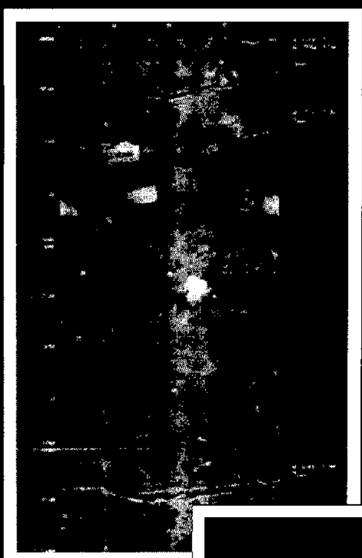
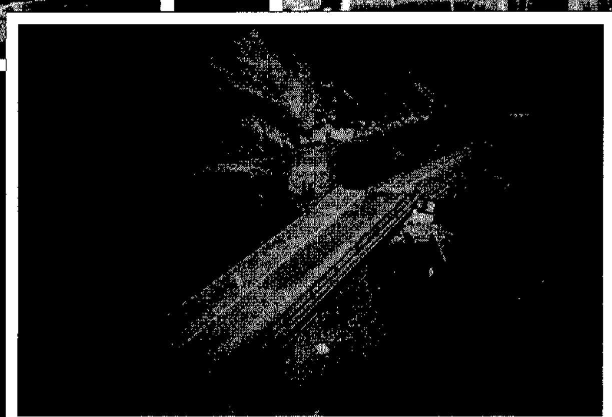
Proceedings of the U.S.-Japan Joint Seminar on Civil Infrastructure Systems





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This workshop was conducted at the Royal Garden Hotel, Honolulu, Hawaii, and was supported by the National Science Foundation and the Japan Society for Promotion of Science with supplementary non-federal support by the Multidisciplinary Center for Earthquake Engineering Research.

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16. Abstract (limit 200 words)

A two and one-half day Bilateral Seminar on Civil Infrastructure Systems (CIS) Research was held on August 28-30, 1997, under the joint sponsorship of the U.S. National Science Foundation and the Japan Society for Promotion of Science, with supplementary support by the Multidisciplinary Center for Earthquake Engineering Research. This seminar provided a forum for identifying and comparing common CIS issues in the U.S. and Japan, exchanging ideas on solutions, promoting cooperative research between the two nations, and formulating action plans, all in the frontier areas of CIS research involving the following main themes of study: 1) Aging and deterioration; 2) Health monitoring and conditioning assessment; 3) Renewal engineering; 4) Socioeconomic issues, including institutional effectiveness and productivity; and 5) Research coordination. The seminar consisted of five plenary technical sessions addressing the five themes, two break-out workshop sessions and three plenary sessions for the development and adoption of a working group report, resolutions and recommendations. Executive sessions dealt with administrative needs and facilitated communication among conference and session chairs in developing the resolutions and recommendations. Each of the twenty-eight participants from the U.S. and Japan presented a paper on the CIS issues of his/her expertise. Four papers concern concrete structures. Two concern steel bridges. Five papers treat health monitoring and condition assessment of civil infrastructure systems, and three papers consider the rehabilitation of civil infrastructure The remaining papers treat economic issues, sustainability, research coordination, and the varied of government, academic and industry on CIS issues.

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Proceedings of the U.S.-Japan Joint Seminar on Civil Infrastructure Systems Research

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Edited by Masanobu Shinozuka[1] and Adam Rose[2]
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Executive Summary

The United States and Japan share common problems of civil infrastructure systems (CIS), such as physical aging and deterioration, functional obsolescence, high maintenance costs, and huge outlays for renewal and upgrading. Both nations urgently need cost-effective strategies for the planning, design, construction, maintenance and retrofit of their respective CIS to enhance and sustain their current economic prosperity into the 21st century, and to further promote their respective competitiveness consistent with the principles of a free and fair global economic system.

A two and one-half day *Bilateral Seminar on Civil Infrastructure Systems (CIS) Research* was held on August 28-30, 1997, under the joint sponsorship of the U.S. National Science Foundation (NSF) and the Japan Society for Promotion of Science (JSPS), with supplementary support by the Multidisciplinary Center for Earthquake Engineering Research from non-federal sources. The objective of this seminar was to provide a forum for identifying and comparing common CIS issues in the U.S. and Japan, exchanging ideas on solutions, promoting cooperative research between the two nations, and formulating action plans, all in the frontier areas of CIS research involving the following main themes of study:

1. Science of aging and deterioration
2. Health monitoring and condition assessment
3. Renewal engineering
4. Socioeconomic issues, including institutional effectiveness and productivity
5. Research coordination

The seminar consisted of five plenary technical sessions addressing the five themes, two break-out workshop sessions and three plenary sessions for the development and adoption of a working group report, resolutions and recommendations, in addition to the opening and closing sessions. Executive sessions dealt with administrative needs and facilitated communication among conference and session chairs in developing the resolutions and recommendations. Each of the twenty-eight participants from the U.S. and Japan presented a paper on the CIS issues of his/her expertise.

This volume contains the proceedings of the seminar and consists of two parts. Part I contains seminar recommendations for potential future cooperative U.S.-Japan research projects on CIS research, and Part II contains papers presented in the seminar. M. Shinozuka and A. Rose, in cooperation with M. Watabe and M. Yoshimura, co-edited this volume. The seminar agenda, list of participants, and resolutions from the five working groups are included in the appendices.

Participants of the U.S.-Japan Joint Seminar on Civil Infrastructure Systems Research

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Part I: Seminar Recommendations

Background

Civil infrastructure systems (CIS) provide society's basic ability to transport people and goods, deliver clean water, electric power, natural gas and liquid fuel, facilitate communications, mitigate the impact of natural disasters, and serve many other functions for the enhancement of the welfare and freedom of the general public. Indeed, this century has seen the United States and Japan investing heavily in highways, airports, canals, dams, embankments, mass transit, telecommunication and other systems. However, acceleratingly rapid growth of societal activities and the resulting demand for CIS over this period is exceeding the rate of new construction and retrofitting of existing CIS. This is generally true everywhere in the United States as well as in Japan, particularly in the urban population centers, where increasingly excessive demand, misuse and lack of funds for maintenance, retrofit and new construction are exacerbating the stress on CIS in the U.S. In addition, recent natural disasters, such as the repetitious floods in the Mississippi River basin, Hurricane Andrew and Northridge earthquake in the United States, and ubiquitous typhoon-related disasters and the 1995 Hyogoken Nanbu (Kobe) earthquake in Japan, demonstrated the fragility of CIS and the devastating degree of socioeconomic and human losses their failures can bring. These trends are common in most industrial nations. Particularly in this respect, the United States and Japan share common problems of CIS such as physical aging and deterioration, functional obsolescence, high maintenance costs, and huge outlays for renewal and upgrading. Both nations are in urgent need of cost-effective strategies for planning, design, construction, maintenance and retrofit of their respective CIS in order to enhance and sustain current economic prosperity into the 21st century, and to further promote their respective competitiveness consistent with the principles of a free and fair global economic system.

In accepting this challenge, the U.S. National Science Foundation (NSF) has developed its institutional strategy in two recent publications entitled *Civil Infrastructure Systems (CIS) Strategic Issues*, NSF 94-129, and *Civil Infrastructure Systems: An Integrative Research Program*, NSF 95-52. These publications indicate NSF's support for CIS research and education initiatives to achieve the following goals: (a) enrichment of the science and engineering base; (b) integration, transfer and utilization of knowledge; and (c) education and training of the next generation of engineers, scientists and other professionals with multidisciplinary emphasis, all for the purpose of enhancing and renewing the state of CIS.

In this context, the seminar addressed and identified these and other fundamental issues of CIS that can most cost-effectively be resolved to the benefit of both countries by carrying out bilateral joint research, as well as synergistically combining and complementing the intellectual, institutional and financial resources of both countries. Furthermore, the seminar identified research institutions and organizations together with their researchers capable of carrying out the CIS research pertinent to the joint effort. The CIS research by its nature must encompass a broad spectrum of subject matters that are scientific, technological, socioeconomic and policy-related. Therefore, multidisciplinary and multi-institutional approaches are essential in addressing and resolving the complex CIS issues of modern society. In particular, the research must be well coordinated among traditionally less cohesive academic, industry and government institutions to maximize the return of the societal investment in CIS research. New funding sources must also be found from among government and private sectors that would not ordinarily participate in the narrowly defined traditional research agenda involving CIS.

This seminar is very timely in the wake of the U.S./Japan common agenda announcement of April 1996 between President Clinton and Prime Minister Hashimoto, in which earthquake disaster mitigation was added as one item to be pursued jointly under the common agenda program. Since the issue of earthquake disaster mitigation is also part of CIS problems, the seminar was cognizant of this and developed strategies for the purpose of this agenda item as well.

Objectives

The objective of this seminar was to provide a forum for identifying and comparing common CIS issues in the U.S. and Japan, exchanging ideas on solutions, promoting cooperative research between the two nations, and formulating action plans, all in the frontier areas of CIS research, involving the following main themes of study:

1. Science of aging and deterioration
2. Health monitoring and condition assessment
3. Renewal engineering
4. Socioeconomic issues, including institutional effectiveness and productivity
5. Research coordination

The seminar consisted of five plenary technical sessions addressing these five themes, two break-out workshop sessions, and three plenary sessions for the development and adoption of working group reports, resolutions and recommendations, in addition to the opening and closing sessions. Executive sessions dealt with administrative needs and facilitated communications among conference and session chairs in developing the resolution and recommendations. Each of the twenty eight participants from the U.S. and Japan presented a paper on the CIS issues of his/her expertise.

This volume represents the proceedings of the seminar and includes: seminar resolutions, recommendations for the development of future cooperative U.S.-Japan research projects on CIS research, and the presented papers. M. Shinozuka and A. Rose, in cooperation with M. Makoto and M. Yoshimura, co-edited this volume.

Working Group Goals and Scope of Discussion

Goals

As a major function of the seminar, working group sessions played a significant role in achieving the seminar objectives, including selection of high priority cooperative research areas. Five working groups were organized in accordance with five themes identified in the Objectives. The chair of each working group was requested to coordinate, with proper emphasis, the discussion within the group to address the objectives of the seminar and to produce a written report to be presented at the plenary session set aside for that purpose.

In selecting high priority cooperative research areas, it was suggested that the following criteria be used:

1. Complementary nature of cooperative research;
2. Fundamental research value as university-based research in cooperation with government and industry;
3. Impact and utility of research results;
4. Multidisciplinary and integrative nature of research;
5. Urgency of research results;
6. Budgetary requirement of leveraging of funds;
7. Compatibility with the Clinton-Hashimoto common agenda.

Each working group also was requested to make comments on the mechanism that would facilitate and support cooperative research in, among others, the following areas:

1. Exchange of research personnel on a long or short term basis;
2. Sharing of experimental, computational and other facilities unique to each country;
3. Development of an information clearinghouse to collect and effectively disseminate the cooperative research results;
4. Identification of potential funding resources for cooperative research projects.

Scope of Discussion

Working Group 1	Science of Aging and Deterioration
Chairs:	S.P. Shah and K. Takanashi
	· Physics, chemistry and micromechanics of deterioration process
	· Integration of material science, structural engineering and system performance
	· Uncertainty in deterioration process
Working Group 2	Health Monitoring and Condition Assessment
Chairs:	R. Eguchi and A. Mita
	· Damage definition
	· Damage measurement
	· Damage index
	· Health monitoring and condition assessment
Working Group 3	Renewal Engineering
Chairs:	J. Roberts and K. Yokoyama
	· Vulnerability analysis for all natural hazards
	· Renewal cost and life cycle cost reduction
	· Performance-based design for CIS renewal
	· Optimum renewal strategies
	· Large scale testing
	· Advanced materials
Working Group 4	Socioeconomic Issues, Institutional Effectiveness & Productivity
Chairs:	W. Petak and R.Shimada
	· Sustainability
	· Engineering, management and political processes for CIS decisions
	· Life cycle planning
	· Strategic CIS development and renewal planning
Working Group 5	Research Coordination
Chairs:	R.N. Wright and I. Okawa
	· University-government and industry partnerships
	· Use of existing coordination mechanisms
	· University education

High Priority Cooperative Research and Activity Areas

In consideration of on-going, planned and potential research projects and activities for the enhancement of infrastructure performance, effectiveness and productivity, the participants identified the following research and activity areas within each of the five major themes of the workshop. These research and activity areas will expand the knowledge base not only relevant to the built environment in general and infrastructure systems in particular, but also to integrated assessment procedures relaying the engineering performance information on these systems to the broader socioeconomic and policy issue constituents for their decision support activities. More specific research and activity areas are identified and listed in the working group reports contained in Appendix D.

Science of Aging and Deterioration

The very basic physical issue that will influence all phases of the life of infrastructure systems (planning, design, construction, maintenance, and operation) is the fact the system physically ages and deteriorates. At this time, unfortunately, the physics of aging and deterioration is not well understood in spite of the paramount importance of this issue. The following research areas are identified as having high priority toward the resolution of the issue:

1. Integration of physics, chemistry, and micromechanics of degradation process.
2. Synthesis of material science, structural engineering and system performance (micro-meso-macro scale) for predicting life cycle performance of infrastructure systems.
3. Development of reliability analysis methodologies accounting for uncertainty involved in the aging and deterioration process.

Health Monitoring and Condition Assessment

A growing consensus exists on the need for an enhanced capability in health monitoring and condition assessment among the parties responsible for controlling the life cycle cost of infrastructure systems, while maintaining acceptable levels of safety and functionality throughout the life of the systems. In fact, health monitoring and condition assessment technology is emerging as a critical subject of research for infrastructure systems in order to continually ensure system safety and functionality. More importantly, however, health monitoring and condition assessment technology, sufficiently enhanced particularly with the aid of rapid development of related advanced technologies, can bring a paradigm shift in the practice of civil structure design by providing the profession with the concepts and procedures of damage tolerance design. Indeed, this shift, while it may take some time, is consistent with the current transition that prevails in the profession moving from response-based design to performance-based design.

The high priority research areas are:

1. Development and synergistic use of advanced sensors and NDE techniques.
2. System identification techniques to correlate sensor data with physical damage.
3. Real-time assessment of damage caused by earthquake and other natural forces.

4. System integration to manage infrastructure facilities with health monitoring and condition assessment as an integral part.
5. Damage tolerance design for civil infrastructure systems.

Renewal Engineering

Renewal of existing civil infrastructure systems entails a continued and sustained effort of national magnitude, requiring an enormous amount of human and financial resources. Politically and economically viable strategic plans must be developed and implemented for this purpose especially being mindful of sustainability of renewed environments. Renewal engineering will primarily deal with technological components of this effort with the following high priority research areas:

1. Improvement of vulnerability analysis methods of civil infrastructure systems for all natural and technological hazards, including fire hazards, to provide planners and political leaders with information to make effective renewal decisions.
2. Evaluation of renewal costs versus potential life cycle cost reduction.
3. Development of performance-based civil infrastructure systems renewal procedures.
4. Large scale testing of civil infrastructure systems to the extent that all states of performance are reproduced in the test.
5. Use of advanced materials, structural systems, and construction methods for rehabilitation and reconstruction.

Socioeconomic Issues, Institutional Effectiveness and Productivity

For the purpose of developing, maintaining and renewing the infrastructure systems within the framework of sustainability, the research community must urgently address socioeconomic issues beyond engineering endeavors to improve on and take advantage of science of aging and deterioration, health monitoring, condition assessment, and renewal engineering. In this connection, the following research areas are identified to have high priority:

1. High level systems analysis to develop perspective for understanding the socioeconomic issues in CIS.
2. Definition of sustainable development with respect to civil infrastructure systems that establish guiding principles and decision criteria.
3. Identification of values and perspectives of stakeholders, such as owners, government, industry and citizens, having an interest in the status of civil infrastructure systems.
4. Documentation of experience in retrofit and reconstruction to provide the understanding for political processes involved.
5. Development of a systems architecture to provide a socioeconomic and political framework of civil infrastructure systems for engineering.

Research Coordination

Efficient research coordination is critically needed to maximize the utility of available financial and human resources for the civil infrastructure systems research, since the research must be carried out under cross-disciplinary, multi-institutional, multi-cultural and international cooperation. While the civil infrastructure research community has been cognizant of this necessity and some effort has been made in this direction, it is recommended that the effort be expanded to achieve more streamlined and enhanced coordination.

The following avenues of action are identified as highly promising to accomplish the purpose:

1. Development of more effective partnerships of government, industry and academia to respond in a timely way to civil infrastructure systems issues.
2. Expansion of successful U.S.-Japan partnerships such as UJNR Panel on Wind and Seismic Effects to address the research recommendation of this seminar.
3. Reinvention of educational and research programs in universities to respond to capabilities of information technologies and needs for the leadership with cross-disciplinary and international perspectives in decision making in academia, industry and government.

Acknowledgments

The *U.S.-Japan Joint Seminar on Civil Infrastructure Systems Research* was held at the Royal Garden Hotel in Honolulu, Hawaii, on August 28-30, 1997. Thirty-one U.S. and Japanese researchers from universities and industries, and government officials, participated in this seminar.

The financial support of the U.S. National Science Foundation and the Japan Society of Promotion of Science is gratefully appreciated. The supplementary support by the Multidisciplinary Center for Earthquake Engineering Research (formerly the National Center for Earthquake Engineering Research) from non-federal sources is also acknowledged.

The seminar would not have been possible without the leadership and advice of Drs. Hiroyuki Yoshikawa and Ken Kikuchi at the Japan Society of Promotion of Science and of Drs. S.C. Liu, C. Astill and W. Anderson of the Earthquake Hazard Mitigation Program and of Dr. L. Weber of the International Program of the National Science Foundation.

It is acknowledged that Ms. A. Dargush, Assistant Director of the Multidisciplinary Center for Earthquake Engineering Research (MCEER) provided most efficient logistic support for the seminar and Ms. J. Stoyle, of MCEER, most skillfully assisted the seminar coordinators in editing the proceedings.

Part II: Workshop Papers

DETERIORATION OF CONCRETE SUBJECTED TO COMPRESSION AND TENSION

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INTRODUCTION

The Center for Advanced Cement-Based Materials (ACBM) was established by the U.S. National Science Foundation to advance the basic science of cement and concrete. As a part of this research effort the deterioration of concrete and fiber reinforced cement-based composites have been studied in both compression and tension. This paper presents a brief summary of the development and/or adaptation of a few of NDE techniques which are essential to obtain basic fracture properties of concrete.

FRACTURE IN CONCRETE

Concrete can be referred to as a "quasi-brittle" material due to the numerous microstructural features which absorb energy during crack growth. This energy absorption manifests itself in the nonlinear stress-strain behavior as well as the strain softening post-peak behavior. The microstructural mechanisms which control the fracture behavior of concrete are often referred to as the fracture process zone. The fracture process zone acts in the vicinity of a macrocrack front. It is characterized by microcracks, particle interlocking, and other inelastic, irreversible material changes which dissipate energy. A substantial global research effort has produced much evidence which supports the nature and effects of the fracture process zone [1].

Experimental and analytical studies conducted over the last 30 years have done much to advance understanding of concrete fracture. The general stress-strain behavior of concrete can be explained in terms of microcracks and macrocracks. Typically microcracks form at relatively low stresses. As the stress increases, the microcracks tend to localize into larger visible cracks (macrocracks) that ultimately grow until failure. Some of the fracture issues that remain unresolved include the stresses at which microcracking begins, stresses at which microcracks localize into macrocracks, and the nature of microcracks in the process zone. Laboratory NDE techniques have been invaluable in addressing these and other fracture issues.

Nondestructive evaluation of concrete has its own inherent difficulties. The heterogeneous nature of concrete makes it difficult to examine it with some of the more traditional techniques. The heterogeneous structure of the material spans a large number of length scales, making it difficult to tune a technique to a wider range of scales. Microcracks can be the same size as pores. The NDE techniques must be extremely sensitive because the fracture processes begin at relatively small loads. A number of NDE techniques have been applied in ACBM sponsored research in fracture processes in cement and concrete. Each of the techniques used have their own particular strengths and weaknesses. These techniques may be broadly classified into surface observations and internal observations. The surface observations include laser holography, moiré interferometry, computer vision, and electronic speckle pattern interferometry (ESPI). The internal observations include acoustic emission (AE) and more recently x-ray microtomography (XMT).

FRACTURE IN CONCRETE UNDER TENSION

Acoustic Emission

For the evaluation of the properties of fracture process zone, the monitoring of acoustic emissions can be a useful technique. Since acoustic emissions in cement-based materials result primarily from microcracking and other fracture processes, AE signals can be used to examine the fracture process zone directly. A number of different AE experiments have been conducted over the last few years. The goals of these experiments included: 1) the understanding of materials behavior based on micromechanical phenomena, 2) the characterization of the mechanisms for damage evolution, and 3) the establishment of a fundamental basis for nondestructive evaluation of concrete. Towards these goals one set of experimental results are described here.

A series of concrete specimens were fabricated and loaded in uniaxial compression. A new method of experimental control was used for stable control of crack growth [2]. As shown in Figure 1, a series of four LVDT displacement gages were mounted on the specimen. The gage which showed the greatest displacement was used as the feedback controller in a closed-loop loading setup. As long as localization occurred within the gage length of one of the four transducers, stable crack growth could be assured.

Four AE transducers were attached to the specimen as shown in Figure 1. Since the specimen was thin compared to its length and width, all analysis was performed in two dimensions. The experimental program consisted of a number of different types of plain (unreinforced) and fiber reinforced concrete specimens. Only the results of the plain concrete are discussed here. The load-deformation behavior for one of the concrete specimens is shown in Figure 2. The deformation was measured by the appropriate LVDT.

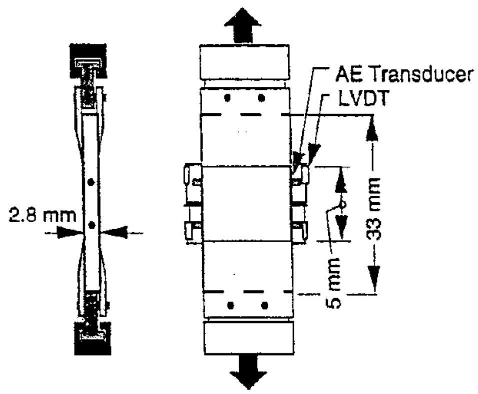


Figure 1. Tension specimen geometry

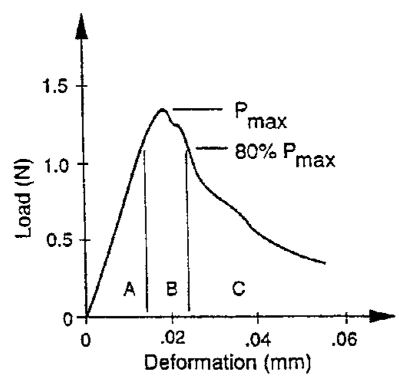
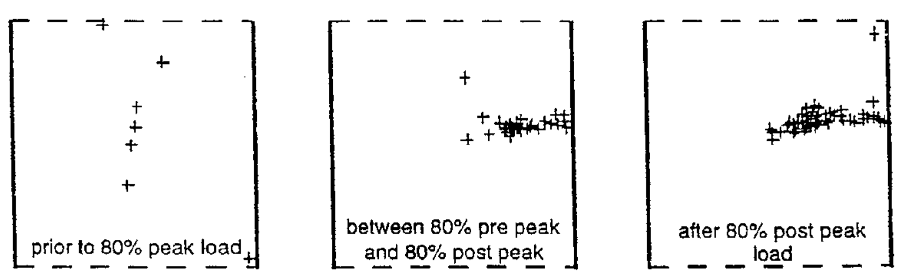


Figure 2. Load vs. displacement



(a) Stage A (b) Stage B (c) Stage C

Figure 3. Locations of AE events in uniaxial tension specimen

The location of the AE sources was determined for each recorded AE event. A least-squares method was used to estimate the location from the different signal arrival times [3]. The locations of the AE sources are shown in Figure 3 for three different stages of the test. The three stages are labeled as A, B and C in Figure 2. Stage A is the initial loading of the specimen. Stage B is the area around the peak load, and Stage C is the softening region of the curve. As can be seen in Figure 3(a) the locations of the AE sources tend to be somewhat randomly distributed through the specimen. At about 80% of peak load the AE sources along a band on the right-hand side of the specimen as illustrated in Figure 3(b). This corresponds to the localization of microcracking into a single critical crack. All subsequent AE sources (in the post peak region) occur along this band as shown in Figure 3(c), indicating that strain softening corresponds to growth of a single critical crack.

The results of this experiment show that AE techniques could be used to observe localization phenomena in concrete specimens. The AE locations showed that localization in this specimen occurred at roughly 80% of the peak load. In summary, acoustic emission techniques have been shown to be a valuable tool for inferring the basic fracture properties of cement-based materials. Microcrack localization phenomena could be followed using the AE source location data. Characteristics of individual microcracks were evaluated using quantitative AE analysis techniques.

Two Dimensional Electronic Speckle Pattern Interferometry

The advantage of acoustic emission techniques is the ability to obtain damage data in three dimensions. The limitation is that the spatial resolution of the technique is limited. An experimental method which compliments AE analysis is electronic speckle pattern interferometry (ESPI) which can be used to accurately detect microcrack formation at the surface of the specimen. While ESPI analysis is limited to the specimen surface, the resolution is detailed such that microcracks on the order of 0.25 micron openings can be detected.

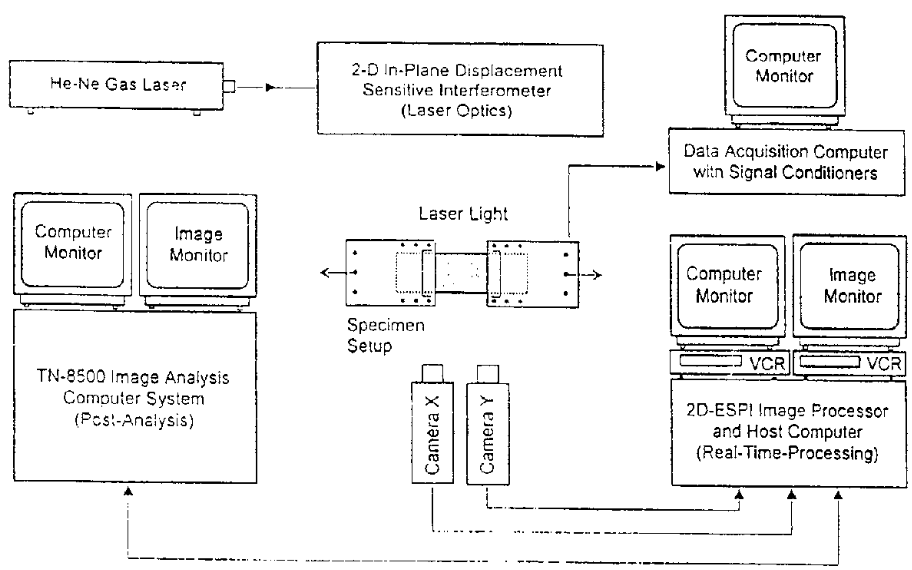


Figure 4. Experimental setup for 2D ESPI

The 2D ESPI system was developed to measure two-dimensional in-plane displacements. The system was capable of real-time monitoring and continuous recording. Figure 4 shows the 2D ESPI system which is a combination of optics and electronics. Details of the optical setup and data processing are presented elsewhere [4]. Some of the experimental results are summarized here.

A carbon fiber reinforced mortar specimen was loaded in uniaxial tension. The load-displacement plot is shown in Figure 5. As the specimen was being loaded, it was monitored by the 2D ESPI system of Figure 4. Throughout the test, the interference patterns (fringes) were recorded by the two CCD cameras on a 101x76 mm area of the specimen. After the test, the fringe patterns were analyzed for discontinuities in the fringe patterns. Any discontinuity represents an active crack.

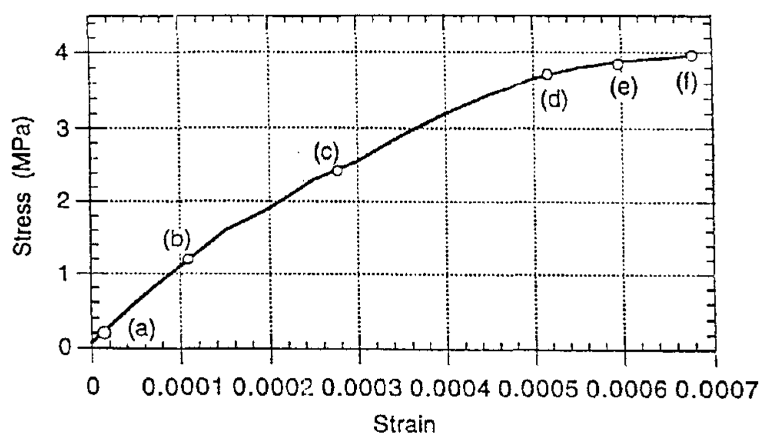


Figure 5. Stress vs. strain for carbon fiber reinforced mortar

Fringe patterns at various loading stages are shown in Figure 6. Each of these patterns corresponds to a load as shown on Figure 5. In Figure 6(a) the fringe pattern is continuous indicating there are no active cracks on the specimen surface. Figure 6(b) shows the first discontinuity in the fringe pattern and thus the first active crack. It should be noted that this crack appears at only 1.11 kN or about 29% of the peak load. This highlights the fact that damage begins to accumulate very early in the loading stage of mortars. As additional load is applied, the initial crack grows. In addition, more cracks form at different locations on the specimen as shown in Figure 6(c). It should also be noted that the addition of fibers to the cement matrix effectively forces a greater distribution of cracking in the specimen. Fibers tend to bridge and stop existing cracks, causing additional load to be distributed elsewhere in the material. This load redistribution is spread throughout the specimen until it reaches a maximum as shown in Figure 6(d). Beyond this point, the additional energy must be absorbed by growth of the largest crack. Figure 6(e) shows the growth of the largest crack, and Figure 6(f) shows the active cracks at the peak load. The term "active cracks" here refers to those that grow between successive points on the load-deformation curve. The large number of cracks shown in Figure 6(d) do not disappear, but in fact do not grow beyond that point.

The strength of the ESPI analysis described here is that it is extremely sensitive to the discontinuities in the displacement field caused by cracks. The crack activity was plainly visible, and could easily be correlated with load data to get a detailed picture of damage evolution and localization in the fiber-reinforced mortar specimen. The three stages of damage evolution: distributed cracking, localization, and critical crack propagation, were clearly seen. The ESPI data thus agrees with the results of the acoustic emission experiments in localization.

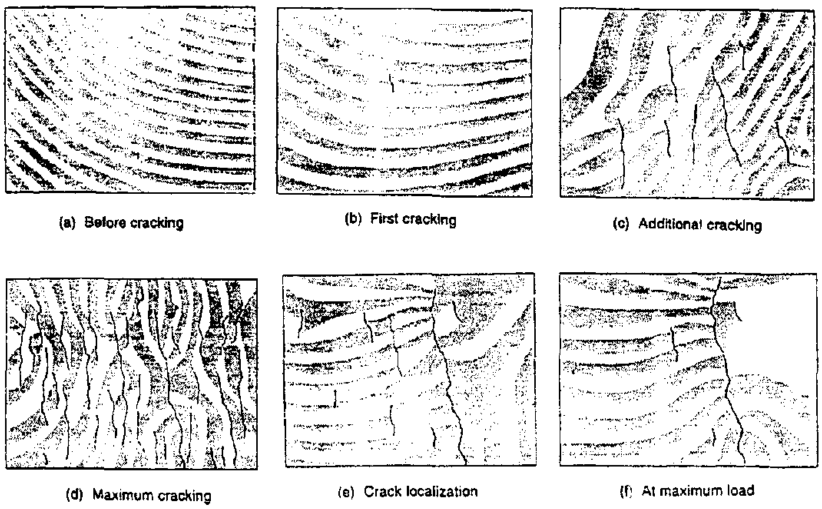


Figure 6. 2D ESPI fringe images with crack patterns

FRACTURE IN CONCRETE UNDER COMPRESSION

Computer Vision

Detailed information about crack development is essential for the study of complex fracture behavior in cement-based materials under compressive loads. Measurement of deformation fields using digital image correlation (DIC) or computer vision utilizes the brightness intensity of specimen surface images [5]. The displacement of an image subset at different loading steps can be computed using a correlation coefficient which returns higher value when two subimages are closer each other. Measurement of crack failure development based on DIC carries some important advantages over other methods. First of all, DIC does not have technical limitations in dealing with multiple cracks which are common in cement-based materials under compression. Second, since gages are not attached to the specimen, measured values would not be disturbed by undesirable crack development. Precise examination is possible based on the full-field measurement data over a reasonably large specimen size. Another important thing is that DIC method can be used at the hydraulic testing machine which often generates vibration problem for some sensitive techniques. Automated measurement routine also reduces processing time a lot for a large quantity of specimens. The measurement accuracy strongly depends on the testing conditions and image quality. The average and standard deviation of errors were estimated as about 0.014 pixels, 0.012 pixels, respectivley [6].

It has been known that shear confinement from the loading platens introduces barrel effect [7]. A series of experimental work was conducted at Northwestern University with different material compositions such as cement paste, mortar, and concrete. In these tests, the barrel effect was clearly observed using computer vision method, and it was found that the end constrains and barrel effect can be removed by inserting friction-reducing materials. Crack patterns in cement paste, mortar, and concrete do not reveal shear band failure (Figure 7). For concrete, cracks were interacted with aggregate interfaces showing more complex patterns. In all cases, cracks were essentially vertical. The magnitude of crack opening was larger in cement paste than in concrete. On the other hand, the number of cracks formed near peak load were more in concrete than in cement paste or mortar.

