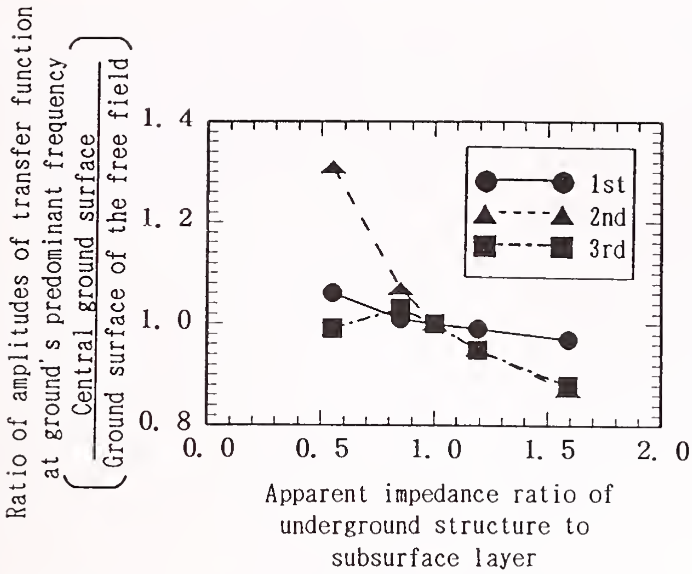


Fig.7 Influence of apparent stiffness of structure on dynamic response characteristics at the central ground surface

Determination of the designable structure and dynamic 2-D FEM analysis of a soil-structure system

Determination of designable structure

With the structures considered earlier, if they were to be constructed at deep levels, dynamic responses of soil above those structures would be amplified and substantial shear reinforcement needed to be considered at the design stage because of the thinness of elements of the structure in relation to the width of the box module. Therefore, analyses similar to those performed for ordinary structures could be conducted by keeping the apparent impedance ratio at 1.0 or above or modifying the structure so that it could be designed ordinarily as under ordinary stationary load. The modified structure was a multiple cellular box structure consisting of a series of 5m high, 7.5m wide hollow rectangular modules set side by side. The thickness of all elements of the structure was set at 1.0m. Structures of widths of 7.5m, 22.5m, and 75m, corresponding to 1, 3, and 10 modules respectively, were considered, and the construction depth was assumed to be 10m. If the width of the structure is 75m, then the apparent impedance ratio is 1.25, indicating, according to Fig. 7, that responses of soil above the structure are smaller than those of the ground surface of the free field.

Model of dynamic 2-D FEM analysis of a soil-structure system

Fig. 8 shows an analytical model for the width of structure of 75m. Since the soil-structure system was symmetric, only the right half of the cross section was analyzed, and the degree of vertical freedom on the left-side boundary was fixed. The lower boundary are assumed a viscous boundary[7)] and the right-side boundary are assumed an energy transmitting boundary by various depth method.[8)] The semi infinity of the ground was considered, but out-of-plane dissipation of waves was not. This model was sufficiently accurate at frequencies between 0Hz and 8Hz. Through the FEM analysis, responses of the ground and responses of various stresses occurring in the structure were examined, and seismic loads acting on the structure were considered.

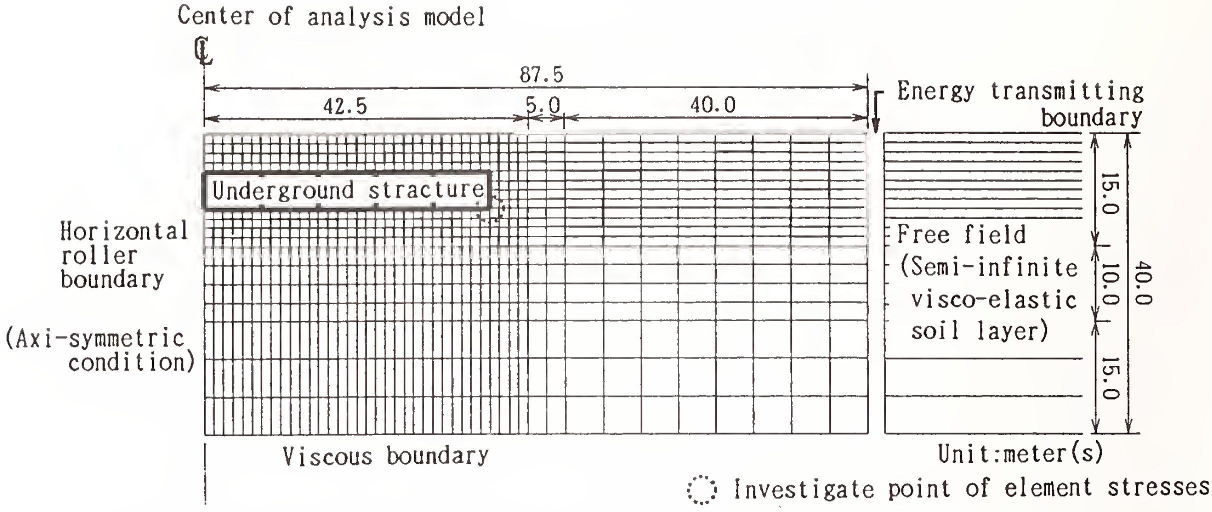


Fig.8 2-dimensional FEM model (75m in width and 10m in depth)

Static Frame Analysis under(Equivalent) Seismic Load

Objective of analysis

Seismic loads that were expected to act on underground structures during earthquakes were divided by causes and components into smaller elements. Those load elements were then assumed to act on a planar frame model having panel points supported by soil springs in the vertical and shear directions. The stresses thus obtained were compared with the results from FEM analysis, and the seismic loads that would be assumed to act on underground structures were considered.

Component of seismic load divided by factors

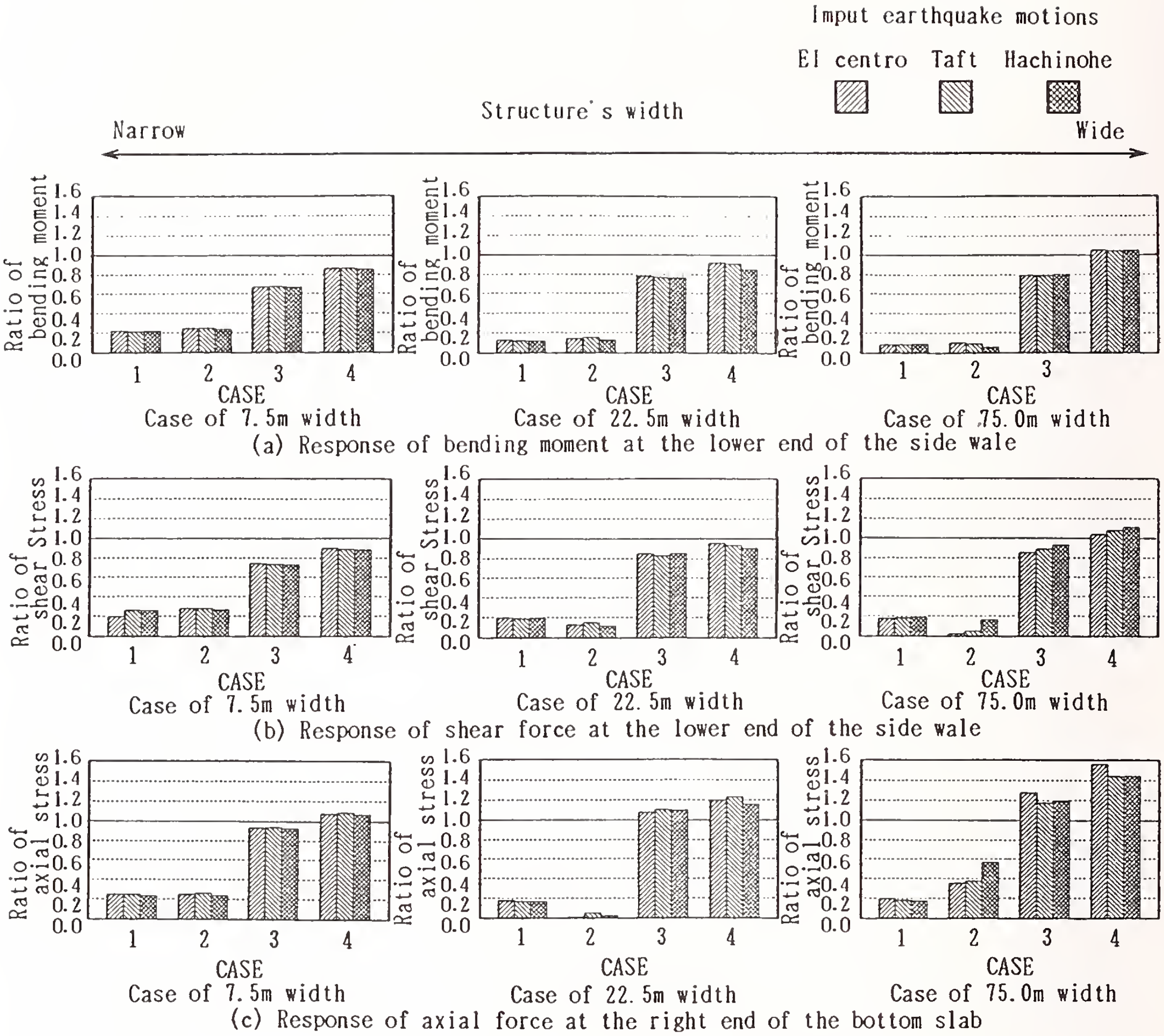
Table 1 shows loads divided by factors, and seismic loads in four cases with different combinations of divided loads. In Case 1, lateral relative displacements of the ground to the structure were assumed to work through soil springs on the sides of the structure; these loads correspond to those obtainable from the conventional seismic deformation method. In Case 2, in addition inertia force was added. In Case 3, shear forces were assumed to act on the top and bottom of the structure. For Case 4, shear forces were also assumed to act on the sides of the structure. Inertia force was calculated based on the acceleration of the ground of the free field. Shear force was calculated as summation of shear stress occurring in the ground at the depth of the structural member in question times the area to be covered by panel points. Soil spring values were calculated through static analysis using an FEM model.

Table 1 Combination of seismic loads for static analysis with frame model supported by soil springs

Factors and components of seismic loads	Combination of seismic loads			
	Case-1	Case-2	Case-3	Case-4
① Relative lateral displacement acting on the side wall	○	○	○	○
② Inertia force acting on the structure		○	○	○
③ Shear force acting on the upper and lower slabs of structure			○	○
④ Shear force acting on the side wall				○

Contribution of each factor and component of seismic load

Fig. 9 shows stresses at end corners obtained from the static analysis, in terms of ratios to the stresses obtained from the FEM analysis. Stresses obtainable from Case 1 accounted for only 10%-20% of the results of the FEM analysis. These ratios became lower as the width of the structure increased. This means that the use of the lateral displacements of the ground acting on the sides of the structure alone, will result in an under estimation of the seismic loads. There are no substantial differences if inertia forces are added to horizontal displacements acting on the sides of the structure. The stresses obtained in Case 3 could be shown to be much closer to the results of the FEM analysis by introducing shear forces acting on the top and bottom slab of the structure. For a 7.5m wide structure, about 70% of bending moments and shear forces, and about 90% of axial forces could be expressed. As the structure became wider, these ratios approached 100% and exceeded 80% when the width of the structure was 75m. It is thought, therefore, that the shear forces acting on the top and bottom of the structure constitute the major part of seismic loads on flat underground structures and have a considerable affect on the member stresses produced. It was confirmed that by introducing all loads that were considered to act on a flat underground structure in Case 4, seismic loads could be expressed as accurately as by the FEM method. It was also confirmed that this tendency did not depend on the types of seismic waves. Note, however, that axial forces were slightly overestimated.



All graphs are plotted as ratio of stress [frame analysis/FEM analysis]

Case-1:① Relative lateral displacement acting on the side wall
Case-2:Case-1 + ② Inertia force acting on the structure
Case-3:Case-2 + ③ Shear force acting on the upper and lower slabs of structure
Case-4:Case-3 + ④ Shear force acting on the side wall

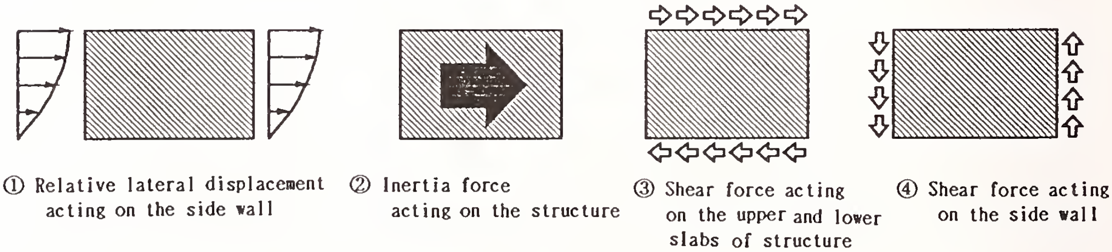


Fig.9 Element stress ratio at the right lower corner of static analyses of frame model to dynamic analyses of FEM model

Conclusions

(1) Responses of the overburden soil above the structure and responses of various member stresses of the structure increase when the apparent impedance of the structure is smaller than that of the ground. Hence, such responses can be reduced by increasing the apparent impedance of the structure.

(2) The responses of a relatively flat underground structure and its overburden soil can be evaluated by one-dimensional multiple reflection analysis.

(3) Analysis of static frames supported by earth springs in the vertical and shear directions revealed that if the consideration is limited to the horizontal displacement and inertia forces acting on the sides of a structure the stresses will be underestimated. By applying shear forces on all faces of the static frame model, the stresses can be evaluated as accurately as by FEM analysis.

(4) Major seismic loads for a flat underground structure are shear forces acting on the top and bottom slab of the structure.

Future Directions

This study revealed characteristic behaviors of a flat underground structure and surrounding ground during an earthquake, along with major seismic loads acting on the structure.

In order to establish a seismic design method for flat underground structures, which is the ultimate goal of this study, there is a need for further investigation of behavioral differences depending on the irregularity of the ground and construction depth, as well as simple methods for calculating seismic load, and methods for calculating soil spring constants.

Acknowledgments

These studies were conducted as a cooperative research into the development of seismic design method of underground structures through the technology research and development regarding use of underground space in a comprehensive research project.

We would like to express our appreciation to Dr. K. KAWASHIMA of P.W.R.I. (Public Works Research Institute) and committeeman in the working group for valuable discussion.

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SEISMIC RESPONSE OF SUPER-DEEP VERTICAL SHAFT WITH CIRCULAR CROSS-SECTION

By

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ABSTRACT

In order to clarify the deformation of super-deep vertical structure due to earthquake, the earthquake observation is carried out at a super-deep vertical shaft with a circular cross-section having a 22 m outer diameter and 99 m depth. It was found that deformation of the EW component of the shaft is predominantly first vibration mode, and that of the NS component is predominatly second vibration mode. The difference of the deformation of the horizontal component of the super-deep vertical shaft is closely related to the connected shield tunnel direction.

INTRODUCTION

There is a great demand for a development of super-deep underground urban space in Tokyo Metropolitan area. In the Tokyo Metropolitan area, subway tubes and other underground structures are crisscrossed, therefore it is difficult to construct underground structures without interfering with such existing structures. Consequently it is necessary to construct the underground structure deeper than existing structure foundations and tunnels in the ground.

A construction technology of super-deep continuous underground wall has been developed to apply to earth retaining wall for large scale excavation and wall thickens by a super-deep diaphragm wall method. In the near future, the construction of the super-deep vertical shaft as a starting shaft for a shield tunnel or ventilation of super-deep tunnel by using super-deep continuous underground wall will be increasing. However, a seismic response behavior of super-deep vertical structures is not clear so far, moreover, observed records of shafts during earthquakes are limited in number (Refs. 1,2).

The earthquake resistant design of underground structure was made by the response displacement method to compute stresses and strains on the structure due to a fundamental deformation of the surrounding ground during earthquakes. In super-deep underground structure such as a vertical shaft, the most important thing is to evaluate fundamental deformation of the surrounding ground. However, the deformations of the super-deep shaft due to surrounding ground conditions and earthquake characteristics, such as seismic wave propagation of bed rock, epicentral distance, magnitude and frequency components, have not yet been sufficiently clarified.

In order to develop rational and economical earthquake-resistant design for super-deep vertical structure, it is necessary to understand seismic response behavior of the shafts. The purpose of this paper is to clarify the deformations of the super-deep shaft and surrounding ground due to observed three earthquakes that have different epicentral distance.

OBSERVATION STRUCTURE, SITE AND SYSTEM

The observation site is located in the West of the Tokyo Metropolitan area. Figure 1 shows a vertical section in NS directional of the super-deep vertical shaft with circular cross-section, soil profiles and location of instruments.

The shaft was constructed using super-deep continuous reinforced concrete underground wall, which was constructed by a slurry-assisted excavation method and a diaphragm wall method. The shield tunnel is at the depth of 50.8 m because of the shield tunnel have to pass through under foundation piles. The depth of shaft is 99 m for shutting off underground water in the shaft. The shaft has circular cross-section with outer diameter of 22 m and the thickness of the wall is 1.2 m.

The surrounding soil layers consist of alluvial silt and sand, diluvial clay, gravel and mudstone. The values of the Shear Wave velocity are assumed by using relation between an N Value of standard penetration test and Shear Wave velocity. These values are good corespondent to seismic wave propagation time delay from observed records.

As shown in Fig. 1, seismographs were installed at three different depths (K1;GL-48m, K2;GL-24m and K3;GL-7m) in the shaft, and at four locations of surrounding ground. At surrounding ground, seismographs were installed at two different depths (at the bottom of the shaft (G1;GL-99m) and G2;GL-48m), and at two surface points (G3, G4;GL-1m). Velocity servo seismographs, which provide high signal-to-noise ratios in the measurement of long-period components, were used, and wave forms of velocity amplitudes have been being observed. Total components are 21 components for velocity wave forms and 11 components for acceleration wave forms from, mainly, the ground. These components are being observed simultaneously. The observation system uses telephone lines for reporting the outbreak of earthquakes and collecting earthquake data.

OBSERVED EARTHQUAKES

More than twenty earthquakes have been observed since the installation of the instruments. So far ten earthquakes (four of which had a maximum acceleration amplitude of 10 gal or more) with a maximum surface velocity amplitude of 0.1 kine (cm/sec) or above have been observed. Three of these with relatively larger magnitude and deference of epicentral distance, such as short epicentral distance (15 km), middle (146 km) and distant (842 km), have been taken as examples. Table 1 summarizes a characteristic of earthquakes and maximum amplitudes of three earthquakes. The location of observation site and epicenter of three earthquakes in Table 1 are shown in Fig. 2. The location of epicenter of Event No.2 earthquake is out of the Fig. 2, so, the symbol indicating this Event is just showing the relative position to the earthquake wave propagation direction.

RESPONSE OF SHAFT AND FREE-FIELD SOIL

Figure 3 shows an example of recorded EW component velocity time histories at the ground (observation points G3 and G2) and in the shaft (observation points K3 and K1) due to Event No.1 and No.2 earthquakes. Event No.1 had a short epicentral distance and Event No.2 had a distant one. In Event No.1, a short period component can be seen and a duration is very short. Moreover, the amplitude of the shaft is smaller than that of the ground. On the other hand, in the response of the shaft and the ground for the distant earthquake, which have long period component and long duration, the time histories of the shaft and the ground are similar.

Figure 4 shows a superimposed Fourier Spectra, which are obtained by the velocity time history of the horizontal component in the shaft (observation point K3) and on the ground (observation point G3) for Event No.1 and No.2. A solid line corresponds to the ground and the dash line corresponds to the shaft. Event No.2 may be regarded as an example of a distant that usually contain considerably strong power in low frequency region. The Fourier Spectra of EW and NS components of the shaft and the ground has a similar shape. Moreover, a peak frequency of the shaft and ground is almost same. On the other hand, Event No.1 shows a typical feature of the short epicentral distance. Very high frequency component is predominant in this earthquake as shown in Fourier Spectra for both shaft and ground in Fig. 4. NS component of the ground has a single dominant frequency at around 6 Hz, and EW component of the ground has two dominant frequencies at around 4 and 6 Hz. However, EW and NS component of the shaft have not so clear dominant frequency. It appears that the dominant frequency and amplitude of the ground due to the short distance earthquakes different from those of the shaft. However, response of the ground and the shaft in distant earthquake is almost same.

A vibration characteristic of the shaft can be recognized by frequency response function of the velocity at the top of the shaft with respect to input motion at the bottom. The frequency response function of the horizontal component at the top of the shaft, K3, corresponding to Fourier Spectra of input motion, G1, for three earthquakes are shown in Fig. 5. On the basis of Fig. 5, the vibration characteristics of the shaft during different epicentral distance earthquakes have the dominant frequencies of 1.0 and 2.0 Hz in the EW component and NS component. The frequency around 1.0 Hz is most predominant in EW component, and the frequency around 2.0 Hz is predominant in the NS component. The frequency of 1.0 Hz is corresponding to fundamental vibration mode of the shaft, and that of NS component is around 2.0 Hz which correspond to second mode of the shaft. It seems that the difference of the predominant frequency of the shaft depends on the connection of the shield tunnel in NS direction.

Figure 6 shows a distribution of three components of amplitude ratio for Event Nos. 1, 2 and 3, which are based on amplitude ratios at each observation point in the ground and the shaft corresponding to the maximum amplitude of earthquake motion observed at point G1. The distribution of the amplitude ratio of horizontal component of the ground is independent of difference of epicentral distance of three earthquakes, and corresponds to fundamental mode shape. On the contrary, UD component on the ground, the distribution and amplification ratio are different from the three earthquakes. The amplitude ratio of the shaft is smaller than that of the ground. EW component corresponds to fundamental mode shape, while NS component is different. This may be explained by the connecting the shield tunnel to the shaft.

CONCLUSIONS AND REMARKS

Through the observation of the dynamic behavior of the super-deep vertical shaft and its surrounding ground due to earthquake, the deformation of the shaft was investigated. The results can be summarized as follows:

1. It is found that the deformation of the shaft is affected by the earthquake epicentral distance and frequency spectra of the input motion.
2. The deformation of the shaft shows an influenced by a connecting of the shield tunnel.

The authors intend to continue seismic observation, and based on accumulated data, investigate the relationships between the magnitude of earthquakes and input earthquake motion, and the behavior of the ground and shafts during earthquakes.

FUTURE DIRECTIONS

In this paper, these are conclusions drawn from the observed three typical earthquake motions. These results show the necessity of the earthquake array observation consisting of more than two or three seismometers installed in the shaft and the surrounding ground. In order to investigate relation of deformation of the vertical shaft and surrounding ground, it is necessary to install the seismograph and measure the reinforcing-bar stress of the shaft.

In the seismic design point of view, it is necessary to investigate the deformation of the super-deep vertical shaft and surrounding ground due to dynamic analysis by axisymmetric dynamic finite element method and seismic design of a very simple cylindrical vertical structure.

Final stage of this earthquake observation of this super-deep vertical shaft is to develop rational and economical earthquake-resistant design for super-deep vertical structures.

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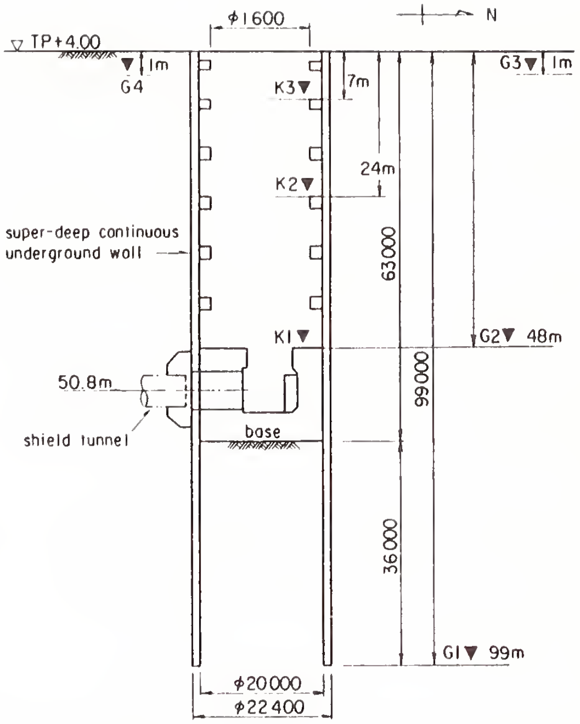
Table 1 Earthquake Event Information and Related Data

Event No.	Origin Time	M	Focal Depth (km)	Epicentral Distance (km)	Maximum Amplitude		I*
					Acc(gal)	Vel(kine)	
1	91:03:01	3.8	35	15	29.6	0.93	II
2	91:05:03	6.8	482	842	2.8	0.20	II
3	91:07:14	5.2	190	146	11.5	0.67	III

＊: JMA Intensity at Tokyo



Depth m	Name	H m	ρ t/m³	Vs m/s
-3.95	Alluvial sand	3.95	1.7	173
-23.10	Alluvial silt	19.15	1.6	153 (I) 200 (II)
-29.3	Alluvial silt	6.2	1.8	275
-34	Alluvial silt	4.7	1.9	245
-50.2	Alluvial sand	16.2	1.9	294
-67.6	Diluvial Clay	17.4	1.8	310
-78	Gravel	10.4	1.8	584 (I) 600 (II)
-99	Mudstone	21	1.8	584 (I) 525 (II)

Fig.1 Vertical Section of Shaft, Soil Profile and Location of Instrument

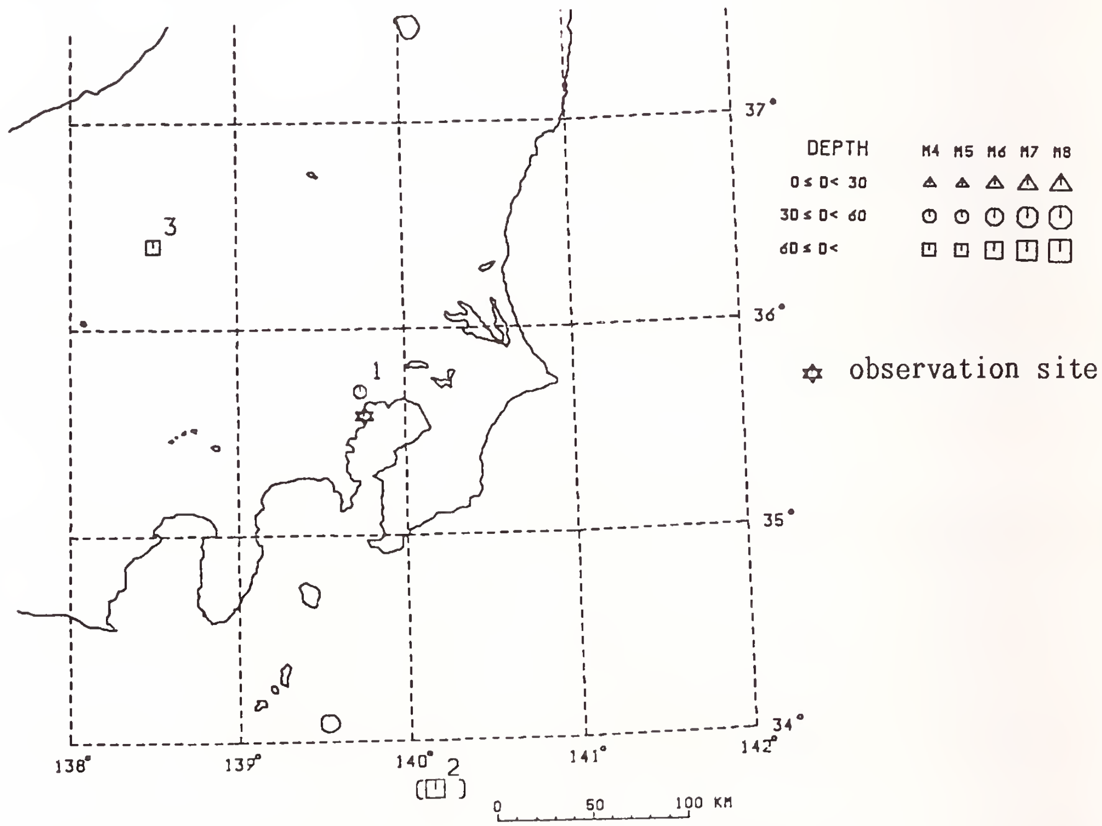
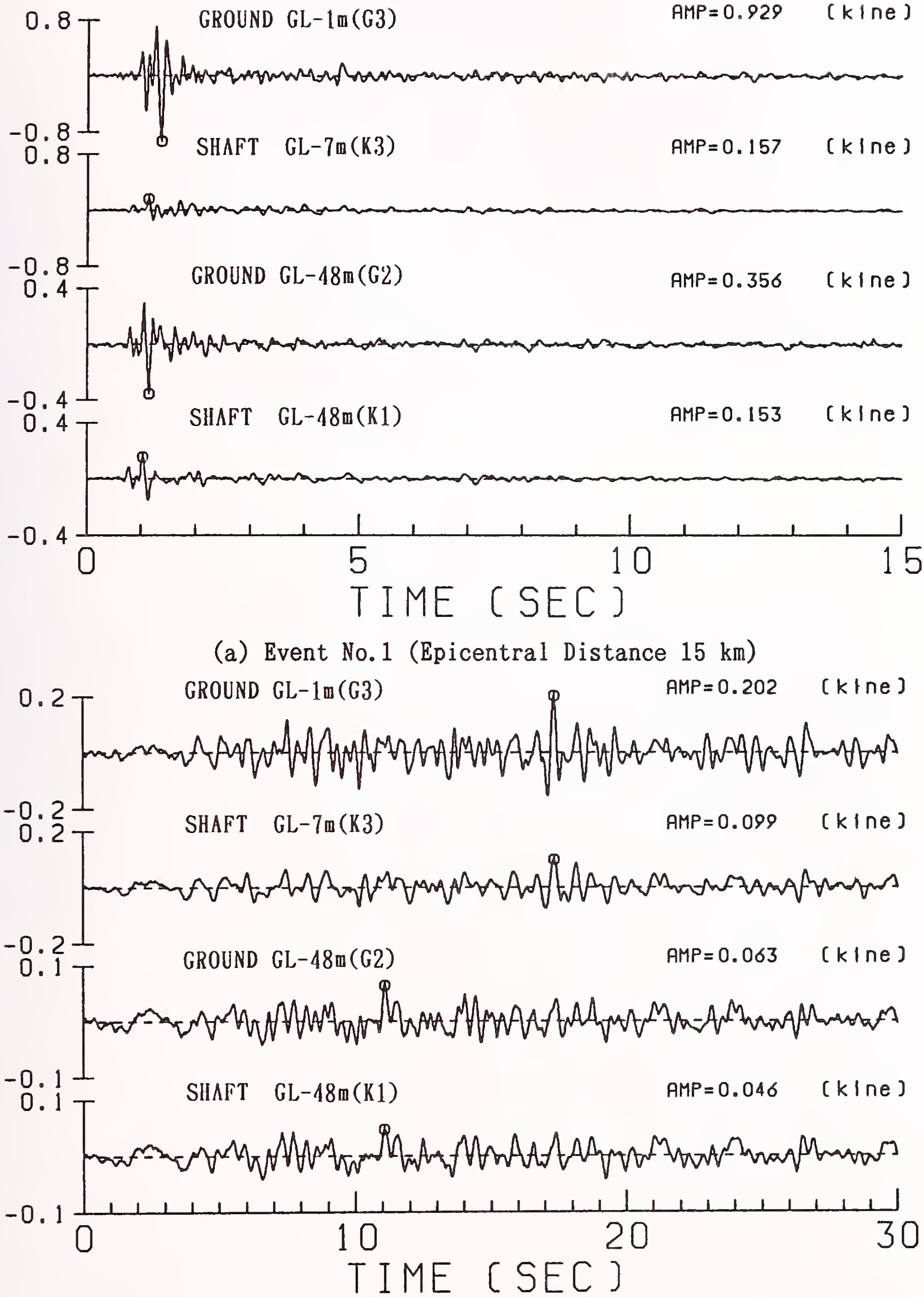


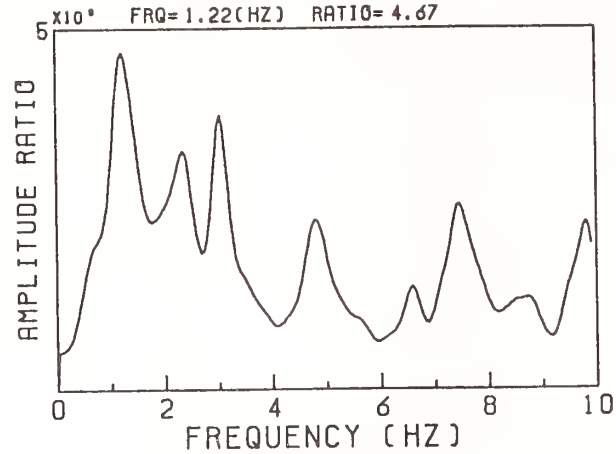
Fig 2. Location of Epicenter and Observation Station



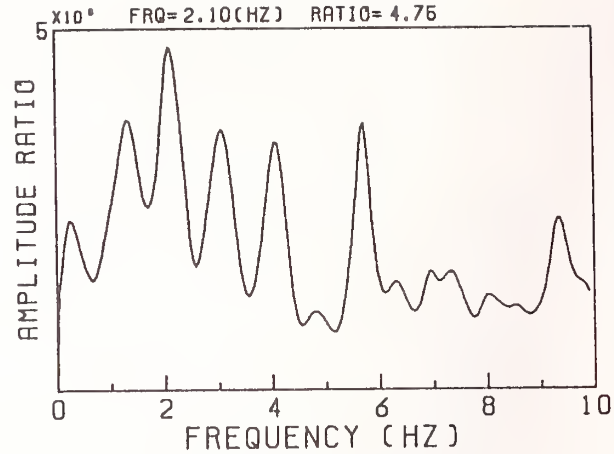
(a) Event No.1 (Epicentral Distance 15 km)

(b) Event No.2 (Epicentral Distance 842 km)

Fig. 3 Comparison of EW Component Velocity Time Histories of the Shaft and Ground

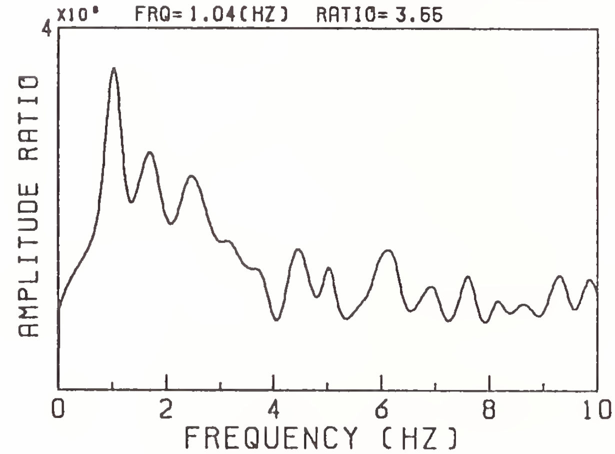


(a) EW Component

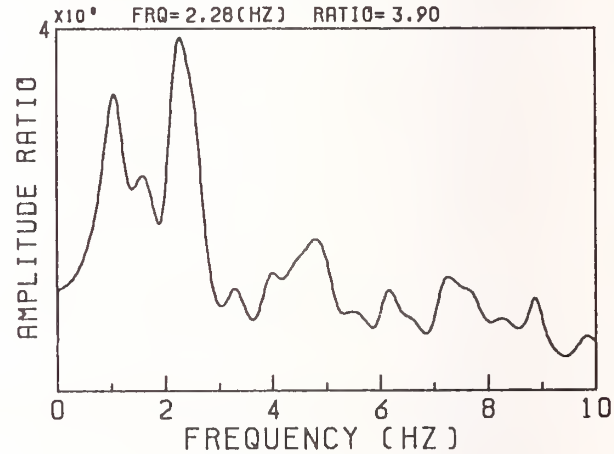


(b) NS Component

Event No.1 Earthquake (Epicentral Distance 15km)

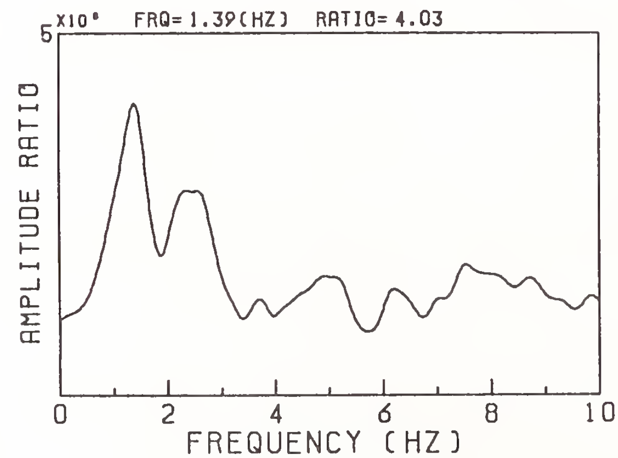


(a) EW Component

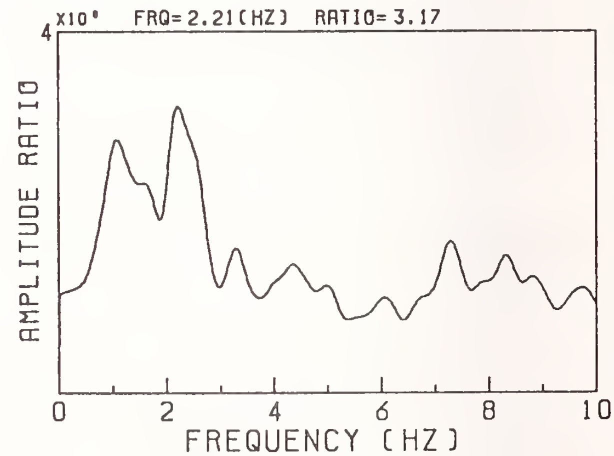


(b) NS Component

Event No.2 Earthquake (Epicentral Distance 146km)



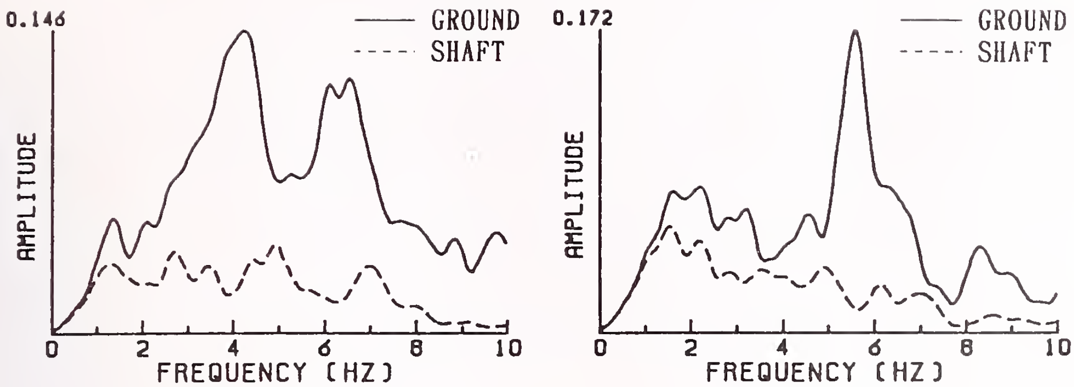
(a) EW Component



(b) NS Component

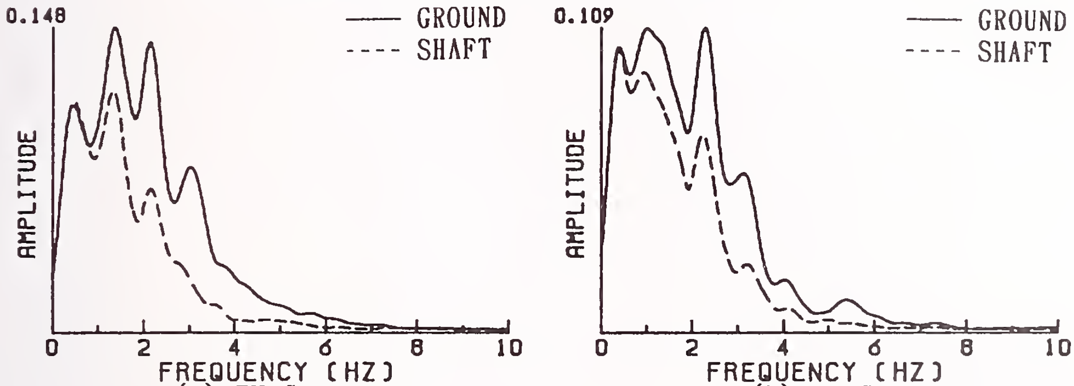
Event No.3 Earthquake (Epicentral Distance 842Km)

Fig.5 Horizontal Component Frequency Response Function of K3/G1 for Three Earthquakes



(a) EW Component (b) NS Component

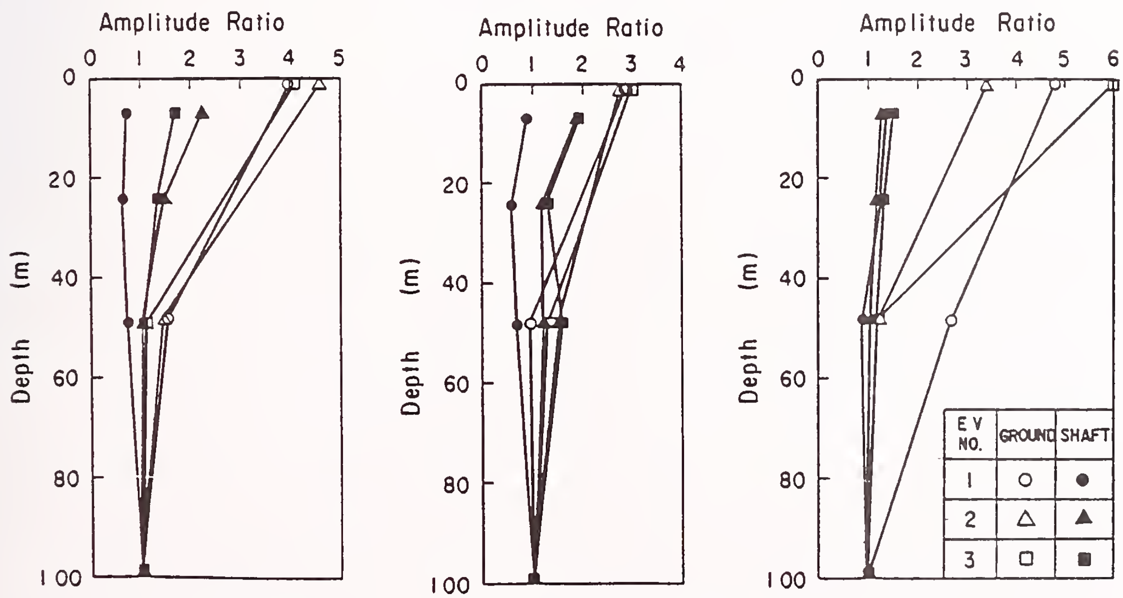
Event No.1 Earthquake (Epicentral Distance 15km)



(a) EW Component (b) NS Component

Event No.2 Earthquake (Epicentral Distance 842km)

Fig. 4 Comparison of Fourier Spectra from Shaft and Ground



(a) EW Component (b) NS Component (c) UD Component

Fig. 6 Spatial Distribution of Amplification Ratio for Three Component of Shaft and Ground for Three Earthquakes

SEISMIC DESIGN METHOD OF SHIELD TUNNELS WITH AXIAL PRESTRESSING BY MEANS OF RUBBER AND PC BAR

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Abstract

This report proposes a method to introduce axial prestress to the primary lining of the shield tunnel by using rubbers and PC bars as the method for improving the axial seismic performance. And the seismic effect of prestressing is verified by the axial alternative loading test and by the calculation of the seismic deformation method based on the analysis of the test results.

Introduction

During a large earthquake, a large axial tensile strain is introduced to a shield tunnel from the surrounding ground. This tensile strain concentrates on the ring joints so that joints plates deform plastically and the chance of water leakage is increased. As a countermeasure for this problem, a method for improving the axial seismic performance by introducing axial prestress to the primary lining of the shield tunnel, can be considered.

In this study, a method to introduce prestress to each ring using PC bars by inserting rubbers between the segments is proposed as a means for securing more improvement of the seismic performance by this prestressing, and the effect of prestressing is verified by the axial alternative loading test using a RC segment model. After that, the test results are analyzed and at the same time, based on the results of the analysis, the response of the shield tunnel is calculated by the seismic deformation method.

Concept of axial prestressing method using rubbers and PC bars

Fig.1 shows the concept of the axial prestressing method using rubbers and PC bars. Using this Fig.1, this method is explained as follows:

(1) Before introducing prestress, segment rings are assembled and at the same time rubbers are inserted in grooves provided between segment rings.

(2) The segment rings are prestressed every a ring by PC bars, and rubbers are given a compressive strain until the ends of the concrete segments come into contact. The purpose of bringing the concrete segment ends to contact under the prestressed condition,

is to receive a jacking thrust during shield driving of excavation by means of the compressive reaction of the concrete segments.
(3) When tensile strain is taken from the surrounding ground during a large earthquake, this tensile strain is absorbed by a large compressive strain which was given to rubbers precedingly by prestressing. Also, as the compressive strain is large enough, the compressive stress will remain in the rubbers even during a large earthquake so to maintain the water cut-off performance.

Outline of the experiment

Purpose of the experiment

Purposes of this experiment are as follows:
(1) To verify that a large compressive strain can be introduced to the rubbers that are between the rings using PC bars, and using a full-scale RC segment model.
(2) To verify that, even for a tensile displacement corresponding to the axial strain level (2.0×10^{-3}) which can thoroughly cover large earthquakes as well, the strain of rubbers between rings is on the compression side and presents a large compressive reaction to afford a high cut-off performance.
(3) To verify that, for similar large tensile displacements, stress of PC bars is below the yield stress to secure the safety performance of the PC bars.

Testpieces and loading apparatus used for the experiment

Fig.2 shows the concept of the shield tunnel with a big diameter which was supposed in the experiment.
Fig.3 shows a general view of the RC segment testpieces used in the experiment. Each of the RC segment testpieces was cut out from a part of the primary lining between the 2 rings of the full-scale shield tunnel in Fig.2. The length of the segment in the axial direction and the thickness of the primary lining were set to be 1500mm and 650mm, respectively the same as those of the supposed full-scale shield tunnel, giving consideration so the same behavior as that of the full-scale structure could be obtained against an axial load.
Fig.4 show the front view of the loading apparatus.
Table.1 shows the specification of rubber, PC bar and testpiece concrete used in the experiment.

Loading conditions

Loading conditions were given based on the following concepts:
(1) In prestressing, prestress of 17tf per PC bar (57.4tf of yield load) is introduced and, concrete ends are brought to contact by compressing a 38.5mm thick rubber by 8.5mm (5.7×10^{-3} in an average axial strain of the tunnel) which are adhered to grooves between rings.

(2) After prestressing, each testpiece is left still for 24hrs to converge stress alleviation of the rubber, and then 184tf of compressive force is acted and released to the testpiece, so to simulate shield driving beforehand the loading test.
(3) The loading test is performed by an alternative loading of the axial tension and compression. Incidentally, the maximum load is set to be that before the yield of the PC bar.
Fig.5 shows the loading cycle.

Measurement items

Major measurement items of the loading test include (1) relative displacement of the joint, (2) rubber pressure, (3) total displacement, (4) change in prestress, (5) PC bar stress, (6) stress of tensile reinforcing bars at rubber grooves.

Results of the axial alternative loading test

(1)Load-relative joint displacement relationship
Fig.6 shows the relationship of load and relative joint displacement. According to Fig.6, a relative joint displacement 3mm corresponds to the average axial tensile strain 2.0×10^{-3} and is included within a smaller range compared with a compressive displacement introduced to the rubber by prestressing. Therefore, it can be expected that the tensile strain acting from the surrounding ground during an earthquake can be thoroughly absorbed by the compressive strain which was introduced before by prestressing to the shield tunnel. Also, from the relative joint displacement between the rings showing a near linear behavior, it is understood that the residual joint displacement is almost zero.
According to Fig.6, a fact that the relative joint displacement does not occur up to around 10tf of the tensile load can be considered, which is due to about 10tf of the prestress which was introduced as a compressive strain into testpieces at the end of prestressing. For this reason, contact of segment ends occurred around 24tf despite the target prestressing load of 34tf.
(2) Load-axial displacement relationship
Fig.7 shows the relationship of load and axial displacement. According to the figure, axial displacement presents almost the same tendency as that of the relative joint displacement between rings. From this fact, the axial displacement of testpieces can be considered to have been caused by the relative joint displacement between rings.
(3) Load-PC bar stress relationship
Fig.8 shows the relationship of load and PC bar stress. According to the figure, PC bar stress corresponding to average axial tensile strain 2.0×10^{-3} is 5400kgf/cm^2, and it is within the the yield stress 10,500kgf/cm^2. From this fact, it can be understood that the safety of PC bars can also be secured sufficiently even during large earthquakes. Incidentally, the PC bar stress at the start of the loading test after prestressing was 2,908kgf/cm^2.
(4) Load-rubber compressive stress relationship

Fig.9 shows the relationship of load and rubber compressive stress. According to the figure, the rubber compressive stress corresponding to the axial average tensile strain 2.0×10^{-3} is 4.9kgf/cm², indicating that a high water cut-off performance can also be maintained even during large earthquakes. Also, the rubber compressive stress presents near-linear behavior, showing that it returns to the initial value when the load is returned to zero. Incidentally, rubber compressive stress at the start of the loading test after prestressing is 9.5kgf/cm².

Analysis of the results of axial alternative loading test

Outline of test results analysis

To calculate the axial seismic stress of the shield tunnel using the seismic deformation method, the relationship of the axial load and axial displacement is required. Here, based on the results of axial alternative loading tests, a simplified axial stiffness evaluation model is set for the shield tunnel with the axial prestressing. And, through comparison with the test results in terms of axial displacement, relative joint displacement, PC bar stress and rubber compressive stress, the applicability of the model is studied.

Estimation of elasticity modulus of rubber on the compression side

The elasticity modulus of rubber is estimated from the prestress-relative joint displacement relationship and the prestress-rubber's compressive stress during prestressing. First the target value of the rubber's elasticity modulus was set at 75kgf/cm², but is was not possible to estimate the influence of the rubber's shape effect to the elasticity modulus correctly, so the estimation of the elasticity modulus is done from test results.

From an average value of the rubber's compressive stress (measured by an earth pressure gauge) at a time point when both relative joint displacement ε_G and rubber's compressive stress σ_G become almost fixed (which vary only slightly) during prestressing (at 24.2tf of prestress), elasticity modulus E_G of rubber are determined as follows:

$$E_G = \sigma_G / \varepsilon_G = 10.33/0.194 = 53.2\text{kgf/cm}^2 \qquad (1)$$

Fig.10 shows the result of the above-mentioned calculation. The figure shows that the calculation value and experiment value nearly correspond with each other. Therefore, E_G= 53.2kgf/cm² is used as the rubber's elasticity modulus for the results of the following studies.

Study on axial alternative loading test

(1)Model of axial stiffness evaluation

It is assumed that the segment ends come into contact at a prestressing force Po during prestressing, and that prestress is introduced to Ppc. Also, it is assumed that the rubber's compressive stress will not increase after the contact of segment ends.

In this case, as the axial stiffness models representing the contact and non-contact of segment ends, the ones shown in Fig.11 are considered. That is, for the contact of segment ends, a model consisting of the concrete segments and PC bar springs as parallel springs. Also, for the non-contact of segment ends, a model consisting of segments and rubbers springs connected serially and PC springs are assembled in parallel with the former, and these are considered.

① Equivalent axial stiffness

An axial force Po at the contact of segment ends is determined by the following formula:

$$Po = \sigma_P A_P - \sigma_G A_G \tag{2}$$

Where, σp: PC bar stress at the start of the test
Ap: Sectional area of PC bar
σ_G: Rubber copressive stress at the start of test
A_G: Sectional area of rubber

When the acting axial force P is equal or less than Po (When $P \leqq Po$), the equivalent axial stiffness $(EA)eq^{I}$ is obtained as follows:

$$(EA)eq^{I} = E_S A_S + \frac{l_S}{l_P} E_P A_P \tag{3}$$

Where, Es: Elasticity modulus of segment concrete
As: Sectional area of segment concrete
ls: Length of segment concrete
Ep: Elasticity modulus of PC bar
lp: Length of PC bar

When $P \geqq Po$, $(EA)eq^{II}$ is obtained as follows:

$$(EA)eq^{II} = \frac{E_S A_S}{1 + \dfrac{E_S A_S l_G}{E_G A_G l_S}} + \frac{l_S}{l_P} E_P A_P \tag{4}$$

Where, E_G: Elasticity modulus of rubber
l_G: Length of rubber

② Relationship of load and axial displacement

$$\text{When } P \leqq Po: P = (EA)_{eq}^{I} \frac{\delta}{l_S} \tag{5}$$

$$\text{When } P \geqq Po: P = (EA)_{eq}^{II} \frac{\delta}{l_S} + Po\left(1 - \frac{(EA)_{eq}^{II}}{(EA)_{eq}^{I}}\right) \tag{6}$$

Where, δ: axial displacement

This relationship is plotted as shown in Fig.12.

③ PC bar stress and rubber's compressive stress

An increment of PC bar stress $\Delta\sigma p$ by loading are determined as follows:

When $P \leqq Po$: $\Delta \sigma_P = \frac{l_S}{l_P} E_P \frac{P}{(E A)_{eq}{}^{I}}$ (7)

When $P \geqq Po$: $\Delta \sigma_P = \frac{l_S}{l_P} E_P \{ \frac{Po}{(E A)_{eq}{}^{I}} + \frac{P - Po}{(E A)_{eq}{}^{II}} \}$ (8)

And a decrement of rubber's compressive stress $\Delta \sigma_G$ by loading are determined as follows:

When $P \leqq Po$: $\Delta \sigma_G = o$ ($\sigma_G = const$) (9)

When $P \geqq Po$: $\Delta \sigma_G = \frac{P - Po}{A_G} \frac{E_S A_S}{(E A)_{eq}{}^{II}} \{ \frac{1}{1 + \frac{E_S A_S l_G}{E_G A_G l_S}} \}$ (10)

(2) Specification and calculation conditions of test materials

The specification of test materials is shown in Table.1. Incidentally, prestress Ppc, PC bar stress σ_P and rubber's compressive stress σ_G at the start of the alternative loading test are as follows:

Ppc= 34.2 (tf)
σ_p= 2,787 (kg/cm^2)
σ_G= 8.8 (kg/cm^2)

Therefore, Po is determined by formulas (2), as follows:

Po= 11.29 (tf)

(3) Comparison of experiment results and calculation results

① Equivalent axial stiffness

The calculation formula of the equivalent axial stiffness is obtained using formulas (3) and (4). Table.2 shows the comparison of calculation value and experiment value about equivalent axial stiffness.

② Relationship of load and axial displacement

The load-axial displacement relationship obtained from experiment results is shown in a solid line in Fig.7. Also, the calculation values by formulas (5) and (6) are shown in the same figure using broken lines, for a comparison with experiment values. Likewise, the load-relative joint displacement relationship is shown in Fig.6.

③ PC bar stress and rubber's compressive stress

The load-PC bar stress determined from the experiment results is shown in Fig.8 using a solid line. Also, calculation values by formulas (7) and (8) are shown in the same figure using broken lines, for comparison with experiment values.

Likewise, the load-rubber's compressive stress relationship is plotted in Fig.9 using values calculated by formulas (9) and (10).

Calculation of seismic stress by the seismic deformation method

Analysis method

The seismic deformation method determines the maximum cross-sectional force generated in the tunnel, supposing that the tunnel is a beam on an elastic floor supported by a ground spring, and giving a ground displacement (supposing the distribution of axial displacement of the tunnel as a sine curve) for the tunnel depth to the spring end. A conceptual view of the analysis model by seismic deformation method is shown in Fig.13.

Analysis conditions

(1) Velocity response spectrum used for the seismic deformation method

A velocity spectrum Sv necessary for calculating the response displacement is set by the following formula, based on an acceleration spectrum S_A shown in Fig.14:

$$Sv(T,h) = \frac{S_A(T,h)\cdot T}{2} \qquad (11)$$

Where, T: Natural period, h: Damping coefficient

(2) Analysis cases

In calculating, specifications including the elasticity modulus of segments and rubber are set to be the same values in the test. With an aim to study the response characteristic of the tunnel, depth, diameter, etc. of the tunnel are varied, and 8 cases are shown in Table.3 and the analysis models in Fig.15. In this analysis, δ_G/l_G = 0.2 and lp/ls = 0.9.

Where, δ_G: Decrement of rubber's length by prestressing

Analysis results

Under the above-mentioned analysis conditions , the seismic response analysis of the shield tunnel is performed. The analysis results are shown in Table.4.

From the table, the following can be concluded:

(1) According to the analysis results performed under these conditions, PC bar stress is within the yield stress (10,500kgf/cm^2), and the safety of PC bars is secured.

(2) For stress generated in rubbers as well, all cases present compressive stress.

(3) For relative joint displacement, Case 3 with a smaller diameter and smaller thickness of lining presents the largest value (1.28mm). It is because an axial stiffness of the tunnel was smaller than ground stiffness, and that increased the transfer ratio of ground displacement.

(4) For the compressive stress of rubber inserted between segments, Case 8 with smaller rubber thickness presents the largest drop of stress at maximum tension. It is because, while relative joint displacement is slightly smaller in Case 8 than in Case 3

where it is the largest, the smaller rubber thickness increased the drop of stress in the same relative joint displacement.

(5) For PC bar stress, stress at maximum tension is the largest in Case 5 where the rubber's cross-section area increases. It is because a larger PC bar stress was required in giving the same compressive displacement to the rubber during prestressing.

Conclusions

In this study, the following can be summarized.

(1) From the results of the experiment, it is possible to absorb the axial tensile strain supposing a large earthquake by compressive strain of rubbers introduced by prestressing within the yield stress of the PC bars. Also, for axial tensile strains up to 2.0×10^{-3} or so as well, rubbers can keep their high compressive stress to afford a high water cut-off performance.

(2) For the contact of segment ends, a proposed model is consisting of the concrete segments and PC bar springs as parallel springs. Also, for the non-contact of segment ends, the model is consisting of segments and rubbers springs connected serially and PC springs are assembled in parallel with the former.

(3) From the results of the seismic response analysis by the seismic deformation method, the effect of prestressing like (1) can be verified.

Future Directions

After this report, the following will be researched.

(1) We will progress in studying the response characteristics of the shield tunnel by the seismic deformation method, and make the chart of the structural design based on the conclusion of the seismic response analysis.

(2) We will investigate the maximum water pressure at which the water cut-off performance can remain by prestressing on condition that the relative joint displacement happens by the experiment.

(3) We will verify the applicability of this proposed method by introducing prestress to the shield tunnel on site in order to make the method the practical use.

Acknowledgement

This research is conducted as a part of the Comprehensive Research Project ''Technology Research and Development Regarding Use of Underground Space (The Cooperative Research in Development of Seismic Design of Underground Structure)''. The authors wish to thank all members of this Comprehensive Research Project for their advice in the preparation of this paper.

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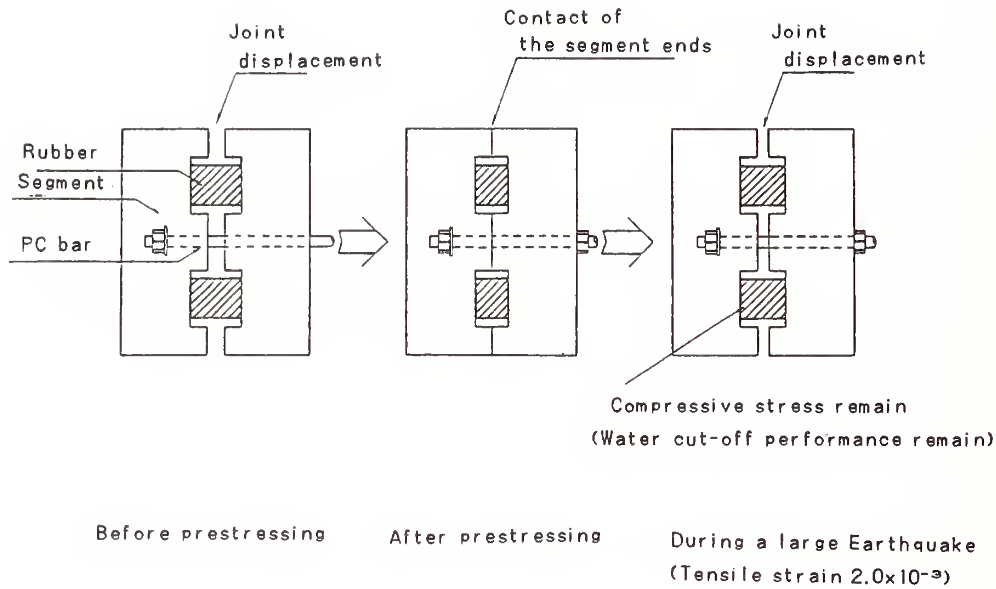
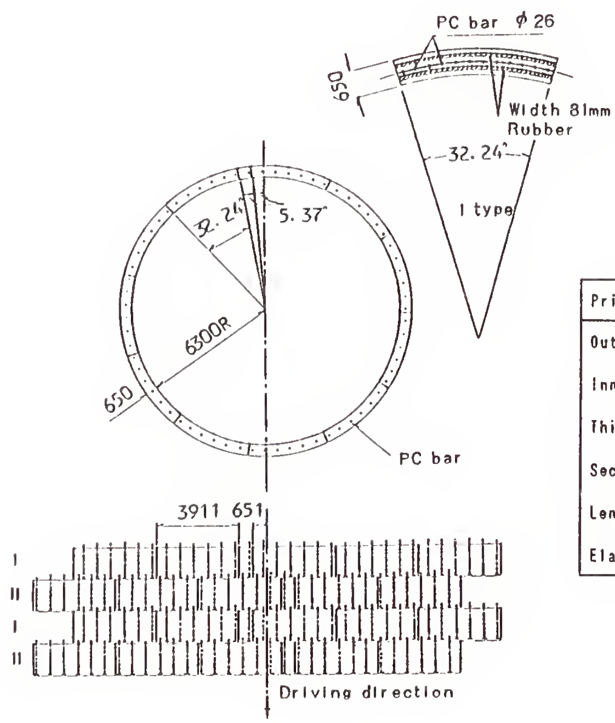


Fig.1 Concept of the axial prestressing method using rubbers and PC bars



Primary lining	
Outer diameter	13900 mm
Inner diameter	12600 mm
Thickness	650 mm
Sectional area	27.057 m²
Length	1500 mm
Elasticity modulus	3.75×10^{5} tf/cm²

Fig.2 Concept of the shield tunnel with a big diameter which was supposed in the experiment

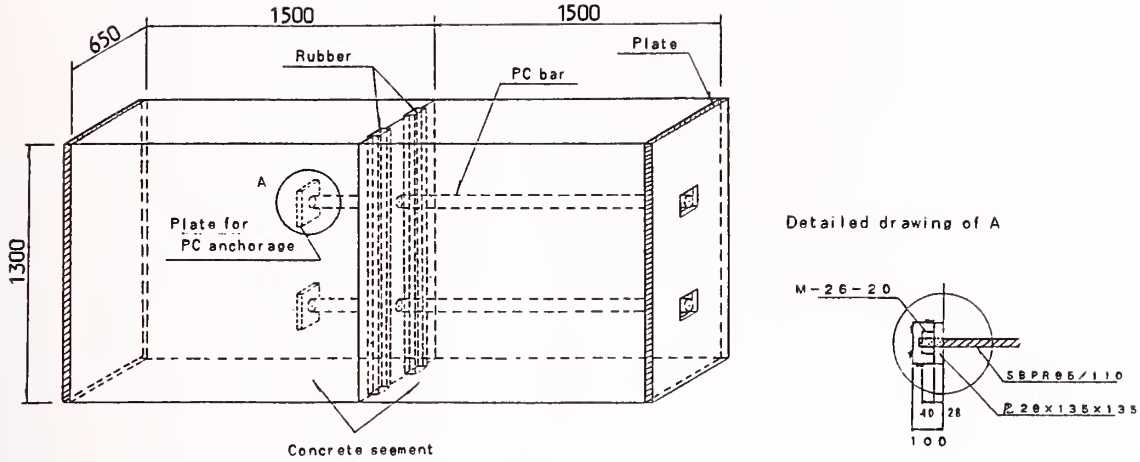


Fig.3 General view of the RC segment testpieces used in the experiment

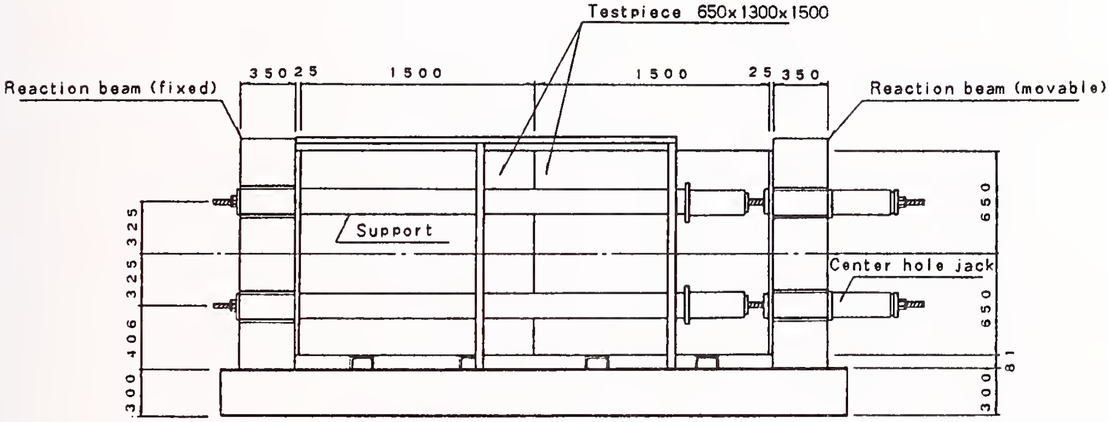


Fig.4 Front view of the loading apparatus

Table 1 Specification of materials used in the experiment

Material	Specification		
Natural rubber	Hardness	(deg.)	41
	Thichness	(mm)	38.5
	Width	(mm)	80
PC bar (type B)	Diameter	(mm)	26
	Length	(mm)	2184
	Yield stress	(kgf/cm²)	10500
	Elasticity modulus	(kgf/cm²)	2.075×10^{6}
Concrete	Compressive strength	(kgf/cm²)	434
	Elasticity modulus	(kgf/cm²)	3.12×10^{5}
	Tensile strength	(kgf/cm²)	32.4

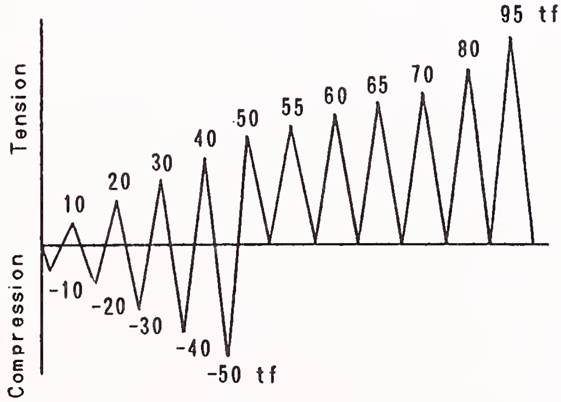


Fig.5 Loading cycle

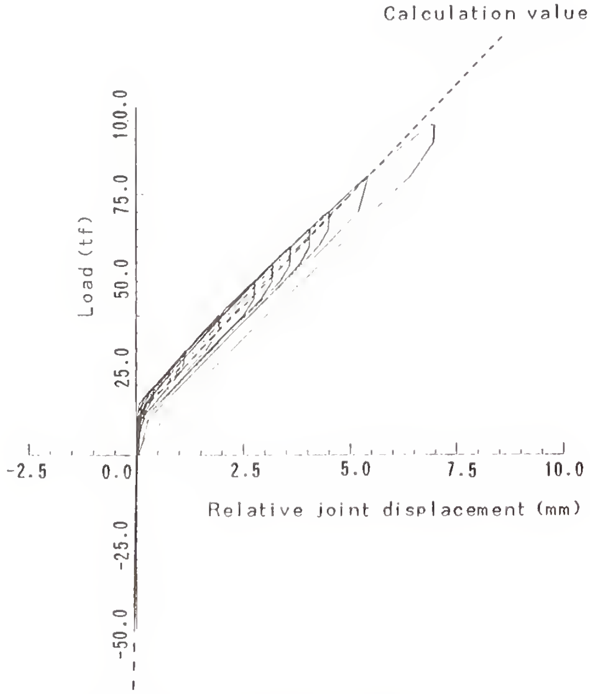


Fig.6 Load-relative joint displacement relationship

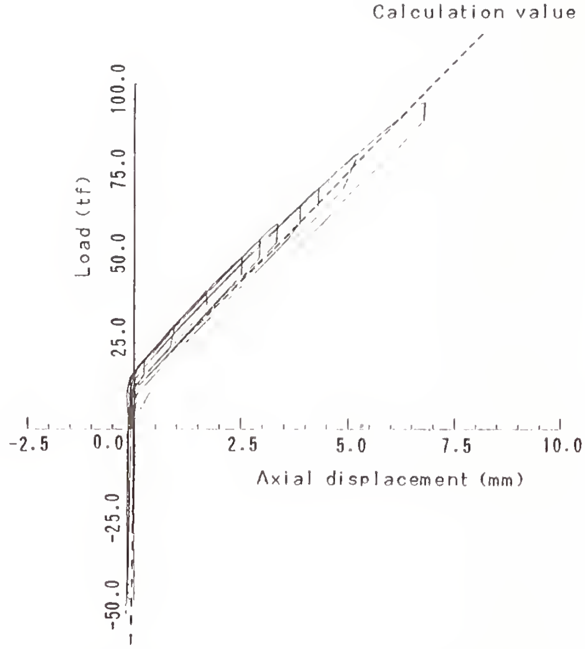


Fig.7 Load-axial displacement relationship

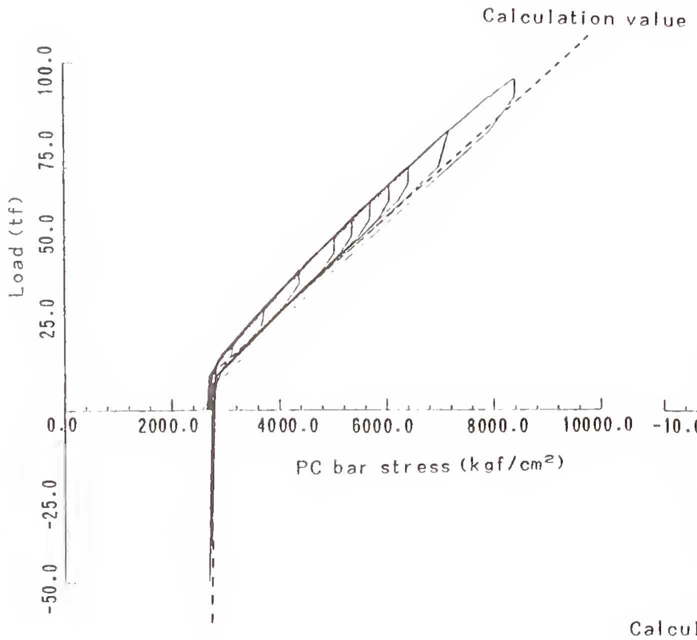


Fig.8 Load-PC bar stress relationship

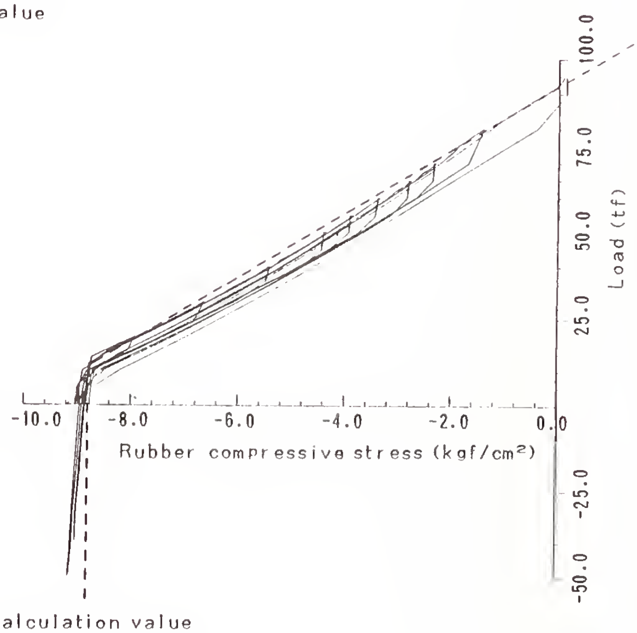


Fig.9 Load-rubber compressive stress relationship

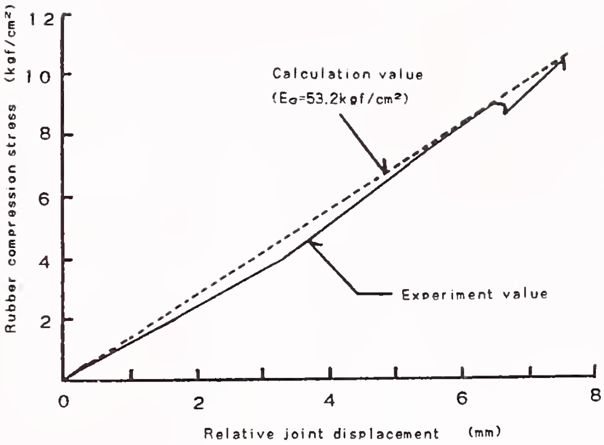


Fig.10 Rubber compressive stress
- relative joint displacement relationship

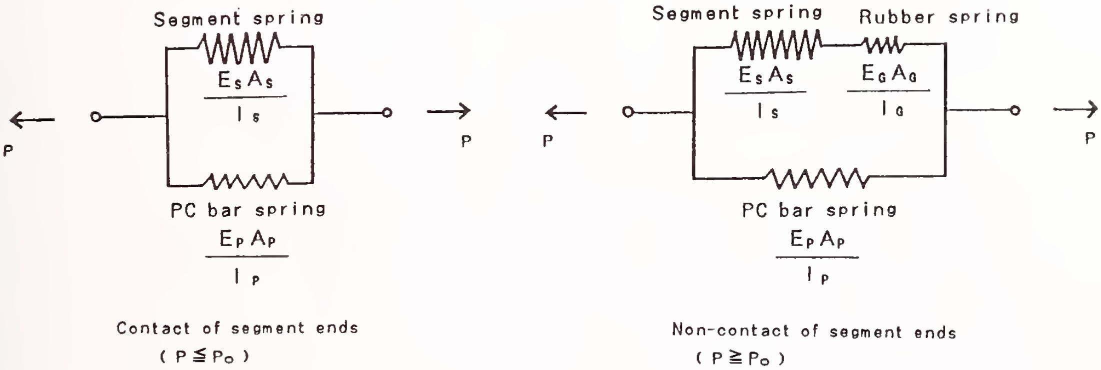


Fig.11 Axial stiffness model

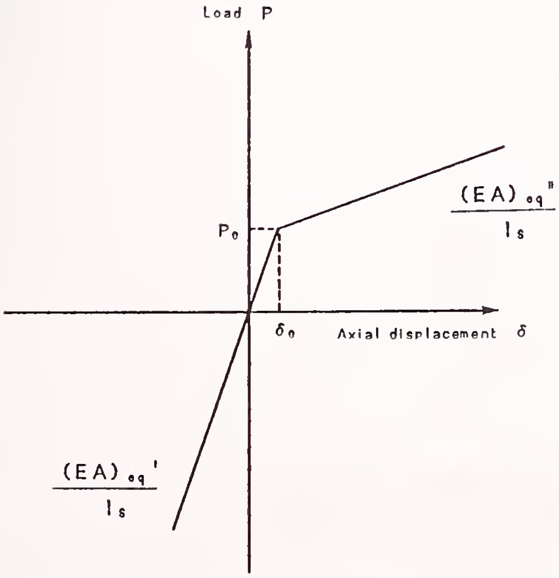


Fig.12 Load-axial displacement relationship

Table 2 Equivalent axial stiffness

	Calculation value	Experimenal value
$(EA)_{eq}'$ (Contact of segment ends)	2.67×10^6	1.69×10^6
$(EA)_{eq}''$ (Non-contact of segment ends)	3.80×10^4	3.61×10^4

(t f)

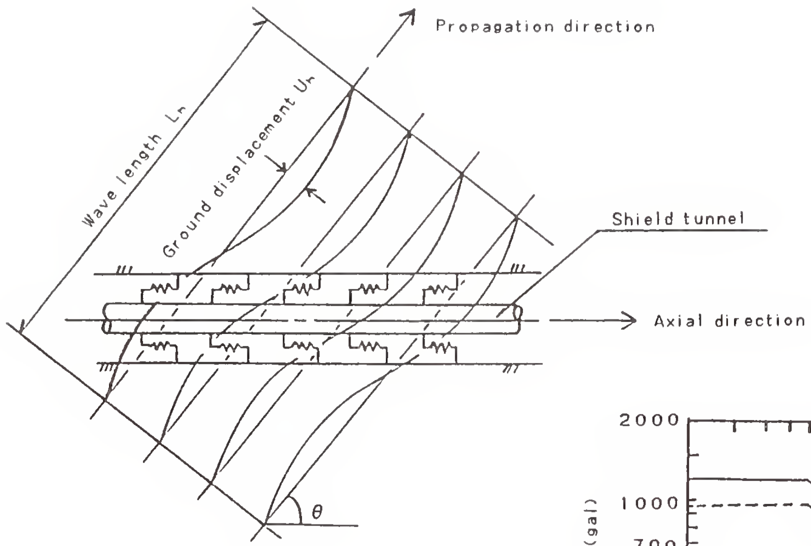


Fig. 13 Conceptional view
of the analysis model
by seismic deformation method

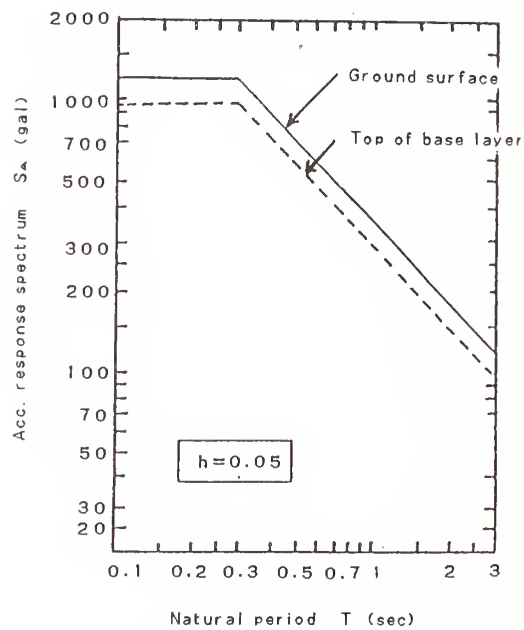


Fig. 14 Acceleration response spectral curve

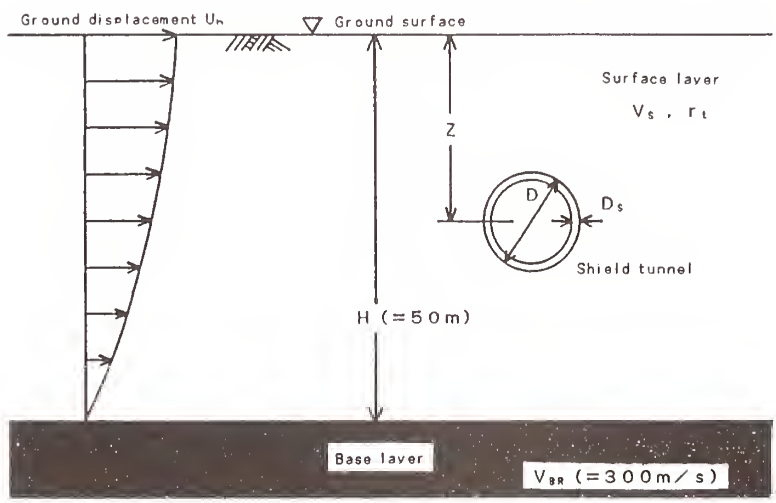


Fig. 15 Analysis model

Table 3 Analysis cases

No.		1	2	3	4	5	6	7	8
Tunnel	Diameter D(m)	16.0	16.0	6.0	16.0	16.0	16.0	16.0	16.0
	Thickness of lining D_s(m)	0.8	0.8	0.3	0.8	0.8	0.8	0.8	0.8
	Depth Z(m)	25.0	40.0	25.0	25.0	25.0	25.0	25.0	25.0
	A_P/A_S ($\times 10^{-3}$)	1.25	1.25	1.25	2.50	1.25	1.25	1.25	1.25
	A_G/A_S	0.25	0.25	0.25	0.25	0.50	0.25	0.25	0.25
	P_{PO}/P_{POO}	1.2	1.2	1.2	1.2	1.2	1.5	1.2	1.2
	Thickness of rubber l_G(mm)	38.5	38.5	38.5	38.5	38.5	38.5	38.5	26.0
Surface layer	S wave velocity V_S(m/s)	200	200	200	200	200	200	100	200
	Unit weight γ_t(t/m^3)	1.8	1.8	1.8	1.8	1.8	1.8	1.6	1.8

Table 4 Analysis results

No.	1	2	3	4	5	6	7	8
Introduced force for prestress P_{PC}(tf)	1219.4	1219.4	171.5	1219.4	2439.8	1524.3	1219.4	1219.4
Maximum compressive axial force P_I(tf)	-23504.0	-10208.0	-8925.1	-23352.0	-23351.0	-23336.0	-16412.0	-23484.0
Maximum compressive axial force P_{II}(tf)	108.1	583.9	183.9	1631.4	1385.1	1378.0	970.5	1133.0
Maximum compressive strain ($\times 10^{-4}$) ε_I	-1.945	-0.849	-5.276	-1.942	-1.941	-1.940	-1.364	-1.952
Maximum tensile strain ($\times 10^{-4}$) ε_{II}	6.777	2.948	8.542	5.969	6.557	6.741	5.926	6.690
Stress under the maximum compressive force $\sigma_S{}^I = P^I/A_S$(tf/m^2)	-615.3	-267.2	-1611.4	-611.3	-611.3	-610.9	-426.6	-614.8
Stress under the maximum tensile force $\sigma_S{}^{II} = P^{II}/A_S$(tf/m^2)	28.3	15.3	34.2	42.7	36.3	36.1	25.4	29.7
PC bar stress under the maximum compressive force $\sigma_P{}^I$(tf/m^2)	21031.0	23579.0	13371.0	8332.7	46596.0	27447.0	22390.0	21035.0
PC bar stress under the maximum tensile force $\sigma_P{}^{II}$(tf/m^2)	41160.0	32333.0	45189.0	26529.0	66236.0	47462.0	39189.0	40960.0
Rubber stress under the maximum compressive force $\sigma_G{}^I$(tf/m^2)	-106.4	-106.4	-106.4	-106.4	-106.4	-106.4	-106.4	-106.4
Rubber stress under the maximum tensile force $\sigma_G{}^{II}$(tf/m^2)	-92.4	-100.3	-88.8	-94.1	-92.9	-92.5	-94.2	-86.0
Relative joint displacement δ_D(mm)	1.02	0.442	1.28	0.895	0.987	1.01	0.889	1.00

Seismic Isolation For Underground Structures

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ABSTRACT

Underground structure such as shield tunnel, shaft, pipe line, etc. strains during earthquakes due to deformation of the surrounding ground. Ground strain caused by earthquake in soft subsurface layer or interface between two different layers with large difference in stiffness is easy to be amplified. Therefore, it should be taken a great care for underground structures to be constructed in such a ground.

First of all, this paper introduces a technique of seismic isolation applied to a shield tunnel under the sea bed. Then, the authors present a unique and yet reasonable idea to protect shield tunnel or shaft from large strain during earthquakes. The idea is to enclose a shield tunnel and a shaft entirely by a cushion material. This cushion is expected to absorb strains mobilized by the deformation of the surrounding ground during earthquakes. The effect of the cushion was evaluated analytically for a shield tunnel and its shaft.

INTRODUCTION

Use of underground space has been attracted in Japan to supply additional land. In particular, underground space in metropolitan areas is getting more precious due to high cost of land caused by concentration of population. However, since underground in metropolitan areas already contain many facilities, deeper part of the ground in these areas has to be utilized.

Underground space in metropolitan areas has been mainly used for shopping malls, parking lots, lifelines of electricity, gas, telecommunications, water supply or drainage. It has been also used for subway system or flood control system. Demand for usage of underground space is getting larger and larger.

Therefore, construction of shafts and shield tunnels will also continue to increase. Since the ground in Japan is very complicated in general, shafts and tunnels sometimes have to be constructed through an interface zone between two different layers such as soft alluvium and hard diluvium. Large strain concentration occurs in such zone during earthquakes. It would be one of the effective techniques to isolate structure from the surrounding ground by a cushion material and to absorb seismic strains by this material. This absorption of seismic strains would result in the increase of the seismic resistance of the structure.

APPLICATION OF SEISMIC ISOLATION TO SHIELD TUNNEL

In the last two decades, the authors have been engaged in seismic design of shield tunnels and their shafts constructed in various ground conditions. Through these experiences, it has been clarified to be more practical to adopt seismic isolation for these structures in stead of strengthening themselves. Following are examples of seismic isolation applied to a shield tunnel.

Seismic Isolation by Elastic Washer

When seismic wave applies to a shield tunnel as shown in Figure 1, compressive and tensile strain and stress are induced into the tunnel. Compressive stress is normally within a level of the strength of the tunnel itself.

However, tensile strain caused by the level of design earthquake is easy to exceed the limiting value of the tunnel, particularly with reinforced concrete segment. Excess of tensile strain has to be absorbed by joints of the tunnel. If joint of a shield tunnel is subjected to tensile strain, the connecting bolts and plates of the joint box are easy to be damaged. Therefore, leaking water from the joint and local failure of the tunnel would cause functional problem in the tunnel after the earthquakes.

Figure 2 shows a shield tunnel constructed under the sea bed. According to the results of the seismic analysis of the tunnel subjected to the earthquake of 150 gal at the base ground of op-200m, joints bolts longer than 50 cm are required for the tunnel. The diameter of the bolts adopted in the analysis is 24mm and this size is minimum allowable in terms of fastening the bolt in the construction. Therefore, length of 50 cm is minimum requirement. However, this length is too long to install.

If flexible segments through which longitudinal movement is free is adopted, they have to be installed at the interval of every 10 to 20 m. Since flexible segment is very expensive in general, adoption of a large number of flexible segment will result in very high cost of the construction. However, very high strain concentration occurs at the joint of shield tunnel and the shaft due to the rocking behavior of the shaft and different vibration mode of the tunnel and shaft. Therefore, flexible segment as shown in Figure 3 was installed at the joint.

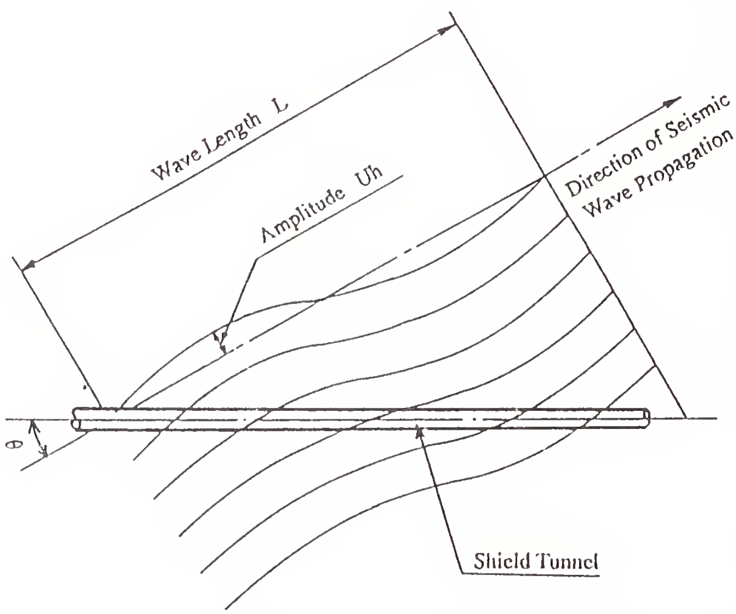
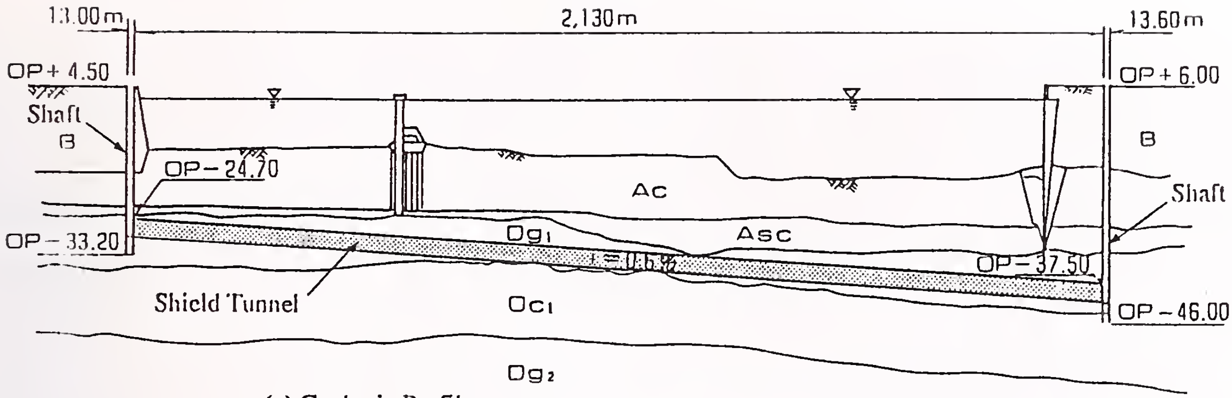
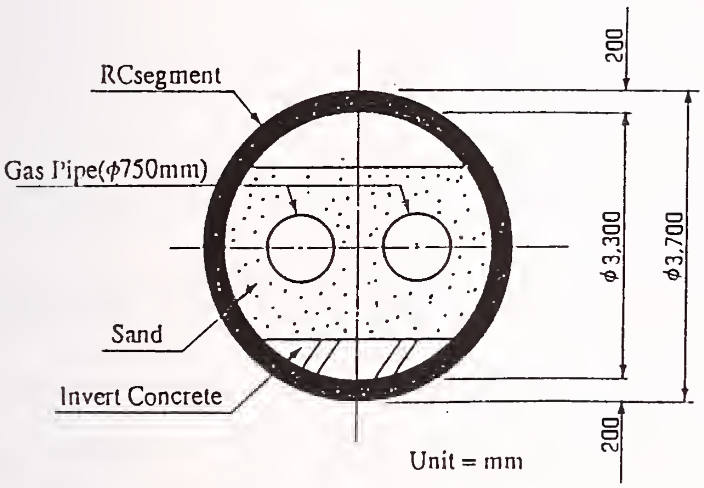


Figure 1 Ground Motion Being Used in Seismic Deformation Method



(a) Geologic Profile

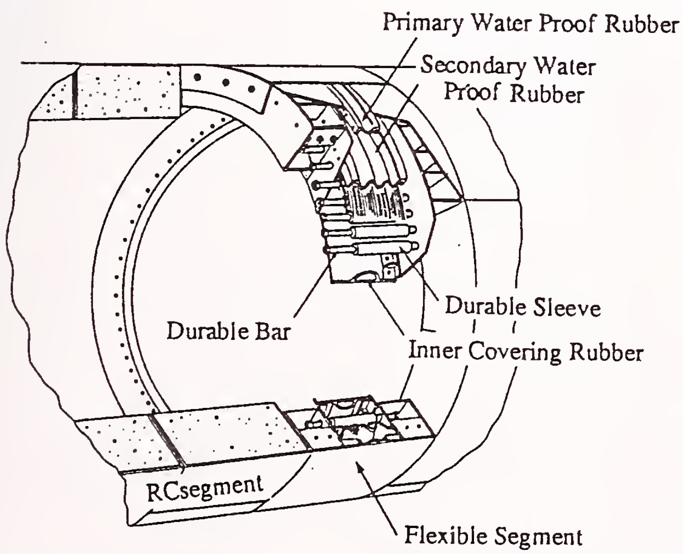


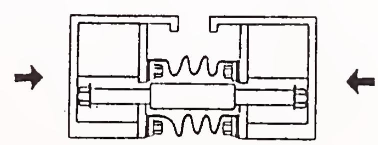
RCsegment ; B=900mm,h=200mm

Ringe joint bolt ; 16-M24

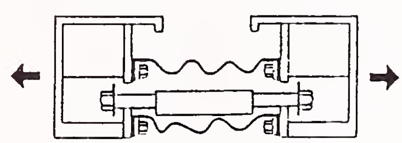
(b) Tunnel Section

Figure 2 Section of the Shield Tunnel

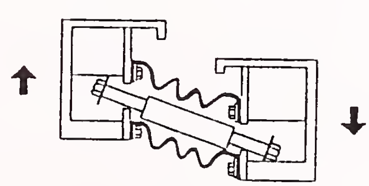




(a) Contraction



(b) Extension



(c) Shear

Figure 3 Flexible Segment

In order to overcome above problems, the authors developed new type of washer called "KRS elastic washer". KRS washer is installed between joint plate and nut of the connecting bolt as shown in Figure 4.

The KRS washer adopted for the tunnel is a ring type with φ 26 mm of inner diameter, φ 44 mm of outer diameter and 13 mm of thickness as shown in Figure 5. This washer mainly consists of epoxy resin. It is very easy to select a specified Young's modulus and yielding strength of the washer by adjusting the content of epoxy resin and the hardener. For this tunnel, 250,000 tf/m^2 of Young's modulus and 12,000 tf/m^2 of yielding strength are chosen for the washer. Figure 6 shows the stress-strain curve of the washer.

According to the results of seismic analysis of the tunnel with KRS washer installed at each joint, the bending stress of the plate at the joint is found to reduce almost proportional to the reduction of the longitudinal stiffness. In this case, the bending stress reduces to one third of the value induced in the plate without KRS washer. The tunnel is found to be safe against the design earthquake. Experimental verification was carried out to check the effect of KRS washer in reduction of the longitudinal stiffness using model tunnel. Model of the tunnel consist of 5 rings and is identical with the real tunnel except for the nonexistence of the surrounding soil.

Figure 7 shows the model and experimental set up. The tunnel is made of ductile segments. The outer diameter is 5.4 m and the height is 5.0 m. Axial tensile force was applied to the model by 12 jacks set at the top of the model. Most of the measurements were focused on the middle ring to avoid local effect which might be induced into the boundary rings.

Figure 8 shows obtained force-elongation relationship for the model with and without KRS washers. In this case, about 40% of the reduction in tensile longitudinal stiffness was observed by the use of KRS washers.

Therefore, same amount of tensile force reduction is expected at each joint of the actual tunnel if KRS washers are used.

KRS washer is also expected to be adopted in the construction of Trans-Tokyo Bay Highway. In this case, the size of the KRS washer will be the order of the size shown in Figure 9.

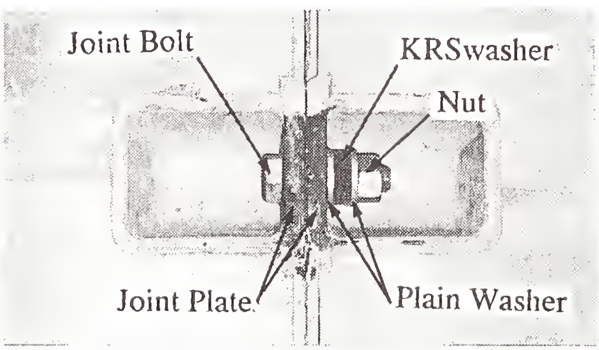


Figure 4 Seismic Isolation Using KRSwasher

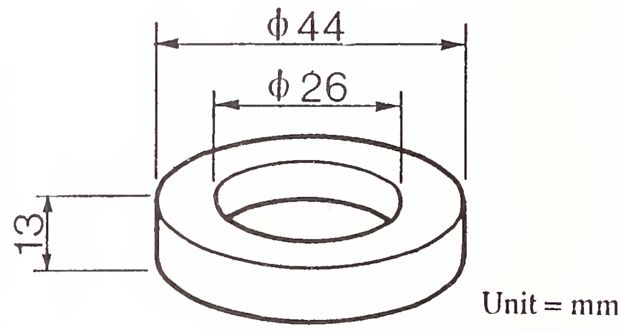


Figure 5 The KRSwasher Used for the Tunnel

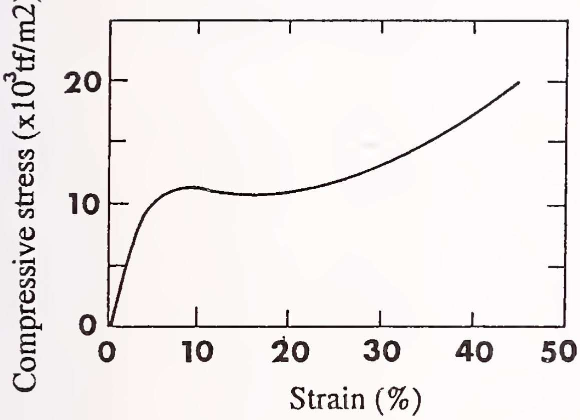


Figure 6 Stress-Strain Curve of KRSwasher

Figure 7 Model of the Tunnel

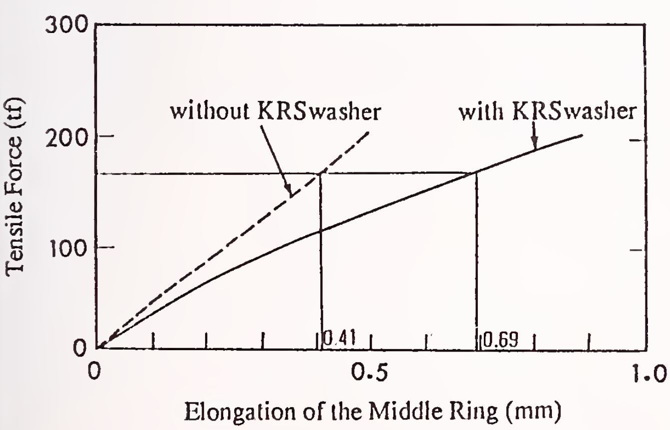


Figure 8 Tensile Force-Elongation Curve

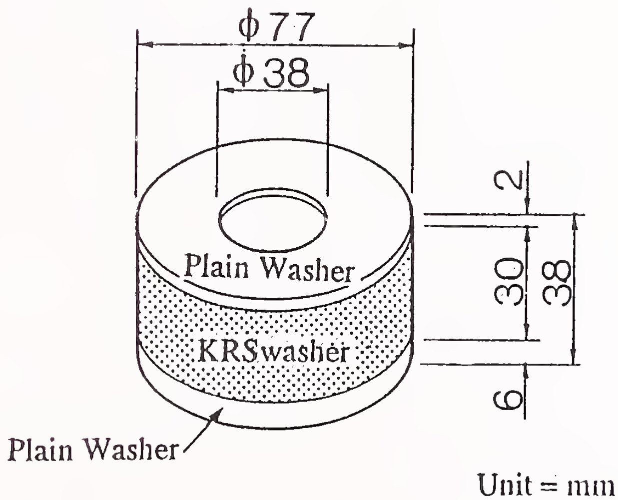


Figure 9 KRSwasher for Trans-Tokyo Bay Highway

SEISMIC ISOLATION BY CUSHION MATERIAL

KRS washer is not capable of reducing a large strain arisen at such a particular zone of the interface between soft ground and hard ground within the limiting value of the tunnel. Similar strain concentration occurs in shaft at such interface zone. In these cases, enclosure of the shaft or the tunnel by soft cushion material would be one of effective methods to release large seismic strain as shown in Figure 10.

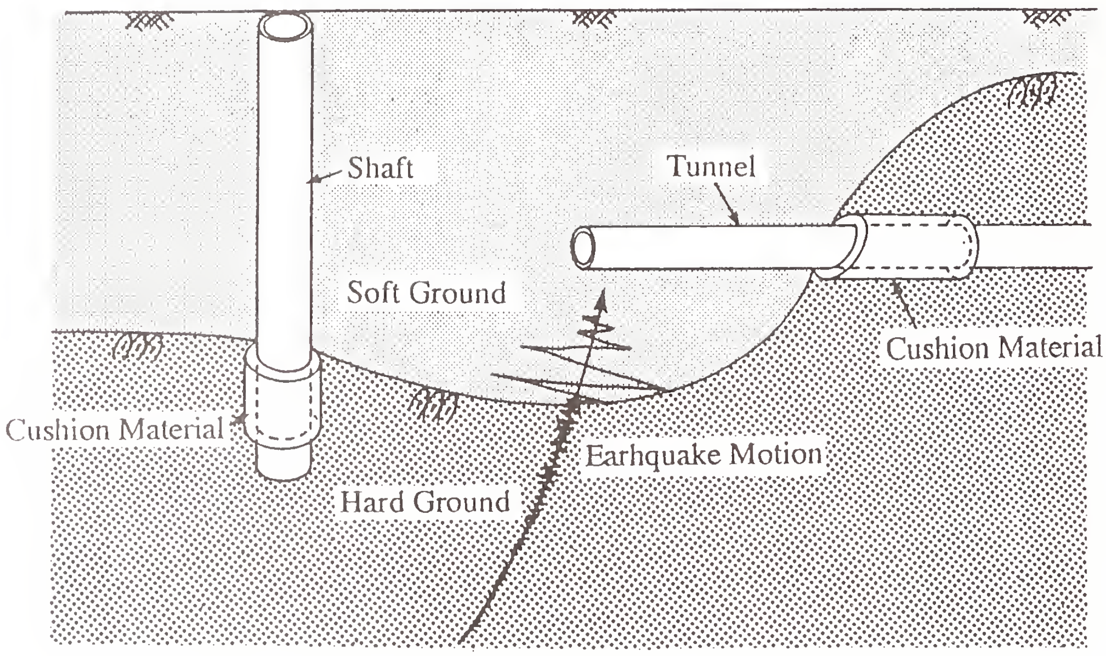


Figure 10 Seismic Isolation by Cushion Material

Case Study for Shaft

Figure 11 shows an outline of a shaft employed for the case study. The shaft of reinforced concrete is 10m in diameter, 90m long and its wall thickness is 1m. Seismic analysis is also done for the case without the isolation layer and the case where the ground is uniform to be alluvial clay. The isolation layer is 0.5m thick and its Young's modulus is 20 tf/m2. This isolation layer is covering the shaft for 20m.

Three-dimentional axisymmetric finite element method is employed for the response analysis. Figure 12 shows the finite element model. Material properties of the shaft, the ground and the cushion material are shown in Table 1. Figure 13 and 14 show input accelerogram with the maximum acceleration of 126.2 gal on the model base of GL-100m and its velocity response spectrum, respectively. In the analysis, the damping factor of each mode is assumed to be 10%.

Figure 15 shows the calculated maximum bending stresses and maximum shear stress arisen in the shaft. According to the case study, large stress concentration occurs in the shaft around the interface zone. The stress concentration factor reaches about 3.5 in this case. Bending stresses both in concrete and steel bars will surpass their limiting values although the shear can be supported easily by installing stirrups. The cushion isolation reduces the stress concentration by 20%, which is not so sufficient.

However, this case study indicates that such a cushion isolation could be come more effective if more rational installation and more effective material are developed.

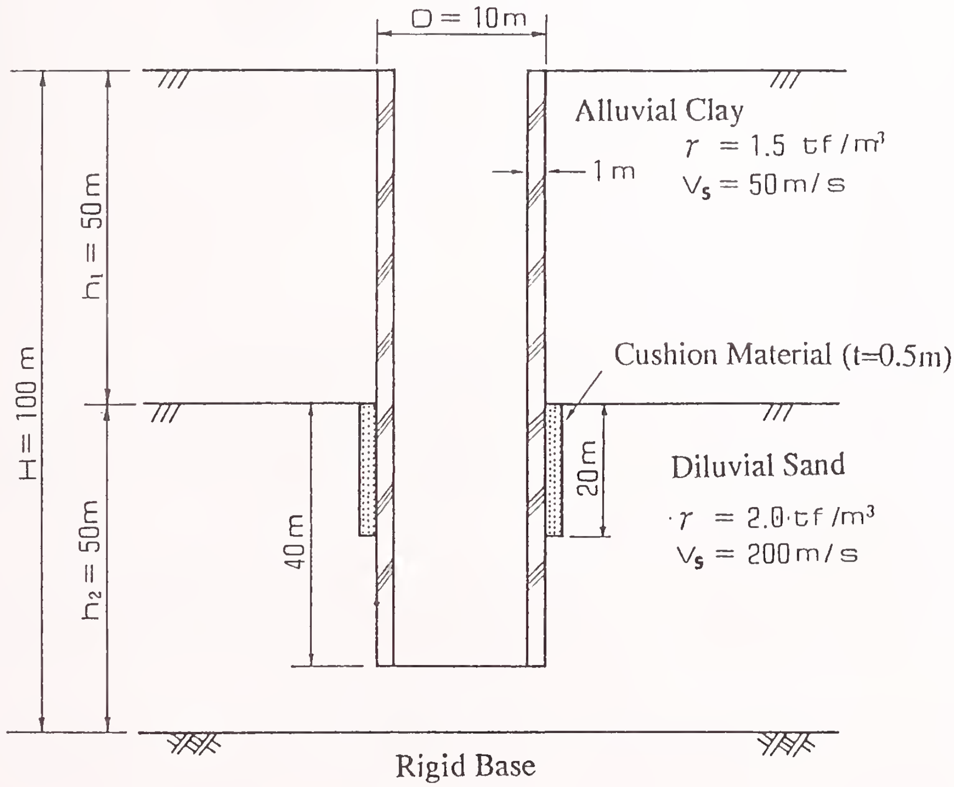


Figure 11 Ground and Shaft Profile

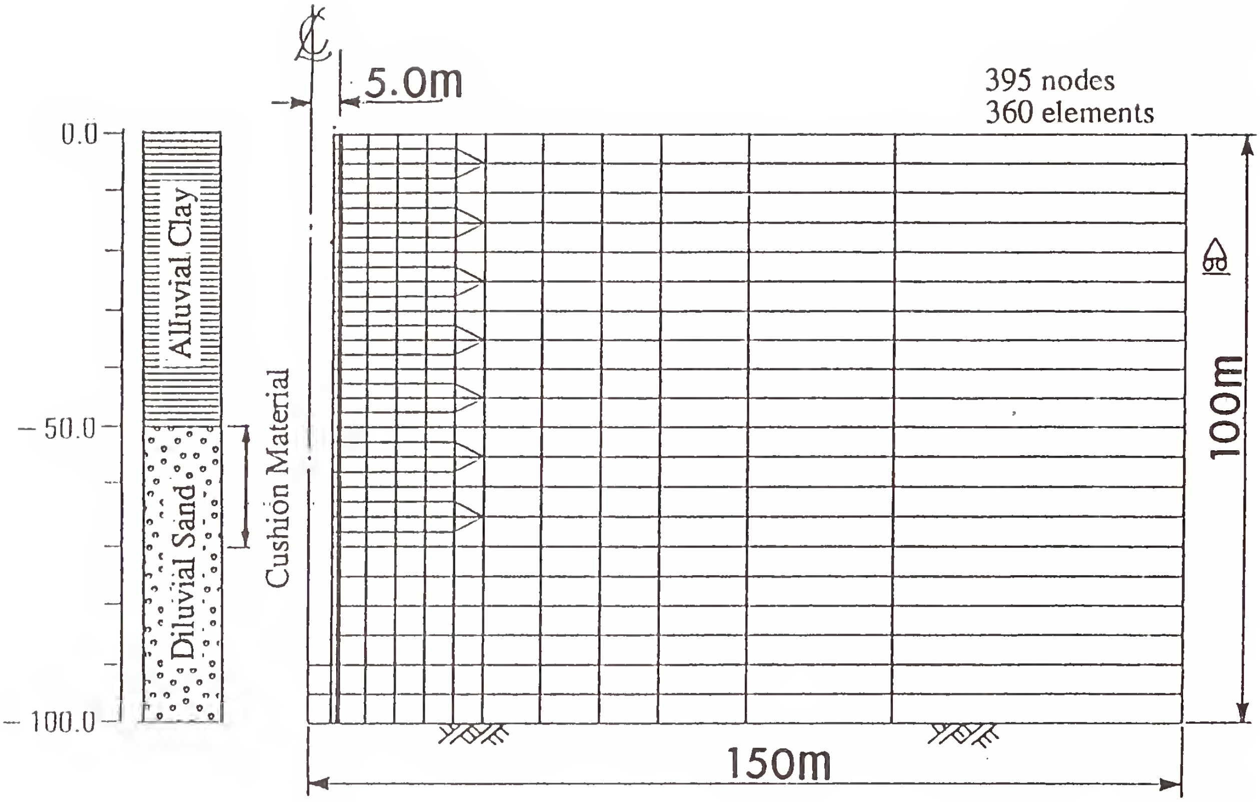


Figure 12 Finite Element Model

Table 1 Material's Properties

Properties	Alluvial clay	Diluvial sand	Shaft	Cushion Material
Unit Weight (tf/m3)	1.5	2.0	2.5	1.8
Young's Modulus (tf/m2)	1,140	24,300	3,000,000	20
Poisson's Ratio	0.49	0.49	0.167	0.49

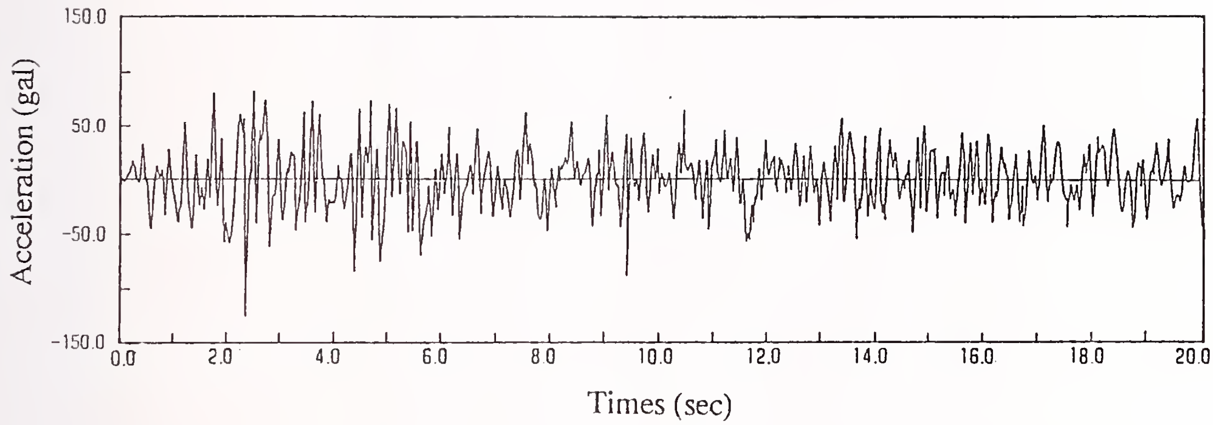


Figure 13 Input Accelerogram on the Model Base

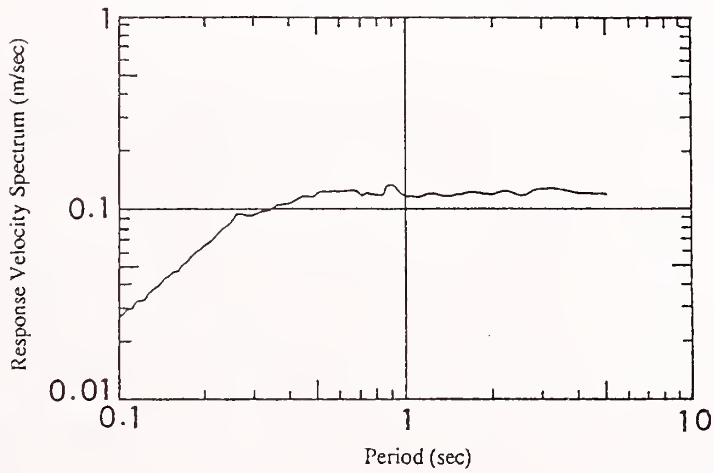


Figure 14 Velocity Response Spectrum of Input Accelerogram on the Model Base

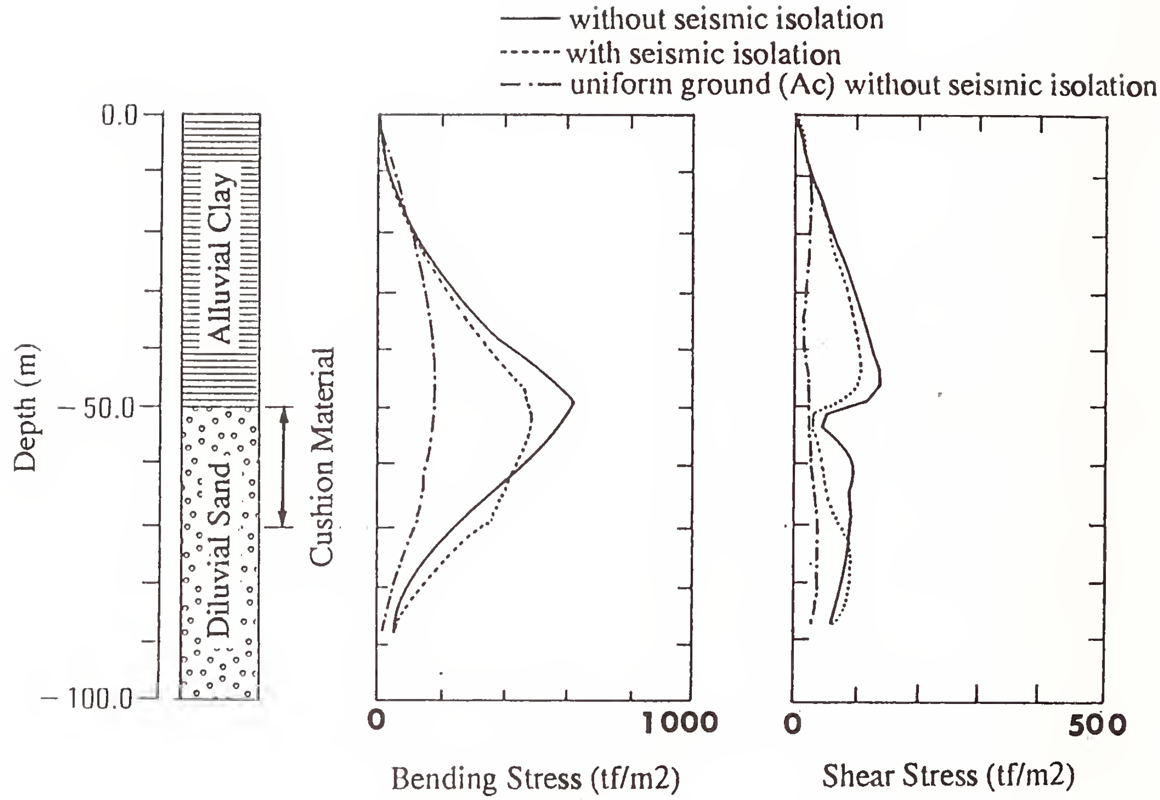


Figure 15 Maximum Bending and Shear Stress along Shaft

Case Study for Shield Tunnel

Figure 16 shows a tunnel and two different ground layers employed for the case study. The tunnel is composed of 0.4m thick reinforced concrete segments and the diameter is 10m.

The tunnel is modeled as beam elements and the surrounding ground is modeled as lumped mass springs. Interaction springs are also installed in the model to link the ground masses and tunnel beam elements . The analytical model is shown in Figure 17. The cushion material covering the tunnel for 40m in the diluvial layer is expressed by spring elements. Young's modulus of the cushion is assumed to be 1/100 of the value of the diluvial sand. Input data for the tunnel and the ground are shown in Table 2 and Table 3, respectively. Same seismic load as used in the analysis of the shaft is also employed in this study. Properties of accelerogram are shown in Table 4. The time history analysis is done by the direct integration method. Damping factor of each mode is assumed to be 10%.

Figure 18 and Figure 19 show the calculated maximum axial force and bending moment arisen in the tunnel. The cushion isolation reduces their peak values by about 30% in this case. Larger reduction effect could be possible by increasing the thickness of cushion and using appropriate cushion material.

Installation of such cushion isolation is also big problem to be solved. Figure 20 and 21 show rough idea of installation. For tunnel, cushion material made by mixing liquid agent with hardener is injected between outer surface of the tunnel and the ground through the grouting hole. For shafts, seismic isolation cushion as mat state is installed together with the reinforcing bar cage of the diaphragm wall.

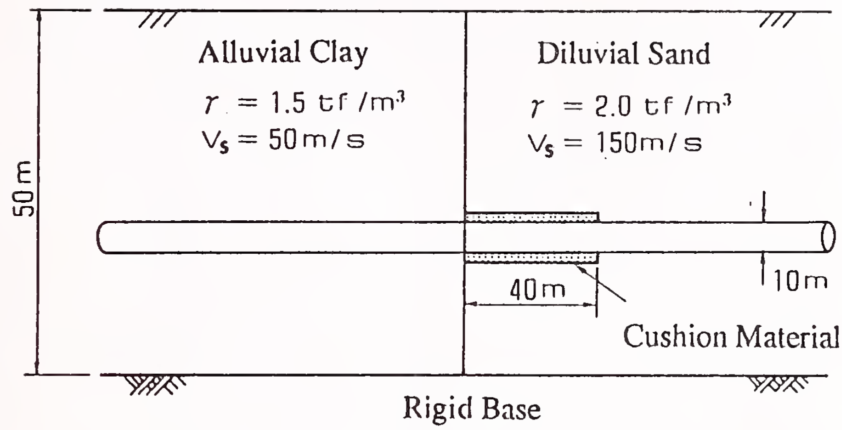


Figure 16 Ground and Tunnel Profile

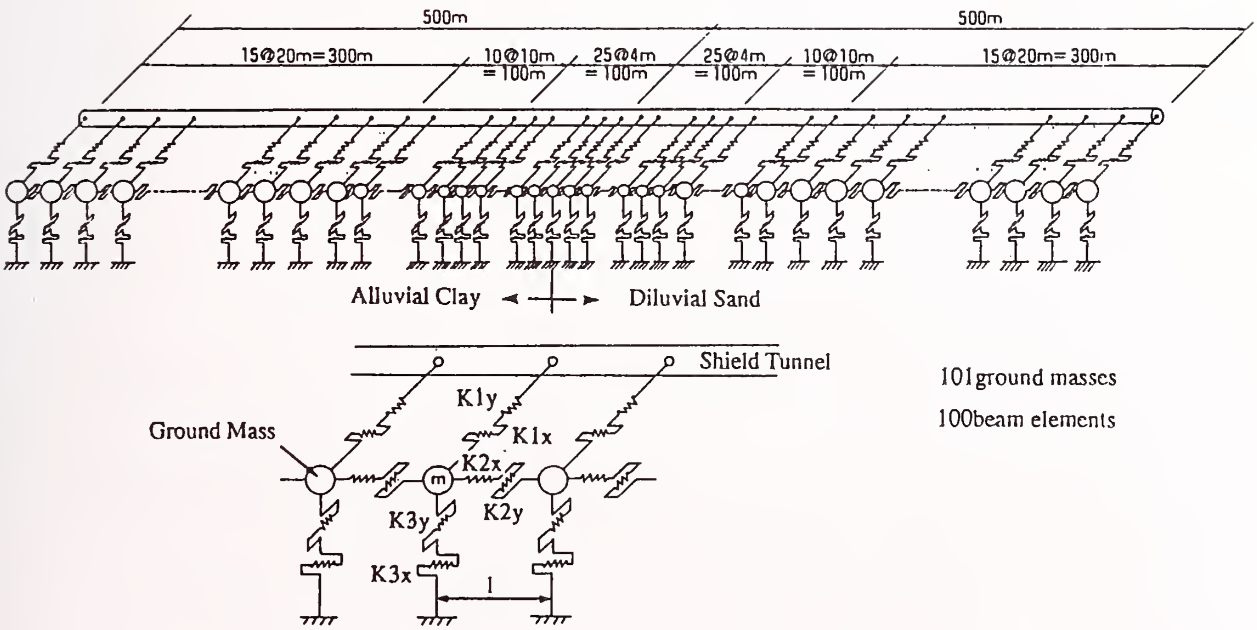


Figure 17 Analysis Model

Table 2 Properties of the Tunnel

Unit Weight (tf/m3)	2.5
EA (tf)	1,810,000
E I (tfm2)	41,800,000

Table 3 Properties of Ground Masses

Properties		Alluvial Clay			Diluvial Sand		
Interval between Ground Masses	l (m)	4.0	10.0	20.0	4.0	10.0	20.0
Mass	m (tfs2/m/m)	24.8	62.0	124.	33.1	82.7	165.
Spring Constants	K1x,K1y (tf/m/m)	4,600	11,500	23,000	55,100	138,000	275,000
	K2x (tf/m/m)	11,500	4,620	2,310	134,000	55,500	27,800
	K2y (tf/m/m)	3,880	1,550	776.	46,500	18,600	9,300
	K3x,K3y (tf/m/m)	61.2	153.	306.	783.	1,850	3,700

These parameters refer to Fig. 17

Table 4 Parameters of Input Earthquake Motion

Parameters		Axial Force Calculation		Bending Moment Calculation	
Superficial Ground		Alluvial Clay	Diluvial Sand	Alluvial Clay	Diluvial Sand
Maximum Response of the Ground Surface	(m)	0.0688	0.0228	0.0973	0.0323
Wave Length	(m)	453	325	320	230
Input Max. Acc. of the Model Base	(gal)	282	353	367	453

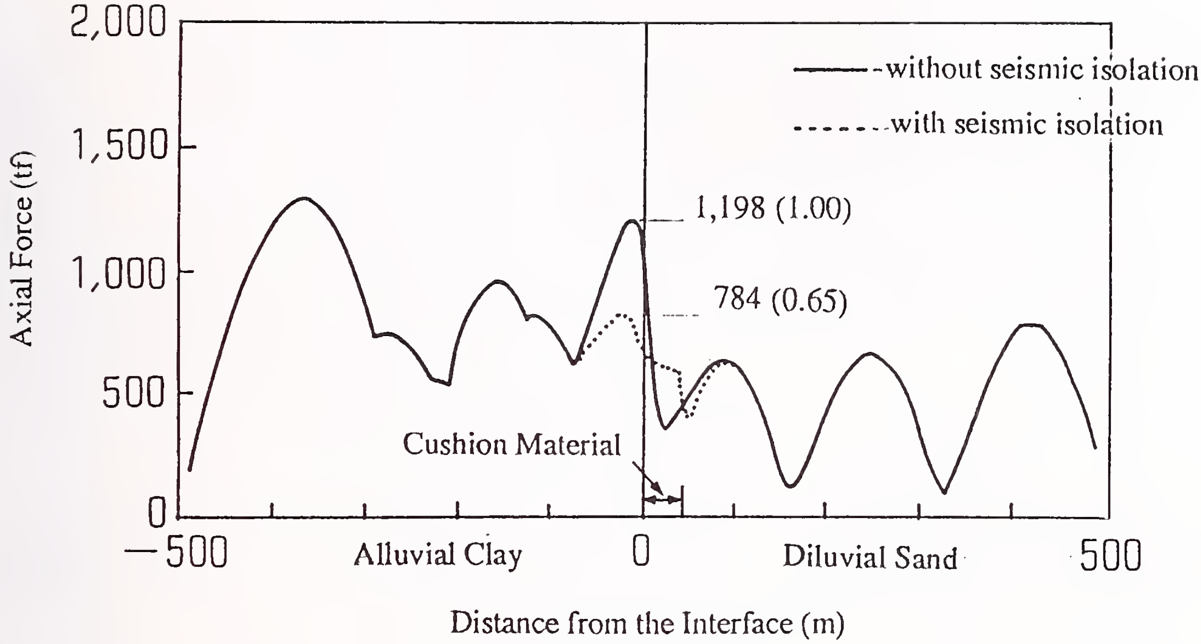


Figure 18 Maximum Axial Force of Tunnel

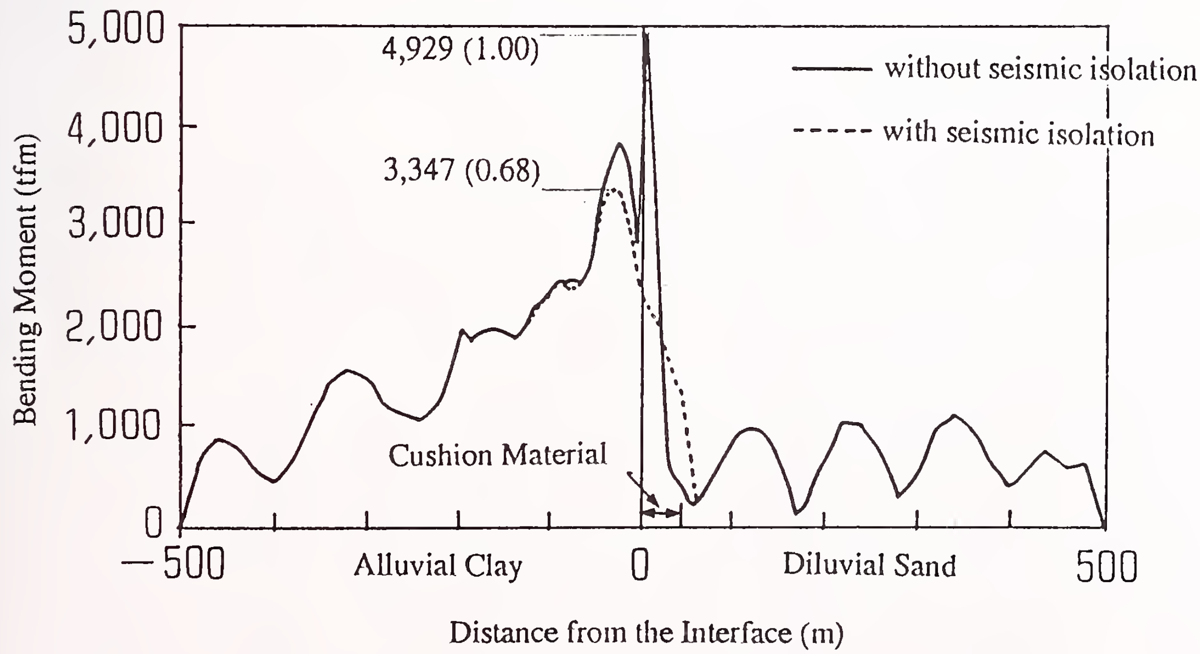


Figure 19 Maximum Bending Moment of Tunnel

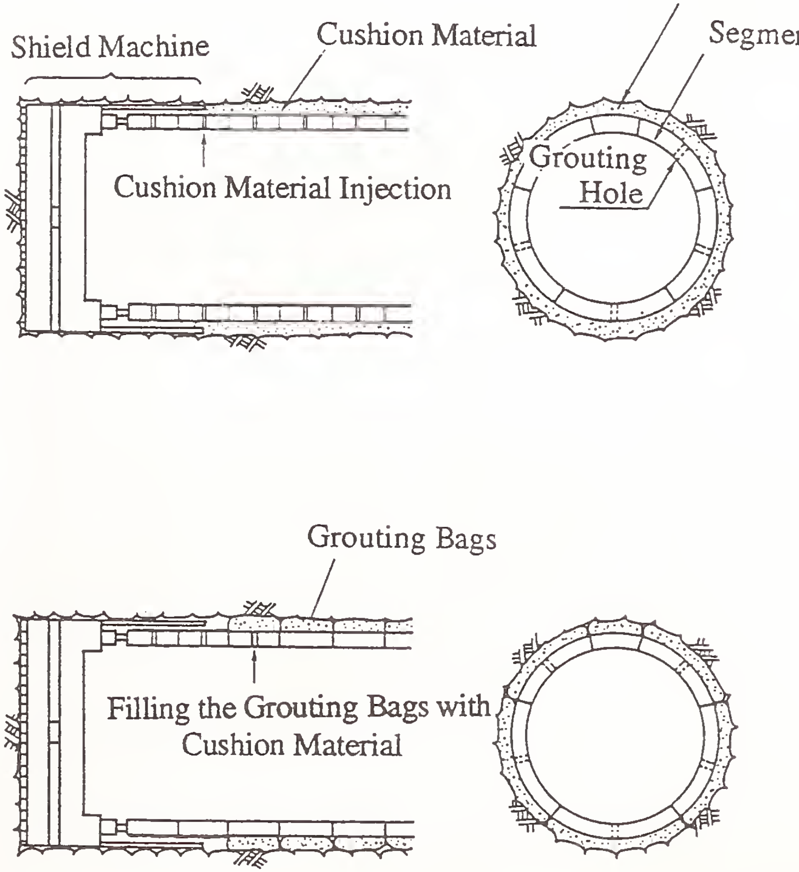


Figure 20 Installation of Cushion Isolation for Shield Tunnel

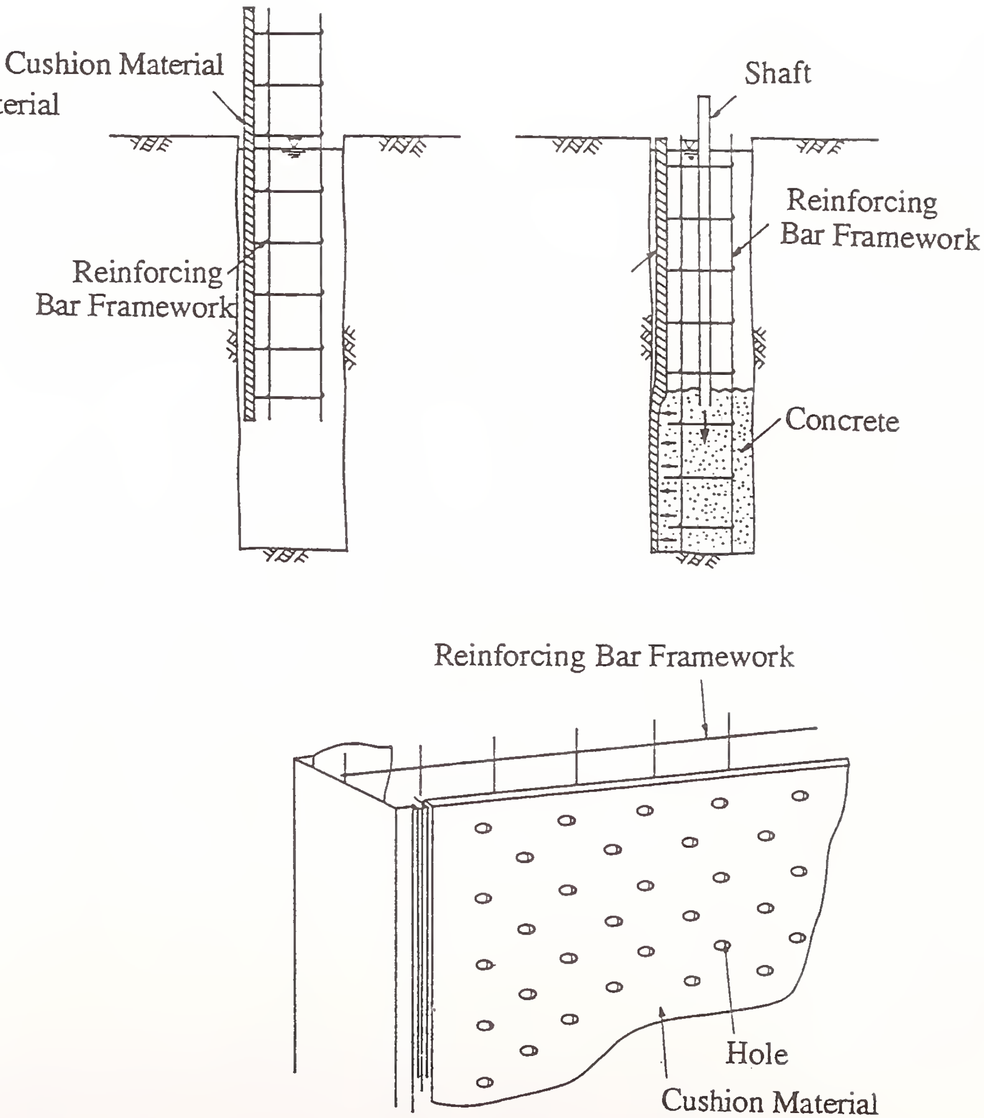


Figure 21 Installation of Cushion Isolasion for Diaphram Wall

CONCLUSIONS

Large stress concentration occurs at the joint of shield tunnel itself or at the joint of tunnel and the shaft during earthquakes. These stress concentration will be released by the installation of the proposed elastic washer and installation of flexible segment. Particularly, installation of the elastic washer is very efficient technique to release the longitudinal tensile strain of the tunnel since the washer is relatively inexpensive and it is easy to install.

At an interface zone between two different layers with large difference in stiffness, larger stress concentration occurs so that it can not be released by the technique of elastic washer or flexible segment alone. Enclosure of the tunnel or shaft by cushion material at near the interface zone looks effective to release the stress concentration. This technique should be discussed more before getting into the practical use.

FUTURE DIRECTIONS

Use of elastic washer like KRS and cushion isolation could improve the seismic resistance of shield tunnel and the shaft. Therefore, improvement and development of these techniques should be done as the future works. Installation technique of cushion isolation should also be studied. Effect of combined use of KRS washer and cushion isolation should also be examined.

ACKNOWLEDGEMENTS

This study was conducted as a part of study on "A Cooperative Research in Development of Seismic Design of Underground Structures" in "Technology Research and Development Regarding Use of Underground Space" of "A Comprehensive Research Project". The authors wish to express their thanks and appreciation to Dr. Kawashima, Public Works Research Institute, Ministry of Construction and to the cooperative research colleagues.

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V. SEISMIC RISK ANALYSIS

PERFORMANCE OF AWSS AND PWSS OF SAN FRANCISCO DURING AND AFTER LOMA PRIETA EARTHQUAKE

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ABSTRACT

The performance of the Auxiliary Water Supply System (AWSS) and the Portable Water Supply System (PWSS) of San Francisco following the 1989 Loma Prieta earthquake is discussed in this paper with reference to the initiation and suppression of the fire in the Marina. The computer program **GISALLE** (Graphical Interactive Serviceability Analysis of **LifeLines** for **Engineering**) is used to simulate observed damage to water mains and hydrants immediately after the earthquake. The results of the computer program, **GISALE**, are compared with actual performance, and recommendations are made for improving AWSS reliability.

INTRODUCTION

The October 17, 1989 Loma Prieta earthquake (Ms 7.1) was the largest event to occur in northern California since 1906. This event demonstrated that earthquakes can still cause severe damage to modern urban areas - although only few building were actually destroyed, major lifelines and vital services were lost. The critical importance and seismic vulnerability of utility systems was strongly emphasized in the Loma Prieta earthquake. In order to understand this vulnerability this paper briefly describes the damage and performance of the San Francisco Auxiliary and Portable Water Supply systems, during the Loma Prieta, earthquake. A graphical interactive computer program GISALLE (Graphical Interactive Serviceability Analysis of LifeLines for Engineering) has been used to simulate the Performance of the Auxiliary Water Supply System (AWSS) of San Francisco during the Loma Prieta earthquake.

SYSTEMS DESCRIPTION

Auxiliary Water Supply System (AWSS): The AWSS is one of only a few high pressure system of its type in the U.S. It is separate and redundant from the municipal water supply system (MWSS) of San Francisco, and is owned and operated by the San Francisco Fire Department (SFFD). It was built in the decade following the 1906 San Francisco earthquake and fire, primarily in the north-east quadrant of the City (the urbanized portion of San Francisco in 1906 and still the Central Business District), and has been gradually extended into other parts of the City, although the original portion still constitutes the majority of the system.

The AWSS supplies water to dedicated street hydrants by a special pipe network with a total length of approximately 125 miles of buried pipe, with nominal diameters ranging from 10 - 31 inches. Nearly 100 miles of the system is cast iron, to which about 25 miles of ductile iron pipe have been added during the past several decades. The system, which is shown in Figure 1, is intended to augment the city's existing fire fighting capacity by providing a supplementary network that would work independently of, and in parallel with, the MWSS. It is separated into upper and lower pressure zones. Each zone operates nominally at a pressure of about 150 psi, which is approximately 2.5 times the pressure in the municipal system.

The main source of water for the AWSS system under ordinary conditions is a 10 million gallon reservoir centrally located on Twin Peaks, the highest point within San Francisco (Figure 2). Water from this source supplies two zones: the Upper Zone, and the Lower Zone where the pressure is controlled at Ashbury Tank (0.5 million gallon) and Jones St. Tank (0.75 million gallon) respectively.

The Twin peaks reservoir supply may not be adequate under emergency conditions. Two pump stations exist to supply water from San Francisco Bay (each has four diesel pumps)- Pump Station No. 1 is located at 2nd and Townsend Streets, while Pump Station No. 2 is located at Aquatic Park - each has 10,000 gpm at 300 psi capacity. Both pumps were originally steam powered but were converted to diesel power in the 1970's. The pipe network has five manifold connections along the City's waterfront, in order to permit the City fireboat *Phoenix* to act as an additional "pump station", drafting from San Francisco Bay and supplying the AWSS. The *Phoenix's* pump capacity is 9,600 gpm at 150 psi.

Lastly, in addition to the above components, San Francisco has 151 underground cisterns, again largely in the northeast quadrant of the City These cisterns are typically of concrete construction (a few are brick and predate the 1906 earthquake) 75,000 gallons capacity (about one hour supply for a typical fire department pumper) and are located at street intersections, accessible by a manhole. They are highly reliable and extremely low maintenance. The cisterns are completely independent of all piping and are filled by hose by fire department pumpers supplied from hydrant. In the event of water main failure, water may be drafted from these cisterns via the manhole. In 1986, due to a recognition that these cisterns are mostly only in the northeast quadrant of the City, a bond issue was passed for construction of an additional 95 cisterns, in outlying portions of the City.

Control of the AWSS is centered at Jones Street tank house, where gauges provide pressure readings at a limited number of locations in the network. A limited number of gate valves can be remotely operated from Jones Street tank house via land lines, the Lower Zone-pressure can be increased by opening valves at the tank house, and the Twin Peaks pressure Zone-can be "cut-in" by remotely operating valves located at Ashbury tank. However, many other gate valves in the system must be operated manually.

In 1906, San Francisco had sustained major ground failures (leading to water main breaks) in zones generally corresponding to filled-in land and thus fairly well defined. Because it was anticipated these ground failures could occur again, these zones (termed "infirm areas") were mapped and the pipe network was specially valved where it entered these infirm areas. Under ordinary conditions, all of the gate valves isolating the infirm areas are closed, except one, so that should water main breaks occur in these infirm areas, they can be quickly isolated. On the other hand, should major fire flows be required in these areas, closed gate valves can be quickly opened, increasing the water supply significantly.

Portable Water Supply System (PWSS): While the AWSS provides high assurance of firefighting water supply in the northeast quadrant of San Francisco, major fires can and do occur at large distances from the AWSS pipe network. In recognition of this, and to provide greater flexibility in deployment and to further extend the "reach" of the AWSS, the San Francisco Fire Department has developed in recent years the Portable Water Supply System (PWSS). The basic components of the PWSS are:

- Hose Tenders, trucks capable of carrying 5,000 ft. of large diameter (5 inch) hose, and a high pressure monitor for a master stream,
- Hose Ramps, which allow vehicles to cross the hose when it is charged,
- Gated Inlet Wye, allowing water supply into large diameter from standard fire hose,
- Gleeson valve, a pressure reducing valve,
- Portable Hydrants, that allow water to be distributed from large diameter hose

The large diameter (five inch) hose is carried on the hose tenders, together with portable hydrants, pressure reducing Gleeson valves and other fittings. Each hose tender carries almost one mile of hose, and is capable of laying this in about twenty minutes. Hose lengths are intermittently fitted with the portable hydrants, permitting water supply at many locations along the hose, which can be gridded and in effect provides an above ground water main, (Figure 3). At the time of the Loma Prieta earthquake, SFFD had four PWSS hose tenders, and has since requested acquisition of an additional eleven.

PERFORMANCE OF WATER SUPPLY SYSTEMS

The locations of pipeline repairs after the 1989 Loma Prieta earthquake are shown in Figure 4. Zones of potential soil liquefaction, based on maps by Youd and Hoose (1975) and Hovland and Darragh (1981) are also shown. The most serious damage in the AWSS was the break of a 12" diameter cast iron main on 7th St. between Mission and Howard Sts. This location is on the boundary of Infirm Area No. 3 and, moreover, the AWSS pipe at this location crosses over a sewer line. Soil settlements in this area are thought to have occurred prior to and also as a result of the earthquake, causing the AWSS pipe to bear on the sewer line, and break. Water flow through this break, supplemented by losses at broken hydrants, emptied the Jones St. Tank. Loss of this supply led to loss of water and pressure throughout the lower zone of the AWSS. This resulted in critical condition in the Marina, where damage in the MWSS had cut off alternative sources of pipeline water.

Other breaks in the AWSS system included: (a) a break in an 8 inch hydrant branch, on Sixth between Folsom and Howard Streets (where the hydrant branch crossed up and over a sewer line) and (b) five 8 inch elbow breaks, four within Infirm Area No. 3 including one on Bluxome Street where a portion of a building collapsed onto an AWSS hydrant.

Hydrants were the most vulnerable parts of the system, with damage being concentrated at elbows. Typical construction involves an 8 inch diameter cast iron elbow affixed to a concrete thrust pad beneath the street surface hydrant. Damage at hydrant elbows occurred as 45 fractures centered on the elbows.

MARINA FIRE AND PWSS

The Marina fire was the largest earthquake-related fire in the loma Prieta earthquake. Ignition occurred in a four-story wood-frame apartment building at northwest corner of Divisadero and Beach Sts. Major leakage resulted from the AWSS breaks such that Jones Street tank completely drained. Leakage continued so that first arriving engines at the Marina fire, found only residual water when they connected to AWSS hydrants. Due to uncertainty as to the number and location of AWSS breaks, valves connecting the Upper Zone to the Lower Zone were not opened, and Pump Stations 1 and 2, although available, were not placed in-service immediately but only at 8 PM, following identification and isolation of broken mains. As a result, pressure in the AWSS Lower Zone was lost for several hours following the earthquake. The pump stations were operated at half capacity so as to fill the AWSS mains slowly, out of concern for entrapped air which was exhausted out of the Lower Zone through Jones Street tank (air could be heard exhausting through the tank) This operation continued until 10 PM when full pressure was restored and Jones Street tank had been filled with salt water.

Water to fight the fire was drafted and relayed from the lagoon in front of the Palace of Fine Arts, approximately three blocks away. The fireboat, *"Phoenix"* and special hose tenders were dispatched to the site. Approximately one and a half hours after the main shock, water was being pumped from the fireboat and conveyed by means of 5 inch diameter hosing, which had been brought to the site by the PWSS hose tenders. Eventually, the supply of water to the fire was about 6,000 gpm at 180 psi, which continued for over 18 hours. The fire was brought under control within about three hours after the earthquake.

ANALYSIS OF AWSS

An extended computer simulation of the AWSS network was performed with the program GISALLE (Graphical Interactive Serviceability Analysis of LifeLine for Engineering). This program was developed to represent the AWSS as part of a special demonstration project to develop advanced techniques of computer graphics for lifeline systems and to prove the feasibility of applying these techniques to a real system (Khater, et al., 1989; Grigoriu, et al., 1989). GISALLE has been checked successfully against special fire flow tests run by the San Francisco Fire Department. GISALLE is developed to (i) perform seismic hazard analysis (ii) generate damage states for lifelines consistent with the seismic intensity at the site; (iii) perform connectivity analysis; and (iv) perform hydraulic analysis for simulated damage states of the system.

Figure 5 shows a plan view of the system that was simulated to reproduce the conditions on the night of the earthquake. Water in the lower zone was supplied by the Jones St. Tank. The lower and upper zones were isolated from one another with closed gate valves. Pump Stations 1 and 2 were not included in the simulation to replicate the system conditions immediately following the earthquake.

Damage of the AWSS considered in the simulation was a broken 12" diameter main on 7th St., four broken hydrants, and two leaking joints. The approximate locations of these damaged components are illustrated in the figure.

Figure 6 pictures the water height in Jones Street Tank during the extended computer simulation (drop in water level in Jones Street Tank was considered in the analysis). Leaking joints were modeled as open hydrants during the simulation. This figure shows that the time required to empty the Jones St. Tank was about 32 minutes.

This estimated time to loss of tank agrees with observations during the earthquake. Scawthorn and Blackburn (1990) report that, when the first engine arrived at the Marina fire approximately 45 minutes after the earthquake, it could not draw water from the

AWSS hydrants. This time for engine arrival exceeds that analyzed for loss of the Jones St. Tank. Moreover, it was estimated that loss of water from a height of 35 to 18 ft in the Jones St. Tank took approximately 15 minutes which is consistent with the analytical results.

CONCLUSIONS

The AWSS performance and computer simulations emphasize how rapidly water can be lost and how important automatic control of isolation gate valves can be. The simulations also underscore the importance of hydrant breaks.

The AWSS, would have survived the earthquake and provided adequate water supply for extinguishment of the Marina fire except that the North-South division of the AWSS, part of the original concept and design, was eliminated in 1964, resulting in South of Market Street breaks draining the section of the AWSS North of Market, including the Marina district.

Although the AWSS pipe network failed in the short term due to a small number of breaks, this system failure could have likely been precluded if the breaks could have been identified sooner, leading to placing the pump stations on-line more rapidly. The need for a rapid damage assessment/reconnaissance technology is emphasized.