The effect of aggregate spacing to the failure mode in concrete was also examined using model concretes which contain different number of aggregates (1, 5, and 13 aggregates). For this experiment, automated scanning stages are equipped together with computer vision setup to increase the accuracy by zooming in smaller subregion. Figure 8 shows the assembled lateral displacement contour maps and each line represents 2 microns of deformation. The specimen which contains 13 aggregates did not form the same failure pattern observed in 5 aggregate case. Instead it revealed localized fracture.

In summary, computer vision (or DIC) method turned out to be suitable for obtaining the full-field deformations on a specimen which carries a complicated compressive failure pattern.

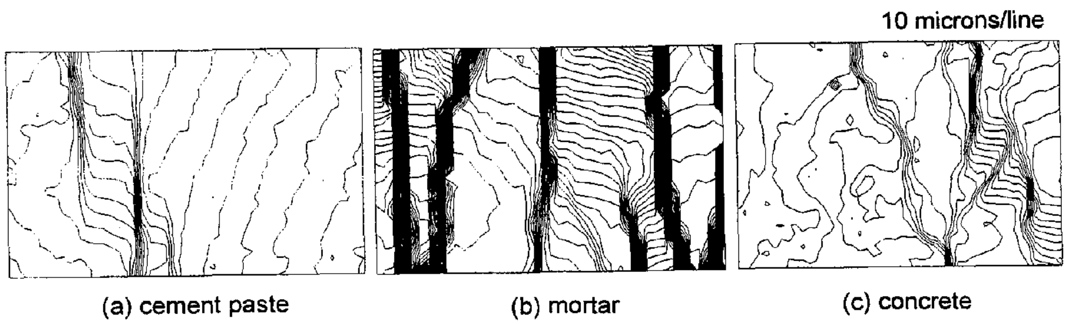


Figure 7 - Lateral deformations in the late post-peak region tested with friction reducing material, Viewing area:98x73mm

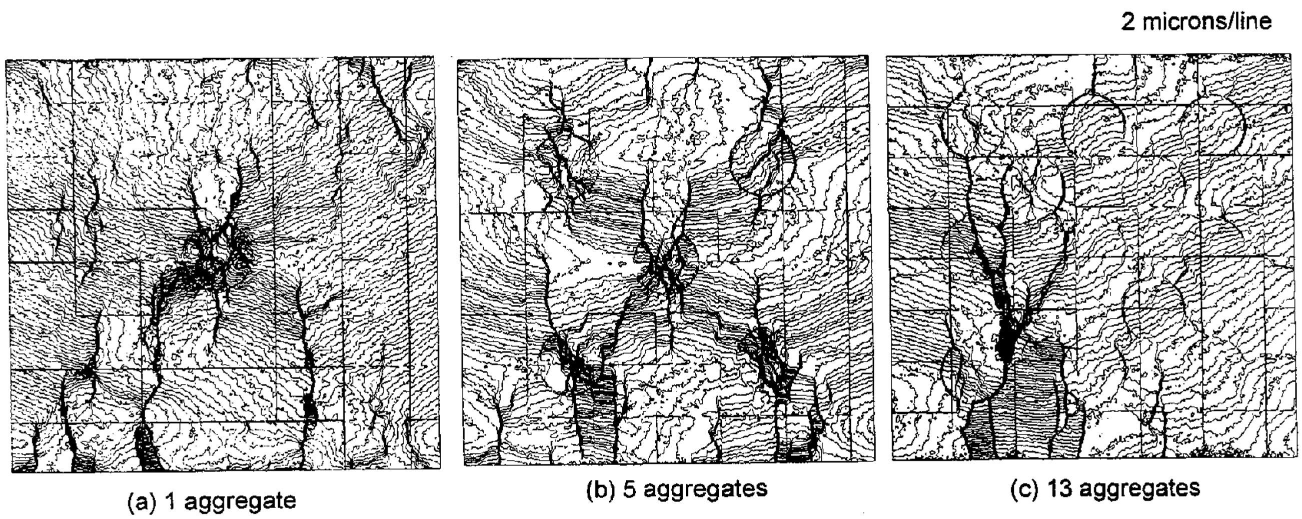


Figure 8. Lateral deformations near peak loads with different aggregate contents Viewing area: 75 mm x 75 mm

X-Ray Microtomography

X-ray microtomography (XMT) is a technique by which the internal structure of a material may be determined from maps of its x-ray absorptivity. Three dimensional maps are reconstructed from hundreds of through transmission radiographs of the sample taken from different angles [8]. A schematic drawing of the apparatus is shown in Figure 9. Microtomography is similar in practice to conventional medical CAT-scans. The primary differences are the x-ray source and detector. Microtomography uses synchrotron radiation for the x-ray source and a high resolution x-ray detector. A spatial resolution of about 2 μm is possible, although 13 μm pixels were used in the preliminary experiments described here. The data that results from a scan is a series of images which represent cross-sectional "slices" through the material. The advantage of microtomography for investigations of damage in cement-based materials is its ability to measure internal structure in three dimensions at high resolution.

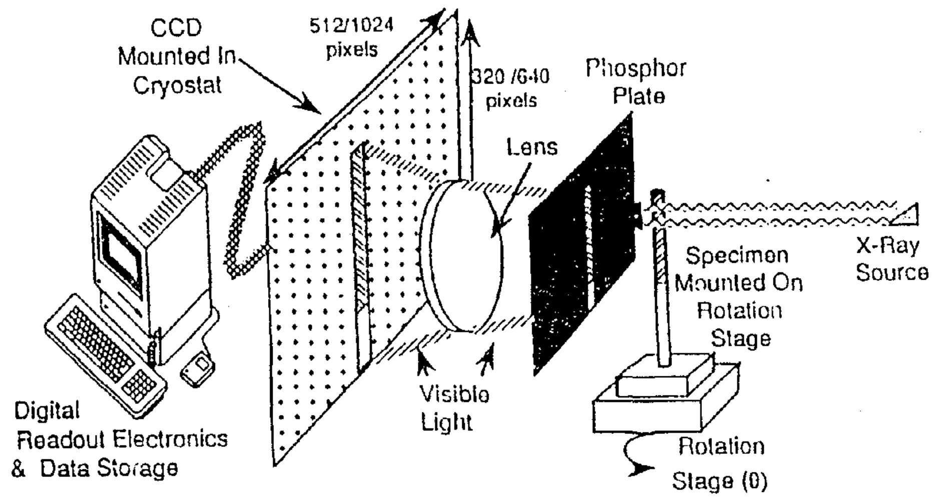


Figure 9. Schematic drawing of apparatus for XMT

An experimental apparatus was developed for scanning samples while under load. This apparatus allowed multiple scans to be made on a single specimen at different load levels. The resulting internal damage could then be correlated directly with bulk material properties such as load and deformation. Image analysis routines were developed to extract three dimensional crack area from the cross-sectional image data. The approach is very straight forward. In a single slice, the total length of a crack is measured. Then the measured crack lengths for all the slices of a particular scan are added to determine a crack area. Crack areas for scans taken at different levels of strain are then compared to the bulk load and deformation response of the specimen.

Results indicate internal crack growth can clearly be observed in three dimensions. Three dimensional renderings of tomographic data are shown in Figure 10. Some of the features critical to the fracture behavior of the material are clearly illustrated, such as crack branching and bridging. It should be noted that due to limitations of the x-ray beam from the synchrotron, investigations are limited to small samples. The cylinders shown in Figure 10 are 4 mm in diameter. Thus any conclusions from the experiments must be qualified. However, in the future test, the technique will be matched up with computer vision, where larger samples can be examined under more varied experimental conditions.

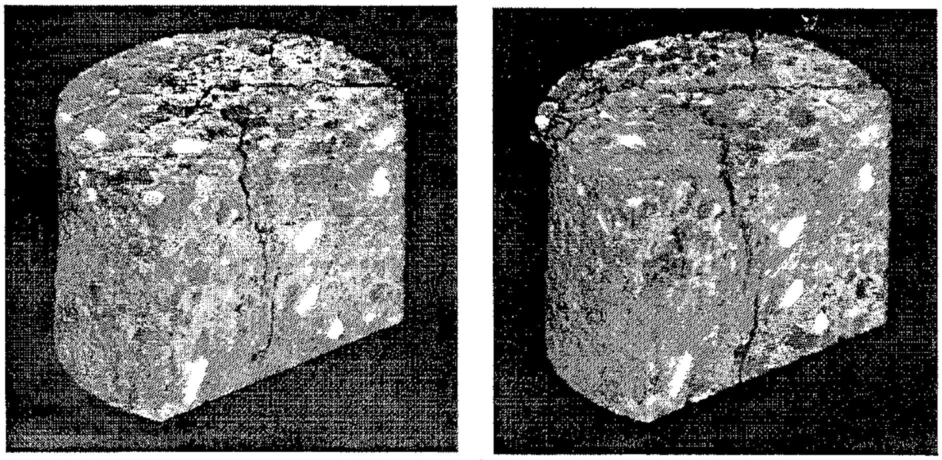


Figure 10. Three dimensional renderings of tomographic data taken at two different levels of damage.

SUMMARY

The primary intent of the AE, ESPI, DIC, and XMT experiments was basic fracture properties. However, there is considerable interest in the development of nondestructive methods to evaluate the condition of in-service structures. Some experiments were conducted as a part of a program to develop NDE techniques to evaluate distributed damage in reinforced concrete bridge decks. Specifically, ultrasonic techniques were used to evaluate the extent of damage in concrete laboratory specimens. A series of plain concrete specimens were cast and subjected to freeze-thaw, salt-scaling, and mechanical stresses. The specimens were inspected with ultrasound before and after being damaged in order to examine the effects of progressive damage on ultrasonic signals. It was found that ultrasonic attenuation, as measured as the decrease in peak-to-peak amplitude of the ultrasonic signal, was much more sensitive to changes in the material damage level, as compared to ultrasonic pulse velocity. In order to verify the damage levels, after all testing was completed, the specimens were thin-sectioned and examined using optical microscopy. The microscopy was used to measure the microcrack density. The microcrack density was correlated to the degree of damage as measured by ultrasonic methods.

The experiments described in this paper show that effective nondestructive evaluation techniques are an invaluable part of investigations of basic fracture properties of cement-based materials. The AE, ESPI, DIC (computer vision), XMT techniques were particularly valuable for making inferences on microscopic damage and localization phenomena. With this knowledge of basic fracture properties, relationships with a variety of NDE parameters can be established, and field tests can be developed.

ACKNOWLEDGMENTS

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ENHANCED MICROPLANE CONCRETE MODEL FOR DAMAGE AND FRACTURE OF CONCRETE

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ABSTRACT

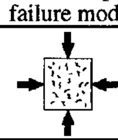
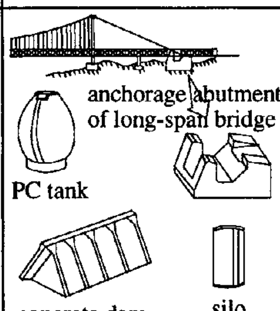
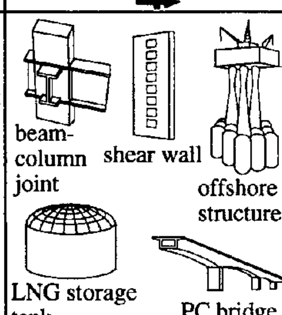
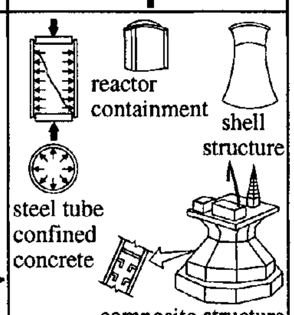
Concrete exhibits highly nonlinear material behavior due to cracking, damage, and plasticity. In evaluating the load-carrying capacity and durability of concrete infrastructures, mechanical and constitutive models that have a physically sound base in view of microscopic damage processes in concrete and can predict damage processes of the material in general mechanical conditions with accuracy are important. One of the promising constitutive models for that purpose is the Enhanced Microplane Concrete Model, whose basic idea and formulation are outlined. The model is shown to be capable of describing the important damage, fracture, and plasticity modes occurring in civil concrete infrastructures.

INTRODUCTION

Concrete, having many advantages over other construction materials such as steel, soil, and wood is one of the most practical engineering materials for civil infrastructure systems. However, concrete's unique material nonlinearity related to cracking, damage, fracture, and plasticity is an important issue which has to be examined in terms of mechanical strength, deformability, and durability of the infrastructures.

Recent developments in computational mechanics make it more feasible and realistic to simulate damage and deterioration processes in concrete infrastructures. Although general and rational material models (constitutive models) are necessary to simulate damage, fracture, and plasticity processes under various stress conditions, only a few models are available for that purpose. The Enhanced Microplane Concrete Model [1, 2] is one of the promising constitutive models for realistically describing these behaviors. It is shown to be capable of characterizing the important damage, fracture, and plasticity modes occurring in civil concrete infrastructures. The basic idea and formulation of the model will be outlined in this paper.

Table 1 Concrete infrastructures and dominant damage modes

	cracking mode	shear failure mode	multiaxial compressive failure mode
damage mode			
concrete infra-structure			
research items	• hydration thermal crack • crack resistance • size effect	• shear strength • ductility • load-carrying mechanism	• strength and ductility • multiaxial effect • confinement effect

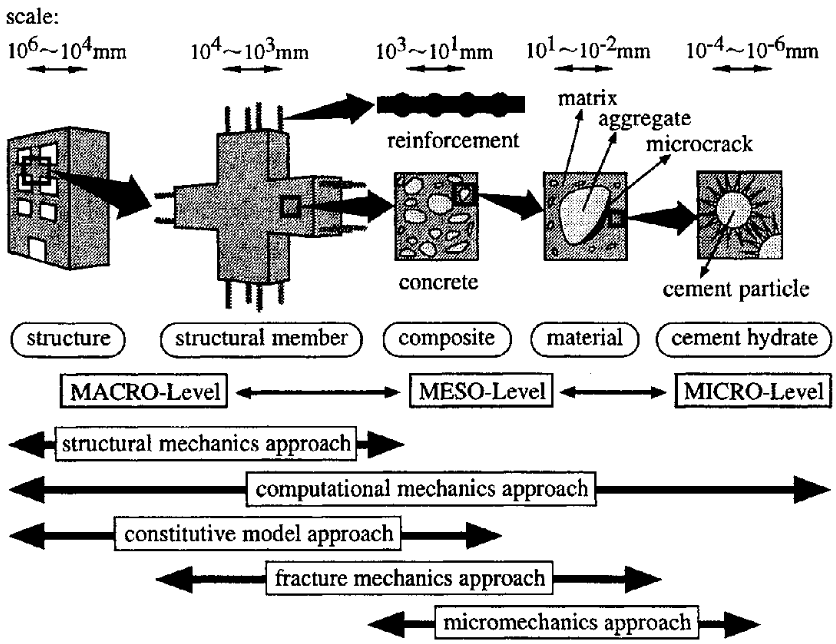


Fig. 1 Structural levels of concrete and mechanical approaches to them

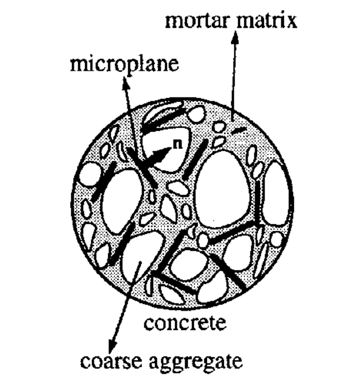


Fig. 2 Microplanes in concrete

DAMAGE AND FRACTURE IN CONCRETE INFRASTRUCTURES

Concrete infrastructures suffer from various kinds of damage and deterioration during public service. From the viewpoint of safety assurance in practical structural design, the following issues related to damage and fracture of concrete structures are very important: 1) mechanism and strength in shear failure; 2) structural ductility and brittleness; 3) structural size effects in strength and ductility. Since there are serious limitations in studying these problems with experimental methods, the approach of numerical simulations with computational mechanics is gaining popularity to investigate the processes as well as the mechanisms of damage and fracture in structures.

There exists a variety of civil concrete infrastructures with individual social functions. Some of these concrete infrastructures are shown in **Table 1**, in which the infrastructures are divided into three groups classified by their dominant damage modes: cracking, shear failure, multiaxial compressive failure modes. Some of the structures have not only one dominant damage mode, but also others depending on their design load and other conditions. Furthermore, material deterioration due to carbonation, shrinkage, and corrosion of steel reinforcements, etc., combines with mechanical damage and fracture due to structural loads, which results in more complicated damage to the concrete structures. To simulate damage under such general and complicated conditions in the concrete infrastructures more sophisticated mechanical models with accuracy and physical bases are necessary.

To consider damage and fracture in concrete three structural levels are often referred to; these are macro-, meso-, and micro-levels. In **Fig. 1** these three structural levels are shown using a concrete building as an example. Damage and fracture occurring in the representative elements for each structural level can be well explained by different mechanical approaches as shown in **Fig. 1**. The computational mechanics approach, when used with proper constitutive models of concrete and fracture mechanics, can be applied to a relatively wide range of structural levels. However, in reality most of the existing constitutive models for concrete are phenomenological and instead of having general applicability they have some serious limitations. To derive constitutive and mechanical models for concrete as a heterogeneous material with much wider applicability, it is important to consider a lower structural level than that used in models for explaining mechanical and structural behaviors. The Enhanced Microplane Concrete Model is one such constitutive model. It has a clear physical image at the microscopic level, and is based on a characteristic hypothesis that the inelastic origin of concrete as a heterogeneous material is microcracks which occur within the interface region (microplane: **Fig. 2**) between the coarse aggregate particle and the mortar matrix.

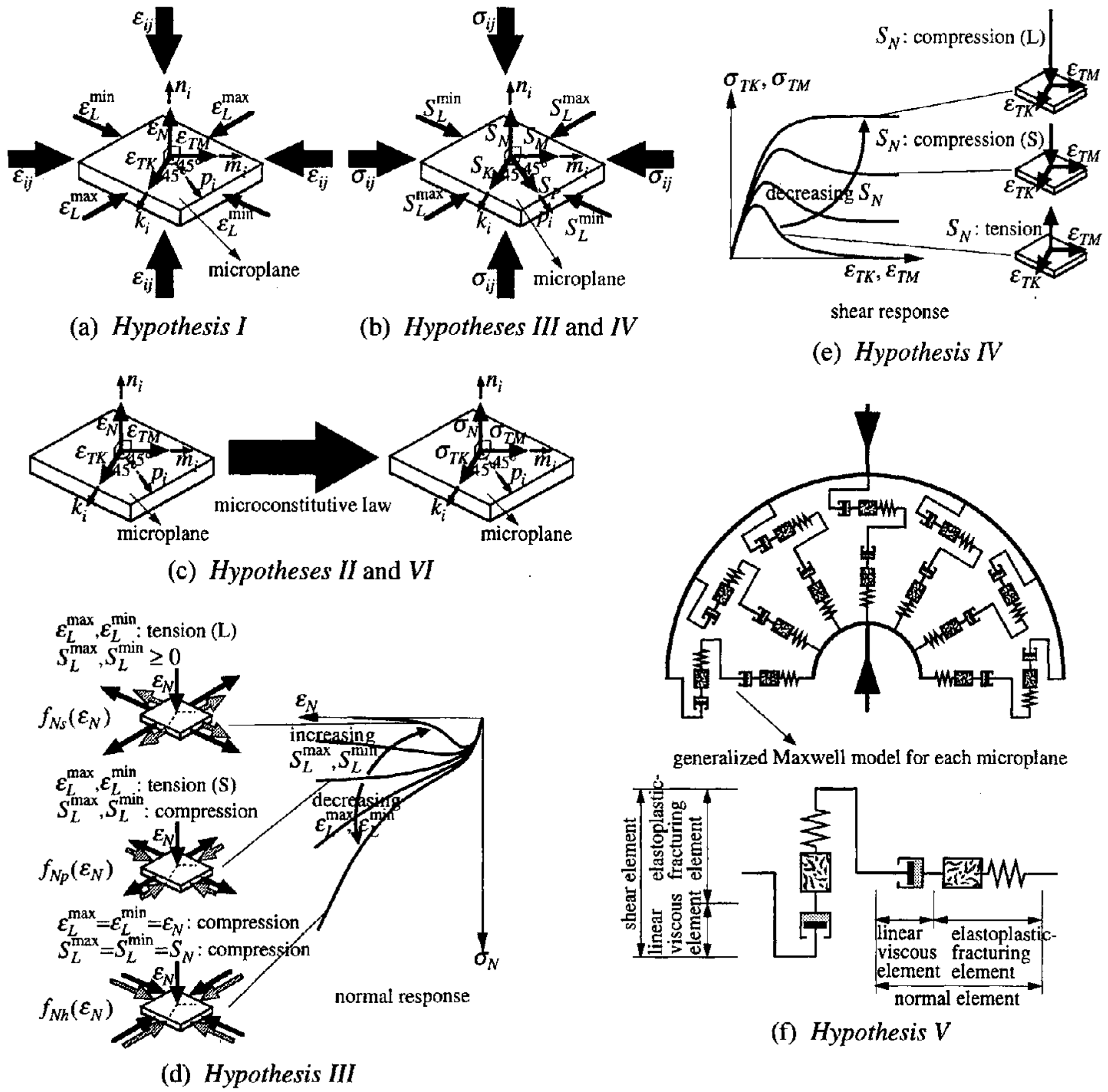


Fig. 3 Hypotheses of the Enhanced Microplane Concrete Model

BASIC FORMULATION OF ENHANCED MICROPLANE CONCRETE MODEL

The following are the hypotheses made in the Enhanced Microplane Concrete Model:

Hypothesis I : Normal strain ε_N, shear strains ε_{TK}, ε_{TM}, and lateral strain ε_L of a microplane are the resolved components of the macroscopic strain tensor ε_{ij} (tensorial kinematic constraint).

Hypothesis II : Normal stress σ_N and shear stresses σ_{TK}, σ_{TM} on a microplane depend on normal strain ε_N and shear strains ε_{TK}, ε_{TM}. The relations between those strains and stresses are described by microconstitutive laws. The directions of the shear stresses on each microplane are the same as those of the shear strains.

Hypothesis III : The inelastic normal stress increment on a microplane depends on the resolved lateral strain ε_L of the macroscopic strain tensor ε_{ij} (lateral strain effect) and resolved lateral stress S_L of the macroscopic stress tensor σ_{ij} onto the same microplane (additional static constraint: lateral stress effect).

Hypothesis IV : The inelastic shear stress increment on a microplane depends on the resolved normal component S_N of the macroscopic stress tensor σ_{ij} onto the same microplane (additional static constraint: transition from brittle to ductile fracture for microplane shear response).

Hypothesis V : The microconstitutive laws for the normal and shear components are based on a generalized Maxwell rheologic model in which a linear viscous element is coupled in series with an elastoplastic-fracturing element.

Hypothesis VI : The microconstitutive laws for the normal and shear components on each microplane are mutually

independent.
Fig. 3 illustrates these hypotheses.

According to *Hypothesis I*, the normal strain component on a microplane with unit normal vector $\mathbf{n}$ is

$$\varepsilon_N = n_j \varepsilon_j^n = n_j n_k \varepsilon_{jk} \tag{1}$$

in which n_i = components of unit normal vector $\mathbf{n}$ of the microplane.
The shear strain components in the $\mathbf{k}$ and $\mathbf{m}$ directions on a microplane with direction cosines n_i are

$$\varepsilon_{TK} = k_j \varepsilon_j^n = k_j n_i \varepsilon_{ij} = \frac{1}{2}\left(k_i n_j + k_j n_i\right)\varepsilon_{ij}; \quad \varepsilon_{TM} = m_j \varepsilon_j^n = m_j n_i \varepsilon_{ij} = \frac{1}{2}\left(m_i n_j + m_j n_i\right)\varepsilon_{ij} \tag{2a, b}$$

in which k_i and m_i are components of in-plane unit coordinate vectors $\mathbf{k}$ and $\mathbf{m}$.
To implement *Hypothesis III*, we need derive equations for the maximum and minimum principal values $\varepsilon_L^{\max}$, $\varepsilon_L^{\min}$ of lateral strain of each microplane. To this end we introduce another in-plane unit vector $\mathbf{p}$ whose angle with the unit vectors $\mathbf{k}$ and $\mathbf{m}$ is 45°, as shown in **Fig. 3(a)**; $\mathbf{p} = (\mathbf{k}+\mathbf{m})/\sqrt{2}$. The lateral normal strains in the directions of $\mathbf{k}$, $\mathbf{m}$, and $\mathbf{p}$ are $\varepsilon_K = k_i k_j \varepsilon_{ij}$, $\varepsilon_M = m_i m_j \varepsilon_{ij}$, and $\varepsilon_P = p_i p_j \varepsilon_{ij}$ respectively, in which p_i = components of the in-plane unit vector $\mathbf{p}$ of the microplane.
Considering Mohr's circle for the in-plane strains of the microplane, we can obtain the maximum and minimum principal values $\varepsilon_L^{\max}$, $\varepsilon_L^{\min}$ of the lateral strain on each microplane

$$\varepsilon_L^{\max},\ \varepsilon_L^{\min} = \frac{\varepsilon_K + \varepsilon_M}{2} \pm \sqrt{\left(\frac{\varepsilon_K - \varepsilon_M}{2}\right)^2 + \left(\frac{\varepsilon_K + \varepsilon_M}{2} - \varepsilon_P\right)^2} \tag{3}$$

The incremental forms of microconstitutive relations are written separately for the normal component and the shear components in the *K* and *M* directions:

normal component: $$d\sigma_N = C_N d\varepsilon_N - d\sigma_N'' = f_{N1}\left(\varepsilon_N, \varepsilon_L, S_L\right) = f_{N2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) \tag{4a}$$

K-shear component: $$d\sigma_{TK} = C_{TK} d\varepsilon_{TK} - d\sigma_{TK}'' = f_{T1}\left(\varepsilon_{TK}, S_N\right) = f_{T2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) \tag{4b}$$

M-shear component: $$d\sigma_{TM} = C_{TM} d\varepsilon_{TM} - d\sigma_{TM}'' = f_{T1}\left(\varepsilon_{TM}, S_N\right) = f_{T2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) \tag{4c}$$

in which $d\sigma_N$, $d\sigma_{TK}$, and $d\sigma_{TM}$ = microplane stress increments; C_N, C_{TK}, and C_{TM} = incremental elastic stiffnesses for the microplane; $d\sigma_N''$, $d\sigma_{TK}''$, and $d\sigma_{TM}''$ = inelastic microplane stress increments; $f_{N1}\left(\varepsilon_N, \varepsilon_L, S_L\right)$ and $f_{N2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right)$ are microplane normal stress increments $d\sigma_N$ expressed in terms of ε_N, ε_L, and S_L, and in terms of ε_{kl}, σ_{kl}, and n_r; $f_{T1}\left(\varepsilon_{Ts}, S_N\right)$ and $f_{T2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right)$ are microplane shear stress increments $d\sigma_{Ts}$ expressed in terms of ε_{Ts} and S_N, and in terms of ε_{kl}, σ_{kl}, and n_r $(Ts = TK, TM)$.
Using the principle of virtual work (i.e., the equality of virtual works of the stress tensor within a unit sphere and microplane stresses on the surface of the sphere), we can write

$$\frac{4\pi}{3} d\sigma_{ij} \delta\varepsilon_{ij} = 2\int_S \left(d\sigma_N \delta\varepsilon_N + d\sigma_{TK}\delta\varepsilon_{TK} + d\sigma_{TM}\delta\varepsilon_{TM}\right) f(\mathbf{n})\, dS \tag{5}$$

in which $\int_S dS = \int_0^{2\pi}\int_0^{\pi/2} \sin\phi\, d\phi\, d\theta$; θ and ϕ = the spherical angular coordinates; and $\delta\varepsilon_{ij}$, $\delta\varepsilon_N$, $\delta\varepsilon_{TK}$, and $\delta\varepsilon_{TM}$ = small variations of the strain tensor and of the microplane strains. The constant $4\pi/3$ means that the work of the stress tensor is taken over the volume of the unit sphere. The factor of 2 on the right-hand side arises because the work of microplanes needs to be integrated only over the surface of the unit hemisphere S. The function $f(\mathbf{n})$ is a weight function for the normal directions $\mathbf{n}$, which in general can be used to introduce anisotropy of the material in its initial state. We will use $f(\mathbf{n}) = 1$, which means isotropy. Expressing $\delta\varepsilon_N$, $\delta\varepsilon_{TK}$, and $\delta\varepsilon_{TM}$ from (1) and (2) and substituting them into (5), we obtain

$$\frac{4\pi}{3} d\sigma_{ij} \delta\varepsilon_{ij} = 2\int_S \left[n_i n_j d\sigma_N + \frac{d\sigma_{TK}}{2}\left(k_i n_j + k_j n_i\right) + \frac{d\sigma_{TM}}{2}\left(m_i n_j + m_j n_i\right)\right] f(\mathbf{n})\, dS\, \delta\varepsilon_{ij} \tag{6}$$

This variational equation must hold for any variations $\delta\varepsilon_{ij}$, therefore, we can delete $\delta\varepsilon_{ij}$. Then, substituting (4), now (1) and (2) may be used here for ε_N, ε_{TK}, and ε_{TM}. This finally yields the incremental form of the macroscopic stress-strain relation

$$d\sigma_{ij} = C_{ijrs} d\varepsilon_{rs} - d\sigma_{ij}'' \tag{7a}$$

$$C_{ijrs} = \frac{3}{2\pi}\int_S \left[n_i n_j n_r n_s C_N + \frac{1}{4}\left(k_i n_j + k_j n_i\right)\left(k_r n_s + k_s n_r\right) C_{TK} + \frac{1}{4}\left(m_i n_j + m_j n_i\right)\left(m_r n_s + m_s n_r\right) C_{TM}\right] f(\mathbf{n})\, dS \tag{7b}$$

$$d\sigma_{ij}" = \frac{3}{2\pi}\int_S \left[n_i n_j d\sigma_N" + \frac{1}{2}\left(k_i n_j + k_j n_i\right) d\sigma_{TK}" + \frac{1}{2}\left(m_i n_j + m_j n_i\right) d\sigma_{TM}" \right] f(\mathbf{n}) dS \tag{7c}$$

in which C_{ijrs} = incremental elastic stiffness tensor; and $d\sigma_{ij}"$ = inelastic stress increment.

Relation (7) can be expressed in another form (8).

$$\begin{aligned} d\sigma_{ij} &= \frac{3}{2\pi}\int_S \left[n_i n_j d\sigma_N + \frac{d\sigma_{TK}}{2}\left(k_i n_j + k_j n_i\right) + \frac{d\sigma_{TM}}{2}\left(m_i n_j + m_j n_i\right) \right] f(\mathbf{n}) dS \\ &= \frac{3}{2\pi}\int_S \left[n_i n_j f_{N2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) + \frac{1}{2}\left(k_i n_j + k_j n_i\right) f_{T2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) \right. \\ &\quad \left. + \frac{1}{2}\left(m_i n_j + m_j n_i\right) f_{T2}\left(\varepsilon_{kl}, \sigma_{kl}, n_r\right) \right] f(\mathbf{n}) dS \end{aligned} \tag{8}$$

As can be seen from the fact that the incremental stress tensor $d\sigma_{ij}$ depends not only on strain tensor ε_{kl} but also on stress tensor σ_{kl}, the interactions between microplanes are modeled in the Enhanced Microplane Concrete Model through the additional static constraint that a microplane response depends on the resolved components of the stress tensor obtained by spherical integration of the stresses of all microplanes. This interaction effect means that the present model deviates from the basic concept that individual microplane responses are mutually independent, which leads to the kinematic constraint. This is necessary to take account of a situation within concrete where microcracks, damage, and plasticity in each direction have mutual effects.

MICROCONSTITUTIVE LAW FOR NORMAL COMPONENTS

The purpose of taking account the lateral strain and stress effects on normal compression response of a microplane according to *Hypothesis III* is to achieve the following (**Fig. 3(d)**):

1) The normal compression response would not be the same as the hydrostatic response except when the lateral strains ε_L are the same as the normal strain ε_N, which is the case of hydrostatic loading.
2) The normal compression response would have a plastic plateau when the difference between the normal strain ε_N and the lateral strain ε_L is large and the resolved lateral stress S_L of the microplane is a large, compressive value, i.e., it would exhibit ductile plasticity.
3) The normal compression response would be more brittle when the difference between the normal strain ε_N and the lateral strain ε_L is large and the resolved lateral stress S_L of the microplane is a small, compressive value or a tensile value, i.e., it would exhibit more strain softening.