The fireboat/PWSS back-up system fulfilled its mission, and the fire was suppressed. This was due in large part to the extremely fortuitous circumstance of the unusual lack of wind. Normal prevailing wind conditions would almost assuredly have resulted in loss of several city blocks, with losses in the hundreds of millions of dollars

ACKNOWLEDGMENTS

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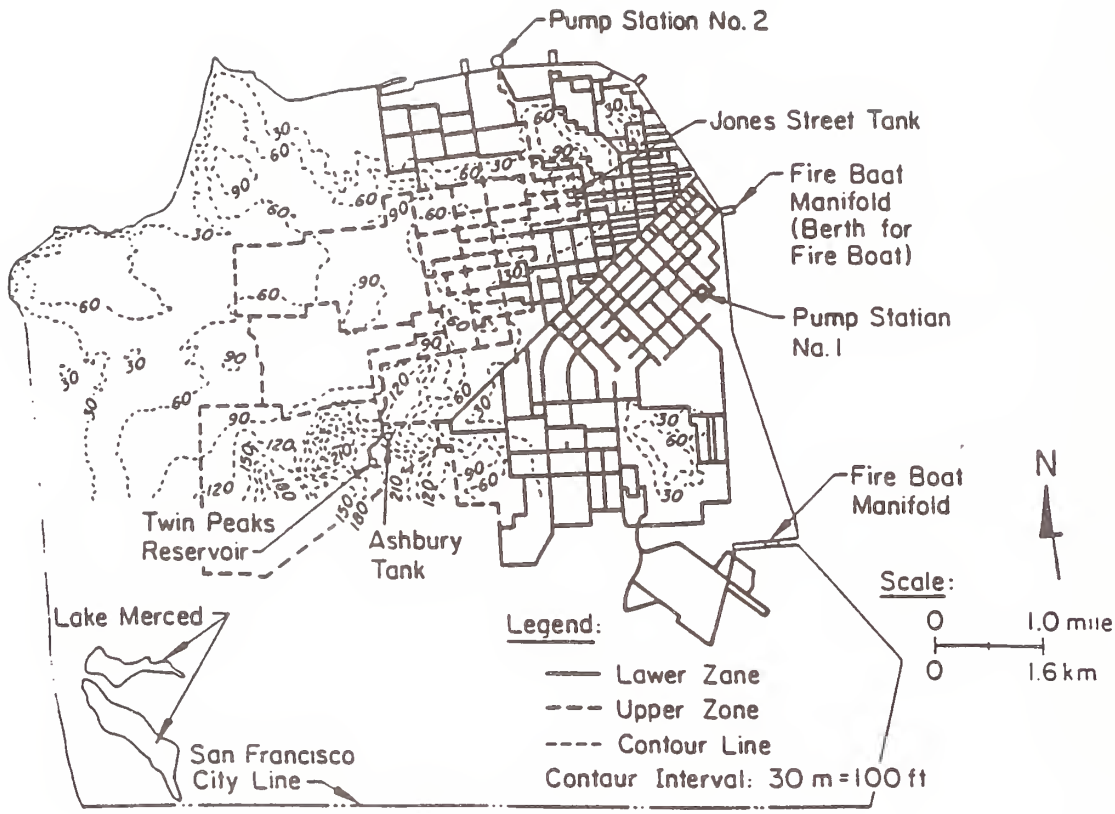

Figure 1: San Francisco AWSS

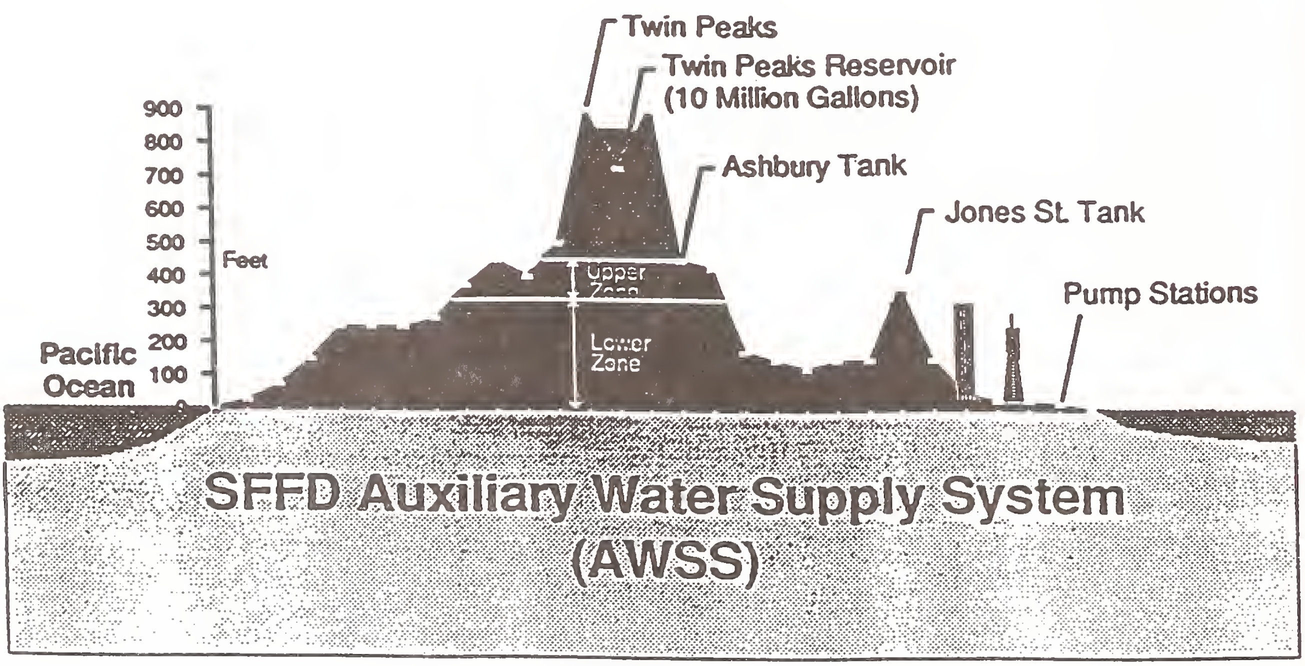

Figure 2: Schematic Cross-section San Francisco Auxiliary Water Supply System

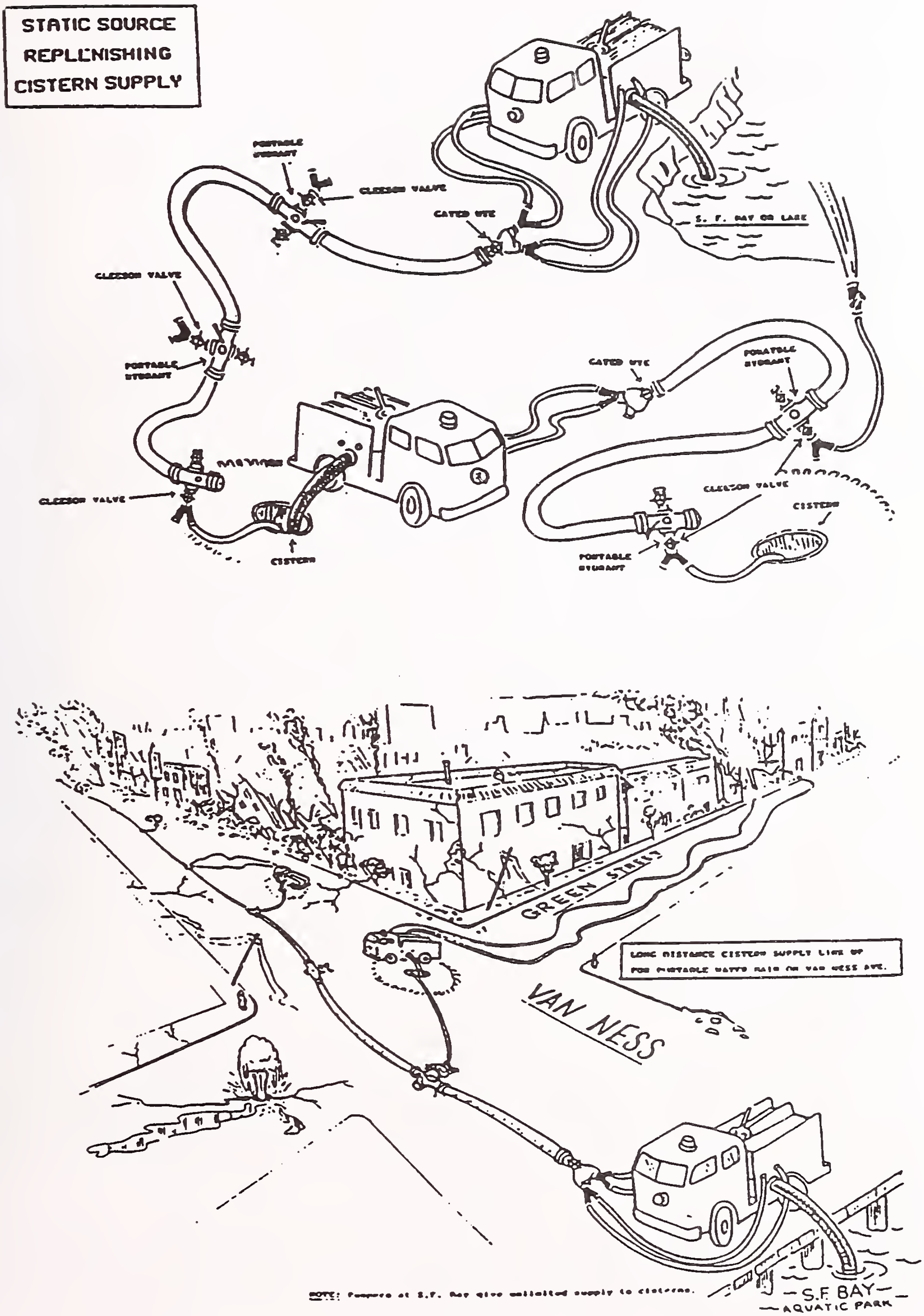

Figure 3: San Francisco Portable Water Supply System (schematic)

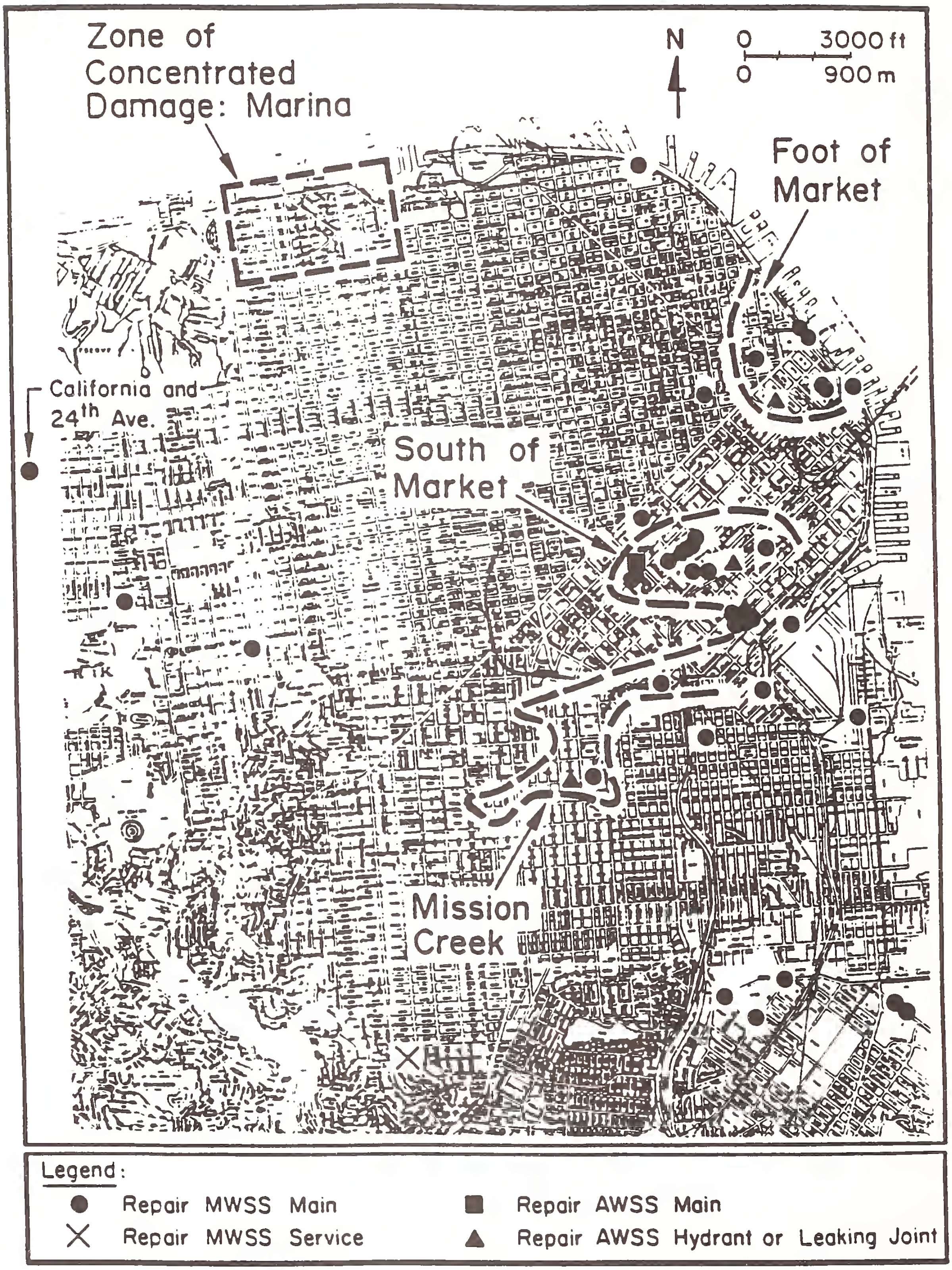

Figure 4: Water Supply Pipeline Breaks and Zones of Soil Liquefaction Caused by Loma Prieta Earthquake (after O'Rourke, et al., 1990)

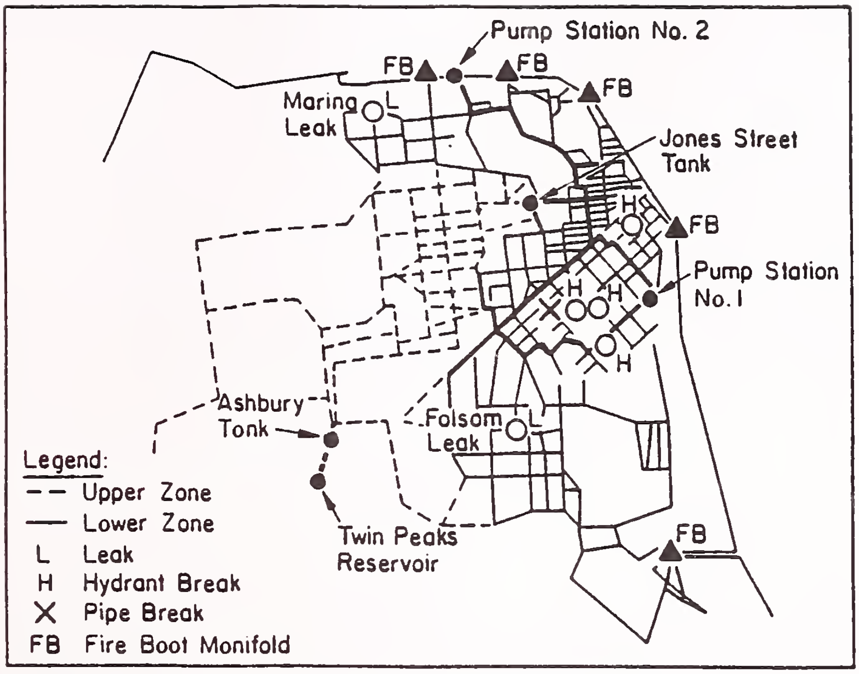

Figure 5: Conditions Used in Computer Simulation of AWSS Performance During the Loma Prieta Earthquake

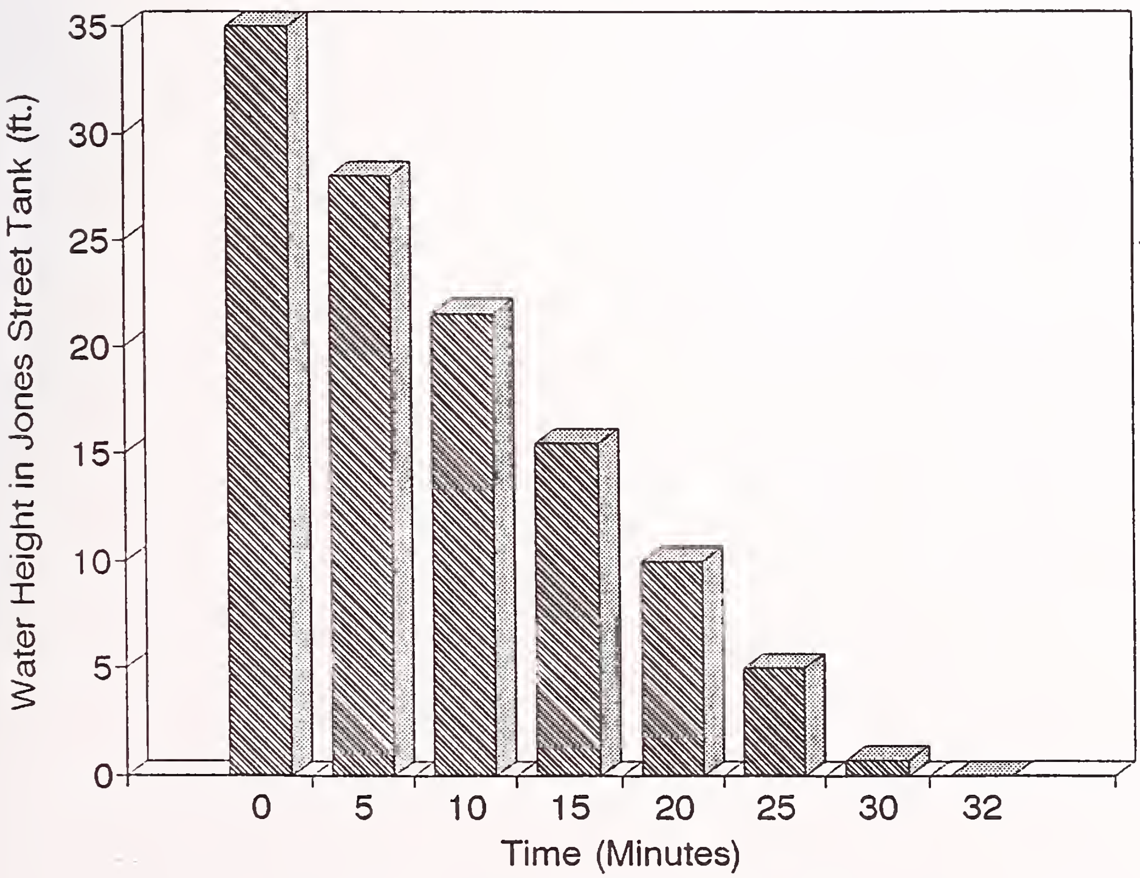

Figure 6: Results of AWSS Extended Simulation

SEISMIC RELIABILITY ANALYSES OF LARGE SCALE LIFELINE NETWORKS TAKING INTO ACCOUNT THE FAILURE PROBABILITY OF THE COMPONENTS

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ABSTRACT

To assess the seismic reliability of large scale lifeline networks and taking into account failure probability of the components, we developed an algorithm which has only polynomial complexity to enumerate paths in the network. Using this algorithm we have developed a new approximate calculation scheme, call the point matching method. The uncertainty of the attenuation law, shear wave velocity of ground and component's failure level are considered to calculate the seismic reliability of the network. This program can be run on a 32 bit desktop computer with the main memory of 16 MB and the operation time of 5.3 MIPS. This program package offers you an interactive capability, to select supply and demand nodes as well as show them on screen. A check of applicability of our new method is made by hypothetical seismic reliability analysis of a gas supply network composed of 1765 nodes and 1764 links.

INTRODUCTION

Because damage to a single location in a utility network often affects a major portion of the entire system, the reliability of lifeline networks during earthquake is a major concern for the prevention of urban hazards. When we are to assess the seismic reliability of lifeline systems, the following items must be considered: seismic activity and the attenuation of seismic waves; microzonation that takes into account to the local geological nature of each area; structural response and system analyses with proper modeling of components; and lifeline serviceability requirements. Seismic reliability analysis should include an efficient method for computing the reliability of the lifeline as a network interconnected nods and links. Because enumeration of network states in a seismic environment usually has exponential complexity, seismic reliability analyses of large scale networks have not been feasible as the number of components increases. To overcome this deficiency we proposed an efficient procedure for assessing the seismic reliability of lifeline networks (Ref.1). For a fixed earthquake on an active fault zone, we proved that the network took at most $2n^2 - 2n + 2$ states, n being the number of components. In the algorithm, we used the concept of the critical distance to the earthquake fault (Ref.2). A network is assumed to consist of links and nodes, and a node and link to fail if the seismic response exceeds a particular level at any point in each component. The intensity of the seismic response, Y, at a particular point in the network components is given through an attenuation law in terms of the earthquake magnitude, M, the shortest distance to the rupture zone,r, and the parameters $C_k(k = 1, 2\cdots)$ which express the source mechanisms and the effect of local geological parameters around the point. If the network component can sustain the magnitude of response expressed by Y^*, the shortest safe distance (the critical distance) between the component and the fault rupture is inversely solved from the attenuation law

$$r^* = f(M, C_k, Y^*) \tag{1}$$

The sphere of the radius with this distance centered at the element point defines the transition boundary for the component because it divides three dimensional space into the component of working and non-working states. The shape of the transition boundary for a node is a sphere. For a link it is a combination of a row of spheres and idealized by a circular cone with spheres at both end provided that there is linearity of the failure distances along the link. An example critical distance r^* obtained from an attenuation curve of strain (full line) and an allowable strain of buried pipe at a certain point ε^* is shown in Fig.1. Because the uncertainties of attenuation law and allowable strain of the component are not considered, the failure probability of the component changes from 0.0 to 1.0 if the shortest distance to an earthquake fault r becomes less than r^* as shown in Fig.2.

The purpose of this paper is to relax this binary condition of component's failure probability. The seismic reliability of a network may decrease if we take into account uncertainties of several factors. Especially the effect of uncertainty of attenuation law must be examined carefully.

DEFINITION OF FAILURE PROBABILITY OF COMPONENT

Network components are assumed to fail when the strain induced in the pipe exceeds a threshold level or the ground is liquefied. The Japanese specification for the earthquake-resistant design of gas pipes (Ref.3) defines the formula with which to calculate pipe strain as

$$\varepsilon = \beta \cdot \delta, \qquad \delta = (1 - \alpha) \cdot U \tag{2}$$

in which β is the coefficient for transferring relative displacement between the pipe and ground into the pipe strain, α the transmission coefficient, U the ground displacement given by

$$U = \frac{2}{\pi^2} \cdot T \cdot S_V \cdot K \cdot cos(\frac{\pi z}{2H}) \tag{3}$$

in which T is the natural period of the ground, S_V the response velocity value per unit seismic coefficient, K the design seismic coefficient and z the depth of the pipe.

By substituting Eq.(3) into Eq.(2) the strain induced in a pipe is expressed as follows:

$$\varepsilon = \beta \cdot (1 - \alpha) \cdot \frac{2}{\pi^2} \cdot T \cdot S_V \cdot K \cdot \cos\frac{\pi z}{2H} \tag{4}$$

Because the values of T, S_V and K scatter around their mean values of $\overline{T}$, $\overline{S_V}$ and $\overline{K}$, Eq.(4) can be rewritten

$$\varepsilon = \beta \cdot (1 - \alpha) \cdot \frac{2}{\pi^2} \cdot \overline{T} \cdot \overline{S_V} \cdot \overline{K} \cdot \cos\frac{\pi z}{2H} \cdot \frac{T}{\overline{\overline{T}}} \cdot \frac{S_V}{\overline{\overline{S_V}}} \cdot \frac{K}{\overline{\overline{K}}} \tag{5}$$

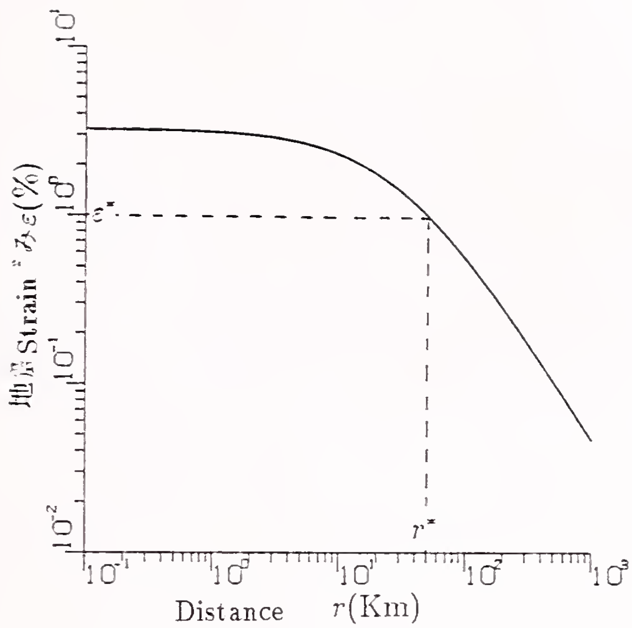


Fig.1 Relation between the seismic pipe strain and critical distance from an earthquake fault

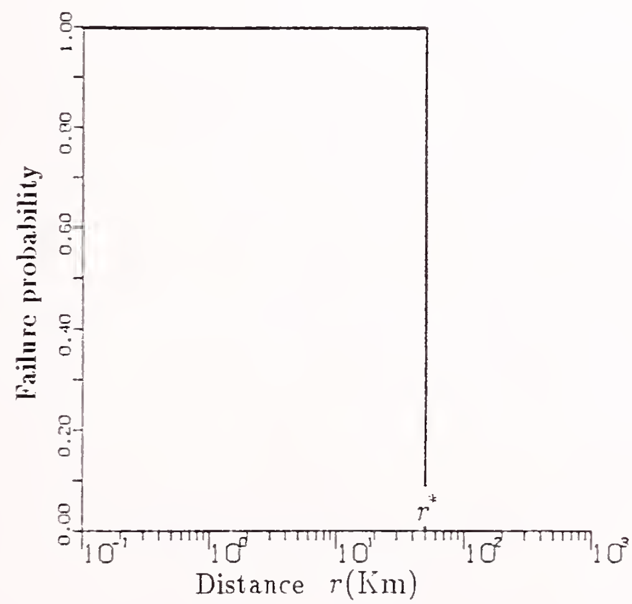


Fig.2 Relation between failure probability of a component and critical distance from an earthquake fault (Not considering uncertainty)

Taking logarithm of Eq.(5) the expression of pipe strain is divided into two parts.

$$\log \varepsilon = \log\{\beta \cdot (1-\alpha) \cdot \frac{2}{\pi^2} \cdot \overline{T} \cdot \overline{S_V} \cdot \overline{K} \cdot \cos \frac{\pi z}{2H}\} + \log \frac{T}{\overline{T}} + \log \frac{S_V}{\overline{S_V}} + \log \frac{K}{\overline{K}} \tag{6}$$

The first term of right-hand side Eq.(6) is derived from mean values of parameters. The remaining second, third and fourth terms are assumed to be random variables with normal probability distributions $N(\mu_T, \sigma_T^2)$, $N(\mu_{S_V}, \sigma_{S_V}^2)$ and $N(\mu_K, \sigma_K^2)$, respectively. If we define ε_0 as follows:

$$\varepsilon_0 = \beta \cdot (1-\alpha^*) \cdot \frac{2}{\pi^2} \cdot \overline{T} \cdot \overline{S_V} \cdot \overline{K} \cdot \cos \frac{\pi z}{2H} \tag{7}$$

$\log \varepsilon$ has a normal probability distribution with a mean value of $\mu_\varepsilon = \log \varepsilon_0$ and a variance of $\sigma_\varepsilon^2 = \sigma_T^2 + \sigma_{S_V}^2 + \sigma_K^2$. To calculate ε_0 the value of K must be expressed by an attenuation law in terms of the earthquake magnitude, m, and the shortest distance to the rupture zone, r. Because the attenuation laws proposed so far are functions of the magnitude and epicentral distance, we have used the following relation for K (Ref.4)

$$K \cdot g = 227 \cdot 10^{0.308m}(r+30)^{-1.201} \tag{8}$$

in which g is the acceleration of gravity.

The failure probability of a component for an earthquake with magnitude m at critical distance $\tilde{r_1}$ is calculated as shown in Fig.3. If the allowable strain level ε^* is a definite value the component loses its function on the condition of $\varepsilon \geq \varepsilon^*$. The failure probability $p_f(m, \tilde{r_1})$, therfore, can be calculated by the area of shaded part in Fig.3 or by the following formula,

$$p_f(m, \tilde{r_1}) = \mathrm{P}(\varepsilon \geq \varepsilon^*) \tag{9}$$

An example relation between failure probability of a component and the critical distance from an earthquake fault is given in Fig.4. The distance r^* at where the failure probability being 0.5 is obtained from the condition of $\varepsilon_0 = \varepsilon^*$.

If the allowable strain ε^* is a random variable obeying a logarithmic normal distribution with the mean value of $\mu_{\varepsilon^*} = \log \overline{\varepsilon^*}$, and the variance of $\sigma_{\varepsilon^*}^2$, then failure probability of the component $p_f(m, \tilde{r})$ is given by

$$p_f(m, \tilde{r}) = 1.0 - \Phi\left(\frac{\mu_{\varepsilon^*} - \mu_\varepsilon}{\sqrt{\sigma_{\varepsilon^*}^2 + \sigma_\varepsilon^2}}\right) \tag{10}$$

in which $\Phi(x)$ is the standard normal distribution function.

SEISMIC ENVIRONMENT

The gross parameters of the seismic source, which are required to reliability analysis of lifeline network, are the fault length and width, the seismic moment and the magnitude of the earthquake. The seismic moment, M_o, is the fundamental parameter used to measure the magnitude of an earthquake caused by a fault slip, but the surface magnitude, m_s, is

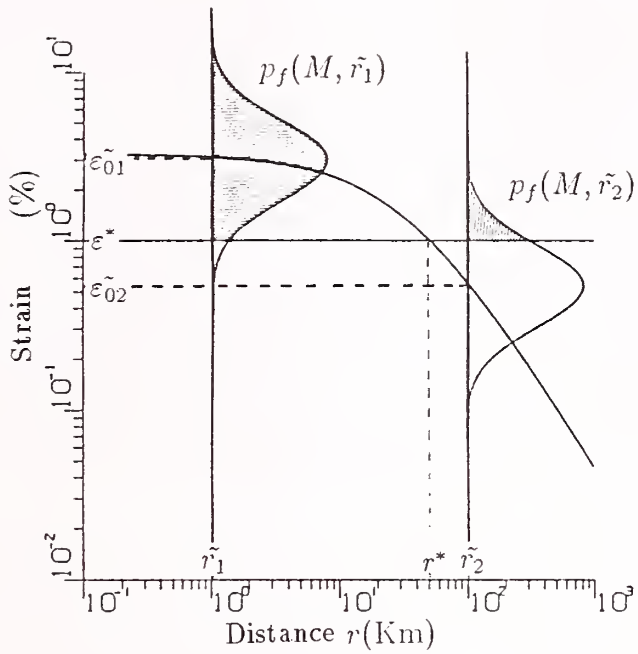


Fig.3 Calculation of failure probability of a component at several critical distances from an earthquake fault

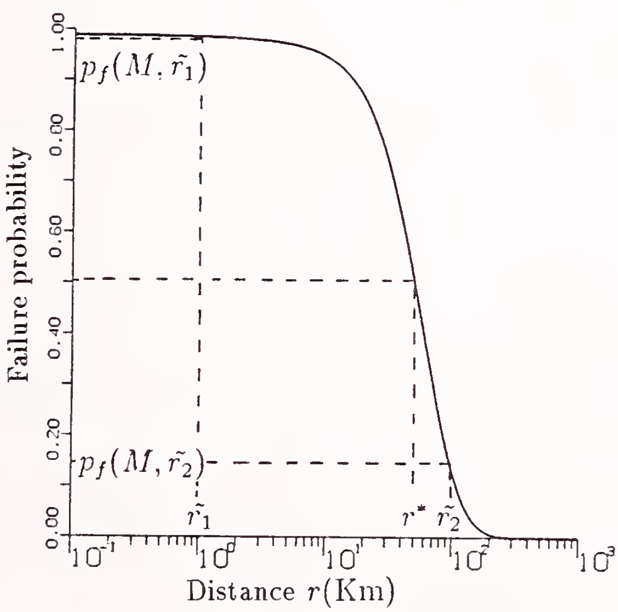


Fig.4 Relation between failure probability of a component and critical distance from an earthquake fault
(Taking into account uncertainty)

used in engineering. We have the relation between m_s and M_o given by Geller (Ref.5). The fault area, S, also is related to M_o;

$$S = 1.88 \cdot 10^{-15} M_o^{2/3} \tag{11}$$

The network component will fail with probability, $p_f(m, \tilde{r})$, if the transition boundary with the critical distance $\tilde{r}$ intersects the earthquake rupture area. Taking into account the probabilistic nature of an earthquake occurring, we have defined an active fault zone. The collocation of earthquake rupture on this active fault zone is determined from a probabilistic concept. We assume rectangular areas (the length of fault to be twice of the width) for the active fault zone and for a rupture caused by an earthquake as shown in Fig.5. The collocation of both rectangles is arranged with the axes of symmetry parallel. We define the transition area with failure probability of $p_f(m, \tilde{r})$ as the intersection between the critical boundary of a component of the network and the active fault zone as shown in Fig.5. For a node it is a circle; for a link it is the cross section between the active fault plane and a circular cone with a sphere at both end.

To formulate network reliability, we must assume that this transition area with failure probability p_f may intersect an earthquake rupture with magnitude m; therefore, it is convenient to look at the positions of rupture from the center of the earthquake. We defined a coordinate transformation (Ref.1) by which the position of earthquake rupture can be represented by the center of the rupture. An example of the collocation of this newly defined transition area for a simple network composed of twelve nodes and eleven links is shown in Fig.6. We assume that only critical boundaries of nodes D and E compose transition areas. The relation between the failure probability and critical distance was a continuous function as shown in Fig.4. For purpose of numerical simplicity we discretize this relation as shown in Fig.6. There are four transition areas for each node with different failure probability and these areas divide the active fault zone into 12 regions (defined as influence regions) that are mutually exclusive and collectively exhaustive regions. The network condition changes from one region to the other. For example the region 8 expresses the network condition that nodes of D and E fail simultaneously with probabilities of 0.5 and 0.25, respectively.

NETWORK RELIABILITY COMPUTATION

In order to illustrate the reliability computation a multi-terminal reliability measure is used that is defined as the probability that the given source nodes can be connected to all specified terminal nodes in the network. Let $I_i(x, y/m)$ be an reliability index of influence region i defined by the probability to satisfy the function of a network after a magnitude m earthquake with a rupture centered at (x, y). How to calculate the reliability index of region 8 in Fig.4 (I_8) is explained in Table 1. The connectivity from nodes A and B to nodes K and L is chosen as the reliability measure. There are four cases based on component's failure condition. $I_8(x, y/m)$ becomes 0.75 by adding all probabilities of network functioning.

For simplicity we consider only one active fault zone in the following analysis. The transition areas divide the active fault zone into some M influence regions A_i $(i = 1, 2, \cdots, M)$. Each A_i region corresponds to some state of the network and $I_i(x, y/m)$ is constant within A_i, so that the reliability of the network, G, conditioned on magnitude m is

$$R(G/m) = \sum_{i=1}^{M} \{I_i(A_i/m) \cdot A_i\} / \sum_{i=1}^{M} A_i \tag{12}$$

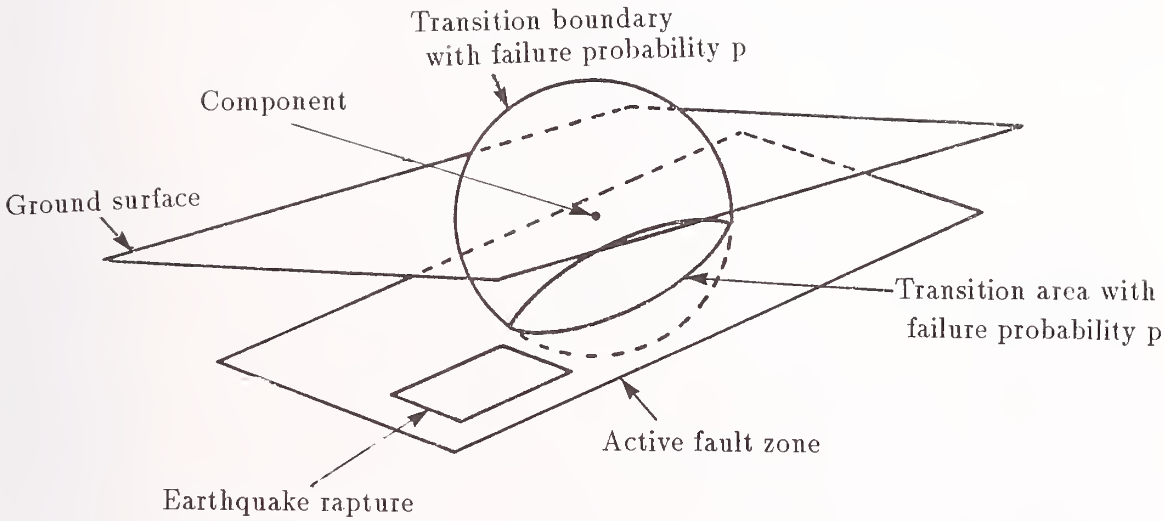
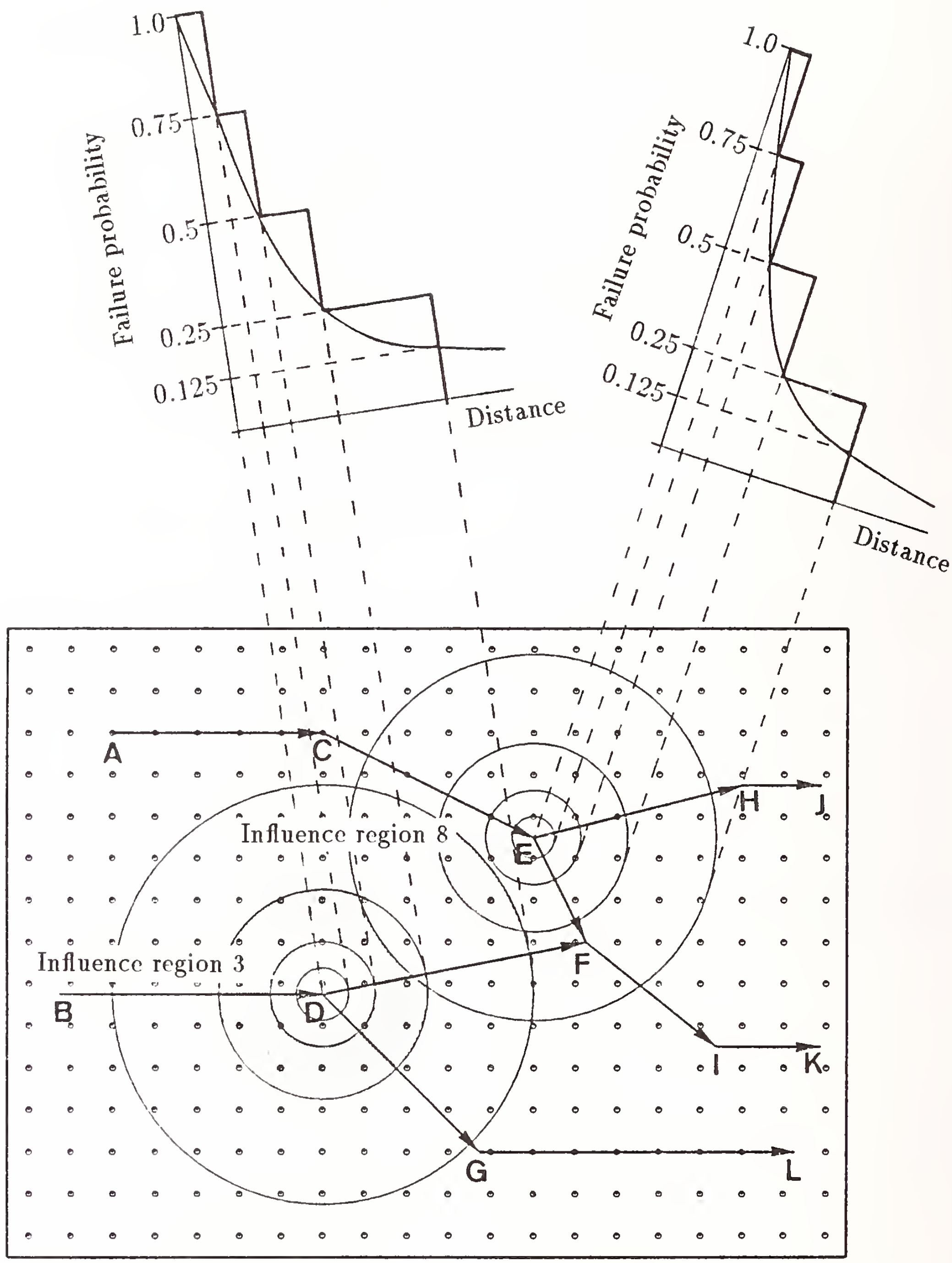


Fig.5 Collocation of transition boundary, an earthquake rapture in the active fault zone and transition area of a component



Evaluation point

Active fault zone

Fig.6 Example of the collocation of transition area with different failure probability for simple network

(Only the transition areas of nodes D and E are considered)

Table 1 Calculation of reliability index

	Node D	Node E	Reliabilty measure	Probability
1	F	F	NS	
2	F	NF	NS	
3	NF	F	S	$(1-0.25)\times 0.5 = 0.375$
4	NF	NF	S	$(1-0.25)\times(1-0.5) = 0.375$
				Reliability index $I_8 = 0.750$

F: Failure S: Satisfy

NF: Non failure NS: Not satisfy

Table 2 Calculation of reliability for the connectivity from nodes A and B to nodes K and L

Region No.	Reliablity index I_i	Number of evaluation points N_i	$N_i \cdot I_i$
1	1.0	177	177
2	0.75	44	33
3	0.5	12	6
4	0.25	6	1.5
5	0.0	1	0
6	0.5	2	1
7	0.75	11	8.25
8	0.75	2	1.5
9	1.0	31	31
10	1.0	8	8
11	1.0	4	4
12	1.0	2	2
Total		300	273.25
Reliability $R = (\sum_{i=1}^{12} I_i \cdot N_i)/\sum_{i=1}^{12} N_i = 0.9108$			

in which we assume uniform distribution of earthquake occurrence on the active fault zone.

Point matching method : The most time consuming part of the developed program is the calculation of area of influence region A_i. After detail check of program statements we found that the order of enumeration for a network with n elements was $O(n^6)$ due to the existence of nested DO ROOPS to calculate A_i although our algorithm guarantees the order of $O(n^2)$ (Ref.1). To overcome this deficiency we have developed an algorithm named "point matching method". The idea of the method is simple. Evaluation points are uniformly distributed on the active fault zone and the area of region A_i is assumed to be proportional to the number of evaluation points included in the region A_i. The value of reliability index defined by $I_i(x, y/m)$ is assigned to each evaluation point. If the number of evaluation points in the region A_i is N_i then Eq.(12) is rewritten as follows:

$$R(G/m) = \sum_{i=1}^{K}\{I_i(N_i/m) \cdot N_i\} / \sum_{i=1}^{K} N_i \tag{13}$$

The reliability for the network shown in Fig.6 is calculated in Table 2 using the above mentioned procedure. The number of evaluation points is 300 and the included evaluation points in each influence region is also given in Table 2.

Application of minimal cutset: In Table 1 we counted out all possible case for calculating reliability index. But this is not feasible as the number of collectively exhaustive regions increases. For an illustrative purpose a network shown in Fig.7 is considered. The reliability measure is connectivity from nodes S_1, S_2 and S_3 to nodes D_2 and D_3. The transition areas of nodes A, B, C and D are intersected with active fault zone and only single failure probability for each transition area is assigned as given in Table 3. If we calculate the reliability index of shaded influence region by taking into account all possible cases we have to count out 16 (2^4) cases because this region is included inside of four transition areas composed of nodes A, B, C and D. The computation time to calculate the reliability index expands exponentially as the number of related transition areas of an influence region increases. To avoid this difficulty we use minimal cutset which is the subset of network components. If only one component in this subset survives the connectivity of the network is satisfied. In the example given in Table 4 the minimal cutset is the combination of nodes C and D because both nodes must survive to satisfy the reliability measure. The reliability index of this influence region, therefore, can be calculated by counting out all cases satisfying reliability measure for nodes A and B and multiplying survival probability of nodes C and D as shown in Table 5.

A MODEL OF GAS NETWORK AND AN ACTIVE FAULT ZONE

We considered a gas supply network (Fig.8) composed of 1765 nodes and 1764 links. The network shown in Fig.8 is an idealization of the southern portion of the middle pressure A line (gas pressure: $10>p>3kg/cm^2$) and B line ($3>p>1kg/cm^2$). Six source nodes expressed by astrisks are connected to the high pressure line through regulator stations, and 640 terminal nodes are connected to source nodes of the low pressure line ($p < 1kg/cm^2$) through district regulators. Based on the gas demand 116 terminal nodes are chosen as the demand nodes. To evaluate the reliability of this network we have chosen a simple connectivity measure, the probability that gas can reach all the demand nodes.

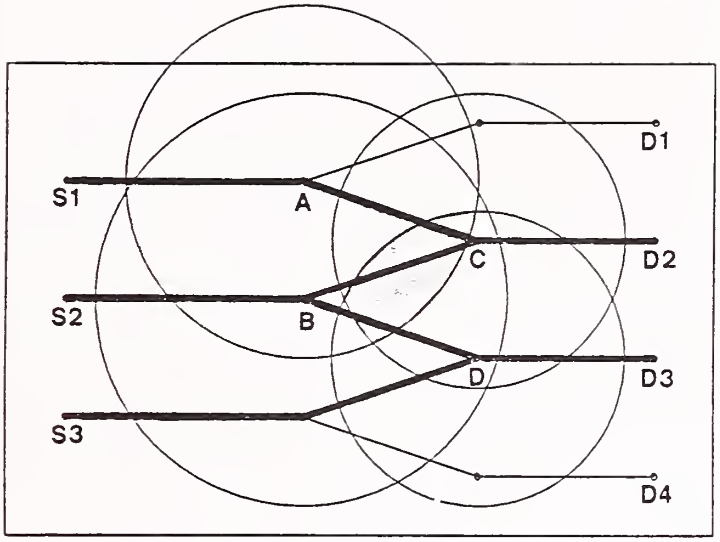


Fig.7 A network to apply the minimal cutset

Table 3 Failure probability and survival probability

Node	P_f	P_l
Node A	0.3	0.7
Node B	0.4	0.6
Node C	0.9	0.1
Node D	0.8	0.2

Table 4 Calculating reliability index without using minimal cutset

	Node A	Node B	Node C	Node D	Reliability measure	Probability
1	o	o	o	o	o	0.7×0.6×0.1×0.2=0.0084
2	o	o	o	×	×	
3	o	o	×	o	×	
4	o	o	×	×	×	
5	o	×	o	o	o	0.7×0.4×0.1×0.2=0.0056
6	o	×	o	×	×	
7	o	×	×	o	×	
8	o	×	×	×	×	
9	×	o	o	o	o	0.3×0.6×0.1×0.2=0.0036
10	×	o	o	×	×	
11	×	o	×	o	×	
12	×	o	×	×	×	
13	×	×	o	o	×	
14	×	×	o	×	×	
15	×	×	×	o	×	
16	×	×	×	×	×	
					Reliability index	0.0176

Table 5 Calculation of reliability index using minimal cutset

	Node A	Node B	Reliability measure	Probability
1	o	o	o	0.7×0.6=0.42
2	o	×	o	0.7×0.4=0.28
3	×	o	o	0.3×0.6=0.18
4	×	×	×	
				0.88
			Reliability index	0.1×0.2×0.88=0.0176

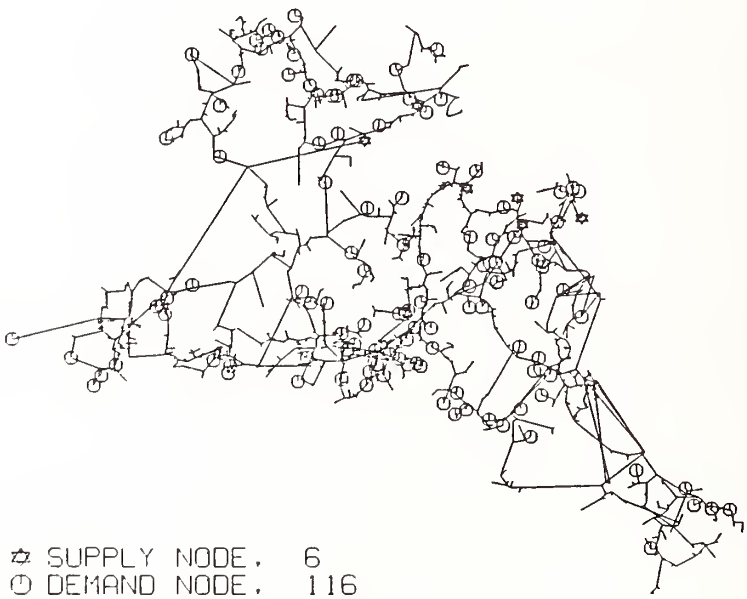


Fig.8 Middle pressure gas supply network model in Shounan area (6 supllv nodees and 116 demande nodes)

An active fault zone is considered (Fig.9). This zone includes the fault area of the 1923 Kwanto Earthquake. The shear wave velocity and thickness of the ground at each node are determined by using the geological data base constructed from the results of standard penetration tests conducted at 1618 points in the area concerned. The depth of the buried pipe elements is assumed to be 1.2m.

COMPARISON OF CPU TIME FOR EXECUTING THE PROGRAM

The main system of the data processing center of Kyoto University consists of a large-scale general-purpose computer, FACOM M-780/30 and two super computers, VP-400E and VP-200. We use M-780/30 and VP400E systems for comparing CPU time to analyze seismic reliability of a sample network. A FORTRAN program developed by using M-780/30 can be executed by a vectorized computer (VP-400E) without any modification. Decrease of execution time is based on the rate of vectorization of each program unit. To decrease efficiently execution time of the program unit there is a software with an interactive capability, to show the execution time of each program unit on VP-400E and on M-780/30; the number of execution times of each program unit; a possibility for vectorization and procedures to increase computation speed of each program unit. A comparison of needed CPU time between a vectorized computer (VP-400E) and a general purpose computer (M-780/30) for analyzing seismic reliability of a gas supply network with the 3529 elements gives a result that the vectorized program attains 5.5 times computation speed at the earthquake magnitude 6.6, 17.4 times at the magnitude of 6.8 and 34 times at the magnitude of 7.0. As the earthquake magnitude becomes large the efficiency of vectorized program increases because the vectorized length (the number of data processed by vectorizer) becomes long.

To check the accuracy of the point matching method the seismic reliability of above mentioned gas supply network is calculated for earthquake magnitude of 6.6, 6.8 and 7.0. The number of evaluation points is 5000. The results also show in good agreement between the seismic reliability calculated by the exact method and that by the point matching method. Although the program of this method is executed on M-780/30, the CPU time is attained almost 6 times lesser than that for exact solution obtained by using super computer (VP-400E).

APPLICATION

A work station, NEWS-1860, with 5.3 MIPS processing speed is mainly used for calculating seismic reliability of the gas network shown in Fig.8. In the network there are 640 terminal nodes. We classified these terminal nodes into five levels based on their gas demands. The case of 116 demand nodes is the fourth level. The standard deviations of random valuables appeared in Eqs.(6) and (10) are determined by using the observed and experimental data as follows:

$$\sigma_T = 0.151, \quad \sigma_{S_V} = 0.105, \quad \sigma_K = 0.244, \quad \sigma_{\varepsilon^*} = 0.04$$

The relation between the failure probability of a component and critical distance is discretized every 1The calculated seismic reliability against a random earthquake for the fourth level is shown in Fig.10. Three cases are presented in the figure. First is the case of without

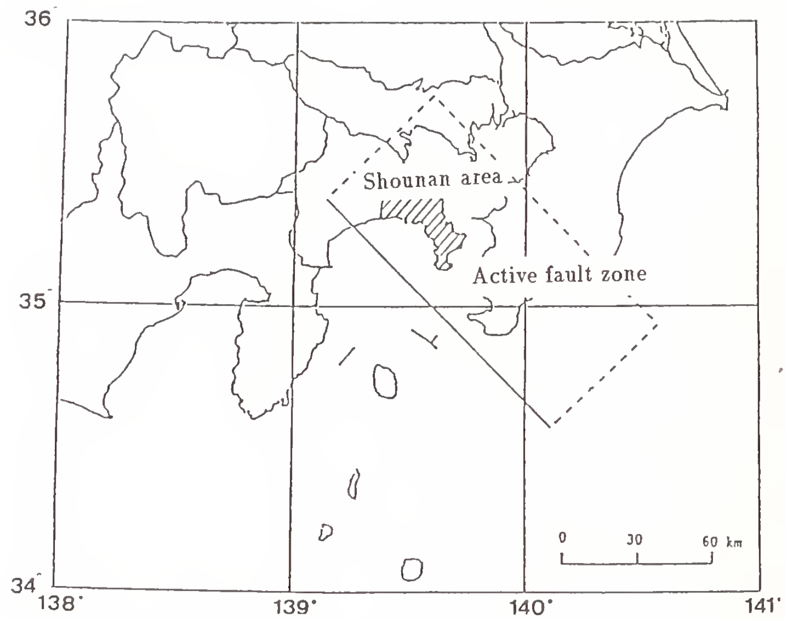


Fig.9 An active fault zone

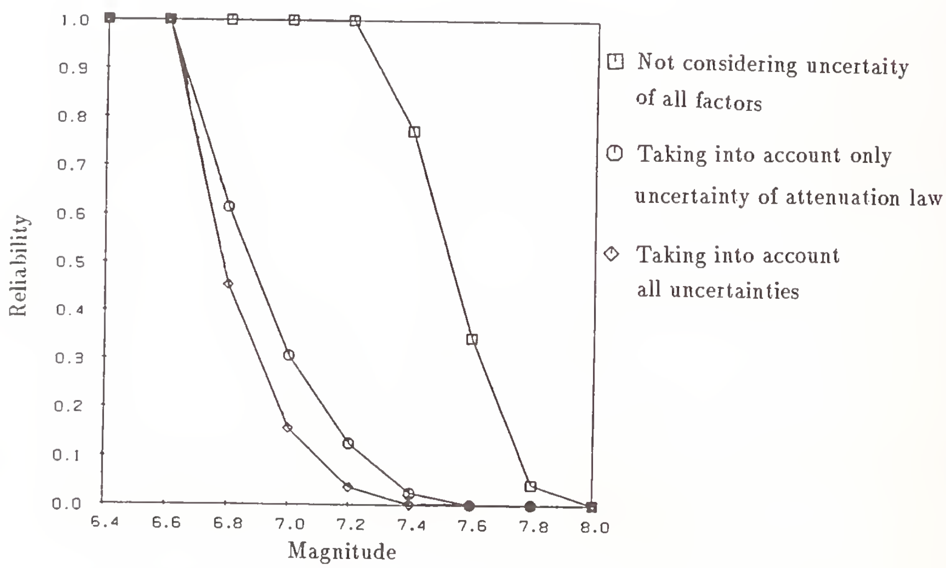


Fig.10 Relation between seismic reliability and earthquake magnitude

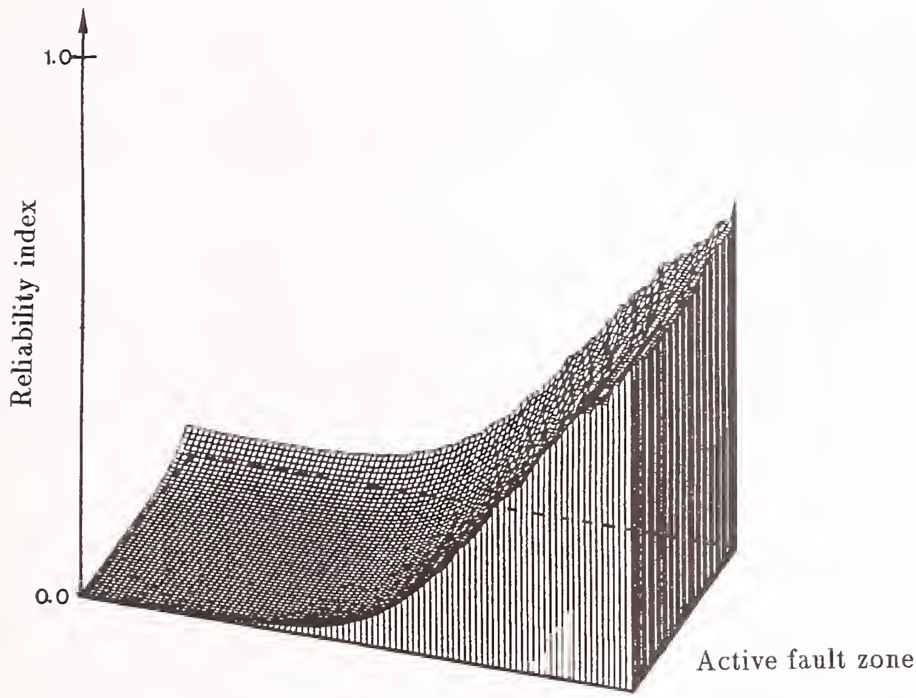


Fig.11　Distribution of seismic reliability index on the active fault zone

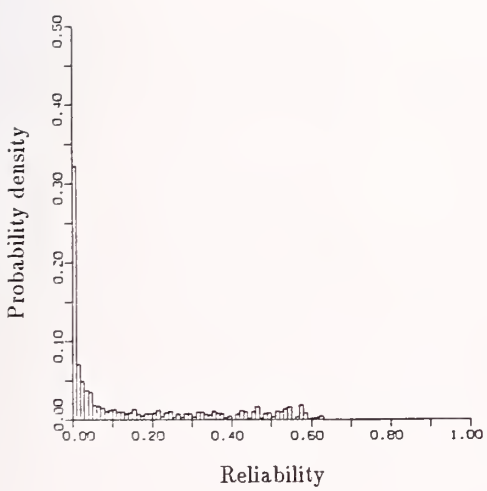


Fig.12　Probability density of seismic reliability

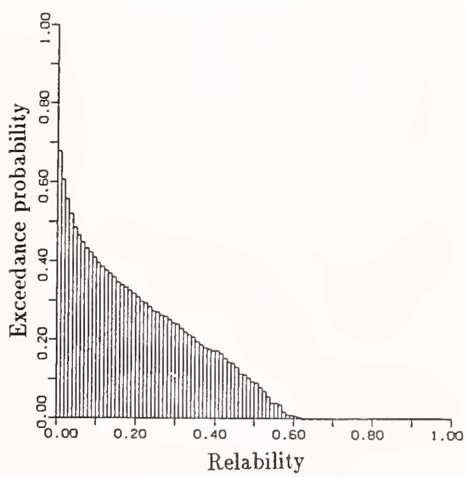


Fig.13　Cumulative distribution of seismic reliability

any uncertainties, so that only the binary failure probability for each component is assumed. Second is that of taking into account only the uncertainty of attenuation law, and third includes all uncertainties. Because the uncertainty of attenuation law is largest the seismic reliability of network decreases drastically when the uncertainty of attenuation law takes into account.

The distribution of reliability index on the active fault zone is calculated for each earthquake magnitude as shown in Fig.11. This is the result for earthquake magnitude 7.0. The probability density function of seismic reliability of the network is obtained from Fig.11 as shown in Fig.12 based on Eq.13. By integrating the probability density function we can draw the cumulative distribution function of seismic reliability of network as shown in Fig.13. Form this figure we can estimate the probability guaranteeing a certain level of seismic reliability of network.

CONCLUSION

Uncertainties of several factors for assessing seismic reliability of large scale lifeline system have been implemented in the algorithm with which to analyze the seismic reliability. It uses a new approximate calculation scheme called point matching method and minimal cutset theory. Calculation of the seismic reliability of an example network shows that newly developed program package can be used with large networks. The result reveals that the effect of uncertainty of attenuation law is the largest on the seismic reliability analyses of networks.

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EARTHQUAKE DAMAGE ANALYSIS ON TELECOMMUNICATION CONDUITS

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ABSTRACT

A telecommunication conduit was severely damaged at a threaded joint during the 1983 Nihonkai Chubu earthquake, which hit the northern part of Japan, as well as other underground structures. The damaged telecommunication conduit was a 75 mm diameter steel pipe connected with threaded joints.

In order to clarify the cause of the damage, the relationships between the strength of the threaded joint and permanent ground displacement are investigated in this study, which displacements were occurred along the conduit due to liquefaction of sandy soil.

Experiments on the threaded joint were conducted to investigate the deformation characteristics against compressive force, tensile force and bending moment. Critical axial strain of the conduit subject to compressive force was 0.085%, for example.

Compression tests of the buried conduits were conducted in a soil container with different soil conditions. Deformation of the joints were very similar to the damaged joint observed at the site after the earthquake in the both soil conditions. The load versus displacement curves, however, were not so different each other.

The permanent ground displacements along the conduit were measured by comparing the aerial photographs before and after the earthquake. As a result of the investigation of the ground movement, compressive ground strain along the conduit reached to 0.35%, which was large enough to cause the damage of the joint of the conduit.

It is concluded that the axial rigidity and deformability of the joint of the conduit is a critical parameter for the seismic design especially in the liquefiable area, where the permanent ground displacement are expected.

INTRODUCTION

Structural safety and maintenance of communicational function of lifeline communication systems at the time of an earthquake are important subjects for disaster prevention, hence investigations have been made from various viewpoints .[1,2] Investigation of the earthquake resisting strength of conduits for buried communication cables is one of those subjects. This report describes the results of analysis of damage to conduits for communication cable in Nohiro City. The damage was caused by Nihonkai-chubu earthquake in 1983. The analysis concern the relationship between the strength of the conduit joint and the permanent displacement [3] of the ground.

DAMAGE TO CONDUITS FOR COMMUNICATION CABLES

An example of damage to a conduit joint is shown in Photo. 1. This conduit is a steel pipe with 89.1 mm outer diameter and 4.2 mm wall thickness. It is connected by a threaded joint of 120 mm total length. As shown in the photograph, jointing parts of the conduits show crank like bending by rotation around the socket. However, because the rigidity of the socket was higher than the conduit, deformation of the socket was hardly found, and one of the conduits was bent until the section of pipe was completely closed, while the section of another conduits remained open because of the plastic deformation of the thread ridge in spite of the bending.

DISTRIBUTION OF GROUND PERMANENT DISPLACEMENT

Fig. 1 shows the plan of one section (length: about 250 m) of Telecommunication conduits which includes damaged positions (▲). Also, permanent displacement vectors, which were obtained by an aerial photographic survey, are shown. Each measuring point was set on both sides of the conduits at intervals of about 20 m and permanent displacements of the conduits were obtained by interpolating measured data from pairs of measuring points which hold conduits between them. Fig. 2 shows the results.

As seen in Fig. 2, the distance between the manholes at each end shows a relative reduction of about 85 cm, distribution of permanent displacement in the direction of the pipe axis shows nearly linear change over the total length of about 250 m, and compressive strain of about 0.35% was generated. In addition, the horizontal distribution of displacement in the direction which is rectangular to the pipe axis is small within an interval of about 50 m around the damaged position, and shows 34 cm maximum displacement at the front and rear ends of the above interval. Displacements in the vertical direction (amount of subsidence) varies in the range of 26 to 47 cm but, as a whole, subsidence is nearly uniform.

Permanent displacement in the direction of the pipe axis and permanent displacement in the direction rectangular to the axis, which were described above, are called lateral shifts and can be shown by the models in Fig. 3. Permanent displacements which were observed in Noshiro City were expressed by a model as seen in Fig. 3. The upper limit of the relationship between the length of transition interval and relative displacement δ, and the relationship between the width of the permanent displacement W and permanent displacement δ is shown by Table 1. Accordingly, in the case of displacement in axial the direction of the pipe, axial strain of the ground ($\varepsilon_g = \delta/L$) is ε_g = 3% (constant) when $L \leqq 50$ m, but axial strain of the ground in the case of 50 m < L is ε_g = 0.6% which is inversely proportinal to L. For instance, when L = 250 m, which is the damaged interval, is substituted in the above equation, ε_g = 0.6%, which is about 1.7 times the value of the actual measurement (about 0.35%), is obtained. For the direction rectangular to the pipe axis, W = 100 m, δ = 0.2 m is obtained in the neighborhood of damaged position. Accordingly, δ = 1 m is obtained when W = 100 m is substituted in the above equation. The upper limit of displacement by actual measurement is about 5 times the damaged position.

EXPERIMENT COMPRESSIVE STRENGTH OF THE CONDUIT JOINT

Fig. 4 shows the Load-Displacement Curve which was obtained from compression tests on a conduit joint of conduits of 220 m in total length. Yielding load was about 20 tonf and maximum load was 26.7 tonf. Yielding displacement and displacement of maximum loading point were about 1 mm and about 3 mm, respectively. Axial stress on the conduits which corresponded to yielding load and maximum load were 1785 kgf/cm^2 (0.085%) and 2330 kgf/cm^2 (0.111%), respectively. As shown in Photo. 2, for the inside of a joint, the screwed part of one conduit was pushed in causing it to penetrate and ride over the inside surface of the other conduit.

SIMULATION EXPERIMENT OF DAMAGE IN A SOIL TEST TANK

To simulate the damage to a conduit joint, one conduit was buried in a soil test tank made of steel (LBD = 5×2×2 m) and was deformed by applying an axial force using a hydraulic jack, as shown in Fig. 5. To investigate the effect of spring properties of the ground, hard condition (Case H/N = 17 to 21) and soft condition (Case S/N = 4 to 5) were used for test ground. Spring properties of the ground in the direction rectangular to the pipe axis were directly measured by pulling up a single pipe as shown in Fig. 6.

Fig. 7 shows the Load-Displacement Curve which were obtained from experiments on the spring properties of the ground. As shown in the figure, there is a large difference between the spring properties of the ground in Case H and Case S. Namely, in Case H, in the early phase of deformation where displacement is less then 5 mm, the spring factor is so large that k = 29.3 kgf/cm^3, and the load shows decrease after the maximum load is reached with a displacement of 20 to 25 mm. It then decreases to about 70% of the maximum load in the phase of displacement of 100 mm. On the other hand, the Load-Displacement Curve in Case S shows a rounded appearance as a whole. Particularly, the spring factor in the early phase of deformation is k = 7.0 kgf/cm^3, and it shows an increase of load without clear yielding. Maximum load is reached in the phase of displacement of 100 mm, and after that load shows a gradual decrease.

The relation between load and displacement, obtained by simulation experiment, is shown in Fig. 8. Maximum loads are 23 tonf for Case H and 21 tonf for Case S which are nearly the same, and proportional factors in the early phase of deformation are 49.8 tonf/cm for Case H and 40.5 tonf/cm. In this case, such a large difference in spring properties as in Fig. 7, is not observed. The joints of specimens which were dug out show deformations as shown in Photos 3 and 4. These deformations simulate actual damage very well. In addition, protective conduits which were connected to sockets, were bent by rotating deformation, and plastic deformation remained.

ANALYSIS OF DAMAGE

For long column buckling of a straight elastic beam which is placed on an elastic support, and has infinite length, the fundamental differential equation is expressed by (1). The buckling load N and the buckling wave length L are given by (2) and (3) respectively.

$$EI\frac{d^4y}{dx^4} + N\frac{d^2y}{dx^2} + k_L Dy = 0 \quad (1) \qquad N = 4EI\beta^2 \quad (2) \qquad L = \frac{\pi}{1.414\beta} \quad (3)$$

Where $\beta = (k_L D/4EI)^{1/4}$. Figs. 9 and 10 show the results of calculations for the buckling load (Eq. (2)) and buckling wave length (Eq. (3)), taking k_L which is the spring factor of the ground in the direction rectangular to the pipe axis, as parameter.
L = 0.98 m, N = 470 tonf for Case H, and L = 1.40 m, N = 240 tonf were obtained. In these cases, apparent strains generated in the protective conduits are 1.90% and 0.97%, respectively and both values exceed the elastic limit (0.085%) of strain which is

generated in the conduits. Thus, elastic buckling in the long column mode is found not to occur in the ground condition of the soil tank.

Consideration is now given to the case in which the threaded joint shows plastic deformation under axial force as shown in Photo. 2, and shifts to deformation by rotative movement of the socket as shown in Photos. 3 and 4. In this case, conduits becomes elongated because of the loss of axial rigidity of the socket, and rotative movement of socket results. Hence, the total length of conduits is assumed to become shorter by the rotative displacement of the socket. The behavior of conduits at the time of an earthquake can be estimated by approximating the half of value of reduction as displacement of free and of semi-infinite straight conduits in uniform field of strain of ground.

When displacement of the ground is distributed in an axial direction of a semi-infinite straight conduits, the equation of balance is expressed by (4). Assuming the spring property of the ground by elasticity to complete plastic body, displacement of pipe end Upe is expressed by equation (5), and the length of slipping ℓ_{slip} is expressed by equation (6) [5]. Results of calculations using equations (5) and (6) with U_{cr} = 0.1 cm are shown in Figs. 11 and 12.

$$EA\frac{d^2U_P}{dx^2} + \pi Dk_L\,[U(X) - U_P(X)] = 0 \qquad (4)$$

$$U_{pe} = \frac{1}{2\alpha^2U_{cr}}(\varepsilon_g^2 + \alpha^2U_{cr}^2) \qquad (5)$$

$$\ell_{slip} = \frac{1}{2\alpha^2U_{cr}}(\varepsilon_g - \alpha U_{cr}) \qquad (6)$$

where, $\alpha = (\pi Dk_L/EA)^{1/2}$, k_L : Spring factor and U_{cr}: Relative displacement of yielding.

As described formerly, assuming that ε_g = 0.35% and displacement of the pipe end is 10 cm, $k_L \fallingdotseq 5.5$ kgf/cm^3 is obtained from Fig. 11. Frictional force of the ground is estimated to be about τ = 0.55 kgf/cm^2 because U_{cr} = 0.1 cm. This frictional force is significantly larger than the frictional force in a completely liquefied state, so the strength of the ground in the neighborhood of the conduit is assumed to be little reduced. The length of slip is estimated to be about 30 cm from Fig. 12.

CONCLUSION

Earthquake damage to conduits for buried communication cables, which was caused by Nihonkai-chubu earthquake, was analyzed in relation to permanent displacement of the ground, and simulation experiments were carried out using a soil tank. The results of the expriments showed that a compression strain of 0.35% was caused in the ground in an axial direction by permonent displacement. Hence, the collapse of the joint is assumed to be caused by this compressive strain. Spring properties of the ground at the time of generation of permanent displacement was estimated from fundamental analysis of deformation. Frictional force in the neighhorhood of the conduit was τ = 0.55 kgf/cm^2 at least, and this value is assumed to be sufficiently larger than the value in the state of complete liquefaction.

FUTURE DIRECTIONS

We have studied the cause of the conduits damage due to each of ground motion and ground deformation. Particularly, as concerns ground deformation, the actual damaged conduits by Nihonkai-chubu earthquake was an object of study, and we have investigated the cause of the conduits damage due to ground deformation with analytical method. The experiment of this paper has two purposes. One is simulation of conduit damage by Nihonkai-chubu earthquake, and the other is verification of propriety of above analysis results.
In the future, we will applicate these results, and investigate how to construct the high reliability conduit facilities and reinforce of earthquake resistant for existing conduit facilities.

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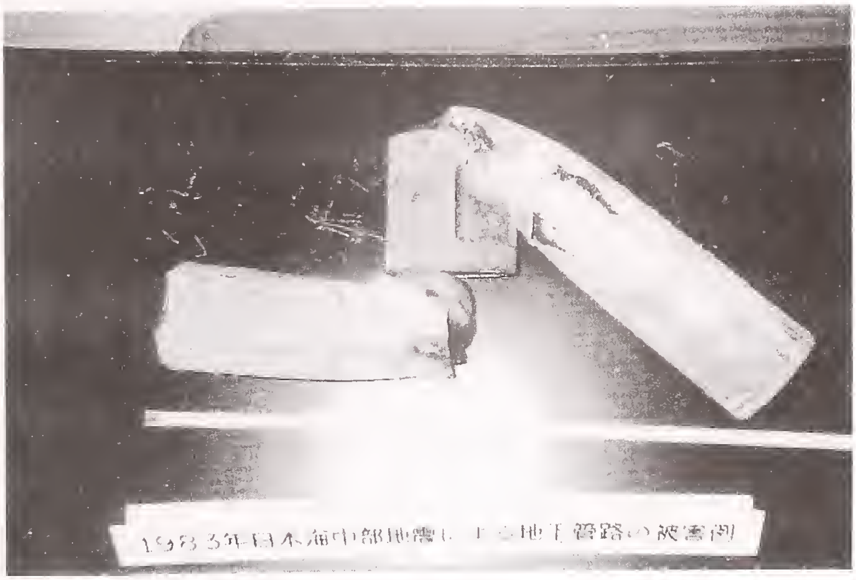


Photo. 1　Example of Telecommunication Conduits damage caused by Nihonkai-chubu Earthquake

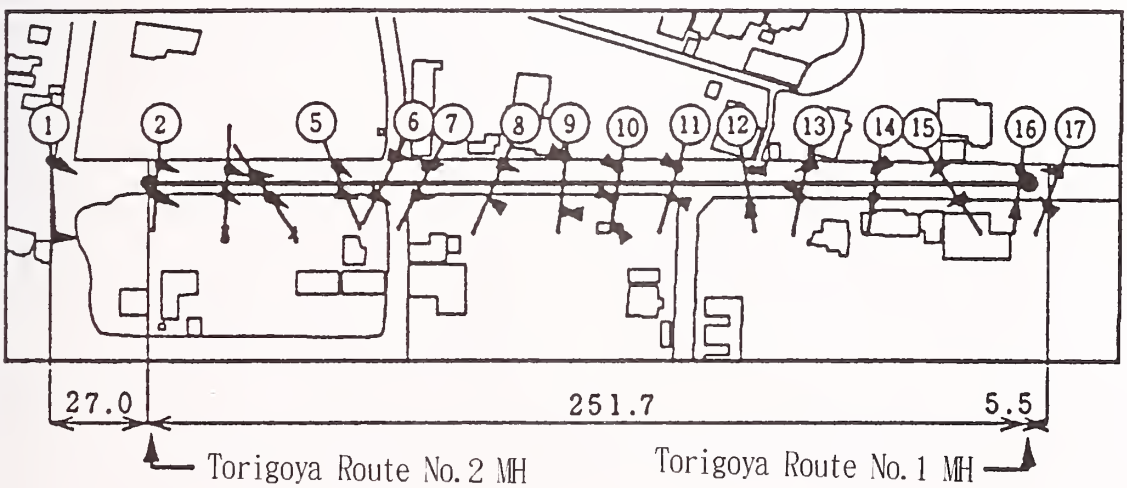


Fig. 1　Laying line of Telecommunication Conduits in the interval of damaging.