To formulate the lateral stress effect, we resolve the stress tensor σ_{ij} into the lateral normal stresses S_K, S_M, and S_P in the directions of the in-plane unit coordinate vectors $\mathbf{k}$, $\mathbf{m}$, and $\mathbf{p}$ (**Fig. 3(b)**), in which $S_K = k_i k_j \sigma_{ij}$, $S_M = m_i m_j \sigma_{ij}$, and $S_P = p_i p_j \sigma_{ij}$.
Considering Mohr's circle for the lateral normal stresses of the microplane, we can obtain the maximum and minimum principal values $S_L^{\max}$, $S_L^{\min}$ of the lateral stress of each microplane

$$S_L^{\max},\ S_L^{\min} = \frac{S_K + S_M}{2} \pm \sqrt{\left(\frac{S_K - S_M}{2}\right)^2 + \left(\frac{S_K + S_M}{2} - S_P\right)^2} \tag{9}$$

We define a lateral confinement stress S_{LC} that combines $S_L^{\max}$ and $S_L^{\min}$ into one stress invariant for the microplane

$$\begin{aligned} S_{LC} &= S_L^{\max} + S_L^{\min} && : \text{when } S_L^{\max} < 0 \text{ and } S_L^{\min} < 0 \\ &= 0 && : \text{when } S_L^{\max} \geq 0 \\ &= 0 && : \text{when } S_N \geq 0 \text{ on any other microplane} \\ &= S_{LC}^p && : \text{when } S_{LC} \leq S_{LC}^p \end{aligned} \tag{10}$$

in which $S_{LC}^p \leq S_{LC} \leq 0$; and $S_{LC}^p = S_{LC}$ value corresponding to the case of plastic response.
The lateral-deviatoric strain ε_{LD}, which is the difference between the normal ε_N and lateral ε_L strains of a microplane, is defined using the maximum and minimum principal values $\varepsilon_L^{\max}$, $\varepsilon_L^{\min}$ of the lateral strain on the microplane

$$\varepsilon_{LD} = \left|\varepsilon_N - \varepsilon_L^{\max}\right| + \left|\varepsilon_N - \varepsilon_L^{\min}\right| \tag{11}$$

In the present model, the following hardening-softening function $\phi(\varepsilon_{LD})$ based on the lateral-deviatoric strain

ε_{LD} and the lateral confinement stress S_{LC} is introduced:

$$\phi(\varepsilon_{LD}) = \frac{1}{1+\left(\varepsilon_{LD}/\varepsilon_{LD}^{1}\right)^{m}} \quad : \text{when } S_{LC} < 0$$

$$= \phi^{p} \quad : \text{when } \varepsilon_{LD} = \varepsilon_{LD}^{p}$$

$$= 0 \quad : \text{when } S_{LC} \geq 0 \tag{12}$$

in which ε_{LD}^{1} = ε_{LD} value when $\phi(\varepsilon_{LD}) = 0.5$; ε_{LD}^{p} = ε_{LD} value corresponding to the case of normal plastic response; m = a constant that specifies the shape of the curve $\phi(\varepsilon_{LD})$; and ϕ^{p} = $\phi(\varepsilon_{LD})$ value corresponding to the case of normal plastic response.

Weight functions are defined in terms of $\phi(\varepsilon_{LD})$ and S_{LC}, and utilized to obtain a gradual transition from hydrostatic response to plastic response and softening response for the virgin loading curve of the normal component of the microplane (**Fig. 3(d)**).

When $1 \geq \phi(\varepsilon_{LD}) \geq \phi^{p}$ and any S_{LC}:

$$\sigma_N(\varepsilon_N, \varepsilon_{LD}, S_{LC}) = \left(\frac{\phi(\varepsilon_{LD}) - \phi^{p}}{1-\phi^{p}}\right) f_{Nh}(\varepsilon_N) + \left(\frac{1-\phi(\varepsilon_{LD})}{1-\phi^{p}}\right) f_{Np}(\varepsilon_N) \tag{13a}$$

when $\phi^{p} > \phi(\varepsilon_{LD}) \geq 0$ and $S_{LC} \leq S_{LC}^{p}$:

$$\sigma_N(\varepsilon_N, \varepsilon_{LD}, S_{LC}) = f_{Np}(\varepsilon_N) \tag{13b}$$

when $\phi^{p} > \phi(\varepsilon_{LD}) \geq 0$ and $S_{LC}^{p} < S_{LC} < 0$:

$$\sigma_N(\varepsilon_N, \varepsilon_{LD}, S_{LC}) = \left(\frac{S_{LC}}{S_{LC}^{p}}\right) f_{Np}(\varepsilon_N) + \left(\frac{S_{LC}^{p} - S_{LC}}{S_{LC}^{p}}\right) f_{Ns}(\varepsilon_N) \tag{13c}$$

when $\phi^{p} > \phi(\varepsilon_{LD}) \geq 0$ and $0 \leq S_{LC}$:

$$\sigma_N(\varepsilon_N, \varepsilon_{LD}, S_{LC}) = f_{Ns}(\varepsilon_N) \tag{13d}$$

in which $f_{Nh}(\varepsilon_N)$ = hydrostatic loading curve (when $\phi(\varepsilon_{LD}) = 1$); $f_{Np}(\varepsilon_N)$ = plastic loading curve (when $\phi(\varepsilon_{LD}) = \phi^{p}$); and $f_{Ns}(\varepsilon_N)$ = compression softening loading curve (when $\phi^{p} > \phi(\varepsilon_{LD}) \geq 0$ and $0 \leq S_{LC}$).

The details of the microconstitutive model for normal components are described in the previous study [1].

MICROCONSTITUTIVE LAW FOR SHEAR COMPONENTS

Since the difference between K-shear and M-shear is only its direction on the microplane, the microconstitutive laws for those shears must be identical. Therefore, we do not differentiate K-shear from M-shear, but consider a unique microconstitutive law for shear with subscript T which refers to K-shear (TK) and M-shear (TM). In the present model, shear loading curves are defined individually for softening (subscript TT) under resolved normal tension stress, softening (subscript TC) under resolved normal compression stress, and plasticity (subscript Tp) under resolved normal compression stress. A shear friction law is applied to evaluate shear peak stress for pre-peak and post-peak curves under resolved normal tension stress and for pre-peak curves under resolved normal compression stress. On the other hand, post-peak shear response under resolved normal compression stress is calculated by weighting the softening and plasticity loading curves with resolved normal stress. This results in a transition model from brittle to ductile fracture for shear response on the microplane (**Fig. 3(e)**).

The resolved normal component S_N of the stress tensor σ_{ij} on a microplane whose direction cosines are n_i is

$$S_N = n_j \sigma_j^{n} = n_j n_k \sigma_{jk} \tag{14}$$

The loading curve $\sigma_T(\varepsilon_T, S_N)$ under resolved normal stress S_N is calculated by weighting each loading curve $f_{Tp}(\varepsilon_T)$, $f_{TC}(\varepsilon_T)$, and $f_{TT}(\varepsilon_T)$ with resolved normal stress S_N as follows.

when $S_N \leq S_N^{p}$:

$$\sigma_T(\varepsilon_T, S_N) = f_{Tp}(\varepsilon_T) \tag{15a}$$

when $S_N^{p} < S_N < 0$ and in pre-peak:

$$\sigma_T(\varepsilon_T, S_N) = f_{TC}(\varepsilon_T) = f_{Tp}(\varepsilon_T) \tag{15b}$$

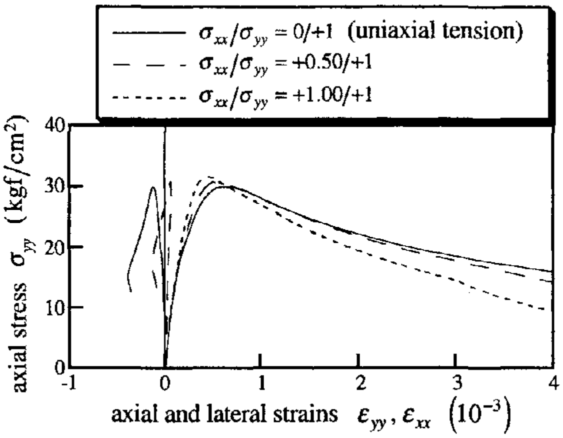


Fig. 4 Biaxial tension analysis

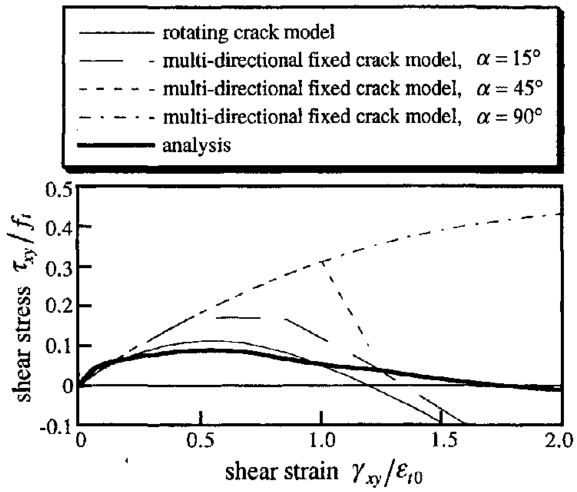


Fig. 5 Biaxial tension-shear analysis

when $S_N^p < S_N < 0$ and in post-peak:

$$\sigma_T(\varepsilon_T, S_N) = \left(\frac{S_N}{S_N^p}\right) f_{Tp}(\varepsilon_T) + \left(\frac{S_N^p - S_N}{S_N^p}\right) f_{TC}(\varepsilon_T) \tag{15c}$$

when $0 \le S_N$:

$$\sigma_T(\varepsilon_T, S_N) = f_{TT}(\varepsilon_T) \tag{15d}$$

in which $f_{Tp}(\varepsilon_T)$ = plastic loading curve (when $S_N \le S_N^p$); $f_{TC}(\varepsilon_T)$ and $f_{TT}(\varepsilon_T)$ = softening loading curves under resolved normal compression and tension stresses; and S_N^p = S_N value when the shear response corresponds to the plastic loading curve.

The concept of shear frictional coefficient μ_{TT}, μ_{TC} is utilized to model the dependence of shear peak stress τ^0 on resolved normal stress S_N.

For tension of shear $(\varepsilon_T > 0)$:

$$\text{when } S_N < 0: \quad \tau^0 = +\sigma_{TC}^0 - \mu_{TC} S_N \tag{16a}$$

$$\text{when } S_N \ge 0: \quad \tau^0 = +\sigma_{TT}^0 - \mu_{TT} S_N \ge +r_{\min}^0 \sigma_{TT}^0 \tag{16b}$$

for compression of shear $(\varepsilon_T < 0)$:

$$\text{when } S_N < 0: \quad \tau^0 = -\sigma_{TC}^0 + \mu_{TC} S_N \tag{16c}$$

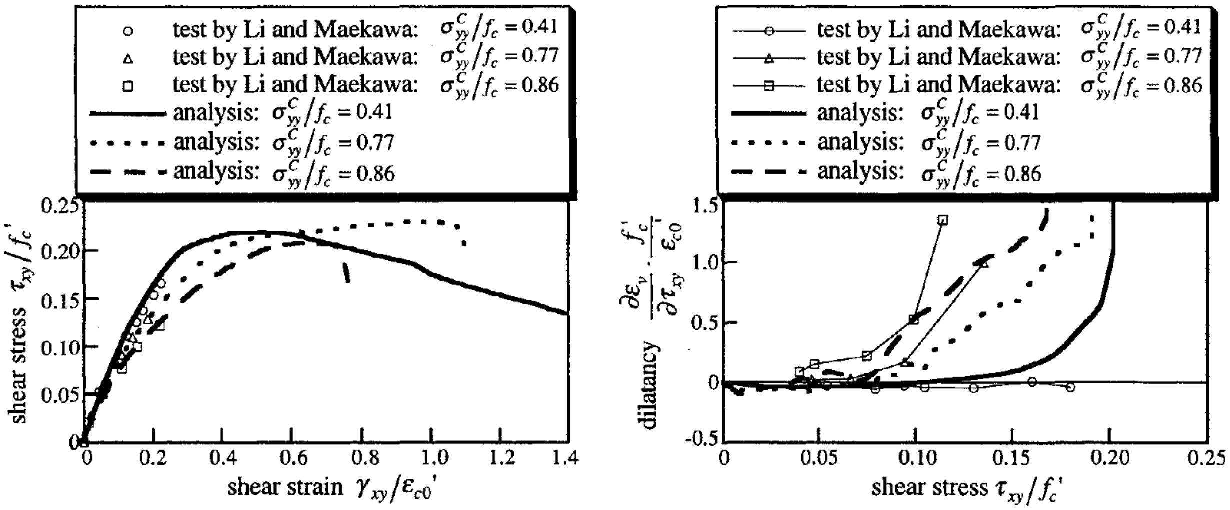
$$\text{when } S_N \ge 0: \quad \tau^0 = -\sigma_{TT}^0 + \mu_{TT} S_N \le -r_{\min}^0 \sigma_{TT}^0 \tag{16d}$$

in which $\sigma_{TT}^0 (> 0)$ and $\sigma_{TC}^0 (> 0)$ = shear peak stresses at $S_N = 0$ under resolved normal tension and compression stresses; $\mu_{TT} (> 0)$ and $\mu_{TC} (> 0)$ = shear frictional coefficients under resolved normal tension and compression stresses; and $r_{\min}^0$ = a constant specifying a lower limit on shear peak stress under resolved normal tension stress $\left(0 < r_{\min}^0 \le 1\right)$.

The details of the microconstitutive model for shear components are described in the previous study [1].

CONSTITUTIVE RELATIONS AND DAMAGE PROCESSES OF CONCRETE

The Enhanced Microplane Concrete Model can provide good predictions for the damage modes shown in **Table 1**. The results for uniaxial tension analysis as well as biaxial tension analysis in which the cracking mode is dominant are shown in **Fig. 4**. The analytical responses under biaxial tension exhibit considerable nonlinearity in the pre-peak regime, while typical average stress-strain relations in the experiments show almost perfect elasticity under biaxial tension. This model is thought to be capable of evaluating nonlinear behavior in a highly localized damage region such as a fracture process zone.



(a) Shear stress-strain relation (b) Dilatancy-shear stress relation

Fig. 6 Uniaxial compression-shear analysis

In **Fig. 5** the shear response obtained in the biaxial tension-shear analysis is compared with the results calculated by Rots [3] using a rotating crack model and a multi-directional fixed crack model, in which f_t = uniaxial tensile strength; ε_{t0} = the axial strain corresponding to f_t; and α = threshold angle. The present model predicts a flexible shear response similar to the result achieved with the rotating crack model that has been shown capable of simulating shear-tension failure. This means that the present model could predict shear failure modes such as diagonal tension failure of reinforced concrete beams. To evaluate prediction accuracy of the present model under uniaxial compression-shear, the experiment of Li and Maekawa [4] is simulated in this study and the comparisons are shown in **Fig. 6**, in which σ_{yy}^C = the fixed uniaxial compressive stress; f_c' = uniaxial compressive strength; ε_{c0}' = the axial strain corresponding to f_c'; and ε_v = volumetric strain. This model can accurately simulate dilatancy of concrete which is a typical and important phenomenon in shear damage under compression stress.

In **Fig. 7** calculated cyclic and monotonic responses under biaxial compression are compared with the experiment by van Mier [5], in which the biaxial stress ratio was $\sigma_{xx}/\sigma_{yy} = -0.05/-1$. **Fig. 9** shows the normal, *K*-shear, and *M*-shear responses of microplanes (integration points) 2, 3, and 14 (**Fig. 8**) as well as the average volumetric response ε_{av} for the biaxial compression analysis. The model well describes strain-softening stiffness and the degradation of unloading and reloading stiffnesses during strain softening under biaxial compression; however, lateral strain ε_{xx} is overestimated in the calculation as compared with the experiment. The curvature of the macroscopic hysteresis loop at lower stress levels under biaxial compression depends on the alternating cyclic loading response of microplane when normal compression unloading proceeds into normal tension loading. As shown in **Figs. 9(a), (b)**, macroscopic tensile strain and cyclic loading cause tensile microplane stress to be induced on microplanes where there is no resolved tensile component of the macroscopic stress tensor. This tensile microplane stress causes microscopic damage, which then results in macroscopic damage and stiffness degradation. In a heterogeneous concrete material with no macroscopic tensile stress, microscopic tensile strain and stress are induced on a microscopic level or meso-level, becoming the origin of macroscopic damage [6, 7]. The model accounts for the microscopic damage mechanism in macroscopic constitutive modeling in a simple and reasonable way without a complicated micromechanics model.

Fig. 10 shows the analytical result for the biaxial strength envelope compared with experiments by Kupfer et al. [8]. It confirms that the biaxial strength of concrete can be estimated with accuracy using the present model. The obtained ratio of uniaxial tensile strength to uniaxial compressive strength in the analysis agrees well with typical experimental values. **Fig. 11** shows the result of triaxial compression analysis along the compressive meridian in comparison with the experiments by Smith et al. [9], in which σ_c = confinement pressure. The model predicts increases in strength and ductility with confinement pressure under triaxial compression with practical accuracy, i.e., the transition from brittle to ductile fracture. This is due to rational modeling of responses on the microplane. The compressive and tensile meridians of the failure envelope are evaluated from maximum stresses obtained in the analyses, and shown in **Fig. 12** with experimental results from the literature [10], where

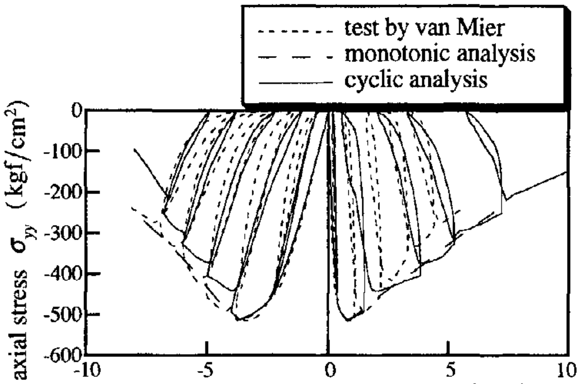


Fig. 7 Cyclic biaxial compression analysis

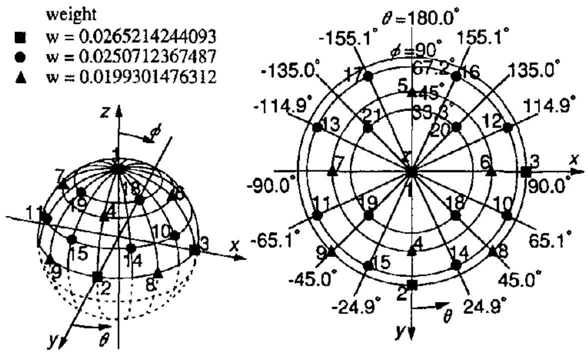
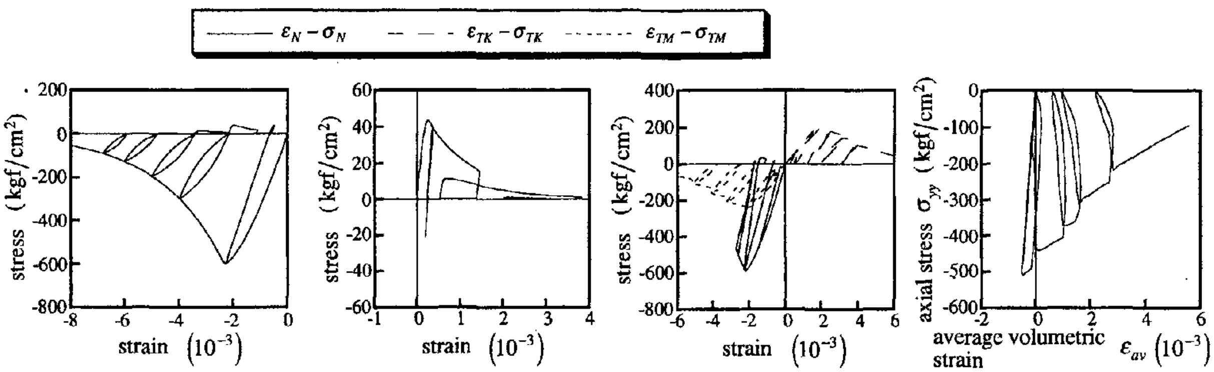


Fig. 8 Numerical integration points on unit hemisphere



(a) Microplane 2 (b) Microplane 3 (c) Microplane 14 (d) Average volumetric response

Fig. 9 Responses in cyclic biaxial compression analysis

$\sigma_{oct} = I_1/3 = \sigma_{ii}/3$ and $\tau_{oct} = \sqrt{2J_2/3}$ are octahedral normal and shear stresses (J_2 = the 2nd invariant of deviatoric stress tensor). The model predicts the compressive meridian very well, but it slightly overestimates the tensile meridian.

CONCLUSION

The Enhanced Microplane Concrete Model has a clear physical image at the microscopic level, and is based on a characteristic hypothesis that the inelastic origin of concrete as a heterogeneous material is microcracks which occur within the interface region (microplane) between the coarse aggregate particle and the mortar matrix. In this paper the basic idea and formulation of the model are described. It is verified that the model can predict well the experimentally obtained constitutive relations and important damage modes of concrete which occur in civil concrete infrastructures.

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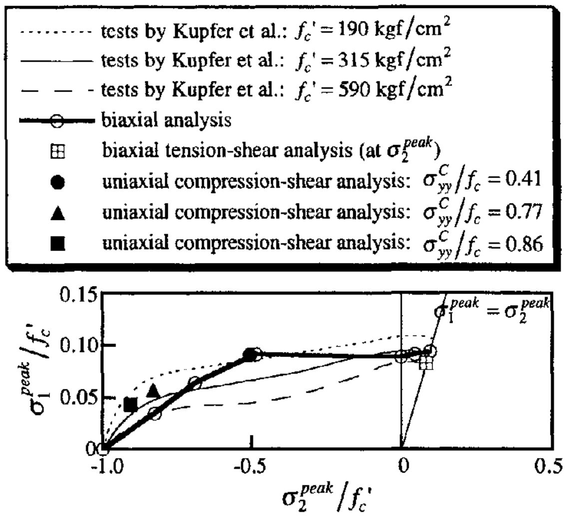
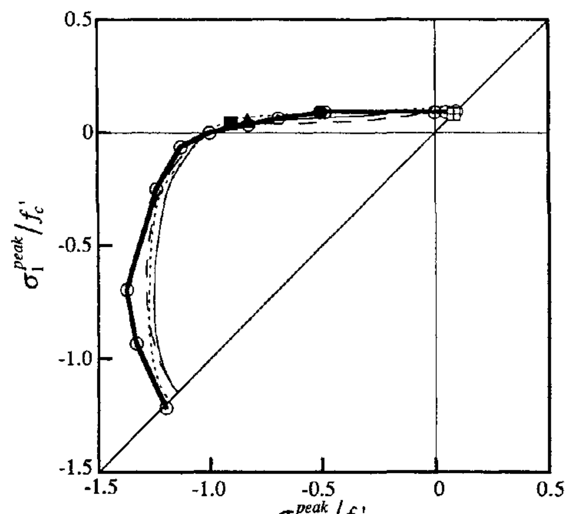


(a) Compression-tension and tension-tension stress regions



(b) Entire region of biaxial stress

Fig. 10 Biaxial strength envelopes

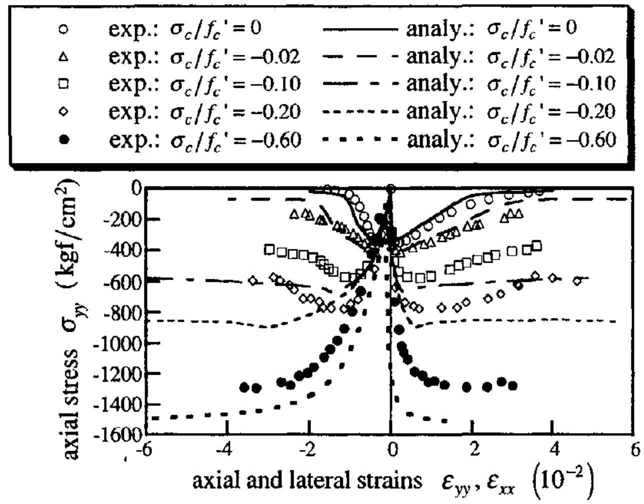


Fig. 11 Triaxial compression analysis

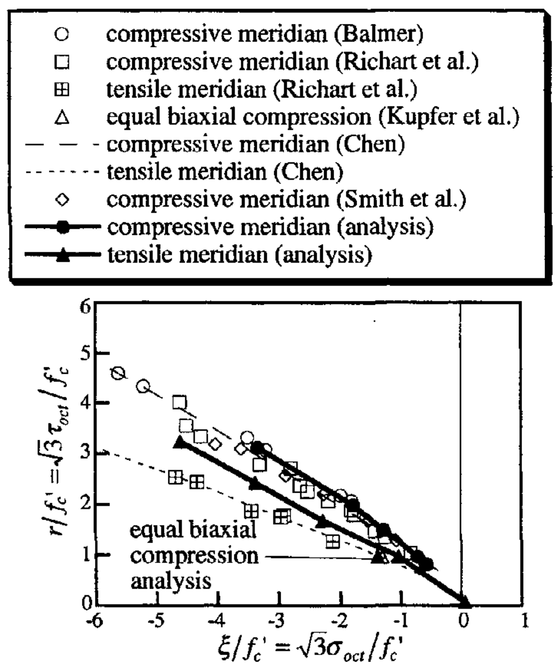


Fig. 12 Compressive and tensile meridians

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A COMPUTATIONAL APPROACH FOR DURABILITY EVALUATION OF CEMENTITIOUS MATERIALS

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ABSTRACT

The computational framework of durability analysis based on microscopic mechanisms are crucial for rational durability design of reinforced concrete, in which performance to be achieved is examined. Thermodynamic couplings of moisture transport, powder material hydration and the microstructure formation phenomena must be the core for it. In this scheme autogeneous shrinkage can be treated in the same manner as drying shrinkage by regarding capillary pore tension as driving force for them. Further, considering corrosion of the reinforcing bars in concrete as one of the primary long-term deterioration causes, the governing equations of moisture and chloride transport through the concrete microstructure are formulated. As a corrosion model a micro-cell approach is applied. Preliminary simulation studies show the versatility and extensibility of the proposed schemes.

INTRODUCTION

The design of concrete structures is typically done based upon structural performance considerations in reality though realizing the service potential of the structure for long term is also expected. This situation exists - not because that the requirements on the structural performance criteria are more stringent, but because sound design methods for the durability design of concrete structures does not exist. The codes of practice have traditionally addressed the durability design based upon a general set of guidelines or "engineering notes". The traditional approach has been generally prescriptive in nature. However, recently it has been realized that the future durability design systems must include the performance-based criteria into the evaluation system [1]. Though elementary, the JSCE durability design proposal which was first issued in 1989 can be termed as one such proposal. In this scheme, the durability of structural concrete is numerically scored as a durability index value. The durability index is obtained by a linear summation rule, which includes various influencing factors like water to cement ratio, water content, slump and spacing of reinforcement etc. The goal of this rather simple performance based design method is to have the durability index higher than a durability limit state value. This concept made clear that the evaluation of the limit state of durability of concrete structure is the core of development.

In the performance based design scheme for reinforced concrete structure, the design process which includes material selection, structural design and construction planning as initial decision making is repeated to find appropriate overall design satisfying requirements. In this design value setting of design items and overall performance checking should be clearly divided and it can make possible free design irrelevant to individual prescriptions and pursuit of benefit due to rationalization. To get these fruits from performance based design however, it is indispensable to establish performance evaluation technology which can predict behaviors of reinforced concrete under arbitrary conditions supposed. Owing to the development of enhanced constitutive laws for concrete and reinforced concrete, it is now possible to numerically predict the structural response and the mechanical states of constituent materials in time and 3D space under any external mechanical action with certain reliability [2]. Especially in the latest standard specification of JSCE for seismic design, it was obligated that the seismic performance of structure must be checked directly by nonlinear dynamic response analysis for important structures. This is because achieving performance is not guaranteed by only regulating strictly the specifications of structure. However, the numerical analysis to simulate transition of properties and states of RC for durability problem is still green. It is the technical problem to be solved on this matter to describe numerically how RC structure behaves without any experiments under arbitrary material properties, mix proportion and environmental conditions. To realize more sophisticated performance based durability analysis systems that would be in harmony with the technological progress of future, we should seek for an integrated durability analysis system of concrete structures.

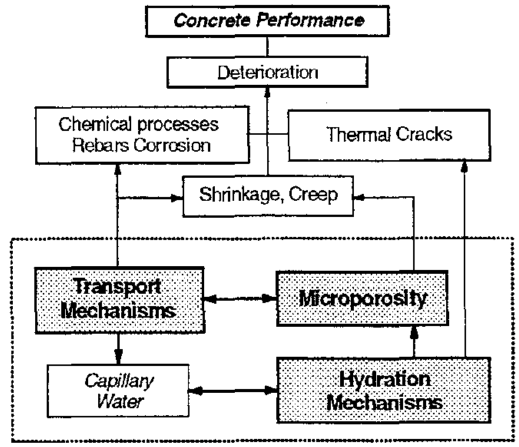

Fig. 1 Framework for the rational evaluation of concrete durability. [4]

This work is a part of the project where the ultimate aim is to seek for a so called life-span simulator of structural concrete based upon the microscopic modeling of concrete in a manner that is similar to the established methods of structural analysis in the field of structural engineering. A preliminary report on this methodology has already been published elsewhere [3]. The method involves an integration of various physical phenomena that occur over the lifetime of a typical concrete structure. The goal is to be accomplished by a computational tool supported by experimental studies in the general framework as shown in Fig. 1. The core of the simulation tool is a coupled microstructure, hydration and moisture transport analysis computations for arbitrary powder materials and structures exposed to arbitrary environmental conditions over the life span. Basically, in this scheme development of the cementitious microstructure, heat generation and moisture transport inside concrete structures can be traced over time and space domains for a given material design and boundary conditions. To extend this system to consider long-term deterioration process of reinforced concrete structures, several phenomena such as various shrinkage and crack occurrence, mass transport, carbonation and corrosion etc. need to be considered. For these further development of the system, numerical modeling and analytical approach for drying/autogeneous shrinkage, chloride ion transport and micro-cell corrosion are discussed in this paper.

MICORSTRUCTURE DEVELOPMENT, HYDRATION AND MOISTURE TRANSPORT THEORY

Most of the deterioration mechanisms of concrete structure, such as cracking due to drying shrinkage, carbonation, corrosion and sulfate attack etc. are related to the water content in concrete. It is therefore indispensable to predict the water content in concrete under any environmental conditions for a rational and quantitative durability design of the concrete structures. For the purpose mentioned above, the inter-relationship of microstructure development, hydration and moisture transport are analytically treated by three dimensional FEM code, DuCOM based upon fundamental physical material models pertaining to each physical process [3,4]. This scheme is supposed to form the core of an analytical durability evaluation method where an integrated approach is taken. This methodology serves as a basis for the quantitative evaluation of parameters relevant for the durability of concrete structures. Considering its initial mix-design, the curing conditions and the environmental conditions, parameters like permeability could be automatically obtained. The integration is primarily done by analytically examining the inter-relationships of hydration, moisture transport and pore-structure development processes. These processes and their interrelationships have been translated into simulation models that can be solved in a dynamically coupled manner. In the framework of durability evaluation system, physical processes related to moisture transport are formulated at the micro pore scale and integrated over a REV to give macro scale mass transport behavior. The hydration process is based on a multi-component hydration heat model of powder materials and is dependent on the free water available for hydration. Thus the average degree of hydration as well as the hydration of each clinker component of cement can be obtained. The development of the pore structure at early ages is obtained using a pore structure development model based on the average degree of hydration. The predicted computational pore structures of concrete are used as a basis for moisture transport computations. In this way, applying a dynamic coupling of pore-structure development to the moisture transport and hydration models, the development of strength along with moisture content and temperature can be traced with the increase of degree of hydration for any arbitrary initial and boundary condition. This integrated approach calls for a highly computation intensive method - a task that could be easily relegated to a computer now. The details of a coupled cementitious microstructure development, hydration and associated moisture transport model under arbitrary environmental conditions have been discussed elsewhere [3]. This technology can be executed on our Web site through Internet (http://concrete.t.u-tokyo.ac.jp/database/ducom).

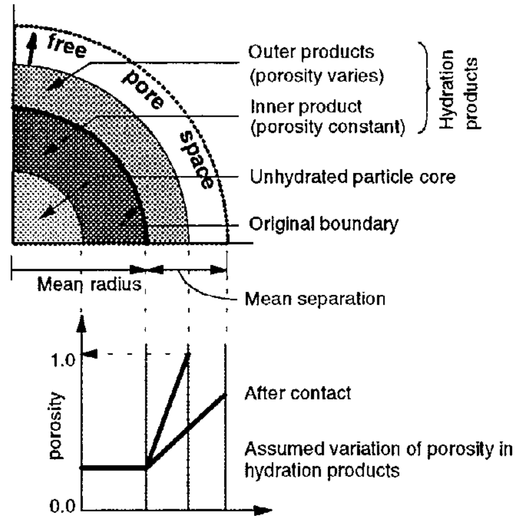

Fig. 2 Microstructure development model [4]

Pore structure development [4]

As a physical basis for pore -structure development computation at early ages of hydration, overall pore space is broadly divided into interlayer, gel and capillary porosity. The powder material is idealized as consisting of uniformly sized spherical particles. Fig. 2 shows a schematic representation of various phases at any arbitrary stage of hydration. Precipitation of the pore solution phase at any surfaces of particles leads to the formation of outer products whereas so called inner products are formed inside the original boundary where the hydrate characteristics are more or less uniform.

Characteristic porosity of the CSH hydrate crystals precipitated as inner and outer products are assumed to be constant throughout the process of pore structure formation. It has to be noted that this porosity includes both the interlayer as well as micro-gel porosity and the capillary porosity corresponds to the space which is not occupied by CSH hydrate crystal in the outer product. Undertaking these assumptions, weight W_s and volume V_s of gel solids can be computed, provided average degree of hydration α and the amount of chemically combined water β per unit weight of powder material are known. By overall volume balance thus, the interlayer(ϕ_l), gel(ϕ_g) and capillary(ϕ_c) porosity are computed as

$$\phi_l = \frac{t_w s_l \rho_s}{2} \qquad \phi_c = 1 - V_s - (1-\alpha)\frac{W_p}{\rho_p}$$
$$\phi_g = \phi_{ch} V_s - \phi_l \qquad V_s = \frac{\alpha W_p}{1-\phi_{ch}}\left(\frac{1}{\rho_p} + \frac{\beta}{\rho_w}\right) \tag{1}$$

where t_w : Interlayer thickness(2.8Å), s_l : specific space area of interlayer, W_p : weight of the powder materials per unit volume, ρ_p : density of the powder material, ρ_s : dry density of solid crystals, ρ_w : density of the chemically combined water. ϕ_{ch} is the characteristic porosity of the CSH crystal assumed as 0.28. In this model, the volumetric change of overall objects accompanied by hydration called as hydration shrinkage is expressed by considering density change of water due to reaction. After surface areas of capillary and gel pores per unit volume of matrix are obtained from porosity of outer product and weight of gel crystals, the overall microstructure is expressed by porosity distribution function $\phi(r)$ as [5]

$$\phi(r) = \phi_l + \phi_g\{1 - \exp(-B_g r)\} + \phi_c\{1 - \exp(-B_c r)\} \tag{2}$$

where, r : pore radius (m). The distribution parameters B_c and B_g can be easily obtained from the computed porosity and surface area values for the capillary and gel region by assuming a cylindrical pore geometry.

Multi-component cement hydration model [6]

The hydration process is simulated using multi-component model for hydration heat of concrete based on cement mineral compounds. In this model the hydration process of each mineral compound present in cement is combined to represent the overall hydration phenomenon. In view of the concept of a multi-component powder material, the effect of arbitrary types of cement or powder materials can be rationally taken into account to predict the overall heat generation rate. The influence of variable moisture content or free water in the hydrating concrete is also taken into consideration. The total heat generation rate of concrete H per unit volume is idealized as

$$H = C\sum p_i H_i \qquad H_i = H_{i,T_0}\exp\left\{-\frac{E_i}{R}\left(\frac{1}{T} - \frac{1}{T_0}\right)\right\} \tag{3}$$

where C : the cement content per unit volume of concrete, p_i : the corresponding mass ratio, H_i : the specific heat generation rate of individual compound, $H_{i,T0}$: the referential heat rate of i-th component when absolute temperature is T_0, E_i : the activation energy of i-th component, R : gas constant, T : absolute temperature at each location. The referential heat rate of each reaction is dependent on the contact probability between free water and unhydrated chemical compounds and it is expressed as function of amount of free water, the thickness of inner hydrated cluster and the accumulated heat of each compounds. The amount of free water adopted is in fact the total condensed water in developing microstructure, which are computed by models of microstructure formation and moisture transport. The free water for hydration is the primary source of non-linear coupling between moisture transport and hydration heat model. In the multi-component hydration heat model the degree of hydration of each compound and the total amount of water consumed per unit volume concrete, $\beta \cdot W_p$ due to chemical reaction are incrementally computed at any point of hydration. State variables described above are crucial parameters for microstructure development model and the amount of water fixed as chemically bound water is considered in mass conservation of moisture in concrete.

Moisture transport process [4]

The ingress of moisture into the pores of concrete is a thermodynamic process, driven by the pressure and temperature potential gradients. In this study, total water present in matrix pores is subdivided into interlayer, adsorbed and condensed water. The amount of water in concrete microstructure in the form of condensed and adsorbed phases can be obtained by integrating the degree of saturation of individual pores over the computed porosity distribution and the modified B.E.T. equation. Interlayer water is differently treated since it is probably under the influence of strong surface forces and does not move readily under the application of pore pressure potential gradient. In this model, the hysteresis behavior of moisture isotherm of concrete where the water content in concrete exposed to the same relative humidity is different under drying and wetting stages is expressed by considering the physical phenomenon called the ink-bottle effect in porous medium [7]. With the water consumption due to hydration at early age of concrete, the overall moisture balance of concrete can be expressed as

$$\rho_L\left(\sum \phi_i \frac{\partial S_i}{\partial P}\right)\frac{\partial P}{\partial t} - div\big(K(P,T)\nabla P\big) + \rho_L \sum S_i \frac{\partial \phi_i}{\partial t} - W_p \frac{\partial \beta}{\partial t} = 0 \tag{4}$$

where, ϕ_i : porosity of each component(interlayer, gel and capillary), S_i : degree of saturation of each component, P : equivalent liquid pore pressure, ρ_L : density of pore water. Moisture conductivity K is obtained from the flux models using the computed porosity distribution function [4].

MODELING FOR DRYING AND AUTOGENEOUS SHRINKAGE [5, 8]

Under equilibrium conditions moisture content of a porous media is dependent on the ambient relative humidity. This is because, for a given relative humidity certain group of pores whose radii are smaller than the pore radius in which liquid-vapor interface is formed are completely filled with water, whereas lager pores remain empty or partially saturated. Liquid water in fine pores develops a curved interface with vapor, because of the cohesion and adhesion of water molecules. Considering local thermodynamic and interface equilibrium in a porous media, the relationship between relative humidity and the pore size in which the interface is created can be determined by the Kelvin's equation as

$$\ln\left(\frac{P_v}{P_{v0}}\right) = -\frac{2\gamma M_w}{RT\rho_L}\frac{1}{r_s} \tag{5}$$

where, P_v/P_{v0} : relative humidity of vapor phase, γ: surface tension of liquid water (N/m), M_w : molecular mass of water (kg/mol), R : universal gas constant (J/mol·K), T : absolute temperature of the vapor liquid system (K), ρ_L : density of liquid water (kg/m^3), r_s : the radius of the pore in which the interface of liquid and vapor is created (m). Here, the pressures of vapor and liquid phases at their interface are not equal. This fact can be hydrostaticically explained from the view point of the surface tension of liquid water. The pressure gap between vapor and liquid phases is described by the Laplace's equation as follows.

$$\Delta P = P_G - P_L = \frac{2\gamma}{r_s} \tag{6}$$

where, P_G, P_L : pressures in vapor and liquid phases (Pa). Since the pressure of liquid phase is lower than vapor phase due to the surface tension of liquid water, tensile stress should be applied on pore walls where contact is made during the liquid phase. Based on the concept of the transmitted stress between water and the solid wall of the pore structure, it is assumed that this stress causes the macroscopic shrinkage of concrete. The total intensity of the tensile stress per unit concrete volume dependent on both the magnitude of the tension and the area is formulated as

$$\sigma_s = A_s \frac{2\gamma}{r_s} \tag{7}$$

where, σ_s : capillary stress (Pa), A_s : area factor which represents applied area of capillary stress (m^3/ m^3) . In this study area factor is given as the amount of liquid water per unit concrete volume computed by moisture transport and content model with distribution function of porosity. For deformation due to capillary stress, a simple linear elastic relation is assumed between evaluated capillary stress and volumetric change as follows.

$$\varepsilon_{sh} = \frac{\sigma_s}{E_s} \tag{8}$$

where, ε_{sh} : drying shrinkage strain (including autogeneous shrinkage strain), E_s : elastic modulus for capillary stress (Pa).

Model simulations and results [8]

The relative humidity of pore in concrete decreases due to drying caused by difference of relative humidity between pore in concrete and surroundings. However, even under isolated condition it is originated by so called self desiccation due to hydration accompanied by water consumption. In the proposed scheme, there is no distinction between drying and autogeneous shrinkage. According to environmental and curing conditions applied to structure, water content and microstructure development are computed and then those shrinkage can be computed by eqn. 8. In this study the elastic modulus for internal capillary tension, E_s is assumed as one fourth of that for external force, E_c [5]. By the porosity computed in time and space domain, the compressive strength of concrete is expressed as [8]

$$f_c' = A\exp\left(-B\cdot V_{pore}\right) \tag{9}$$

where, f_c' : compressive strength (MPa), V_{pore} : Volume of pores per unit volume of paste (m^3/m^3), A, B : constants. Here, the total volume of pores over 50nm is adopted for V_{pore} since it is well known in concrete

Table 1 Input data for analyses and experimental conditions

No.	Type of Portland cement	W/C	Dimensions (cm)	Curing condition
1 (concrete)	Ordinary	30 %	10×10×120	Sealed
2 (concrete)	Ordinary	30 %	10×10×120	Dried at RH 60 %
3 (mortar)	Early hardening	30 %	10×10×40	Dried at RH 50%
4 (mortar)	Early hardening	30 %	10×10×40	Sealed
5 (mortar)	Ordinary	23 %	10×10×16	Sealed and Dried at RH 60 %
6 (mortar)	Ordinary	55 %	10×10×16	Sealed and Dried at RH 60 %

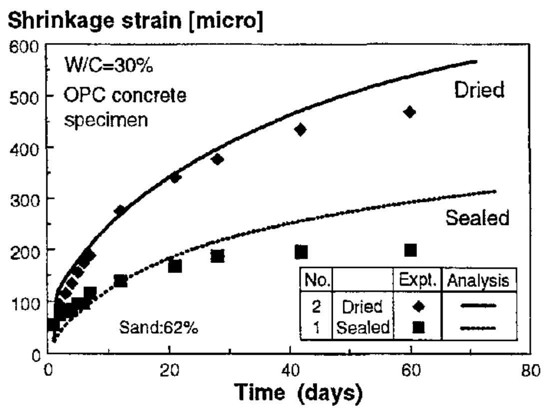

Fig. 3 Shrinkage behaviors in concrete

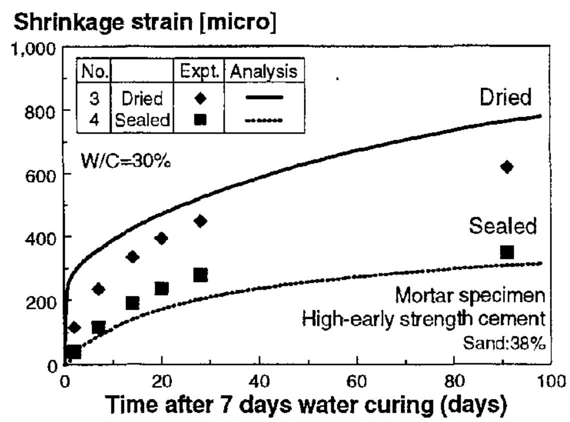

Fig. 4 Shrinkage behaviors in mortar

and mortar that compressive strength has high correlation with volume of pores from 50nm to 2μm. The elastic modulus E_c is calculated from compressive strength by empirical formula.

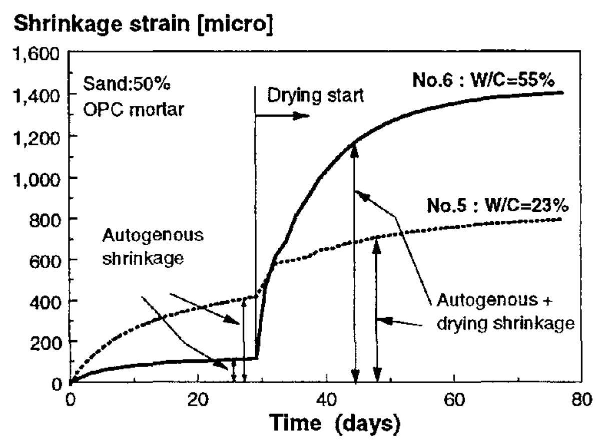

Fig.5 Shrinkage behaviors computed in different W/C

Several verifications and simulation are conducted and mix proportions as shown in Table 1. Input data for computation includes only mix proportions, type of binder, dimensions of specimen, curing and surrounding conditions. In these analyses average value of free shrinkage strain at each location is given as overall shrinkage of specimen without stress analysis. Fig. 3 and 4 show comparisons of experimental and computed shrinkage both sealed and exposed to ambient relative humidity in concrete and mortar respectively. In Fig. 3 ordinary Portland cement is used for concrete and shrinkage are measured after 24 hours sealed curing. In Fig. 4 early hardening Portland cement is used for mortar and shrinkage are measured after 7 days water curing. It can be seen that shrinkage behaviors are reproduced successfully in both of sealed and dried cases. It should be noted that in the case of shrinkage measured in drying condition at early age, experimental result includes both of drying and autogeneous shrinkage. Further, as a case study of the proposed scheme, autogeneous shrinkage from casting to 28 days in sealed curing and following shrinkage under drying condition are computed. Two different water cement ratio cases, 23 % and 55 % are computed respectively as shown in Fig. 5. RH 60 % is applied for drying curing. The qualitative tendency well known about shrinkage behavior between conventional concrete and low water to cement ratio concrete can be observed in this study. That is to say, in the case of lower water to cement ratio shrinkage under sealed condition at early age is relatively large and following shrinkage with drying does not increase so much, whereas in the higher W/C case shrinkage after exposure to ambient RH is remarkably larger in spite of smaller shrinkage observed under sealed condition.

CHLORIDE ION TRANSPORT AND CORROSION IN CONCRETE [9]

Chloride salts are well known to assist in the corrosion phenomenon of concrete reinforcements. The presence of chloride salts in the concrete leads to the destruction of the passive layer provided by an alkaline environment around the bars. Furthermore, presence of chloride ions might accelerate the corrosion process since, the pore solution rich in chloride serves as an electrolyte and reduces the conductivity of concrete around corrosion zone. It is important therefore, that any analytical durability evaluation system should incorporate the chloride ion transport into the same framework to forecast the expected service life of a structure. Traditionally, the movement of chloride ions in concrete is considered to be a diffusion process where the transport occurs by the ion concentration gradient. Fick's law of diffusion has been used to simulate the ion penetration in the concrete [10]. There have also been attempts to empirically correlate the Fick's diffusion coefficient with concrete properties like water to cement ratio and strength etc. Such treatment however neglect the bulk movements of pore water that might occur in real life situations for alternate drying wetting conditions. This has led to a wide scatter in the apparent diffusion coefficient data for similar concrete when exposed to different environmental conditions. While a simple diffusion theory based approach might be suitable for relatively static conditions like completely submerged concrete, the bulk transport of the pore water as the medium carrying chloride ions must be considered for a more general treatment of the problem. The problem is more important for cases where the concrete cover might contain cracks and the salt movement due to bulk movements of pore water may far outweigh the movement due to simple Fick's diffusion.

Ion transport formulation [9]

Salt transport in porous materials under usual conditions is an advective-diffusive phenomenon. Apart from the diffusion of ions and molecules of the salts in the pore solution phase due to concentration difference, there is an associated advective transport due to the bulk movement of pore solution phase. In a three dimensional porous continuum, the flux of ions can be therefore expressed as

$$\mathbf{J} = \phi S\left(-D\,\nabla C + \mathbf{u}\,C\right) \tag{10}$$

where, $\mathrm{J}^{\mathrm{T}} = [\, J_x \quad J_y \quad J_z \,]$: flux vector of the ions (mol.m^{-2}.s^{-1}), ϕ : porosity of the porous media, S : degree of saturation of the porous medium, D : diffusion coefficient of the chloride ions in pore solution phase (m^2.s^{-1}), $\nabla^{\mathrm{T}} = [\, \partial/\partial x \quad \partial/\partial y \quad \partial/\partial z \,]$: the gradient operator, $\mathrm{u}^{\mathrm{T}} = [\, u_x \quad u_y \quad u_z \,]$ is the advective velocity of ions due to the bulk movement of pore solution phase (m.s^{-1}), C : concentration of ions in the pore solution phase (mol.l^{-1}). In the case of chloride ion transport in concrete, S represents the degree of saturation in terms of the free water only, as adsorbed and interlayer components of water are also present. Furthermore, it is a well-known fact that chlorides in cementitious microstructure have a fixed (bound) and free components. The bound components exist in the form of chloro-aluminates and adsorbed phase on the pore walls, essentially making them unavailable for free transport and subsequent corrosion of bars. The parameter C represents the free chloride component. It has been suggested that the free and bound components of chlorides in concrete are under an equilibrium condition given by the relationship [10].

$$\alpha_{fixed} = \begin{array}{ll} 1 & C_{tot} \leq 0.5 \\ 1-0.5(C_{tot}-0.5)^{0.39} & 0.5 \leq C_{tot} \leq 4.5 \\ 0.141 & 4.5 \leq C_{tot} \end{array} \tag{11}$$

where $C_{tot} = C_{free} + C_{bound}$; is the total amount of chloride expressed in weight percent of the amount of cement per unit volume of concrete, $\alpha_{fixed} = C_{bound} / C_{tot}$: equilibrium ratio of fixed chloride component to the total chloride ion components. In this paper, eqn. 11 has been used to obtain an instantaneous equilibrium relationship of free and bound chloride ions. That is, it has been assumed that a local equilibrium between bound and free chloride is reached immediately. Furthermore, at low salt concentrations, the chloride ion velocity components might be quite close to the actual bulk velocity of the pore solution phase. However, at increased concentrations ionic interaction effects may arise in the fine cementitious microstructure, thereby reducing the apparent bulk velocity of chlorides. The exact mechanisms, for this phenomenon are not understood clearly at this stage. Therefore, following expression is used to represent the actual advective component of chloride ion transport.

$$\mathbf{u} = \tau \mathbf{u}_w \tag{12}$$

where, u : velocity vector of chloride, u_w : the velocity vector of pore solution phase obtained directly from moisture transport formulation, τ : the reduction factor representing the interaction effects in the pore solution phase (0.3). Using eqn. 10, the mass balance condition for free chloride can be expressed as

$$\frac{\partial}{\partial t}(\phi S\, C) + \nabla^{\mathrm{T}}\mathbf{J} + Q_C = 0 \tag{13}$$

where, Q_C : the rate of binding or the change of free chloride to bound chloride per unit volume of concrete ($mol.m^3.s^{-1}$). Theoretically, it should be possible to obtain Q_C from eqn. 11. However, this relationship does not appears conducive to the computation of rate terms especially at low free chloride content. Tentatively, we have assumed Q_C as zero and assumed an equilibrated relationship of eqn. 11 to obtain bound chloride directly, once the free chloride content is known. It must be noted that eqn. 11 does not explains the condensation phenomenon observed in practice. That is, the experimentally obtained "free" chloride concentrations near the exposed surfaces of concrete could be much higher than that would be expected from simple equilibrium conditions of environmental liquid and pore solution concentrations. It has been proposed, that a significant amount of chloride might exist as the adsorbed phase that does not takes part in the transport, but are observed as free chloride when pore water is squeezed out of the concrete. The relationship of such adsorbed and/or condensed chloride to the actual chloride ion concentration of the pore fluid is currently not known. However, it is expected on intuitive grounds that the adsorbed phase of chloride would be generally a function of the pore solution chloride ion concentration and the amount of pore fluid existing in adsorbed phased. In this paper, no attempt has been made to formulate such a relationship and it is seen as a part of the future work. Tentatively we assume the relationship $C = C_{free}$. Coupling eqn. 13 with the moisture transport formulation [4], it is possible to obtain the distribution of free and bound chloride in the concrete with time. This information could be used for predicting the relative risks of corrosion based on the rate of loss of passivation layer and estimates of corrosion currents. In this paper, we have used a simpler micro-cell based model that has been proposed to explain the corrosion phenomenon of steel bars.

Micro-cell based corrosion model [11]

The corrosion induced due to chloride ions often result in the formation of 'macro-cells'. In these systems, the concentrated and corroding anode can be quite separated from the cathodes. High moisture content, low resistivity of the concrete and localized unavailability of oxygen are conducive to the formation of macro-cells, that often lead to localized pitting and loss of cross-sectional areas of the reinforcing bars. However, for more or less uniform conditions of moisture and oxygen availability over the length of the steel bars in concrete, the resulting corrosion may occur uniformly with the formation of red-rust. In this paper, the rate of corrosion is obtained from an empirical rate type equation that considers the dependence of corrosion rate on the availability of water, oxygen and chloride. Mathematically, it can be expressed as the corrosion rate of unit surface of the reinforcement per unit time, as

$$q = k\left(\alpha_0 + a\,C\right)\phi S\, C_{O_2} \exp\left\{-\frac{E}{R}\left(\frac{1}{T} - \frac{1}{T_0}\right)\right\} \tag{14}$$

where, q : rate of corrosion ($g.cm^{-2}.day^{-1}$), k : conversion factor = 55.8×2/32.0, α_0 : referential rate coefficient (0.151 $cm.day^{-1}$), a : coefficient for sensitivity of chloride effect (16.9), C_{O_2} : concentration of oxygen in the pore solution phase ($gm.cm^{-3}$), T : Temperature (K); E : activation energy. Currently, not sufficient data exists for the determination of activation energy parameter E. In all the subsequent analysis, we have assumed the temperature T as uniform and equal to T_0 (293K) to neglect the temperature effects. It has been assumed that the pore solution phase has sufficient availability of oxygen in all conditions.

Furthermore, C_{O_2} value has been tentatively fixed as 9.31×10^{-6} g.cm^{-3} in all the analysis of this paper. The consideration of oxygen diffusion in the same framework is seen as a part of the future work. The treatment in use of this model should be considered qualitative as the role of chloride ions as the driving force for corrosion, beyond the de-passivation of the protective alkaline passive layer is not clearly expressed in the analytical model. Also, for a more complete analysis of corrosion in structures, where various zones might be exposed to different environmental conditions that may lead to pitting effects, macro-cell based analysis models are required. In-spite of these shortcomings, the use of eqn. 14 would generally enable us to search qualitatively the relative risks of corrosion, given a choice of initial materials, reinforcement layout, curing conditions and exposure history.

Model simulations and results [9]

The chloride transport and the relative corrosion rate models were installed into a three-dimensional finite-elements based computational program DuCOM. Since, microstructure at any location and the moisture transport parameters are obtained as a natural solution of the computational scheme; these can be directly used in the salt transport and corrosion models. Few cases have been simulated to show the viability of the scheme. First case is the analysis of chloride transport into an initially dry concrete, under alternate drying (3 days) and wetting (4 days) cycles. The aim of this simulation is to show the rapid ingress of chloride in concrete due to the rapid suction of pore water under wetting phase. Wetting is simulated by exposing one face of the member to a chloride solution of 1.4 mol/l. Drying is simulated by an environmental relative humidity of 50%. The simulation clearly shows a deep ingress of chloride within 150 days (Fig. 6), that also roughly corresponds to the test results [10].

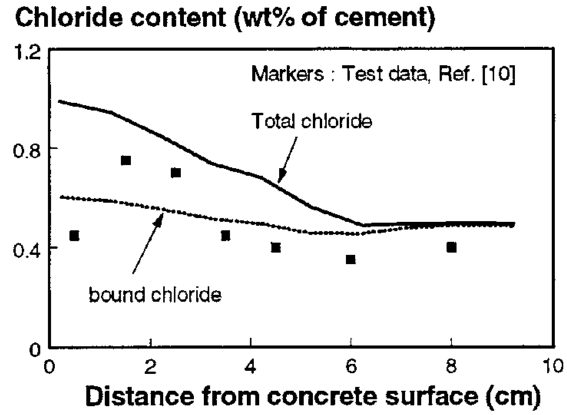

Fig. 6 Chloride content profile in concrete exposed to cyclic wetting an drying.

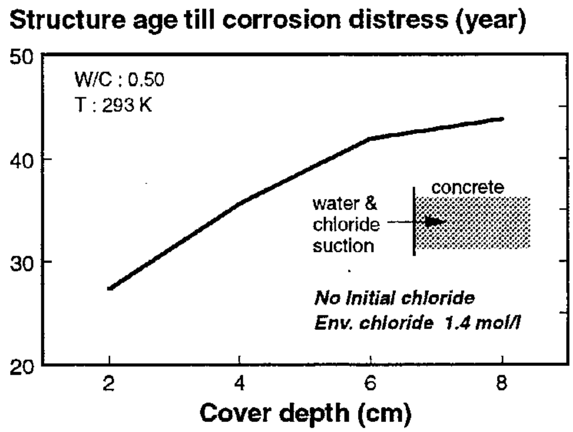

Fig. 7 Time till first signs of corrosion distress for concrete exposed to salty water.

Next, simulation results are shown in view of the effect of chloride ions on the rate of corrosion. In this paper, we assume that about 0.05mm of rebar loss would show an initial corrosion related distress of the structure. In the subsequent simulations, we have computed, the time required at different locations inside concrete, for the corrosion level to reach a level of 0.05mm that roughly corresponds to about 0.04g/cm^2 of steel. The total corrosion loss can be obtained by integrating eqn. 14 in time domain and coupling it with the moisture and chloride ion transport model. It is reiterated here that a full availability of oxygen in the pore water is assumed here, tentatively. For this simulation, a one-dimensional concrete member of 0.5 w/c ratio with only one face exposed to the environment has been considered. The boundary condition is an exposure to environmental fluid containing 5% chloride by weight. The member was initially dried at 0.5 relative humidity for 600 days before the salt-water exposure. The simulation shows that the concrete nearer to the exposure zone would show early signs of distress. Interesting observation is however, that the subsequent increase in service life decreases as the cover depth is increased. In fact, corrosion would be imminent at certain stage in such conditions, irrespective of the cover depth (e.g., about 45 years in Fig. 7). Proper preventive measures against corrosion, beside a reasonable cover depth are absolutely essential for such structures.

CONCLUSIONS

The combined system of micro-structure development, hydration and moisture transport was extended to have more versatile framework for durability prediction of RC structure. Based on microscopic mechanism volumetric changes due to drying shrinkage as well as autogeneous shrinkage can be treated uniformly. Further, as a preliminary attempt the salt transport and subsequent corrosion phenomenon are integrated into the same computational framework. It is hoped that through this computational approach it might be possible to make a more informed guess for predicting the durability service life of concrete structures under arbitrary external and internal control conditions, in a rational way. As for future development, required is a combination of these integrated theory with the structural mechanics model of reinforced concrete for examining total performance of reinforced concrete from birth to death.

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CRITICAL RESEARCH NEEDS IN STEEL BRIDGES

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INTRODUCTION

The major challenges in the maintenance and expansion of our highway infrastructure in steel bridges is the prevention of failures due to fatigue crack extension and corrosion of members. Traditionally design has emphasized strength. Corrosion and fatigue are often classified as serviceability issues and are not given the same consideration as the static strength of the structure. During the last thirty years, most of the bridge failures that have occurred have either been due to extension of fatigue cracks or loss of section due to corrosion and pack rust at hanger joints. The large failures which have occurred by earthquakes have lead to demolition and replacement of deficient structures, large scale retrofit programs, and revised design standards. The less rapid degradation of our structures due to cracking and corrosion has not triggered a significant change in the design and construction methods employed in the design of the structures. As we plan for the future, we must recognize the need to develop new strategies to maintain the existing structures and for development of new design criteria.

FATIGUE CONSIDERATIONS

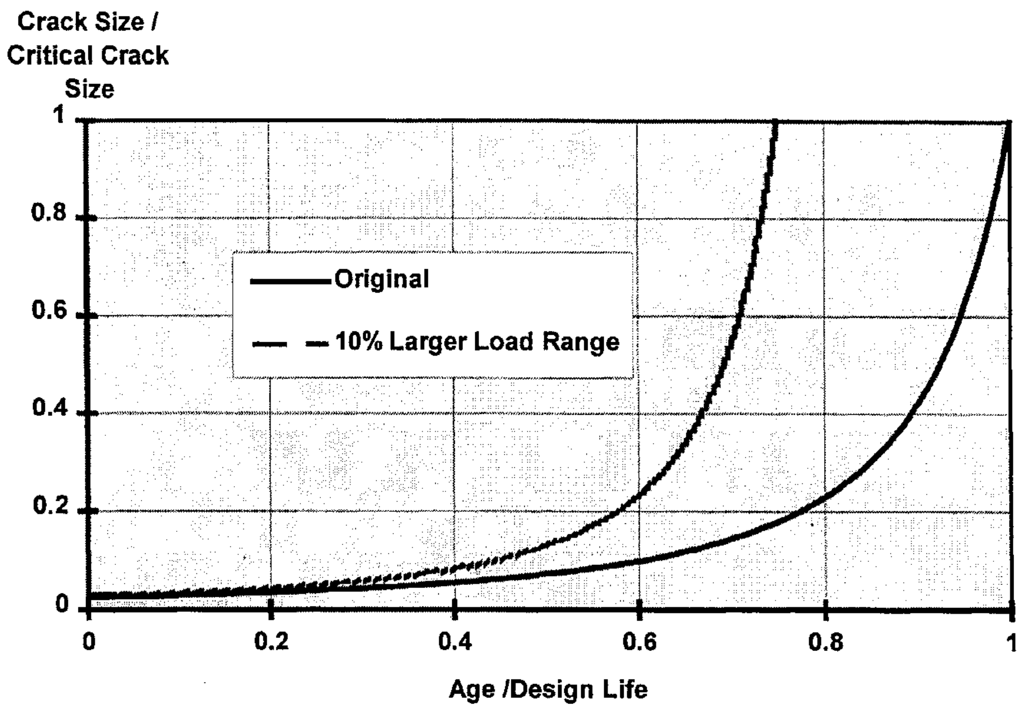
The fatigue life of a structure is dependent upon the magnitude of the stress concentrations and initial crack like discontinuities at a connection detail, the magnitude of the applied stress range, and the rate of application of the loading. The relationship between typical joint details employed in bridge construction and the fatigue life is reasonably well understood. Most structural codes have a classification system which provides the designer with knowledge of the fatigue severity of the connection detail. The stress range at the detail is adjusted in the design to provide the required design life. The design life in the U.S. is between 50 to 100 years.

The two key assumptions in the fatigue design process are that the vehicular loading and traffic count of the trucks will remain constant over the life of the structure. The designer has no control over these variables. Often the number of vehicles assumed in the original design are surpassed within the first year of operation. Economic pressure is placed upon the governing bodies to increase the legal vehicle weight of trucks or to ease restrictions and fees for overweight vehicles. The cost of the increased damage to the bridge is often not considered in the decision making process nor in the charge for the permits. The economic benefit of increased truck pay load versus reduced service life needs to be carefully considered.

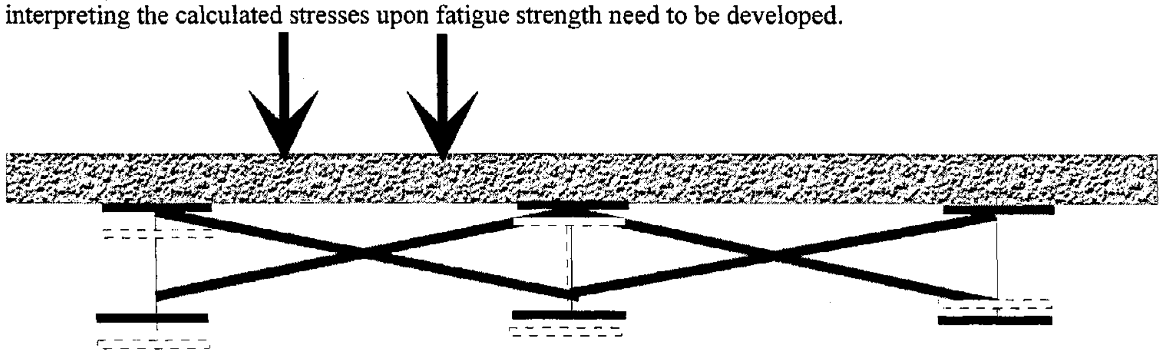
A further issue is the future of vehicular traffic in the design life of the bridge. Who knows the type and size of vehicles which will use our roadway 100 years from now? Within the past 100 years we have seen the development of the modern transport truck which has replaced the train and horse drawn wagons as the primary means of moving goods on the surface. Will we see a return to trains in the future and therefore a reduction in truck traffic? What will be the source of fuel in the future for the vehicles using our roadways?

It is very likely that the future transport vehicles will be heavier. Heavier vehicles are more efficient and therefore more economical. We can see the migration to heavier trucks in the increase in the number of permit loads allowed on our highways. If the truck loads increase and the truck rate stays the same or increases a significant reduction in the fatigue life of our bridges will result.

The life of welded steel structure subjected to cyclic loading is proportional to the inverse of the cube of the applied stress range. Once a structure is in service, the primary method of extending the structures fatigue life is the reduction of the applied loading. In typical highway bridges in the U.S., the opposite has been true. The loads as well as their frequency on the bridges has increased. The change in fatigue life from a 10% increase in load range is shown in the figure below. The life to failure is 75% of the original design. The difference in crack size in the early part of the structures life is not changed significantly by the increase in load. Consequently, Initial inspections will not indicate the substantial loss in fatigue life from this increase in load.



Many of the fatigue cracks that have appeared in bridge structures are caused by secondary or distortion induced stresses. These are stresses which are ignored in standard bridge design practice. Some like the cracking at floor beams and transverse stiffener welds at cross frames are not calculable in the one dimensional analysis often used in bridge design. Investigations into the cause of these cracks have shown that modern three dimensional analysis techniques can be used to estimate these stresses and to develop retrofit schemes. The differential deflections of the girders introduces the forces into the transverse cross frames and floor beams spanning between the girders. Obviously, considerable improvement in the life of these bridges could be gained by using an analysis model in the design process which more closely represents the actual behavior of the bridge. The required modeling parameters and the methods of interpreting the calculated stresses upon fatigue strength need to be developed.



Differential Deflection Introduces Forces in Cross Frames and Floor Beam

New connection methods with improved fatigue performance need to be developed. Hybrid systems using adhesive with powder actuated fasteners look promising for thin material. Similar systems for larger components need to be developed. The economy of higher strength steels in shorter span structures cannot be utilized until more fatigue resistant methods of connection are developed.

CORROSION

Corrosion of steel structures is a major concern in the northeast of the U.S. This area has some of the oldest structures and a cold climate which requires the use of road salt. Bridge decks in the northeast often last less then twenty years. Numerous approaches to increasing the service life of the structural elements have been tried. Improved barrier coating have been developed for reinforcing steel and the structural steel. These systems generally are more costly and more stringent VOC regulations have reduced or prevented the application of some of the most effective systems. Combinations of barrier and galvanic protection systems such as zinc rich paint offer improved coating life and tolerance to coating holidays. Future regulations may limit the use of zinc which will limit the application of these coatings as well as hot dip galvanizing.