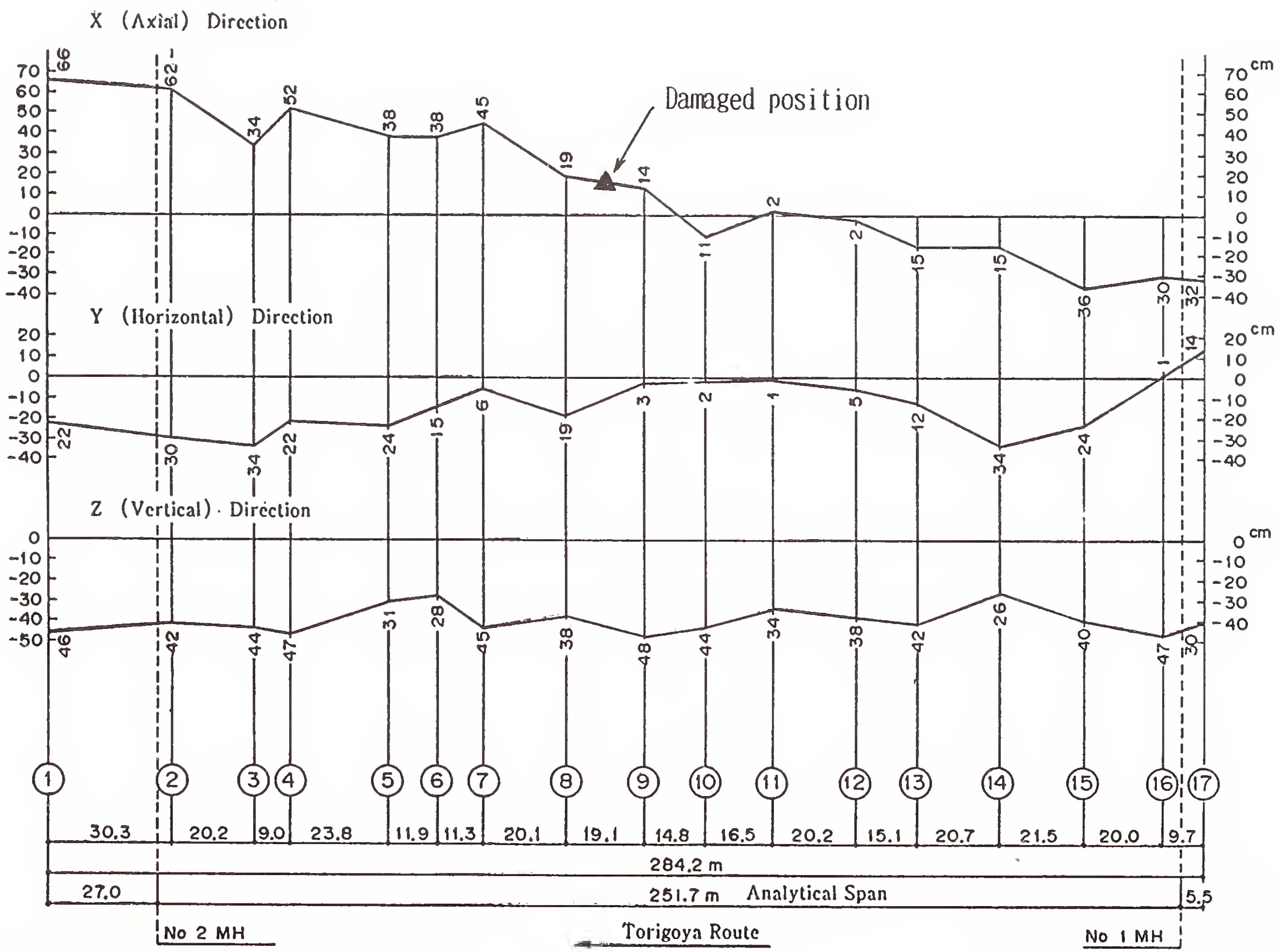
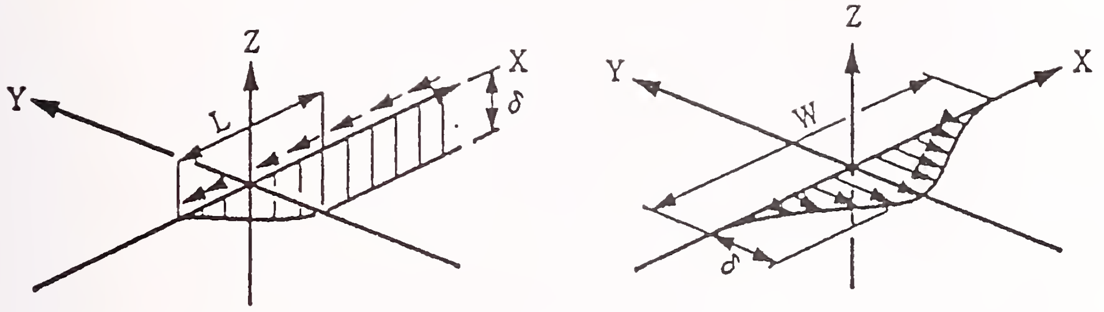


Fig. 2 Distribution of permanent displacement in the damaged interval.



① Displacement in the direction of pipe axis

② Displacement in the direction rectangular to pipe axis

Fig. 3 Model of lateral shift of ground

	L	δ
Displacement in the direction of conduit	L≦50	3/100*L
	50<L<400	1.5
Displacement in the direction of rectangular to conduit	W≦150	1/100*W
	150<W<500	1/300*W+1

(m)

Table 1 Upper limit of distribution of permanent displacement (lateral shift)

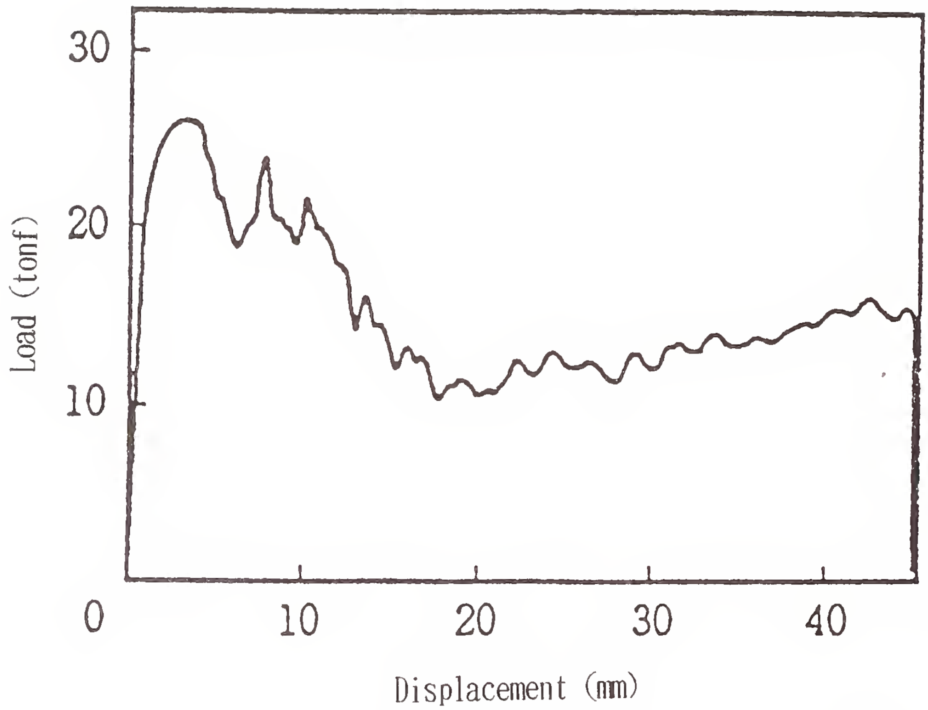


Fig. 4 Load to Displacement curve of threaded joint

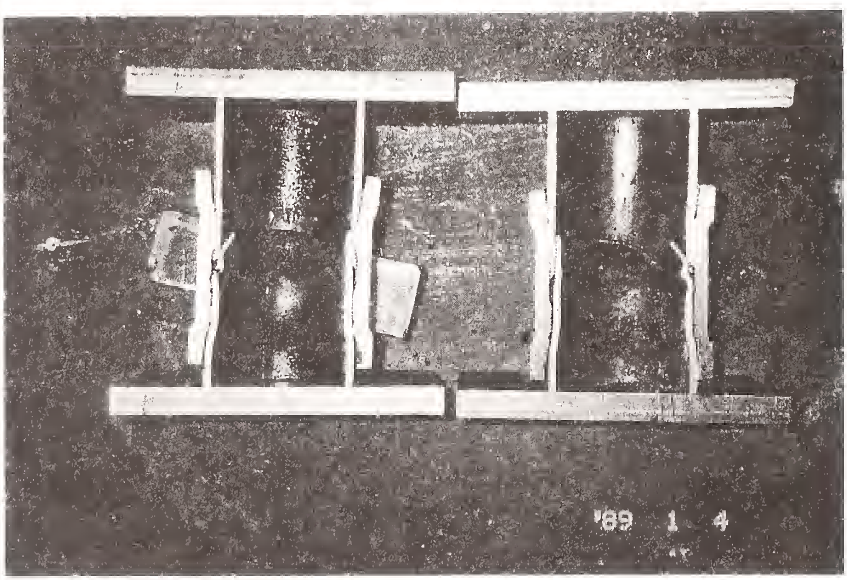


Photo. 2 Section of joint after compression test

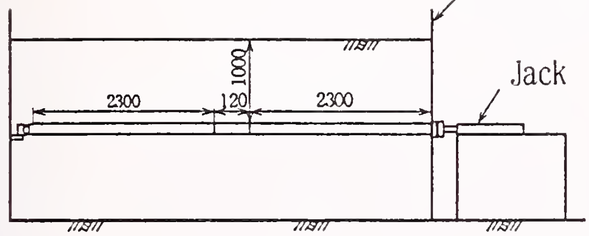


Fig. 5 Simulation experiment of damage

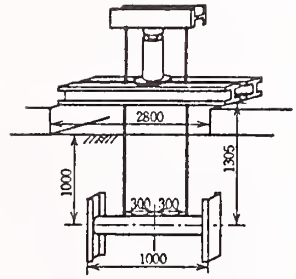


Fig. 6 Measuring method of ground spring property

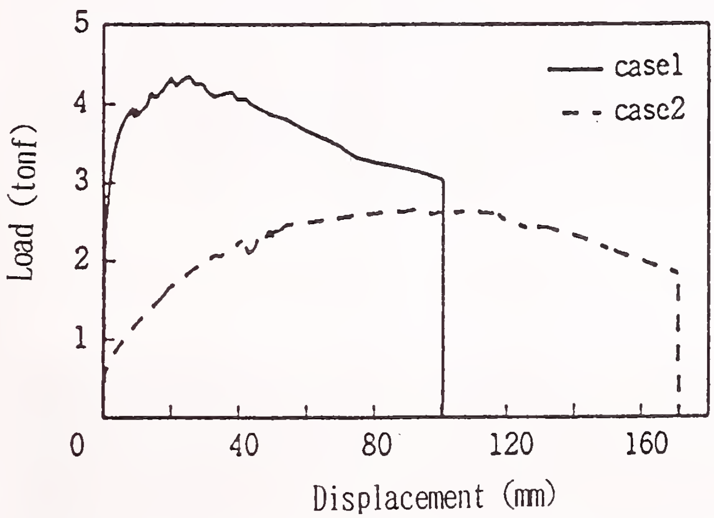


Fig. 7 Vertical and upward ground spring property

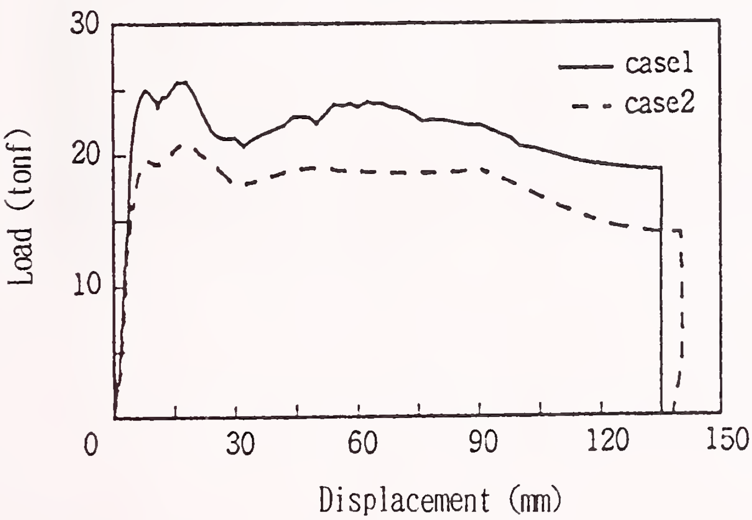


Fig. 8 Load to Displacement Curve obtained by Simulation Experiment

Photo. 3 Specimen after test (Case H)

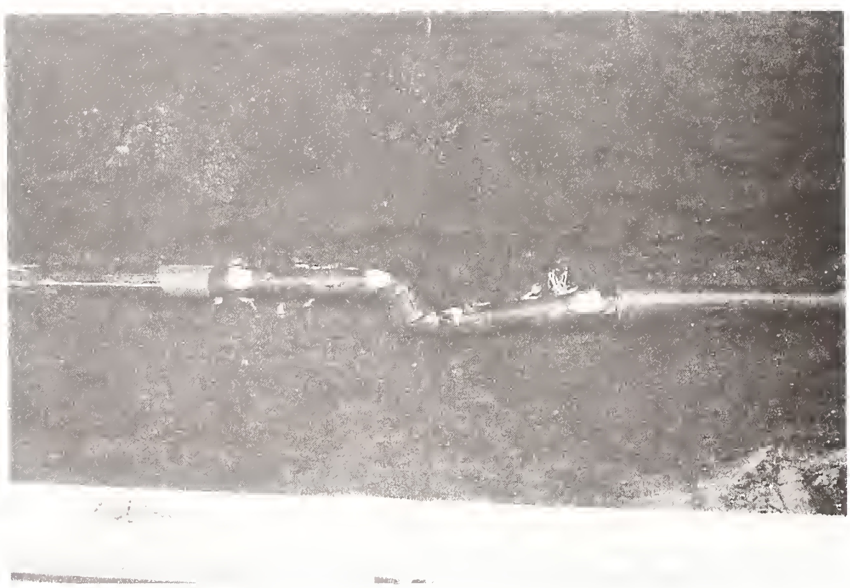


Photo. 4 Specimen after test (Case S)

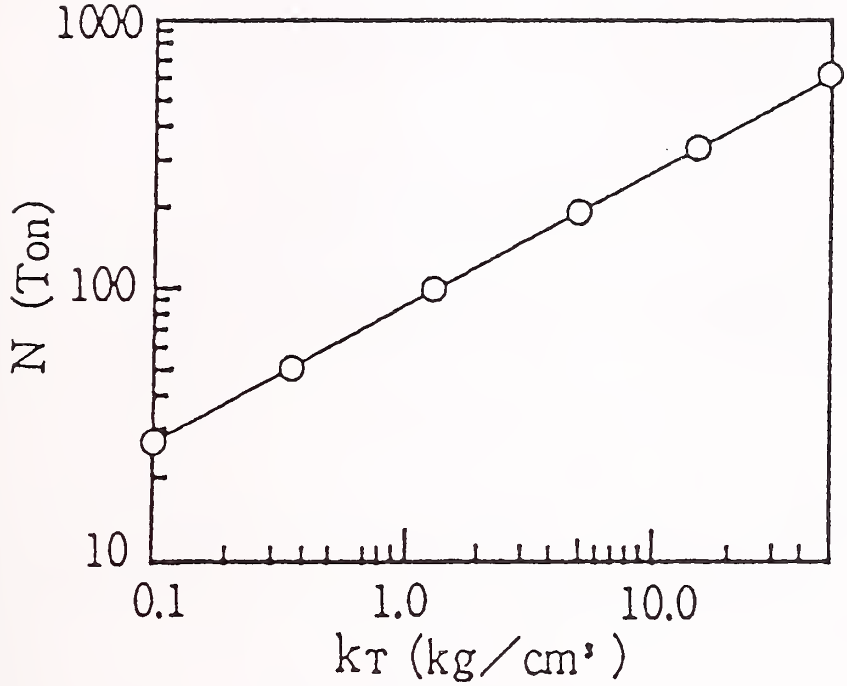


Fig. 9 Buckling load

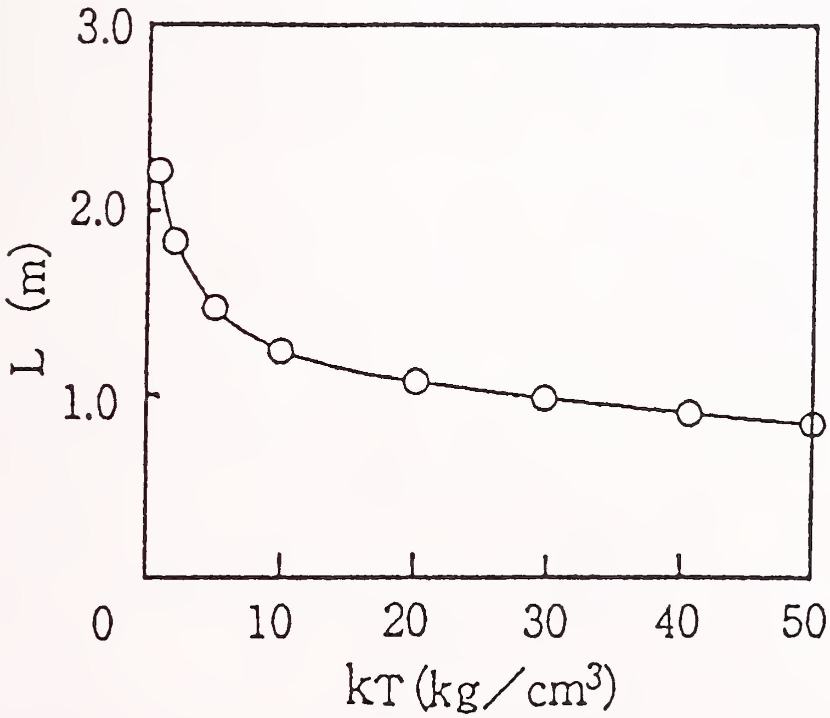


Fig. 10 Buckling wave length

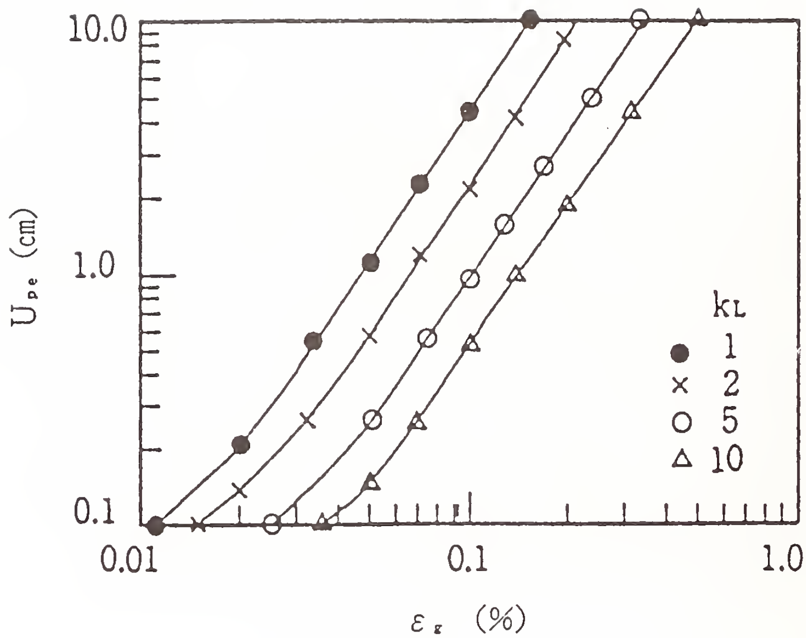


Fig. 11 Displacement of semi-infinite straight conduit

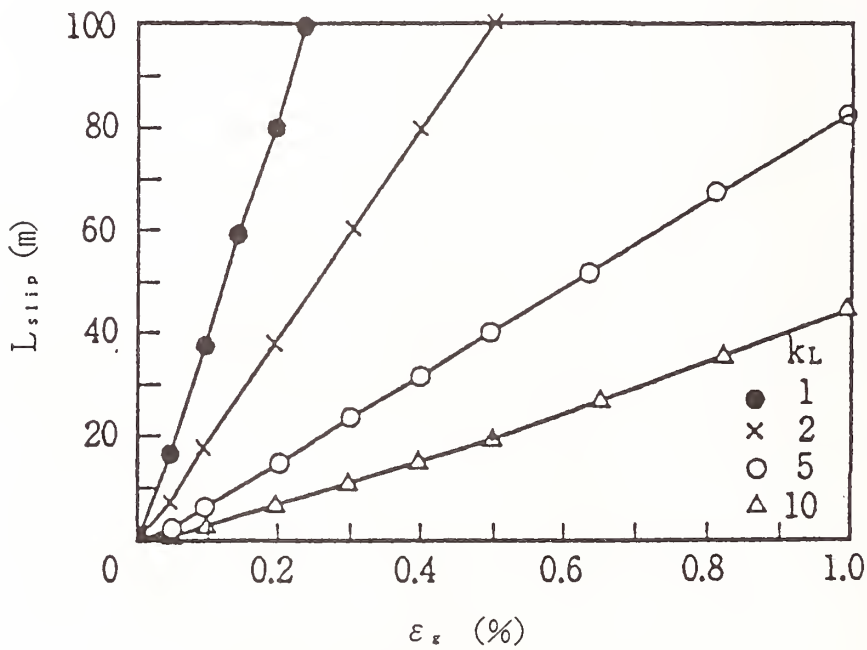


Fig. 12 Slip length of semi-infinite straight conduit

REGIONAL RISK ASSESSMENT OF ENVIRONMENTAL CONTAMINATION FROM OIL PIPELINES

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ABSTRACT

This paper presents a methodology for assessing the risk of environmental contamination from oil pipeline leaks due to earthquakes. Risk is measured both as volume of oil released and remediation cost. The methodology was developed for use on a regional scale and thus relies on a limited amount of input data. Monte Carlo techniques are used to simulate earthquake events, while a deterministic model is used to estimate the volume of oil released at a particular site. A library of cost models is used to estimate the contamination and resulting remediation cost based on the volume of oil released and the general site conditions. This methodology has been implemented in a computer program, OILOSS, and the results are presented as frequency of exceedence curves for volume of oil released and cost of remediation. The methodology is applied to two crude oil pipelines near the New Madrid Seismic Zone (NMSZ) and preliminary results presented. This study is being sponsored in part by a contract with the National Center for Earthquake Engineering Research (NCEER).

INTRODUCTION

There are currently many seismically-active areas in the United States with petroleum pipelines. While there have been many incidents of piping to tanks breaking during earthquakes, there have not been any major incidents in the United States involving transmission lines. This has been in part due to the relatively short time frame that large transmission lines have been constructed and the few number of major earthquakes in recent history in the United States.

In other countries, however, there have been significant losses resulting from petroleum transmission pipelines damaged during earthquakes. Most notable is the loss associated with the 1987 Ecuador Earthquakes of Richter magnitude 6.1 and 6.9 (Crespo, et al. 1987). In these events, two areas of the Trans-Ecuador pipeline, 6.5 and 10 mi long, were severely damaged. This resulted in the loss of at least 70,000 bbl of crude oil, complete disruption of oil export, and major rebuilding costs.

With the increasing possibility of a major earthquake in the New Madrid Fault Zone, there is also the possibility of damage to major pipelines carrying crude oil from the south to refineries in the midwest. Loss of a single pipeline would not create a complete disruption of oil flow to the refineries, however, it could have a significant impact. The losses may be direct, such as environmental contamination, or indirect, such as business interruption losses. The present methodology addresses the direct losses associated with environmental contamination. Other study elements within the larger NCEER project are focussing on other socioeconomic losses attributed to oil pipeline spills.

The approach used here is based on probabilistic methods. Another common approach for assessing loss potential is to use a worst case or probable maximum loss (PML) approach. In these methods, though, the likelihood of the assumed break locations and their associated losses ever occurring are rarely estimated. A risk based approach, such as that presented here, is preferable because each loss level is accompanied by a concrete estimate of the likelihood of the loss occurring or being exceeded. This enables a realistic approach to preparedness or mitigation planning.

The general approach and simulation procedure is first presented, followed by a description of the contamination and cost estimation models. An example is then discussed. The paper concludes with some comments on future applications.

APPROACH

In developing this methodology, there were three primary objectives:

1. the methodology should be applicable on a regional basis without excessive computational or input requirements,

2. the models should utilize available data, but not be dependent on site-specific information,

3. the output should provide probabilistic estimates of loss levels, rather than worst-case values.

These objectives, which include a pragmatic approach to using regional seismic hazard data, essentially dictate the choice of models used for estimating the volume of oil released, the extent of contamination, and the associated remediation cost.

The general approach followed in this study was to use Monte Carlo techniques to simulate earthquake events. Although the primary focus of this paper is on seismic events, both earthquake and normal operational risks can be assessed using this approach. In the case of seismic events, each simulation trial represents a possible outcome of the earthquake. The results of the entire simulation allow us to represent the distribution of losses which may occur from a single earthquake event. To obtain a distribution of losses for all earthquakes possible in the region, a series of events is simulated and the results combined and weighted based on their probabilities of occurrence.

The procedure for estimating losses due to a single event is outlined in Figure 1. A library of cost models is used to determine the remediation cost. This cost is estimated once the appropriate environmental model has been selected (i.e. river crossing, flood plain, etc.) and the volume of oil released has been estimated. The environmental and oil release models are described in later sections of this paper. Because the cost estimates do not depend on a particular event, they are evaluated prior to the simulation and accessed via data arrays.

Pipeline vulnerability models developed in Eguchi (1983) and updated in Eguchi et al. (1989) are used to determine the failure probability of each pipe segment that is exposed to an earthquake hazard. For each simulation trial, a random number is selected and then compared to the failure probability of a pipe element to determine if the pipe element has broken.

Once the locations of all breaks for a particular trial are known, a deterministic "drain down" model is used to estimate the volume of oil released at each location. This model assumes that all oil originally above the break drains out, with the exceptions of oil trapped in valleys, and oil blocked by valves and pump stations. Conservation of mass is maintained such that the same oil can only leak out of one location. This type of model is in some ways overly conservative because it does not consider breaks of different sizes or the time required for the pipe to drain. However, the model does provide an estimate of the volume which could leak out if the pipe were to break and the leak were uncontrolled. The drain down model is also easily applied, that is, the only data required to estimate drain down volume is the elevation at the endpoints of the pipe segments and the pipe diameter.

With the volume released at each location known and the appropriate environmental cost model specified, the remediation cost can then be determined. The losses, both volume and dollar, are aggregated for all pipe failures simulated in the analysis to determine the total losses for the event. At this point, the simulation trial is completed and a new set of break locations are generated. The process is continued until stable distributions of loss are obtained. The number of trials required is dependent on the number of pipeline elements and system failure probability. After all trials are completed, annual frequency of exceedence curves are formed by normalizing the resulting frequency curves by the number of trials performed, and then unconditionalizing these curves by the annual probability of the earthquake.

LIBRARY OF COST MODELS

A library of cost models was used to determine the dollar loss associated with each leak. These models estimate both the extent of environmental contamination and the cost of remediation. Five generalized models were developed to approximate the various types of contamination which may occur. The appropriate library model for each pipe segment is assigned based on the general topography of the area, depth to ground water, and proximity to surface water.

The contamination models use Darcian flow to estimate the vertical extent and rate of oil migration in soil. Using this analytic model, an estimate of the scale of ground water contamination can be made. Surface water is contaminated if the break is near a river, lake, or wetlands. Estimates of remediation costs for soil, ground water, and surface water were obtained from experts in the field. The models were formulated such that the type of contamination and its associated cost could be estimated based on the library model specified and the volume of oil released.

Even with these relatively simple analytic models for contamination, there are many parameters which significantly effect the remediation cost. Furthermore, to be general, the models must be applicable for any season of the year and a variety of soil conditions. To obtain an estimate of the average cost for a given library model and volume released, event trees were used. The parameters in the event trees included variables such as depth to water table, soil permeability, oil type, response time, and cost schedule. Each variable was allowed to take two or three states to approximate its range of values, and a probability was assigned to each state. The cost associated with the parameters in each branch of the tree was evaluated along with its likelihood of occurrence. For each model, the average or expected cost of remediation was obtained for the range of oil spill volumes specified. These arrays were used in the simulation procedure.

APPLICATION OF METHODOLOGY

The methodology has been applied to two major crude oil pipelines near the NMSZ. Modified Mercalli Intensity isoseismals for a M=8.6 earthquake anywhere along the NMSZ were developed by Algermissen and Hopper (1984). MMI's for an event anywhere along the seismic zone is approximately equivalent to a single earthquake with a very long rupture length, or a combination of events, such as the 1811-1812 series of earthquakes. This map was digitized and used as the strong ground shaking scenario. Figure 2 shows the strong ground shaking in the study area. A single event of this magnitude (MS >= 8.3) has a return period of 550 (+/- 125) years and 0.3-1.0 percent probability of occurrence by the year 2000 CE (Johnson and Nava, 1984). For this example a 0.5 percent probability of occurrence by the year 2000 is assumed.

A significant seismic hazard in the Mississippi Valley and along rivers is liquefaction-induced ground failure. Areas of moderate to high liquefaction potential in a large earthquake were mapped for seven states in the Central United States by Obermeier (1985). No distinction was made on these maps between areas of liquefaction and landslide, or areas of greater and lesser potential. These maps were digitized and combined into a single map. Figure 3 shows the areas of moderate to high liquefaction potential included as seismic hazard input to the simulation.

The Shell Capline and Mobil line number 68 were selected to demonstrate the methodology. These pipelines are shown in Figure 4. The Shell Capline is a 40 in diameter pipeline built in 1968 from API 5LX-X52 grade welded steel pipe with arc welded joints (Ariman et al., 1990). The Mobil pipeline is 20 inches in diameter in the south and 18 inches in diameter in the north. This pipeline was built around 1948 from grade B steel with welded joints.

The results obtained for this example are preliminary and should be viewed as such. The annual frequency of exceedence of the volume of oil released is contained in Figure 5. This volume is the sum of spill volumes from all leaks occurring in a single event. The range of total volumes from this scenario is 150 thousand to 2350 thousand barrels, with an expected volume of 400 thousand barrels. While this would be quite a large volume for a single location, the expected number of leaks in this scenario is 87, and the expected spill volume at a single leak site is 4700 bbl. The maximum frequency on this curve, 0.005, is the frequency with which at least one spill occurs. Because this is a severe scenario, at least one leak occurred in every simulation of the event. Therefore, the maximum frequency is also the probability of the event.

Dollar losses for remedial measures were also estimated for this scenario. Figure 6 contains the frequency of exceedence of dollar losses. The range of possible losses is 30 million to 2.4 billion dollars. This range is very large because of the large difference in the cost models between remedial measures for surface water and those for soil. In a simulation where an extraordinarily large number of leaks occurs, and many of them are in flood plains or rivers, the cost of clean up can be quite high. However, many more moderate than extreme cases are possible and the expected loss given the event is 310 million dollars. The expected loss at a single leak site is 3.6 million dollars.

CONCLUSIONS

The importance of using a probability or risk-based analysis can clearly be seen from this example. If a worst case analysis had been performed it is possible that losses in the 600 thousand bbl and 2 billion dollar range would have been determined, without an estimate of the likelihood they would actually occur. Compared with the annual frequency of exceedence curves, it is seen that the worst case analysis may produce loss values with frequencies three to four orders of magnitude less than the frequencies of the expected values.

This example also demonstrates the large variation in both spill volumes and remediation costs which can occur in a given earthquake. The present methodology provides estimates of the frequency of exceedence for the entire range of volumes and costs. Using these curves, the risk associated with the operation of existing pipeline systems or the design of new systems can be quantified.

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APPENDIX

1 bbl (barrel) = 42 U.S. gallons = 159.0 liters
1 mi = 5280 ft = 1609 m

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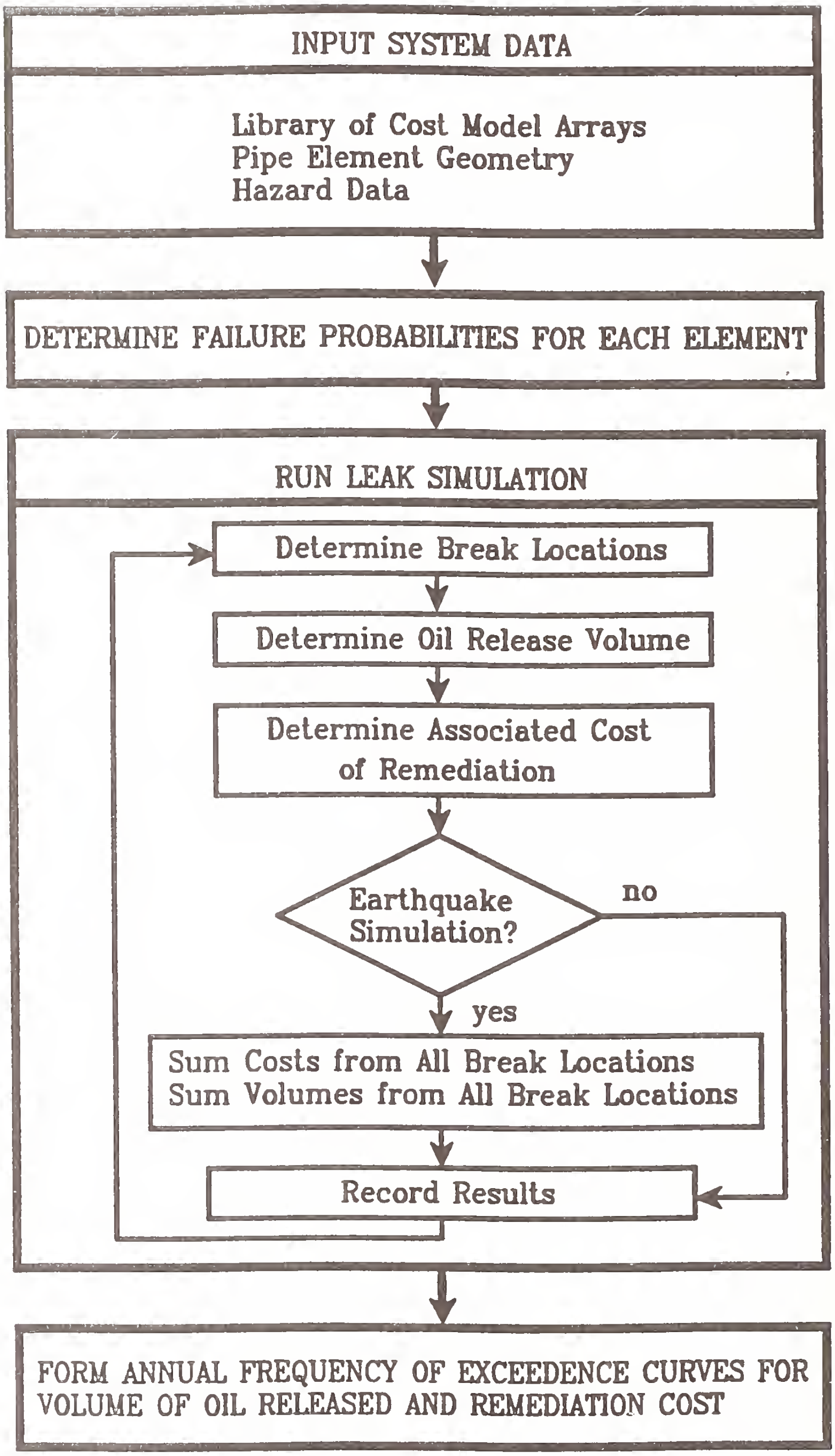

FIGURE 1. Procedure for Simulating Events to Estimate Frequency of Exceedence Curves

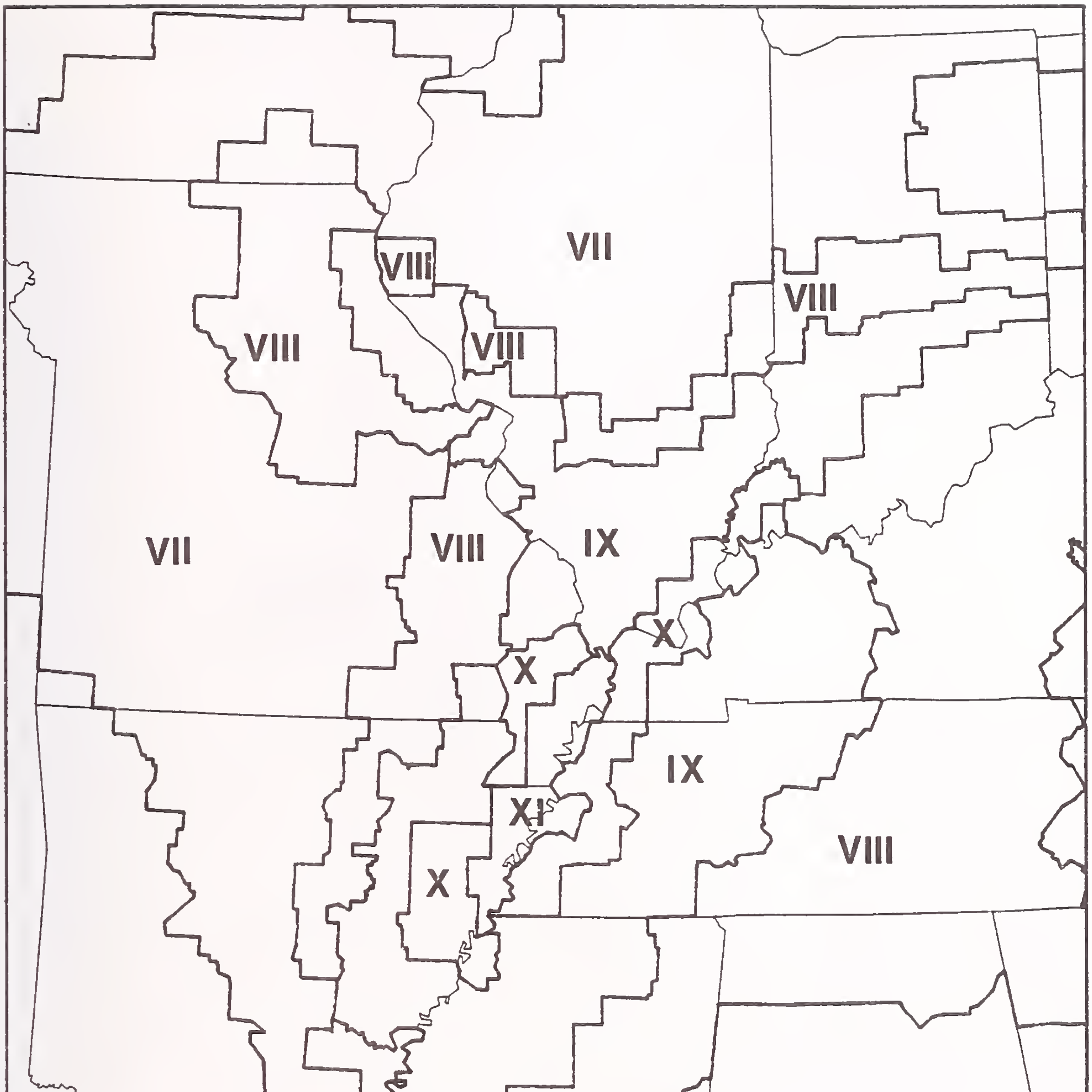

Figure 2. Modified Mercalli Intensity for a Magnitude 8.6 Event Anywhere Along the New Madrid Seismic Zone. (Digitized from Algermissen and Hopper, 1984)

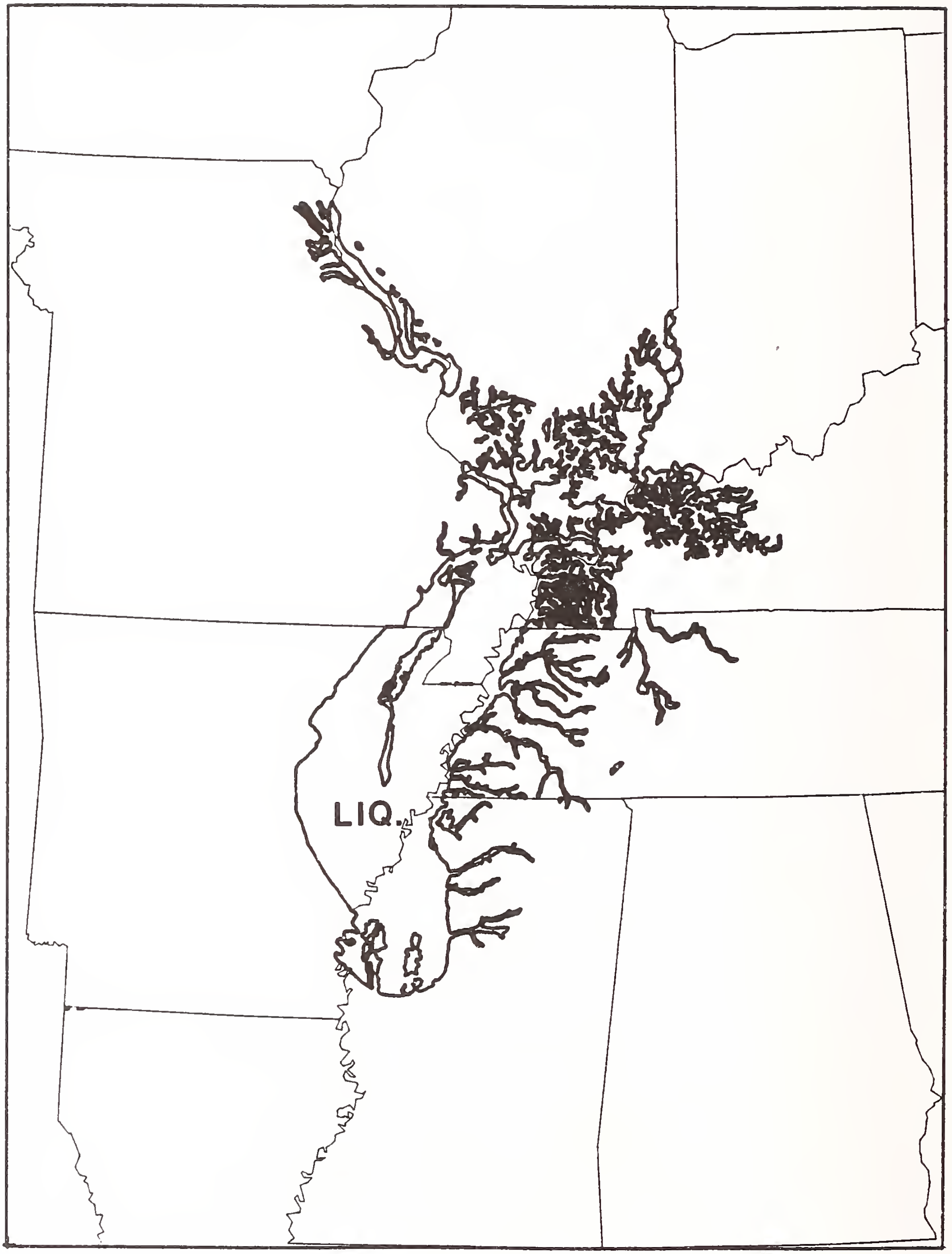

Figure 3. Areas of Moderate to High Liquefaction Potential in the New Madrid Seismic Zone During a Major Earthquake. (Digitized from Obermeier, 1985)

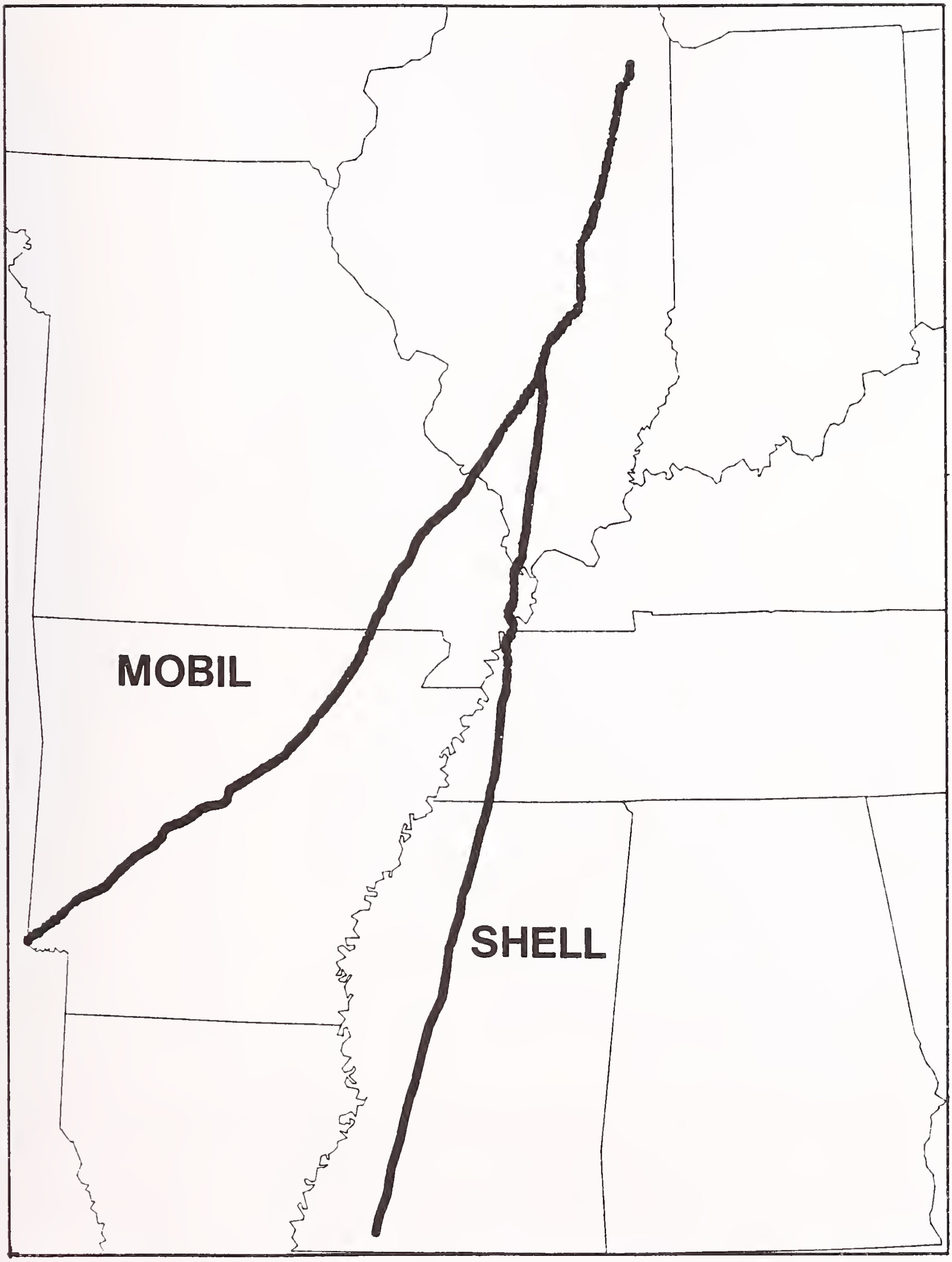

Figure 4. Pipelines Studied

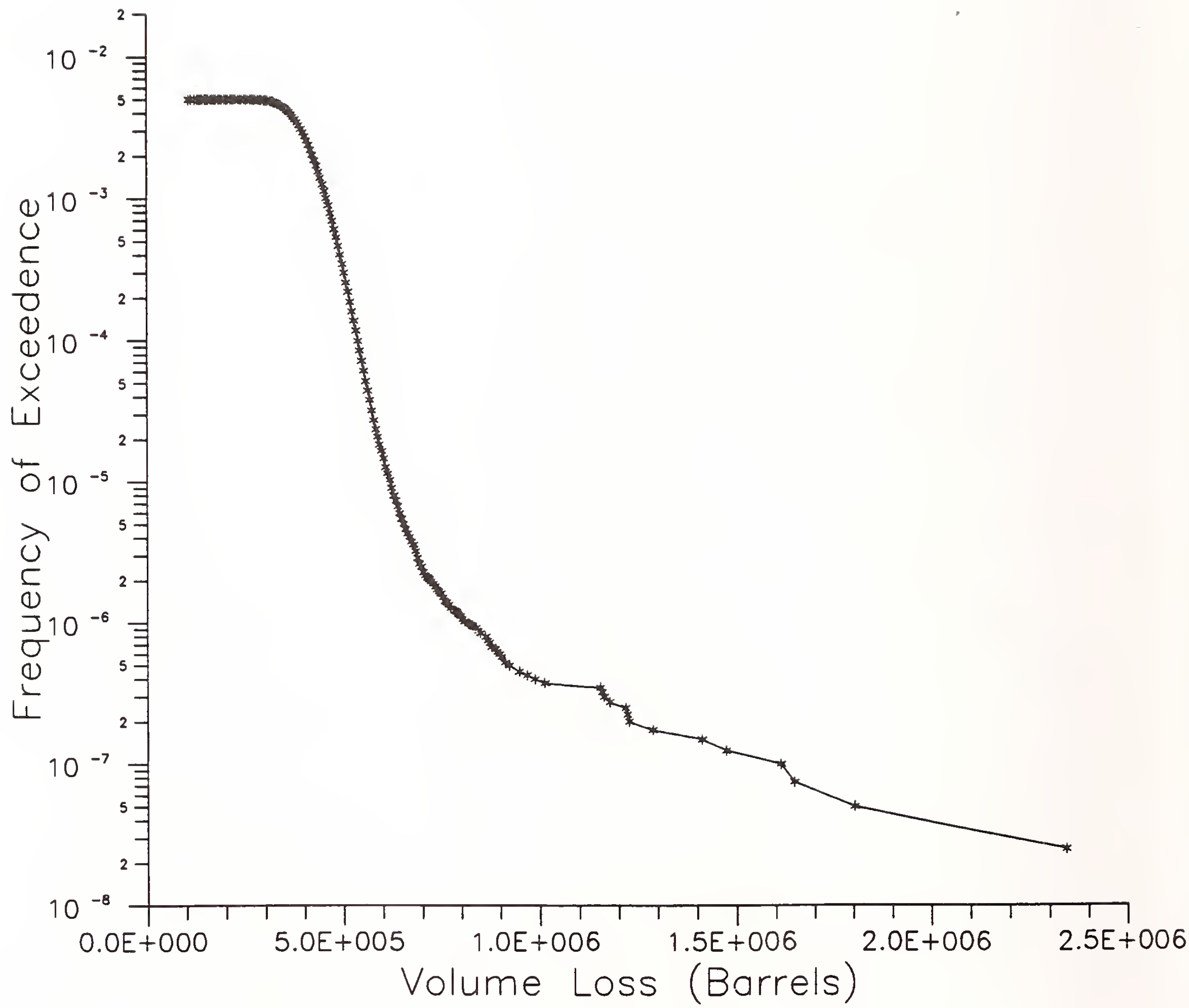

Figure 5. Frequency of Exceedence of Total Spill Volume in a Major Event in the New Madrid Seismic Zone

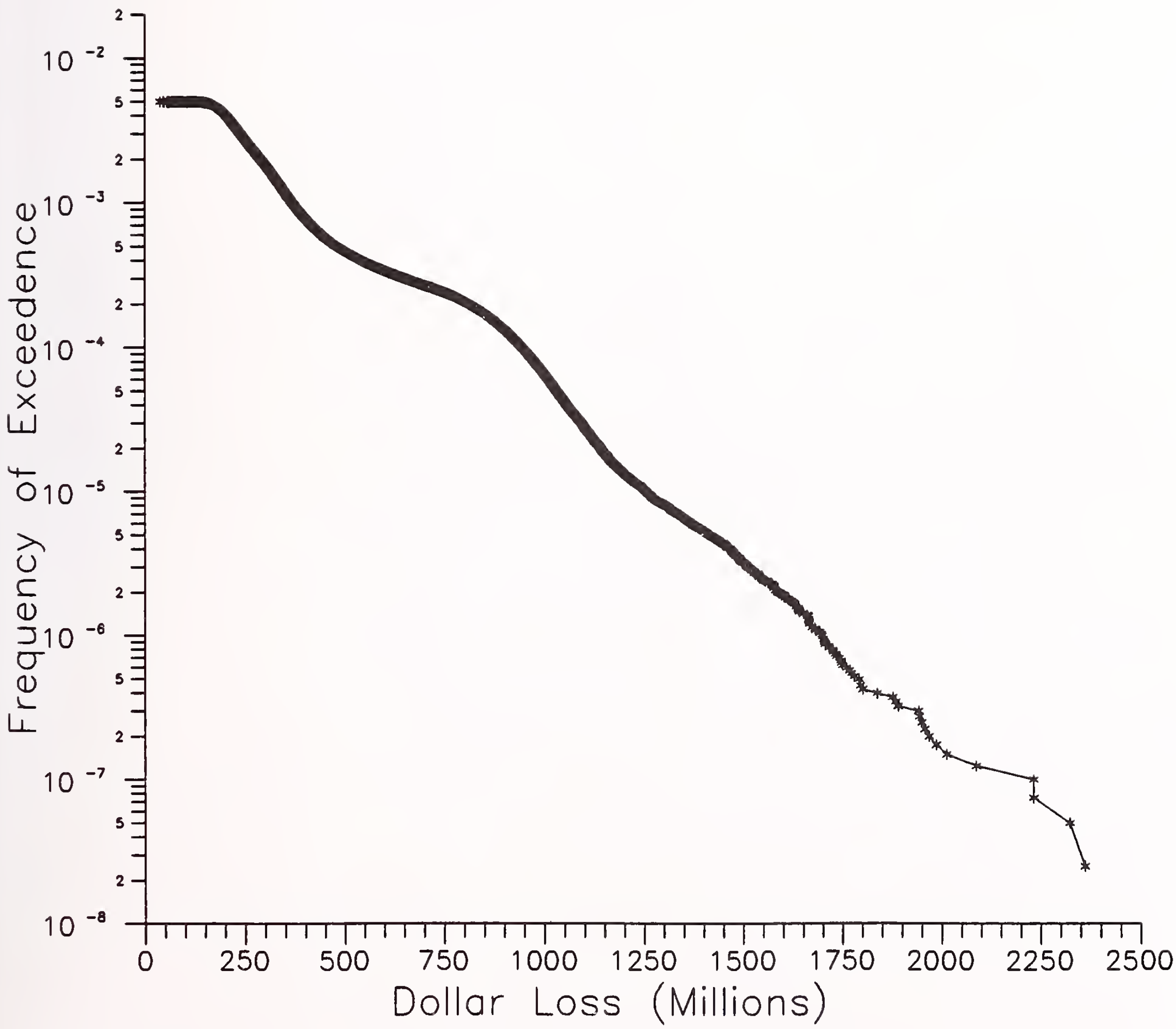

Figure 6. Frequency of Exceedence of Dollar Loss

DAMAGE ASSESSMENT OF LIFELINE SYSTEMS IN JAPAN

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ABSTRACT

This paper first introduces the current status of damage assessment of lifeline facilities, including the estimation of the loss of function. The methods of the estimation of earthquake motion on the bedrock as well as on the ground surface were outlined, and a general flow for the evaluation of the damage probability of the lifeline facilities is shown.

Secondly, the present paper describes the future research subjects such as dynamic response of soft ground, liquefaction-induced permanent ground displacement and effects of permanent ground displacement on the lifeline facilities, which shall be studied for the damage assessment with a high accuracy.

INTRODUCTION

Large-scale development projects are now proceeding on the waterfront in many major Japanese cities, including Tokyo and Osaka. These waterfront projects are often conducted on soft ground reclaimed ground from the sea or a river. In the 1989 Loma Prieta earthquake, lifeline facilities were seriously damage as a result of liquefaction of reclaimed land in the San Francisco Bay area. The Loma Prieta earthquake taught us the importance of earthquake resistant design of structures on and in reclaimed ground. Accordingly, local governments such as Tokyo Metropolitan Government and the companies providing lifeline services, such as electricity and gas companies, are now conducting assessments of the damage resulting from a strong future earthquake. Pre-earthquake measures to reinforce facilities and post-earthquake recovery strategies are also being investigated based on the results of the damage assessment. In these studies, the effects on lifeline facilities of liquefaction-induced ground displacement are also being examined. This paper describes the current status of damage estimation of lifeline facilities, including estimations of the loss of function and the research subjects still to be solved in making these estimations in Japan.

DAMAGE ASSESSMENT OF LIFELINE FACILITIES AND SYSTEM FUNCTION

The damage estimation of lifeline facilities, the loss of the function, and the recovery process follows the work flow shown in Figure 1. Lifeline facilities are generally distributed over a wide area, so seismic activity is surveyed for the whole area and an anticipated earthquake, the subject of the study, is determined based on the survey. For cities on the Pacific coast, such as Tokyo and Osaka, an undersea earthquake of the magnitude 8 class and an inland one of magnitude 7 class with the hypocenter right under a city, are sometimes assumed. If the earthquake under the ocean is assumed, the effects of tsunami must be taken into consideration because some lifeline facilities such as thermal power stations are located on the coast.

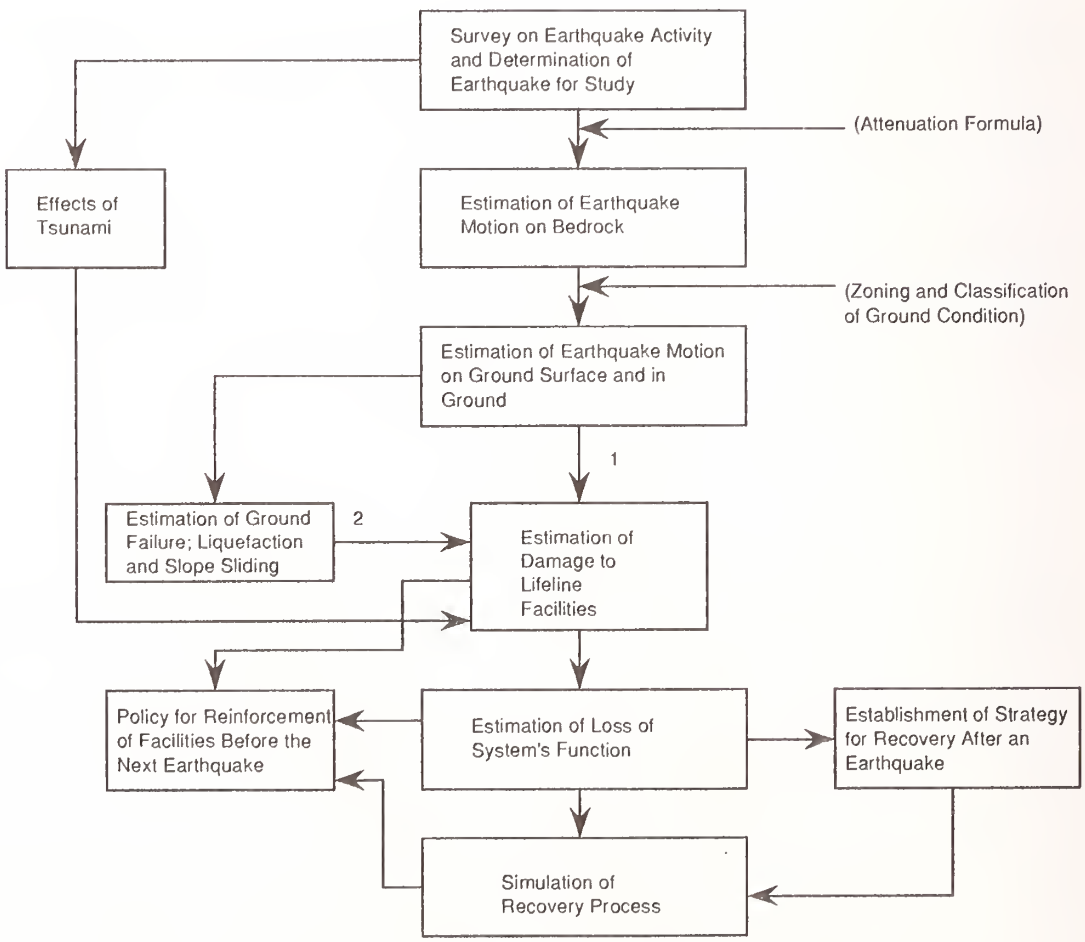

Figure 1 General Flow of Damage Estimation of Lifeline Systems

The earthquake motion on the bedrock throughout the area on which lifeline facilities are distributed is estimated by using attenuation formulae. In this case, correct estimation of the earthquake motion in the neighborhood of earthquake faults will be one of the important subjects, for the reason described later.

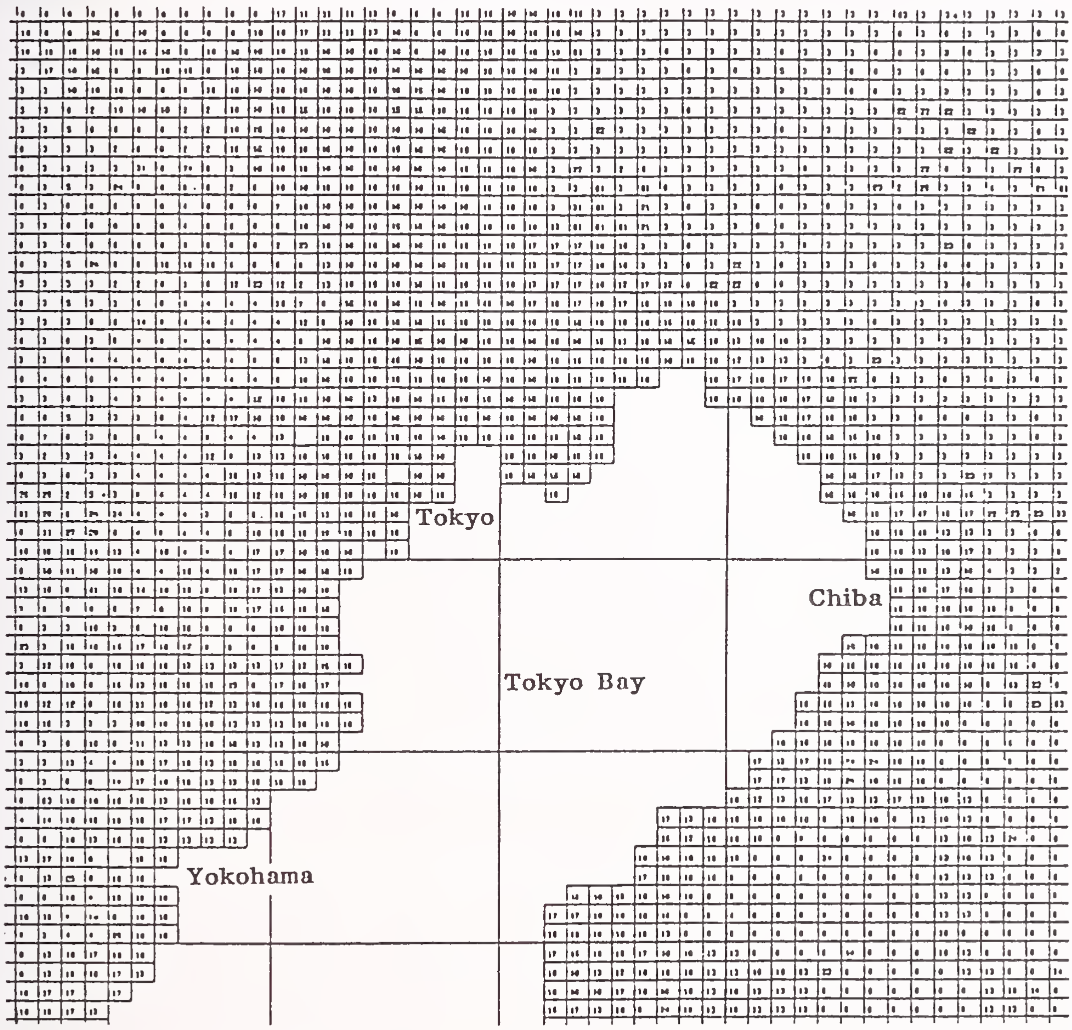

Figure 2 Example of Meshes for Calculation of Ground Response in Tokyo Metropolitan Area (1 km Mesh)

The next step is to estimate the earthquake motion on the ground surface as well as in the ground by dynamic response analysis, the input for which is the earthquake motion on the bedrock. Besides acceleration, velocity, and displacement, it is required to determine the relative displacement and strain in order to estimate the damage to buried lifeline facilities. Since lifeline facilities are distributed over a wide area, the area of interest is generally divided into meshes with a size of 1 kilometer to 500 meters, and the earthquake motion for each mesh, where the ground conditions is assumed to be uniform, is calculated. For example, the ground in Tokyo area is divided into about 10,000 meshes as shown in Figure 2 with 150 kinds of ground conditions.

Based on the calculated earthquake motion on the surface and in the ground, the probability of liquefaction and slope sliding is examined. Since many areas of Japanese large coastal cities are constructed on reclaimed land, the exact estimation of liquefaction potential is one of the key subjects for damage assessment of lifeline facilities. The damage to lifeline facilities is estimated based on the calculated earthquake motion, by taking into consideration the effects of ground failure such as liquefaction.

The damage degree to facilities is generally represented as a probabilistic value. For example, the damage degree of buried pipes is given as the average number of failures per unit length of pipe or as the damage probability at connection point with other structures such as the manhole.

As for the effects of liquefaction, we must consider the effect of the permanent ground displacements, subsidence and heaving of ground and floating up of manholes. In places where lifeline pipes cross a river, the effects of the subsidence of the embankment behind bridge abutments must also be taken into consideration.

The following two methods can be used to obtain a damage probability. The first method is by empirical formulae obtained by statistically analyzing the damage caused to lifeline facilities in past earthquakes. For example, the following empirical formula is proposed for the buried pipes.

$$R_f = C_G \cdot C_L \cdot C_P \cdot C_E \cdot R_s$$

R_f: Damage probability of buried pipes
R_s: Standard damage probability
C_G: Factor by ground condition
C_L: Factor by liquefaction
C_P: Factor by pipe's material and diameter
C_E: Factor by strength of earthquake motion

Figure 3 shows damage rate of buried pipes (mean number of the failure points per 1 km) obtained from 1971 San Fernando and 1978 Miyagiken-Oki earthquakes. From these data the standard damage probability R_s can be determined. However, most of the damage resulting from these earthquakes were caused to pipes with a relatively small diameter and low strength, so it is necessary to make a correction to the standard damage probability according to strength and ductility when it is applied to strong, large-diameter pipes. Recently, large-diameter steel pipes or ductile iron pipes with flexible joints have come into wide use for lifeline system mains. It is one of the most difficult subjects to determine a correction factor suitable for these pipes, since almost no actual damage data are available.

The second method of the evaluation of the damage probability of facilities is to compare the stress, the strain and the deformation which are calculated based on predicted earthquake with the ultimate strength of the facilities. Figure 4 shows the process of calculating the damage probability for buried pipes. Besides the relative ground displacement and the ground strain resulting from earthquake motion, the permanent ground displacement due to liquefaction is taken into consideration in the calculation of the stress, the strain and the deformation of buried pipes.

The functionality of lifeline system after an earthquake is evaluated based on the damage assessment of the facilities. A lot of numerical methods of network analysis have been proposed. However, practical networks of lifeline facilities consists of an enormous number of elements and such an analysis covering all elements is actually impossible. Thus, to achieve the objectives, some simplifications are conducted by taking into account the characteristics of the network. An electricity supply substation, for example, consists of many transformers, circuit breakers, and other components, but is substituted by a simple system with fewer elements upon the judgement of experts.

The post-earthquake recovery strategy right after an earthquake is studied on the basis of estimated damage to facilities and the function. The recovery process is simulated in accordance with several probable recovery strategies, and based on the simulation, the best strategy is chosen by referring to expert's opinions. The pre-earthquake policy for the reinforcing facilities is also determined by the damage estimation and by the simulation of recovery process.

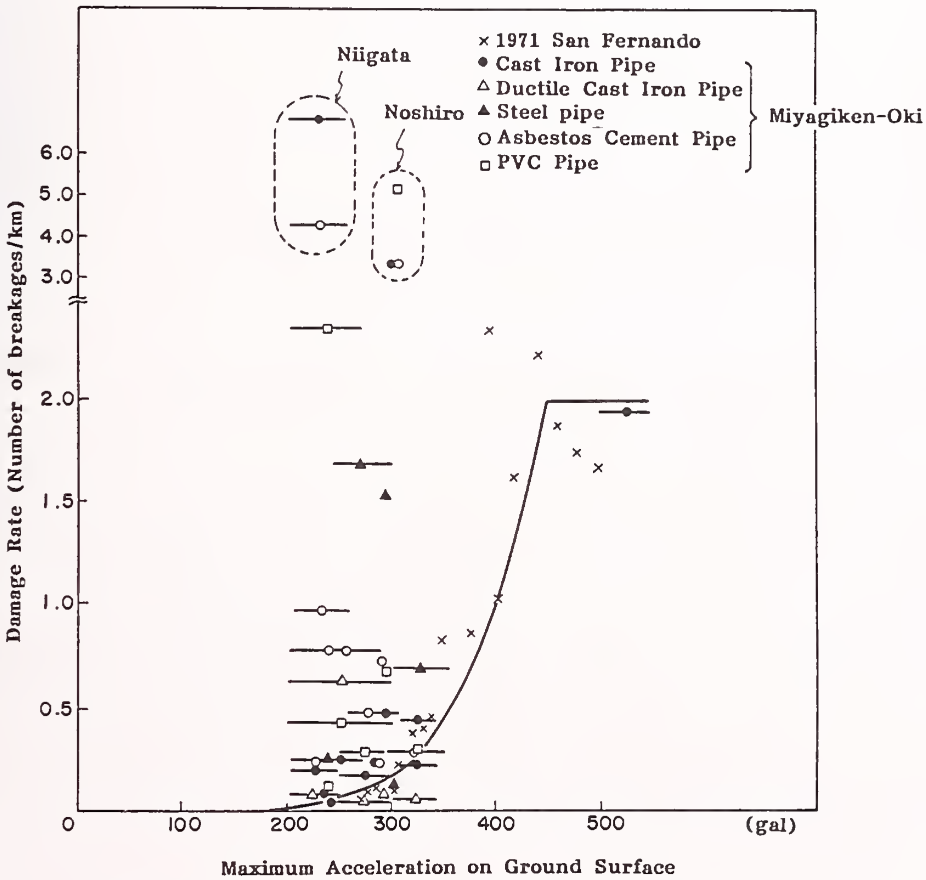

Figure 3 Water Pipe Damage Rate in 1971 San Fernando and 1978 Miyagiken-Oki Earthquake

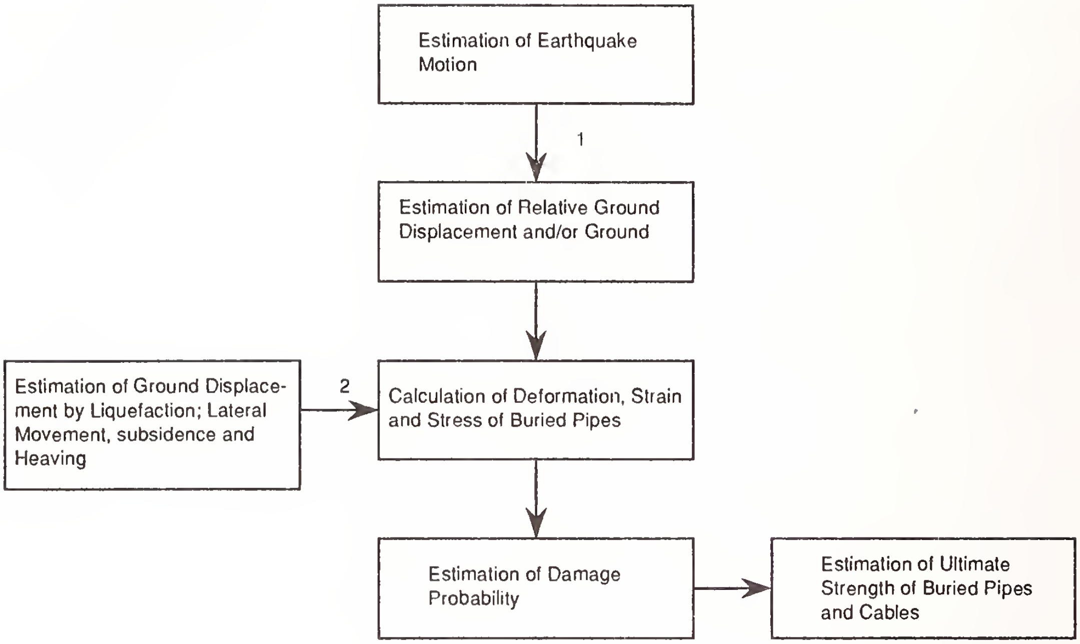

Figure 4 General Flow for Estimating Damage Probability for Buried Pipes

FURTHER SUBJECTS ON EARTHQUAKE RESISTANCE OF LIFELINE SYSTEMS

The areas needing future study to enable accurate damage estimation of lifeline facilities and their functions are described below.

Earthquake Motion in the Neighborhood of Fault

Many attenuation formulae, which take earthquake magnitude and epicentral or hypocentral distance as functions for estimating earthquake motion, have been proposed. Some examples are shown in Figure 5. Most of the earthquake records which were used to develop these attenuation formulae were collected relatively far from the earthquake fault during large or medium-magnitude earthquakes, or were measured during small-magnitude earthquakes. Earthquake motions which are recorded near faults are insufficient to establish a reliable attenuation formula. Thus, ground motions within an epicentral distance of 0-20 km due to earthquakes with a magnitude of 7-8 are estimated by extrapolation.

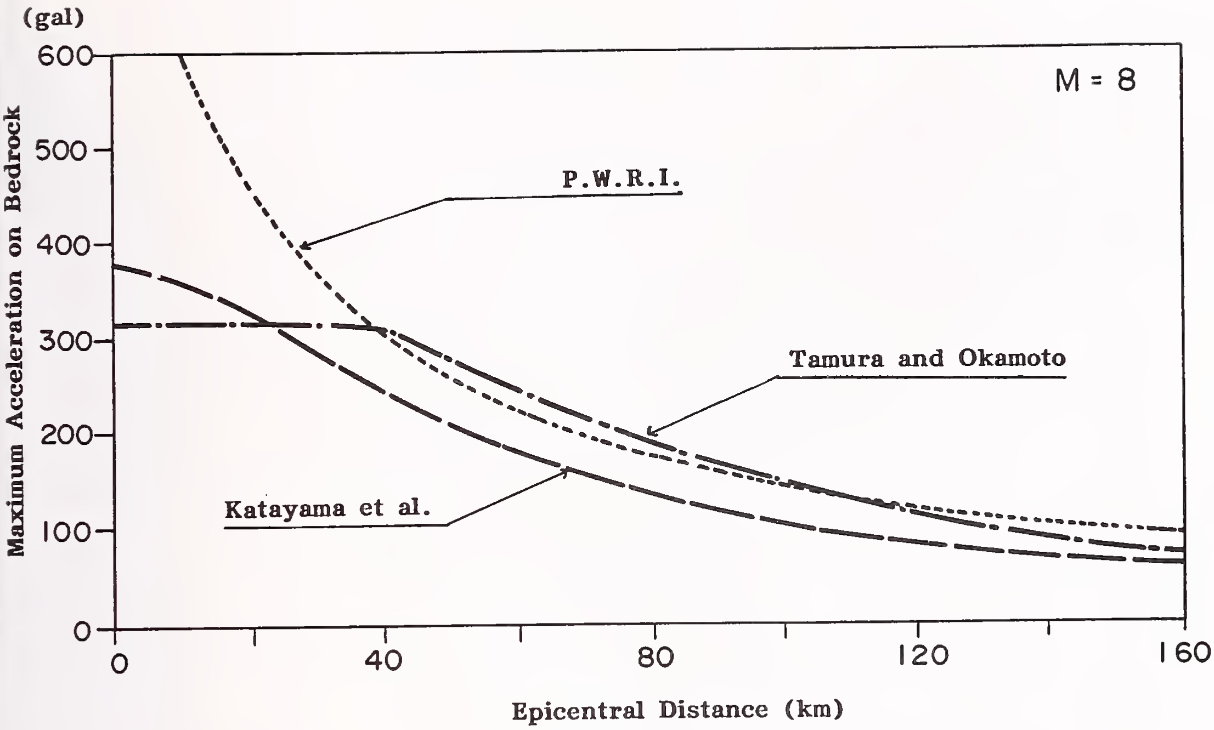

Figure 5 Attenuation Formula for Estimation of Ground Motion

When an inland earthquake occurs, some facilities are inevitably located in the fault zone because lifeline facilities are distributed over a wide area. The damage probability for facilities in the fault zone is high, and the degree of damage to those facilities has a great influence on the overall functioning of the system. Therefore, one of the most important tasks is to estimate correctly the earthquake motion in a fault zone for the damage estimation of lifeline facilities.