Painting of in service structures has become very costly due to regulations on the lead contained in the old paints. The cost of the removal of the old coating will exceed the cost of the application of the new coating. Coatings which do not require removal of the existing coating have been introduced into the market but further development is needed.

Corrosion at expansion joints continues to be a problem even in areas where deicing salt is not used. One solution has been the elimination of joints. A durable, reliable, and easily repaired joint is needed. The present designs are often proprietary and based upon very little independent study of their performance. Design standards for the joints are needed which consider the durability of the joints, the required expansion performance, and the ability of the joints to clean themselves.

RESEARCH TOPICS

A list of research topics which can be used to develop new methods of reducing and forecasting the strength of our highway bridges is given below. Shared and coordinated research between the U.S. and Japan on these topics will lead to an improved and more economical infrastructure.

Fatigue:

1. Forecast of Future Fatigue Loads for Highway Bridges

    The structures we are designing and maintaining at present are expected to remain in service into the 21st century. The present U.S. design loadings are based upon a hindcast rather then a forecast of future loading. A study of expected future truck configurations and traffic should be undertaken and the results evaluated to determine the required static and fatigue resistance of our bridges for these future loads

2. Methodology for the Determination of Fatigue Damage Based Costs for Overweight Vehicle Permits

    The majority of the cumulative damage done to our highway bridges occurs from vehicles which exceed the legal weight laws and also exceed design loads. These heavy vehicle travel on our roads legally by obtaining a special permit. The cost of these permits is not related to the damage caused by these vehicles. A study to develop a cost structure for the permits based upon the amount damage caused by these vehicles is needed.

3. Development of Three Dimensional Modeling Standards for The Fatigue Assessment of Highway Bridges

The majority of the fatigue cracking occurring in highway bridges is due to stresses and forces not considered in the normal one dimensional bridge design procedure. The interaction of cross frames and floor beams with the girder system when the bridge cross section is loaded non uniformly is not considered in present design methods. Modern three dimensional analysis techniques can be used to determine these forces. However, the method of integrating these complex analysis techniques with present loading rules has not been established. A study to develop recommend methods of analysis and loading is needed.

4. Development of Connection Techniques with Improved Fatigue Strength

High strength steels have no advantage in shorter spans due to the increase in the live load stress range from the reduction of the size of the bridge members. The fatigue strength of the welded connections which is the same for low and high strength steel limit the live load stress range. New connection methods with improved fatigue performance must be developed to take advantage of the higher strength and toughness of these modern steels. A survey of available connection techniques should be undertaken. Based upon the survey, laboratory studies of the most attractive methods should be undertaken to determine their fatigue performance, costs, and applicability to large construction.

Corrosion:

1. Improved Coatings for Covering of Corroded Steel

Development of new techniques and coatings which encapsulate the existing lead based coating and prevents further corrosion are needed to prevent the further deterioration of existing structures. The cost to remove the existing lead based paints is prohibitive. A coating system which is easily applied and has corrosion protection comparable to a conventional coatings is needed.

2. Durable Coatings with Low VOC's

Paints with low volatile organic solvents, VOC, are needed if shop and field painting of structural steels is to continue. An ideal paint would have nil VOC and provide protection and ease of application comparable to existing coatings.

3. Corrosion Protection vs. Environmental Concerns for Zinc Rich Paints and Galvanizing.

Zinc coatings have long been used to protect steel structures from corrosion. Galvanized sign structures and guard rails are used through out the highway system. Zinc rich paints are commonly specified for structural steel in coastal environments. Concern over the toxicity of zinc has been raised. Some manufacturers are planning on phasing out the use of galvanizing for corrosion protection. A study of the use of zinc coatings in the highway industry is needed to develop the environmental risks associated with its versus the benefits of zinc coatings corrosion protection.

Damage of Connections in Steel Buildings

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Abstract

In the Hyogoken-nanbu earthquake, which caused serious damage to the densely populated Hanshin megalopolis (Kobe city and its vicinity), complex natures of damage sustained by steel structures pose new and highly perplexing challenges for steel engineers. Various types of damage in steel structures were found around connections. In the worse, many ruptures and cracks were detected in the welds. This paper describes such damage in detail and discusses possible causes. Finally, some improvements are suggested in order to avoid serious results due to severe earthquakes.

1. Introduction

Steel is popular in building construction. Steel has a large share in Japan than in Europe and in U. S. A. Figure 1 shows the total constructed floor area per year with respect to various structural materials. Wood has been ranked first for years, but it gave the first place to steel recently. Sudden drop since 1990 is due to the current recession. Figure 2 shows the total floor area of steel buildings constructed each year with respect to the number of stories. It suggests that more than 90% of steel buildings are shorter than 5 stories. These are offices, shops, shop and residence combined, warehouses and industrial facilities. Most of such steel buildings are constructed in large cities. And, it is of our fortune, strong earthquakes have not attacked the crowded large cities and brought us few experience that steel buildings suffer a lot of damage due to earthquakes until the Hansin-Awaji earthquake.

Immediately after the earthquake, the steel committee of the Kinki Branch of the Architectural Institute of Japan (AIJ) conducted a detailed survey into the damage to steel buildings. According to the report, 90 were rated as collapsed, 332 as severely damaged, 266 as moderately damaged and 300 as slightly damaged among 988 building damaged. Figure 3 shows the number of buildings with respect to the damage level, indicating that the severely damaged including the collapsed are 2 to 5 story. The number of the severely damaged buildings is a few in the taller than 10 stories though the total number of the taller buildings constructed is also small.

The damage in low-rise buildings was founded around connections such as the beam-to-column connections, particularly being attracted attention as it was after the Northridge earthquake in U. S. A. While, severe damage in tall buildings are not reported until now, though an exceptional case was precisely examined as discussed later. The above-mentioned two types of damage are described.

2.Damage in Low-rise Buildings

<u>2.1 Types of damage</u>

Damage was found in various portions of building structures; columns, beams, beam-to-column connections,column base connections, braces and brace connections. In the past earthquakes, failure in members due to severe plastic buckling and fracture at the brace connections were already found and reported. Fracture around the beam-to-column connections, however, was found this time in so many buildings, particularly in moment resistant frames. Most of them consist of wide-flange beams and columns of cold-formed square-tube sections. These types of frames have been used more frequently for the past 15 years. The moment-resistant frames without brace are commonly chosen for office buildings. This trend was accelerated after the revision of the seismic design regulation in the Building Standard Law, probably because the square-tube sections available in the market can provide necessary moment resistance strength, and also the design procedure becomes very simple in unbrace frames. Therefore, this type of connection has widely spread before the earthquake.

2.2 Damage in beam-to-column connections

The widely-used details of the beam-to-column connections in damaged buildings are shown in Figures 4 (a) and 5 (a). In the through-diaphram connection, a long square tube is cut into three pieces; the upper column, the connection's panel zone and the lower column. Two diaphram plates inserted are shop-welded to the column ends and the panel zone ends. The details in Figures 4 (a) and 5 (a) are distinguished each other by the beam end connections. In Figures 4 (a), the brackets are shop-welded to the ends of the diaphragm and then the brackets are connected to the beams by bolt connections, while the beams in Figure 5 (a) are directly site-welded to the diaphram ends. The statistics of the damage types is shown in figures 4 (b) and 5 (b). In the shop welding case, fracture or crack is found at 3.3% welded joints out of 2,396 inspected joints. There are several types of the cases of fracture, but in 37.2% of the ruptured joints the crack starts from the tack weld of the run-off tab as shown in Figure 6. On the other hand, fracture or crack is found at 8.5% of welded joints which are site-welded. Almost a half of the ruptured cases show the cracks initiated at the crater of the weld metal. The failure of the beam flange started from the weld access hole as shown in Figure 7 is not dominant in these statistics.

3. Damage in High-rise Buildings

3.1 Damage survey of high-rise buildings

According to the regulation attached to the Building Standard Law, structural design of all buildings taller than 60 m must be reviewed at the Tall Building Appraisal Committee at the Building Centar of Japan. 53 projects planned in Hyogo Prefecture were reviewed so far. The Appraisal Committee carried out the interview with the designers and the constructors who were involved in the above projects. The responses to the interview can be summarized as most of them suffered minor damage or nothing. Only one exception is the Ashiyahama residential complex, where there are 4 types of buildings different in their heights. 19 story and 24 story buildings suffered serious damage as shown in Table 1. The design base shear coefficients CB and the fundamental periods T of these 4 types are summarized in Table 2. Probably the frames which fundamental periods are around 2 seconds were vibrated strongly by the ground motion there.

3.2 Structural damage to the Ashiyahama residential complex

The Ashiyahama residential complex has a particular structural configuration as shown in Figure 8. The truss-typed beams and columns form a so-called mega-frame. The members of the beams and the columns are not covered by the fire protection material and exposed into oped air. The apartment unit made of pre-cast concrete panels are piled into the mega-frames. Most serious damage occurred at the vertical chord members of the truss-typed columns. The chord members have square hollow sections, the plate thicknesses of which are 16mm to 55mm. The members were connected by site-welding. Some members made of plates thicker than 32 mm are ruptured at sections away from the site-connections, while some members made of thinner plates less than 28 mm are exclusively ruptured at the site-welded connections.

The types of fracture are summarized in Figure 9. These ruptured sections are now tentativery repaired by providing 4 vertical stiffners at two opposite surfaces of the chord members.

4. Possible Causes of Failure and Countermeasure

4.1 Brittle fracture

Structural engineers know well steels are ruptured finally, but the rupture takes place after considerable plastic deformation in usual structural steels. After the earthquake,however, it poses highly perplexing challenges for engineers that members and connections in steel structures may rupture in the brittle manner. Many research projects launched in order to identify real causes of the fracture or possible conditions that initiate the fracture: Factors associated with member's shape and dimensions might be much related to the conditions. Also, the distortion speed due to the violent quake motions might be one of those. Temperature at members and connections, in some cases, may play an important role. Of course, this fact is a fundamental knowledge in the fracture mechanics, but it is not taken into consideration in structural design of usual buildings.

4.2 Concentration of plastic deformation

It is widely recognized that the stress level at the weld metal in the beam flange must be reduced, providing a horizontally tapered flange shape or enhancing the connection strength with a bracket below the lower beam flange. From the viewpoint of seismic design of the frame, structural frameworks must have enough redundancy until collapse and be provided with a multi-phase seismic resistance [2]. These two requirements reflect large variety in earthquake intensity and various waveforms of ground motion. We should disperse earthquake resistance elements in the structures as widely as possible. Structural performances required in coming seismic design may be partially fulfilled by utilizing so-called seismic devices. There are a lot of seismic devices invented and utilized in actual design. One of key performances required is how to dissipate energy induced by earthquake excitation. Most reliable energy dissipation is provided by plastic deformation.

5. Further Research Needed

The damage analysis brings us a lot of problems to be solved. Namely,

1) Material issues including
Upgrading the quality of steels and welds in trend of increased use of weld connections and ensurring the potential of inelastic deformation and fracture toughness.

2) Fabricationand erection issues including
Welding procedures and processes in the shop and the site,
Inspection and quality control in welding.

3) Design issues including
Identifying the ground motion for design,
Examining the sensitvity of seismic performance to structural configuration,
Improving the detail design of steel components and connections.

To respond to these demands, the following research projects are underway.

1) Japanese Society of Steel Construction (JSSC)
Special Task Committee on Hyogo-ken Nunbu Earthquake

2) Scientific Research Aids by the Ministry of Education
Quantative Evaluation of Limit States of Steel Connections
3) Japan Weld Engineering Society (JWES)
Accumulated Plastic Deformation (APD)
4) A National Project
Development of Structural Safety Improvement Tecnology Utilizing New-generation Steels
5) Six University Joint Research Program
Experimental Study on Plastic Deformation Capacity of Typical Japanese Welded Beam-to-Column Connections.
A project team among them recently published the report describing the research works done so far [3].

The research projects are particularly aiming at development of methods for
1) Upgrading seismic performance of existing steel buildings
2) Repair and retrofiting of damaged steel buildings
3) Design of details in weled connections and reliable workmanship in welding.

References

1) Steel Committee of the Kinki Branch of the Architectural Institute of Japan: Reconnaissance Report on Damage to Steel Building Structures Observed from the 1995 Hyogoken-nanbu Earthquake,(1995)
2) The Architectural Institute of Japan:Preliminary Reconnaissance Report of the 1995 Hyogoken-nanbu Earthquake,(1995)
3) JSSC: Kobe Earthquake Damage to Steel Moment Connections and Suggested Improvement,(1997)

Table 1 Number of damage found in Ashiyahama residential complex

Types of Buildings	Columns	Beams	Braces	Trusses	Total
14 Story	5	18	4	1	28
19 Story	26	41	18	12	97
24 Story	26	27	14	14	81
29 Story	0	2	1	0	3
Total	57	88	37	27	209

Table 2 The base shear coefficients and the fundamental periods of Ashiyahama

Bldg Height	14 stories 40.8 m		19 55.0		24 70.3		29 84.7	
Directions	X	Y	X	Y	X	Y	X	Y
C_B	0.236	0.230	0.190	0.172	0.145	0.150	0.126	0.125
T	1.41	1.50	1.75	1.97	2.30	2.32	2.66	2.70

Notes C_B : Base shear coefficient
T : Fundamental period (sec.)

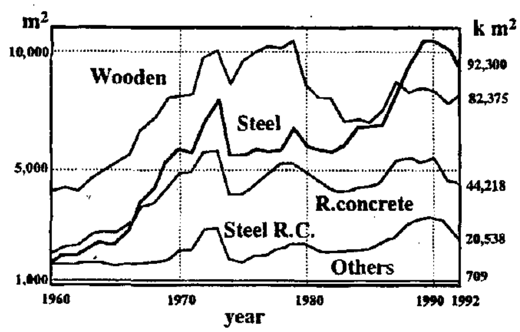


Figure 1 Constructed area of buildings with respect to structural materials

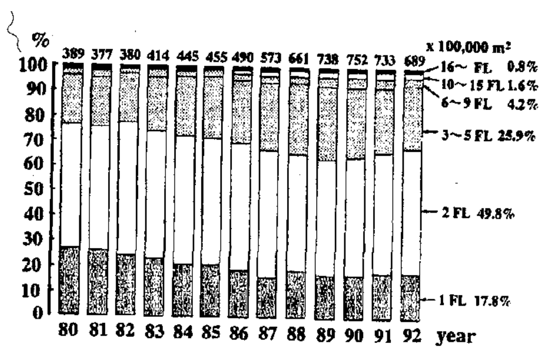


Figure 2 Constructed areas of various types of buildings

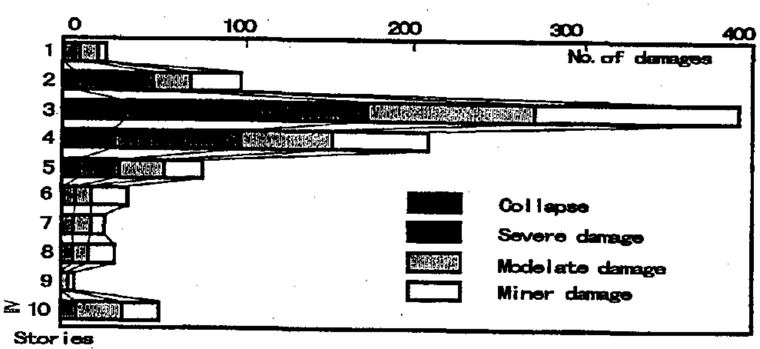


Figure 3 Damage level with respect to number of stories [3]

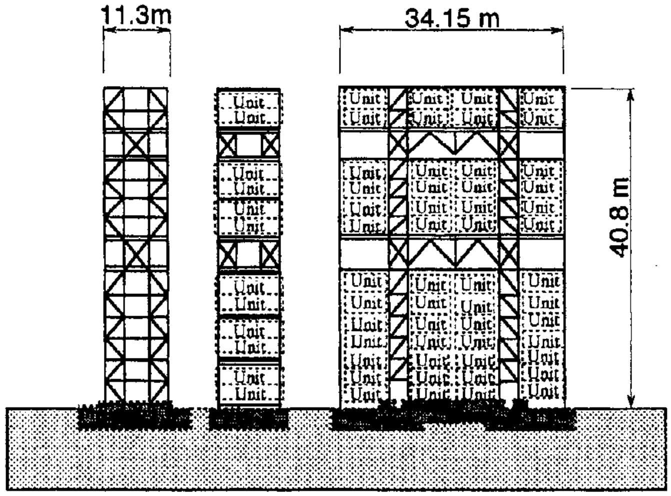


Figure 8 Structural configuration of Ashiyhama residential complex

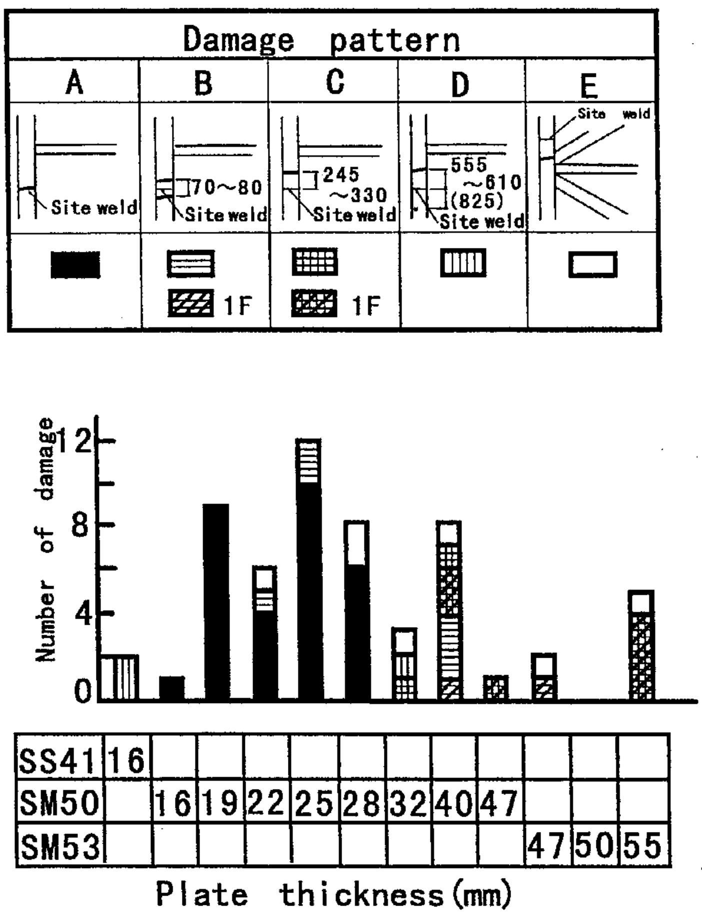
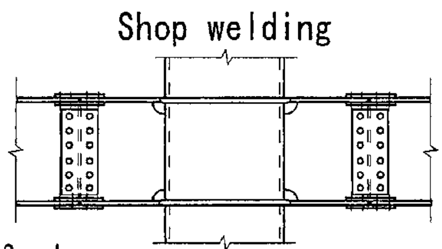


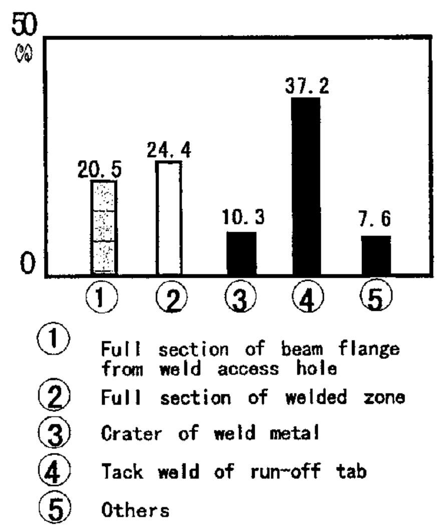
Figure 9 Crack pattern and plates thichness



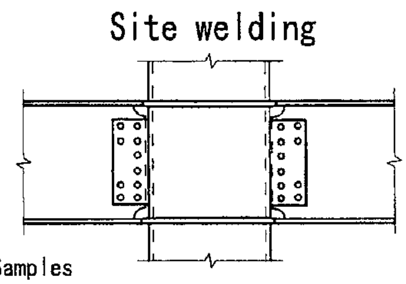
Samples

Buildings	Welded joints	Fracture/Crack
4	2396	3.3 %

Fiqure 4(a) Shop-weld joints inspected



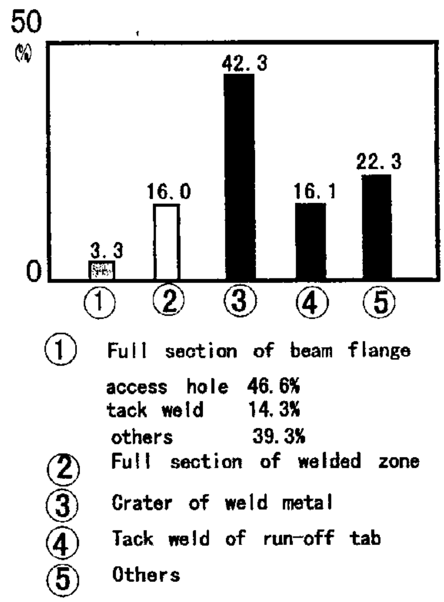
Fiqure 4(b) Damage patterns of the shop-welded



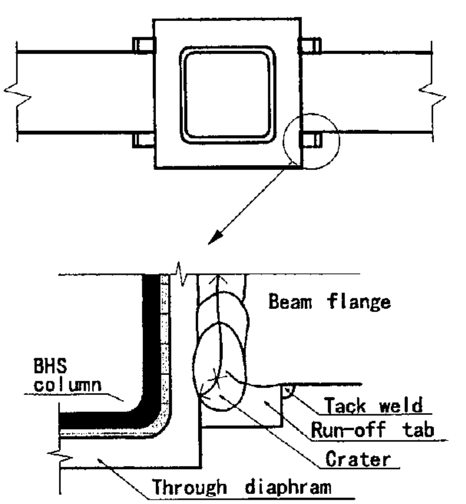
Samples

Buildings	Welded joints	Fracture/Crack
4	10112	8.5%

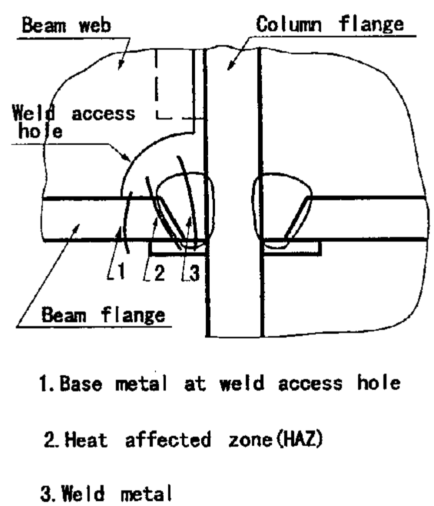
Fiqure 5(a) Site-weld joints inspected



Fiqure 5(b) Damage patterns of the site-welded



Fiqure 6 Crack paths at the weld metal



Fiqure 7 Crack patterns of the lower beam flange

ADVANCED SENSORS AND INSTRUMENTATION FOR HEALTH MONITORING AND CONDITION ASSESSMENT OF CIVIL INFRASTRUCTURE SYSTEMS

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ABSTRACT

Experimental analysis of structural systems is necessary for assessment of both new and existing facilities. While available sensor technology has advanced considerably over the last few decades, the problem of health monitoring and condition assessment is still one of material and structural modeling. No matter how good the information about the state of the structural elements or connection details, the assessment still must be based on basic structural mechanics. This paper describes current sensor technology, the relationship between that technology and the material and structural performance issues, and the role of both sensing technology and performance modeling in the greater problem of condition assessment.

INTRODUCTION

The role of health monitoring and condition assessment of civil infrastructure systems is continually growing in importance. This increase in importance can be traced to both changes in system needs and the development of appropriate sensing technologies. System needs include new and innovative structural systems and materials, smart structures, and perhaps most important of all, the need to objectively evaluate an aging infrastructure. In turn, an explosion in sensing and data analysis technology has been fed by the microelectronics revolution. The combination of these two factors has led to an unprecedented interest in the development of sensing technologies specifically tailored for civil infrastructure applications.

The need to develop tools for assessing and evaluating new and existing structures is implicit. The importance of such tools cannot be understated. For example, the total replacement value of concrete-based structures alone in the U.S. has been estimated to be over six trillion dollars.[1] It can be inferred that any improvements in technology for evaluating these structures could lead to substantial savings through a more rational allocation of maintenance and rehabilitation funds. The problem of resource allocation becomes particularly acute following a natural disaster (e.g. Northridge, Loma Prieta, Kobe earthquakes) where decisions regarding structural safety must be quickly and efficiently made. In order to assess the service life of existing structures it is necessary to answer two questions about the structure:

1. What is the current state of the structure?
2. Given this state what kind of performance or reliability estimates can be made?

The answers to these questions are rarely simple, especially for existing structures. To answer the first question we must define what the relevant "state" is, and then decide what sensing technology is appropriate to measure it. The answer to the second question is perhaps more difficult because it requires an understanding of the intricate relationships between individual components and overall structural systems. Complicating the questions are the range of scales involved. Knowledge of the "state" of the structure requires knowledge of materials, connections, and subsystems. Widely different techniques may be required to investigate the different scales. Indeed, performance and reliability estimates of a global system are based on performance and reliability estimates of the different components. Performance or reliability estimates therefore require knowledge of the different scales, plus the relationships between the scales.

Overview of This Paper

The main thesis of this paper is that in this point in time, our sensing technology has outpaced our ability to make accurate structural performance predictions and reliability estimates. This statement is especially true for complex structures constructed of heterogeneous materials. A case is presented below that the weakest link in the process of health monitoring and condition assessment of civil infrastructure systems is in fact our understanding of aging, damage, and failure properties of the system components. The greatest advances in health monitoring and condition assessment of civil infrastructure systems will take place when sensing technology and the relevant structural and material models are developed in concert. The most important issues relating specifically to sensing technology are the strategies for implementation in the most economical manner.

This paper presents a brief overview on the current state of sensing technology in the context of structural testing and monitoring. A description of barrier issues that need to be addressed for more advanced implementation of sensing technology for health monitoring and condition assessment of civil infrastructure systems is followed by recommendations for future development. Examples of salient U.S. research is included throughout the paper.

BACKGROUND

Structural testing has a history as long as mankind. While the methods and instruments have changed radically, the basic motivation has been and always will be to acquire information on the structure that is impossible to obtain any other way. Advances in structural mechanics theory and modeling techniques have perhaps shifted the emphasis somewhat from one of wholesale understanding of the structural system, to one of verification and tuning of a model. Regardless of the test details or exact motivation, experimentation in structural engineering is a critical part of the growth of knowledge, as it is in the scientific method.

The different applications of structural testing can be wide in their objectives and their scope. Within the context of health monitoring and condition assessment of civil infrastructure systems, this paper is limited to experimental analysis of full-scale field structures (as opposed to laboratory models).

Assessment of Existing Structures

While focus of this paper is on the assessment of existing structures, the motivations for doing this are many. The following is a list (certainly not all inclusive) of the more common motivations for structural assessment experiments:

- verification of models developed for design purposes
- evaluation of loads that were difficult to analyze prior to construction
- measurement of stresses in a particular element, or displacements at a particular connection
- assessment of new materials or design details
- monitoring of changes in overall behavior over time, or after a specific high impact event such as an earthquake, hurricane, or flood
- evaluation of the damage or deterioration of a particular element, connection, or other detail, including cracks, delaminations or corrosion
- remote monitoring

An important issue here is that these different applications require different sensing technologies. No single sensing technology will handle all these problems. Therefore for a rigorous structural assessment, we must think in terms of a sensing *system*, which utilizes as many different technologies as is required. The second important issue is *what do we do with the information we gather?* Sensing technology only deals with information gathering. As our ultimate problem is one of condition assessment, the collection of structural data only is half the problem.

In order to break down the big problems of sensing and condition assessment into a series of smaller problems, I will make the following classifications: local versus global testing, instrumentation of new structures, relationship to nondestructive evaluation, and the relationship of all these items to structural analysis and modeling.

Local versus Global Testing

Of all the different approaches to instrumenting structures, perhaps the simplest classification is local versus global. Here local refers to the examination of a single element or small region, whereas global refers to the entire structure or system. Obviously the approaches to these two applications are quite different. As it is impossible to instrument an entire structure for every possible quantity of potential interest, a hierarchy can be established between global and local analysis. In theory, it would seem that local problems should manifest themselves in the global response of the structure. If that were the case, a coarse global monitoring system could be used to point to a local problem. A more detailed analysis of the local problem could then be done with the appropriate testing technology. The problem with this scenario is this manifestation of a local variation may be of such a small magnitude, that its effects on the global response of the structure may be within the normal fluctuations.

Smart Structures

Instrumenting new structures for self-diagnosis is an area that is gradually gaining wider application in the U.S. In some respects any instrumented structure may be referred to as "smart" in the sense that it is able to detect changes in the state of the structure. How "smart" a structure is may be defined by the level of sophistication by which the data is collected, processed, transmitted, and acted upon. Although there has been some work in structural systems that can actually repair themselves, the primary form of intelligence designed into a smart structural system is one of behavior monitoring, and to a limited extent, assessment.

In the U.S., a group of researchers led by Drs. D. Huston and P. Fuhr at the University of Vermont have instrumented a number of structures using a variety of sensor technologies including fiber optics, strain gages, and thermocouples. A newly constructed classroom and laboratory building on the University of Vermont campus included an array of sensors for monitoring vibrations, wind pressures, service loadings, creep, cracking, and point measurements of strain.[2] The data is being logged for future long-term assessments of structural performance.

In the context of whole civil infrastructure systems, implementation of smart structure technology may provide opportunities to centralize infrastructure management systems.

Relationship to Nondestructive Evaluation

Nondestructive evaluation (NDE) technology has long been viewed as the means to answer questions of condition assessment and performance prediction, as it has been doing for some time in the aerospace industry. NDE typically connotes local testing. As with any structural testing, successful application of nondestructive evaluation requires fundamental knowledge in both sensing technology and materials theory. The sensing technology addresses the question of condition assessment, whereas materials theory is applied to answer the question of performance prediction.

Unfortunately NDE technology for infrastructure applications is not nearly at the level of sophistication of that in the aerospace industry due to problems of both sensing technology *and* materials theory. The simple reason for this is the extreme variability of construction materials such as concrete, wood and composites, due to their inherent heterogeneity. Heterogeneous materials are both difficult to inspect and predict. This can be contrasted to many homogeneous materials where defects such as cracks can be directly related to fatigue life and fracture toughness through elementary fracture mechanics theory. Heterogeneous materials do not have a widely accepted relationship such as that established.

Relationship to Structural Modeling and Analysis

Because of the success of finite element methods and other modeling techniques, our ability to analyze the global response of a structure exceeds our knowledge individual elements for many types of materials. That is, our modeling techniques allow us to accurately predict the response of a large system of components even though our understanding of the component response may be somewhat limited. This is the primary reason that large system models must be tuned to experimental data. It is the only way that the models can be adjusted to represent the real behavior of the structure.

In the U.S., a group of researchers at the University of Cincinnati, led by A. E. Aktan have spent several years looking at ways to instrument, model and assess highway bridges by integrating local and global monitoring schemes. The focus of their research effort has been to replace traditional (somewhat subjective) load rating schemes with objective, rational methods using load history and advanced testing techniques.[3] Their approach has been to combine long term strain gage monitoring (local) with dynamic analysis and structural modeling (global).[4] The questions of condition assessment are addressed through a "structural identification" methodology. The methodology serves as an umbrella covering all issues of performance, serviceablity, and reliability.

The most exciting aspect of the work Aktan and his colleagues is the *integration* of structural performance modeling with specific sensing and testing technologies.

OVERVIEW OF TECHNOLOGIES

Although the purpose of this paper is not to provide a comprehensive report on different sensing and instrumentation technologies available, a brief summary of some of the different commonly used techniques are described here for background purposes. Numerous references are available for each of the technologies described. This overview has been broken down into the following broad categories: mature sensing technologies, developing sensing technologies, instrumentation and data analysis.

Mature Sensing/Interrogation Technologies

For this paper, mature technologies refer to those that have been used on past instrumented structure projects, and therefore may be considered to have a reasonable "track record." In this section a sensor refers to a devise which can actually record some kind of signal, whereas an interrogation technology is one by which a structure is probed for a particular response. Interrogation technology in general must be coupled with some sort of sensing capability.

Traditional Techniques

Traditional techniques are those whose use predates the recent electronics revolution, such as electric resistance strain gages, extensometers, and load cells. A good reference on traditional techniques may be found in reference 5. For advanced monitoring applications, only those sensors that can be directly connected to an electronic monitoring system are considered here.

Of the traditional techniques, electrical resistance strain gages are still a widely used and viable technology. Perhaps the most common method of "instrumenting a structure" is to attach a series of strain gages, and monitor their output. The reason for their popularity is low cost, relative ease of use, and substantial knowledge base supporting their behavior. The disadvantages include their fragility, long term durability in extreme environments, and the often substantial surface preparation required for use. Despite these drawbacks, strain gages are an important sensing technology that will continue to have wide use in structural monitoring for the foreseeable future. Strain gages may be considered a local sensing device. However, a large enough array of strategically placed strain gages can provide a fairly comprehensive picture of global structural behavior.

Modal Testing

Modal testing is a method to use dynamic properties of a structure to estimate its integrity. It can be relatively simple in its approach or quite complicated depending on the structure and the desired information. Dynamic sources can be either ambient vibrations or various impact hammers or shakers. Monitoring is done using either single or multimode accelerometers, or in some cases, strain gages and fiber optics. Although accelerometers typically produce data for a specific point, modal testing and analysis may be considered a global technique since the measured accelerations are a function of the global structural response.

One approach to structural monitoring using modal analysis is to monitor the structure long enough so that a dynamic fingerprint can be developed. A change in the fingerprint over time may indicate damage evolution in some part of the structure. Work to date using this approach has shown that global behavior changes can be detected through modal analysis, however, these changes as yet can not necessarily be traced to specific local sources.[6]

Active Acoustics

The term "active acoustics" here refers to interrogation technologies where stress waves are introduced into the structure in such a way that the response provides local information about material condition. This ranges from relatively high-frequency ultrasonics for homogeneous materials to the relatively low frequency impact-echo method for heterogeneous materials. Both of these are strictly local condition assessment.

Ultrasonic testing has a relatively long history in the aerospace industry, and has found use in civil engineering in particular for crack detection in steel structures. As is mentioned above, the fracture mechanics-based relationships between crack length and fatigue life provides a valuable technique for nondestructive evaluation. While impact-echo techniques can be effectively used to estimate certain types of crack sizes in concrete structures, there is no accepted companion theory to make reliable service life predictions based on the test results.

Attempts have been made to apply ultrasonic techniques to concrete, but the problems of aggregate scattering and attenuation make measurements difficult to interpret. With the exception of impact-echo, ultrasonic applications to concrete have been aimed more at bulk properties than location of defects. Although many have tried to relate ultrasonic pulse velocity to compressive strength, the relationship is weak due to the lack of any fundamental basis.

Developing Sensing/Interrogation Technologies

While the techniques just listed have been useful and valuable in a wide range of structural monitoring applications, there has been an explosion of new sensing technologies introduced over the last 10 or more years, fueled primarily by the microelectronics revolution. The methods considered below are some of the more promising for civil infrastructure systems.

Fiber Optics

Although fiber optics are quickly becoming the standard for structural monitoring applications, they are listed in the developing technologies section due to the ever-expanding list of applications. Fiber optics have been a much-hyped technology for advanced instrumentation and monitoring of civil engineering structures. This is with good reason. Probably no other technology has as wide a range of possible applications. The optical fibers can act as both sensor and signal pathway. Sensing schemes can be set up as either localized or distributed in nature. An array of fiber optic sensors can be integrated with a multiplexing data acquisition system to monitor a wide range of system parameters including strain, crack monitoring, pressure, and temperature.[7]

Fiber optics are perhaps most associated with "smart structures" in civil engineering, primarily due to the multitude of ways the optic sensors can be integrated into the structure. A smart structure which includes an array of fiber optic sensors can be set up to monitor vibrations and strain, as well as locate cracks. The technology is especially appropriate where composite materials are used. Optical fibers can be embedded directly in the matrix for an integrated material-sensor system.

Electromagnetics

A wide range of sensing technologies are emerging as other engineering and scientific disciplines take on the problems of condition assessment of civil engineering systems. Many of these are based on utilization of electromagnetic waves including radar, microwaves, and x-rays. While all these techniques are showing great promise, the greatest technological barrier seems to be interpretation of data. Ground penetrating radar, for example, has been shown to be sensitive enough to detect subtle variations in concrete bridge decks, but it generates considerable volumes of data that is not yet easily interpreted.[8]

As is described below, success of advanced electromagnetic interrogation technologies will likely require a multidisciplinary research effort that includes expertise in both the particular electromagnetic medium and the particular structural assessment application.

Passive Acoustics

Passive acoustic or acoustic emission (AE) techniques have shown some promise as a part of a global monitoring system. Passive acoustic here refers to sound that is generated within the structure or material by microstructural changes in the material. Cracks are the most common sources of AE signals. This includes load-induced cracks as well as reinforcing steel corrosion cracking. Since the rate of acoustic emission activity is directly related to the rate of damage growth, AE techniques are a logical way to detect real-time damage growth in structures. The biggest problem, however, is the low energy acoustic signals generally are not strong enough to be detected by a reasonable-sized AE sensor array. Therefore recent laboratory research has focused on ways to extract AE signals from deep inside the structure.

An innovative use of acoustic emission technology in the U.S. has been advanced by David Prine of the Northwestern University Infrastructure Technology Institute. He has been using AE techniques to examine fatigue cracks in steel bridge girders. Through advanced AE signal processing he is able to determine if a crack effectively dormant or if it is actively growing.[9]

2D Imaging

There are a number of techniques for condition assessment that are based on extraction of information from two dimensional images of the structure. The most common ones include thermographic imaging and surface crack detection. Additional techniques include digital image mapping and shearography. In the case of thermographic imaging, inferences about the interior of the object are made through surface temperature profiles. It has the advantage of being a far-field, non-contact technique, but the disadvantage of producing data that requires considerable interpretation.

While image-based sensing techniques have proven to be valuable laboratory tools, field use has been limited.

Instrumentation and Data Analysis

While the sensing techniques described above represent the "nerves" of a structural monitoring system, the associated instrumentation to some degree represents the brain. In the simplest implementation of a monitoring system, there must be some method to record, display, and or process the data. In a sophisticated implementation, there can be considerable automated analysis that reduces large amounts of data into the most basic structural parameters. In either case the essential components include signal conditioning, data recording, data processing, and communications. The communications component may either be an interface for connection with an external device, or a transmitter/receiver for sending data and receiving instructions from a remote site.

The most common instrumentation associated with structural monitoring are data loggers that record the analog output of the sensors. In the past these were analog-based tape recorders, but have since been replaced by portable computer systems. Data-logging schemes have in the past been limited by the storage capacity of the particular device. Since tape or disk space was limited, there was great incentive to process the measurements prior to archival storage, thereby reducing storage requirements. Although there may still be a motivation in some cases to process the data and reduce the storage requirements, modern low-cost data storage has made this less important. As an example of the advances in storage capacity, if we instrumented a structure with fifty sensors, and logged data at 8-bit resolution every five seconds, we could fit over three years worth of data on a single one gigabyte hard drive. Two years of that data would fit on a single compact disk. Thus, hardware has progressed to the point where we need to be less concerned with data handling problems. The advantage of keeping the raw data is that it can always be referred to for future questions that arise regarding the structure's history. If the data is reduced prior to storage, that option is removed.

As is described in more detail below, sophisticated instrumentation could possibly provide a partial substitute for traditional structural models.

BARRIER ISSUES FOR FURTHER DEVELOPMENT

There are several hurdles that are slowing incorporation of advanced sensors and intelligent analysis into civil infrastructure systems. Most of these hurdles are not on the sensing technology end, but rather on the implementation and analysis end. Some of these are easily handled through some modest organization efforts, while others are much more difficult. Also, some of these problems require civil engineering expertise, while others require expertise outside of civil engineering. Each is summarized below, listed in no relation to difficulty of solution.

Performance Prediction of Heterogeneous Structures

The most difficult problem of using advanced sensors for condition assessment is the uncertainty associated with structural systems at the component or material level. While structural steel has a large theoretical basis for performance prediction (elasticity, plasticity, fracture mechanics), heterogeneous materials such as concrete, wood, and composite materials do not. The reason this is so intimately linked with sensor technology has to do with the questions stated early on in this paper. The greater problem of health monitoring and service life prediction can only be solved by answering the following:

1. What is the current state of the structure?
2. Given this state what kind of performance or reliability estimates can be made?

Answering this first question with advanced sensors and instrumentation does nothing for our greater problem unless we can answer the second question. Nondestructive evaluation techniques such as impact-echo may be able to find cracks in a concrete structure, fiber optic sensors may be able to measure their openings, microwave probes might be able to estimate moisture contents and pore-size distribution, and electric potential measurements might be able to estimate the degree of corrosion in the reinforcing steel. While this is all valuable information, we now need to be able to take that information and make rational reliability and serviceability predictions. The state of the art in materials science of heterogeneous construction materials does not yet allow us to do that.

Interpretation of Data

The general phrase "interpretation of data" has two meanings here. One has to do with the interpretation of data from advanced probing or sensing techniques, while the other has to do with interpretation of an array of signals from more conventional sensors.

Interpretation of data is mentioned above as a problem with techniques such as ultrasonics and radar when applied to heterogeneous materials. This problem comes back to one of the general problems of heterogeneous materials. Critical defects are difficult to detect because they are often of the same size as the inhomogenieties in the material. Ground penetrating radar produces a tremendous volume of data that must be filtered. Optical signals carried through fibers require processing to interpret. Really any indirect sensing technology requires a basis by which sensor output can be transformed to physical measurements. For mature sensing technologies this is well established. However, for developing techniques considerable baseline work will be necessary prior to field implementation.

In the case of multiple sensor array interpretation, techniques must be developed that can make better sense of the large amounts of data that can be generated for a large structure. The approach of Aktan described above includes finite element models to interpret the output of the large sensor array. The bridges Aktan describes however, are relatively simple structures. An approach that may be more appropriate for large complex systems is to employ neural networks to the data array, such as was done by Stuart Chen of SUNY Buffalo. He trained the neural networks to identify and locate damage using strain gage and accelerometer data.[10] Although his conclusions were based on a small scale experiment, the results show the feasibility of the approach as a tool to compliment finite element models. Clearly more work is needed in this area.

Interdisciplinary Nature of Problem

The problem of sensing for structural system assessment is an inherently interdisciplinary problem. In most cases, the sensing technology falls outside the realm of civil engineering. While for something as simple as electrical resistance strain gages, this is not important. However, for more advanced technologies, this is a critical issue, and requires a true multidisciplinary research and development team. In general, civil engineers do not have the background required to develop an advanced microwave nondestructive test method. Conversely, an electrical engineer with expertise in microwave sensing is not likely to have sufficient background in mechanics or materials to understand the relevant assessment issues. To move a technology forward the two must work together so that the sensing technology will be focused toward the correct problem.

Performance and Reliability versus Sensing and Nondestructive Evaluation

Given the considerable uncertainty of material performance, the problem of system performance and reliability becomes compounded. The role of sensing technology is to improve on that uncertainty. The sensing technology and reliability estimates should be intimately linked. Unfortunately the problem is so broad that very few are able to take on such an "holistic" approach. (An exception is George Hearn at the University of Colorado who has done work relating NDE technology with structural reliability estimates.[11]) In order to move health monitoring and condition assessment to the next level, sensor output should be used to feed global structural performance models.

RECOMMENDATIONS

As a result of the above-mentioned barriers to technological advancement, as well as the current state of sensor and implementation strategies, the following actions are an initial slate of recommended research approaches.

1. At the material level, research toward better relationships between *measurable* microstructural features and material performance parameters for heterogeneous construction materials is needed. This is a major undertaking with wide scope and approaches. While much work is presently being conducted on microstructure-performance relationships for concrete (for example), in the background of that work should be an underlying theme that material models need to be based on measurable microstructure properties.
2. Advanced sensing and instrumentation technology development must be driven by structural performance questions, as opposed to driven by technology looking for application. This will require multidisciplinary teams of researchers to accomplish. The bottom line is to get the people who know the technology teamed up with the people who know the problems.
3. Focus NDE research on the problem of structural reliability. The two need to be inseparable. Either NDE should answer questions of reliability, *or*, reliability models should point to locations on a structure where NDE should be performed.
4. Continue to develop advanced sensing and NDE technologies, with particular emphasis on heterogeneous materials.
5. Continue to advance data interpretation techniques.
6. Promote a Japanese-U.S. partnership where the relative technological strengths may be complimented.

SUMMARY

The thesis of this paper is that sensing technology will advance together with advances in relevant materials science and structural modeling technology. The problem of health monitoring and condition assessment must be considered as a whole so that particulars of the sensing or experimental technique do not obscure the primary goal.

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ADVANCED SENSORS AND NETWORKS FOR HEALTH-MONITORING OF CIVIL INFRASTRUCTURES

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ABSTRACT

Three fiber optic sensors and a smart sensor are introduced. The first fiber optic sensor utilizes bragg grating for correlating bragg wavelength to strains or temperatures. This sensor is basically a point sensor. However, multiplexing the sensors allows us to measure several points along a single fiber path. The following fiber optic sensors are based on optical time domain reflectometry (OTDR). One uses Brillouin scattering for detecting strains and temperatures. The other uses Raman scattering for temperatures. Thanks to the use of OTDR, both sensors are distributed sensors so that physical response along the fiber cable can be continuously monitored. As an alternative device, a smart sensor consisting of carbon fiber glass fiber reinforced plastics is introduced. The sensor is unique since it can be an element of structural system as well. The smart sensor memorizes its maximum strain ever experienced. To show the applicability of advanced sensors to a large structure, application of a Raman OTDR to a large underground storage tank for liquefied natural gas is demonstrated. For quick and accurate assessment, a recently developed monitoring system utilizing the world wide web technology is introduced. The combination of the advanced sensors with such an advanced network will provide us vast possibilities.

INTRODCUTION

Health-monitoring systems for infrastructures are intended to ensure the functionality and safety under normal operation conditions or sever environmental hazards such as earthquakes and typhoons. The recent devastating natural hazards, such as, the Northridge Earthquake in 1994 and the Hyogo-Ken Nanbu (Kobe) Earthquake in 1995 (more than 6,000 people dead and 40,000 buildings destroyed) have indicated the immediate need for health assessment of structures. However, it has been widely understood that the current sensor and network technologies have certain limitations for such assessment. For example, nondestructive damage assessment of columns and beams of tall steel buildings is currently very difficult without removing coating materials for fire protection. Evaluation of embedded piles is further difficult since excavation of foundation soil is necessary in most cases.

For many years, fiber optic sensors have been considered to be revolutionary sensors due to their many advantages. However, engineers have been deterred by the astronomical cost for fiber optic equipment. Most of fiber optic sensors were thus limited to experiments in laboratories. Recently some of fiber optic sensors have evolved to achieve affordable price-tags and sufficient specifications for use in the field measurements. Three most promising intrinsic fiber optic sensors include fiber bragg grating sensors, Brillouin OTDR and Raman OTDR. They can be used for measuring strains or temperatures. However, experiences to apply them to civil infrastructures are still lacking. The field implementation experiences may have to be accumulated by civil engineers for wide use of such advanced sensors. One of the fiber optic sensors introduced here, ROTDR, was recently applied to a large underground storage tank for monitoring temperatures to control curing processes of concrete. The implementation process is briefly explained.

As an alternative approach, a smart sensor that is capable to memorize the maximum response they experienced without any external power supply is introduced. The sensor itself can be also a structural element. Since no permanent measuring equipment is necessary to be installed in the target structure, the installation cost may be significantly small. The material introduced here, a carbon fiber glass fiber reinforced plastics (CFGFRP), has capability to memorize the maximum strain by a residual change in electrical resistance of the material itself.

When a health monitoring system is required to identify complicated conditions, the use of multiple sensory systems is inevitable. If that is the case, networking of all sensory systems becomes important for quick and accurate assessment. As a candidate for such a purpose, a monitoring system using world wide web (WWW) technology is briefly introduced.

EXAMPLES OF ADVANCED SENSORS

Among many advanced sensors, intrinsic fiber optic sensors have been considered to be promising due to many reasons [1]. Some of them are:

- electrical and magnetic immunity,
- small size that minimizes degradation of structural performance,
- distributed sensing,
- long operation life, etc.

Three most feasible fiber optic sensors are chosen here. In addition, a new smart material which could be fabricated as a smart sensor is introduced.

Fiber bragg grating (FBG) sensor

A fiber bragg grating sensor is a simple intrinsic fiber optic sensor based on the photosensitivity of optical fibers [2]. Until recently, the production of bragg grating elements was technically difficult. As a result a bragg grating element was prohibitively expensive. The recent photo-imprinting technology and the strong demand in the communication industry are the key thrusts to cut the cost drastically. Application of the bragg grating technology to detect strain and/or temperature in civil infrastructures has now become feasible. The mechanism of fiber bragg grating is briefly explained below.

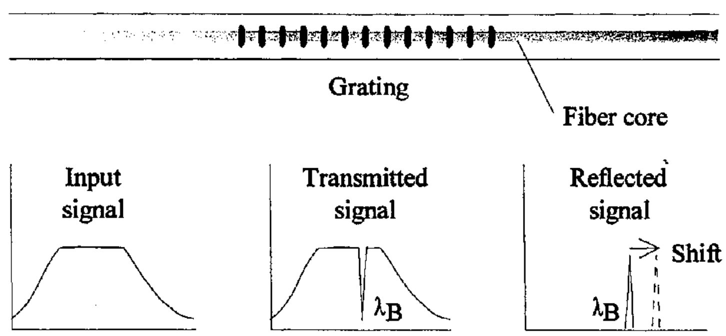

Fig. 1 Basic mechanism of a fiber bragg grating sensor

The resonance condition of a grating is given by the expression

$$\lambda_B = 2n\Lambda$$

where, Λ is the grating pitch and n is the effective index of the core. The resonance wavelength λ_B is called bragg wavelength. As depicted in Fig. 1, a narrow-band component of the bragg wavelength is reflected at the bragg grating with the pitch Λ. The input source should be a broad-band source such as an LED source. The strain arises due to the physical elongation of the sensor. The resulting fractional change in grating pitch shift the bragg grating wavelength as shown in Fig. 1. If it is possible to detect the change of the bragg wavelength accurately, the physical elongation can be obtained. The physical elongation is associated with local strains. The bragg grating sensor is thus capable to detect strains. Since the elongation occurs also due to the temperature change, the same mechanism is applicable to detect temperature as well. In other words, the effects of strain and temperature may be coupled. Therefore, when measuring strain responses, the temperature compensation is necessary.

A typical FBG sensor system is depicted in Fig. 2. As shown in Fig. 2, it is possible to multiplex several FBG sensors under the condition that each FBG sensor has an enough bragg-wavelength band with sufficient separation with neighboring bragg-wavelength bands. The number of possible multiplexing sensors is dependent on the band of the input source, a typical LED source allows us to multiplex 20 or more devices along a single fiber path if the peak strains experienced by the gratings do not exceed 1000 μ strain. Specifications of a commercially available FBG sensor system is listed in Table 1.

In real-world application, complicated and sophisticated fabrication of optical fibers at a construction site is not possible. Therefore, the field implementation process should be as simple as possible. Especially when multiplexing of several FBG sensors is necessary, many connecting processes are involved. Such fabrication should be conducted in a factory or a laboratory.

Table 1 Specifications of FBG sensor

Maximum strain measurement range	0.04
Strain measurement accuracy	$\pm 4\times10^{-6}$
Strain measurement resolution	$\pm 1\times10^{-6}$
Temperature measurement range	-40°C to 300°C
Temperature measurement accuracy	±0.5°C
Temperature measurement resolution	0.1°C
Sampling frequency	50Hz

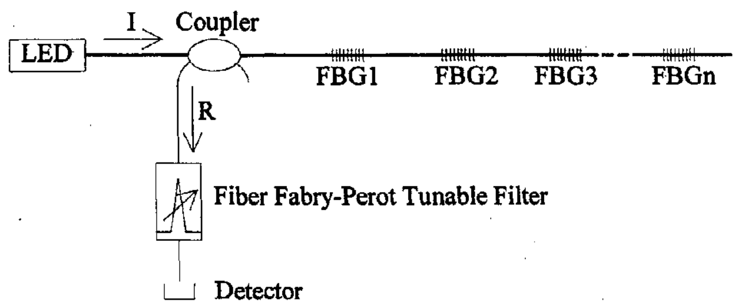

Fig. 2 Typical configuration of a fiber bragg grating sensor system

Brillouin optical time domain reflectometry (BOTDR)

The optical time domain reflectometry is commonly used in the communication industry [3]. It can be considered as an one-dimensional guided radar. This method utilizes the scattered lights in fiber cores due to series of input pulses. Some components of input lights are scattered back due to reflectors, bending cores and so on. The time delay of scattered lights determines the distance between the source and the target point. The physical characteristics of the scattered lights can be associated with some physical response of the fiber at the point where the light was scattered.

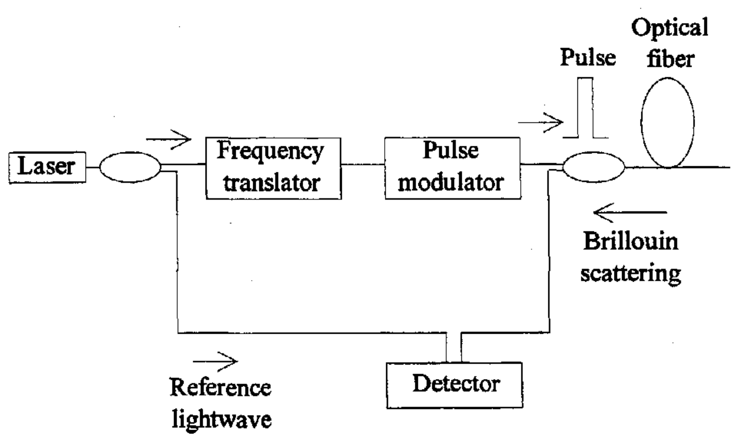

Fig. 3 Configuration of BOTDR

Rayleigh, Raman and Brillouin scattering are commonly known phenomena. The Brillouin scattering reflects the smallest power among these three scattering phenomena and was found to be sensitive to both strain and thermal response of the fiber core. However, the scattering energy is so small so that it had been so difficult to discriminate the Brillouin scattering component from noises. The Brillouin OTDR recently developed by a team of NTT reached a feasible performance level to be used in civil engineering field [4]. The specifications of their system are listed in Table 2 . Although it is not realistic to replace conventional strain gauges by the BOTDR sensors, capability of distributed sensing over a large distance is very attractive for certain applications.

Table 2 Specifications of BOTDR

Maximum sensor length	10.0km
Spatial resolution	1.2m
Maximum strain measurement	0.03
Measurement accuracy	$\pm 6.0\times10^{-5}$

Raman optical time domain reflectometry (ROTDR)

The Raman OTDR is similar to the BOTDR except that this system utilizes Raman scattering instead of Brillouin scattering. The Raman scattering is sensitive to temperature only. In addition, scattering power is much larger than the Brillouin scattering. Therefore, an interrogation system based on the Raman OTDR can be much simpler than the one for the BOTDR. Several systems are already commercially available.

Typical specifications of an ROTDR system are listed in Table 3. If necessary, the spatial resolution can be managed to be as short as 0.3m. The temperature range listed in the table are under the assumption that the optical fibers used are those commonly used for communications. When a specially fabricated fiber is used, the maximum temperature can be much higher.

Table 3 Specifications of ROTDR

Maximum sensor length	2.0km
Spatial resolution	1.0m
Temperature range	-10°C to 75°C
Measurement accuracy	±1.0°C
Measurement resolution	0.1°C

Carbon fiber glass fiber reinforced plastics (CFGFRP)

Some class of CFGFRP composites containing carbon fiber bundles have curious physical characteristics. Their electrical resistance has a correlation with its strain arising from external load [5]. Interestingly, the electrical resistance increased due to the loading remains even after unloading. A schematic relation between strain and electrical resistance is shown in Fig. 4. Therefore, a CFGFRP composite can work as a memory for storing its maximum strain ever experienced.

It is the significant advantage if the material itself has its own memory without any power supply. The only thing we should do for reading its memory is to measure the electrical resistance. The relation between strain and electrical resistance is controllable by adjusting the fiber materials. Strain sensors fabricated from the CFGFRP composites have been used for monitoring cracks in the slab concrete in a tall building in Singapore. The CFGFRP mesh was fabricated and used for implementing into walls of a vault to detecting a robbery through the walls. In this case, the mesh was used for sensing as well as reinforcing concrete.

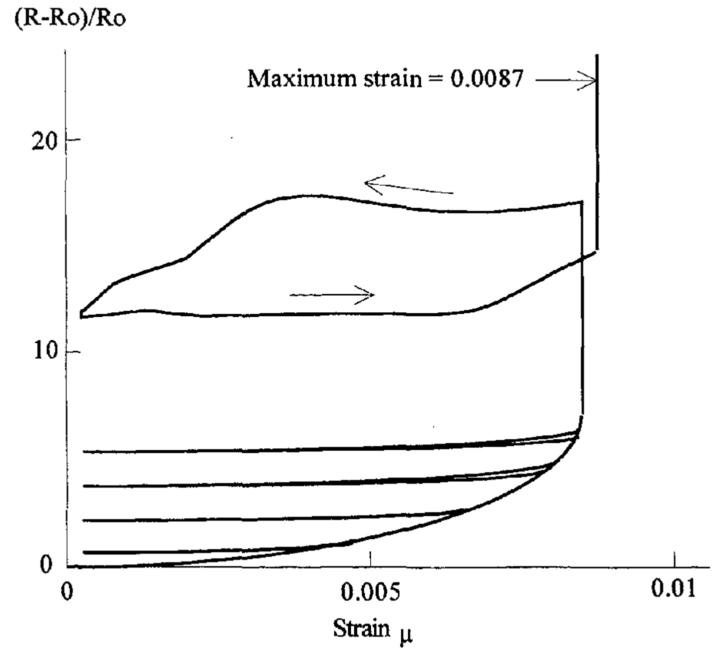

Fig. 4 Electrical resistance versus strain for a CFGFRP composite

APPLICATION OF ROTDR TO UNDERGROUND STORAGE TANK

An ROTDR system was used for controlling curing processes of a large concrete slab which is a bottom plate section of an underground storage tank for liquefied natural gas [6]. The diameter of the is about 70m. The corresponding depth of the slab is more than 10m. Due to the large volume of concrete poured at once, measuring curing temperatures is always required to ensure designed performance of the concrete. Major reasons to employ the ROTDR for the storage tank were:

- A single optical fiber covers many measuring points so that the damage to the slab structure is negligible.
- Installation of sensor fibers requires less time compared to conventional sensors.
- The number of measuring points can be 2-digits more than the conventional sensors with the same installation cost.

The installation layout of the ROTDR is presented in Fig. 5. The accuracy of the sensor was secured by adding a few conventional sensors at the key measuring points. The total length of the optical fibers exceeded 2km. Therefore, the number of measuring points was more than 2,000. This large number of measuring points allowed us to employ a more elaborate control of curing processes. At a location where good spatial resolution was necessary, spot fiber sensors were used. A spot fiber sensor consists of an optical fiber of a few meters wound up in a small circle. The fiber is protected by plastic.

The optical fiber cable used for this application is the one commonly used in the communication industry. The diameter of the fiber cable is 12.5mm. A single cable can case six single-mode or multi-mode fibers. The mass of the cable is only 150kg per 1km. This light weight helped conduct the installation works quickly. This type of fiber cable is a mass-produced one. Therefore, the price-tag for it is affordable.

In the above case, the ROTDR was used only for controlling the curing processes. However, this sensor is more promising to be used for permanent monitoring of soil temperature after completion of the storage tank. The liquefied natural gas should be kept at a very low temperature so that the soil deposits surrounding the tank are being gradually frozen. If no heating control is made for the soils, the storage tank will be eventually floated and fractured. To avoid this catastrophic event, heating pipes are embedded in the surrounding soils to regulate the soil temperature. The thickness of the frozen soils is thus limited within a certain range. In this heating system, the ROTDR is ideal as a temperature sensor. Any sensors using electricity have a certain degree of danger to ignite a fire. It is not feasible to apply such sensors for flammable natural gas. Hence, a sensor which does not use electricity such as the ROTDR is very attractive.

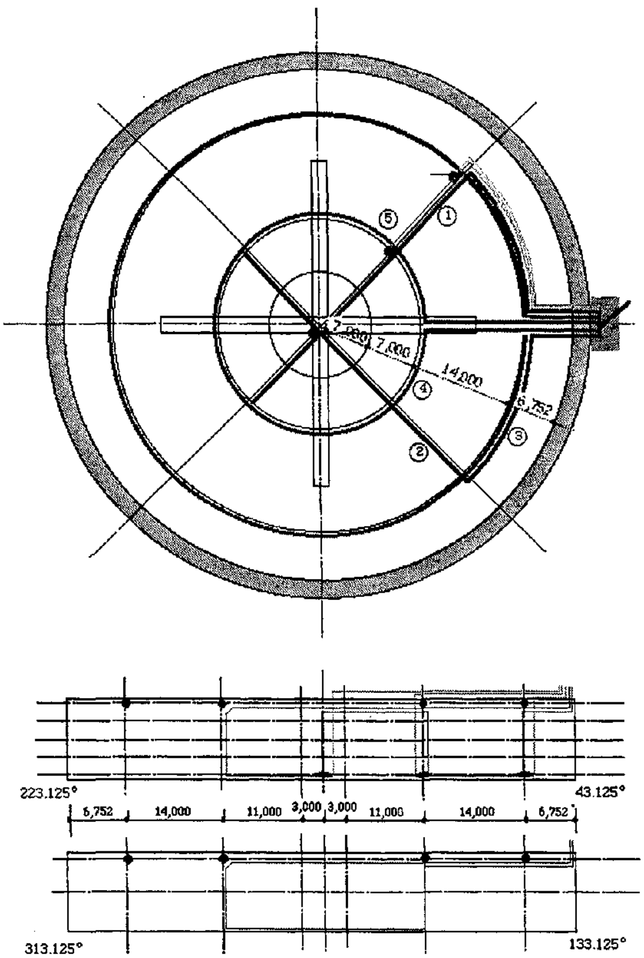

Fig. 5 ROTDR sensor layout in a large slab structure

MONITORING SYSTEM FOR CONSTRUCTION SITE USING WORLD WIDE WEB

In practice, a monitoring system consists of variety of sensors including advanced sensors as well as conventional sensors. Each group of sensors may have a controller, which is a PC in most cases. Data from the sensors are gathered through floppy disks, networks or even via printed materials. Therefore, processing all data is an extremely tedious task.

At a large construction site involving ground excavation in an urban region in Japan, monitoring of ground deformation and associated physical values is mandatory. For each monitoring item, an appropriate engineering company is usually assigned. Each engineering company has their own sensory system which is not compatible to one another. The engineering companies are responsible for data acquisition only in most cases. Interpretation of the acquired data is made by site engineers or research engineers at research and development divisions of the construction companies. Therefore, the incompatible sensory systems were the headache for construction management.

A monitoring system recently developed by Shimizu Corporation [7] solved this situation. The configuration of the monitoring system is depicted in Fig. 6. The server utilizes the World Wide Web technology to communicate with client PCs. The data acquired by engineering companies are transferred to the server using a simple protocol called NFS. The modification of the programs installed in the controllers is minimal. All measured data are automatically stored in the monitoring server. Every engineer is now able to access any data with a common format which is compatible to common spread-sheet applications. The current and previous conditions can be easily checked through dynamically generated graphs or tables. Any data can be downloaded from the server under the condition that the client PC is connected to the network and that the user has an access privilege. The server was employed at construction sites for a large building complex in Tokyo and a large storage tank for liquefied natural gas. A typical view window for a storage tank is shown in Fig. 7. The red, yellow and green colors are used to tell the warning level at each measuring location. When a red color is lit, the construction works should be immediately stopped. A single click of the circle indicating a measuring point prompts generation of graphs and tables.

For a large-scale monitoring system, introduction of the WWW technology is extremely attractive. Even when many incompatible sensory systems are involved in the monitoring system, engineers can assess the conditions through the WWW monitoring server as an interface. By interfacing the system by the server, the data formats can be automatically converted into a common format. Engineers do not need to know the detailed functions of each sensory system at all. If necessary, any identification processes can be also implemented into the server with a reasonable effort using CGI or JAVA.

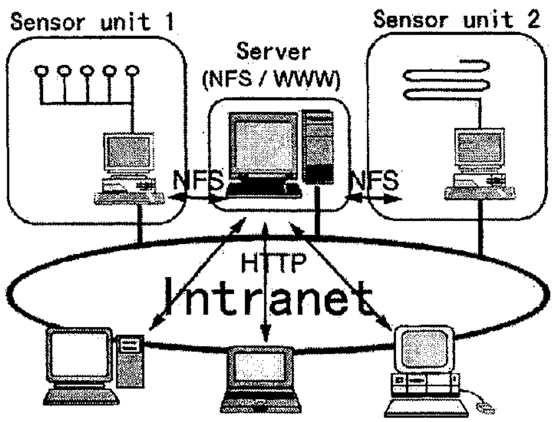

Fig. 6 Monitoring system for construction management

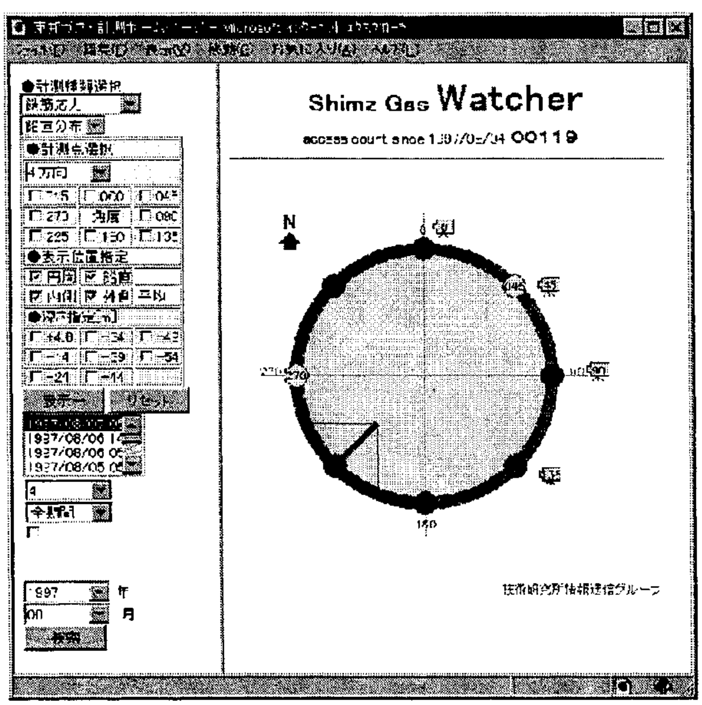

Fig. 7 Typical view window of monitoring system

CONCLUSIONS AND RECOMMENDATIONS

The potential of advanced sensors and networks was examined for applying them to a health-monitoring system for civil infrastructures. Three fiber optic sensors, fiber bragg grating sensor, Brillouin OTDR and Raman OTDR are considered to be in a practical stage for use in the field measurement. The latter two OTDR systems are attractive especially for continuous measurement along a single fiber cable. The fiber bragg grating sensors are perfect when a long-life and an accurate measurement is necessary. In addition, the fiber bragg grating sensors can be multiplexed to measure several points along a single fiber path though the possible number of sensors is not very large. A smart sensor using CFGFRP composite materials has capability to memorize the maximum strain response experienced by the sensor since its fabrication. This type of sensor is considered to be smart since no permanent equipment for measurement is needed. When necessary, a conventional tester for evaluating the electrical resistance is sufficient for interrogation. Smart sensors similar to the CFGFRP composite are cost competitive and feasible for particular applications where only a small number of physical responses is necessary to be recorded. A monitoring system using WWW technology employed for measuring variety of physical responses for construction management was proven to be very effective for quick and accurate assessment. It can be easily achieved to conduct more complicated tasks such as online identification using the same system. The combination of advanced sensors and networks promises a revolution for health-monitoring systems.