Figure 6 shows an example of a maximum acceleration map for the ground surface. This map was used to estimate the damage to lifeline systems resulting from a future earthquake in the metropolitan area. The maximum acceleration was calculated by non-linear dynamic analysis. According to this map, the maximum acceleration in Shinjuku Ward, on the diluvial plateau, is greater than that in Koto Ward along the Sumida River on alluvial low land. This map shows that the acceleration on soft ground is less than that on firm ground.

Dynamic Response of Soft Ground

In the Loma Prieta earthquake, the acceleration on reclaimed land around San Francisco Bay was two to three times greater than that on firm ground. And in the 1985 Mexico City earthquake, too, acceleration was amplified significantly on land reclaimed from the lake in the city and many tall buildings were seriously damaged.

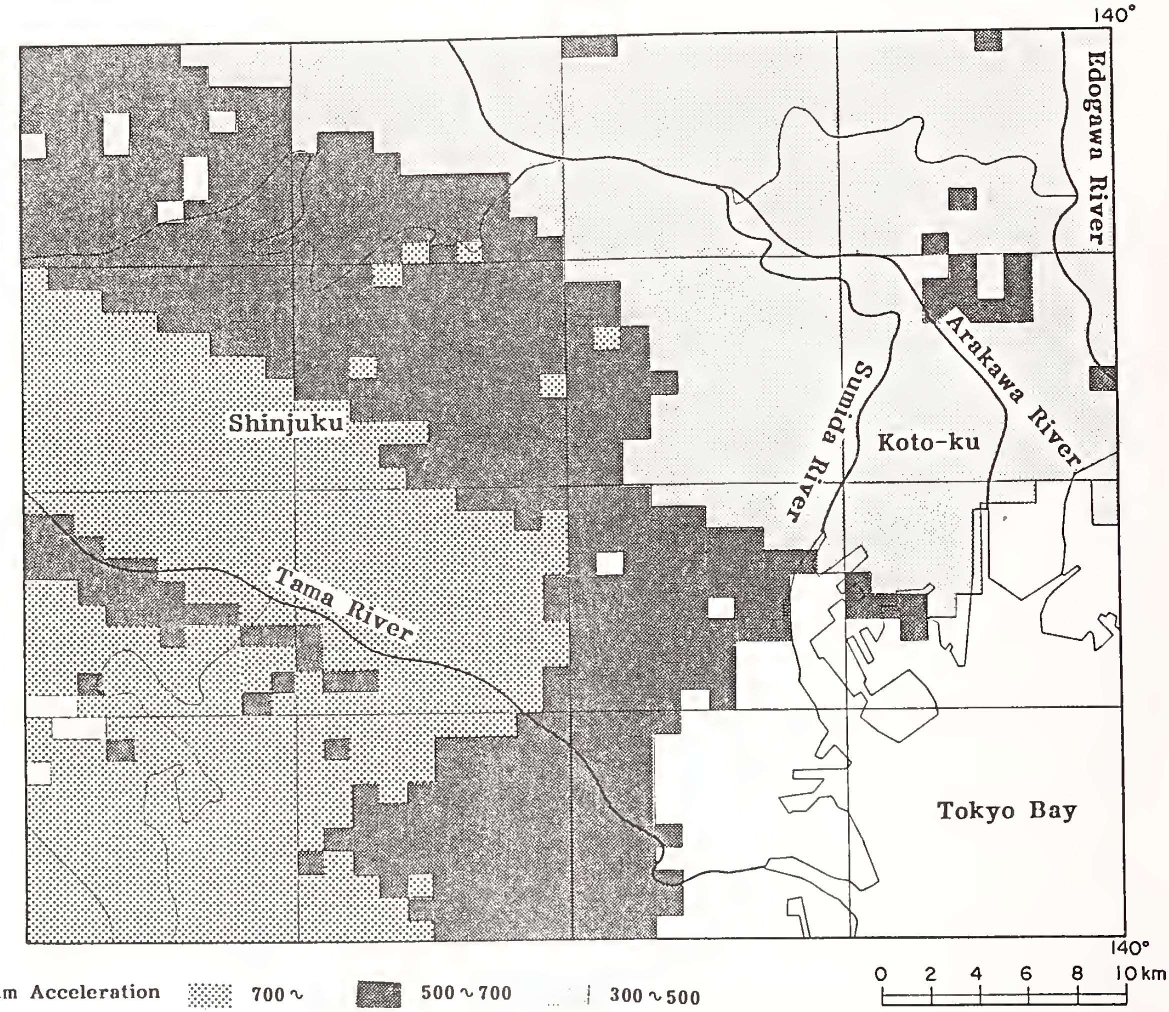

Figure 6 Maximum Acceleration Map of Tokyo Metropolitan Area

Figure 7 shows maximum accelerations on the ground surface and on the bedrock as measured on reclaimed land around Tokyo Bay. The maximum acceleration on reclaimed land is two to three times greater than that on the bedrock. It seems that the maximum acceleration given in Figure 6 somewhat contradicts the actually measured accelerations in Figure 7.

The results of a dynamic analysis of reclaimed land in Tokyo, shown in Figures 8 and 9, illustrate this contradiction. Figure 8 shows the ground conditions and the numerical model while Figure 9 shows the acceleration on the surface calculated by non-linear response analysis using the R.O. model. The calculated maximum acceleration on the surface is 121 Gal, assuming the maximum acceleration on the bedrock to be 100 Gal. Some amplification of earthquake motion from the bedrock to the ground surface can be seen. However, the surface acceleration becomes 156 Gal, if the maximum acceleration on the bedrock is 200 Gal, so the amplification factor is less than 1.0. The reason for this is that, for a larger acceleration on the bedrock, the natural period of the ground increases and the damping effect is enhanced by the more significant influence of non-linear soil characteristics. Consequently, the accelerations on soft ground are estimated to be less than those on firm ground, as shown in Figure 6.

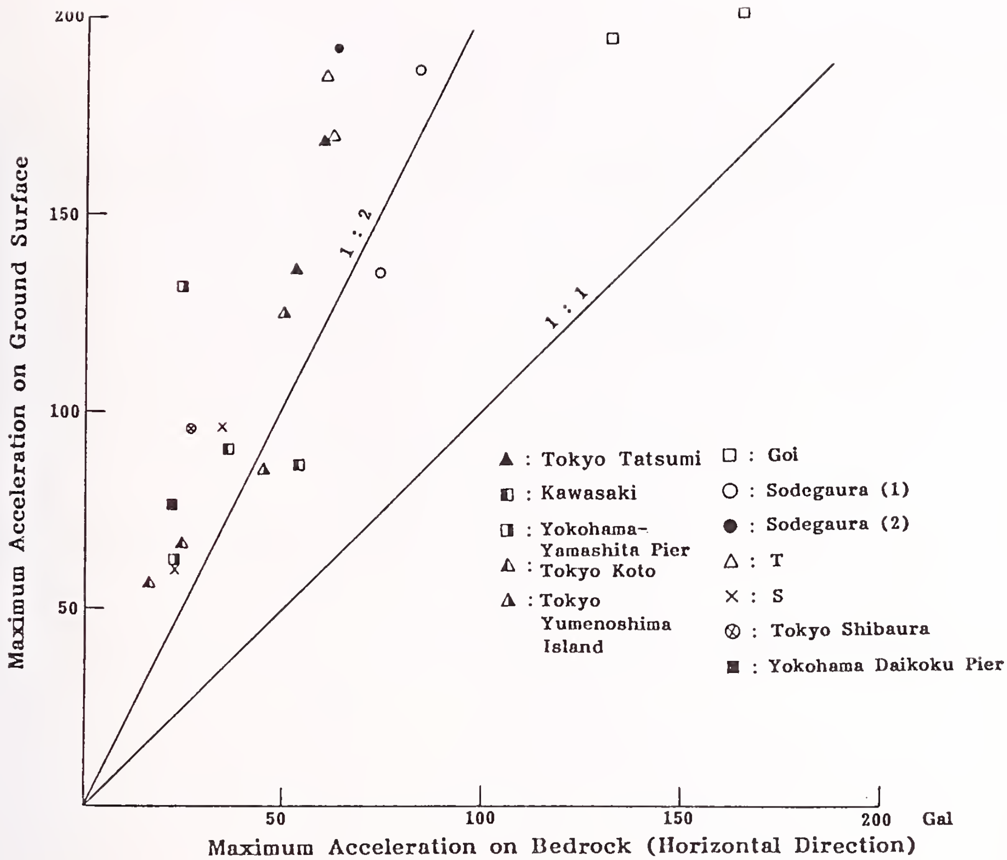

Figure 7 Maximum Acceleration on the Ground Surface and Bedrock Recorded on Reclaimed Land in Tokyo Bay Area

The probable reasons for the lower acceleration on the soft ground are as follows.

1) Evaluation of soil properties for the analysis is not adequate. In particular, there may be problems in evaluating the damping coefficient.

2) The numerical model is not adequate. One-dimensional model such as SHAKE is generally used for the calculation of the ground acceleration. In these numerical models, the effect of seismic waves propagating in a horizontal direction, which are caused by variations in ground conditions are not taken into consideration.

In order to carry out accurate damage estimation of lifeline facilities, it is critical that earthquake motion of soft ground such as reclaimed land be evaluated. Research activity on this subject should be promoted by utilizing the observed earthquake motions on soft ground, such as those recorded during the Loma Prieta earthquake.

Liquefaction-Induced Permanent Ground Displacement

The author and his research team reported that liquefied ground was displaced as much as several meters in the horizontal direction, depending on topographical conditions, at the time of the 1983 Nihonkai-Chubu earthquake.[1] Since then research case studies of liquefaction-induced ground displacements caused by eight earthquakes in Japan and the U.S. have been conducted by the Japan-U.S. joint research team.[2),3),4] The mechanism of liquefaction-induced ground displacement has been investigated by shaking table test. However, no mechanism which gains a consensus of researchers has been found so far.

Depth (m)	Soil	Thickness (m)	Shear Wave Velocity (m/s)	Density	Numerical Model
	Fine Sand	3.8	170	1.65	
	Silt	2.0	170	1.50	
-10	Sandy Silt	4.5	130	1.50	
	Sandy Clay	2.0	200	1.50	
	Sand	6.3	330	1.80	
-20	Fine Sand	1.5	330	1.90	
-30	Hard Clay	11.1	380	2.00	
-40	Tertiary Layer	10.8	440	2.00	
-50, -60	Tertiary Layer	—	530	2.00	

Figure 8 Ground Conditions and Numerical Model for Dynamic Response Analysis (Alluvial land in Tokyo)

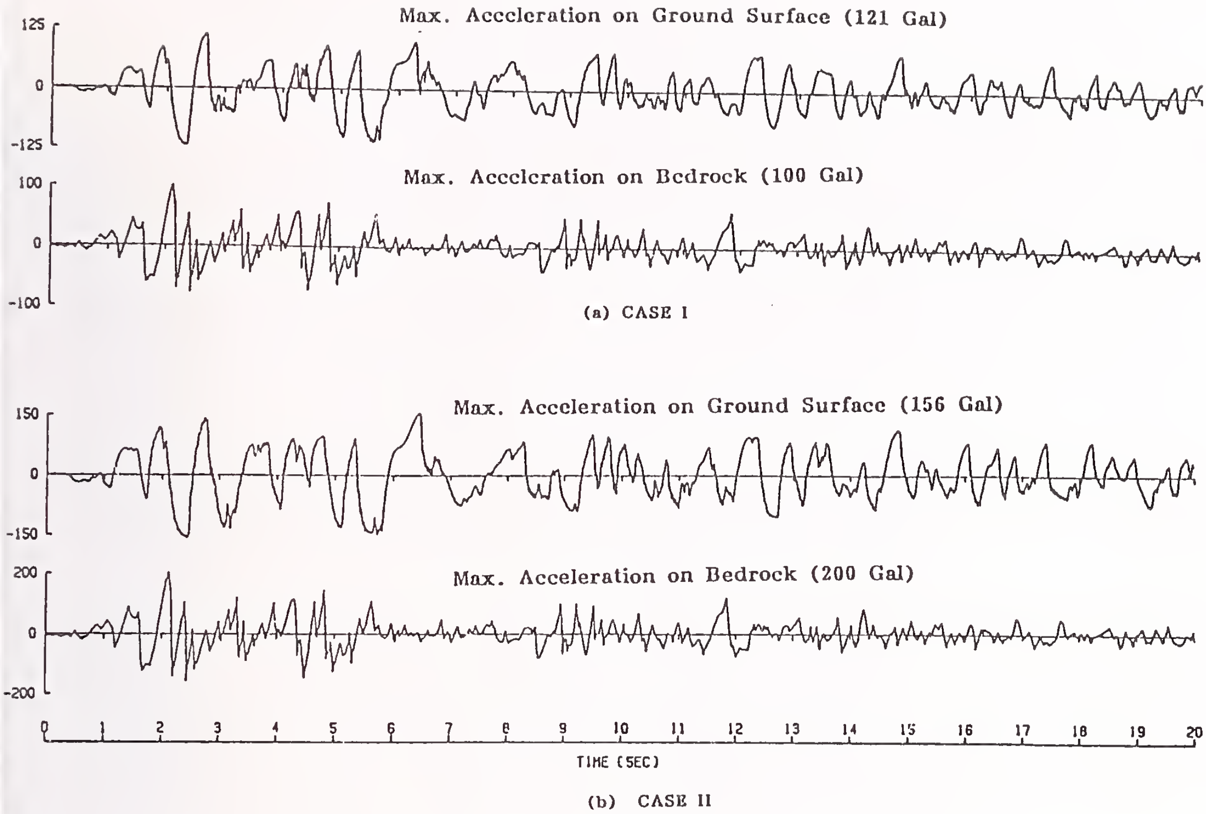

Figure 9 Response Acceleration on Ground Surface

Only the following empirical formula has been proposed;

$$D = 0.75 \sqrt{H} \sqrt[3]{\theta}$$

- D: Liquefaction-induced ground displacement (m)
- H: Thickness of liquefied soil layer
- θ: Gradient of ground surface or of liquefied soil layer (%)

Estimating the effects of liquefaction-induced displacements on lifeline facilities, especially on buried pipes, requires accurate estimates of the displacement magnitude, its direction and pattern. To make this information available, the mechanism of ground displacement needs to be fully clarified. Model tests using shaking tables are now being undertaken by several institutes in Japan, while work on numerical models to calculate the permanent ground displacement is proceeding.

Effects of Permanent Ground Displacement on Lifeline Facilities

Many buried pipes and foundation piles were damaged by liquefaction-induced ground displacement during the 1964 Niigata and 1983 Nihonkai-Chubu earthquakes. The effects of liquefaction on lifeline facilities are as follows.

1) Settlement and inclination of structures due to reduction of the ground's bearing capacity.

2) Floating of underground structures such as manholes due to buoyancy in the liquefied soil.

3) Liquefaction-induced permanent ground displacement.

Effects 1) and 2) are already taken into consideration in the earthquake-resistant design of facilities, but effects of permanent ground displacement 3) are at present not considered.

When large permanent ground displacements, with a magnitude of several meters, are considered in the earthquake-resistant design of buried pipes two matters become a problem. One is whether "the seismic response displacement method," which has conventionally been used for the design of underground structures, is applicable or not. And if it is applicable, there is also the problem of how to evaluate the coefficient of the subgrade reaction of partially liquefied ground.

The other is the evaluation of the ultimate strength of facilities. It is impossible to design the facilities using the conventional allowable-stress method when the permanent displacement is several meters, and information on the ultimate strength of lifeline facilities such as buried pipes is insufficient at present.

The former problem has been studied in an experiment on a pile foundation model in liquefied soil, and it was reported that the force acting on the pile during lateral movement of liquefied soil is similar to the drag force in a liquid. The second problem is now being studied by laboratory tests on piles and pipes.

CONCLUSIONS

In this paper, the current status of damage estimation of lifeline facilities in Japan and problems needing solution in future are described. As stated in the introduction, large-scale waterfront projects have already progressed greatly in major cities. To create an earthquake-proof urban society, the study of damage to lifeline facilities must proceed much more quickly.

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3) M. Hamada, K. Wakamatsu and S. Yasuda, Liquefaction-Induced Ground Displacement during the 1948 Fukui Earthquake, Proceedings of 2nd U.S.-Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines.

4) T.D. O'Rourke, B.L. Roth and M. Hamada, A Case Study of Large Ground Deformation during the 1971 San Fernando Earthquake, Proceeding of 2nd U.S.-Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines.

VI. POST-EARTHQUAKE RESPONSE/ DAMAGE DETECTION

"Data Acquisition and Emergency Response System for City Gas Pipeline Operation During a Major Earthquake"
Y. Yoshikawa

"Damage Inspection Systems Immediately after an Earthquake"
K. Kawashima, H. Sugita, K. Kanoh

"Estimation of Degree of Anxiety Felt by People in Underground Urban Spaces During Earthquakes"
E. Saito, H. Ikemi, H. Nakano, M. Nakamura

"Performance of Lifeline and Emergency Response in Watsonville, California to Loma Prieta Earthquake"
J. Isenberg

"Strategies for Repair and Restoration of Seismically Damaged Gas Pipeline Systems"
M. Shinozuka, M. Murata, T. Iwata

DATA ACQUISITION AND EMERGENCY RESPONSE SYSTEM FOR CITY GAS PIPELINE OPERATION DURING A MAJOR EARTHQUAKE

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Abstract

Tokyo Gas more narrowly divided its entire service area by block and sub-block to preclude the possibility of a secondary disaster in the event of major earthquake, and introduced a new remote controlled system capable of limiting service shutoff solely to badly damaged blocks.

Tokyo Gas has recently developed a data acquisition system capable of transmitting real-time seismic and damage information to its headquarters by means of a multiple radio telemeter system. This enables the company to render timely operations in the event of emergencies, service shutoff to badly damaged blocks.

Introduction

The Tokyo Metropolitan area is characterized by an extreme concentration of population, economic and industrial production, and information--rare among the cities of the world.

This area, however, is threatened by a higher possibility of earthquake than any other cities of Japan where seismic activity is brisk. For this reason, public lifeline utilities commissioned to support urban functions are required to make extra safety provisions against damage from future earthquakes.

Above all, city gas utilities are publicly obligated to take cautious steps to preclude the possibility of any secondary disaster resulting from seismic damage inflicted on the gas service system. Because the city gas pipeline network spreads over a wide area and consists of a variety of structures and buried conduits, exact assessment of seismic damage is difficult and time consuming.

Tokyo Gas has divided its service area into blocks that are remote-controllable by a multiple radio telemeter system. The system stops gas supply to selected blocks according to the intensity of damage inflicted, and enables uninterrupted gas supply for the unaffected areas.

Public city gas utilities, however, are required to maintain a stable supply even in the event of a major earthquake, and to make a decision for or against service shutoff after prompt and exact assessment of seismic behavior and damage information by block. Therefore, the construction of a system is under way that collects in real-time such seismic information and makes immediate decisions in emergencies.

The next chapter gives a brief system overview.

Gas Production and Service Facilities

Tokyo Gas supplies gas to Tokyo and its neighboring cities, catering to about 7.2 million clients over a service area of about 2,500 km^2 (Fig. 1).

Gas supply is classified by high pressure, medium pressure A, medium pressure B, and low pressure, with a total pipeline length of about 39,000 km (Fig. 2). High-pressure gas delivered directly from the plant is reduced to medium pressure A by at regulator stations, and then down to medium pressure B by district governors, and finally to low pressure by 4,000 district governors before being supplied to customers.

The Center for Disaster Management and Supply Control at the Head Office (hereafter called the "headquarters") radios operating instructions to the plant. Headquarters by means of radio is able to control operations, as well as the pressure and flow rate of gas holders and governor stations. Communication and computer facilities assist in this effort (Fig. 3).

Simultaneously, headquarters is in charge of supervision of gas production and gas supply in the event of an earthquake.

Provisions for Earthquakes

Prevention of secondary gas-induced disasters

- o Main equipment and facilities are designed in robust construction for high seismic resistance.
- o Timely service shutoff by block/sub-block is ensured in the event of an earthquake.

Uninterrupted gas supply for the unaffected areas

- o Where service blocks are damaged in different severity, the system isolates severely damaged blocks and stops service to these blocks.

- o Maximum efforts will be made to continue service to undamaged or slightly damaged blocks.

Prompt gas-suspension recovery

- o Except for badly damaged blocks, maximum efforts will be made to accelerate recovery of moderately damaged blocks.

Subdivision of Blocks to Minimize Service Shutoff

Division of gas pipeline network by block/sub-block

Tokyo Gas caters to a broad service area where seismic damage has a regional difference according to the location and intensity of the quake and the properties of the ground involved.

Allowing for maximum availability of consumer service, time-consuming recovery work, and manpower requirements, it is preferable to partition the service area into the smallest possible blocks for flexible service upkeep or shutoff by block, depending on the severity of damage to each one.

Therefore, allowing for regional characteristics and the service station's management area, a gas pipeline network is classified in two different blocks by pressure of gas supplied.

(1) "King blocks" (for medium-pressure pipelines)

The medium-pressure pipeline network is divided into nine King (K) blocks (Fig. 4), each block serving several-hundred thousands of customers. K blocks have service facilities such as plants and governor stations by block and are provided, along each block boundary, with valves remote-controllable by radio from headquarters in emergencies to facilitate service shutoff by selected blocks.

(2) Large (L) blocks for low-pressure pipelines

In the event of earthquakes immediately below, where damage is localized, K block-level system shutoff can unnecessarily interfere with customer service. Therefore, a network of low-pressure pipeline is subdivided into 100 L blocks (Fig. 5), each of which remains separated from its adjacent block and has its own district governor incorporating an SI sensor to permit block-by-block service shutoff according to seismic intensity.

Shutoff by customer

Customers' gas meters incorporate microprocessors. As soon as a seismic sensor detects a substantial earthquake, the gas meter shuts off gas flow and stops gas supply to customers. As of now, 67% of customers use microprocessor built-in gas meters.

This type of gas meter automatically shuts off gas flow where the flow rate is abnormal or when continuous use of gas is disclosed over extended hours.

Data Acquisition Regarding Seismic Behavior and Damage

In the event of an earthquake, the system not only promptly collects information on seismic behavior and damage inflicted and isolates badly damaged blocks from others, but also invokes emergency operations, including service shutoff to damaged blocks to preclude the possibility of a secondary disaster.

The following paragraphs present a network of communication systems and the scope of information to be collected.

Communication network

An earthquake can disrupt wire communication circuits or keep them "busy," thereby disabling data acquisition and telecontrols.

Accordingly, Tokyo Gas implements a network of multiplex radio communication systems (Fig. 6). A private, looped radio communications system is laid between headquarters and major service facilities including plants, service stations, pressure-regulating stations, governor stations, etc.

This system monitors gas pressure, flow rate, etc. at these service footholds as well as collecting operating data and information. Where the system detects any serious damage to a block, remote-controlled K block valves are immediately shut and gas flow is stopped.

Service stations have access to field damage information by means of mobile or portable radio communication equipment.

Data Acquisition

(1) Seismic information system

The service area is equipped with 31 seismographs and SI sensors--an average of three to five per K block--and the moment any of these seismographs detects an acceleration exceeding five galileos, the acceleration and SI values are transmitted to headquarters by telemeter.

Telemetering is made through the communication network, and headquarters can collect entire data within four minutes. Upon receipt of these seismic data, headquarters preferentially displays acceleration and SI value distribution on the CRT screen to facilitate assessment of seismic intensity, rough estimation of damage, and distribution of damaged areas.

(2) Seismic Data Acquisition at L blocks

In each L block, telemetering and telecontrol points (operating about 300 slave telecontrol points) are installed on three representative regulators so that the service station can acquire seismic data (acceleration and SI values, service shutoff, and pressure). The points supplement seismic information on K blocks.

(3) Damage Data Acquisition

(a) Information from service stations

The engineers of each service station patrol an affected area after an earthquake and operate remote monitor TV cameras installed at the service station to collect information relating to damage to buildings and structures, fire, gas leakage, etc. This information is transmitted to headquarters by radio.

(b) Other information

We have also established a system capable of collecting a broad range of seismic damage information over a large service area as follows.

- o Wide service area damage data acquisition by helicopter, which stands by for emergency operations around the clock
- o Wide service area damage data acquisition by remote-monitor TV cameras installed at headquarters and service stations
- o Mass media information transmission by TV and radio
- o Disaster control agencies (Tokyo Metropolitan Government, Fire Defense Agency, police stations, and National Land Agency)
- o Patrol information

Emergnecy Control System

In the event of a major earthquake, headquarters will take the following emergency measures based on information on seismic intensity, damage, etc.

Headquarters operates around the clock and is staffed with five operators responsible for emergency decisions and operations.

Identification of damaged K blocks

Acceleration and SI values are radioed from 31 telemetering and other points of the service area to headquarters for display on the CRT screen. This facilitates analysis for identification of damaged blocks (Fig. 7).

K block based operation

- If a given K block records an acceleration of more than 250 gal or an SI value exceeding 25 kine in two or more locations, the system radio transmits information on damage to the affected blocks to the applicable service station.

- The system then shuts off all the K block valves by telecontrol to isolate them from other K blocks.

- The system also reduces the pressure of gas supplied to the service area from the plant and governor stations.

Monitoring of K blocks

The flow-rate assessment system checks pipelines for gas to find if a medium-pressure pipeline network in a K block has been badly damaged. The system compares preseismic and postseismic flow rates for possible gas leaks in excess of a prescribed threshold value (currently, 100,000 m^3/H) including an error.

Flow-rate assessment system

$$Q_t > Q_o\,(1-K\alpha) + Q_L\text{: Warning}$$

Q_t: Postseismic flow rate
Q_o: Preseismic flow rate
K : Household supply by block
α : "microprocessor built-in gas meter" diffusion rate
Q_L: Projected leaks (threshold value: 100,000m^3 in terms of flat rate)

Service shutoff to K blocks

- If the system indicates abnormal flow rates, but fault recovery is impossible at the discretion of the service station concerned, the affected plant and governor station stop supply to the affected K block.

- Steps are also taken to preclude the possibility of a secondary disaster by venting medium-pressure gas into the air through vent stacks installed in K blocks.

FUTURE PLANS

In the event of an earthquake in a major urban location such as the Tokyo Metropolitan area, the city gas service system as an urban infrastructure works to prevent a secondary disaster by means of service shutoff to the damaged K block as soon as the severity of damage is determined.

Accordingly, Tokyo Gas is involved in the development of a real-time pipeline-failure estimation system designed to determine the severity and distribution of damage to gas pipelines and customers' facilities based on information transmitted from seismographs and SI sensors.

This system displays mapping data combined with disaster information and damage estimation on CRT screens. This helps disaster system operators make intuitive decisions. The system is configured with four major features that process three types of real-time information, ground information for city gas-supplied areas, and other associated information (Fig. 8). Major system functions:

Real-time information collection (telemeter-aided seismic data acquisition over a wide area)

o Acquisition of SI sensor data by 300 telemetering and telecontrolling points in L blocks

o Acquisition of data monitored by ten accelerometers buried at basement depth in the block

o Acquisition of data monitored by liquefaction sensors

Projection of damage

o Estimation of potential damage based on SI values

o Estimation of epicenter location and potential damage based on the epicenter information

o Estimation of potential damage to structures and buried pipes

o Estimation of severity of ground liquefaction

Graphic representation of information

o Graphic display of real-time information

o Graphic display of potential damage information

Epilogue

Tokyo Gas is commissioned to supply gas to the Tokyo Metropolitan area, which is characterized by sophisticated and complex urban functions and structures. The company has a compelling social obligation to prevent secondary disasters in the event of an earthquake, and must promptly exert all means to enable the restoration of public utilities.

To meet these requirements, the critical point is how and rapidly damage can be remedied. To achieve this, it is extremely important to commercialize the real-time pipeline-failure estimation system that supports emergency decision-making. For this purpose, Tokyo Gas will strive diligently for the timely completion of this system.

References

SI sensor

To determine the effects of ground motion upon structures, it is necessary to assess how intensively such motion can vibrate the structures.

One of the major barometers for this is the "velocity response spectrum." Most structures range from 0.1 to 2.5 sec. in natural periods. G. W. Housner defines mean spectrum intensity (SI value) as a major indicator of the intensity of ground motion.

Past records of earthquakes to a large extent illustrate the presence of a correlation between SI value and damage to structures.

We developed a prototype SI sensor jointly with the Production Technology Research Institute of Tokyo University and are endeavoring to commercialize the product.

Ground liquefaction sensor

This sensor detects a rise in the water level in sand layers as a result of ground liquefaction. This is detected by changes to the waterhead flowing into underground pipelines.

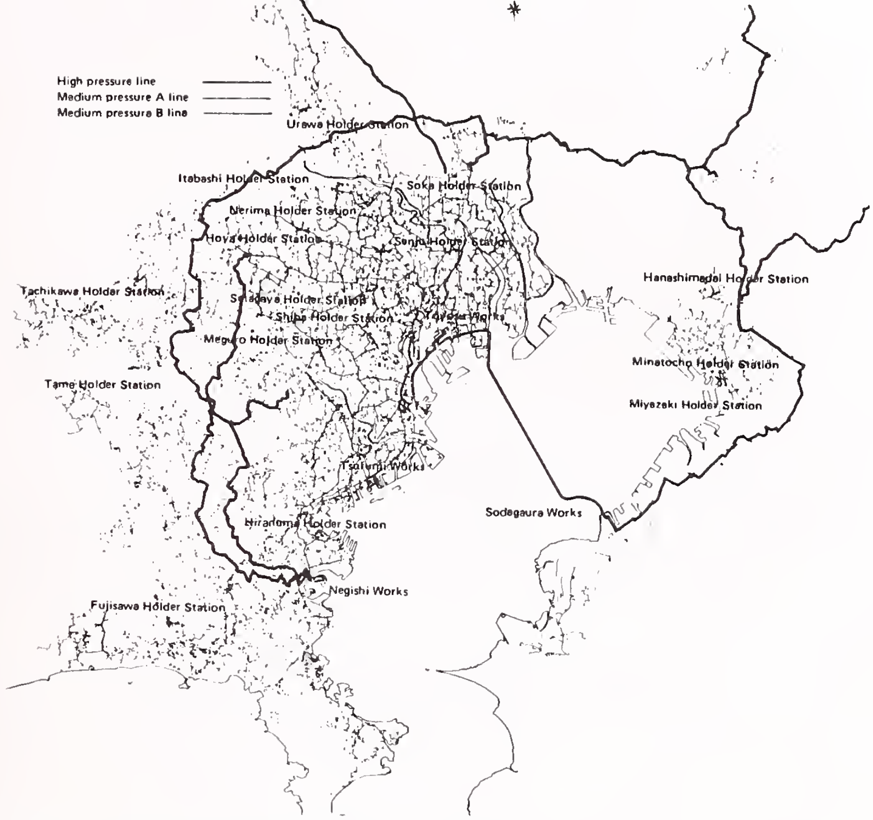

Fig. 1 High and Medium Pressure Gas Pipeline Network

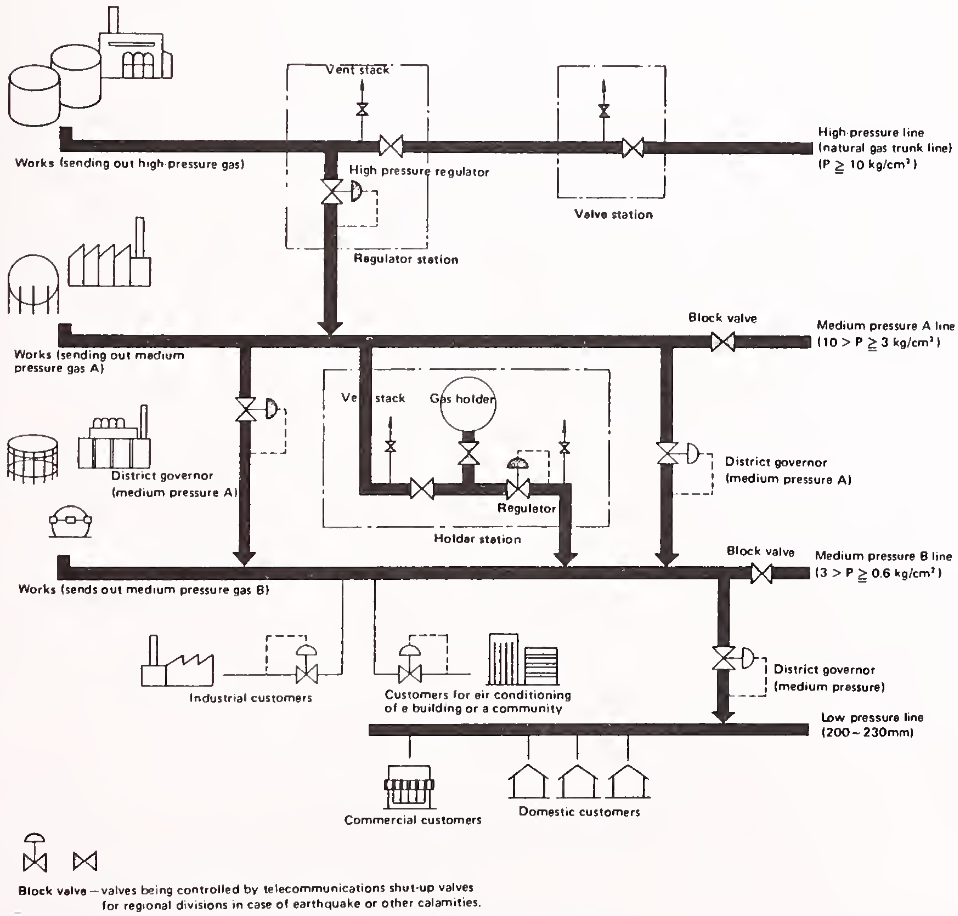

Fig. 2 Gas Supply System

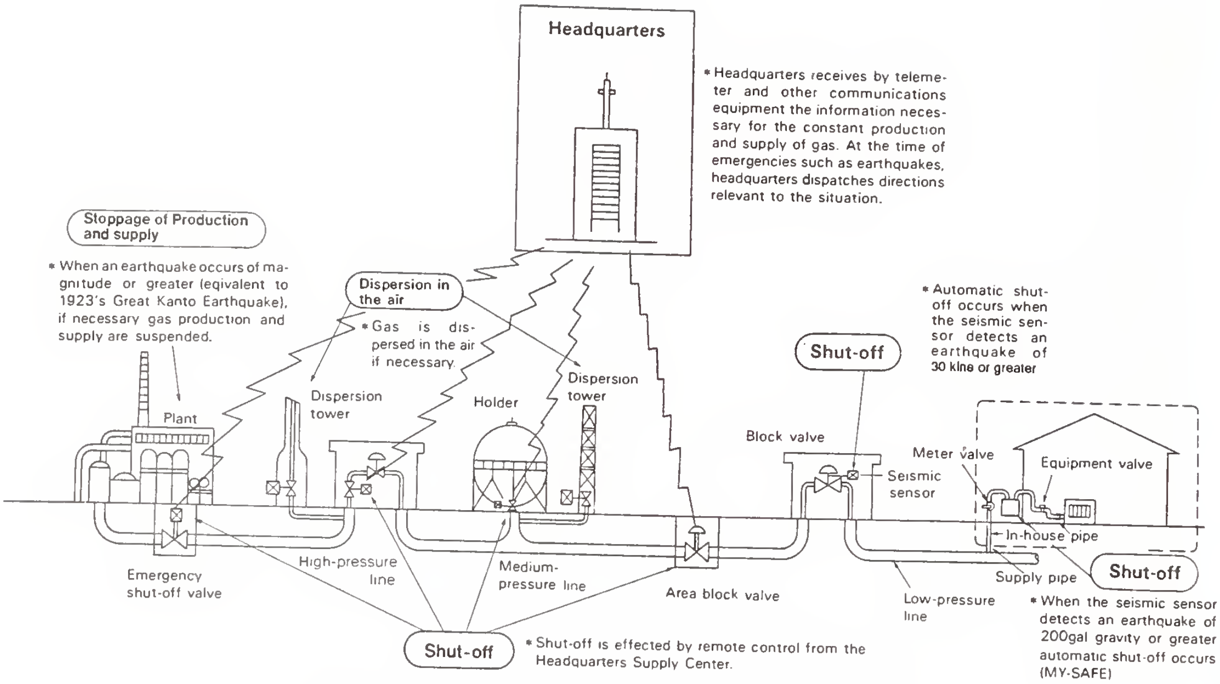

Fig. 3 Hazard Countermeasures System

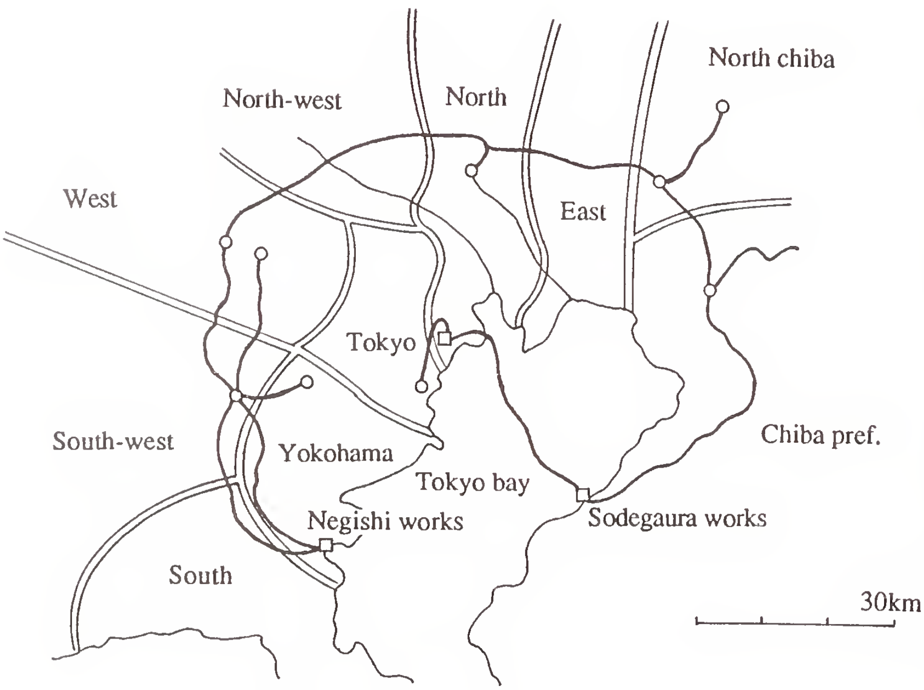

Fig.4 Isolation Area of Medium-Pressure Pipeline Network (K-block)

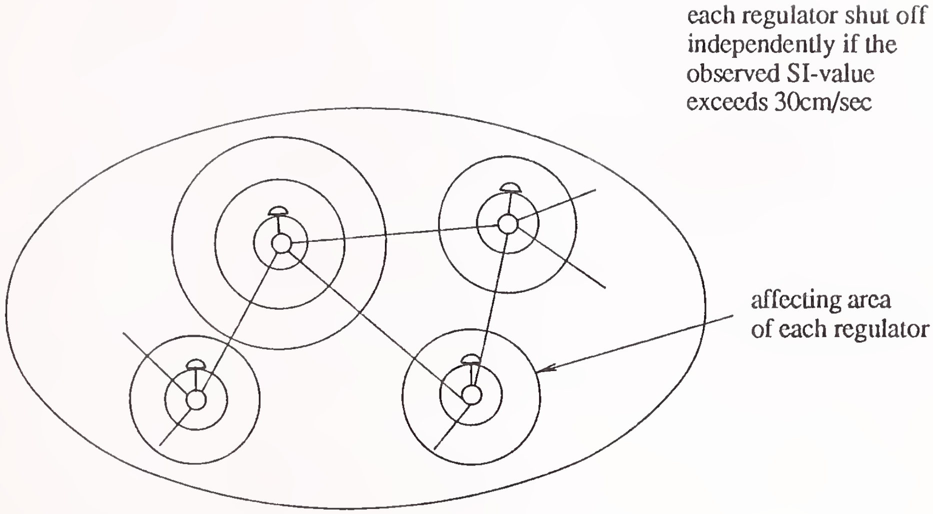

Fig. 5 Isolation Area of Low-Pressure Pipeline Network (L-block)

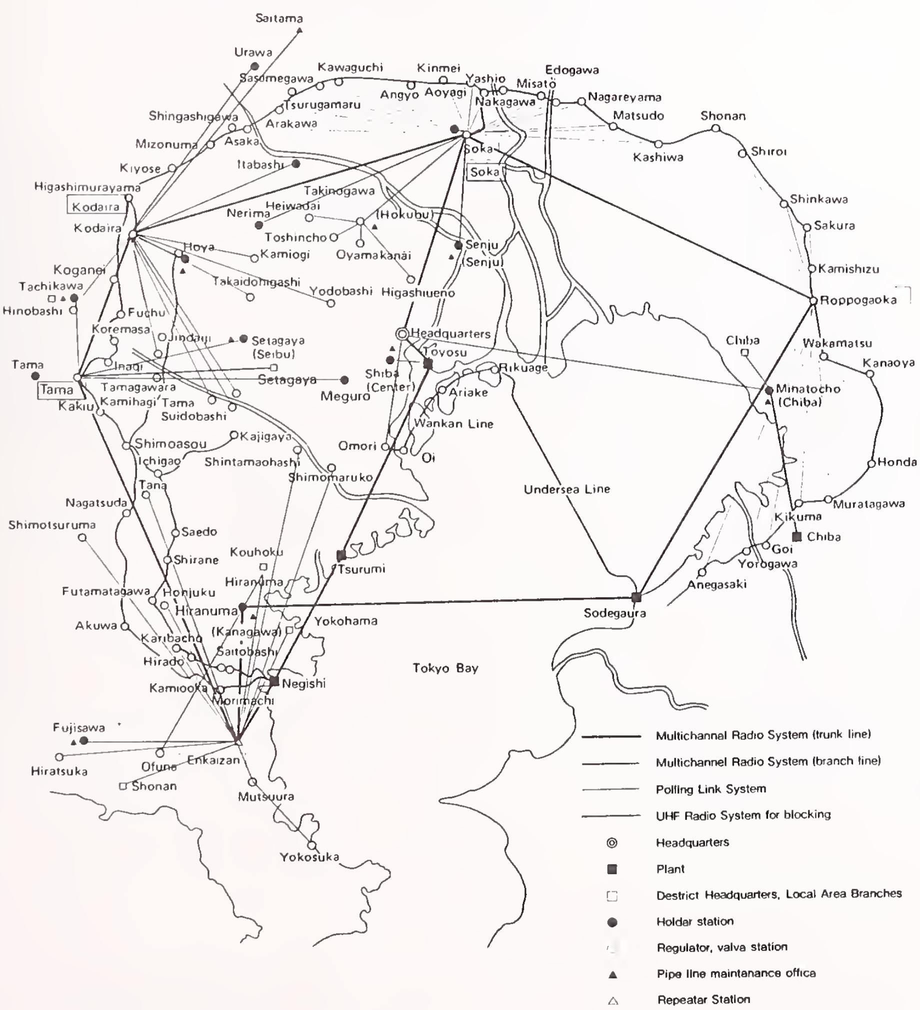

Fig. 6 Telecommunication Networks

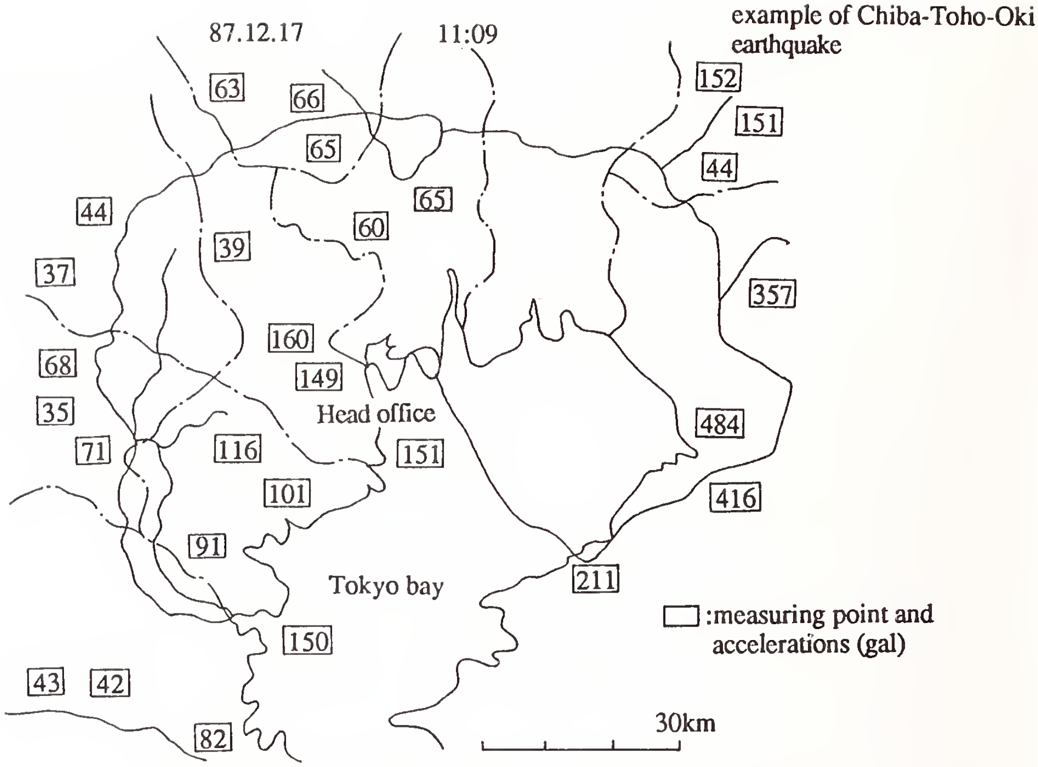

Fig. 7 Earthquake Data Acquisition System

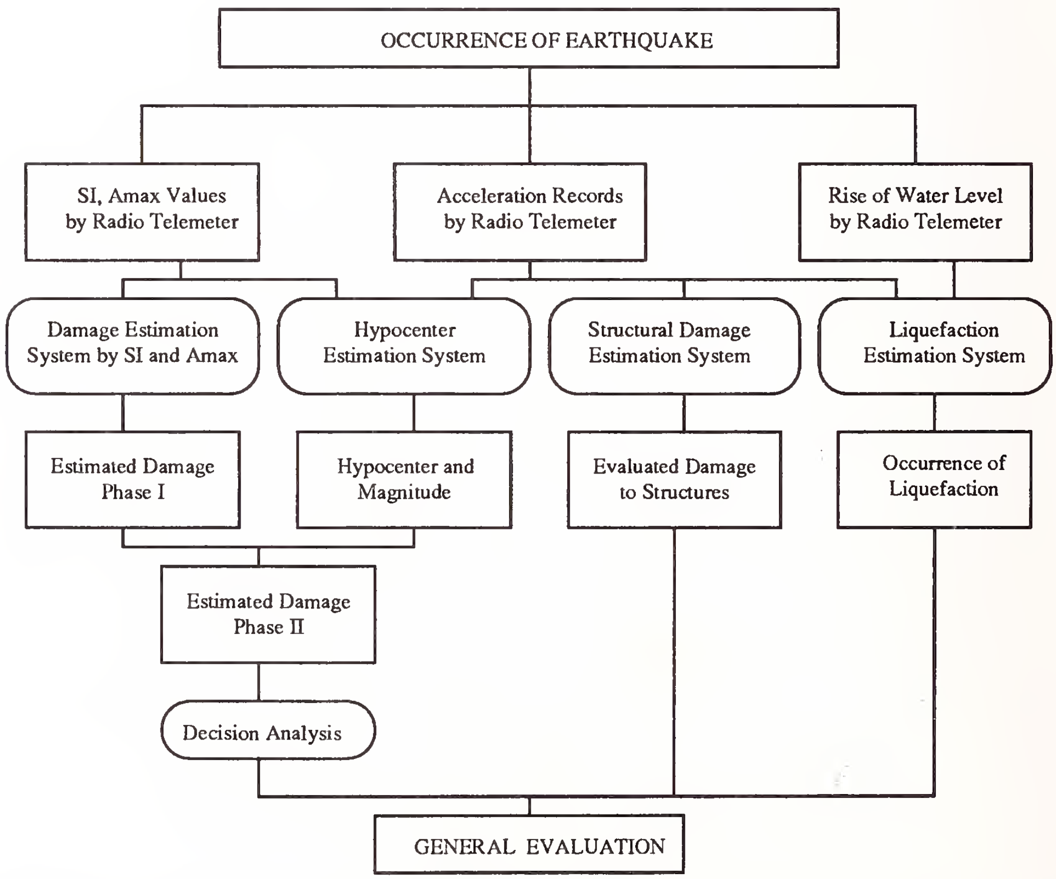

Fig.8 Flowchart for Damage Assessment and Decision Making in a Supply Block

DAMAGE INSPECTION SYSTEMS IMMEDIATELY AFTER A MAJOR EARTHQUAKE

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Earthquake Engineering Division, Earthquake Disaster Prevention Department, Public Works Research Institute, Ministry of Construction

ABSTRACT

This paper introduces a part of a comprehensive 5 year research program entitled "Development of Information Collecting and Processing Systems for Disaster Mitigation" of the Ministry of Construction. Description is given to the current problems for correcting damage information immediately after a destructive earthquake in urban area and basic requirements for developing a new system. Emphasis is placed for a proposed seismic damage information systems. Damage inspection way with use of video camera from a helicopter is also presented.

INTRODUCTION

When a destructive earthquake occurs, it is of significant importance to immediately collect accurate information on the extent of damage in the area affected by the earthquake. Road transportation is likely to be interrupted in urban area such as Tokyo as a direct result of damage of not only transportation facilities but buildings and lifeline facilities along the road. Confusion caused by people in evacuation and automobiles left on road are anticipated, and they would also cause considerable disruption of transportation. It is required therefore to develop a new information collecting and processing systems which will enable the extent of damage to be recognized immediately after an earthquake for formulating the repair and restoration strategy promptly.

A comprehensive 5 year research program entitled "Development of Information Collecting and Processing Systems for Disaster Mitigation" was initiated in 1987 by the Ministry of Construction. Public Works Research Institute, Building Research Institute and Geographical Survey Research Institute are involved in the research. The objectives of this research project were to develop new information collecting and processing systems, using advanced new technologies, for various disasters such as earthquake, floods and debris flows. As shown in Fig. 1, key items of the research development are :

a) Bird's eye information collecting systems from helicopter,
b) On-line information processing systems, and
c) Real-time digital mapping systems.

For this purpose, measuring and assessing technology of the damage degree of

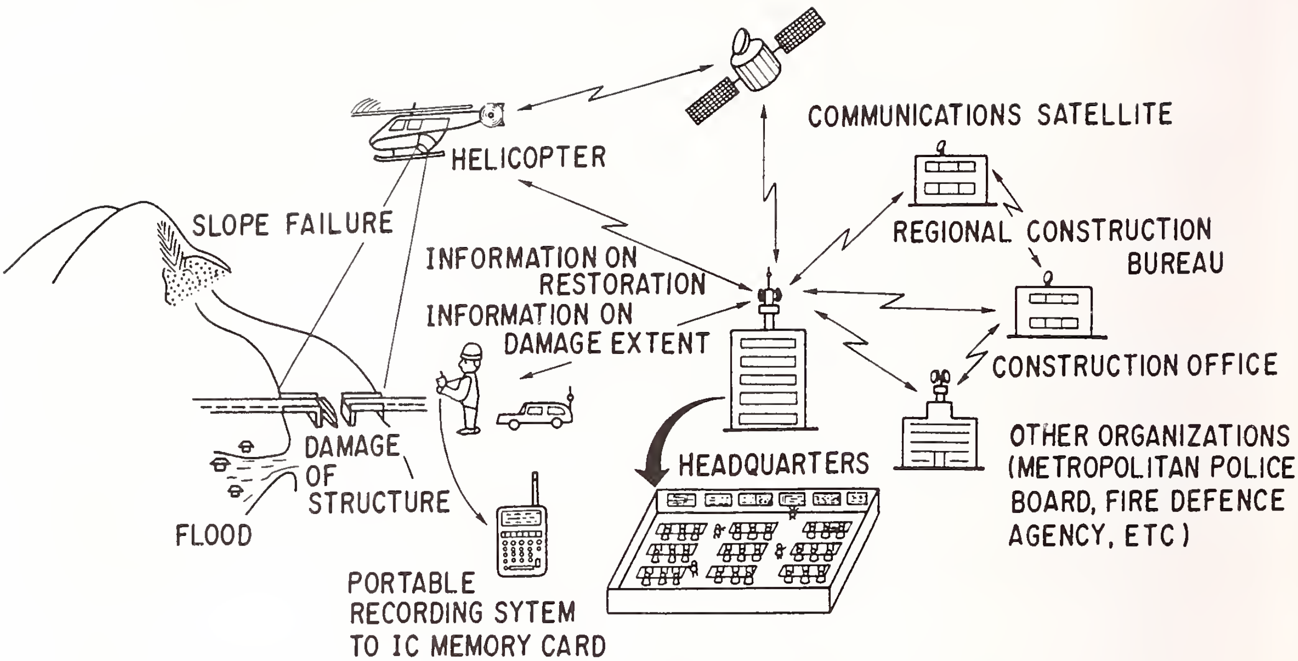


Fig. 1 Information Collection and Processing Systems

structures using moving picture information detected from helicopter and new media oriented high-grade disaster prediction methods as well as planning systems for repair and restoration in the damaged area is now under development. To avoid distortion of information during transferring from regional office to headquarters, it is aimed to adopt visual motion picture systems.

This paper briefly describes a part of the comprehensive research program with emphasis on damage information collecting and processing systems for road and river facilities[1].

PROBLEMS OF CURRENT DAMAGE INFORMATION COLLECTING SYSTEMS AND BASIC REQUIREMENTS FOR DEVELOPING NEW SYSTEMS

Although there were not destructive earthquakes which seriously hitted urban area in the last 40 years in Japan, it has been repeatedly pointed out from the experience of local seismic damage that information collection and dissemination are the most important but difficult issues for central and regional governments in charge of control of public facilities[2)3)]. This was also pointed out at the Loma Prieta, U.S.A., Earthquake in October 1989[4)5)]. From the past experiences, various obstacles for collecting damage information are anticipated[2)3)]. The most important one is unexpectedly long time required for collecting damage information due to interruption of road and damage of information transferring media such as telephone. Even if the telephone could survive an earthquake, disruption of communication repeatedly happened due to concentration of call to a specific region.

Inadequate accuracy of the collected information is the second problem. Unconfirmed and sometimes uncorrect information are sent. Only a part of total information arrives. It is difficult to know the accuracy and correctness. Sometimes the same information arrives through different routes, and they are misunderstood as different information. Confirmation of source of information is very important. When a series of events occurred, information on subsequent event sometimes arrives earlier than the information of the preceding event. There were examples that this caused significant misunderstanding on what were going on the events and the repair.

Because most of information is transferred by a voice and letters, correct evaluation on the information is difficult. Distortion of information always occurs. Therefore it is required to send information in a form of visible information. Still picture and moving picture information are quite effective.

Basic requirements for solving the problems of the current damage information collecting systems are:

(1)Prompt Information Collection
Collection of damage information in wide area has to be made at not only day time but night. Because accuracy of the damage information required changes in accordance with time following an earthquake, it is important to be able to collect damage information with reasonable accuracy depending on the time after an earthquake. Visible information through video camera is quite important.

(2)Delivery of Updated and Same Information for All Bodies Involved in Repair and Restoration
Although quality of the information required depends on the task of individuals involved in repair and restoration, it is essentially important that the same and updated damage information is able to be delivered for all individuals from site engineers to decision makers. The real-time damage information has to be displayed in the form of not only letters but moving picture. Because it is important for site engineers involved in inspection and emergency treatment to know real-time progress of damage and repair in other area for which they are not in charge of, the systems should have capability to display the damage information of not only inside but outside of the controlled area.

(3)Data Processing Systems for Repair and Restoration
The data processing systems has to be capable to control all the damage information in a system, and can be used from the first stage (immediately after an earthquake) to third stage (stage of permanent repair and restoration)[2)3)]. It should also have a capability to execute various simulation for studying restoration strategy.

PROPOSED SEISMIC DAMAGE INFORMATION SYSTEMS

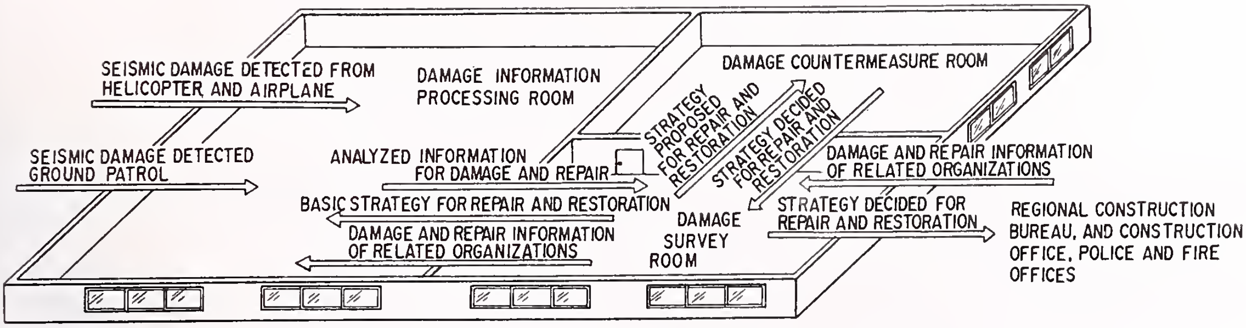
For enabling the requirements presented in the preceding chapter, Seismic Damage Information Systems is proposed for collecting and processing seismic damage information for repair and restoration of road and river facilities. The Seismic Damage Information Systems is to be operated at the Main Headquarters for Disaster Countermeasures (Headquarters Office of the Ministry of Construction), the Headquarters for Disaster Countermeasures (Headquarters Office of Regional Construction Bureaus) and the Branch for Disaster Countermeasures (Construction Offices).

Fig. 2 shows layout and role of three sub-sections of the Seismic Damage Information Systems. At Damage Information Processing Room, all damage information are collected from various sources such as ground patrol teams, helicopter patrol teams, other organizations and public. From such every types of information, important information required for repair and restoration of road and river facilities are identified. Preliminary statistical analysis of the damage are made. At Damage Survey Room, repair and restoration plan based on the damage information compiled at the Damage Information Processing Room are formulated. Communication and adjustment for restoration with related organizations such as police, fire department and local governments have to be made at this stage. At Damage Countermeasure Room, basic policy and strategy for repair and restoration based on the information compiled at the Damage Survey Room is decide.

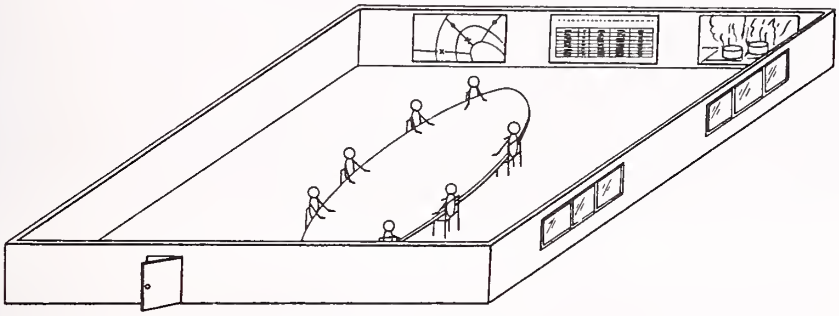
Fig. 3 shows an image of the Damage Survey Room and the Damage Information Processing Room which should be placed at the Headquarters Office of Regional Construction Bureaus.

The Seismic Damage Information Systems is of four sub-systems for collecting, transferring, processing and maintenance of the damage information. The outline of the sub-systems are as follows.

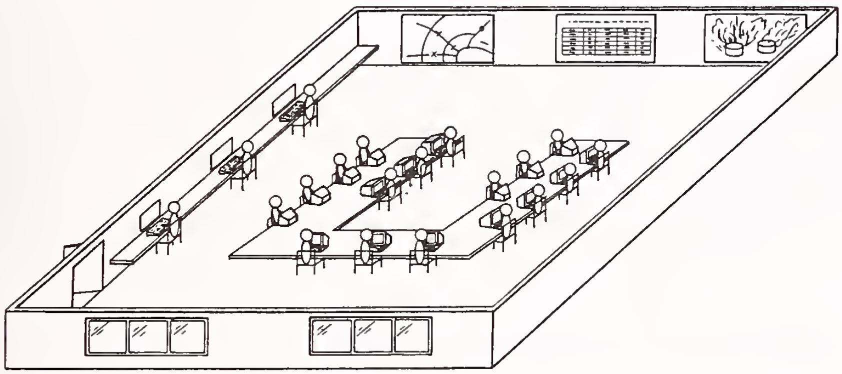
(1) Seismic Damage Information Collecting Systems
Information on seismic damage detected by helicopter patrol teams and ground patrol teams is collected and processed in this systems. The damage information is of moving picture, still picture and letters. They have to be detected by video camera, electric still camera and electric hand notes. The electric hand notes are specially designed man-machine handy type personal computers for exclusive use for recording seismic damage. When the electric notes are operated by field engineers, they automatically display and ask important items for identifying the damage situation such as location, size, type and degree of damage. The data are stored on floppy disk, and they can be sent to host computer at headquarters by cables or radio communication systems.



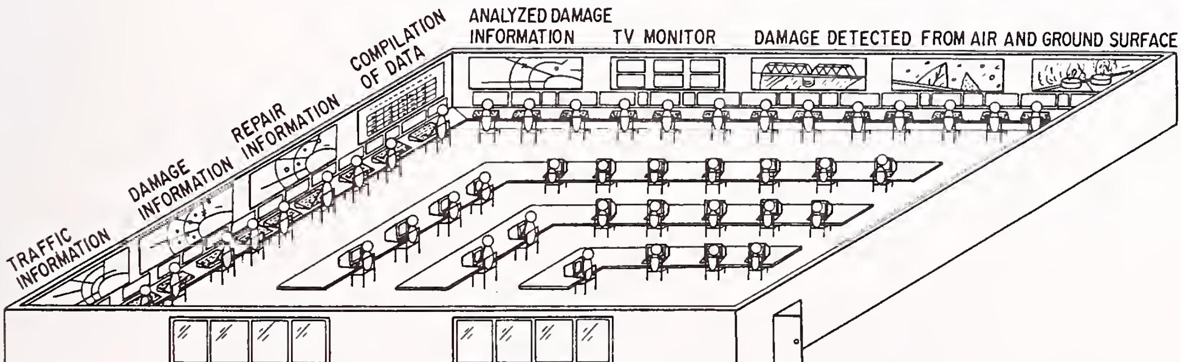
(a) FLOW OF INFORMATION



(b) DAMAGE COUNTERMEASURE ROOM

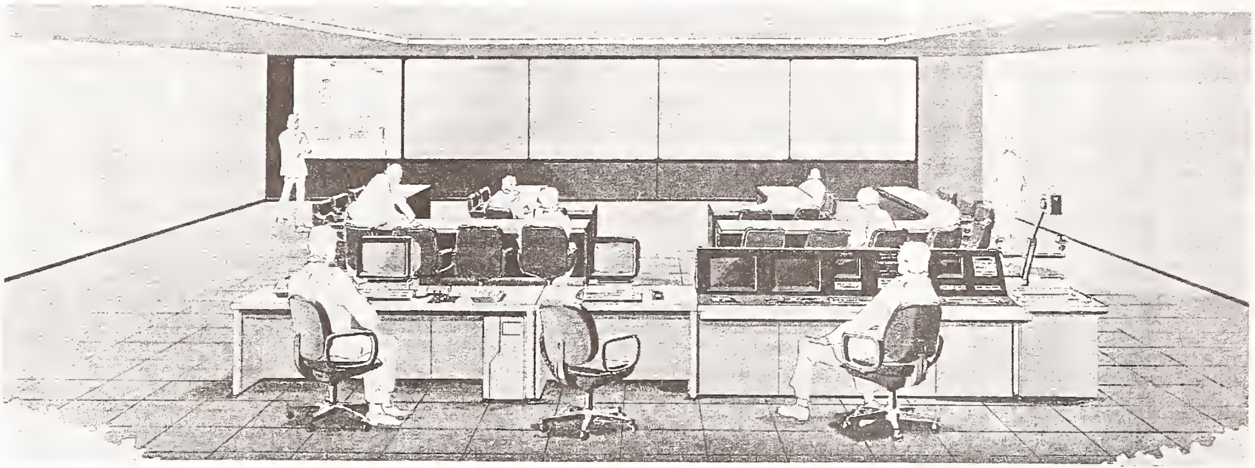


(c) DAMAGE SURVEY ROOM



(d) DAMAGE INFORMATION PROCESSING ROOM

Fig. 2 Damage Information Processing Room, Damage Survey Room and Damage Countermeasure Room of Seismic Damage Information Systems



(a) DAMAGE SURVEY ROOM (REGIONAL CONSTRUCTION BUREAU)

(b) DAMAGE INFORMATION PROCESSING ROOM (REGIONAL CONSTRUCTION BUREAU)

Fig. 3 Image of Damage Information Processing Room and Damage Survey Room

(2) Data Transferring Systems

Fig. 4 shows an image of the Data Transferring Systems for transferring the damage information between Head-quarters, Regional Construction Bureaus, Regional Construction Offices, patrol helicopters and ground patrol teams. The information to be covered by the systems includes;

a) moving picture motion detected by video camera on helicopter
b) moving picture motion by video camera of ground patrol team
c) still picture motion by still camera of ground patrol team
d) character and mapping data used in the Seismic Damage Information Processing Systems
e) Data on earthquake and meteorology which are telemetered
g) Information through telephone and faximile

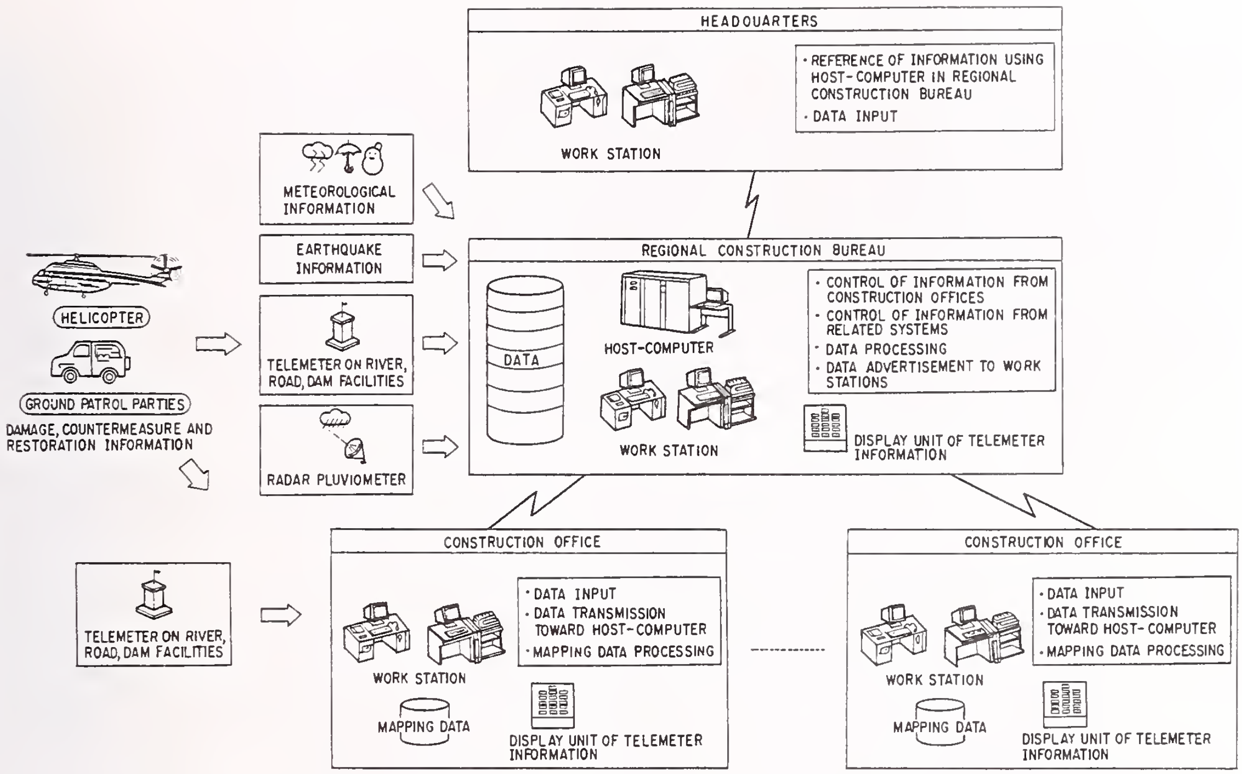


Fig. 4 Image of Data Transferring Systems

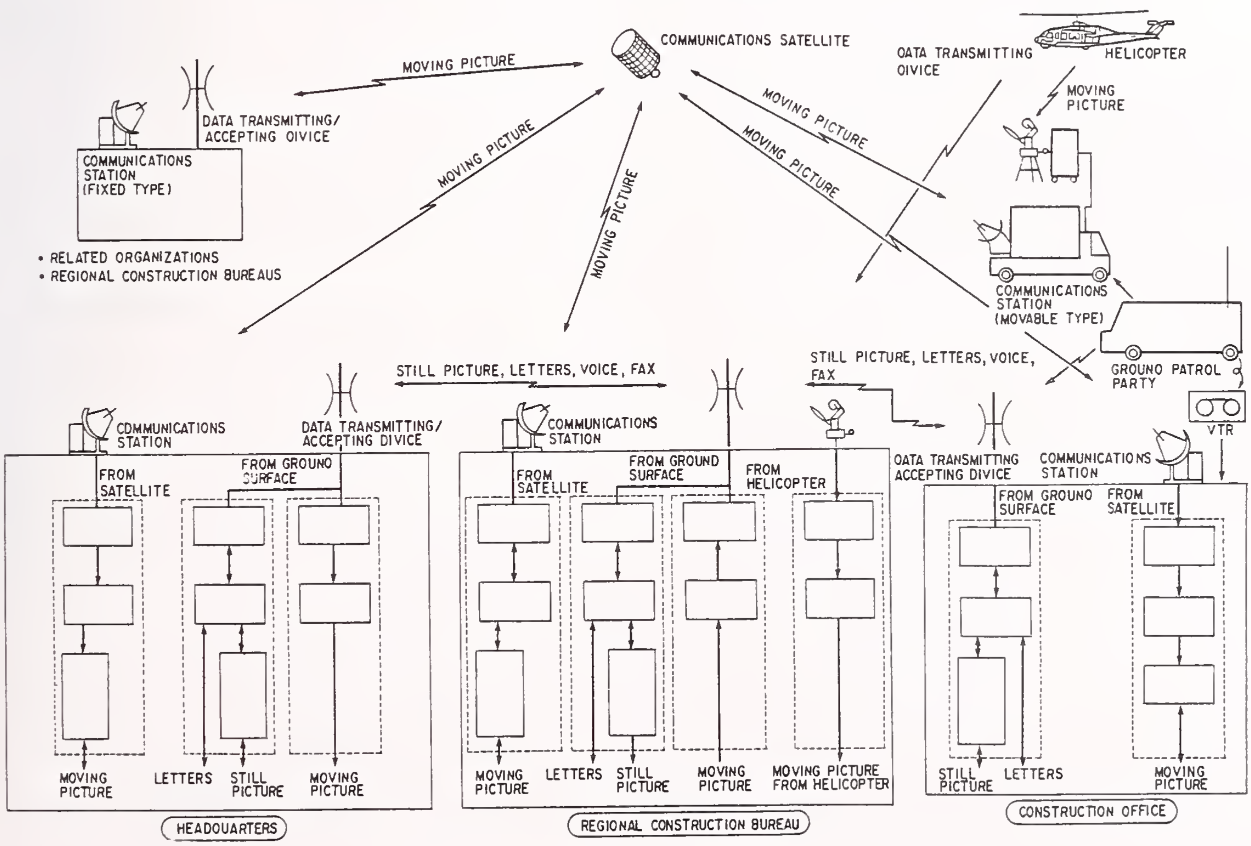


Fig. 5 Main Facilities for Data Transferring Systems

Main facilities required for the Data Transferring Systems are presented in Fig. 5. Main facilities included in the systems are of satellite communication systems and ground communication systems. The satellite communication systems includes satellite, fixed receivers, portable receivers and moving picture transmitting systems. The fixed receiver and portable receivers were already installed at two and five of eight Regional Construction Bureaus, respectively. The ground communication systems are of radio network for exclusive use of the Ministry of Construction and picture motion communication systems. The radio network of the Ministry of Construction covers about 800 locations including Regional Construction Offices and local governments throughout the country.