Some recommendations for future studies in the health monitoring field are listed below:

- A feasibility study of health monitoring systems installed in civil infrastructures,
- Study of standard specifications for sensor devices to reduce production cost,
- Development of WWW-based identification and simulation tools,
- Study of implementation techniques for new sensors such as fiber optic sensors,
- Strategies for minimizing life-cycle-cost.

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DEVELOPING NDE TOOLS FOR THE INSPECTION OF HIGHWAY BRIDGES IN THE UNITED STATES

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ABSTRACT

The goal of the Special Project in Nondestructive Evaluation is to develop implementable tools for the evaluation of in-service highway bridges. This process involves thorough field testing and evaluation of research prototypes and commercially available instrumentation. Applications of NDE technologies are developed through this evaluation process, and instrumentation is modified and improved. This paper will discuss several of the technologies currently being developed by the SPNDE. The paper will also discuss the new FHWA NDE Validation Center, a project that is developing a national resource for the validation testing of NDE technologies.

BACKGROUND

The national bridge inventory of the United States includes more than 470,000 bridges, of which approximately 187,000 are rated as deficient. Deficient bridges are classified as either being structurally deficient of functionally obsolete. The reasons that a structure may be rated as structurally deficient include low load rating, deficient deck or superstructure, or deficient substructure. Functionally obsolete structures are simply not designed to meet modern requirements for geometry and traffic capacity. Approximately 1%, or 5,000 bridges, become deficient each year. This large bridge inventory requires a significant investment of resources to maintain the bridges in safe operating conditions, and to rehabilitate and improve the condition of existing structures. The United States currently builds or rehabilitates approximately 10,000 bridges each year[1].

Effective methods for evaluating the condition of the bridges is critical to the management of this massive bridge inventory. Current methods of evaluating in-service highway bridges consist primarily of visual inspection. During a typical bridge inspection, the various components of a structure are examined at close range by trained inspectors, who evaluate the condition of the component and give it a ranking. For example, bridge decks may be rated on a scale from 1 to 7, with a rank of 7 being perfect condition and 1 being completely deficient. The inspector will judge the deck based on visible damage, percentage of deck delaminated, evidence of corrosion etc. and assign a rank value for the deck. This ranking is based on the subjective evaluation by the inspector, who bases his or her ranking on the guidelines provided through training and the particular inspector's experience with similar components. For many situations, this type of evaluation in appropriate and effective. However, due to the subjective nature of this evaluation, rankings of similar bridge component condition can vary widely from inspector to inspector, and from one state to another. Additionally, this type of ranking does not provide reliable and quantitative information about the condition of the structure.

This type of inspection is simply not appropriate for many circumstances. For some component geometries, such as pin and hanger connections, the area of interest is hidden from view. In other cases, such as concrete bridge decks, the surface condition may not be indicative of the internal condition of the component. Internal defects, typically delaminations occurring due to corroding steel reinforcing bars, may not be detected visually. The same is true for many other defects that may occur in highway bridges.

Perhaps the greatest shortcoming of visual inspection is the ability to provide objective and quantitative information on the condition of a bridge component. Current inspection practice relies on a somewhat arbitrary rating system and inspector notes to determine the condition of various bridge components. This leaves little hard data to determine structural capacity rating, appropriate maintenance action, and remaining life of the structure. Additional tools that provide this type of objective and quantitative data are needed to improve the way that bridges are inspected, evaluated and maintained.

The Federal Highway Administration's (FHWA) Nondestructive Evaluation (NDE) Research Program has focused on the development of specific tools to address the need for objective and quantitative NDE of highway bridges. The Special Project in NDE (SPNDE) is charged with field testing the prototype systems developed through this research program. The goal of the Special Project is to develop implimentable tools for the nondestructive evaluation of bridges. This development process involves thorough field evalution of NDE technologies, development of application and procedures, and design modifications to research prototypes. The Special Project also investigates commercially available technologies that may be suitable for highway bridge inspection. Three key areas are currently being investigated: Detection of Fatigue Cracks, Advanced Bridge Deck Inspection, and Advanced Bridge Instrumentation. This paper describes the efforts in each of the Special Projects focus areas. The paper also describes the FHWA approach to validating the performance of NDE methods with applications suitable for bridge inspection.

FATIGUE CRACK DETECTION

Of the 475,850 bridges included in the National Bridge Inventory, more than 40% are constructed with a superstructure of steel[2]. Fatigue cracking in these steel bridges is a significant problem for bridge inspectors, because the cracks are often difficult to detect by visual inspection. Many methods for detecting cracks nondestructively have been developed, but few have found acceptance in the bridge community. The application of many techniques is complicated by the large size of a bridge, and the relatively short time allowed for the inspection of each structure.

Two electromagnetic (EM) techniques for detecting cracks are currently being investigated by the SPNDE. The advantage of EM techniques is that they typically require little surface preparation prior to inspection. One technique being tested is the eddy current method. This method is widely used in the aerospace industry, where typical materials are non-ferromagnetic. The eddy current method being employed uses two coils on edge at the specimen surface. The on-edge coils are intertwined and oriented normal to each other. This orientation scheme eliminates the effects of spatially varying magnetic permeability, which may be significant is ferromagnetic materials such as steel. This coil configuration significantly increases the signal to noise ratio, and allows for effective testing of the weld bead. This method has been employed on a painted, high strength steel welded box girder to detect surface breaking, hydrogen assisted cracking in a newly constructed bridge. The method was successful at detecting cracks on the order of 3 mm in length and 1 mm deep through a conductive zinc coating. Current laboratory studies are examining the ability of the method to determine crack depth.

The Alternating Current Field Measurement (ACFM) method, which is related to the eddy current method, is also being evaluated. ACFM was originally developed for the offshore oil and gas industries, where crack detection methods capable of penetrating up to 5 mm of coating are required. The method uses an induced magnetic field and a unique probe detection scheme to detect and quantify longitudinal cracks at the weld toe. The method is sensitive in a variety of conductive materials, including steel and aluminum, and can penetrate typical bridge coatings. Current research is aimed at determining the accuracy of the crack depth and length measurements and exploring how the method may be used for the detection of cracks in fillet welds on light poles and sign supports.

Another active area of research in area of fatigue crack detection is ultrasonics. Although this method is widely used in other industries, the field application of ultrasonics for highway bridge inspection has been limited. The ultrasonic method is currently applied regularly to the inspection of pin and hanger connections. The pins are difficult to visually inspect because the primary area of crack growth is at the hanger/web interface, an area that is not accessible to visual inspection. The pulse-echo ultrasonic technique is commonly used to detect cracks in this area.

A "hands-free" ultrasonic instrument has been developed by the FHWA, and is currently being field tested. The computer-based system is worn by the inspector as a backpack, powered by batteries attached to the inspector's belt. The ultrasonic signals are displayed in a "heads up" fashion, a unique computer display worn in front of one eye that appears as a virtual screen in front of the inspector. This allows the inspector to maintain visual contact with the surrounding environment, while simultaneously viewing the real time display of test results. Keyboard operations for the instrument are conducted on a virtual keyboard, allowing the inspector to operate all I/O operations from a single hand held mouse. This hands free system operation is intended to increase safety and convenience during the inspection of problematic bridge details. The system also has a magnetic perturbation channel that can be used quickly determine the length of a crack, and to confirm ultrasonic results.

Time of flight diffraction is being investigated for potential field application. This ultrasonic technique is effective for determining crack depth. The method consists of measuring the time of flight of elastic waves diffracted at the crack tip. Previous research by FHWA has investigated the application of this technique for detecting and sizing cracks in eyebars. Current research activities are investigating this technique for determining crack growth rates in welded plate girders. This method has been used to track the crack growth in 2" thick flange during fatigue loading.

ADVANCED BRIDGE DECK INSPECTION

The vast majority of bridges in the National Bridge Inventory are constructed with a reinforced concrete deck. These decks are typically replaced every 15 to 20 years, due primarily to deterioration caused by corrosion of the reinforcing steel (rebar). The expansion of the rebar due to corrosion causes *delaminations*, cracks formed along the plane of the rebar mesh. The delaminations typically fill with water, and propagate during freeze-thaw cycles.

The FHWA has sponsored the development of an improved thermography system for the detection of delaminations. Thermography has been used in recent years to determine where delaminations exist on bare concrete bridge decks. Thermal images of the bridge deck show areas where heat transfer properties of the bridge deck are perturbed by the existence of a delamination. These areas typically heat up more quickly than sound concrete when exposed to solar heating during the day, and cool down more quickly at night. Typical application of this technology is hampered by emissivity variations caused by surface clutter on the deck. The Dual Band Infrared Thermography System is designed to compensate for emissivity variations by simultaneously acquiring infrared images at two different wavelengths. The data is then processed to create a map of emissivity variations on the deck to aid in the interpretation of data. This system is currently being field tested to determine if the method is effective and practical under actual field conditions.

The FHWA has also sponsored the development of a Ground Penetrating Radar Imaging System that uses a new generation of micro-radar antennas. The system uses impulse radar, synthetic aperture techniques and advanced signal processing to create 3-dimensional images of internal structures in concrete. The radar system consists of a of a 64 element antenna array mounted in a towable trailer. The system is designed to image an entire lane width of a bridge deck in a single pass at highway speeds. A two dimensional slice of a concrete deck is shown in figure 1. Shown in the image is a void located approximately 10 centimeters below the surface of the asphalt covered concrete deck. This image was created in the field with a single radar antenna, scanned over the bridge deck to simulate the operation of the towable radar system. The system is currently being constructed at Lawrence Livermore National Laboratory. Field testing of the complete system is scheduled to begin in October, 1997.

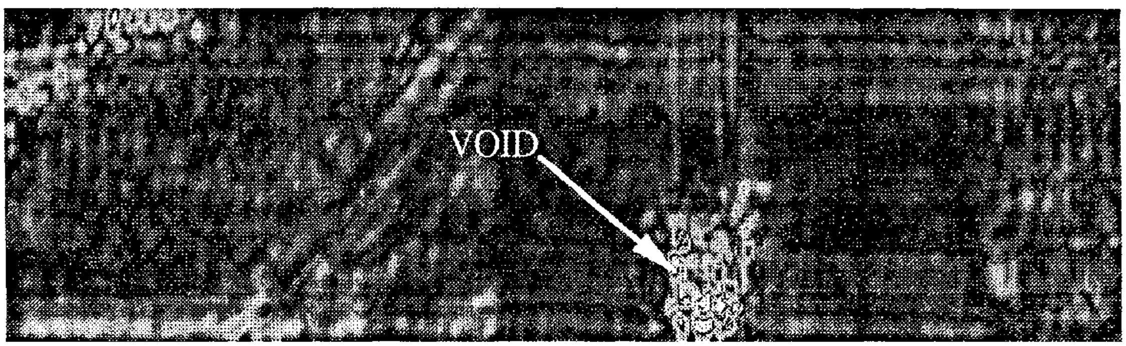

Figure 1 GPR Image of concrete bridge deck containing a void.

ADVANCED BRIDGE INSTRUMENTATION

The nondestructive testing of highway bridges often includes the analysis of the load carrying capacity of the structure, or load rating. This is typically accomplished by instrumenting the bridge with strain gages, accelerometers, deflection gages and tiltmeters, all hardwired to a data acquisition system. The response of the bridge to an applied load is then measured and analyzed. A tremendous advantage could be gained if measurements could be made without the time consuming and costly task of wiring the instrumentation to a single data acquisition system. Two systems have been developed through the NDE research program to address this need for wireless or remote measurement of bridge performance.

The Coherent Laser Radar System is designed to provide remote, non-contact deflection measurement of structures under load. The system uses a frequency modulated, eye-safe laser to make precise range measurements. The laser is fitted with servo-controlled motors to allow scanning over large sections of a structure from a single point. No targets are required to make the measurements. The laser radar system has a range of approximately 30 meters, and a range measurement accuracy of less than 1.0 mm. The system has been used to measure the deflection of an abutment wall under load, out of plane distortion in a welded plate girder undergoing lateral torsional buckling, and deflections of bridge beams over a roadway open to traffic. Field testing is currently underway to investigate how the laser system can be integrated into load rating procedures used by the State Departments of Transportation.

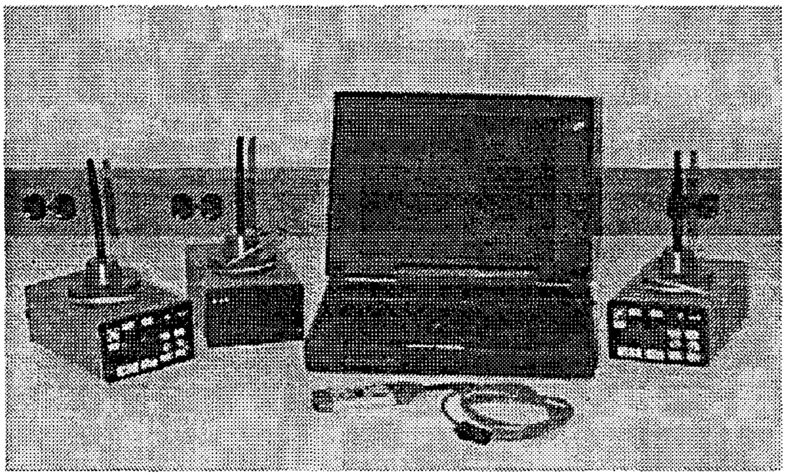

Figure 2 Wireless data acquisition system with 2 radios, 1 home base and laptop interface computer.

A wireless bridge monitoring system has also been developed. This system uses a network of low cost microwave transponders to telemetry data from locations on the bridge to a local controller. The transponder units can measure strain, rotation, deflection and vibration. Data is collected at the transponder, then downloaded to the local controller for analysis. This greatly reduces the time and cost associated with running wires from the instruments to a data acquisition system, as well as reducing the noise that often results from long wire runs. The system can be quickly configured with up to 10 transponders in a single network. Sensors configurations for use with these transponders, such as fieldable tiltmeters, slide wire potentiometers and full bridge strain sensors are being developed, as well as an improved software interfaces for use by bridge inspectors..

FUTURE DIRECTIONS IN NDE

Perhaps the greatest problem facing the development and implementation of new NDE technologies is the lack of suitable evaluation methods. The diversified and highly technical nature of most NDE tools makes this evaluation complicated and costly, particularly for State Departments of Transportation. The need for objective, reliable evaluation of NDE technologies for the inspection of highway bridges is clear.

The FHWA is answering this need with the development of the FHWA NDE Validation Center, located at the Turner Fairbank Highway Research Center in McLean, VA. The objective of this new center is to provide researchers, industry, and State highway agencies with quantitative, independent, and reliable validation of NDE methods. The Center will develop specimens and methodologies to validate NDE performance both in the laboratory and in the field and serve as a resource for the highway and bridge inspection community.

The NDE Validation Center includes elements of both laboratory and field testing for the evaluation of NDE methods:

1. Developing Validation Protocols - The primary role of the NDE Validation Center will be to develop and execute effective test protocols for validating the performance of NDE methods. Component specimens and test bridges will serve as a critical resource in the process by simulating the many factors that effect the

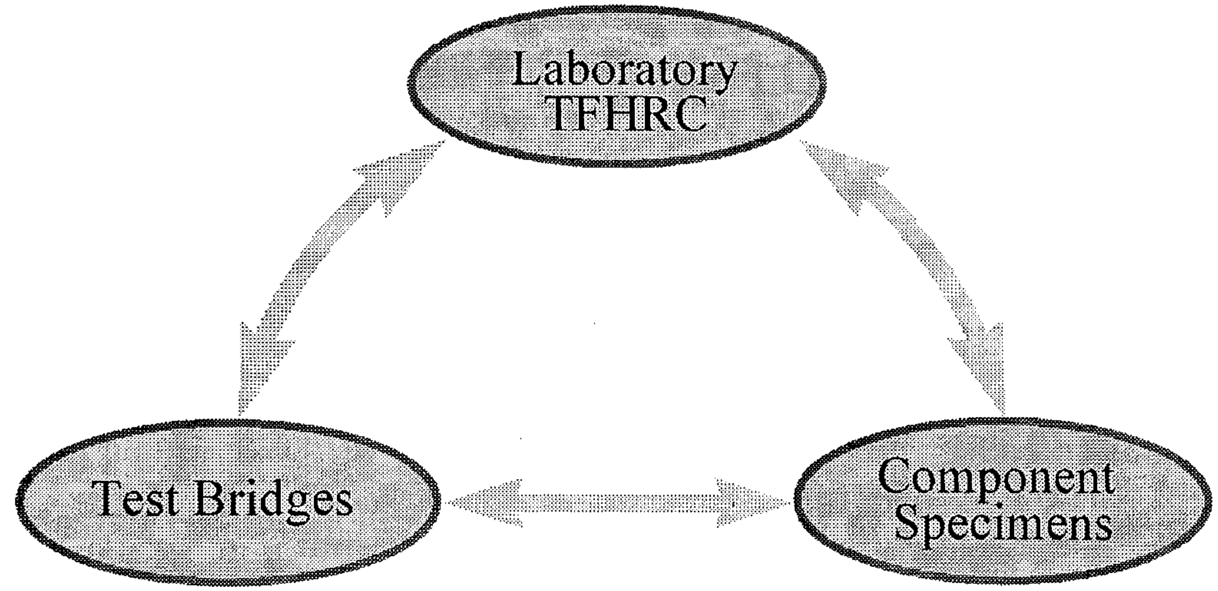

Figure 3 NDE Validation Center elements.

reliability of NDE methods. Testing at the Center will help define the proper application and limitations of NDE methods, and determine the ability of NDE methods to provide quantitative measurements of flaw size, material properties or structural condition. The validation process will provide performance parameters for NDE methods, procedures and systems to enhance safety and maintenance inspections of the highway system.

2. Component Specimens - The NDE Validation Center will develop, test and catalog a library of bridge component specimens containing flaws. Typical components to be included in this library are welded details containing fatigue cracks, prestressed concrete beams with broken and corroded strands, and concrete decks sections containing delaminations. The specimen library will contain a scientific distribution of flaws of different types and sizes, suitable for determining the probability of the detection for NDE methods. These component specimens will play a critical role in the validation process, allowing for quantitative evaluation of NDE methods at the Center. Component specimens will be available on loan to researchers and State highway agencies wishing to evaluate new or currently used technologies.

Test Bridges - Seven test bridges have been selected for use as part of the NDE Validation Center. This test bridge population includes five decommission highway bridges that will be used to evaluate NDE technologies. These structures were built in 1940, and were part on the Pennsylvania turnpike through 1968 when the turnpike was rerouted. There are three concrete T-beams, one riveted steel structure and one concrete rigid frame. A thorough inspection of these structures using state of the art methods will be conducted to determine the size and location of damage. Defects will be implanted in the structures where appropriate. For instance, cross frame connections containing fatigue cracks will be installed on the steel bridge to provide a realistic testbed for evaluating crack detection methods. The structures will be utilized to determine the ability of NDE methods to detect and quantify defects under actual field conditions.

The test bridge population will also include a large steel bridge that is open to traffic, and will be used to test NDE methods affected by live loading. This structure will be fully instrumented and linked via a modem to the Center so that the environmental and structural conditions during a test are well defined. A detailed finite element model of the bridge will be created and maintained at the center. This bridge will be instrumental to evaluating techniques for dynamically evaluating structures. Another steel bridge, open to traffic but seldom used, will be instrumented for use in evaluating load rating instrumentation and methods. Since this structure has a very low volume of traffic, it will convenient for applying the calibrated loads used to load rate structures.

These test bridges will be critical to evaluating the effect of restricted access, structure geometry, surface conditions, platform stability, and human factors on the application of NDE methods during normal bridge inspections. Through the use of actual bridges and realistic component specimens, the NDE Validation Center will provide the most comprehensive test facility for evaluating NDE methods for highways in the world.

The NDE Validation Center is designed to provide a national resource for the development and application of NDE methods. The Center will provide consistent and reliable evaluation of NDE methods, and accelerate the implementation of NDE in the inspection process.

Researchers, industry and users are encouraged to participate in the development and operation of the NDE Validation Center. The resources of the center can be available to researchers from around the world that would benefit from having more effective test specimens for developing NDE technologies. For more information, please contact the author at the address listed.

CONCLUSION

It is evident from the discussions at this seminar that both Japan and the United States are experiencing difficulties in evaluating the condition of their existing infrastructure. Quantitative methods for determining the in-situ strength and load carrying capacity of highway bridges, buildings and other infrastructure facilities are badly needed. This applies not only to disaster mitigation, but to the routine maintenance required to ensure public safety and extend the life of the facilities.

Quantitative nondestructive evaluation methods being developed in the U.S. and Japan can have significant impact on these difficult problems. Research efforts in each country are not currently coordinated, and improved technology transfer and cooperation is needed to expedite the development of these critical technologies.

Additionally, the development of quantitative NDE technologies in both countries is hindered by the difficulty in evaluating the reliability and effectiveness of new methods and instrumentation. The NDE Validation Center described in this paper is aimed at reducing the cost and time required to perform these evaluations, and the participation from researchers in both countries is strongly encouraged.

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INSPECTION METHODS OF DAMAGED STRUCTURES CAUSED BY EARTHQAUKES

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ABSTRACT

The establishment of inspection techniques for seismic damage to civil infrastructure systems, such as highway viaducts, is an urgent need to diagnose the condition of the structures just after the earthquake because those structures have to be immediately assessed either reusable or not. However, inspection of seismic damage to underground structures or covered structures such as piles or jacketed columns are difficult because the damage to such structures is not easily to see. Therefore, research and development on non-destructive techniques to inspect the seismic damage condition of such structures is necessary. This paper presents introducing advanced non-destructive inspection techniques for the seismic damage to the viaduct piles suffered the great 1995 Hanshin earthquake; bored-hole camera inspection and pile integrity test methods. These methods successfully inspected the number and the location of the cracks along the pile axis. This paper also discusses the inspection and evaluation techniques that should be prepared for the future events.

INTRODUCTION

The establishment of inspection techniques for seismic damage to civil infrastructure systems, such as highway viaducts, is an urgent need to diagnose the condition of the structures just after the earthquake because those structures should be immediately assessed either reusable or not. Many highway viaducts suffered seismic damage in the previous earthquake; however, the damage was generally developed at bearings or longitudinal bar cut-off zone of reinforced columns. Therefore only visual inspection has been employed to assess the seismic damage to viaduct structures. In other words, no other advanced inspection techniques had been developed yet. In case of the great Hanshin earthquake, minor cracks on piles caused by excessive earthquake load from the piers and/or liquefaction of the ground, which had not been observed previously, were found. However, there was no sufficient preparation in terms of both inspection and evaluation methods to seismic damage to such underground structures. The same thing will happen to covered structures such as jacketed RC members for retrofitting. The damage is not easily to see from the outside because the damage occurs to the inside of RC members. Therefore, research and development on non-destructive techniques to inspect the seismic damage condition of the structures such as underground structures or covered structures such as piles or jacketed columns is necessary.

Figure 1 shows the possible location of principal plastic hinges of viaduct structure designed according to the present design code[1]. In case of the conventional design method, principal plastic hinge is designed to occur at the bottom of the column and also secondary plastic hinge will occur at the pile top section.

In case of the menshin design (seismic isolation design), principal plastic deformation will occur at menshin device or seismic isolation bearings that are designed to absorb the seismic energy and also secondary plastic hinge will occur at both the pile top section and the bottom of the column. In case of the bridge supported by a wall pier, principal plastic hinge is designed to occur at the pile top section. From the view point from seismic damage in future, the damage will be developed into underground if menshin design is not introduced. From this point, it is also an urgent need to develop inspection and evaluation method to detect and assess seismic damage occurred in underground structures.

This paper mainly introduces non-destructive investigation methods and the result on the bridge pile foundations damaged by the Great Hanshin Earthquake with emphasis on inspection method. And later part of this paper discusses the inspection and evaluation techniques that should be prepared for the next seismic damage that will happen in underground or other invisible part of structures in future.

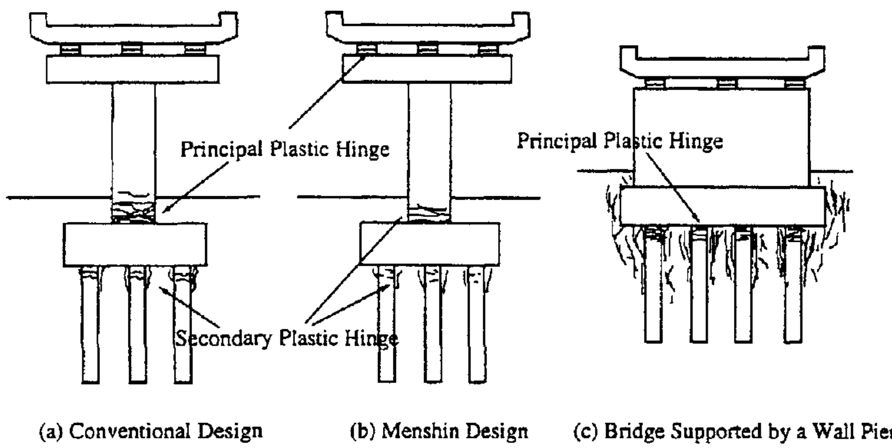


Figure 1. Location of primary plastic hinge in case of significant earthquake

INSPECTION AND EVALUATION OF DAMAGED PILES DUE TO THE GREAT HANSHIN EARTHQUAKE

INSPECTION METHODS FOR PILE DAMAGE

In case of the great Hanshin earthquake in 1995, a lot of elevated structures suffered significant damage from the Great Hanshin Earthquake in January 1995. The size of the region affected and severity were extensive. What is the most notable in this event is that a lot of damage was found to the piles of viaducts. As shown in earlier, there is no inspection technique firmly developed yet at that time. Therefore the damage of the pile had to be inspected without sufficient preparation in terms of both inspection and evaluation methods. Three inspection methods were primary employed. The most reliable method of inspection was direct visual inspection which was expensive and took time, so two advanced non-destructive inspection techniques; bored-hole camera inspection and pile integrity inspection that required much less time, were mostly employed.

The inspection results revealed that the damage to the piles are different between the Kobe route, which passed over the inland area, and the Wangan route, which connects recently-reclaimed island. The seismic damage to the pile of the Wangan route, especially along the perimetric sections of the reclaimed land where the ground underwent liquefaction were severely damaged. However, the damage to the foundations of the Kobe route and of the in-island areas of the Wangan route was comparatively small so they could be reused.

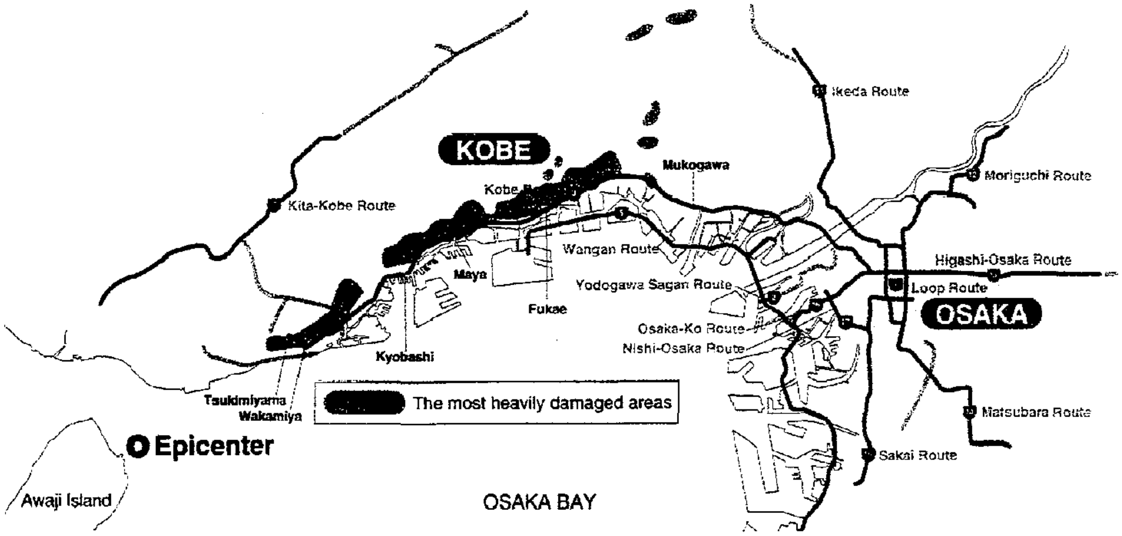


Figure 2 The location of the Kobe and Wangan route and affected area

INSPECTION METHODS FOR PILE DAMAGE

General

Investigation of pile damage should be conducted from the view point of investigating both damage to pile body itself and ground condition around bridge. The investigation of the ground condition around piers was conducted by visual inspection. GPS (Global Positioning Survey) was also conducted to survey the ground movement and the movement of piers as well. As concern to the inspection of pile body it self, the methods listed in table 1 were mainly employed. However, these inspection methods were not well developed for inspecting the seismic damage to piles so that there is much room for improving these techniques.

Table 1. Inspection methods for damage to bridge foundations

Investigation	Inspection Methods
Eye inspection	Direct visual inspection of surface cracks on a pile body.
Bored-hole camera inspection	Indirect visual inspection of inside pile cracks with a small camera inserted into a hole bored in a pile.
Pile integrity test	Nondestructive inspection of cracks and defects in piles by analyzing reflected impact wave.

Direct Visual Inspection

Direct visual inspection was the most direct and reliable way in determining the damage because it allows their direct observation by excavating and exposing the pile top section, though only externally. Orientation and intensity of cracks in the piles can be indispensable in determining the degree of damage. On the other hand, due to the need for excavation, the benefits of this inspection method depend on the labor and equipment available for excavation, and inspection area is only limited to the pile top section, not the entire length. Prevention work for secondary shock disaster by putting temporary bent to girders and pier beams may prevent these inspection work so that it is impossible to check the pile damage of severely damaged piers. It may also be necessary to prevent water ingress near the sea.

Bored-hole Camera Inspection

Direct visual inspection is very reliable but it takes time to expose the piles, and only the pile tops can be inspected, not the whole length. An alternative method was an indirect visual method using a bored-hole camera, which allows inexpensive, rapid investigation to piles to a similar extent to direct visual inspection. Bored-hole cameras have been used to investigate bedrock, mainly to check for joints, cracks and faults prior to dam construction. Here, a hole is bored down the center of the target pile and a small camera is inserted in the hole to allow assessment of the conditions inside. For this inspection, a 66-mm-diameter bored-hole was made in a pile and a small TV camera was inserted into the hole to look for any cracking in the hole wall. Figure 4 shows the configuration of the inspection equipment. This method evaluates the damage based on the information of a 66mm ϕ hole in a 1000~1500m ϕ piles so that it needs careful consideration to assess the whole pile damage based on observed inside crack width and inside crack density.