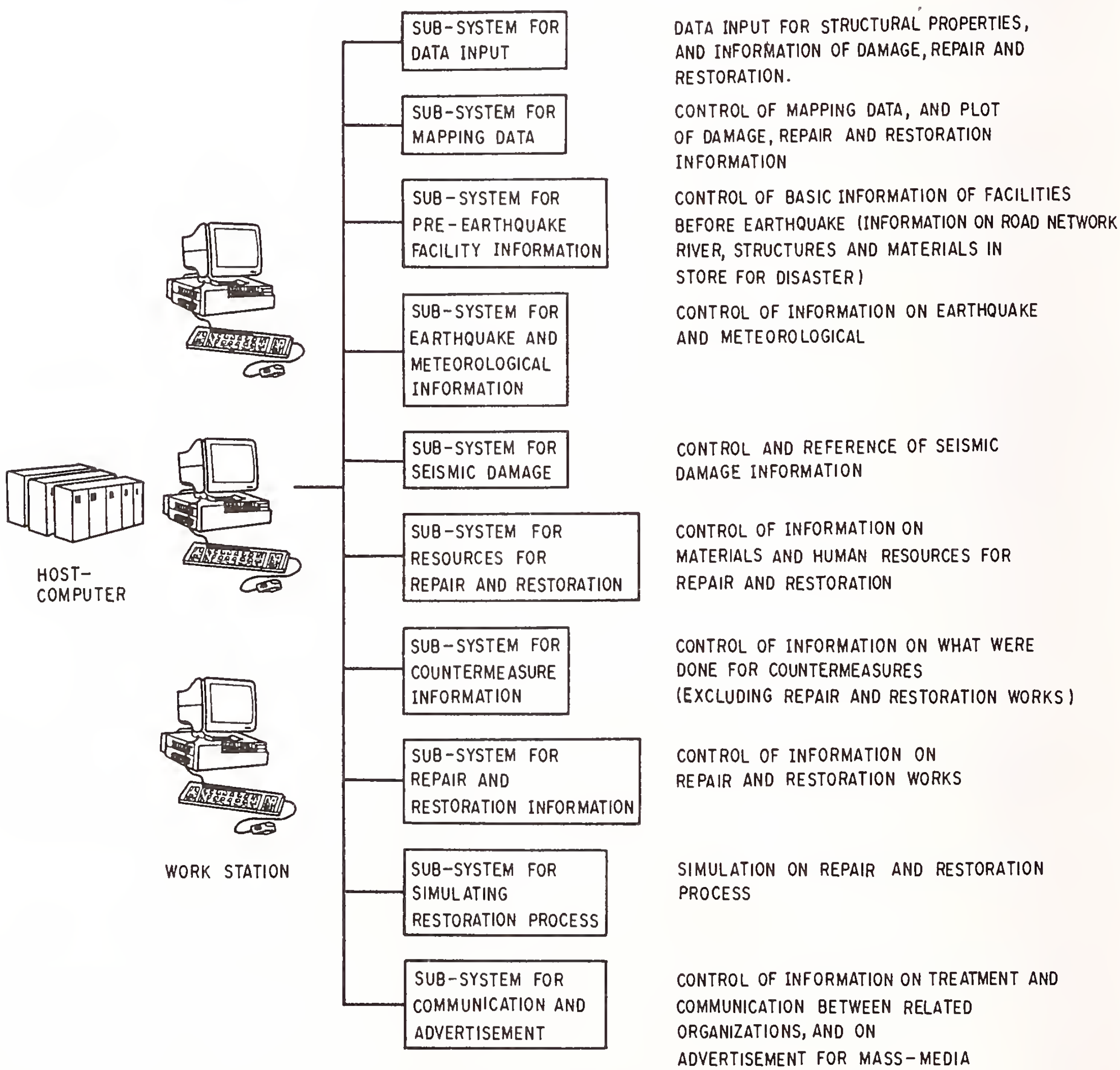


Fig. 6 Sub-systems of Seismic Damage Information Processing Systems

(3) Seismic Damage Information Processing Systems

Fig. 6 shows ten sub-systems for operating the Seismic Damage Information Processing Systems. Whole seismic damage data, repair and restoration data, mapping information, and facilities information prior to an earthquake are compiled and controlled in this systems. Simulation of restoration strategy can be made for clarifying the most appropriate way of restoration. Control of information on adjustment with other organizations are made. Notice and advertisement of damage situation and repair to mass-media is also controlled by this systems. It is of host computer for data processing and work-stations for data input and output. Analyzed data should be displayed on monitor in a well organized form for deciding repair and restoration strategy.

(4) Audio Visible Systems

Fig. 7 shows an image of the Audio Visible systems. It includes screens with 200 inches and 70 inches, monitor of telemeters, projectors and electric black boards. The Audio Visible Systems should have the capability to monitor the damaged area through large screen, to understand the damage feature by comparing the still and moving pictures taken prior to the earthquake, and to conduct TV conference.

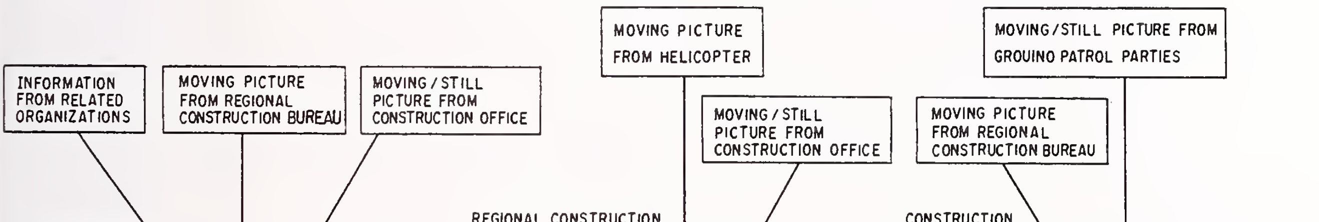


Fig. 7 Image of Audio Visible Systems

DAMAGE INSPECTION FROM HELICOPTER

Damage inspection from helicopter seems attractive to collect damage information promptly because it is not interrupted by traffic congestion and road suspension. To investigate the effectiveness of the damage survey from helicopter, model damages were intentionally placed on various road and river facilities. A helicopter owned by the Ministry of Construction was used for this test. It is for exclusive use for disaster prevention as shown in Photo 1.

Photo 1 Helicopter for Disaster Prevention Owned by Ministry of Construction

A video camera which is set in a specially designed camera case for isolating the camera from vibration of the helicopter was used to detect artificial damage. The focal distance of the video camera is from 12.5 mm to 550 mm. By using the focal distance of 550 mm, a flame of 4.8 m by 3.6 m can be recorded when the height of the helicopter is 300 m which is regulated minimum height at city area. Real time transferring of the picture from the helicopter to the ground receiver can be made. A thermal image sensor is also provided on the helicopter. It can detect the difference of temperature with the resolution of 0.25 degree in Centigrade. Because zoom as large as 4 times is provided in the thermal image sensor, a flame with 37 m by 21 m can be recorded.

Various seismic damage was intentionally developed on bridges, pavement, road embankment and river structures. Photo 2 shows cracks which were made by placing white vinyl tape with width of 5 cm for aiming to represent a damage developed at mid-height of reinforced concrete pier and a damage of prestressed concrete deck. The former damage is likely to be developed when reinforcing bars are terminated at mid-height with inadequate anchoring length[5], and the

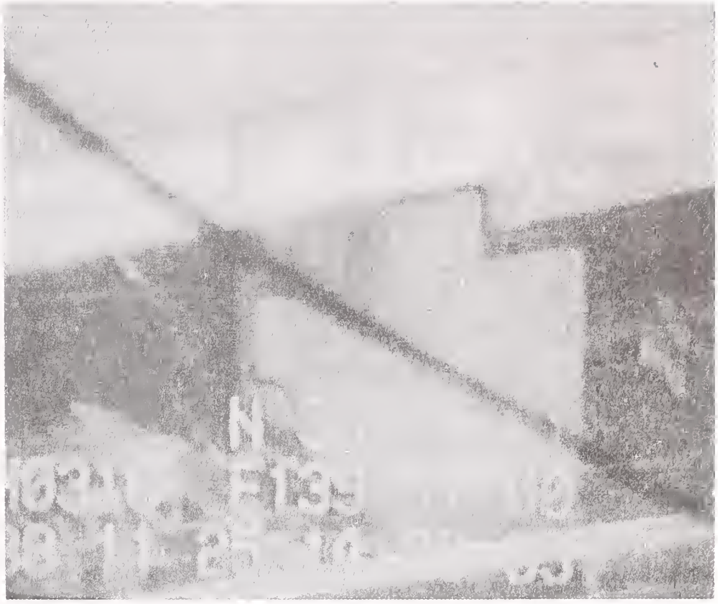


Photo 2 Cracks Placed on Mid-height of Pier and Girder

Photo 3 Damage Detected by Video Camera Placed on Helicopter (Height is 321 m, Focal Distance is 550 mm)

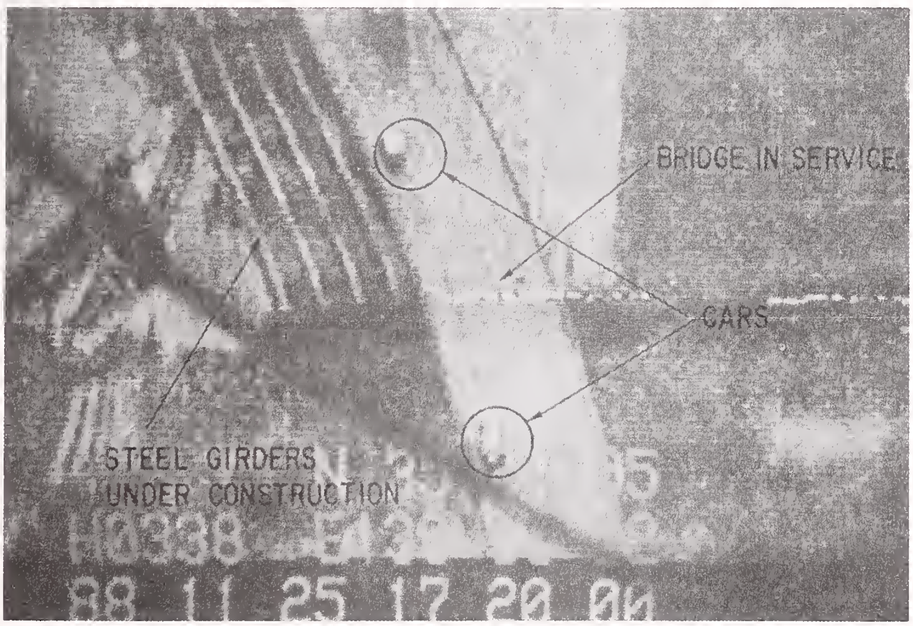


Photo 4 Bridges Detected by Thermal Image Sensor at Night (Height is 338 m)

latter damage tends to be developed on the concrete near the bearing supports[7] when the deck is pulled laterally. Photo 3 shows how such damages are detected from helicopter flying 321 m above the ground surface with velocity of 60 km/h[8]. The focal distance was 550 mm. It is apparent that the cracks at pier and deck can be clearly detected. The test was made by varying the velocity and height of flight.

Photo 4 shows picture representing a new bridge construction along an existing bridge in use for traffic. This was detected by the thermal image sensor at night. Flight height was 338 m from the ground surface. Automobiles driving on the existing bridge and four girders under construction can be apparently seen[8]. The thermal image sensor is effective to detect large damage of steel and concrete structures.

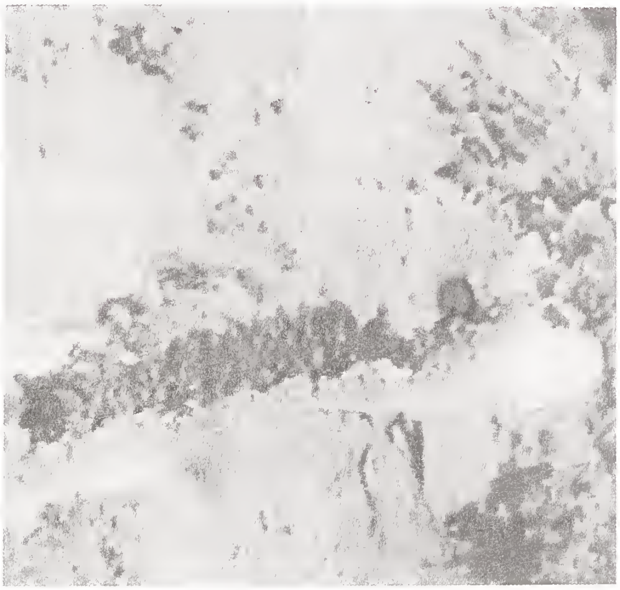


Photo 5 Slope Failure at Nashimoto District of Izu Peninsula Detected by Video Camera from Helicopter

Photo 6 Repair Work Detected by Video Camera from Helicopter

Photos 5 and 6 show the seismic damage of road along natural slope , which was caused by the Izu Ohshima-kinkai Earthquake[9] with the Richter magnitude of 7.0 in January 1978. Extensive slope failure was developed along the road with total distance of about 125 m and height of 30 to 50 m , and this embedded a public bus with three being killed. Photo 6 shows the repairing job by a crane. Because this is a part of video taken by a TV company for broadcasting, the flight information such as height and velocity is unknown. By carefully examining the video record, length, depth and soil volume were surveyed by three geologists with different professional careers[8]. The geologist with 17, 10 and 5 years professional career is designated herein as A, B and C geologist, respectively.

Table 1 Accuracy of Estimation by Three Geologists with Different Professional Career

DAMAGE	GEOLOGISTS			MEASURED AT SITE
	A	B	C	
AVERAGE WIDTH	70(CRANE(10m)x7)	60~80	10~30	100m
AVERAGE DEPTH	10(AVERAGE WIDTHx 1/7~1/10)	5~10	*	5m
FAILURE AREA	*	320~480	*	-
SOIL VOLUME	20,000(30mx70mx10m)	20,000	*	26,500
LENGTH OF ROAD INTERRUPTED BY SLOPE FAILURE	100(AVERAGE WIDTH +α)	80	*	125

1. * MEANS THAT IT WAS UNABLE TO ESTIMATE FROM VIDEO TAPE
2. ( ) REPRESENTS HOW GEOLOGISTS EVALUATED LENGTH, DEPTH AND DISTANCE

Table 1 compares the accuracy of the estimation by three geologists based on the video film. It is interesting to note that the geologist A gives the most appropriate estimation, and that the geologists with shorter professional career tends to give 30 to 50 % smaller estimation. Because estimation was made through video picture which is, of course, much smaller in size than the actual damage, the surveyors tend to give smaller estimation in size.

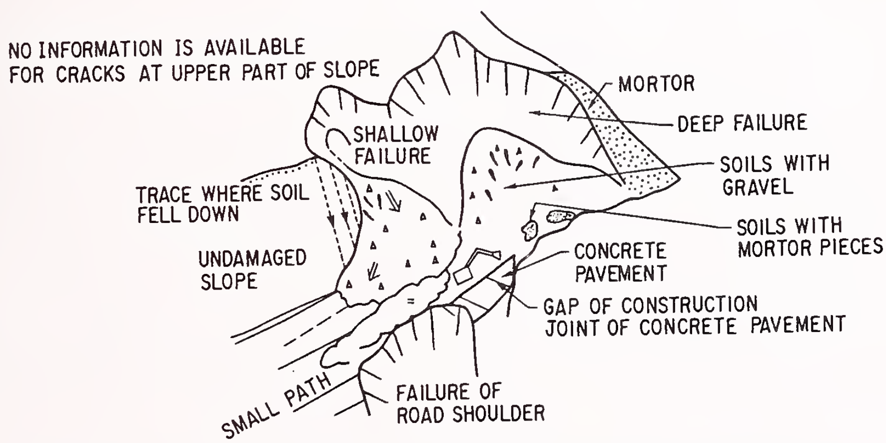


Fig. 8 Geological Feature Detected by Geologist A based on Video Film

Fig. 8 shows the geological feature detected by the geologist A based on the video film. Various information which is useful to judge the cause of failure and probability to cause further progress of the failure can be obtained if the video picture is carefully evaluated by a person with enough professional knowledge.

It was found from the study that estimation of the volume and length of failure is easier when artificial goods and facilities such as car, gird rails, house are found in the moving picture because they can be used as a reference to measure the distance[10]. On the other hand, it was found difficult to estimate the distance and length when the failure is enormously large as compared with the size of such artificial goods and facilities.

The study presented here is now being extended for studying the maximum flight velocity to detect a certain size of damage[11], effect of temperature and wind velocity for detecting damage by thermal image sensor[12], and the most appropriate way of flight[13].

CONCLUDING REMARKS

When the Kanto Earthquake which resulted in about 120 thousands of death occurred in September 1923, there were only about 4,500 vehicles in Tokyo City. People evacuated by either walking or bicycle. This clearly indicates the great difference of anticipated damage situation, between in 1923 and current time, in urban area hitted by a destructive earthquake. It is important to note that there have not be destructive earthquakes which hitted badly urban areas in the last 40 years, and during this time the size, structures of living, lifelines and techniques completely changed. It is therefore anticipated to face a new situation when a large event happens near urban area. It is quite important to develop and introduce a new systems for collecting and processing seismic damage information.

FUTURE DIRECTIONS

Because the proposed systems would become a large systems consisting of a number of hardwares and softwares, further study has to be made for an integrated architecture so that they are capable to be used even after a destructive earthquake. Special training for operating the systems is inevitably important. Because sudden operation immediately after an earthquake can not be made, it has to be used as a part of the comprehensive systems for daily use. Appropriate form of the emergency operation systems in normal time has to be well examined.

Although the thermal image sensors are effective to detect large damage at night, they can be used for surveying more detailed damage if the camera with longer focus distance was developed. The use of new technology such as the high vision camera is promising.

ACKNOWLEDGEMENTS

In executing the project, the Technical Research Center for National Land

Development established the Committee for the Development of Disaster Information Systems, which is chaired by Dr. H. Umemura, Professor Emeritus of the University of Tokyo, with the commitment of the Ministry of Construction. The reports presented in this paper are being executed with the supervision by the Subcommittee for the Development of Supporting Systems for Disaster Restoration Plan (Chairman : Professor Emeritus K. Kubo) and the Working Group for the Development of Supporting Systems for Seismic Disaster Restoration Plan (Chairman : Professor E. Kuribayashi) . The authors express their sincere thanks to all committees members for their invaluable suggestions and guidance. Special thanks go to Prof. T. Katayama, University of Tokyo and Prof. H. Kameda University of Kyoto, for their invaluable suggestions. Final accomplishments of the project are to be complied in March 1992 in the form of guidelines.

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ESTIMATION OF DEGREE OF ANXIETY FELT BY PEOPLE IN UNDERGROUND URBAN SPACES DURING EARTHQUAKES

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ABSTRACT

This study aims at discussing measures to be taken in underground urban spaces utilized by many people for disaster prevention during earthquakes. The objectives of this paper are; to identify a structural model of anxiety felt by people in the underground during earthquakes; to extract major factors which will help to reduce people's anxiety; and to estimate the degree of anxiety by the model through a survey on existing underground shopping malls.
(1)The most effective measures for reducing the degree of anxiety in underground spaces during earthquakes include the training of guides, the construction of emergency facilities and equipment, and training for disaster prevention.
(2)Comparison between new and old underground mall shows that the degree of anxiety regarding to information about refuge, and to facilities and equipment for refuge have improved in new underground mall. However, the degree of latent anxiety and anxiety regarding to leading people to refuge have improved only slightly.

INTRODUCTION

The current social background including over-development and a rise in land prices in large cities of Japan requires new spaces to be developed and utilized in order to maintain and improve city functions. Recently, underground spaces have been spotlighted because they are considered to be appropriate to this purpose. The underground spaces must be artificially closed. Hence, the underground spaces to be utilized by general people should be fully examined legally, economically, technically, and from the standpoints of utilization and disaster prevention. This study aims at discussing measures to be taken in underground urban spaces utilized by many people for disaster prevention during earthquakes.

As an example of existing underground spaces, shopping malls are amongst them. The first underground mall in Japan is "Subway Store" which was opened in 1932 in Kanda-Suda-Cho. Existing large-scale underground malls were mostly built in the 1960s-1980s. There have been only a few disasters in underground spaces. Most of them were caused by fires. Although earthquakes are frequent in Japan, no earthquake has ever caused a disaster in underground spaces.

According to the questionnaire[1] which was conducted in 1990 on the count of 720 people, people were most anxious about fire occurrences, then earthquakes, and finally other probable accident related to underground malls. During an earthquake, they were anxious about suffocation by smoke, an explosion caused by gas leak, people in a panic state who are pushing and shoving toward exits, and the collapse of malls. The other questionnaire[2] which was conducted in 1978 on the count of 1,713 managers and employees of stores in underground malls showed that about half of them were anxious of an attack by an earthquake while staying underground.

As part of this study, brainstorming[3] on "Disaster Prevention Caused by Earthquakes in Underground Urban Spaces" was carried out on the assumption that anunderground structure was attacked by an earthquake. Results of this brainstorming are; the basic points of prevention of disasters caused by an earthquake which are to protect people and to reduce people's anxiety. These points include providing emergency equipment, protecting underground facilities, furnishing adequate informations, realizing the similar environment as that above the ground, giving disaster drills, training against disasters, and making the best use of underground environment.

The objectives of this paper are; to identify a structural model of anxiety felt by people in the underground during earthquakes; to extract main factors which will help to reduce people's anxiety; and to estimate the degree of anxiety by the model through a survey on existing underground shopping malls.

STRUCTURAL MODEL FOR ANXIETY

Procedures for making the structural model are as follows:
Extracting factors of the proposition "Anxiety during Earthquakes".
Evaluating the relation between the factors through a questionnaire.
Identifying an objective structural model by the FSM (Fuzzy Structural Modeling) method[4]. The FSM method, an extended version of the ISM(Interpretative Structural Modeling) method[5], can consider fuzziness related to correlation of the factors.

Factors of Anxiety

A. Factors of latent anxiety
B. Factors related to damages (direct and indirect damages)
C. Factors related to information about refuge
D. Factors related to leading of people to refuge
E. Factors related to facilities and equipment for refuge
F. Factors related to actions to be taken during an earthquake (action by the general public in the underground spaces)

Based on the results of the brainstorming carried out as part of this study, these above major factors for people's anxiety during earthquakes are classified with reference to causes for a panic during a disaster[6], causes for a panic in underground malls[7], and an analysis of behavior in houses[8]. Subordinate factors for each major factor are shown in Table 1. The structural model is made on the assumption that major factors are independent from each other.

Evaluation Between Factors and Structuring

Pairwish relations between every major factor and its subordinate factors are determined by questionnaire. Then the questionnaire must include a total of 1,002 questions. Answers for the questions range from 0.0 to 1.0 so that fuzziness is taken into account in structural modeling. The number of examinees was sixteen: nine men including seven civil engineers and two students specializing in civil engineering and seven women including five office workers and two housewives. Because answers were expected to be different between men and women, answers were classified into three groups; men's, women's, and all examinees. Then each group answers' average were taken. Based on these averages, a binomial matrix (a fuzzy dependent matrix which is equivalent to an adjacent matrix by the ISM method) was determined. The parameters p and λ for the FSM method described in the next section refer to the threshold and fuzzy structural parameter respectively. Unless otherwise specified, they are set as follows; p=0.5 and λ =-0.3.

Structural Model

Structural Model for Major Factors

The highest hierarchy in Figure 1 is the proposed structural model. It apparently shows that the proposition, "Anxiety during Earthquakes", is very closely connected to "factors of latent anxiety" and "factors related to damages". Moderately connected are the "factors related to information about refuge", "factors related to leading of people to refuge", and " factors related to facilities and equipment for refuge". The "factors related to actions to be taken during earthquakes" is slightly connected.

Structural Model for Latent Anxiety

In calculation by the FSM, thresholds to which the asymmetrical law applies are set as follows for the three groups of men, women, and all examinees respectively; p=0.67, p=0.82 and p=0.77. The higher threshold for the women group shows that women consider relations between factors to be strong although the causal relations between them are not clear. The hierarchy of factors of latent anxiety in Figure 1 is the basic structural model which was established by using the FSM method after examining the adequacy of this structural model. Factors of latent anxiety are resulting directly from the "unfavorable impression in the underground"(1). More specifically, the causes are "sounds echo"(5), "lack of the sense of time"(6), and it is "difficult to escape"(9). The degree of their relations shows that being " difficult to escape" is the strongest cause for the "unfavorable impression in the underground". The "unfamiliar space"(2), "closed space"(3), "difficulty to rescue people under the ground"(4), and "artificial space"(7) are factors in the lowest hierarchy. Therefore, latent anxiety can be reduced most effectively by providing measures for these factors.

Structural Model Related to Damages (in Figure 1)

Just like factors of latent anxiety, the threshold is higher for women than men. The "recognition of damages"(11) is the factor in the highest hierarchy. Factors in the intermediate hierarchy can be divided into three blocks. The first block includes damages which people recognize, the second block includes indirect damages of underground facilities caused by an earthquake, and the third block includes direct damages. Factors in the lowest hierarchy are factors of insufficiency whose counter measures for earthquakes are in regard to underground facilities and equipment, fires caused by an earthquake, and prevention of the expansion of damages due to the fact that underground space is an artificial space which easily becomes a closed space. Therefore, providing measures for these factors in lowest hierarchy is the most effective measures to prevent people from recognizing damages.

Structural Model Related to Information about Refuge (in Figure 1)

Just like factors of latent anxiety, the threshold is higher for women group than men. The "inadequate information about refuge"(36) is resulting directly from "information transmitted by word of mouth"(37) and "low reliability of information about refuge"(39). Causes of "information transmitted by word of mouth" are "impossible to confirm information about refuge"(38), "delayed transmission of information about refuge"(40), and "lack of unity of information about refuge"(42). Causes of "low reliability of information about refuge" are "delayed transmission of information about refuge", "lack of unity of information about refuge", "impossible to confirm information about refuge", "shortage of transmitted information about refuge"(43), and "misunderstanding and insufficient confirmation of information about damages"(46). To avoid inadequate information about refuge, providing measures for factors in the lowest hierarchy is the most effective measures. Factors in the lowest hierarchy are: "impossible to confirm information about refuge", "operators of underground stores are not well trained"(47), "means to transmit information is not sufficient"(41), "shortage of means to gather information"(45), and "the centralized monitoring system is not sufficient"(48).

Structural Model Related to Leading of People to Refuge (in Figure 1)

Experiments and instances shows that even when people apparently know the route to escape, the degree of confusion is different according to whether there is someone or some announcements to lead people to refuge. For these models, there are few differences in between men and women. According to this model, the fact that "the guides do not know sufficient information or the information is inconsistent"(52) and "the guide is not well trained"(53) lead to "the shortage of guides"(54), which means one guide must leads more than the adequate number of people to refuge. As a result, "the guides become upset"(50) and "the guides' instructions become unclear"(49). Moreover, the guides "hurry people"(51), resulting in "inadequate guidance"(55). The key to minimize the inadequate guidance is to provide adequate measures for factors in the lowest hierarchy which are: "the guide does not know sufficient information or the information is inconsistent" and "the guides are not well trained".

Structural Model Related to Facilities and Equipment for Refuge (in Figure 1)
Due to the shortage of facilities and equipment for refuge, it takes a longer time to escape, resulting in greater anxiety. The most effective measures for this problem is to provide measures for factors in the lowest hierarchy: "the shortage of routes to refuge"(59), "the shortage of the refuge"(61), "the narrowness of routes to refuges"(62), and "the shortage of equipment to prevent damages"(64).

Structural Model Related to Actions to be Taken during Earthquakes (in Figure 1)
The structural model for all examinees (p=0.7) is adopted as the basic structural model for these factors. According to this model, the "people have no experience in a disaster"(69) which leads them to "make light of a disaster"(70). In this case ,the reason being that "they do not have a disaster drill in facilities they attend"(68). Then, including the fact that "they have not been trained against disasters"(66) lead to the result of "not knowing how to behave when an earthquake occurs"(65)

AHP INFERENCE MODEL

Figure 2 a) shows the basic structural model determined by the FSM method (p=0.5 and λ =-0.3) of the major factors. The proposition, "Anxiety during Earthquakes", is completely described by the six major factors. On this assumption, the degree of connections between the proposition and each major factor can be shown by the proportional scale. The AHP (Analytic Hierarchy Process)[9] estimate the degree of overall importance (or weight) of the whole hierarchy. The degree of importance of elements (or factors), the proportional scale, in each level of hierarchy, with respect to an element of a next higher level is determined by pairwish comparisons. In Figure 2 b), the degree of connections is transformed into the degree of importance. This hierarchical model was adopted as the AHP inference model for the first and second level. The AHP inference model for the third level or lower hierarchies were also determined in the same way as the higher levels.

Figure 3 shows a hierarchical model for major factors of latent anxiety. Factors in the lowest hierarchy act as higher level elements for input elements. In this model, the input elements are from Q1 to Q8. The degree of latent anxiety is inferred by choosing between two options for each question. For example, based on the assumption that you are in a certain underground space, Q1 asks how much one feel annoyed by echoing sounds in closed space. You answer how much you feel annoyed by choosing a number from 1 to 9 for intensity of importance. The value which show how much one does not feel annoyed is the reciprocal value of intensity of importance. Table 2 shows the definition of the values from 1 to 9. In each binomial matrix of answers, the eigen values is calculated. The eigen vector of the maximum eigen value shows the degree of importance. Table 3 shows the degree of importance for each value from 1 to 9 as an alternative answer.

FACTORS REDUCING ANXIETY EFFECTIVELY

Figure 4 shows the results of the sensitivity analysis by the AHP inference model. Procedures for it are as follows:

(1) Enter 1 for all input elements.
(2) Enter 9 for one input element.
(3) Estimate the overall importance (weight), which is adopted as the sensitivity of the input element chosen in procedure (2).

The factor which shows the highest sensitivity in each major factor is : "the lack of sense of time"(6) caused by "closed space" for factors of latent anxiety; "the shortage of facilities and equipment for emergency"(32) for factors related to damages; "the centralized monitoring system is not sufficient" for factors related to information about refuge; "the guides are not well trained"(53) for factors related to leading of people to refuge; "the shortage of equipment to prevent damages in routes to escape and refuges"(64) for factors related to facilities and equipment for refuge; and "people have not been trained against disasters"(66) for factors related to actions to be taken during earthquakes. Among these, the factor with the highest sensitivity is "the guide is not well trained" for factors related to leading of people to refuge, followed by "the shortage of facilities and equipment for emergency" for factors related to damages and "people have not been trained against disasters" for factors related to be taken during earthquakes. Therefore, providing sufficient measures to train guides, furnishing facilities and equipment for emergency and training the general public will help to reduce anxiety felt by people during earthquakes.

ESTIMATION OF DEGREE OF ANXIETY EXISTING IN UNDERGROUND MALLS

Using the AHP inference model, we have estimated how much anxiety people feel about existing underground malls during earthquakes. In the AHP inference model, values for input were collected through questionnaires.

Questionnaire

A questionnaire consists of 28 questions for all input elements for the inference model. To verify the results of the inference, a question to the proposition about anxiety during earthquakes was added. The questionnaire was attached with criteria for underground malls and related data for reference. In the questionnaire, examinees answered the questions after seeing a simulation film. Two underground malls which were built in the 1950s and after 1985 respectively were used in the simulation film. Therefore,the examinees answers were in regard to the new and old underground mall. The number of examinees was twenty one, of which fifteen were men and six were women. The breakdown of the men examinees are: ten civil engineers, four university students and an elementary school student. The breakdown of the women examinees are: two office workers, three housewives, and an elementary school student. The questionnaire was based on the assumption that the examinees were hit by the Kanto Earthquake while staying in the underground mall.

Estimated Results

Figure 5 shows the frequency distribution for the estimated degree of anxiety related to each major factor. The ordinate represents the frequency and the abscissa the degree of anxiety which is shown in Table 4. For the old underground mall, the sections from "almost yes" to "absolutely yes" showed higher frequency for each major factor excluding factors related to action to be taken during earthquakes. In contrast, for the new underground mall, the sections from "rather no" to "rather yes" showed higher frequency. Therefore, we predict that anxiety related to each major factor excluding factors related to action to be taken during earthquakes in regard to old underground malls are stronger than new underground malls. Since questions for factors related to actions to be taken during earthquakes ask about experiences of an earthquake and a disaster drill, answers should be the same between old and new malls. Therefore, differences between them may be attributed to the examinees' vague memories. The frequency distributions spread more widely for the new underground mall than the old underground mall. In particular, factors related to leading of people to refuge and factors related to facilities and equipment for refuge showed wide frequency distributions. This may show that although elaborate disaster measures are expected to be taken for new underground malls compared to old ones, public estimation of these new measures is diversified.

Figure 6 shows frequency distributions for the estimated degree of anxiety and results of the questionnaire which show the degree of anxiety during earthquakes while staying in the old and new underground mall. Both the estimates and the questionnaire results show that anxiety is weaker for the new underground mall than the old ones. However, the results of the estimates and the questionnaire did not fully coincide with each other.

Figure 7 shows the average values of anxiety related to each major factor and anxiety during earthquakes. Characters from A to F correspond to the major factors. "G" in the figure means values of estimated degree of anxiety during earthquakes and "H" means the values of results from asking about the degree of anxiety during earthquakes.

Figure 7 a) shows that there are big differences between the new and old underground mall regarding to factors related to damages and factors related to facilities and equipment for refuge. In view of this, we predict that these factors for the new underground mall will thought to have greatly improved. However, it is obvious that factors of latent anxiety and factors related to leading of people to refuge have improved only slightly. Moreover, the results of the questionnaire and the estimates coincide fairly well with each other. The figure also shows that anxiety regarding to the new underground mall is weak compared to the old underground mall in the sections from "yes" to"cannot say".

Figure 7 b) shows that there are big differences between the new and old underground mall regarding to factors related to damages and factors related to facilities and equipment for refuge. In view of this, we predict that for the new underground mall will thought to have greatly improved. However, it is obvious that factors of latent anxiety and factors related to leading of people to refuge have improved only slightly. In particular,improvements in factors related to leading of people to refuge have increased only a small amount. Moreover, the results of the questionnaire and the estimates of the proposition coincide fairly well with each other. The figure also shows that anxiety regarding to the new underground mall is weak compared to the old underground mall in the sections from "rather yes" to "cannot say". The samples' average value for the degree of anxiety related to major factors is higher for women than men. The value for the proposition, "Anxiety during Earthquakes", shows the same trend between men and women.

CONCLUSIONS

Conclusions of the study are as follows:

(1) The results of the sensitivity analysis tell that the most effective measures for reducing the degree of anxiety in underground urban spaces during earthquakes include the training of guides, the construction of emergency facilities and equipment, and training for disaster prevention.

(2) Comparison between the new and old underground mall shows that the degree of anxiety regarding to factors related to information about refuge, and factors related to facilities and equipment for refuge have improved in the new underground mall. However, the degree of anxiety regarding to factors of latent anxiety, and factors related to leading of people to refuge have improved only slightly. This trend is more apparent for women than men.

(3) Women examinees have stronger anxiety in underground malls during earthquakes and also feel more anxious about major factors causing the anxiety in underground malls than men examinees do.

(4) The samples' average of the questionnaire carried out in the new and old underground mall on the degree of anxiety show nearly the same tendency as those of estimation by the structural model. This indicated that the structural model can be considered to be almost appropriate.

FUTURE DIRECTIONS

We will continue the survey by questionnaire so as to gain more reliable data. We will also examine measures to reduce anxiety during earthquakes in view of the results of the survey so far.

Main items of measures are as follows.

1) Reducing the degree of latent anxiety.

2) Training method to guides and people.

3) Making new emergency facilities and equipment for disaster prevention.

We would like to continue collaboration between present researchers and with other related fields in this study.

ACKNOWLEDGMENTS

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Table 1 Factors for Anxiety

● FACTORS OF LATENT ANXIETY

1.Unfavorable impression in the underground
2.Unfamiliar space (have never been in the space)
3.Closed space 4.Difficult to rescue people under the ground (far from over the ground)
5.Sounds echo (noisy, cannot catch information)
6.Lack of the sense of time (difficult to tell what time is it)
7.Artificial space (power supply, ventilation and supply of oxygen may be cut off)
8.Lack of the sense of direction (position and direction, difficult to tell where you are)
9.Difficult to escape (unable to escape)
10.Secondary disaster may occur.(Ex. a fire,filling smoke)

● FACTORS RELATED TO DAMAGES

11.Recognition of damages 12.Feeling unusual heat 13.Eyes tearing from the gas
14.Having difficulty in breathing
15.Catching an unusual smell (smell of gas and something burning)
16.Hearing something to get damaged 17.Hearing a scream 18.Things fall
19.Things shake 20.The underground structure cracks. 21.A fire breaks out.
22.The underground space is filled with smoke. 23.Water Leaks.
24.The electric supply is cut off. 25.Hazardous gas is generated.
26.The underground space is closed.
27.Facilities and equipment to protect the underground environment are damaged.
28.Structures are not designed to absorb vibrations caused by earthquakes.
29.Shortage of facilities and equipment for fire extinguishing purposes
30.Things are not made sufficiently fire-proof.
31.Facilities and equipment are not sufficiently waterproof.
32.Shortage of emergency facilities and equipment
33.Facilities and equipment are not sufficiently of seismic design.
34.Fire is used 35.Walls crumble.

● FACTORS RELATED TO INFORMATION ABOUT REFUGE

36.Inadequate information about refuge (people do not believe the information or listen to it)
37.Information transmitted by word of mouth (rumor etc.)
38.Impossible to confirm information about refuge (or the general public)
39.Low reliability of information about refuge
40.Delayed transmission of information about refuge
41.Means to transmit information is not sufficient.(communication media)
42.Lack of unity of information about refuge
43.Shortage of transmitted information about refuge
44.Impossible to confirm whether information has been transmitted (one-way transmission)
45.Shortage of means to gather information (by store operators)
46.Misunderstanding and insufficient confirmation of information about damages
47.Operators of underground stores are not well trained.
48.The centralized monitoring system is not sufficient.

● FACTORS RELATED TO LEADING OF PEOPLE TO REFUSE (BY GUIDES)

49.The guides' instructions become unclear.(about route and refuge)
50.The guides become upset. (a panic may occur.) 51.The guides hurry people.(make people run)
52.The guides do not know sufficient information or the information is inconsistent.
53.The guides are not well trained.
54.The shortage of guides (one guide must lead more than the adequate number of people to refuge)
55.Inadequate guidance

● FACTORS RELATED TO FACILITIES AND EQUIPMENT FOR REFUGE

56.People cannot find routes to refuge easily. 57.People take a long time to escape.
58.Guides cannot find facilities and equipment to lead people for refuge easily.
59.The shortage of routes to refuge 60.Routes to refuge are complex.
61.The shortage of refuges 62.The narrowness of routes to refuge
63.There are no definite refuges. 64.The shortage of equipment to prevent damages

● FACTORS RELATED TO ACTIONS TO BE TAKEN DURING EARTHQUAKES

65.People do not know how to behave during an earthquake
66.People have not been trained against disasters.
67.People do not know how to use emergency equipment.
68.People do not have a disaster drill in facilities they attend.
69.People have no experience in a disaster. 70.People tend to make light of a disaster.
71.People do not know where emergency equipment is.

Table 2 Intensity of Importance

Intensity of Importance	Definition	Intensity of Importance	Definition
1 3 5 7	Cannot say Almost yes Yes Rather yes	9 2,4,6,8	Absolutely yes Intermediate values between the two adjacent judgements

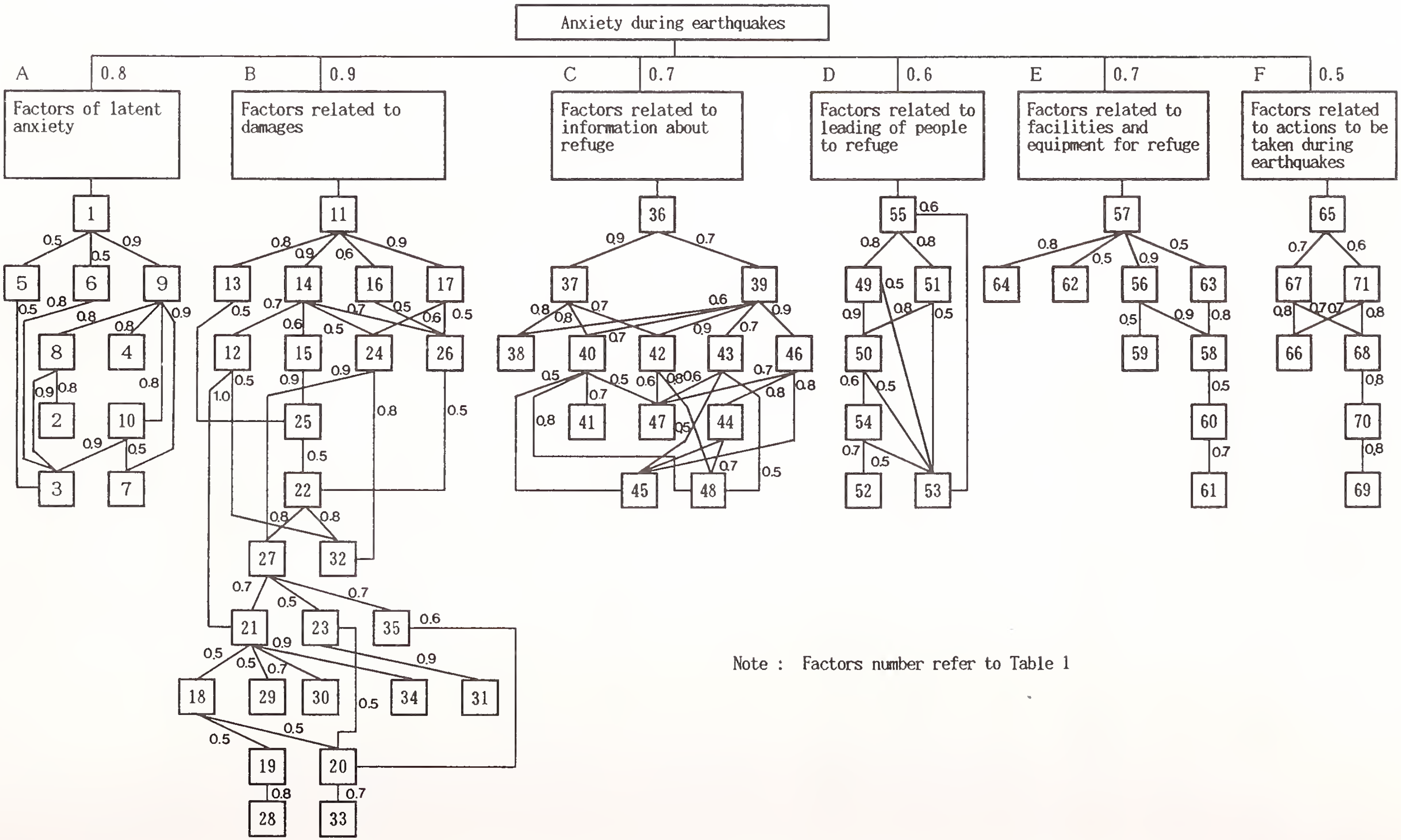
Note: The reciprocal value is used for the opposite meaning.

Table 3 Degree of Importance of Alternative Answer

Answer	1	2	3	4	5	6	7	8	9
Degree of Importance	0.50	0.57	0.75	0.80	0.83	0.86	0.88	0.89	0.90

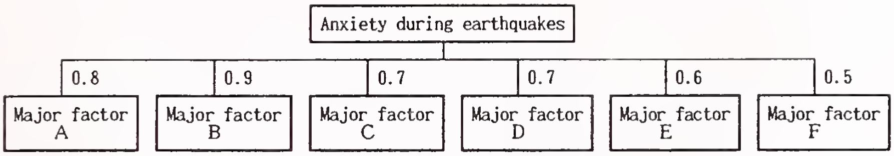
Table 4 Degree of Anxiety

Scale	Degree of Anxiety	Definition
1	0.10-0.12	Absolutely no - Rather no
2	0.12-0.17	Rather no - No
3	0.17-0.25	No - Almost no
4	0.25-0.50	Almost no - Cannot say
5	0.50-0.75	Cannot say - Almost yes
6	0.75-0.83	Almost yes - Yes
7	0.83-0.88	Yes - Rather yes
8	0.88-0.90	Rather yes - Absolutely yes

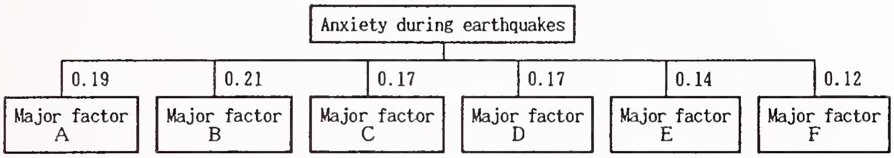


Note : Factors number refer to Table 1

Figure 1 Hierarchy for Anxiety during Earthquakes



a) Structural model by the FSM method



b) Hierarchical model for the AHP

Figure 2 Hierarchical Model for Major Factors

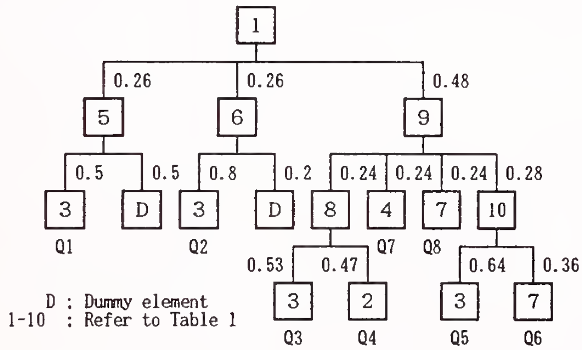


Figure 3 Hierarchical Model for Factors of Latent Anxiety

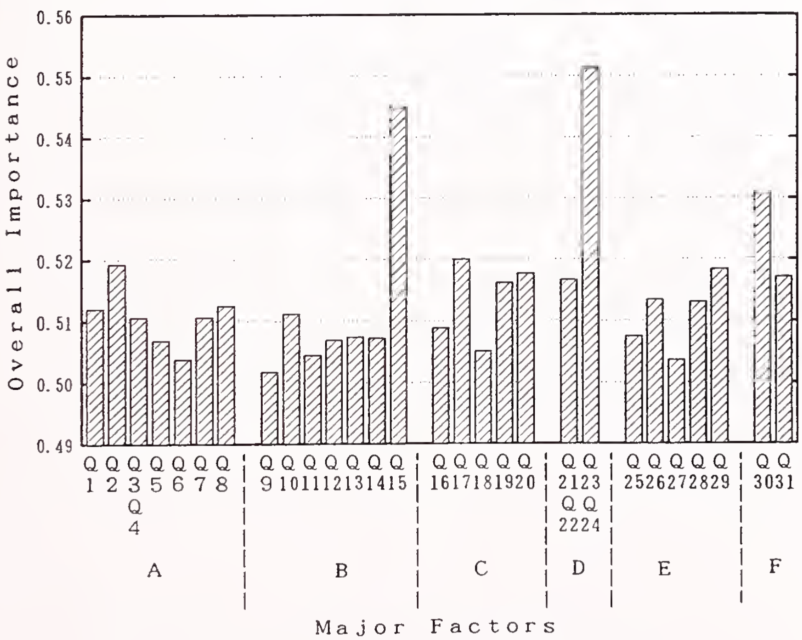


Figure 4 Sensitivity in Each Major Factor

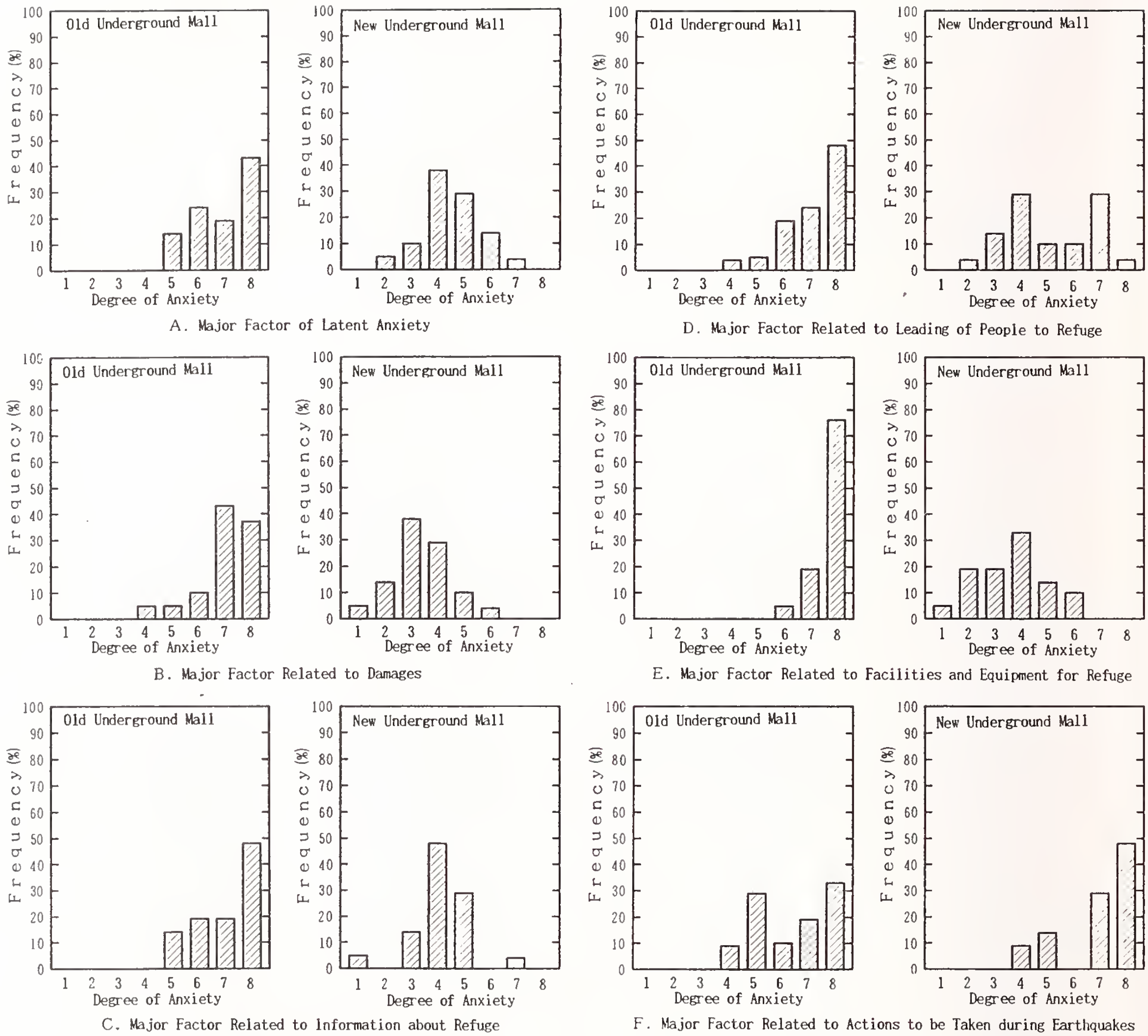
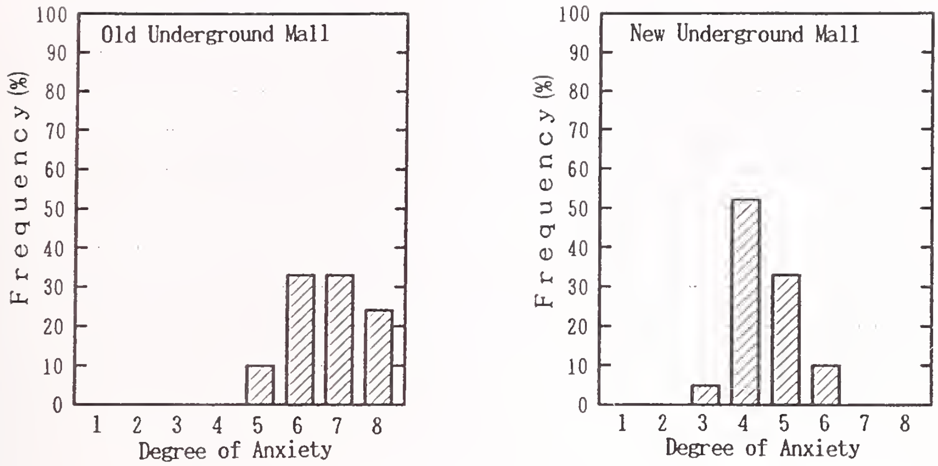
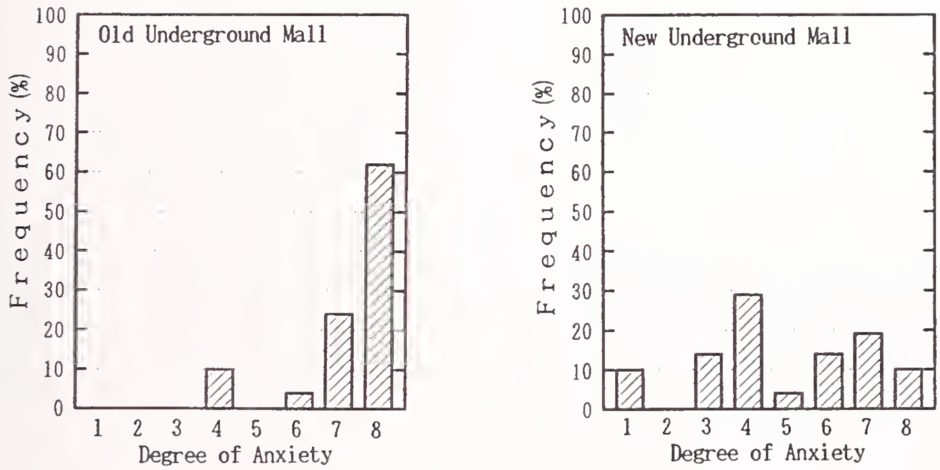


Figure 5 The Frequency Distribution for Estimated Degree of Anxiety Related to Each Major Factor

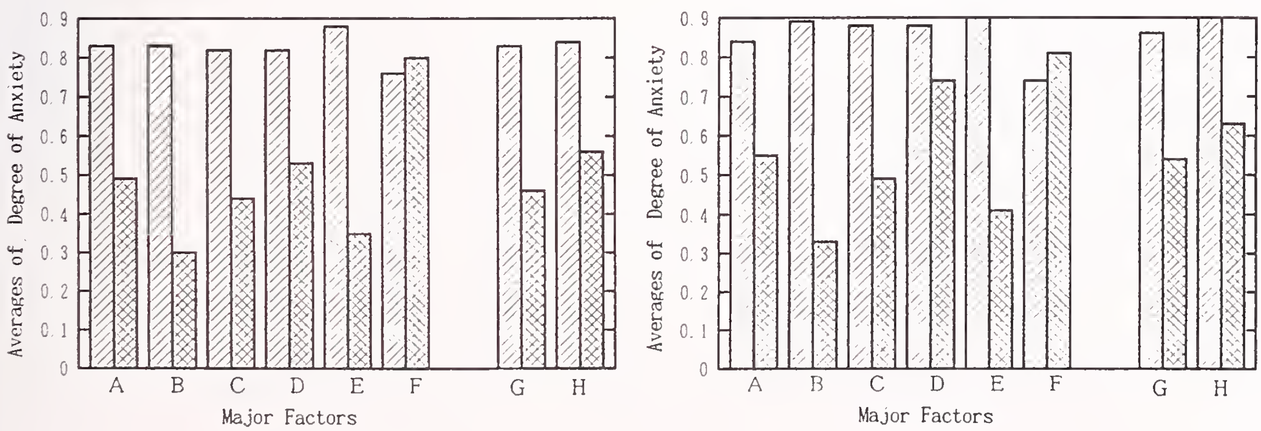


a) Estimate by The AHP



b) Questionnaire Results

Figure 6 The Frequency Distribution for Estimated Degree of Anxiety



a) Examinees (Men)

b) Examinees (Women)

Figure 7 Samples' Averages of Degree of Anxiety
(▨ : Old Underground Mall, ▩ : New Underground Mall)

PA9146

Performance of Lifelines and Emergency Response in Watsonville, CA To Loma Prieta Earthquake

4th U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems

Los Angeles, August 19-21, 1991

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Abstract

Due to disruption of lifelines in the Loma Prieta earthquake, communities such as Watsonville, CA suffered economic losses and their emergency response staffs were severely challenged to provide essential public safety and health services. Watsonville is typical of many cities in that services are partly provided by municipal utilities (water, sanitary sewage) and partly by outside utilities (gas, electricity, telecommunications). The most significant problem for Watsonville was failure of electrical power. Until power was restored several days after the earthquake, the City depended entirely on emergency generators to operate water and sanitary sewer systems, support hospital services and operate emergency fuel pumps. The resources of the Watsonville Fire Dept. were completely deployed; no additional emergency situations could have been met effectively.

The impact on public safety and convenience resulting from damage to the water system was minimized because damaged regions were quickly isolated. Watsonville Community Hospital was able to provide uninterrupted medical care because community members volunteered to provide water and fuel for emergency generators. Watsonville businesses suffered inconvenience due especially to loss of electrical power. The major costs to businesses were due to physical damage, cleanup and revenue lost due to suspended operations.

INTRODUCTION

Due to disruption of essential utility services, or lifelines, in the Loma Prieta earthquake, communities such as Watsonville, CA suffered economic losses and their emergency response staffs were severely challenged to provide essential public safety and health services. Under its Lifeline Systems Program, the National Center for Earthquake Engineering Research (NCEER) has begun to study how a community provides essential services despite damage to lifeline systems and the costs of restoring those services following the Loma Prieta earthquake. Watsonville, located 15 miles southeast of the city of Santa Cruz, is situated in a valley near the Pajaro River; most of the city is within 10 miles of the fault rupture. The size of Watsonville (6 sq. miles, population 29,000, $1.4 billion total assessed valuation of property) makes it feasible to consider the whole machinery of utility and emergency services working in the aftermath of a damaging earthquake.

Watsonville is typical of many cities in that services are partly provided by municipal utilities (water, sanitary sewage) and partly by outside utilities (gas, electricity, telecommunications). As pointed out in Ref. 1, it is widely assumed that the costs of repairs will be borne by the Federal Government in the case of public utilities; by local tax payers; and by adjustments in rate structure for private utilities whose repair costs are not covered by grants in aid. The actual experiences of utilities serving Watsonville provide valuable insight into the merits of lifeline hazard mitigation and emergency preparedness from the standpoints of sustaining public safety functions and of minimizing costs.

The total cost of earthquake damage to property in Watsonville was about $60 million. The total cost of repairing earthquake damage to municipally-owned property was about $4 million. This is about $140 per person and about 0.29% of assessed valuation. Of this amount, about $2.5 million was reimbursed by outside agencies; the remaining $1.5 million will be financed by the City over Fiscal Years 1989/90 and 1990/91. Though primarily a residential and commercial community, Watsonville also has industry which relies on water and electric power for continued operation. The costs of plant closure and lost produce may be considered together with costs of restoring lifeline services in justifying expenditures for lifeline hazard mitigation and emergency preparedness.

OVERVIEW: PROMPT DAMAGE AND EMERGENCY RESPONSE

The most significant problem for the Watsonville community was failure of electrical power supplied by the Pacific Gas and Electric Co. (PG&E) from its Moss Landing substation located about 8 miles south of the city. The substation sustained damage to its 500-kv and 230-kv switchyards (Ref 2). Until power was restored several days after the earthquake, the City depended entirely on emergency generators to operate water and sanitary sewer systems, support hospital services and operate emergency fuel pumps. The resources of the Watsonville Fire Dept. were completely deployed; no additional emergency situations could have been met effectively. City officials stated that their emergency response, while effective, could have been enhanced by having more emergency generators available and by recognizing and accepting the willingness of neighboring communities to provide emergency aid.

The impact on public safety and convenience resulting from damage to the water system was minimized because damaged regions were quickly isolated by the use of valves and because emergency generators were already in place to operate pumps. However, the topography of the service area dictates a system that has large areas serviced by a single source. This caused large areas to be without service due to a single source failure. Repair operations were seriously restricted because the regional emergency response plan did not provide for passage of water and sewer maintenance vehicles and equipment. With stop signals not functioning, this became a serious problem for the repair forces. The impact of damage to the sanitary sewer system was mitigated because a private contractor with a portable generator operated the lift pumps until power was restored. Watsonville Community Hospital was able to provide uninterrupted medical care. Only one of its two emergency generators was useable immediately after the earthquake, however, and the emergency fuel reserve was inaccessible due to inability to operate the fuel pump; community members volunteered to provide water and fuel. Watsonville businesses suffered inconvenience due to loss of lifeline service, especially loss of electrical power. The major costs to businesses were due to physical damage, cleanup and revenue lost due to suspended operations; loss of frozen inventory also contributed to economic losses.

GEOLOGY AND SEISMOLOGY

Zones of potential liquefaction in the Watsonville area are described in Dupre and Tinsley (Ref. 3). Liquifaction-related ground failure occurred within about 150 feet of major water courses draining the area around the Pajaro River, to the south of the City, and Corralitos Creek which passes through a residential district near downtown Watsonville. Approximately 80% of the areas exhibiting such failures are also reported in Youd and Hoose (Ref. 4) as having failed in a similar manner following the 1906 earthquake.

The Modified Mercalli Intensity of shaking in Watsonville due to the main Loma Prieta event has been estimated at VIII-IX. California Division of Mines and Geology strong motion station No. 1615, which is located in the Pacific Bell telephone building in downtown Watsonville, recorded a peak horizontal acceleration of 0.39g and a duration of strong shaking of about 6-8 seconds (Ref. 5).

COST OF PROMPT DAMAGE

The value of losses to municipal property in Watsonville has been estimated at $4 million. About 65% of costs of restoring property has been paid by or claimed from the Federal Emergency Management Agency (FEMA) of the U.S. Government. Although some tax collections, especially sales taxes, are below the level of the preceding fiscal year, the City is able to meet its fiscal commitments.

Interaction between FEMA and City officials following the earthquake sometimes left the City without clear guidelines on what costs would be paid by the government. For example, application to FEMA for funds to assess damage to the sanitary sewer collection system did not receive consistent response. FEMA is going to assist in restoring damaged sewer lines based on a 25% deductible on the repair of sewer line. In the first week following the earthquake, FEMA told the City it would pay to have all lines inspected by video camera; however, in the second week FEMA told the City to reduce the scope of the inspection; in the third week FEMA said if video showed no damage, it would not pay for the inspection. A major reason for such confusion is the lack of experience with local conditions on the part of FEMA representatives and with federal disaster relief procedures on the part of the City. Efforts to improve communications are in progress.

PERFORMANCE OF INDIVIDUAL LIFELINE COMPONENTS

Water System

The present system was built in the 1920's. There are nine pressure zones in the city, with interconnections between eight of these. The 11,800 connections in the system supply water to approximately 45,000 persons. In the earthquake, damaged areas were isolated by manual valves, so that leaks downtown did not bleed the entire system. No structures were lost due to a lack of water. Three mobile homes and one fixed dwelling were engulfed by the time fire trucks arrived; according to the Watsonville Fire Department, they would have been lost under any circumstances. However, if water had been available, adjacent buildings would have been less damaged.

Except for a few isolated areas, service was restored to all customers within 24 hours. Lack of water was regarded as an inconvenience for customers, but without major impact. There was a period of a few days when water pressures were low enough to present the threat of contamination. Residents were advised to use chlorine bleach and not to drink murky or dirty water. Subsequent testing found no contamination.

Water Supply

Eleven wells supply the main needs of the system (approximately 85% of the pre-earthquake capacity of 13,300 gpm or 19 mgd). All water supplies are chlorinated. Raw water supply, from two watersheds, is obtained through steel transmission pipes, and purified in a slow sand filtration plant.

A chlorine leak in a 150 pound cylinder could have threatened nearby residents. Power failures interrupted pumping at two small wells; however, these two small stations represented less than 1% of the system capacity. The remaining wells continued to pump using emergency generators. Fuel tanks for these generators were too small, and a person was completely dedicated to refueling them. The earthquake cracked floors of the sedimentary basin in the slow sand filtration plant. This damage to the plant (it is currently not operable) has reduced the total capacity by 15%.

Storage

One redwood, four concrete, and four steel tanks varying in capacity from 0.3 to 3.0 mg provide total storage of 11.7 mg. A one million gallon steel tank (built 1971) buckled at the wall-roof juncture, due to failure of a bracket. The water level transmitting device was broken also. There was a leak of the pilot line to the altitude valve. The tank did not leak, however.

Water Transmission/Distribution

Twenty five major breaks appeared immediately. There had been 56 main breaks as of January 31, 1990. The Zone 1 (downtown) system had breaks in 8"-14" pipes. With the power outage and the breaks, maintenance of pressurization was impossible, creating the potential for contamination (water testing revealed no contamination). According to the Watsonville Fire Department, the estimated flow capacity in the zone 1 area was completely inadequate (150 gpm) immediately following the earthquake. There was no flow in the higher areas for several hours, due to the need to close valves manually that interconnect parts of the system.

Storm Drain System

The storm drain system serves the entire City and surrounding tributary areas. This system collects and transports stormwater originating within the five drainage basins in and around the City. The stormwater is then discharged into downstream natural drainage channels. Stormwater is typically conveyed by gravity flow in open channels and pipes ranging from 12 inch to 60 inch diameter. This system also includes thirteen pump stations located along the Salsipuedes Creek and Pajaro River to handle stormwater discharge during periods of high tailwater. To date there have been 123 damage sites discovered. Two pumping stations were damaged. The impact to residents of the city was minimal, generally due to the lack of rainfall at the time of the earthquake, and immediately afterward.

Sanitary Sewer System

The sanitary sewer system serves the entire city and Pajaro Dunes, a total population of about 29,000. Average flow is about 8 mgd, with peak flow up to 13 mgd. Effluent is chlorinated and discharged to Monterey Bay through a 36 inch concrete lined steel outfall. It is mainly a gravity fed system, with 9 or 10 lift stations (4 or 5 from Pajaro Dunes). The system is comprised mainly of vitrified clay pipe, which was originally laid about 1900.

The earthquake did not immediately block sewer lines. Inspection of several sewers indicated broken pipes, misalignment and grade changes. Soil liquefaction was a major cause. Due to loss of power, sewage could not be lifted into the gravity part of the system. The City hired a private firm to provide local pumping until power was restored. About 150 sewer repair spots have been identified, primarily in the downtown area. The City considered installing inner liners, but the cost was prohibitive. The land portion of the outfall from the treatment plant developed two leaks where it crossed unstable soil in a low lying area. There were no serious interruptions of sewer service due to the earthquake. Minor overflows occurred. Surface flow of sewage was confined to the break in the outfall line described above.

Gas

The response by PG & E to damaged gas lines is described in Ref. (6).

Watsonville System

There were two major gas leaks due to the earthquake; the intersection of Beach and Brennan streets, and on Hill Street near Tuttle Avenue. There were numerous smaller leaks, including the building fires caused in part by the building shearing the service line at the meter. The fire department shut off the gas to some buildings. The fire department also supplied PG&E with a portable radio to allow faster communication of serious gas leaks.

Watsonville is in PG & E's Mission Trail Region. There were initially 29,000 customer outages in the region (breakdown for Watsonville not available). Approximately 200 homes fell off their foundations and severely damaged the gas lines, and the old cast-iron and steel low-pressure system was shutdown because of widespread leakage (see also water department report on fires in mobile homes).

Most of the downtown Watsonville gas system consisted of old steel pipes, varying in size from 2" to 4" and in age from pre-1930 to 1974. Ref (6) details damage to the main distribution pipes (Exhibit 9 of report). About 85% of the system has been replaced since the earthquake. New 2" plastic pipes were either inserted through the old pipes, whenever possible, or laid into new trenches. The plastic pipes can take higher pressure (30 psig) but a regulator is needed at the service connection to step the pressure down to the required 2-3 psig. According to a PG&E spokesperson, most new pipes installed will be these high pressure plastic pipes.

Emergency Preparedness

PG&E has emergency preparedness plans which include an emergency operations center to coordinate all crews and actions in the aftermath of an earthquake. Details are given in Ref. (6). Each region maintains a stockpile of emergency material, including pretested pipes. Several regions and divisions have established plans for mutual aid within the region and from other regions. PG&E recognizes the problem with fires, especially for mobile homes.

The distribution network is divided into emergency shutdown zones that are separated by valves. Each zone can range up to 40,000 customers. PG&E would not consider shutting off a zone unilaterally upon the start of an earthquake because:

1. It is difficult to tell which is the right zone to shut off; information from customers is not reliable.

2. If a zone is shut off unnecessarily, customers would be inconvenienced or would need the fuel for emergency purposes.

3. The relighting process is very slow, labor-intensive and expensive.

PG&E has a Supervisory Control and Data Acquisition system (SCADA) which is used to monitor the status of transmission and distribution facilities. An operator can shut off particular transmission lines, but this is not an automatic post-earthquake procedure.

Communications

The City emergency communication center was not available due to loss of power. Officials relied on face-to-face communication at hourly intervals. They had a generator to run the emergency communications, but the exhaust went into the AC system, so it was not useable. Radios in water trucks were used until power was restored. The need for additional tactical frequencies was noted; too many emergency crews for the number of bands available.

Roads, Bridges

The only road closures were the Struve Slough Bridge (Highway 1) and the Pajaro Bridge. Following the earthquake, traffic was slowed due to debris in the streets from damaged and collapsed buildings, gas leaks (above), lack of traffic lights due to a power outage, and from slow "sightseers". Some roads in the heavily damaged areas were closed afterwards, for public safety. Fire trucks were able to traverse all roads (except bridge closures); detours were made around intersections with the gas leaks. The Watsonville Fire Department said that response times were increased due to lack of accessibility. Due to lack of coordination in the emergency response plan, some pipeline repair crews were not allowed into closed off areas to repair pipelines. Watsonville will pay half the cost to replace Pajaro Bridge. This is not included in the $4 million estimate of damage.

Emergency Response

Seventy percent of the Watsonville Dept. of Public Works staff reported for work. Resources of the Watsonville Fire Department were completely deployed. One added emergency, such as another building collapse, would have been met with diminished resources. The Asst. Fire Chief said that the Disaster Plan was effective and would not be substantially altered to meet future emergencies. An exception is that the City mistakenly assumed that there would be no outside help, so they never requested any. In fact there were significant resources available from neighboring communities. Eventually, coordinating volunteer helpers became a burden.

Watsonville was the first city or among the first in California to complete a Hazard Mitigation and Emergency Operations Plan; it was completed in 1985. Several City employees attended California Special Training Institute Workshop on Disaster Preparedness conducted by the California State Office of Emergency Services.

Medical Services--Watsonville Community Hospital

Watsonville Community Hospital provided essentially uninterrupted medical care for the community after the earthquake. While medical care was available, service was affected by the earthquake, at least for a number of days after the event. The need for modified medical service resulted primarily from interruptions to hospital lifelines supplies. Key lifeline interruptions are highlighted below.

Electric Power

The hospital's electric power supply, provided by PG&E, was interrupted for several days following the earthquake. Emergency power is provided by the hospital's two emergency generators. During the Loma Prieta Earthquake, one of the generators "walked off of its foundation". The lack of adequate seismic anchorage of the generator rendered it unusable after the earthquake. Among the elements served by the unusable generator were life support systems. Fortunately during the time of the earthquake, no patients were dependent on life support systems. In less than an hour after the earthquake, in-house maintenance engineers were able to transfer demand from the unusable emergency generator to the operational one. Because each generator was sized to accommodate one hundred percent of the hospital's emergency power demand, the remaining functioning generator was capable of fulfilling all emergency power needs. Recovery efforts were greatly facilitated by the prompt response of in-house staff and by the reserve capacity provided in each generator.

Difficulties arose in refueling the functioning emergency generator due to a broken fuel supply line. The reserve fuel supply, a 5000 gallon tank below grade, required an auxiliary system to bypass the damaged supply line and fuel the generator. The hospital's portable pump was used to transfer fuel from the underground storage tank to a 55 gallon drum which in turn was transferred to the generator fuel tank. The fueling procedure was streamlined when a local corporation, Granite Construction, provided the hospital with a fuel truck.

Communications

The primary communication problem faced by the hospital related to the difficulty in placing outside calls. Because of the high demand placed on the phone system, outside calls were subject to lengthy delays. The hospital's in-house phone system suffered some minor damage which required reloading of their computer database. This operation was completed in less than an hour.

Water

A sprinkler line, which crosses between two structurally independent buildings without a flexible connection, ruptured during the earthquake. Several additional steam and water pipes also broke. Damaged portions of the piping systems were isolated by manually turning off flow valves. Approximately 80 percent of the hospital was without water for some period of time as a result of in-house damage. The city water supply was never lost entirely, but pressure was severely limited at times.

Portable toilets were delivered to the hospital the day after the earthquake as a precautionary measure. Gallo Winery provided the hospital with supplemental drinking water the day after the earthquake.

Sewage

Several sewer lines in the ground were broken during the earthquake. These breaks were not immediately identified, however. After one month, a tremendous increase in the number of flies was observed. This prompted a video camera survey of the sewer lines to assess the nature and scope of the problem. Where breaks occurred in underground piping within the hospital building, new overhead piping was installed to bypass the damaged pipes.

Additional

One leg on an upright liquid oxygen tank buckled during the earthquake. As a consequence, piped oxygen was unavailable for medical use. Available portable oxygen tanks were sufficient to meet all patient demands until repairs to the piped oxygen system could be made.

Watsonville Businesses

Many Watsonville businesses suffered substantial losses as a result of structural damage, nonstructural damage, and/or lifeline interruptions. The experiences of 5 businesses which suffered little or no structural damage are highlighted below to provide some insight into the impact of lifeline interruptions on business operations.

Based on interviews with 13 Watsonville companies, the continuity of lifelines was cited as a key to sustained business operations and efficient recovery efforts. Power was most commonly reported to be the most vital lifeline for restoring business operations. While power outages did slow recovery efforts in Watsonville substantially, the power interruptions alone did not appear to cause the majority of losses to the business community. This is in part because a large portion of the local industry consists of cold storage facilities which are somewhat insulated from power interruptions lasting only a few days. It takes on the order of a week or more to increase the temperatures of stored goods sufficiently to render them unfit for commercial distribution.

Del Mar

Del Mar is a medium-sized firm which packs and stores frozen foods. At the time of the earthquake, the peach harvest had recently concluded and their freezers were full of peaches. Power was lost at Del Mar for approximately 2 days. Although the firm has no emergency source of power, the freezers maintained a sufficiently low temperature that no fruit was lost. There was a small ammonia leak on the site which was handled by in-house maintenance crews. Total losses on the order of $150,000 were reported.

Company 2 (Prefers To Remain Anonymous)

Company 2 is a large firm which packs and stores frozen foods and has facilities at many locations including Watsonville. At the time of the earthquake, 40 million pounds of product were stored on site. Power was lost at Company 2 for approximately 40 hours. This interruption of power did not cause a significant rise in temperature of the stored goods and no inventory was lost as a result of a lack of power. The biggest problem experienced by this company was that about half of their stored products fell over. Recovery efforts consisted primarily of cleaning up the site, repackaging goods where possible and disposing of goods as needed. Miscellaneous repairs to sprinkler piping were also needed. Emergency generators were brought to the site to facilitate clean up operations.

Company 2 estimates their losses at approximately $1 million - approximately $100,000 in physical damage, approximately $450,000 in direct expenses incurred during the cleanup effort, and the remaining $450,000 in lost revenue resulting from the diversion of business during the company's one-month shutdown.

The Pillsbury/Green Giant Company

The Pillsbury/Green Giant Company was apparently most severely impacted of all cold-storage facilities in Watsonville. According to Ref. (2), moderate structural damage was suffered at the facility. The structural damage did not, however, place the building in danger of collapse nor lead to substantial operational interruptions. Substantial business interruptions resulted instead from damage to "nonstructural" components.

Cold storage vaults at the facility are cooled via ceiling-mounted cooling coils which operate with ammonia refrigerant. The ceiling-mounted coils were not braced laterally at the time of the earthquake and hence, swayed under the seismic excitations. The attached refrigerant tubing was not able to accommodate the relative movement of the coils and cracked or separated from the coils at several locations. The City of Watsonville Fire Department, EPA and Hazardous Materials Board were notified of the ammonia leak soon after the earthquake. The Watsonville Fire Department arrived on site and assisted in venting the cold storage room affected by the leak and conducting a survey to verify the extent of damage to the refrigerant tubing. Since the Loma Prieta Earthquake, the Pillsbury/Green Giant Company has undertaken measures to minimize future earthquake damage. Specifically, lateral bracing has been added to refrigerant lines and coils, and isolation doors were installed to reduce the size of the storage area that could be exposed to a single ammonia leak.

Undisclosed financial losses resulted from the need to discard the produce present in the 30,000 square foot cold storage area affected by the ammonia leak. The facility was able to resume partial production approximately two weeks after the earthquake and was in full operation one month after the earthquake.

Due to the nature of damage at the Pillsbury/Green Giant facility, the interruption of some lifelines serving the facility posed relatively minor inconveniences to recovery operations.

Monterey Savings

Banking operations at Monterey Savings, located in downtown Watsonville, were suspended for approximately three days. The loss of operations was directly dependent on the loss of commercial power at the facility. Once power was restored, the bank was fully operational.

Skyway Freight Systems

Skyway Freight Systems is a transportation company providing air freight, truck and transportation information services. Skyways corporate headquarters are located in Watsonville. Approximately 120 are employed at the Watsonville facility. Commercial power was restored to Skyway approximately one week after the earthquake. To accommodate operations prior to power restoration, an emergency generator was shipped from the company's Los Angeles facility. The generator was delivered one day after the earthquake. It was fueled with gasoline pumped from the company's trucks. The generator was used to supply power primarily for computer data entry.Phone service was fully restored at Skyway about a week after the earthquake. During the service interruption incoming calls were routed to Los Angeles and Chicago facilities.

Lingering Damage

There is evidence from previous U.S. earthquakes to suggest that seismic ground shaking accelerates the normal process of aging in underground pipelines (Ref. 7). Ground shaking loosens metal already weakened by corrosion and disturbs caulking already distressed by settlement and traffic vibrations. The possibility that this will happen in Watsonville, as happened in Coalinga after the 1983 earthquake, is a matter of concern because the extra cost of above normal maintenance is an ongoing, earthquake-related cost that may not be compensated by outside sources. The City of Watsonville is currently examining the condition of its sanitary sewer system with this consideration in mind.

Conclusions: Interaction Among Lifeline Components

The experience of Watsonville in the Loma Prieta Earthquake shows that the loss of electric power degrades emergency response. In Watsonville, the availability of electric generators enabled the City to make best use of its resources. This is most apparent in the water supply system, where well pumps operated by emergency generators were able to meet the water demand everywhere except in the downtown area, which was deliberately isolated to prevent multiple breaks from degrading the rest of the system. Portable generators also were used by a private company to operate pumps at lift stations in the sanitary sewer system. The inability to use an emergency generator in the town Emergency Communication Center made it necessary to use radios in vehicles, which was unsatisfactory due to limited bandwidth. It was difficult to pump gasoline from below-ground tanks into vehicles for transport to emergency generators. Among the few things the City would do differently next time is to have more portable electric generators and more convenient storage of fuel to operate them.

Future Directions For Research, Applications And Design

A seismic risk study to consider interaction between lifelines, analogous to the one recently completed by the Water and Sewage and Seismic Risk Committees of the ASCE Technical Council on Lifeline Earthquake Engineering, should be developed. The goal should be to examine the effects caused by failure of electric power on other lifelines, especially water and communications. The first stage of the study should explore qualitative historical evidence of impacts, such as failure of pump stations, of electric power failure. Research is needed to develop a theoretical and computational framework to accommodate the uncertainties of two lifeline systems and the connections between them.

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STRATEGIES FOR REPAIR AND RESTORATION OF SEISMICALLY DAMAGED GAS PIPELINE SYSTEMS

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INTRODUCTION

The purpose of this study is to develop a strategy for the repair of seismically damaged gas networks. Based on the concept taken from a study by Isoyama et al. (1985), a series of programs have been coded and a prototype computer systems developed. An important characteristic of the computer systems is the implementation of GIS (Geographic Information System). GIS is a computer tool for acquisition, manipulation and retrieval of spatial data. In this study, a commercial GIS package, ARC/INFO, available from Environmental Systems Research Institute (ESRI), California, was used.

GAS PIPELINES

A city gas network usually consists of two types of networks; high or medium pressure lines and low pressure lines. Figure 1 shows a model of the medium and low pressure lines. As shown in the figure, the medium pressure network includes two types of nodes; sources and regulators. Source nodes which include gas plants, send out medium pressure gas to the network. Regulators on the medium pressure pipes from which the low pressure pipelines originate control gas supply to the low pressure lines.

RESTORATION OF SEISMICALLY DAMAGED NETWORK

Restoration of a seismically damaged gas network involves two major steps. One is the restoration of medium pressure lines, and the other is the restoration of low pressure ones. Before starting the restoration of low pressure lines, however, medium pressure lines must be completely restored. In the study by Isoyama et al, the restoration work of medium pressure lines is assumed to be performed pipeline by pipeline, while low pressure lines are assumed to be restored small district by district. This study primarily deals with the restoration of medium pressure lines.

Figure 2 shows a flow chart that describes the procedures for determining the priority of repair. A brief description is given below.

ARC/INFO Data Base

A hypothetical network on which the example analysis will be performed is created in the form of a digital map or a so-called "coverage", according to the ARC/INFO terminology (ESRI, 1989). Figure 3 shows the network coverage used in this study, which consists of 30 links and 27 nodes. Among the 27 nodes, two are source nodes, 13 are regulators and 12 are junctions. Each feature has attributes which are necessary for the determination of repairing priority. These attributes are stored in the relational database, and can be retrieved as properties of features. The information of the coverage is converted into text files, and can be processed by other computer programs.

Link Data

Table 1 indicates the attributes of the links such as link ID number under the columns identified by the numbers 06 through 10 (see the first two rows), starting and ending nodes indicated by the numbers 11 through 15 and 16 through 20, respectively.

Node Data

In Table 2, the node attributes are shown in a similar fashion to Table 1.

Pipe Breaks in Medium Pressure (MP) Lines

Pipe breaks in MP lines are simulated using a Monte Carlo Simulation technique. First, the occurrence rate of pipe breaks (per unit length) is calculated as follows; For a given Modified Mercalli Intensity (MMI) = I which is assumed to be common over the entire area where the network is located.

$$r_j = C_1 \bullet 10^{C_2(I-C_3)} \tag{1}$$

where r_j is the occurrence rate for pipe j and C_1 , C_2 , and C_3 are constants. These constants are assumed to take values listed in Table 3, based on the pipe elements (Eguchi, 1984).

Second, assuming that the pipe breaks follow the Poisson law, the probability P_{jk} of exactly k breaks occurring in pipe j, is computed as,

$$P_{jk} = \frac{(r_j \bullet L_j)^k}{k\,!} \exp\,(-r_j \bullet L_j) \tag{2}$$

where, L_j is the pipe length (km).

Third, applying the standard Monte Carlo technique, the number of breaks in each pipe will be simulated. In this Monte Carlo simulation, a random number x uniformly distributed between 0 and 1 is generated and the number N of breaks is assumed to have occurred if $\sum_{k=o}^{N-1} P_{jk} < x \leq \sum_{k=o}^{N} P_{jk}$.

Pipe Breaks in Low Pressure (LP) Lines

The number of breaks in LP lines connected to regulator i is calculated as follows,

$$Dn_i = r \bullet Lt_i \tag{3}$$

where Lt_i is the total pipe length of LP lines, and r is the occurrence rate calculated using Eq. (1). The element type 4 (see Table 3) is assumed for all LP lines, which are assumed to be made of cast iron.

Link Distance

Link distance Ld_j of pipe j to be used in the shortest path analysis which will follow, is expressed as a function of the flow rate Qd_j in pipe j, and of the factor which quantifies the level of difficulty in repairing each break, referred to as the difficulty factor for repair t_j. It is defined as,

$$Ld_j = p/Qd_j + q \bullet t_j \tag{4}$$

where p and q are coefficients to which the following values are assigned for numerical purposes.

$$p = 10000, \quad q = 100 .$$

The gas flow rate Qd_j in the pipe is a function of the diameter d and length L of the pipe as shown below.

$$Qd_j = \gamma \sqrt{\frac{d_j^5}{L_j}} \tag{5}$$

where γ is a constant. The difficulty factor of repair is calculated as,

$$t_j = a \bullet L_j + b \bullet Dl_j \tag{6}$$

where Dl_j is the number of breaks in pipe j, "a" represents the efficiency of inspection (day/km) and "b" represents the efficiency of repair (day/break).

Node Weight

Node weight Nw_i represents the importance and necessity of restoring the function at node i. It is defined as

$$Nw_i = \beta_i \bullet \frac{Nc_i}{Dn_i} \tag{7}$$

where Nc_i is the number of customers serviced by the regulator, Dn_i is the number of breakage in LP lines connected to the node, and β_i is a parameter which represents the importance of the node.

Shortest Path

In this study, the restoration between either one of the sources and a node proceeds along the shortest route between them to be determined based on link distances described above.

Link Weight

Link weight is determined by adding all the node weights associated with the shortest routes which include the link under consideration. For example, link 1 in Fig. 4 is shared by two shortest routes to node 3 and node 2, and hence the link weight is Nw3 + Nw2 .

Priority of Repair

For each seismic intensity, steps (4) - (9) as indicated in Fig. 2 are performed repeatedly for all the trials of the simulation. After all trials are carried out, the resulting link weights are averaged and the repair priority is determined based on the averaged link weight. Figure 5 shows the histograms of link weight under seismic intensities VIII and IX. As shown in these figures, links which are closer to source nodes have higher priority as expected. Also, the priority order does not show any significant change as MMI scale increases.

In this study, seismic intensity was assumed to distribute identically over the entire network. Since the service area of a gas network is generally wide-spread, the spatial distribution of the ground motion intensity should be considered in future studies. Furthermore, the method discussed here should be improved and modified for more reasonable modeling of the restoration work on the network. These improvements should be implemented with consultation with repair and restoration experts with gas utilities.

ACKNOWLEDGEMENT

This work was partially supported by Project # NCEER-903005 under NSF Grant ECE-8607591. The support by Tokyo Gas Co., Ltd. and Pasco Corporation is also acknowledged.

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Isoyama, R., Iwata, T. and Watanabe, T., "Optimization of Post-Earthquake Restoration of City Gas Systems", Proceedings of the Trilateral Seminar-Workshop on Lifeline Earthquake Engineering, National Taipei University, Taipei, Taiwan, 1985.

Iwata, T., "Restoration Planning System for Earthquake Damaged Gas Pipeline Network", Proceedings of the Trilateral Seminar-Workshop on Lifeline Earthquake Engineering, National Taipei University, Taipei, Taiwan, 1985.

TABLE 1 Link Data Created from a Coverage

```
    000000000111111111122222222223333333333444444444455555555556666666666777777
    123456789012345678901234567890123456789012345678901234567890123456789012345
 1  LINK   30
 2      1    1   21   19     6.488     76.20    2  10.00000    0.0600    0.0400
 3      2    2   26   25     1.315     76.20    2  10.00000    0.0600    0.0400
 4      3    3   11    3     4.158     40.64    1   3.00000    0.0048    0.0060
 5      4    4    2    1     1.917     76.20    2  10.00000    0.0600    0.0400
 6      5    5    1    7     4.564     76.20    2  10.00000    0.0600    0.0400
 7      6    6    7   10     1.761     76.20    2  10.00000    0.0600    0.0400
 8      7    7    9   21     5.041     76.20    2  10.00000    0.0600    0.0400
 9      8    8    6    9     1.415     76.20    1  10.00000    0.0300    0.0200
10      9    9   19   27     8.202     76.20    2  10.00000    0.0600    0.0400
11     10   10   16   11     4.035     76.20    2  10.00000    0.0600    0.0400
12     11   11   25   24     5.512     76.20    2  10.00000    0.0600    0.0400
13     12   12   23   16     4.678     76.20    2  10.00000    0.0600    0.0400
14     13   13   10    8     2.245     76.20    2  10.00000    0.0600    0.0400
15     14   14    9    8     2.856     76.20    2  10.00000    0.0600    0.0400
16     15   15   23   24     6.565     76.20    2  10.00000    0.0600    0.0400
17     16   16   26   27     3.859     76.20    2  10.00000    0.0600    0.0400
18     17   17    8   14     2.602     40.64    4   3.00000    0.0192    0.0240
19     18   18    2    4     2.537     40.64    4   3.00000    0.0192    0.0240
20     19   19   26   22     2.739     40.64    4   3.00000    0.0192    0.0240
21     20   20   11    5     4.046     76.20    2  10.00000    0.0600    0.0400
22     21   21    5    2     6.169     76.20    2  10.00000    0.0600    0.0400
23     22   22   18   24     6.865     50.80    3   3.00000    0.0180    0.0180
24     23   23   18   13     2.722     50.80    3   3.00000    0.0180    0.0180
25     24   24   13    5     3.700     50.80    3   3.00000    0.0180    0.0180
26     25   25   13   17     1.985     40.64    4   3.00000    0.0192    0.0240
27     26   26    7   12     3.066     50.80    3   3.00000    0.0180    0.0180
28     27   27   12   18     3.215     50.80    3   3.00000    0.0180    0.0180
29     28   28   12   15     2.349     50.80    3   3.00000    0.0180    0.0180
30     29   29   15   19     2.383     50.80    3   3.00000    0.0180    0.0180
31     30   30   15   20     1.554     40.64    4   3.00000    0.0192    0.0240
```

Record 1

1- 5	A5	LINK
6-10	I5	The number of links

Record 2-

1-5	I5	Sequential record number
6-10	I5	Link ID number
11-15	I5	Startinf node ID
16-20	I5	Ending node ID
21-30	F10.3	Link length (km)
31-40	F10.2	Pipe diameter (cm)
41-45	I5	Pipe element
46-55	F10.5	γ (in the flow formula)
56-65	F10.4	Efficiency of inspection (day/km)
66-75	F10.4	Efficiency of repair (day/break)

TABLE 2 Node Data Created from the Coverage

```
     0000000001111111111222222222233333333334444444444
     1234567890123456789012345678901234567890123456789
1    NODE    27
2        1    1    1   4400  1.00     17.000
3        2    2    0      0  1.00      0.000
4        3    3    2      0  1.00      0.000
5        4    4    1   2200  1.00     12.000
6        5    5    0      0  1.00      0.000
7        6    6    2      0  1.00      0.000
8        7    7    0      0  1.00      0.000
9        8    8    0      0  1.00      0.000
10       9    9    0      0  1.00      0.000
11      10   10    1   3000  1.00     21.000
12      11   11    0      0  1.00      0.000
13      12   12    0      0  1.00      0.000
14      13   13    0      0  1.00      0.000
15      14   14    1   2600  1.00     14.000
16      15   15    0      0  1.00      0.000
17      16   16    1   3800  1.00     19.000
18      17   17    1   2400  1.00     11.000
19      18   18    1   4200  1.00     20.000
20      19   19    0      0  1.00      0.000
21      20   20    1   2000  1.00     13.000
22      21   21    1   3200  1.00     17.000
23      22   22    1   2800  1.00     10.000
24      23   23    1   4000  1.00     18.000
25      24   24    0      0  1.00      0.000
26      25   25    1   3600  1.00     15.000
27      26   26    0      0  1.00      0.000
28      27   27    1   3400  1.00     16.000
```

Record 1

1- 5	A5	NODE
6-10	I5	The number of nodes

Record 2-

1- 5	I5	Sequential record number
6-10	I5	Node ID number
11-15	I5	Node type 0: Junction 1: Regulator 2: Source
16-22	I7	The number of customer
23-28	F6.2	Social Importance
29-38	F10.3	Total length of LP lines connected to the regulator (km)

TABLE 3 The Occurrence Rate of Pipe Failure

$$r_j = C_1 \cdot 10^{C_2 (I - C_3)}$$

r_j : The occurrence rate of pipe failure
I : MMI scale

PIPE ELEMENT	CONSTANTS C_1	C_2	C_3
1	0.00937	0.33800	10.2
2	0.06560	0.33800	10.2
3	3.28000	0.33800	10.2
4	3.28000	0.21300	13.6

* PIPE ELEMENT
1: Welded Steel pipes with Arc Welded Joints (WSAWJ) - X-Grade
2: Welded Steel pipes with Arc Welded Joints (WSAWJ) - Pre-Modern
3: Welded Steel pipes with Gas Welded Joints (WSGWJ)
4: Cast Iron and Others

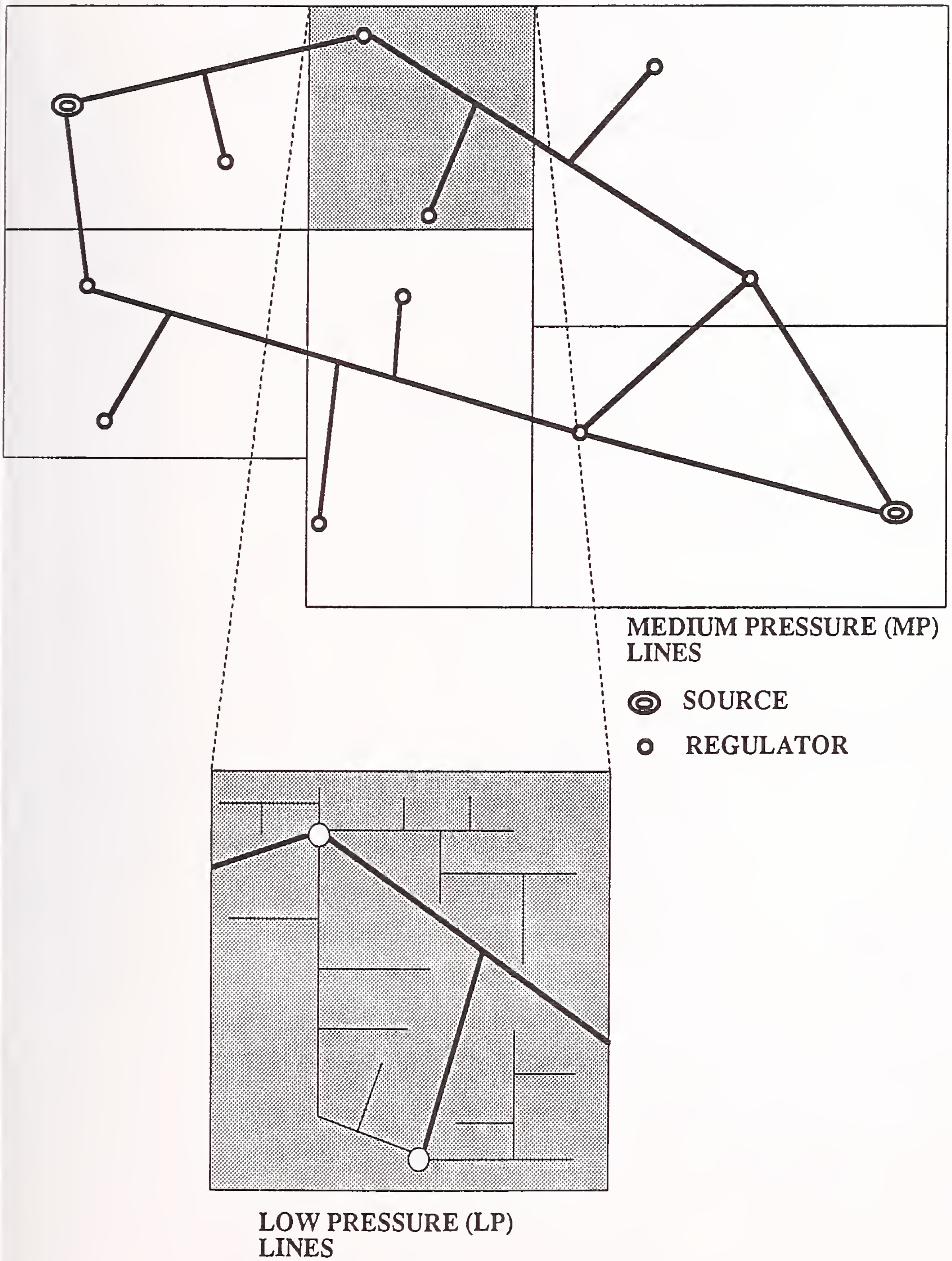


FIGURE 1 The Medium Pressure (MP) Lines and the Low Pressure (LP) Lines

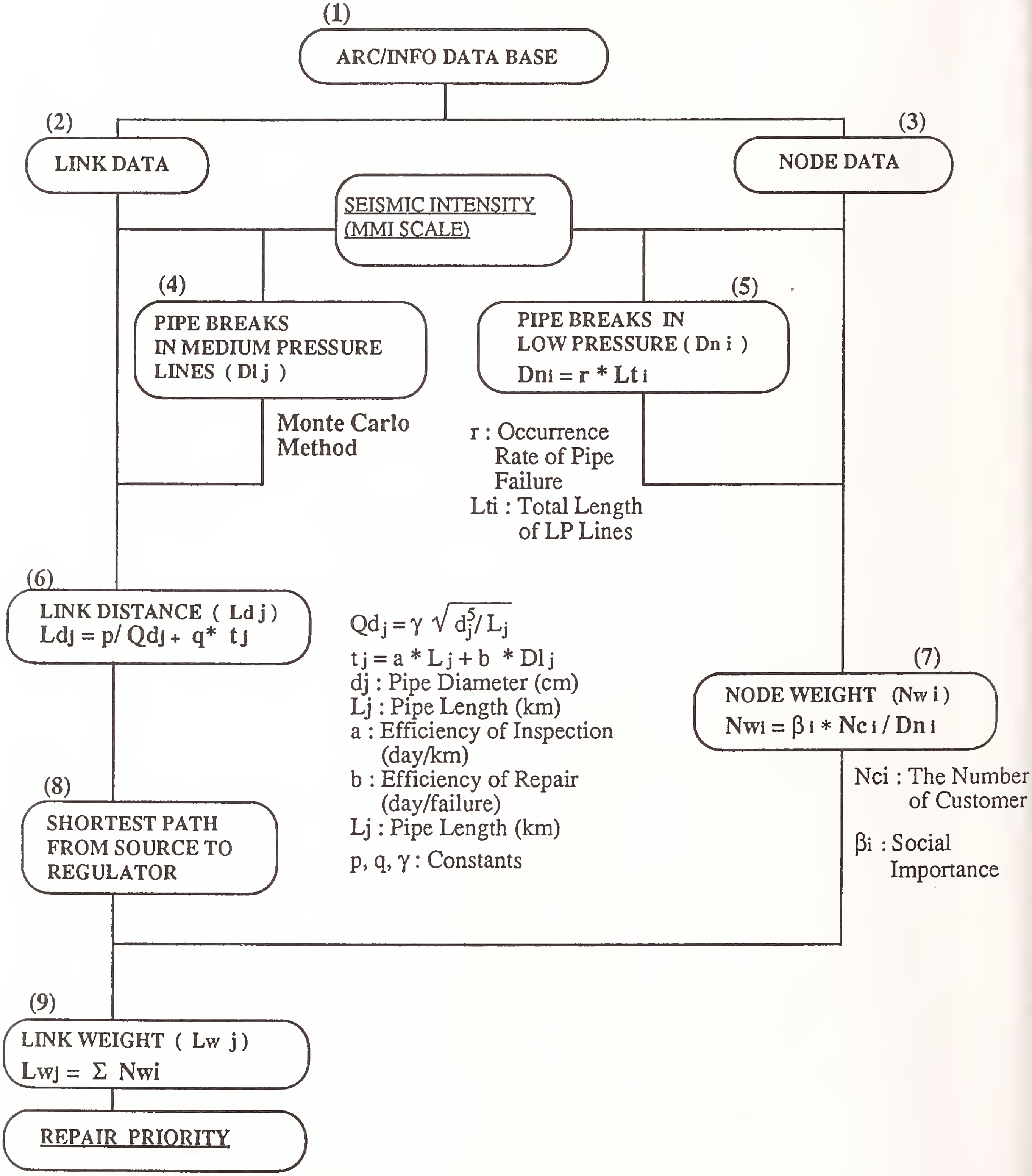


FIGURE 2 GENERAL FLOW OF
DETERMINATION OF REPAIR PRIORITY OF LINKS

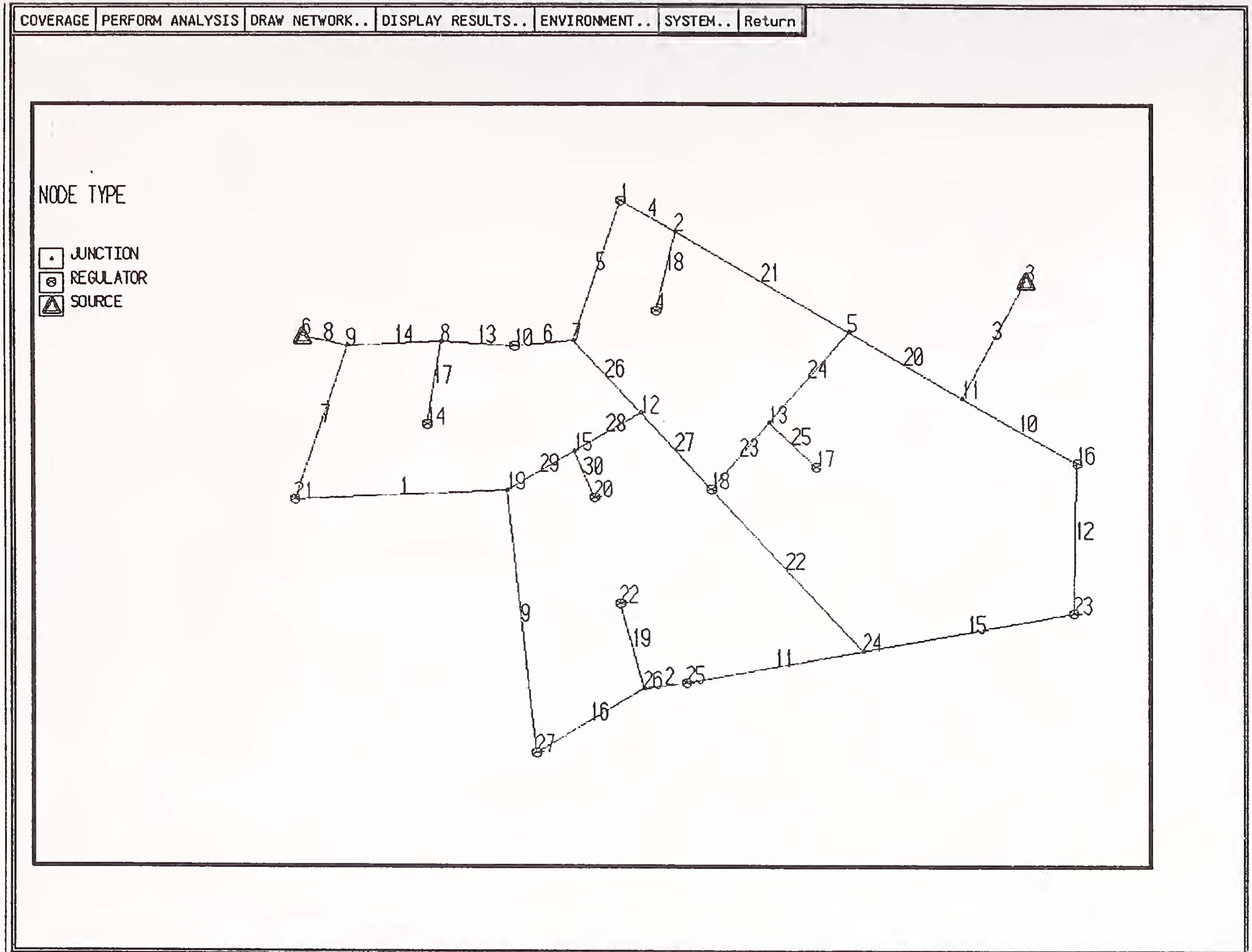
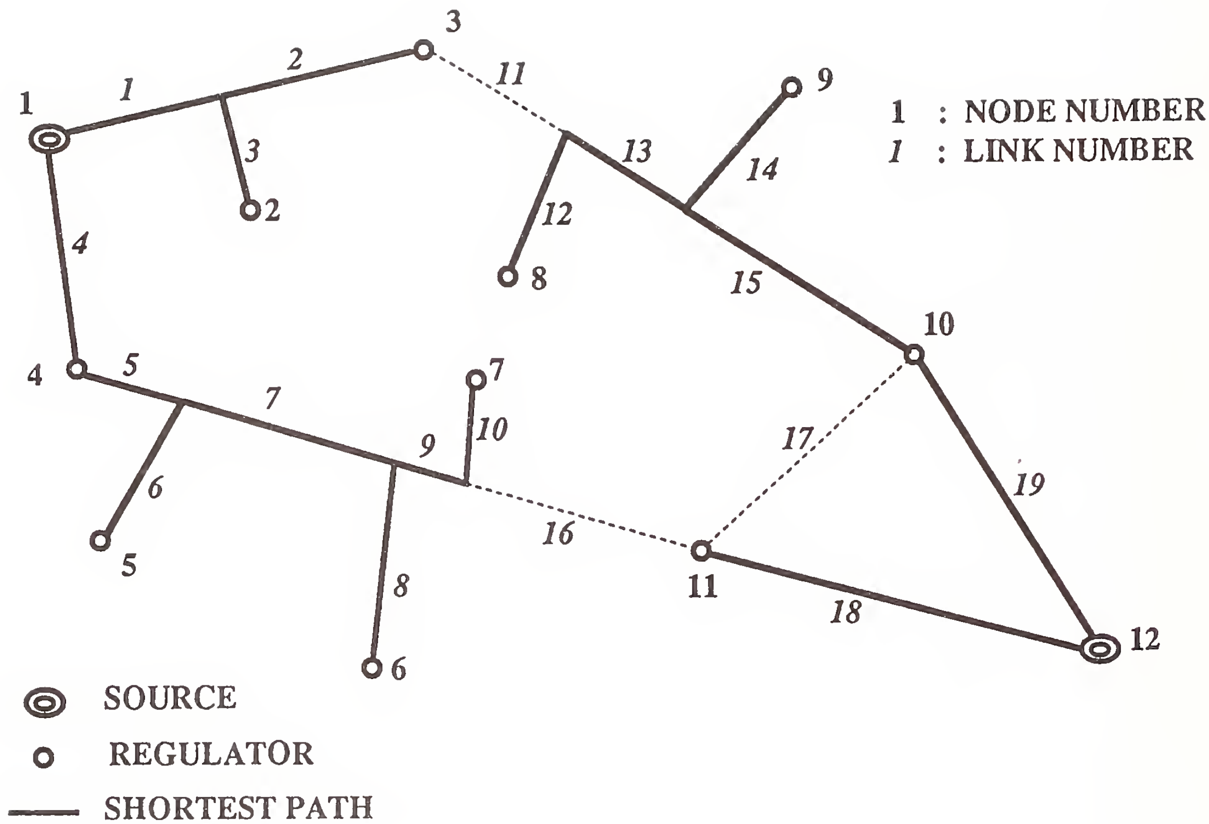
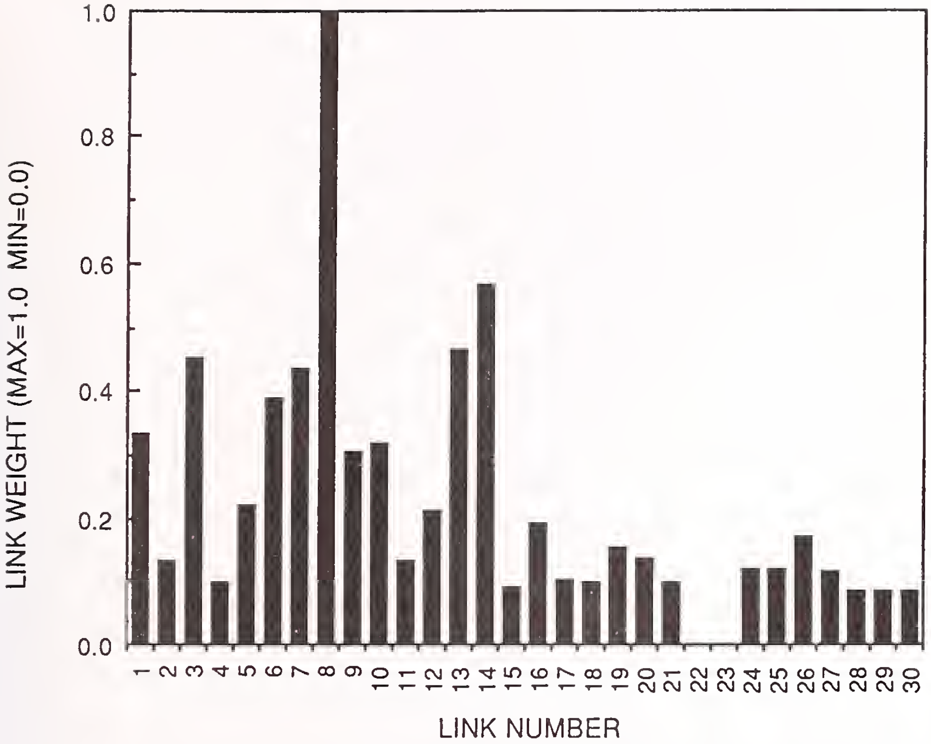


FIGURE 3 The Sample Network Created for Analyses

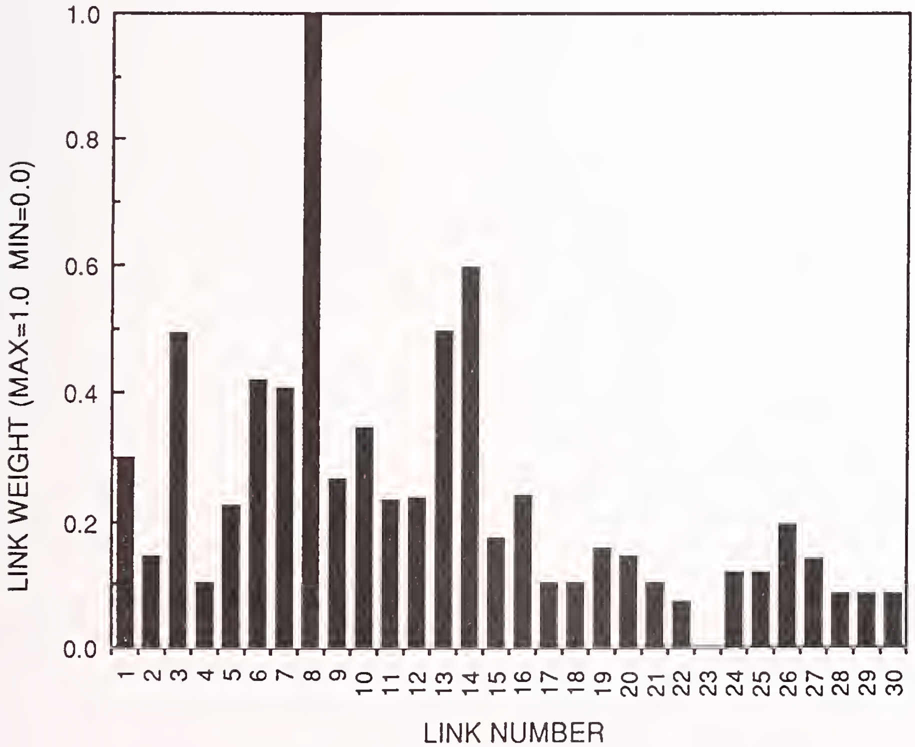


LINK NUMBER	LINK WEIGHT
1	Nw3 + Nw2
2	Nw3
3	Nw2
4	Nw7 + Nw6 + Nw5 + Nw4
5	Nw7 + Nw6 + Nw5
6	Nw5
7	Nw7 + Nw6
8	Nw6
9	Nw7
10	Nw7
11	UNDEFINED
12	Nw8
13	Nw8
14	Nw9
15	Nw8 + Nw9
16	UNDEFINED
17	UNDEFINED
18	Nw11
19	Nw8 + Nw9 + Nw10

FIGURE 4 An Example of the Definition of Link Weight



LINK WEIGHT : MMI SCALE = VIII



LINK WEIGHT : MMI SCALE = IX

FIGURE 5 Averaged Link Weight

VII. SOCIOECONOMIC IMPACTS

"Lifeline Earthquake Hazard Zonation in
Socioeconomic Aspects"
E. Kuribayashi

"Organizational Features of U.S. Lifeline Systems
And their Relevance for Disaster Management"
K.J. Tierney

"A Methodology for Assessing the Risk of Hazardous
Materials Release Following Earthquakes -
A Demonstration Study for the Los Angeles Area"
H.A. Seligson, R.T. Eguchi, K.J. Tierney

LIFELINE EARTHQUAKE HAZARD ZONATION IN SOCIO-ECONOMIC ASPECTS

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ABSTRACT

Within the context of earthquakes, urban lifelines are defined as transportation, water, gas, sewer, power, telecommunications and computer networks. Experiences from recent earthquake disasters in the North American continent seismic zones, Japan and the Mediterranean have provided lessons on measures to be taken for the protection of urban lifelines.

Concerning the aspect of zonation, these events are categorized in three types of recovery of functions in lifeline systems:
1) EMERGENCY repairs, 2) ALTERNATIVE measures and 3) SUSPENSIVE functions.

G. Suzuki
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Toyohashi University of Technology, Toyohashi, Japan

INTRODUCTION [1][2]

"Urban lifeline systems," or "lifelines," includes the following four kinds of networks:

1).Transportation systems through land, sea, and air: moving people and goods for multiple purposes, in many directions by road, railway, waterway and airway.

2).Water supply and sewer treatment systems: reserving, pumping, disposing, and letting natural flow occur through waterworks, sewerage, and urban rivers.

3).Energy supply systems: continually supplying power sources, luminous sources and heat sources such as electricity, city gas and other fuels, steam, and cool air.

4).Communication and computer network systems: exchanging, transmitting and broadcasting information by telephone, telegram, mail, newspaper, radio, and T.V., and readying money and money orders by, for example, cash dispensers.[3]

These systems show the nature of the public, and are not only indispensable in presenting one state or city, but also have a close relationship with daily lives.

These systems also consist of two phases, the equipment itself and its operation. Each system is rationally intertwined with the others. These four systems are also implemented according to their necessity. The more they are implemented, the more convenient daily living becomes.

However, what circumstances would result if a part of the system were damaged because of a disaster or warfare? Is it possible, especially, in the case of an earthquake, to prevent this ?

Dealing with these problems, there is the question if it is possible to get some resolutions from the inductional or analytical method. If it's not, what are the other ways? Can we find any effective countermeasures by using the deductional methodwhich stands with the common sense of prevention of disasters?

INCEPTION

The concept of "lifeline systems" was originally discussed by Prof. C.M. Duke [2][4], because of the disaster brought by the San Fernando earthquake with a magnitude of 6.4 on the Richter scale which struck the northern part of the city of Los Angeles on February 9, 1971,[4]. In the past, the concept had been only the group of facilities consisting of points and lines, but then Prof. Duke provided the new concept including their functions.

EXAMPLES OF THE SAN FERNANDO EARTHQUAKE OF 1971 [4]

In this earthquake, Interstate Freeway 5 named the "Golden State Highway", which is located on the coast of California, was greatly damaged at the interchange with the road to the inland areas of northern Los Angeles, San Fernando and Palm Dale. It caused a large interference with traffic on the north-south and coast-inland links. Fortunately, the national highway 1, which runs parallel to the Interstate 5, had no serious damage so the traffic was not held up.

The Van Norman dam had also suffered damage and it caused problems with the water intake from San Fernando reservoir which is used as one of the main water reservoirs for the city of Los Angeles. It was also fortunate that other water supplies were available such as the waterway from the northern part of California and other reservoirs.

Furthermore, one of the large electrical power transformer stations broke down, and the transmission stopped for several days in San Fernando. Damage to telephone system itself was not significant, but the excessive load of calls paralyzed its transmission function. There were some hindrances to the gas service. Pacific Gas and Electric Company received several times more complaints than usual. However, the earthquake was relatively small (M=6.7) and fortunately its epicenter was located in a mountainous region in non-residential area, thus causing less chaos.

EXAMPLES IN JAPAN [5]

At the Miyagi-ken Oki earthquake with a magnitude of 7.4 on June 16, 1978, at one of national expressways, Tohoku-do, the extension of traffic had been regulated through rampways attached along the expressway which flowed into an area within Miyagi prefecture. There consequently was no damage to the expressway, but the traffic was restricted by the provisions of a regulation that, in such a case, presumed that the facility was forced with abnormal external forces, exceeding certain prescriptions. The supervisor of the expressway had to close the expressway until the whole line could be inspected visually, and safety confirmation could be made. On the other hand, a strange phenomenon was observed where regular roads had to be opened for emergency transit even if they were possibly damaged by the earthquake. One part of a local road in the city of Sendai took 267 days to restore, of course the road itself, and moreover the buried facilities. When a road has some additional services such as aerial lines for electricity and/or telephone and underground piping, other than the normal function of "traffic services", its recovery requires a longer period of time. But some municipal governments have set such a system of priorities, similar to tose in the United States, for buried facilities on private property along the roadsides. These "Right of Way" regulations greatly contribute to shortening the term of restoration.

On the other hand, it was impossible to ward off the affluence of undisposed sewage and sludge into the river and sea, no matter how well it was regulated right after the disaster. The disposal system was not part of the same process of restoration as the water supply system. The water supply system should require an especially early recovery since potable water is a necessity to human life. As for the sewage system, it also had the problem that rain water mingling with it. Consideration will be needed for improvement of maintenance on the facilities and operation of the systems to avoid contaminating the dark water in rivers, lakes and seas in view of long term pollution.

At this earthquake, telephone congestion had occurred, but it calmed down within 24 hours, at the evening of the next day. This phenomenon would appear in any country where telephone systems are in common use, and it usually would calm down within a day.

PECULIAR EXAMPLES [5]

Since the stricken area was a rural area, it was not a large-scale disaster by the Southern Italy earthquake with a magnitude of 6.9 on November 23, 1980. However, there was unusual type of trouble that occurred in a place 100km away from the epicenter. There were many relief parties, private organizations sent from the north, that triggered a traffic jam at the interchange of Expressway 2, in a suburb of Naples. It took many hours to tranquilize the traffic with systematic traffic control. This is an example of indirect chaos caused by an earthquake.

On the contrary, the El Asnam earthquake with a magnitude of 7.3 on October 10, 1980, destroyed a city in the northern part of Algeria. It destroyed the whole city of El Asnam which was established by the former suzerain state France, including its disposal system, power supply, city gas and telephone lines, sewer treatment plants, storage facilities, residential houses, store, school, market, and governmental buildings. Damage was so extensive that the whole city had to be abandoned.

EARTHQUAKE RESISTANCE IN LIFELINE FACILITIES [6]

Distribution of property loss ratios, converted from loss of various kinds of facilities in terms of money value induced from material damage caused by earthquakes occurring in Japan during a 13 year period from 1962 to 1975, is shown in Fig. 1.

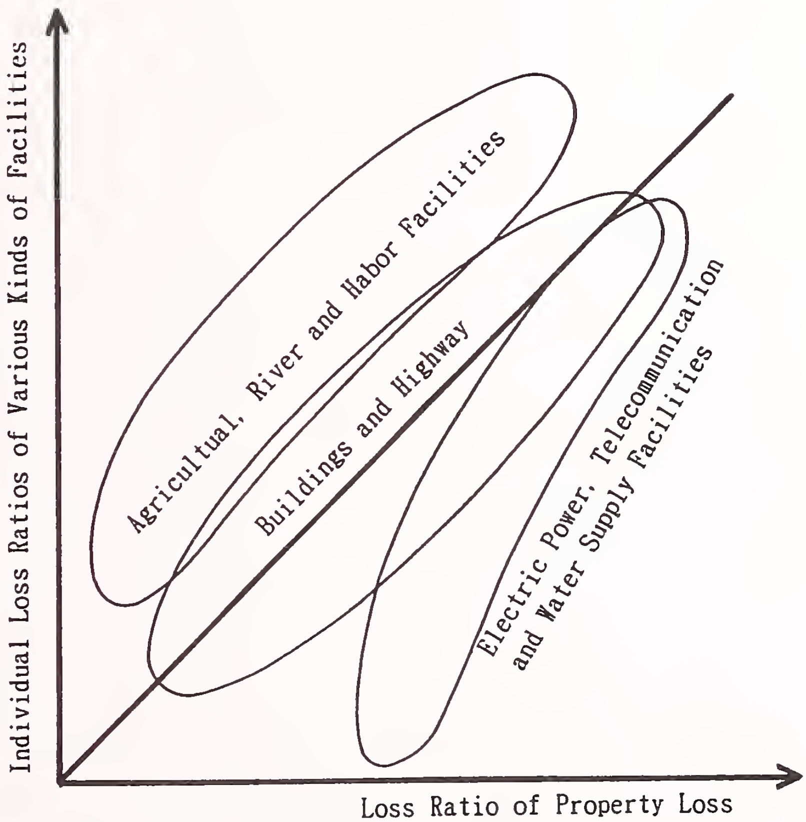


FIGURE 1. Characteristics of individual Loss Ratios of Various Kinds of Facilities [6]

The loss ratios of building and highway facilities are almost of the same degree, and also close to the loss ratio of the total loss. In contrast to this, the loss ratios of agriculture, flood control, and river and harbor facilities have a greater value than the loss ratio of the total loss, but those of electrical power, telecommunication and water supply facilities are less. Distribution of the individual loss ratio for highways is shown in Fig. 2. It includes six earthquakes from the Miyagi-ken Hokubu earthquake in 1962 to the Ohita-ken Chubu earthquake in 1975.

It is slightly meaningless to compare the individual loss ratios in different earthquakes since the damaged area depends on the administrative district, but it is significant to compare the individual with the total loss ratio. For this reason, Fig.3 shows the details of the distribution of the individual loss ratio for buildings.

When the loss ratios for buildings and highways are considered, the tendency of the loss of these two facilities used by humankind since ancient time is almost the same as the total loss. In contrast to this, the loss ratio for the facilities which only have a hundred-year or less history, but have high productivity, including such modern facilities as electrical power, telecommunications and water supply, are lower than that of the total. The greater value of the loss ratio, therefore, makes larger earthquake disaster loss ratios closer to that of the total loss.

On the other hand, the individual loss ratio for harbor facilities is larger than that of the total. Due to the location of the harbor facilities, one should understand from this that the external forces that mainly have to be considered are the waves and the tides. The individual loss ratio will be one of the criteria in estimating the earthquake resistance of lifeline facilities. It is probable that these facilities have a lower loss ratio and surpass others in earthquake resistance.

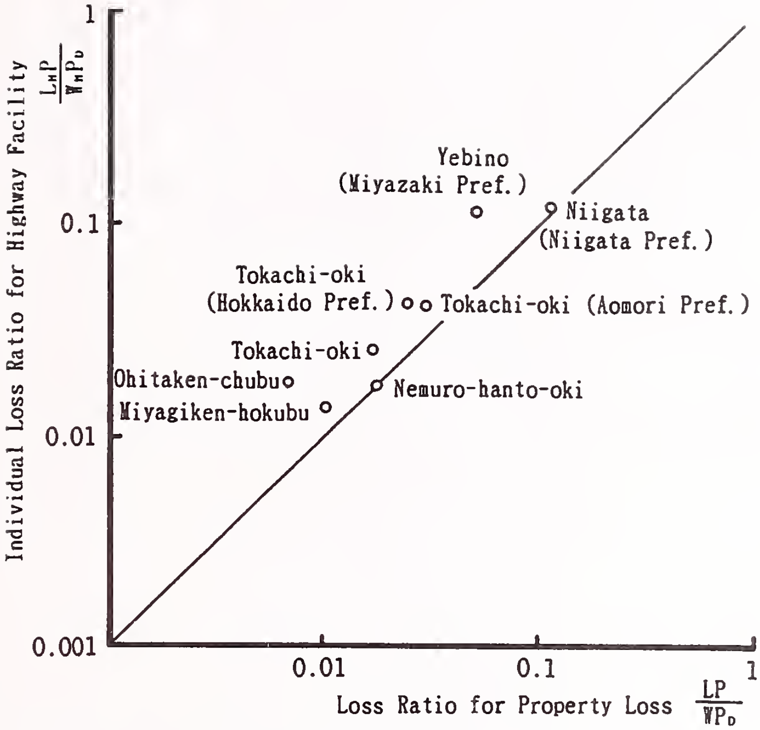


FIGURE 2. Relationships between Loss Ratios of Property Loss and Highway Facility [6]

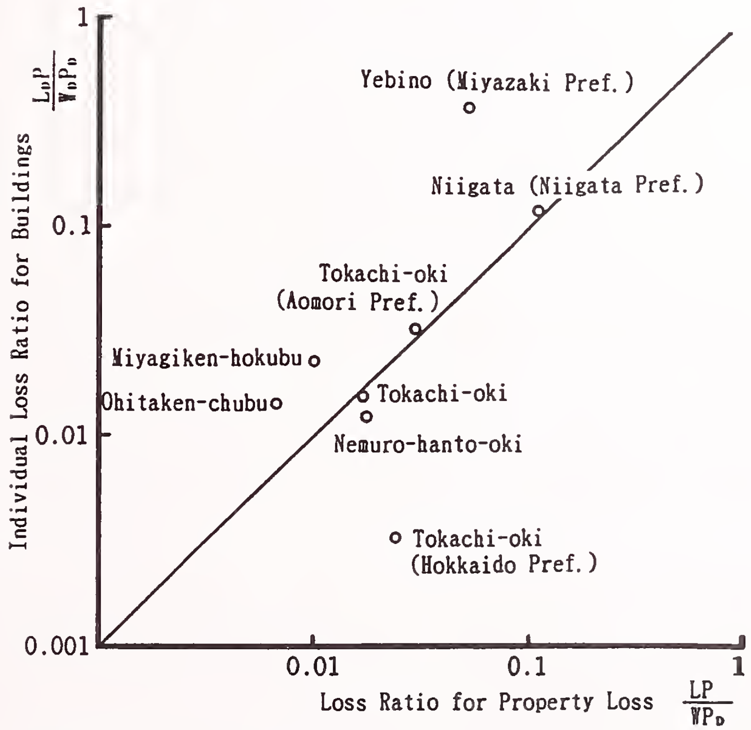


FIGURE 3. Relationships between Loss Ratios of Property Loss and Buildings [6]

FUNCTIONAL LOSS OF LIFELINE SYSTEMS [5]

The recovery of major lifeline systems at the Miyagi-ken Oki earthquake in 1978 is shown in Fig. 4. It took a relatively long period ofd time to get them back on line, especially roads, railway, and city gas. However, having had alternatives, the roads did not cause major difficulties for transportation.

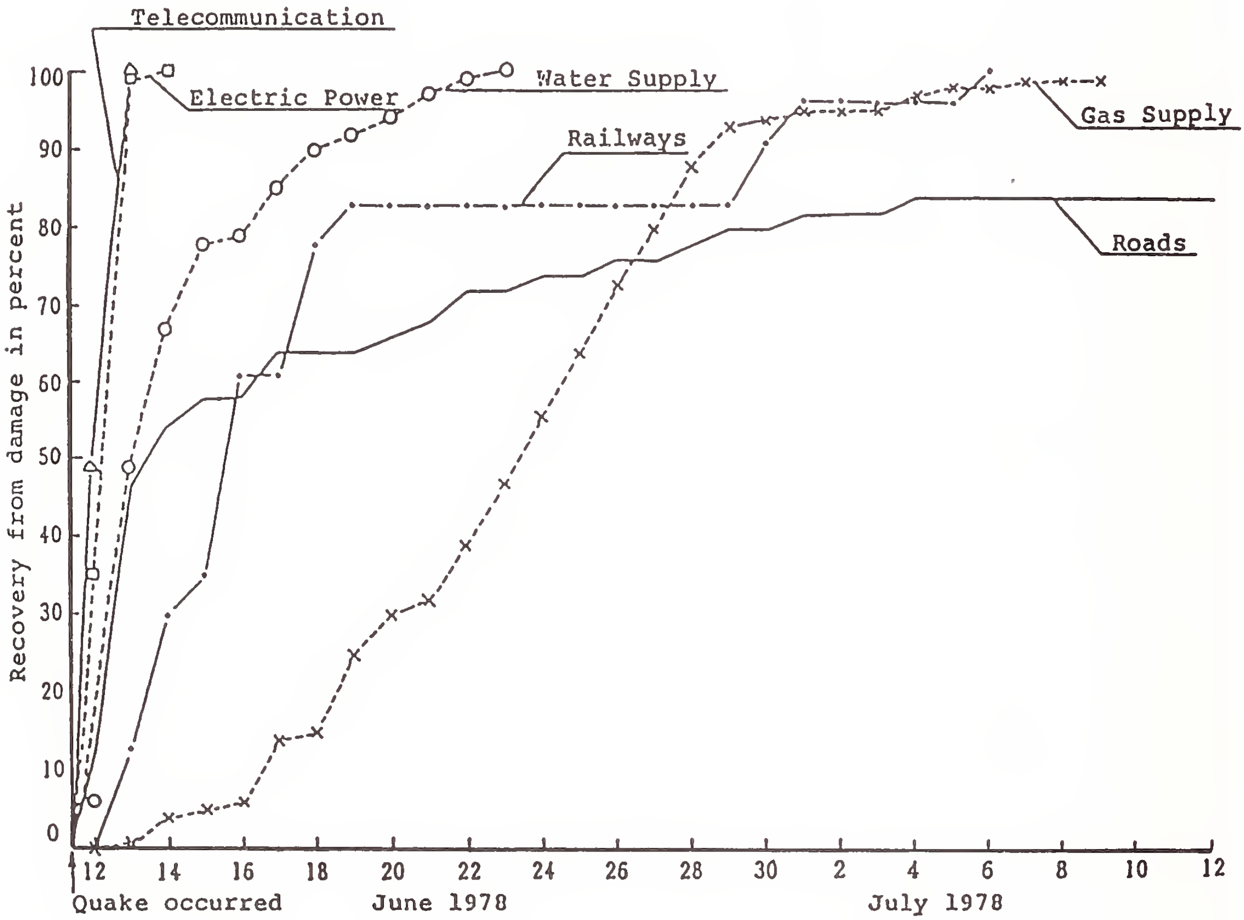
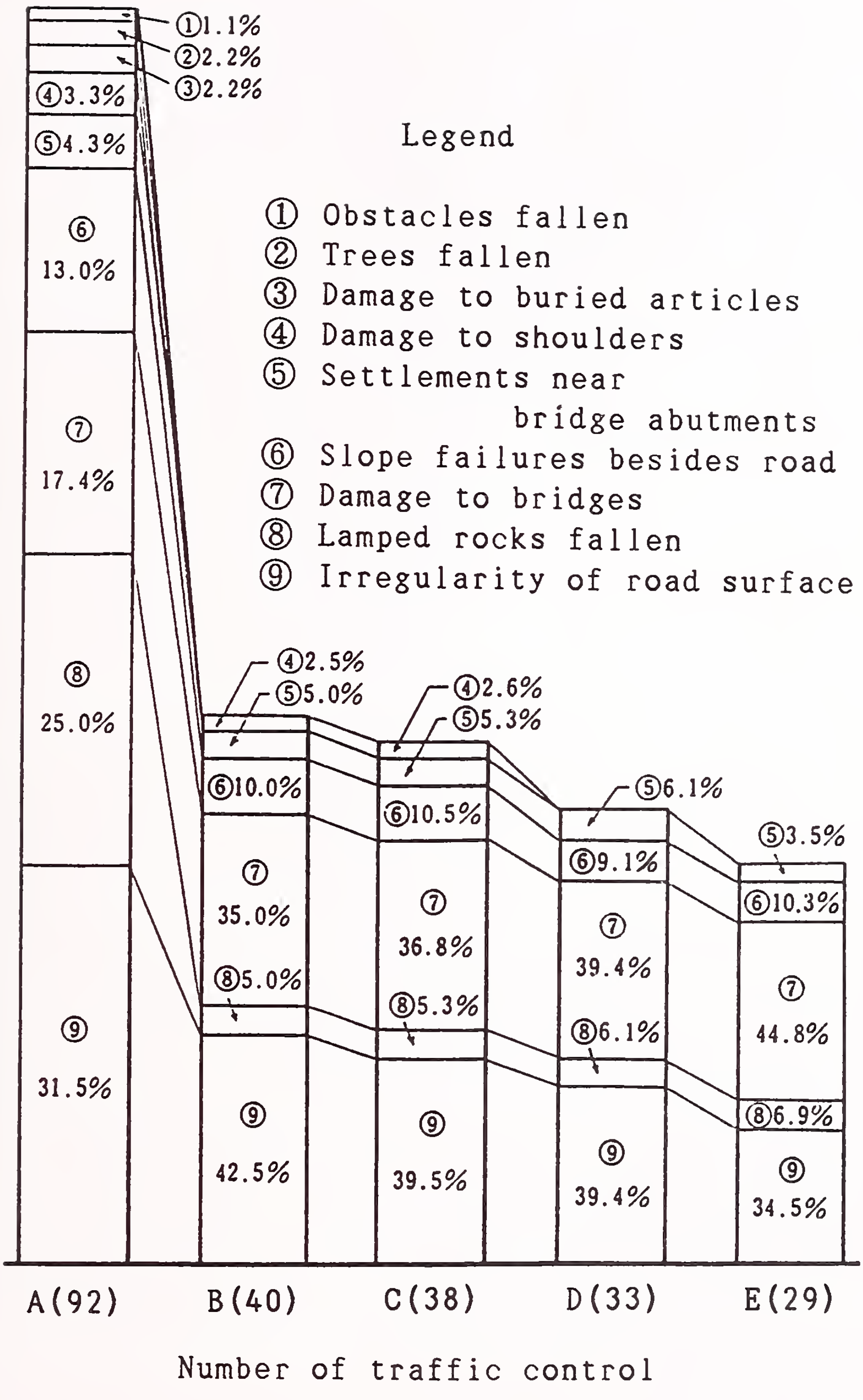


FIGURE 4. Resumption of Lifelines in the Miyagi-ken oki Earthquake of June 12, 1987 (after PWRI)

What are the methods for making recoveries quick? Fig. 5 shows an example from the Miyagi-ken Oki earthquake. It is obvious that the damage to key facilities like bridges takes longer to restore than the damage to such line-like facilities as roadways.

In Fig.6, it can be hypothesized that when there are fewer alternatives available and/or there are weaker key facility, more time is need for recovery.



A: Immediately after the quake
B: One week or more C: Two weeks or more
D: Three weeks or more E: One month or more

FIGURE 5. Recovery of highway traffics after the Miyagi-ken Oki Earthquake (after PWRI)

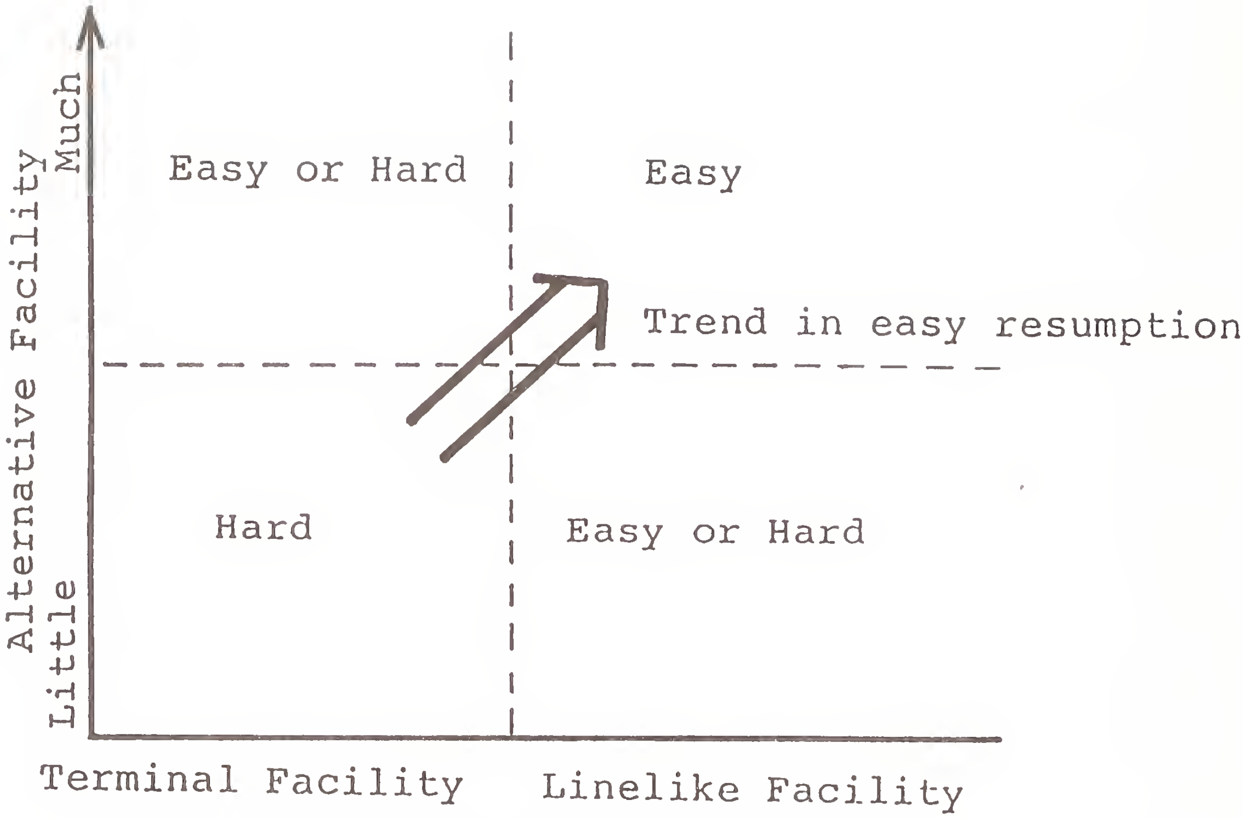


FIGURE 6. A Scheme of Resumption
for Lifelines [7]

FUNCTIONAL RECOVERY OF THE LIFELINE SYSTEMS [5][7]

The considerations for damaged lifeline systems after an earthquake could be patterned as follows:

(1)	EMERGENCY repairs	(E)
(2)	ALTERNATIVE measures	(A)
(3)	SUSPENSIVE functions	(S)

The following are examples of the above:

EMERGENCY repairs: In the Nihonkai-Chubu earthquake with magnitude of 7.7 on May 1983, the national highway 7, which linked the city of Akita and Noshiro, was temporary out of service around Noshiro in the Asauchi region, because of settlement of an underground channel. As a result of emergency repairs, its function was restored in twelve hours after the shock.

In the Algeria-El Asnam earthquake, the national highway which runs from the capital Algiers to Oran via El Asnam, had closed due to fault movement of 4.2 meters in the maximum gap. But the recovery was made by mending the part of a by-pass which was fortunately under construction. It was completed within 2 hours after the shock.

ALTERNATIVE measures: These will need to be taken for water supply, electrical power, gas, telecommunications and broadcasting. It is needless to say, but a tanker would be used for the emergency potable water supply.

SUSPENSIVE functions: This would be a rare occurrence in a post-quake situation. In the Izu-Ohshima Kinkai earthquake with a magnitude of 7.0 on January 14, 1978, a prefectural highway in a mountainous area linking Shuzenji and Shimada was damaged, with the high possibility of a secondary disaster. Partial service was restored using a makeshift road, and full service was resumed 2 years later with a different new road. In other words, the previous road was abandoned. Another example is the case of the El Asnam earthquake, mentioned above.

An extrapolation can be made using these three items, E, A and S with the following two criteria: "Availability of a function after an earthquake" and "Extent of earthquake damage."

If utility services are not functioning according to generally accepted standards, the mutual relationship of E, A and S should be considered. Optimum considerations are shown in (a) and (b) of Fig. 7 and Fig. 8, respectively.

The relationship among each of the two criteria above and E, A and S is shown in (c) and (d) in Fig. 7 and 8, respectively. A closed quadrilateral is drawn in the figures. In both of the figures, as the quadrilaterals get smaller, they show the much greater capability of functions and the less extent of damage. If the lines do not form a complete quadrilateral, it implies the lack of balance in the countermeasures before earthquakes.

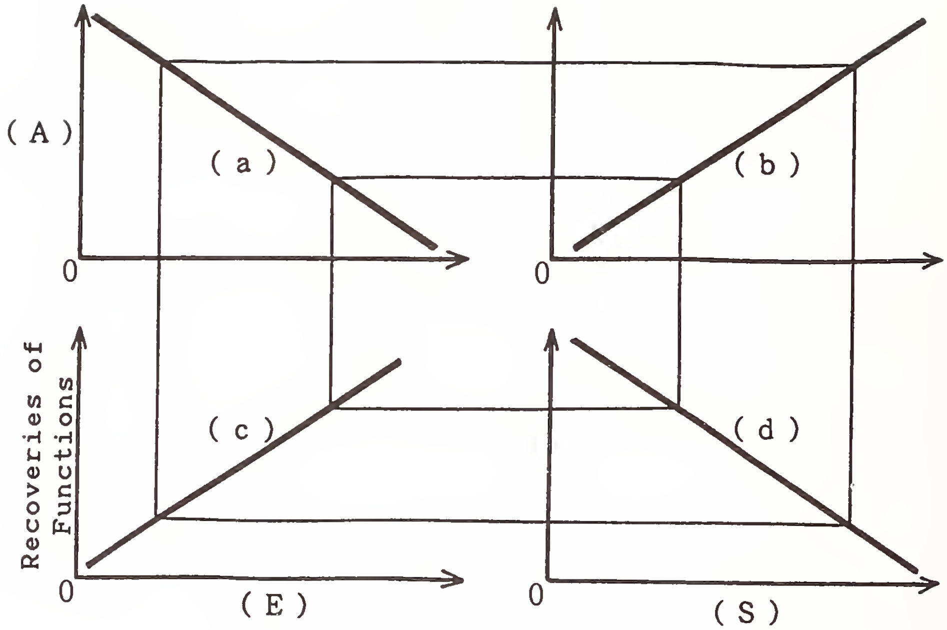


FIGURE 7. Multi-Relationships
in E,A,S and RF [7]

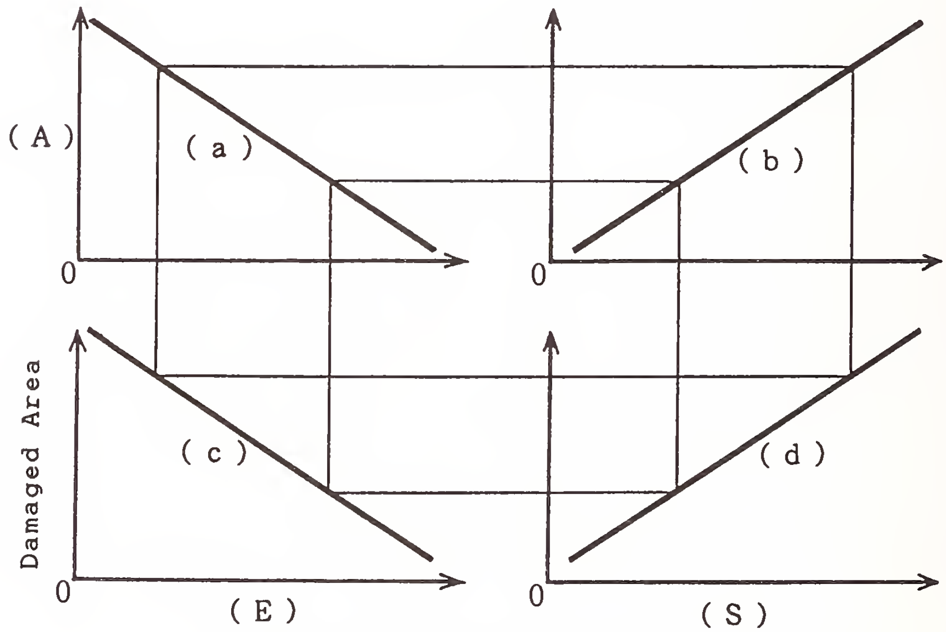


FIGURE 8. Multi-Relationships
in E,A,S and DA [7]

In general, it seems that the state of the suspensive functions is not acceptable. However, it is important to have provisions for realistic situations and constantly assuming the possibility of non-ideal situations.

CONCLUSIONS [7]

The following provisions must be made through the study of lifeline disasters:
(1) The system must have a redundant functional capacity as much as possible,
(2) The system must have economical alternatives to keep its function,
(3) The system must have measures to quickly recover the function after an earthquake, and
(4) The system must be improved in its earthquake resistance.

There are various types of lifeline systems today. Each bears close relationship with another, for example, covering the whole of Japan and making the Japanese islands into one big city. In many aspects, they bring greater convenience to daily life, but may possibly have some weak points when disasters occur.

It is desirable not only to consider each individual system or facility, but also to estimate and undertake the critical study of the system as a whole. A higher density of lifeline systems leads to an easier life, but also contains the possibility of sudden inconvenience or uneasiness in case of a disaster in an urban area. These disadvantages could possibly lead to a critical situation, especially in a large modernized city.

A lifeline system in a city is the equivalent of an artery or a nerve in a human body. It is impossible to strengthen such lifeline systems without strengthening the infra-structure of a whole city and to strengthen a city without strengthening lifeline systems, a good examples being the disaster caused by the Loma Prieta earthquake (M=7.1) occurred near San Francisco, California, U.S.A., on October 17, 1989. [8]

FUTURE DIRECTIONS

To improve earthquake resistance on each lifeline system in accordance with its importances, it is greatly recommended that study on geographic characteristics should be developed. It includes informations from basic conditions, such as physical conditions to much complex data of regional socio-economical conditions.

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ORGANIZATIONAL FEATURES OF U. S. LIFELINE SYSTEMS AND THEIR RELEVANCE FOR DISASTER MANAGEMENT

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ABSTRACT

Disaster-generated damage and disruption to lifeline systems produce a variety of impacts, ranging from direct effects, such as physical damage and service supply interruption, to indirect impacts, such as forced relocation of community residents, threats to ongoing social and economic activity, and delays in the disaster recovery process. Engineering studies focus primarily on the physical vulnerability of lifeline systems and on the technical aspects of mitigating disaster damage. Few studies have addressed the organizational properties of lifeline systems, their interrelationships, or their linkages to broader community disaster preparedness and response efforts. This paper highlights lessons learned in a recent Disaster Research Center project focusing on the performance of lifeline organizations in major community disasters. The paper discusses disaster management strategies at the organizational level as the outcome of three sets of factors: the structural features of the organizations that provide lifeline services; the institutional environment in which lifeline systems operate; and the task-related demands faced by lifeline organizations.

INTRODUCTION

Engineers are concerned with lifelines as physical systems. Their analyses focus primarily on the physical properties of lifelines, such as design features, materials used in construction, system configurations, capacities and outputs, and the spatial distribution of facilities and system components. In attempting to understand issues related to hazard mitigation, preparedness, response, and recovery, engineers are likely to place most emphasis on the physical or material side of the lifeline problem.

In contrast, sociologists are interested in lifeline service providers as organizations. Their analyses focus on the distinctive organizational properties of lifeline service providers and on the implications these organizational features have for disaster mitigation, preparedness, response, and recovery. In sociological approaches to the study of the lifeline problem, variables such as organizational structure, organizational adaptation processes, interorganizational relations, and organizational domains, tasks, and resources are seen as important influences on organizational performance.

In recent years, non-technical issues related to lifeline system performance have begun to receive more attention. For example, in a series of workshops and proceedings, the Building Seismic Safety Council (1987) identified a range of social factors that affect how lifeline organizations manage earthquake hazards and pointed out the need for research to better understand how these factors operate. That same report cited research findings that indicate that, except for a select group of organizations in high-risk areas of the U. S., lifeline service providers generally lack knowledge of the hazard and are far less committed than they could be to hazard reduction.

This paper discusses organizational features of the lifeline service provision system in the U. S. and illustrates how these features influence disaster planning and response activities.The examples and cases discussed in the paper were developed from a study that focused on disaster the preparedness and response of U. S. lifeline organizations. That project involved the collection of data on disaster preparedness activities among lifeline organizations in four U. S. communities and lifeline system impacts and organizational response activities in seven communities affected by recent U. S. disasters, including Hurricane Hugo and the Loma Prieta earthquake.[1] The study involved the collection of data on various types of lifeline organizations, including providers of natural gas, electric, water, and telephone services; transportation departments; port authorities; solid waste management organizations; and departments of public works. The sections that follow discuss what was learned in the course of the study about the organization of lifeline services in the U. S. and the impact organizational variables have on disaster management.

ORGANIZATIONAL AND ENVIRONMENTAL FACTORS AFFECTING DISASTER MANAGEMENT

A number of academic and applied disciplines, including management and business administration, are predicated on the notion that organizational form makes a difference in organizational performance. Given a particular set of goals for an enterprise, some ways of structuring that enterprise and organizing its members are better than others for achieving those goals in a particular societal and historical setting. Organizational performance and effectiveness can thus be attributed in part to organizational form. Examining alternative modes of lifeline system organization and their impact on organizational behavior

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[1] The four communities in which DRC conducted studies on organizational and community preparedness were Seattle, Washington; Richmond, Virginia; Buffalo, New York; and Portland, Oregon. Post-disaster response studies were conducted in Hamden, Connecticut; Charleston, South Carolina; Charlotte, North Carolina; and in Santa Cruz, San Francisco, and Oakland, California.

should provide useful insights on what features enhance their effectiveness and what features interfere with more effective performance with respect to disaster management.[2]

The kinds of disaster management strategies that lifeline organizations in the U. S. have developed and implemented are the result of complex sets of factors. For purposes of this discussion, three categories of factors will be discussed: properties of lifeline systems; characteristics of the institutional and societal environment in which lifeline systems operate; and task-related demands.

Properties of Lifeline Systems

Lifeline systems are characterized by a bewildering degree of variety and complexity. There are several ways in which the system is complex. First, modes of ownership vary, both across lifelines and across communities. Lifeline services may be provided by public, quasi-public, or private organizations, depending on the community context.

Different patterns of sponsorship entail different sets of organizational constraints and incentives. For example, Platt (1991) suggests that private utilities may tend to maintain and upgrade their facilities more readily to enhance their corporate reputations, shield themselves from liability, and protect capital investment. Generally speaking, such practices encourage hazard mitigation. On negative side, private utility providers are likely to be less well-integrated than public utilities with community-wide public sector disaster planning efforts, which can hamper efforts at interorganizational co-ordination in disasters. Government-owned utilities suffer from neglect during times of fiscal crisis and are more subject than private providers to the vicissitudes of the political process.

Recently, there has been a marked trend toward "privatization" of services in the U. S.(David, 1988). Services that were formerly provided by public entities, including services such as garbage removal and sanitation, are increasingly being sold off or contracted out to private entities. This trend has also added to the complexity of lifeline service provision tasks.

Second, different lifeline services vary in the extent to which they are consolidated with one another. In some cases, for example, a single organizational entity supplies more than one

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[2] The term "disaster management" is used here in a broad sense. The term encompasses four phases or task areas in which organizational action can reduce hazards and contain losses: pre-disaster mitigation and preparedness, and post-impact response and recovery.

lifeline service; for example, gas and electric service provision are frequently consolidated. In other cases, the services are separately owned and managed. An individual community may be responsible for providing the majority of lifeline services within its jurisdictional area, while a set of separate service providers may function in a nearby jurisdiction.

Consolidation of lifeline services increases the likelihood that the organizations involved will consider interdependencies among utility services in their planning and response activities and will integrate their disaster management activities. For example, we observed that in the Loma Prieta earthquake, the fact that both gas and electric services were provided by a single organizational entity (Pacific Gas and Electric) helped emergency response and restoration activities proceed more smoothly. On the other hand, lifeline service provision by separate organizational entities appears to hamper integration in planning and response activities.

A third type of complexity stems from the fact that lifeline service provision does not follow consistent jurisdictional boundaries. Some lifeline organizations cover enormous geographic areas--even crossing state boundaries--while others are confined to small geographic areas. Mammoth organizations like the California Department of Transportation, Pacific Gas and Electric, and Bell coexist with smaller entities that provide service to individual cities or to special districts.

This jurisdictional inconsistency makes joint disaster planning and response more difficult for lifeline organizations. Effective emergency planning and response necessitate ongoing relationships and interaction among the organizations responsible for those activities. In order to plan and respond effectively in disaster situations, service providers with large geographic jurisdictions must co-ordinate with a range of more locally-focused organizations that themselves may vary considerably in structure and function. Inconsistency among jurisdictions greatly complicates the problem of establishing these interorganizational linkages.

The fact that some lifeline organizations cover very large geographic areas also means that disaster vulnerabilities may vary considerably within an organization's area of responsibility. Large utilities may find themselves overemphasizing some types of disaster agents (such as those that affect the "home office") and being insufficiently aware of other potential problems. Additionally, when lifeline organizations have responsibility for vast geographic areas, key organizational activities are performed and decisions are made at considerable distance from the local communities those functions affect. As a consequence, the central administrative office of the lifeline utility may lack information on needs and resources in specific localities and may not have an adequate grasp of the factors that could complicate organizational performance in a disaster situation.

Some lifelines, such as crude oil and natural gas pipelines, are operated by organizations that have no counterparts or representatives at the local level. Local communities, and even state-level agencies, may not even be aware of the location of underground pipelines in their jurisdictions, since pipeline operators are typically not required to provide that information (Greene and Schulz, 1987; General Accounting Office, 1991). Lapses in planning and emergency response were evident in recent pipeline accidents in the U. S. (General Accounting Office, 1991).

Of course, as I note later, the fact that some lifelines are operated by very large entities can be an advantage. Lifeline providers with large jurisdictions tend to have large numbers of employees as well, which means that there is a pool of workers that can be utilized for repair and reconstruction in disaster situations. Other things being equal, very large lifeline organizations are probably also in a better position than smaller organizations to implement advanced hazard mitigation techniques. Smaller organizations may lack the financial resources to undertake needed work and may not consider mitigation cost-effective.

Lifeline organizations with large geographic service areas almost invariably decentralize many of their operations to the district level. District offices are likely to be more familiar than central administrative offices with local community organizations and local needs, and it is at this level that co-ordination typically takes place in emergency situations.

A fourth source of complexity is that lifeline systems differ in the manner in which their components are interrelated. Centralization, "tight coupling," and interdependence are three dimensions along which systems can be distinguished. Some lifeline organizations are more highly centralized than others, in that damage to a single key facility affects the operation of the entire system. A high degree of centralization makes a lifeline organization very vulnerable in a disaster situation, and this is one reason why lifeline organizations such as telephone and electric companies try to decentralize their operations and build redundancy into their systems. However, some key lifeline system elements--particularly the transmission systems associated with electricity and water--remain rather highly centralized, in that they depend particular supply sources.

A related property, "coupling," refers to the extent to which system components are dependent upon one another in producing an output, so that damage to one component renders other components in the system inoperative, even if they remain undamaged. Many lifeline systems are relatively "tightly-coupled," which adds to their vulnerability. For example, in Hurricane Hugo, the water system in Charleston was made inoperative as a result of three factors: damage at the filtration plant; water line breaks that reduced water pressure; and loss of electrical power for pumping operations. However, even if the system had not experienced the last two impacts, the loss of the filtration plant would have been sufficient to cause the failure of the entire system, because the operation of the water system depends critically on the plant.

Although electrical system service to the San Francisco area was restored relatively rapidly following the Loma Prieta earthquake, the power supply system was highly dependent on the continued functioning of Pacific Gas and Electric power generation facilities, which sustained severe damage. Because of this relatively "tight coupling," failures in those components led to failure of the entire power supply system.

In contrast, although the damage to the Bay Bridge and the loss of that transportation route for one month following the Loma Prieta earthquake presented severe problems for the Bay Area, transportation difficulties were comparatively easy to overcome, because the transportation system is a "loosely-coupled" and redundant system. The transit system was not paralyzed in the Bay Area, because the subway system and an expanded ferry transportation system were capable of functioning as enhanced alternative transportation lifelines following the disaster.[3]

Finally, rather than operating in isolation from one another, lifeline systems are interdependent. The most obvious manifestation of this interdependence is the reliance of other lifeline functions on electrical power. Electrical power is fundamental to the operation of most other systems, and electrical failures can render other lifelines inoperable even in the absence of direct damage. In Hurricane Hugo, for example, the widespread loss of electrical power affected water distribution systems, waste water treatment systems, local telephone PBX systems, traffic signals, port operations, and other lifeline services. In non-urban areas, communities and households on well systems were unable to pump water. There was an intense demand for emergency generators and for generator fuel following the hurricane, because so many facilities that depend on power needed to be operational.

Extensive dependence upon electrical power also meant that there was conflict about power restoration priorities in some of the disaster-stricken communities we studied. Since in many cases

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[3] The term "coupling" is used in this paper primarily to refer to the physical components of lifeline systems. This concept also has an organizational dimension. Organizations can attempt to partly compensate for "tightly-coupled" technologies by devising organizational solutions. For a more through discussion of "coupling" in physical and organizational systems, see Perrow, 1984.

disaster damage and impacts were unanticipated, priorities had to be established on a case-by-case basis during the emergency period.

Many of the lifeline organizations we studied failed to appreciate the extent to which their own operations were dependent on other services and failed to take interdependence into consideration in their mitigation and preparedness activities. However, as noted earlier, in situations in which lifeline functions are consolidated, e.g., in combined gas and power companies, lifeline interdependence is better understood, and coordination of emergency response and restoration activities is less difficult.

The Institutional Environment

Like all organizations regardless of function, lifeline organizations are affected by their environments.[4] Many of their activities can be traced directly to requirements imposed by outside organizational actors, and the somewhat uncertain and inconsistent environment surrounding disaster management requirements for lifeline organizations has had an impact on the measures these organizations have adopted. The general climate for lifeline organizations might best be described as a combination of a laissez-faire approach and the setting of minimal performance standards. These points are elaborated below.

<u>General Absence of a Requirement to Mitigate and Prepare for Disasters.</u> Lifeline organizations are generally aware of the need to prepare for emergencies that affect organizational functioning. Many lifeline organizations are also aware of the need to engage of mitigation and preparedness for major natural disasters. Some lifeline organizations have concentrated considerable effort on developing hazard mitigation, preparedness, and response capabilities. However, such actions, when undertaken, are largely done voluntarily, not in response to any specific requirements or regulations. In this respect, lifeline organizations can be contrasted with organizations such as hospitals, which are required to engage in disaster planning and to conduct regular disaster exercises as a condition for accreditation.

As noted earlier, effective disaster response, restoration, and recovery effort are typically based on extensive pre-disaster contact and collaboration among responding organizations. In the community studies we conducted, we found several instances of frequent pre-disaster contact and joint planning between lifeline

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[4] For purposes of this discussion, the "environment" consists of other organizational actors external to the various lifeline organizations, including agencies at the various governmental levels, regulators, and other organizations whose activities affect the provision of lifeline services.

organizations and other community response organizations, such as local emergency management agencies. However, we also observed situations in which pre-disaster contact between key lifeline providers and other critical emergency responders was minimal and in which the lack of pre-planning resulted in co-ordination problems during the emergency period. When preparedness networks developed, they were the outcome of distinctive local circumstances and local history, rather than specific mandates to engage in joint disaster management efforts.

The societal environment also affects the behavior of lifeline organizations with respect to pre-disaster mitigation measures. In a recent paper, Buckle (1991) notes that several barriers impede improved hazard earthquake mitigation for lifeline systems. Among these barriers are an absence of authoritative codes and standards for seismic design and construction and for retrofitting existing structures. In the absence of clear standards and guidelines, decisions about mitigation are left in the hands of individual organizations and design professionals. A recent Federal government mandate requiring the development of seismic performance standards for lifelines may ultimately change organizational mitigation practices for earthquakes, but that impact will not be felt for quite some time.

Currently, some lifeline organizations (typically those with abundant resources and those with considerable disaster experience) are very capable of coping with large-scale disasters. For example, several California utilities we studied had invested extensively in both mitigation and preparedness for disasters, particular earthquakes. However, this is not a national pattern; lifeline organizations generally do not give hazard reduction a high priority. In the mitigation area, for example, use of buried electrical lines rather than aerial lines cuts down on wind damage. Both of the two major electrical utilities we studied following Hurricane Hugo were still using above-ground lines at the time of the disaster, and consequently the damage to the electrical distribution system was devastating.

In short, as a consequence of the laissez faire environment in which lifeline organizations undertake their hazard management activities, a "patchwork" situation exists nationally, in which some utility services are more reliable than others in disaster situations, and in which some communities are better served following disasters than others. Large, wealthy, well-managed utilities--particularly those with extensive disaster experience--are often superbly prepared for disasters, while their less well-off, less experienced counterparts lag considerably in all areas.

<u>Weak Incentives to Mitigate and Prepare for Disasters.</u> In the earthquake area, Buckle (1991: 13) notes that for lifeline systems at the present time "There are no tax policies, insurance incentives or rate adjustment schemes which encourage owners to

either upgrade existing facilities or to use higher design standards for new systems." Reducing the disaster vulnerability of lifeline systems can be a very expensive undertaking, involving replacement, retrofitting of components, and in some cases relocation of entire facilities. High costs and the absence of fiscal incentives make hazard reduction impractical for many lifeline organizations.

Legal liability for injury, damage, and losses resulting from lifeline failures and service interruption in disaster situations could conceivably be a factor influencing lifeline organizations to mitigate and prepare for disasters. At some future time, concern about liability may begin to motivate lifeline organizations to mitigate and prepare for disasters. However, at the present time, the legal and regulatory picture is unclear. States vary considerably in terms of their legal provisions; in some states, public and quasi-public entities are immune from liability under most circumstances. In the absence of credible assessments of the risks associated with various hazards in specific localities, formal specifications of performance criteria for lifeline systems and their components, state and local laws requiring improved hazard abatement, and assignment of responsibility for the enforcement of hazard reduction measures for lifeline systems, liability concerns are not likely to motivate organizations to improve their disaster management strategies (for more detailed discussions focusing specifically on the earthquake problem, see Malik, 1987 and Miller, 1987).

Some lifeline organizations voluntarily engage in costly mitigation and preparedness programs primarily because they know it is critical to corporate finances to avoid disaster losses and associated costs. However, such measures are typically seen as cost-effective and justifiable only in those regions of the country where risks are high and disaster experience is extensive. For example, among utility companies, we did not observe any organizations in other parts of the U. S. that had the same degree of commitment to hazard reduction as those in California.

In understanding factors that affect organizational performance, it is also important to note that most lifeline organizations are essentially monopolies.[5] Obviously, those in key

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[5] Since deregulation of the telephone industry, long-distance telephone service is one notable exception to this pattern. Most consumers are able to choose among several long-distance carriers, and concern about the quality of telephone service is one factor that influences that choice. Consumers have some degree of choice about the transportation systems they use for commuting and other daily activities, and they typically make those choices based on quality of service, convenience, and cost. However, choices for most other utility services are more or less fixed for community

positions in these organizations are aware that they provide critical community services, and they doubtless want to provide the best possible level of service. However, being prepared for disasters is typically not a matter of organizational survival for lifeline providers, as it can be for other organizations. Other enterprises that fail to cope well with disaster-related demands can expect to lose business to their competitors, but this is generally not the case for lifeline organizations. Customers who are forced to wait for the restoration of their utilities may be inconvenienced, and the company may lose public good will, but the company will not cease to operate.

Private utility companies certainly have reason to be concerned about company profits if they fail to mitigate and prepare for disasters, but many organizations do not yet think in these terms. Publicly-owned utilities have even fewer fiscal incentives to reduce losses and may have much more difficulty raising the money to make improvements in their systems. Moreover, public entities, including publicly-owned lifeline organizations and special assessment districts, can expect to receive Federal government compensation for their losses in the event of a Federally-declared disaster.[6]

Task-Related Demands

To some extent, the ways that lifeline organizations adapt in disaster situations are an outgrowth of the manner in which these organizations structure themselves to deal with their day-to-day service-provision tasks. Unlike many other types of organizations (e.g, restaurants, banks, commercial establishments in general, service establishments, and many manufacturing firms), lifeline organizations, particularly providers of essential utility services, must be able to deliver service on a continuous basis. In this respect, lifeline organizations resemble major public safety and health care agencies (e.g., fire departments, police departments, hospitals), which must also perform the tasks for which they are responsible on a more or less continuous basis. Lapses in lifeline service do occur from time to time, of course; blackouts, brownouts, and service outages, nonfunctioning traffic signals, and airport shutdowns are not entirely unknown. However, generally speaking, utility customers expect a very high level of

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residents.

[6] Indeed, if used properly, disaster assistance funds may provide an important vehicle for reducing the disaster vulnerability of lifeline systems. Federal law requires a that a proportion of the funds provided to public entities for disaster recovery be used for the mitigation of future hazards. Community utility systems could use these funds creatively to mitigate future losses.

performance from these types of organizations, and lifeline organizations are structured to meet this expectation.

Distinctive structural properties result from these task requirements. For example, staffs of major lifeline organizations are typically large, and some degree of staffing is provided around the clock. Because of the need to maintain continuous operations, lifeline organizations invariably have plans and procedures for "routine," everyday emergencies that might threaten to interrupt service. Mutual aid agreements are also common among organizations providing the same services in different geographic areas. These agreements may involve sharing utility commodities (e.g., electricity, water), personnel, or other resources.

All these arrangements help give lifeline organizations considerable flexibility when they are required to respond to a major disaster. For example, researchers who study organizational adaptation in major crisis situations have noted that organizations alter their structure and functioning in typical ways when faced with the need to meet excessive demands (c.f., Brouillette and Quarantelli, 1971). Expansion and extension are two common types of adaptation. In the first adaptive mode, expansion, the organization performs the same sorts of activities as it did during routine times, but many more people are involved in those tasks. Extending organizations keep their pre-disaster structure, but engage in new, non-routine tasks.

Large staffs and an organizational schedule that involves round-the-clock shifts give lifeline organizations a large pool of trained personnel from which to draw in a serious emergency and permit rapid organizational expansion in critical situations. In this respect, they resemble other key disaster response organizations, such as fire departments, police departments, and hospitals.

Mutual aid agreements among lifeline providers also facilitate expansion in times of crisis. For example, following both Hurricane Hugo and the Loma Prieta earthquake, power companies mobilized very large numbers of people quite rapidly. South Carolina Gas and Electric received assistance from 48 electric companies in 15 states and expanded its staff of linemen from 95 to 2,600. In North Carolina, Duke Power used 3,000 workers in emergency response and service restoration, including workers from 40 other power companies throughout the South.[7] Lifeline

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[7] Extending patterns were also observed in lifeline organizations, but these were less frequent and appeared to be more problematic for the organizations involved. Some organizations had to both expand and extend. For example, following Loma Prieta, one urban public works department, in addition to expanding by mobilizing all its personnel and taking on numerous volunteers,

organizations are, in a sense, well-structured to expand in major emergencies. Organizations that have formal agreements and concrete plans for incorporating personnel mobilized from the outside obviously have fewer problems in major emergencies than those that expand in an ad hoc fashion.[8]

On the negative side, however, the fact that lifeline organizations are aware of the need to plan for "routine" emergencies does not mean that they are automatically well-prepared for major disasters. In the lifeline organizations we studied, we found that it was quite common for personnel to consider disasters simply extensions of everyday emergencies. Rather than representing a quantitative difference in the demands placed on organizations, a disaster situation requires qualitatively different tasks, activities, and decisions. Those organizations that had not taken into account how disasters and ordinary emergencies differ were forced to use costly time and effort to improvise new structures and procedures during the emergency period.

CONCLUDING COMMENTS

Reflecting on the status of disaster management efforts in the U. S., Platt (1991: 173) notes an irony: while our communities may be safer than ever before from direct threats to life-safety and direct disaster losses, "indirect economic and personal hardships, and even threats to life, arise from the failure or disruption of the vital regional circulatory systems over which the private individual and often the local government have no control." Lifelines have clearly not received the emphasis they deserve in either hazard reduction policy or in practice. Because of recent dramatic lifeline failures and losses, such as the Nimitz Freeway collapse in the Loma Prieta earthquake, hazard mitigation for lifeline systems is receiving more attention in the U. S. In considering how to reduce the hazards associated with lifeline performance, along with increasing the resistance of physical lifeline systems, we must consider ways to increase the

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also had to extend its activities into building inspection and damage assessment, which were entirely new tasks for that organization. Accomplishing such profound changes in structure and activities in a crisis situation is extremely difficult for an organization.

[8] However, organizations may not always take into account all they need to do to successfully manage expansion. For example, following Hurricane Hugo, several utilities mobilized large numbers of workers from outside the affected area. The companies were able to use these workers very effectively in repair and restoration activities. However, they had neglected to consider how they would house and feed these additional personnel.

organizational capacity of service providers. Determining how physical systems can be made more resistant, resilient, and redundant is an important topic for research. It is equally important to understand how best to employ principles of organizational design to improve the performance of lifeline organizations in disasters.

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A Methodology for Assessing the Risk of Hazardous Materials Release Following Earthquakes - A Demonstration Study for the Los Angeles Area

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ABSTRACT

A methodology for estimating the risk of earthquake-induced hazardous materials releases was developed for the National Science Foundation and the National Center for Earthquake Engineering Research. Seismic hazard analyses, fragility modeling for facilities handling hazardous materials and data on airborne materials releases were used in the development of the methodology. The risk was estimated in terms of population within the study area exposed to hazardous materials as a result of a postulated earthquake event. The procedure was developed to be used as a tool by communities interested in regional hazard management.

In order to demonstrate the methodology, Los Angeles County was selected as a study area. Population data was integrated into the methodology to predict the population exposure to hazardous materials releases for three earthquake scenarios: a Magnitude 8+ event on the San Andreas fault, a Magnitude 7 event on the Newport-Inglewood fault, and a Magnitude 5.9 simulation of the 1987 Whittier Narrows earthquake.

INTRODUCTION

Exposure to hazardous materials as a result of an earthquake-induced materials release is a threat to the population in the immediate vicinity of any storage, handling and processing facilities, as well as to the surrounding communities. Although there has never been a major incident involving hazardous materials in a U.S. earthquake, smaller releases have occurred in events that were moderate in size. In the 1989 Loma Prieta earthquake, numerous minor releases were reported, including a leak of at least 5000 pounds of anhydrous ammonia from a food processing plant in Watsonville (ABAG, 1990).

Responding to an earthquake-induced hazardous materials release presents challenges not faced in other hazardous materials emergencies. Following a major earthquake event, heavy demands are likely to be made on community emergency response capabilities and resources, making it difficult to effectively deal with secondary emergencies such as hazardous materials releases and fires. Problematic tasks associated with response to a hazardous materials release, including warning the public and evacuating hazardous areas, would be much more difficult following a major earthquake. Further, resource problems will be compounded by possible simultaneous hazardous materials release.

While awareness of the risk is growing, there has been little research to date on the seismic sources of hazardous materials releases, and seismic vulnerability models for chemical facilities are almost nonexistent. The research for this project combines seismic hazard analyses, findings from research on earthquake-related failures in industrial facilities, and data on airborne toxic releases to develop a general methodology that would enable local jurisdictions to determine the magnitude of the problem in their community and identify areas that are susceptible to exposure due to earthquake-generated releases. This paper demonstrates the application of this methodology to the Los Angeles County area.

METHODOLOGY

The methodology developed for this project is diagrammed in Figure 1 and discussed more fully in Tierney et al.,

1991. Its application in this demonstration study may be summarized as follows:

Hazardous Material Inventory

Hazardous materials number in the thousands, and new products are constantly being developed. Before a systematic analysis can be undertaken, it is necessary to determine which hazardous substances are likely to pose the biggest threat to the community in an earthquake. For this demonstration, we have chosen to focus on two hazardous materials; chlorine and ammonia. These substances were selected because: (1) they are responsible for the majority of fatalities and casualties in U.S. hazardous materials incidents; (2) they are present in large quantities in the study area, Greater Los Angeles; and (3) they form clouds that can spread to adjacent areas, thus presenting a hazard beyond the plant gates.

The facilities assumed to be possible sources of hazardous materials in this study are twenty-two of the largest users of chlorine and anhydrous ammonia in the greater Los Angeles area. These users include petroleum refineries, chemical manufacturers, and wastewater treatment plants. Although the methodology as developed calls for data obtained from inventories prepared under state and federal laws, data on the subject facilities were actually obtained from a survey conducted by the South Coast Air Quality Management District.

These facilities, dispersed throughout the study area, store and use varying amounts of chemicals. Facilities have been categorized into three facility types based on chemical usage patterns: chlorine storage facilities, ammonia storage facilities, and ammonia processing facilities. Chlorine storage amounts range from 4 to 1000 tons, while ammonia storage varies from 2 to 206 tons.

Earthquake Scenarios and Ground Shaking Estimates

Seismic hazard estimates have been developed for three different earthquake scenarios. In this demonstration, strong ground shaking was the only hazard considered, although additional hazard estimates could be developed for other earthquake effects, such as fault rupture, liquefaction and other ground failures.

Scenario 1 is a Magnitude 7.0 event on the Newport-Inglewood fault. This fault was the source of the 1933 Long Beach earthquake (M 6.3), which caused 120 deaths and $41 million (1933 dollars) in damage. A major earthquake on the Newport-Inglewood Fault would likely result in numerous fatalities and injuries, billions of dollars in damages, and severe disruption of economic activity at the local, regional, state and even national levels.

Scenario 2 is a Magnitude 8.3 earthquake on the San Andreas fault. This event involves 300 km of fault rupture along the Mojave, San Bernardino Mountains, and Coachella Valley segments of the fault. Such an event would be expected to cause high ground shaking levels throughout the Los Angeles Basin. As with the Newport-Inglewood event, losses and disruption would be significant.

Scenario 3 is a Magnitude 5.9 simulation of the 1987 Whittier Narrows Earthquake. This earthquake, with localized strong ground shaking, caused few deaths and injuries, but produced losses exceeding $350 million. In addition, the earthquake caused a significant hazardous materials incident. A tank in the City of Santa Fe Springs ruptured and leaked 240 gallons of chlorine into the air. The resulting plume, which drifted through the industrial section of the city toward Whittier, prompted evacuation of some areas (FEMA, 1987).

Ground shaking intensities at each facility were computed for each of the three earthquake scenarios, as follows. Peak ground accelerations (PGAs) were calculated at each location using a deterministic magnitude-distance attenuation relationship. Calculated PGAs were then converted to values of ground shaking intensity based on the Modified Mercalli Intensity (MMI) scale. These conversions yield MMI values equivalent to PGA values for sites located on "basement rock". These MMI values were then modified to account for variations in local ground conditions from "basement rock". Figure 2 shows the resulting seismic hazard map for Scenario 1 and indicates the locations of the 22 hazardous materials sources.

Chemical Facilities Modeling

Two "generic" facility models, a "generic" chemical processing facility and a "generic" storage and transfer facility, were developed for reasons of economy and efficiency. It was assumed that facilities that perform the same function have more or less the same components, allowing for analysis by generalized facility type, rather than on an individual facility basis. This assumption is particularly applicable to the Los Angeles area, where the range of chemical facilities types is somewhat limited. Facilities are generally comprised of the same components and follow similar process operations, using gaseous toxic chemicals as reactants in the manufacturing process.

Components of the chemical processing facility model that are subject to failure include the: (1) pressurized storage vessel; (2) exothermic reactor; (3) piping; and (4) separator/regenerator. The storage and transfer facility model is simply a subset of the processing model, consisting of a storage vessel and associated piping.

Facility Vulnerability Assessment

In analyzing complex systems, such as chemical facilities, it is not possible to identify just one or two failure modes that lead to overall system failure. Instead, all conceivable failure modes must be identified, and their individual contributions to overall facility failure must be systematically combined. Fault tree analysis is useful for this kind of assessment. In fault tree analysis, boolean techniques are used to model the interdependency of individual component failures. Cases where several failure modes must occur for some "fault" to occur are modeled using "AND gates." Cases where some "fault" can occur due to one or more failure modes are modeled using "OR gates". For this study, fault tree models were developed for earthquake-generated failures and toxic releases for chemical processing facilities, and storage and transfer facilities. The development of these fault tree models and the resultant failure curves for each facility type is discussed in detail in Tierney et al. (1991).

Plume Modeling

A chemical dispersion analysis was performed to estimate the size and shape of the area exposed to anhydrous ammonia (NH_3) or chlorine gas (Cl_2) following an earthquake-induced hazardous materials release. The dispersion of the resulting hazardous materials clouds was modelled using the SLAB dispersion model (Ermak, 1989), for various meteorological conditions typical of the Southern California Air Basin. The results of this analysis yield a conservative estimate of the zone of vulnerability, or area in which specific health criteria may be exceeded, for a given release and meteorological condition.

In order to determine potential zones of vulnerability, it was necessary to establish health criteria for both Cl_2 and NH_3. The chemical-specific health criteria used were based on the Emergency Response Planning Guidelines (ERPGs) developed by a committee of the American Industrial Hygiene Association (AIHA). The criteria selected for this study was ERPG 3, "the maximum airborne concentration below which it is believed that nearly all individuals could be exposed for up to one hour without experiencing or developing life-threatening health effects." This exposure level is 20 ppm for Chlorine and 1000 ppm for Ammonia.

For each meteorological condition and mode of chemical release, a zone of vulnerability or hazard footprint was determined. As a conservative estimate, the composite maximum width and length were taken to represent a generalized footprint for each release mode (i.e., the largest width and length from all meteorological conditions are used to define the exposure area for each release mode).

Because it would be virtually impossible to account for all of the variables that influence the position of the hazardous materials plume, such as wind speed and direction, a probabilistic approach was used to determine the likelihood that a given site will be within a hazardous material plume. Although hazard footprints are sometimes irregular, varying from tear-drop shape to circular, hazardous materials plumes were modelled as ellipses. This general model was deemed appropriate because it captured most of the characteristics of the irregular footprints.

A mathematical derivation yields the probability of a given site being located within a plume of given dimensions. Given an elliptical plume pattern, the plume must exist somewhere within a circle defined by sweeping the ellipse (fixed at the source) through a 360 degree arc. The plume's exact position within this circle is unknown. Only sites within this circle can be exposed to the chemical plume.

If one draws a circle with the center at the source, and the radius equal to the distance from the source to the site, the site will be within the plume if it sits anywhere along the arc defined by the intersection of this circle and the plume. Since the width of the plume, and hence the length of this arc, varies with the distance from the source, the probability of the site being located within the plume varies with distance from the source. Hence, this probability will depend on three factors; the parameters that define the plume (semi-axes a and b), and the distance, d, from the site to the source of the plume.

The probability, P, that the site will be located along the arc located within the plume is the ratio of the arc length, S, to the circumference of the circle whose radius is equal to the distance from the source to the site: $P=\frac{S}{C}=\frac{2\theta d}{2\pi d}=\frac{\theta}{\pi}$

where Theta, measured in radians, represents the angle between the plume axis and a line connecting the source to the site.

Population Data

1980 census data was obtained for all enumeration districts in the five county Los Angeles Basin area; Los Angeles, Orange, Riverside, San Bernardino, and Ventura Counties. In these five counties, a total of 10,370 enumeration districts represent 11.5 million people. For each enumeration district, the population count is associated with a representative geographic point location.

COMPUTER MODELLING

A computer program was developed to determine the overall risk of exposure, from aggregated information collected during the first six steps of the risk assessment methodology. Program "Plume" was designed to take the collected data as input, and output the number of people exposed to hazardous chemicals in a given earthquake event.

A probabilistic approach was used to develop Program "Plume". The general procedure used to calculate population exposure at a given site from a given hazardous materials source, for a given earthquake event is as follows:

1) Based on the ground shaking intensity (MMI) at the hazardous materials source, calculate the probability of failure in each failure mode for each facility component. Also note the resultant plume size if failure occurs in each component.

2) For each population center, calculate the distance from hazardous materials source to the population site.

3) For each component at the source facility, check whether the population site could be located within the resultant plume if failure occurs.

4) If the population site is within the plume's extent, calculate the probability that the plume will form over the site.

5) Aggregate these probabilities for all components at the source to find the total probability of exposure at the population site for release at this source.

These values may be aggregated such that the total exposure of each site from all sources is produced. In our example, the exposure is further aggregated to the County level.

RESULTS

The computer analysis yielded the number of people exposed to hazardous materials in each of the five counties, as a result of each scenario earthquake. Only Los Angeles and Orange Counties were found to be affected by possible hazardous materials releases from the listed 22 sources within Los Angeles County. As a result of a Magnitude 7.0 earthquake on the Newport-Inglewood fault, 133,000 people in Los Angeles and Orange Counties would be exposed to hazardous materials released from the 22 subject sources. (1.8% of the population in Los Angeles County, and 0.03% of the people in Orange County). These 133,000 people are dispersed throughout more than 3000 enumeration districts. Of these 3000 enumeration districts, only 1% have more than 500 people affected, 90% have fewer than 100 people affected, and 40% have fewer than 10 people affected. The maximum number of people exposed at any one site is approximately 1400.

From these same sources, a total of 20,763 people would suffer exposure to hazardous materials following a M 8.3 event on the San Andreas fault. (0.3% of the population in Los Angeles County, and 0.01% of those in Orange County). The stricken population would be distributed among 2,860 enumeration districts. Of these districts, 99.9% would have fewer than 100 people affected, and 81.5% would have fewer than 10 people affected. The most affected at one site would be only 211 people.

From the smallest of the three events, the Whittier-Narrows simulation, only 6660 people (0.09% of the people in Los Angeles County, and less than 0.01% in Orange County) would be affected by the hazardous materials release. 1800 enumeration districts would be affected; 99.7% of these would have fewer than 25 people affected, and 75% would have fewer than 5 people affected. The largest number of people affected by the release in any one enumeration district is 57 people.

For the event presenting the greatest threat to population, the Newport-Inglewood event, the locations of the 20 enumeration districts with the greatest number of people affected by hazardous releases have been identified, and are plotted in relation to the potential sources in Figure 3. Each of these districts has more than 500 people affected, and the total number of people affected within these districts comprises 12% of the overall number of people affected by hazardous releases in this event.

CONCLUSIONS

The threat of hazardous materials release exists wherever hazardous materials are stored. Earthquake-induced releases are a very real possibility. Based on the 22 sources identified for this study, the most serious releases would occur not in the largest postulated earthquake, but in the earthquake causing the strongest ground shaking at the hazardous materials sources. This earthquake, the Magnitude 7.0 Newport-Inglewood event, would cause ground shaking of at least intensity VIII at all but two of the studied sources. In contrast, the M 8.3 San Andreas event causes MMI VIII or more at only 4 sites. This type of information would be useful in the planning efforts of local communities. If a community could identify those facilities likely to be in areas of strong ground shaking in postulated earthquakes representative of the local seismic hazard, they could concentrate mitigative efforts on these facilities.

One of the most serious hazardous materials threats is presented by the storage of large quantities of chlorine in areas expected to suffer strong ground shaking. Chlorine is stored in vessels as large as 90-ton rail cars, whose failure plumes can extend over 7 miles. The identification of chlorine as the more serious threat enables users to address this risk by concentrating efforts in improving performance of existing vessels, developing smaller safer vessels, or perhaps relocating storage facilities.

The failure models developed for use in this study are based on conservative assumptions regarding failure thresholds. Even with these conservative assumptions, the largest total expected population affected in any of the three scenarios is 132,000 or less than 2 percent of the total population of Los Angeles County. These estimates, however, do not include risks that may result from failure of chemical facilities in counties other than Los Angeles, or from chemicals other than ammonia or chlorine. A more complete analysis of risk must include these other facilities and chemicals.

FUTURE DIRECTIONS IN RESEARCH AND APPLICATION

There are various types of research that would make this methodology more widely applicable. Some are widely explored, such as improved seismic hazard assessments, while others are specific to this type of analysis. Possibilities for this type of research include developing an extensive library of plume patterns for a wide variety of hazardous chemicals, and developing additional chemical facility models and failure curves.

To further explore the benefits to the planning efforts of a local jurisdiction, the completion of a smaller scale, detailed risk assessment including a more extensive inventory of chemicals and sources, tied to the development of detailed response and evacuation plans, would be the next logical step in the development of this methodology.

ACKNOWLEDGMENTS

The authors wish to acknowledge the support of the National Science Foundation and the National Center for Earthquake Engineering Research for this project. Specifically, we would like to thank Dr. William Anderson of NSF and Drs. Barclay Jones and Masanobu Shinozuka of NCEER. The authors would also like to thank Mr. Ken Richmond of TRC Environmental Consultants for his help in performing the air dispersion analysis, and Mr. Robert A. Antonoplis of Sidley and Austin, Los Angeles, for his support in developing the generic chemical facility models.

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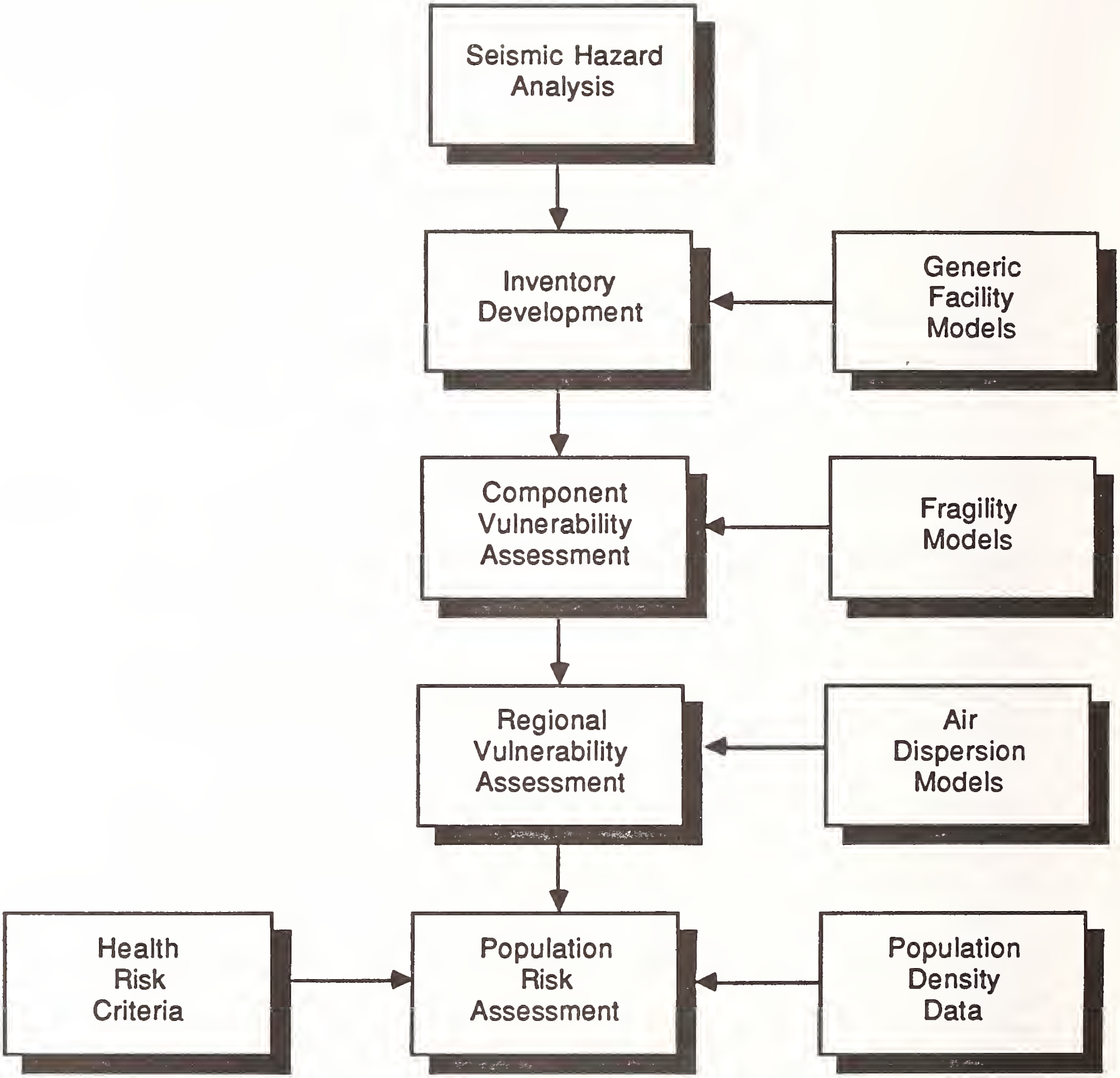
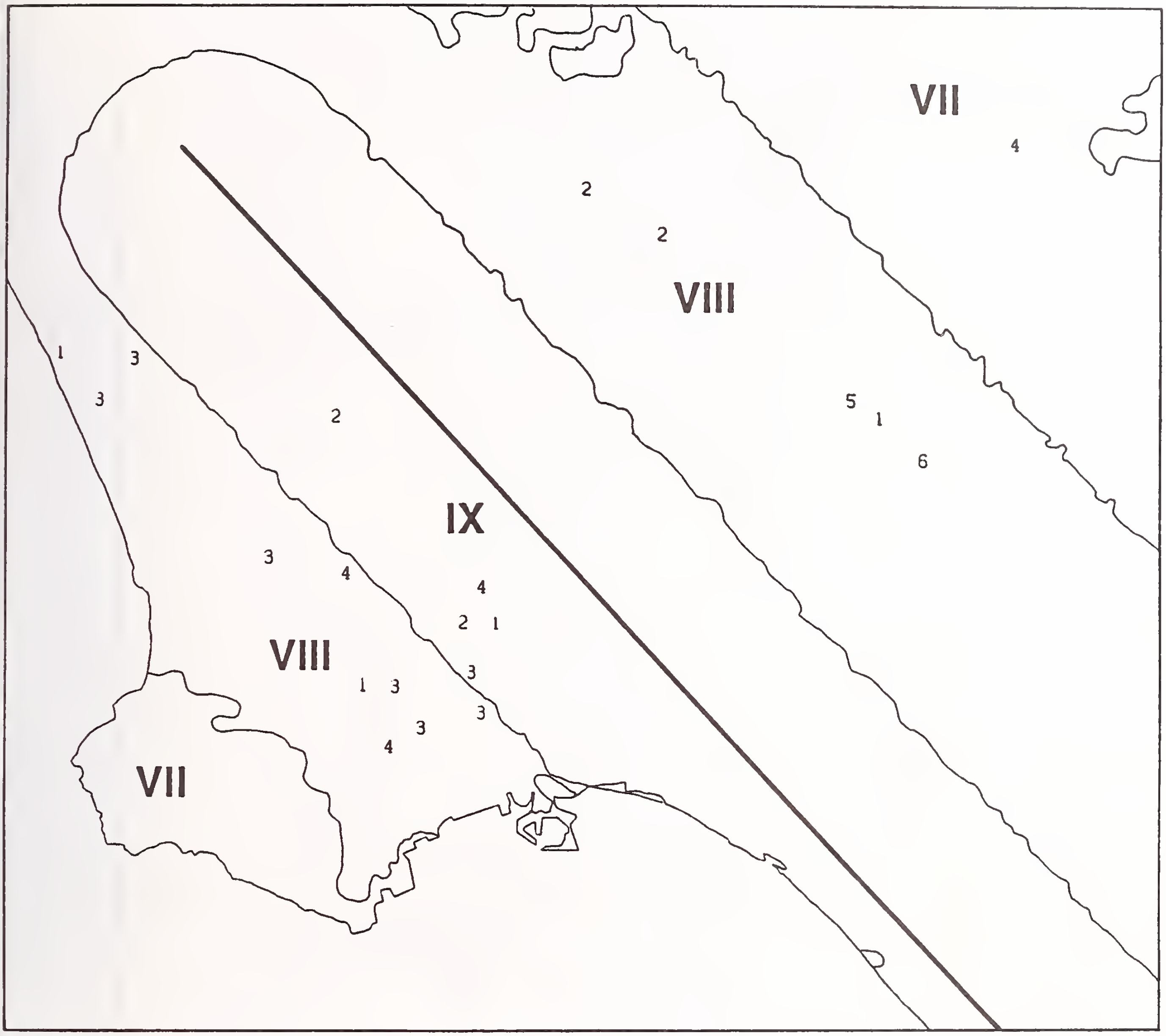


Figure 1. Methodology for Risk Assessment of Hazardous Materials Release During Earthquake



N

EXPLANATION

1 Chlorine Storage

2 Chlorine Processing

3 Chlorine Storage and Ammonia Processing

4 Chlorine Storage and Ammonia storage

5 Chlorine Processing and Ammonia Storage

6 Ammonia Storage

0 4.5 9.0

SCALE IN MILES

Figure 2. Seismic Hazard Map (Modified Mercalli Intensity) for a Magnitude 7.0 Earthquake on the Newport-Inglewood Fault with Site Locations

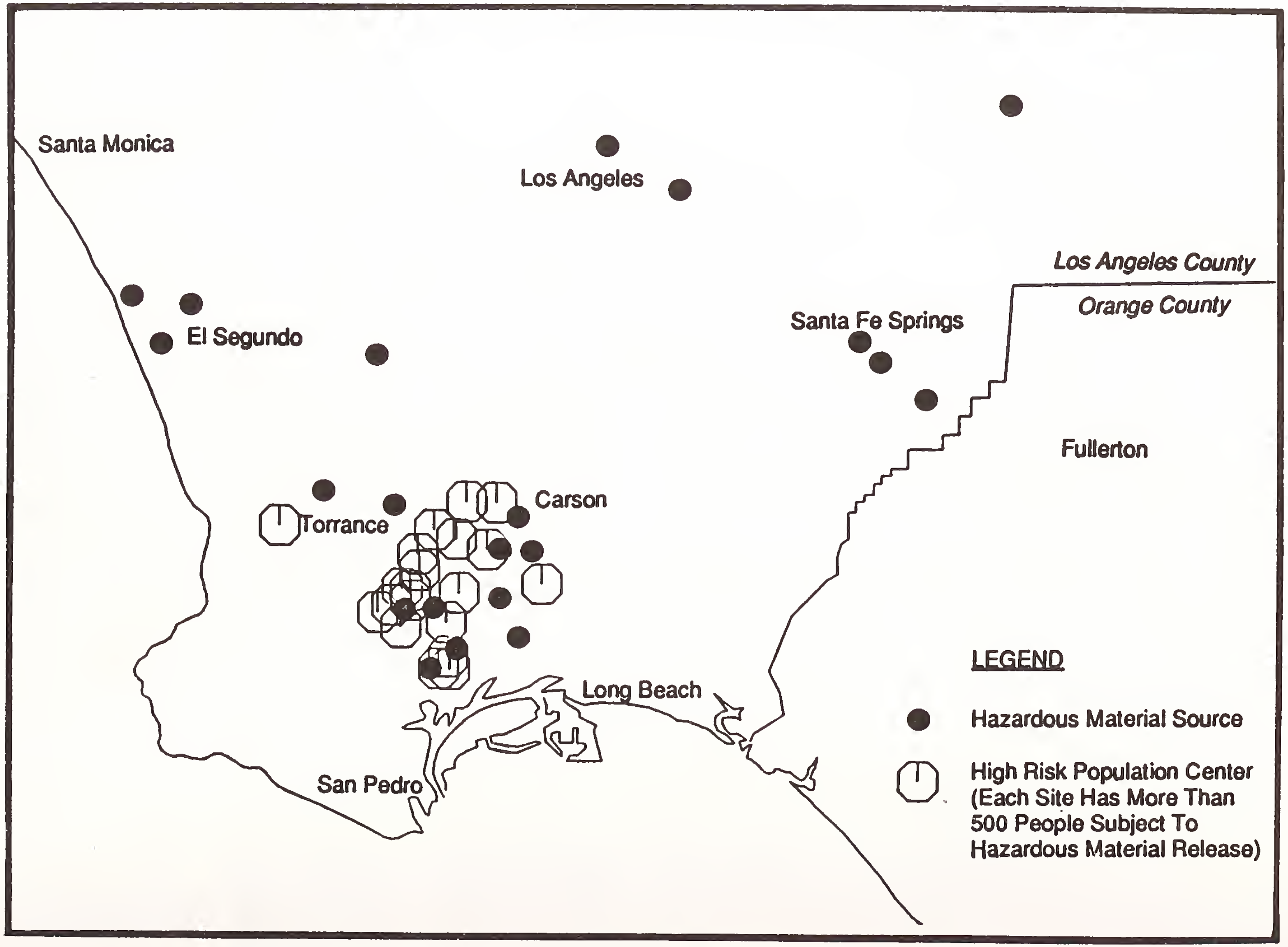


Figure 3. Population Centers with High Risk Potential from Hazardous Materials Release During a Magnitude 7.0 Earthquake on the Newport-Inglewood Fault

VIII. WORKSHOP RESOLUTION

RESOLUTIONS OF THE FOURTH U.S.-JAPAN WORKSHOP ON EARTHQUAKE DISASTER PREVENTION FOR LIFELINE SYSTEMS

Biltmore Hotel, Los Angeles, California
August 19-21, 1991

1. The Fourth U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems was held under the sponsorship of the Public Works Research Institute of Japan and the U.S. National Science Foundation under the auspices of Task Committee F, "Disaster Prevention Methods for Lifeline Systems," UJNR Panel on Wind and Seismic Effects at the Biltmore Hotel, Los Angeles, California, United States on August 19-21, 1991.

2. The workshop provided a valuable forum to exchange information and ideas on technical and socioeconomic issues important to the lifeline earthquake engineering communities of both countries. The following topics were addressed:

   a. Soil Effects
   b. Seismic Design and Analysis: Transportation
   c. Seismic Design and Analysis: Pipelines and Tunnels
   d. Seismic Risk Analysis
   e. Post-Earthquake Response/Damage Detection
   f. Socioeconomic Impacts
   g. Future Directions for Research Application and Design.

3. The workshop identified, without priority order, the following topics relevant to lifeline systems for further cooperative research programs:

   a. Effects of Soils on Lifeline Components;

   b. Prioritization Procedures for the Seismic Inspection and Retrofit of Lifeline Structures;

   c. Use of Passive, Active, and Hybrid Control Techniques for Improving the Seismic Safety and Performance of Lifeline Structures;

   d. Dynamic Response and Design of Lifeline Structures;

   e. System Reliability Methods for Lifeline Systems;

   f. Repair and Rehabilitation of Lifeline Systems;

   g. Post-Earthquake Performance and Restoration of Lifeline Systems;

h. Post-Earthquake Damage Detection Procedures;

i. Socio-Economic and Environmental Impacts of Lifeline System Failures;

j. Emergency and Disaster Response Management of Lifeline Systems;

k. Information Dissemination on the Development of Seismic Performance Standards for Lifelines, Particularly Telecommunication Systems;

l. Use of Innovative Materials, Advanced Sensors and Intelligent Control Systems for Lifeline System Operations, Emergency Control, Damage Isolation, Replacement and Restoration;

m. Collection of Data (e.g., strong-motion response, laboratory testing and earthquake damage data) to Verify Methodologies and Models Used in Lifeline Earthquake Engineering; and

n. Lifeline Interaction Concerns, i.e., Dependency on Other Lifeline Services.

4. The workshop acknowledges current efforts in both countries to establish seismic design guidelines and standards for lifeline systems. Encouragement in the actual development of such standards should be placed on the U.S. effort. Existing UJNR channels should be fully utilized to facilitate the exchange of relevant technical information concerning standards development. Possible collaboration of joint development of lifeline system standards and guidelines should be pursued.

5. The importance of the "International Decade for Natural Disaster Reduction" (IDNDR) in its relation to lifeline earthquake engineering and disaster mitigation research under the UJNR program continues to be emphasized. Future joint activities between both countries in the area of earthquake disaster prevention for lifeline systems shall consider their contribution to the achievement of the goals of the IDNDR program. These future activities shall be defined by the Co-Chairmen of Task Committee "F" of the UJNR Panel on Wind and Seismic Effects.

6. The workshop recognizes the importance to continue exchange of research personnel, technical information, research data and use of available research facilities in both countries. The workshop further recognizes the benefit of identifying specific activities to compare the results of different design, analysis and experimental methods, and disaster mitigation strategies of both countries.

7. In view of the importance of cooperative research programs on seismic effects and earthquake disaster prevention methods on lifeline systems, it is recommended to hold the 5th Workshop in late 1992 in Japan. The location and time of the workshop will be determined through correspondence between the Co-chairmen of Task Committee "F" of the UJNR Panel on Wind and Seismic Effects.

IX. APPENDICES

WORKSHOP PROGRAM

WORKSHOP PARTICIPANTS

PHOTOS

FOURTH U.S. - JAPAN WORKSHOP ON

EARTHQUAKE DISASTER PREVENTION FOR LIFELINE SYSTEMS

AUGUST 19 - 21, 1991

BILTMORE HOTEL

LOS ANGELES, CALIFORNIA

WORKSHOP PROGRAM

MONDAY, AUGUST 19, 1991

8:30 a.m. - **Registration - Heinsbergen Room**

9:00 a.m. - **Session 1 - Opening Session**

Welcoming Remarks by Mr. R.T. Eguchi
Dames & Moore

Address by Dr. S.C. Liu
National Science Foundation

Address by Dr. Y. Sasaki, Director
Earthquake Disaster Prevention Department, PWRI

Address by Mr. R. Dikkers
Group Leader, Structural Evaluation, NIST

9:30 a.m. - **Session 2 - Soil Effects**

(Chairmen: J. Cooper, E. Kuribayashi)

9:30 U-1 "Performance of Water Supply Pipelines in Liquefied Soil"
(K. Porter, **C. Scawthorn,** M. Khater, T. O'Rourke)

9:45 J-1 "A Proposal of Practical Earthquake Response Analysis Method fo
Cylindrical Tunnels in Soft Ground"
(Y. Shiba, **S. Okamoto)**

10:00 J-2 "Visual Information System for Seismic Ground Hazard Zoning"
(K. Tokida, H. Matsumoto, **Y. Sasaki**)

	10:15	Discussion
10:30 a.m. -	**Break**	
10:45 a.m. -	**Session 3 - Seismic Design and Analysis: Transportation**	
	(Chairmen: H.S. Lew, T. Sato)	
	10:45 U-2	"Seismic Risk Identification and Prioritization in the CALTRANS Seismic Retrofit Program" (**B. Maroney**, J. Gates)
	11:00 U-3	"An Old Bridge gets a Seismic Facelift" (G. Snyder, E. Lindvall, **R. Lyons**)
	11:15 J-3	"Seismic Inspection and Seismic Strengthening of Highway Bridges in Japan" (**K. Kawashima**, S. Unjoh, H. Iida)
	11:30	Discussion
12:00 p.m. -	**Lunch - Corinthian Room**	
1:00 p.m -	**Group Photo**	
1:30 p.m. -	**Session 4 - Seismic Design & Analysis: Pipelines & Tunnels**	
	(Chairmen:	T. Lew, N. Nishio)
	1:30 U-4	"Repair and Rehabilitation of Buried Water and Sewer Lifelines" (**L. Wang**, H. Kennedy)
	1:45 J-4	"Investigations on External Force Evaluation in the Seismic Deformation Method" (**N. Takahashi**, M. Takeuchi, K. Irokawa)
	2:00 U-5	"Estimation of System Reliability for Lifeline Standards and Example Using the City of Everett, Washington Lifelines" (D. Ballantyne)
	2:15 J-5	"Dynamic Response of Twin Circular Tunnels during Earthquakes" (**T. Okumura**, N. Takewaki, K. Shimizu, K. Fukutake)
	2:30	Discussion

2:45 p.m. - **Break**

3:00 p.m. - **Session 5 - Seismic Design & Analysis: Pipelines & Tunnels**

(Chairmen: M. Celebi, M. Nakamura)

3:00 U-6 "Data Base of Pipeline Failures, Loma Prieta Earthquake, October 17, 1989"
(L. Lund)

3:15 J-6 "Behavior of Flat Underground Structures during Earthquakes and Seismic Loads Acting on Them"
(S. Mori, T. Ikeda, K. Matsushima, **H. Tachibana**)

3:30 J-7 "Seismic Response of Super-Deep Vertical Shaft with Circular Cross-Section"
(**N. Ohbo**, K. Hayashi, K. Ueno)

3:45 J-8 "Seismic Design Method of Shield Tunnels with Axial Prestressing by Means of Rubber and PC Bar"
(**K. Matsubara**, K. Urano)

4:00 J-9 "Seismic Isolation for Underground Structures"
(K. Ono, **S. Shimamura**, H. Kasai)

4:15 Discussion

5:00 p.m. - **Adjourn**

6:30 p.m. - **Reception at the Biltmore Hotel - Imperial Suite**

7:30 p.m. - **U.S. Hosted Dinner - Imperial Suite**

<u>TUESDAY, AUGUST 20, 1991</u>

8:30 a.m. - **Session 6 - Seismic Risk Analysis**

(Chairmen: H. Andress, N. Ohbo)

8:30 U-7 "Performance of AWSS of San Francisco During and After Loma Prieta Earthquake"
(**M. Khater**, C. Scawthorn, T.D. O'Rourke, F. Blackburn)

8:45 J-10 "Seismic Reliability Analyses of Large Scale Lifeline Networks Taking into Account the Failure Probability of the Components"
(**T. Sato**, K. Toki)

9:00 J-11 "Earthquake Damage Analysis on Telecommunication Conduits"
(K. Yagi, **S. Mataki**, N. Suzuki)

9:15 U-8 "Regional Risk Assessment of Environmental Contamination from Oil Pipelines"
(S. Pelmulder, **R. Eguchi**)

9:30 Discussion

9:45 a.m. - Break

10:00 a.m. - Session 7 - Post-Earthquake Response/Damage Detection

(Chairmen: H. Lagorio, S. Mataki)

10:00 J-12 "Data Acquisition and Emergency Response System for City Gas Pipeline Operation during a Major Earthquake"
(Y. Yoshikawa)

10:15 J-13 "Damage Inspection Systems Immediately after an Earthquake"
(**K. Kawashima**, H. Sugita, K. Kanoh)

10:30 J-14 "Estimation of Degree of Anxiety Felt by People in Underground Urban Spaces During Earthquakes"
(**E. Saito**, H. Ikemi, H. Nakano, M. Nakamura)

10:45 U-9 "Performance of Lifelines and Emergency Response in Watsonville, California to Loma Prieta Earthquake"
(J. Isenberg)

11:00 U-10 "Strategies for Repair and Restoration of Seismically Damaged Gas Pipeline Systems"
(M. Shinozuka)

11:15 Discussion

12:00 p.m - Lunch - Corinthian Room

1:00 p.m. - **Session 8 - Socioeconomic Impacts**

(Chairmen: J. Gates, E. Saito)

1:00 J-15 "Lifeline Earthquake Hazard Zonation in Socio-economic Aspects" **(E. Kuribayashi)**

1:15 U-11 "Organizational Features of U.S. Lifeline Systems and their Relevance for Disaster Management"
(K. Tierney)

1:30 U-12 "A Methodology for Assessing the Risk of Hazardous Materials Release Following Earthquakes - A Demonstration Study for the Los Angeles Area"
(**H. Seligson**, R. Eguchi, K. Tierney)

1:45 U-13 "Recent Advances in Seismic Design and Retrofit of California Bridges"
(J. Roberts)

2:00 Discussion

2:15 p.m. - **Break**

2:30 p.m. - **Session 9 - Future Directions for Research Application and Design**

(Chairmen: R. Eguchi, K. Kawashima)

2:30 J-16 Japan Perspective
(Y. Sasaki)

3:00 U-14 U.S. Perspective
(R. Dikkers)

3:30 p.m. - **Break**

4:00 p.m. - **Adoption of Resolutions**

5:15 p.m. - **Closure**

5:30 p.m. - **Adjourn**

7:00 p.m. - **Japan Side Hosted Dinner - The Castaways**
(Bus leaves in front of Biltmore Hotel at 6:30 p.m.)

WEDNESDAY, AUGUST 21, 1991

8:30 a.m.	Bus arrives at Biltmore for start of Field Trip. All who are going onto the ASCE Conference should be downstairs with their luggage at this time.
9:00 a.m.	Bus leaves.
9:30 a.m. - 11:00 a.m	Tour of the Ralphs Grocery Company Automated Warehouse
11:15 - 11:45	Tour of the Retrofitted Bridge Structure of the 134 and 2 Freeways
12:00 - 1:p.m.	Lunch at the Odyssey Restaurant
1:30 p.m. - 3:30 p.m.	Tour of the Jensen Filtration Plant
4:15 p.m.	Tour of the Retrofitted Bridge Structure of the 405 and 10 Freeways (if time allows)
5:00 p.m.	Bus drops ASCE participants at Holiday Inn Brentwood/Bel Air
5:45 p.m.	Bus stops at the Miyako Hotel
6:00 p.m.	Bus returns to the Biltmore Hotel

PARTICIPANTS FOR THE 4TH U.S.-JAPAN WORKSHOP ON LIFELINE EARTHQUAKE ENGINEERING (FROM JAPAN SIDE)

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Photo 1

Photo 2

Photo 3

Photo 4

Photo 5

Photo 6

Photo 7

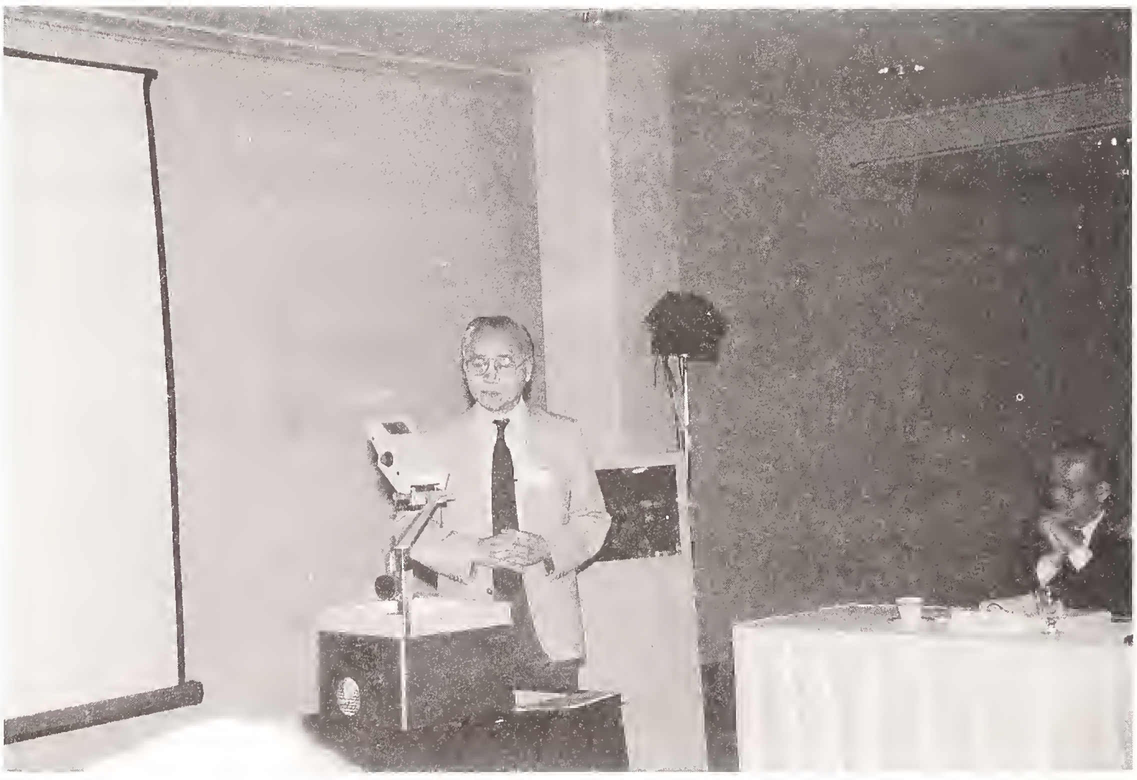

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Photo 11

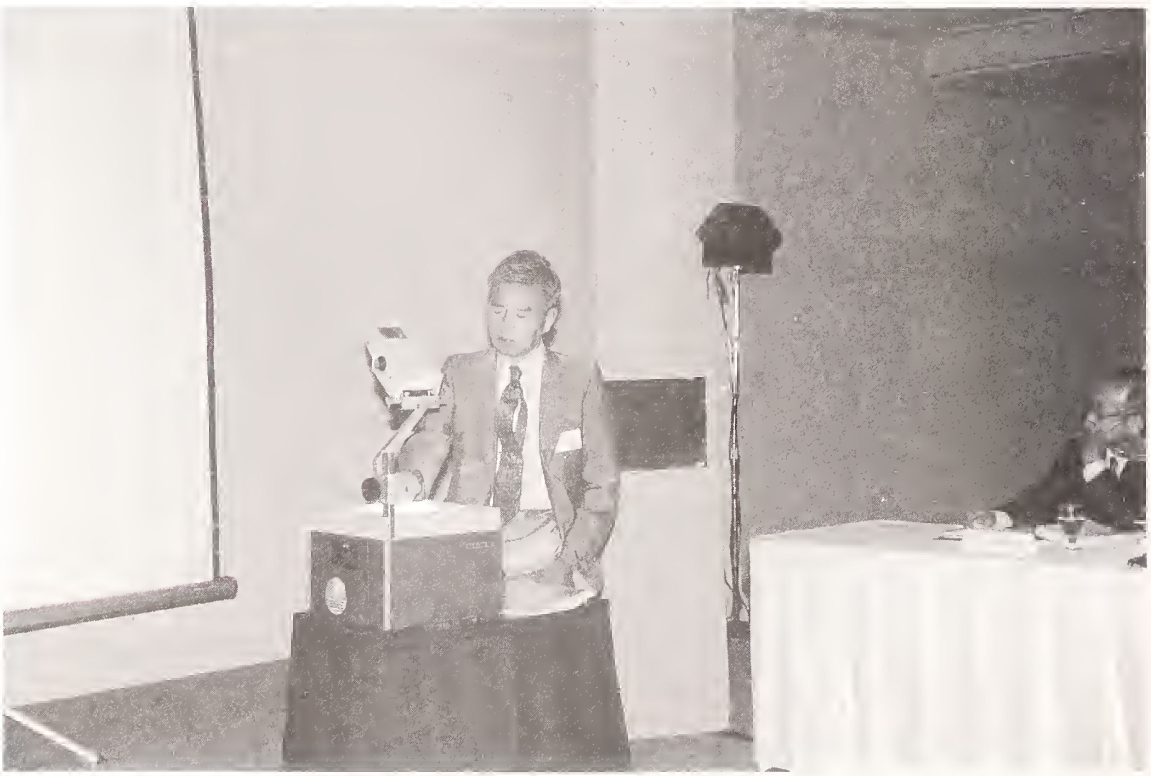

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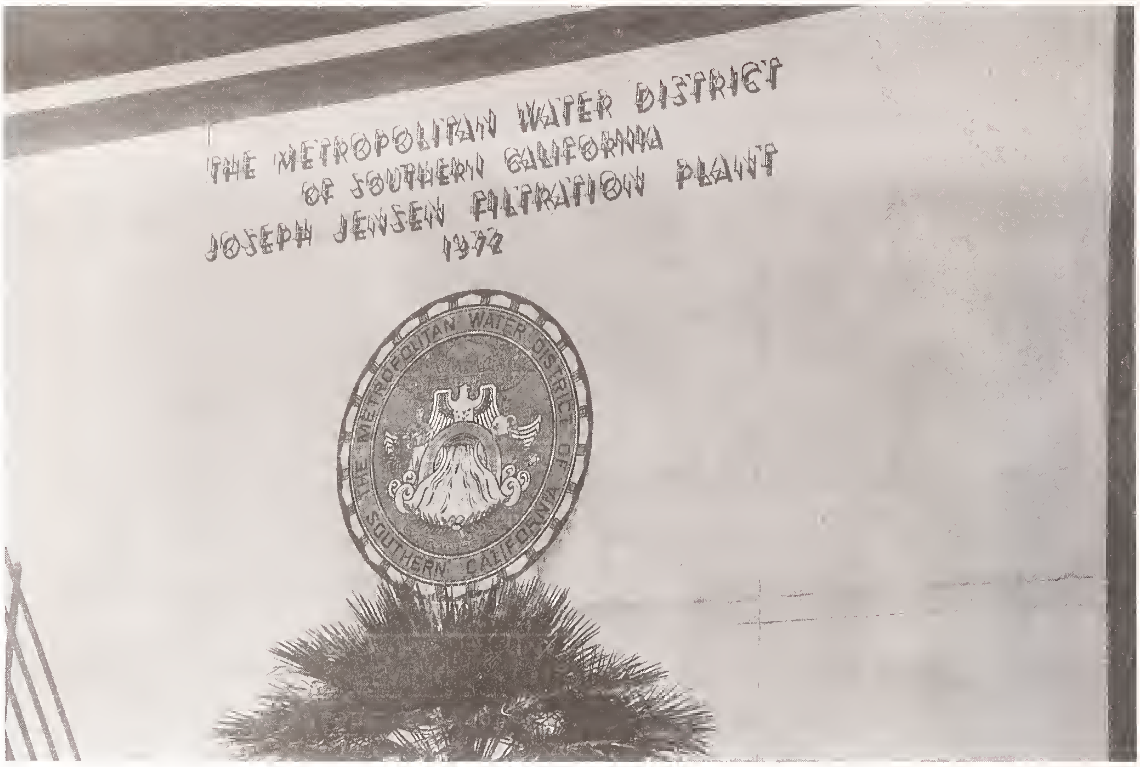
Photo 14



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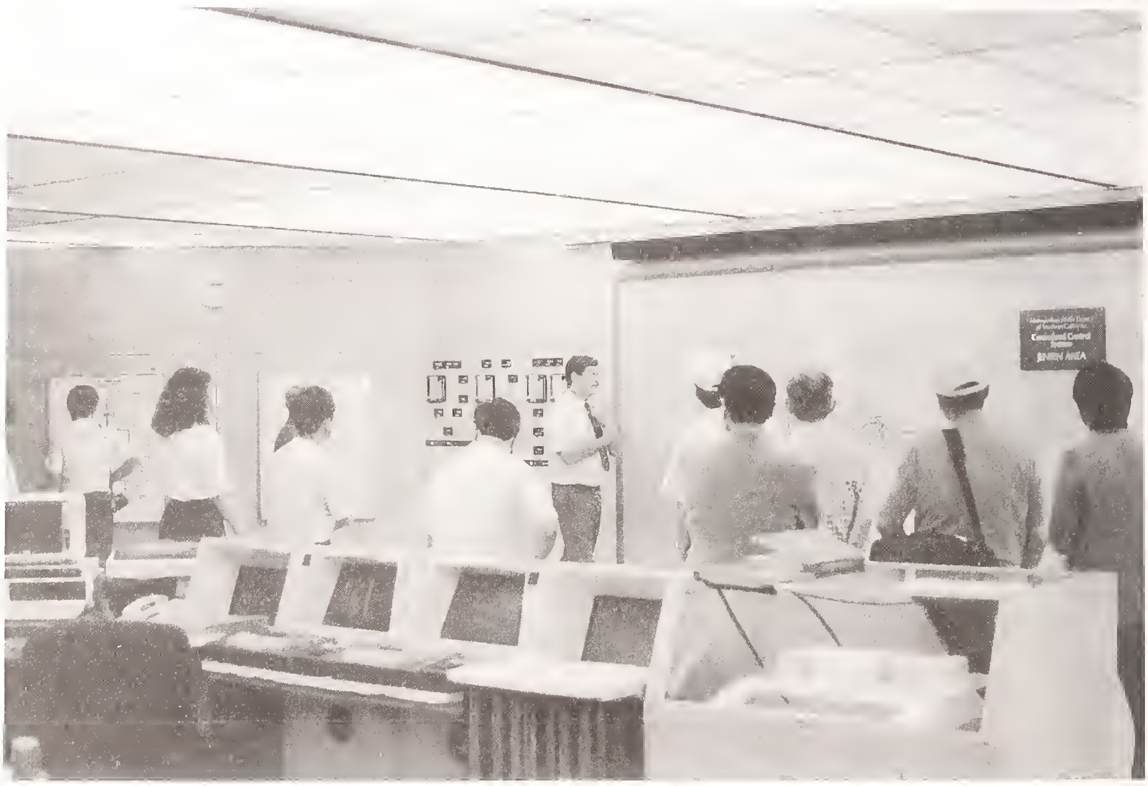

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Photo 19

Photo 20

Photo 21

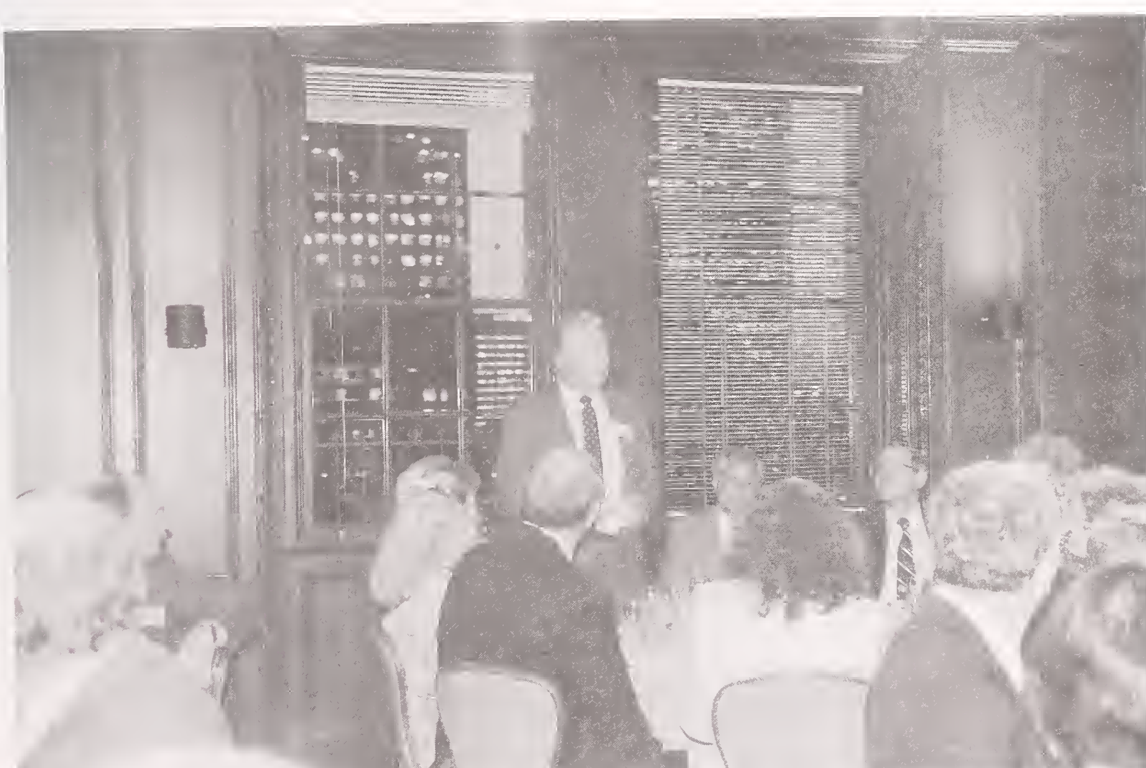

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Photo 27

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Photo 29

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These proceedings document the results of the Fourth U.S.-Japan Workshop on Earthquake Disaster Prevention for Lifeline Systems held on August 19-21, 1992, in Los Angeles, California. The theme of the workshop focused on "Future Directions for Research, Application, and Design of Lifeline Systems." Technical topics discussed include: effects of soils on lifeline components; seismic design and retrofit of lifeline systems; dynamic response and analysis of lifeline systems; repair and rehabilitation of lifeline systems; system reliability methods for lifeline systems; post-earthquake damage detection procedures; socioeconomic and environmental impact of lifeline system failure; and emergency and disaster response management of lifeline systems. Thirty papers were presented in two days of plenary sessions; 16 papers from Japan and 14 papers from the U.S.

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