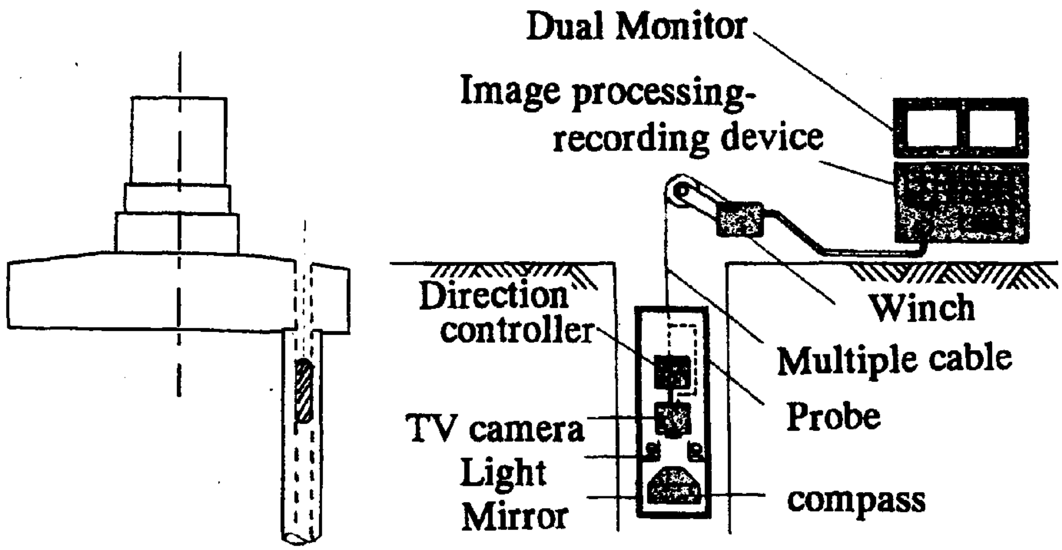


Figure 4 Indirect visual inspection (Bored-hole camera inspection)

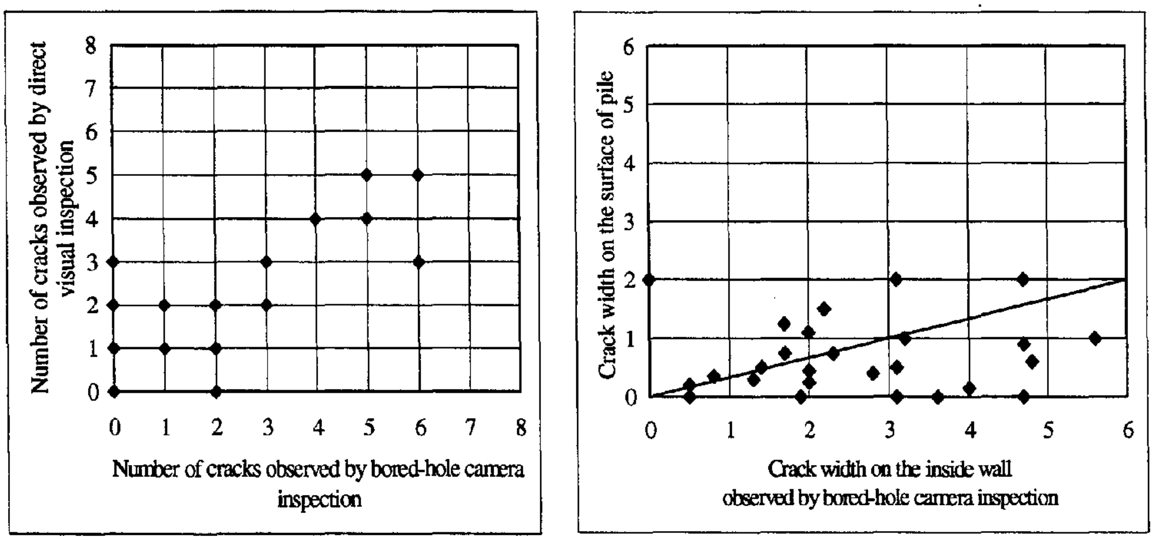


Figure 5 Number of cracks and crack width compared between external observation and bored-hole camera observation

A comparison between the external observation and this result as shown in Figure 5 indicates that the crack positions as detected by the camera generally coincided with those identified by direct visual inspection, verifying the accuracy of the bored-hole camera method. One problem, however, was that the widths of the cracks detected by the camera were different from by direct visual inspection. Possible reasons for this are that scraping during the boring process made the cracks larger; and that shadows from the lighting lead to incorrect measurements. In any case, there is room for improvement when evaluating the total damage to a 1000mm-diameter pile by checking the wall of a 66mm bored-hole.

Pile Integrity Test

The principle of the pile integrity test is to generate an impact wave at the pile top with a plastic hammer and measure the reflected wave with an accelerometer. And the basic idea of this test method is to analyze a reflected elastic wave, which can be achieved rapidly and at a fairly low cost. It is a very useful, effective way of roughly assessing the damage to a pile. The principle of the pile integrity test is to generate an impact wave with a hammer, and measure the reflected wave with an accelerometer. The wave contains information on changes in the pile cross section and cracks collected while it travels through the pile body as shown in Figure 6. Analyzing disturbances in the wave allows the presence of cracks and other defects to be judged, leading to an estimation of the amount of damage to the pile. Besides many advantages of this method, there are two problems: it requires fairly expert knowledge to analyze the inspection results, and the evaluation method is not yet to be developed.

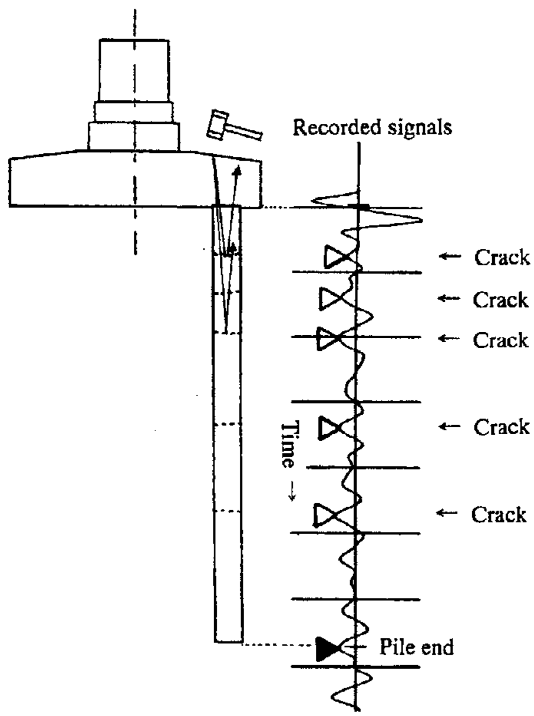


Figure 6 Nondestructive inspection (Pile integrity test)

Figure 7 shows the comparison result between pile integrity test and bored-hole camera inspection. Here, the result was modified from the original output wave and arranged using the idea of wave irregularity . The wave irregularity is the original idea that amplifies irregularity of the observed wave. As shown in this figure, the defect considered as crack can be located and the crack damage can be seen as irregularity. Pile integrity test results tend to underestimate the number of multiple adjacent cracks. However, both inspections show similar results in terms of the locations of damage.

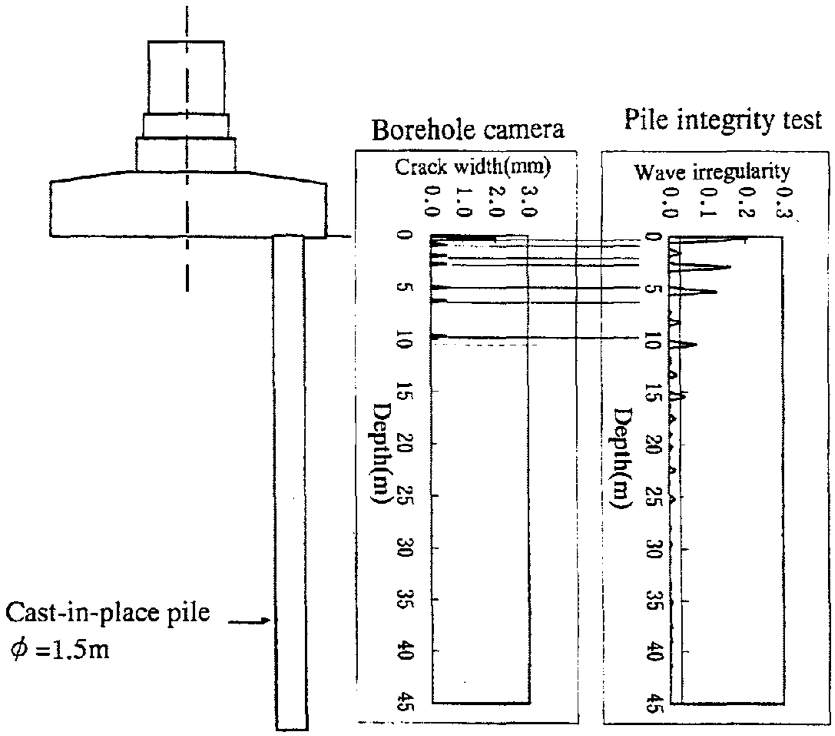


Figure 7 Comparison result between pile integrity test and bored-hole camera inspection.

Photo 1 The worst damage found on the pile of the collapsed piltz bridge

INSPECTION RESULT

Typical Inspection Result of the Kobe Route

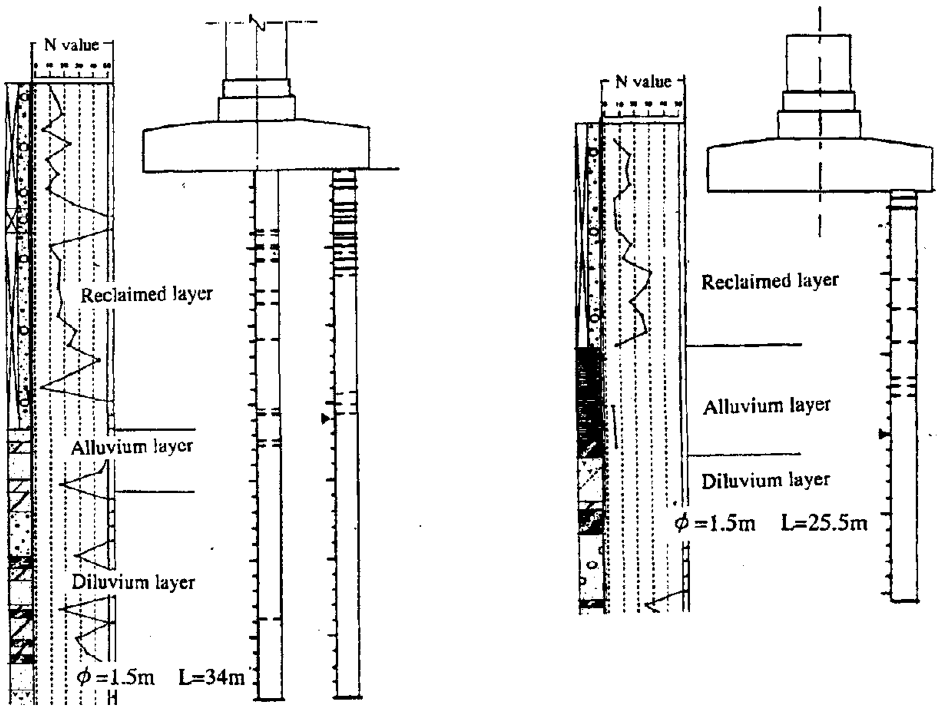
Here, the inspection results of the piltz bridge foundations, an elevated section which fell down, are introduced. The elevated, 18-span continuous concrete bridge collapsed due to flexural-shear failure of the columns. Photo 1 shows the worst example of the direct visual inspection results. As the foundation was totally exposed, all the pile tops were checked visually. Cracks were found in all the pile tops, the widest being about 2 mm in width. On the other hand no cover concrete was spalled from any pile, and hammer tests revealed that only normal concrete acoustics were heard. So the condition of the piles of the Kobe route was judged good enough to reuse. A typical captured crack obtained by bored-hole camera is shown in Photo 2. This photo shows the 360 degree spread-out picture of the inside wall of the 66mm diameter hole. The black zigzag lines shows the cracks.

Photo 2 The bored-hole camera inspection result on the pile of the piltz bridge

Typical Inspection Result of the Wangan Route

Along the Wangan route that connects recently-reclaimed islands, liquefaction occurred after the earthquake, causing large displacements in piers, especially along the waterfront of reclaimed lands. Many piers near the waterfront had moved horizontally toward the sea side, and that the ground near the waterfront had settled much around a half meter.

Figure 8 show the typical inspection results of the foundations both near the waterfront and inland. The foundations along the waterfront suffered cracks that were distributed evenly along the piles in the reclaimed layer, in alluvial deposits, and even in diluvial deposits. The foundations in inland area also had cracks down to the boundary with the diluvial layer as well as in the reclaimed layer, and in the pile tops; however, the density of cracking was less than that of waterfront area.



(a) near water front (b) in reclaimed-island

Figure 8 Crack distribution of piles near the waterfront and inland.

DAMAGE EVALUATION TECHNIQUE

Using the result of non-destructive inspection of viaduct piles, the damage level was categorized according to the criteria in Table 2. The results are listed in Table 3. This result shows that some foundations of the Kobe route ranked in category c, but the majority of them were ranked d; in general there was little damage to the foundations of the Kobe route. The foundations along the perimetric area of reclaimed on the Wangan route, however, suffered b ranked damage while many on the newly-reclaimed land c. The foundations in areas near the center of the reclaimed land ranked d.

Table 2 Foundation damage evaluation criteria

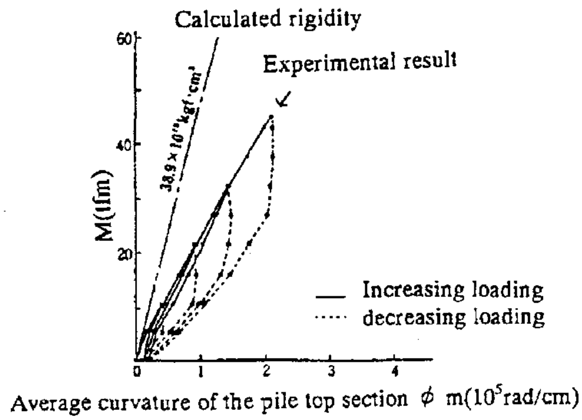
	Definition
a	Both settlement of the foundation and large residual displacement are found.
b	The foundation has a large residual displacement and piles have flexural cracks.
c	Piles have small flexural cracks.
d	Pile has no damage or has fairly small, negligible flexural cracks.

Table 3 Foundation damage evaluation results (No. of foundations)

Degree of damage	**a**	**B**	**c**	**d**	Total
the Kobe route	0	0	17	92	109
the Wangan route	0	17	57	79	153

In order to verify the possibility of reusing c-ranked piles, loading tests were conducted for the piles typically ranked **c**. Three kinds of load test (a pile bending test, vertical load test and horizontal load test) were conducted. The pile bending test is discussed here.

The pile bending test was carried out for the pile top section where cracks had developed. The pile was cut off 2.8 m below the pile cap and the top section was used as the test specimen. A horizontal force was applied in one direction at several levels. A maximum applied force was equivalent to about 60% of the calculated yielding bending moment at the pile top section. The observed M- ϕ relation of the pile top section is shown in Figure 9. As can be seen from the figure, no residual curvature but a certain drop in rigidity is observed in the pile top section. This indicates that the steel bars of the pile had experienced just beyond yield stress. The bearing capacity of the entire foundation w/ or w/o damaged top pile section was calculated using the observed pile top rigidity. As shown in Figure 10, the bearing capacity of the entire pile foundation is not affected much w/ or w/o damage at the pile top section. This indicates that the c ranked piles had enough strength so that they were judged to reuse.



F igure 9 Observed load-curvature relation and calculated result w/o damage

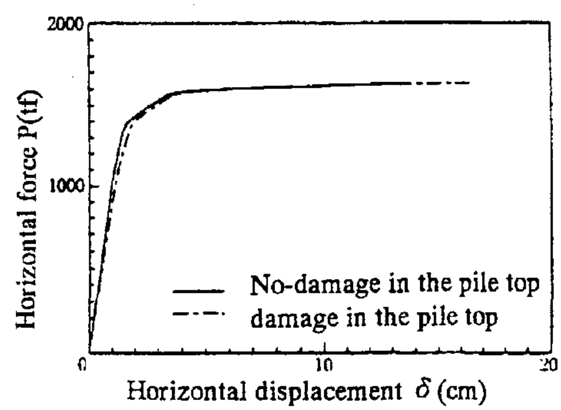


Figure 10 Calculated bearing capacity of the pile of the Piltz bridge

FUTURE NEED ON INSPECTION AND EVALUATION TECHNIQUES OF SEISMIC DAMAGED STRUCTURES

DETECTION OF DAMAGE:

In the future earthquake event, damage will be developed at invisible part of structural members so that damage detection techniques for covered members or underground members should be established.

- Damage to covered members: Jacketed columns
  - To detect the damage inside concrete and bars
- Damage to members under the ground: Column bottom and foundations

-To detect the damage at invisible section of members

- Monitoring methods to detect damage
  - -Sensoring techniques detect and specify the damage and the location

POSSIBLE DAMAGE DETECTION TECHNIQUES:

The non-destructive techniques listed below are possible damage detection techniques for covered members and underground members.

- Damage detection method
  - -Bored-hole camera inspection
  - -Pile integrity test
- Damage detection method using the response characteristics
  - -Changing of natural period
- Sensor technology
  - -Application of fiber sensors and smart materials

EVALUATION OF DAMAGE DEGREE:

The evaluation technique of remained strength or reusable capability is difficult to be established so that continuing effort is needed to accumulate experimental data.

- The relation between detected damage information and damage degree or reusable capability
  - -Accumulation of experimental data
  - -Development of evaluation method of seismic damage

OTHERS:

The design details such as re-bar arrangements for RC columns were sometimes missing for older bridges. When evaluating the seismic performance or designing the retrofit methods, advanced non-destructive technologies are effective.

-Ultrasonic wave, electromagnetic wave, and so on

-Removing cover concrete by water jet

SUMMARY

In this paper, the need for developing inspection techniques for seismic damage structure, especially underground or covered structure, is summarized. And the techniques that were employed for inspecting the seismic damage to piles are introduced. However, the methods for evaluating the remained strength or seismic safety of the damaged structures based on the information of non-destructive inspection results has not been well developed yet. Therefore, more research on these topics should be conducted.

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Integrated Real-Time Disaster Information Systems
The Application Of New Technologies

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ABSTRACT

This paper provides a brief overview of how new technologies were used after the Northridge earthquake to improve lifeline response and recovery. Technologies that are discussed include real-time availability of earthquake source data, improved loss estimation techniques, Geographic Information Systems and satellite-based monitoring systems. This paper also identifies areas in which further integration of these methods into emergency response would improve the survivability or reliability of lifeline facilities or services.

INTRODUCTION

At the time of the Northridge earthquake, a number of new technologies, including real-time availability of earthquake source data, improved loss estimation techniques, Geographic Information Systems and various satellite-based monitoring systems, were either available or under consideration as emergency management resources. The potential benefits from these technologies for earthquake hazard mitigation, response and recovery however, was largely conceptual. One of the major lessons learned from the January 17, 1994 earthquake was that these technologies could confer significant advantages in understanding and managing a major disaster, and that their integration would contribute a significant additional increment of utility.

The technologies discussed in this paper developed as discrete entities, in diverse disciplines and at different rates relative to one another. At the time of the Northridge earthquake, near real-time broadcasts of earthquakes had been available to utility and lifeline operators in southern California since 1990, although emergency management application of this information remained mainly experimental. Loss estimation, on the other hand, has been in use for two or three decades by both government and the private sector but has evolved, in recent years from a single event scenario format to a more versatile automated one.

Geographic Information Systems (GIS), like loss estimation, have a history spanning the last quarter century but only within the last several years have they been applied as an emergency management tool, and only in the Northridge earthquake were they used extensively in disaster response and recovery. As part of the "defense conversion" in the aftermath of the Cold War, satellite monitoring systems have been made available for peacetime purposes and have already been successfully applied in weather and storm surveillance and have been applied, to a limited extent, in other natural hazard analysis.

In this paper, we will examine these various new technologies, their contributions to response and recovery after Northridge and their actual or potential application to improving lifeline performance. Although this discussion will treat these technologies as discrete scientific developments, we will attempt to identify areas in which there has been convergence and where further integration could produce significant improvements in the survivability and continued functionality of lifeline facilities and services.

NEW TECHNOLOGIES

REAL-TIME SEISMOLOGY

Although real-time seismic monitoring has been applied to promote public safety in Japan, similar applications have been slow to materialize in the United States. Following the Loma Prieta earthquake, a survey was conducted to assess the perceived utility of an earthquake early warning system which would provide a few seconds of warning in advance of damaging levels of shaking . Researchers found that most groups which were expected to enthusiastically endorse this new technology were luke-warm, at best, considering a few seconds warning too little time to take meaningful action in advance of a major earthquake. Nor was such a system regarded as a cost-effective strategy for hazard reduction and thus, unlikely to receive much support from users (Holden et al., 1989).

Despite this somewhat disappointing beginning for real-time monitoring, Caltech and the U.S. Geological Survey pushed ahead with plans to upgrade the Southern California Seismic Network (SCSN) to provide real-time broadcasts of earthquakes and seek the support of government agencies, large utilities and corporations for the potential post-event benefits these broadcasts might provide. Gradually, participants were recruited to participation in the Caltech-USGS Broadcast of Earthquakes (CUBE) and by the time of the Northridge earthquake, there were 18 Level III participants in the Earthquake Research Affiliates of Caltech (ERA), the organizational vehicle for provision of real-time earthquake broadcasts from SCSN. These participants made an annual contribution of $25,000 to the Caltech Development Fund (which helped support the purchase of new seismic monitoring equipment) and received reports of earthquakes via pagers which carried the magnitude, location, time and other earthquake data within 2-5 minutes of occurrence.

For approximately four years, CUBE reliably broadcasted earthquakes up to about M5.5 to participants. At 4:31 a.m. on January 17, however, the CUBE system experienced both hardware and software difficulties which resulted in delays of approximately one hour in providing information on the Northridge earthquake. A study currently under way is designed to determine how response of CUBE participants was affected by the performance of CUBE after the Northridge earthquake, and in general, how receipt of real-time earthquake information from SCSN has been integrated into the response plans of these organizations (Goltz, 1995).

Preliminary results from this study indicate that the CUBE system usually provides the first data upon which action may be taken. To assist in determining whether a response is justified, many CUBE participants have customized the Q-pager software (which displays spatial and other data on an earthquake) by adding overlays to the basic southern California map on which earthquake epicenters are plotted. These overlays include facilities such as transmission lines, dams, power stations or other critical structures. In a few instances, participants reported monitoring the patterns of small earthquakes in a particularly critical location under the assumption that these earthquakes may be precursors to a large damaging event.

Statewide real-time broadcasts are now available and have been adopted by those agencies (OES and Caltrans) and companies (utilities and railways) with statewide jurisdiction, facilities or distribution networks. These statewide broadcasts are available through a cooperative agreement between Caltech (CUBE) and the University of California at Berkeley (REDI or Rapid Earthquake Data Integration). In March, 1995, CUBE participants received software which permits the mapping of ground acceleration data broadcast in real-time from the SCSN. Most users consider this to be a very important development in that the potential impact on facilities can be more precisely estimated and mobilizations of personnel for inspections can be expedited.

The development of real-time monitoring and broadcast of seismic information appears to be in transition from concept and pilot project to a practical operational system. Under the strictest definition of real-time availability, the Northridge earthquake must be considered a missed event. It appears, however, that the January 17 event exposed a plethora of technical vulnerabilities in the CUBE system and triggered a concerted effort to remedy most, if not all of them. It does not appear that the failure of the CUBE system to provide rapid and accurate information on the earthquake had a dampening effect on the interest and enthusiasm of users.

LOSS ESTIMATION

The Northridge earthquake was the first earthquake disaster to occur in the United States in which important emergency response and early recovery decisions were based on loss estimates as well as actual data gathered through standard reconnaissance procedures. The utilization of loss estimation techniques in the immediate post-earthquake context is a key development and marks a significant departure from conventional applications. For the last two decades, earthquake loss studies have addressed the pre-earthquake planning needs of utility operators and government agencies. Although the needs of these users vary, understanding risks and maintaining public safety have generally required similar scenario events, that is, those with the greatest impacts on the local population and economy.

Historically loss estimation studies for planning purposes have identified scenario earthquakes as the focal point estimate development. These studies use existing knowledge of regional geology and seismology to generate maps with estimated intensities and, based on projected ground motion and other factors, estimate damage to buildings and structures, lifelines and impacts on population. Some studies also address potential secondary hazards such as fire, flood and hazardous materials releases. Presented in report or document form, often with fold-out and sometimes overlay maps, these earthquake scenarios have been employed by government and utilities to prepare and mitigate potential earthquake losses. Thus, the typical loss study has been single-event focused, applied in the long-term pre-event period and utilized primarily by those concerned with seismic safety planning and risk management.

Even before the Northridge earthquake, technological developments were rapidly rendering these scenarios and the formats in which they are presented obsolete. The advent of high speed computing, satellite telemetry and Geographic Information Systems (GIS) have made it possible to electronically generate loss estimates for multiple earthquake scenarios, provide a nearly unlimited mapping capability, and perhaps most important, develop estimates for an actual earthquake in near real-time given the source parameters of magnitude and location. For the last five years, real-time broadcasts of earthquake data including magnitude, location, depth and time of occurrence, have been available in southern California on an experimental basis from the Southern California Seismic Network operated jointly by the California Institute of Technology and the United States Geological Survey.

Currently under development for the California Governor's Office of Emergency Services (OES) is a GIS-based system capable of modeling building and lifeline damage and estimating casualties in near real-time given the source parameters of an earthquake (Eguchi, et al, 1994). The Early Post-Earthquake Damage Assessment Tool (EPEDAT) will be completed in mid-1995, will utilize real-time seismic information from the Southern California Seismic Network and will operate on a personal computer. EPEDAT will utilize fault and seismicity data to locate the most likely source of an earthquake, as broadcast by CUBE. Applicable ground motion and soil amplification models will be employed to estimate the expected intensity patterns in the affected area. These intensities will then be overlaid onto the computerized data files containing an aggregate listing of buildings and lifelines in the region. Based on damage and casualty models already developed under contract with the State of California, and the intensity patterns computed, building and lifeline damage as well as casualties will be estimated for the impacted area. EPEDAT will also allow the user to update early estimates with actual reconnaissance data. Thus, as more accurate and specific data become available, they can be incorporated to refine or correct initial predictions.

Real-time loss estimation offers direct and tangible benefits over conventional loss studies. For the first time, loss estimation can be used in the emergency response and early recovery phases of a disaster as a decision support tool. Based on rapid receipt of magnitude and location, estimates of ground shaking and intensity can be calculated and mapped giving emergency responders a sense of the overall scope of an event minutes after it has occurred. Reconnaissance efforts can focus on those areas projected to be hardest hit based on estimates of intensity. Inspection teams can be dispatched, staging areas can be identified, medical emergency resources mobilized and sheltering needs considered, as responders are guided by estimates of damage, casualties and displaced persons. Recovery can be hastened by calculation of total dollar loss estimates and breakdowns of losses by facility or structural type, usage or other categorizations expedite governmental disaster assistance.

Immediately following the January 17 earthquake, data and models which will be the basis for EPEDAT in Southern California were utilized and, though the system was not fully operational at the time, produced timely and accurate information which was used by state and federal officials to support key policy and program decisions. Estimates of dollar losses and ground shaking intensity were used by the California Office of Emergency Services to define the general regional scope of the disaster during the critical 24-48 hours after the event. The shaking intensity map was instrumental in approximating the locations of heaviest damage, used in briefing state agency executives, including the Governor, and in making decisions regarding shelter needs, locating Disaster Application Centers and "fast-tracking" the federal Disaster Housing Assistance Program. A total dollar loss estimate of $15 billion, generated within 24 hours of the earthquake, served as the basis for negotiation of a supplemental Congressional appropriation of $8.6 billion in federal disaster assistance.

GEOGRAPHIC INFORMATION SYSTEMS (GIS)

In the context of emergency management, GIS technology has rapidly evolved over the last decade and, in the aftermath of the Northridge earthquake, is regarded as a vital response and recovery decision support system. An obvious advantage for response and recovery officials is the ability to gain both a regional and highly localized assessment of situations requiring rapid decision making. Such decisions are facilitated by the interrelationship of structural, demographic, economic and environmental data in mapped as well as tabular formats. The utility of GIS has been enhanced by technological developments in related areas including remote sensing, global positioning, software and hardware improvements and desktop applications (Topping, 1993).

A GIS unit had been established within the California Governor's Office of Emergency Services prior to the Northridge earthquake; after the January disaster, however, this unit was greatly enhanced with additional personnel, new equipment and technical expertise available on a consulting basis. Co-located in the Pasadena Disaster Field Office with its GIS counterpart in the Federal Emergency Management Agency, this unit provided an important source of information in both the response and recovery phases of the Northridge earthquake. In addition, this unit has taken major strides toward development of a disaster management database for California.

In the immediate aftermath of the Northridge earthquake, GIS was used to develop a shaking intensity map based on model estimates. This map was instrumental in gaining a rapid assessment of the regional scope of the disaster and supported key decisions regarding the location of shelters and disaster application centers and documenting the state's request for a presidential disaster declaration (Goltz, 1995). As recovery programs were initiated, GIS was instrumental in identifying and displaying language and demographic characteristics, geographically locating disaster assistance applications and in hazard mitigation planning (Topping, 1994).

For lifeline operators, GIS also played an important role in Northridge response and recovery. The wastewater collection system for the City of Los Angeles consists of a complex network of underground sewers, both gravity driven and forced mains. In total, over 7,000 miles of sewer pipe traverse the City of Los Angeles. As in past earthquakes, damage to underground sewer pipes has been difficult to detect because the effects are not immediately visible, unless ground failure has occurred. In some cases, leaks and breaks are only detected when adjacent water mains are filled and wastewater spills onto the ground and street because of blockages caused by internal damage or collapse. Other indicators of severe sewer damage include street settlement, crushed or buckled curbs, damage to sidewalks and failed water mains.

In an attempt to quickly identify areas of severe sewer damage after the Northridge earthquake, the City of Los Angeles relied on GIS (Solorzano et al., 1994). First, GIS was used by the Bureau of Engineering to identify areas of extensive building damage, water main repair and surface disruption (i.e., damage to sidewalks, roads, etc.). This information was then overlaid onto maps of the sewer system in order to prioritize close circuit television (CCTV) surveys. Areas that fell within relative high risk areas (e.g., areas with extensive building damage) were surveyed first. This assessment revealed that approximately 16% of the inspected sewers required emergency repair, 49% sustained some damage and the remaining 35% sustained no damage.

Similarly, the Los Angeles Department of Transportation used a GIS-based Automated Traffic Surveillance and Control system (ATSAC), installed several months prior to the earthquake to monitor traffic flow, to plan detour routes and implement signal timing strategies in the Santa Monica I-10 Freeway corridor. The ATSAC system is also used to control the message signs along the freeways and, after the earthquake, these signs were used to advise motorists of the various detour routes and traffic conditions (LADOT, 1994).

AERIAL AND SATELLITE-BASED MONITORING SYSTEMS

Images, measurements and other data obtained from the vantage point of high altitude aircraft and satellites has recently entered the technological arsenal of emergency managers. These technologies have been employed to monitor geological changes and as a source of damage assessment during and after major emergencies including fires, bombings and flooding. These technologies including aerial photography and satellite imagery have recently been proposed for assessing the effects of large earthquakes (Crippen, 1992; Crippen and Blom, 1993). Using SPOT satellite images acquired approximately one month after the 1992 Landers earthquake, the Jet Propulsion Laboratory in Pasadena was able to capture spatial details of terrain movements along fault breaks associated with the earthquake that were virtually undetectable by any other means.

The Japanese have used aerial photographs to identify areas of significant ground failure after several large earthquakes. By comparing pre- and post-earthquake aerial photos of liquefaction affected areas, they have successfully mapped areas of ground movement including both magnitude and overall displacement. Figure 14.1 shows aerial photographs of the Kawagishi-cho and Hakusan area before, and four hours after, the 1964 Niigata, Japan earthquake. Evident in these photographs is significant planar distortion of buildings after the event. Also clearly visible is the change in curvature of the shoreline after the earthquake.

Another example of aerial reconnaissance after an earthquake is shown in Figure 14.2. This photograph shows earthquake damage detected after the 1971 San Fernando earthquake. Explosion craters resulting from gas leakage along Glen Oaks Boulevard in Sylmar can be clearly detected. Also visible in the center of this photo are ground cracks that resulted from extensive ground shaking during the earthquake. In this particular area, over 100 pipeline breaks were recorded in water and natural gas pipelines (Eguchi, 1982). It has been demonstrated historically that extensive ground distortion generally leads to significant damage to underground lifeline components (Eguchi, 1991).

Until recently, it was not clear how satellite data could be used effectively to identify areas of earthquake impact. In general, the data that are publicly available are limited by the level of resolution possible. For example, the SPOT image pixel size used to detect ground movement during the 1992 Landers earthquake (See Figure 14.3) is 10 meters (Crippen and Blom, 1992), which is larger than many of the displacements observed during the event. However, through the use of image matching by correlation analysis, JPL has been able to computationally detect displacement vectors revealing horizontal ground strain patterns. The most effective means of using this data in a post-earthquake situation is in identifying large areas of tectonic ground movement. Data resulting from satellite imaging may suggest areas for detailed aerial reconnaissance. A technique derived by Gabriel and others (1989) has been used to map small elevation changes over large areas using differential radar interferometry. This technique could be used to supplement satellite imaging (which only estimates horizontal changes) to identify areas of significant tectonic changes or perhaps ground failure settlement.

Global Positioning Satellite (GPS) systems were employed after the Northridge earthquake to assess the extent of crustal deformation and displacement. Based on measurements taken before and after the earthquake, the extent and nature of co-seismic displacements were determined. These data indicated that measurable displacements were produced over a 4,000 square kilometer area including much of the San Fernando Valley and adjacent mountainous areas (Hudnut et al, publication pending). Although these measurements were not available immediately following the earthquake, they could be made within sufficient time in future earthquakes for lifeline organizations to make response decisions based on the probable location and magnitude of displacements.

DISCUSSION AND CONCLUSIONS

Lifeline operators, both public and private, have been receptive and, in many cases enthusiastic, users of new emergency management technologies. The application of these technologies in the Northridge earthquake was, for many organizations, the first real test of their actual or potential utility in a major regional crisis. In some cases, existing systems proved to be successful and of critical importance, in others, performance was disappointing but important lessons were learned and vulnerabilities exposed. In addition, many new applications were discovered and the potential integration of these technologies moved from the conceptual to actual pilot testing. In this final section, we will identify areas in which the further application or integration of technologies could improve lifeline performance.

Expanded application of real-time earthquake monitoring holds considerable promise in improving performance during and hastening recovery after an earthquake for lifeline organizations. Most of the participants in the Caltech USGS Broadcast of Earthquakes are utilities and rail transportation providers. While most CUBE users, including lifeline organizations, have used the receipt of real-time source data for determining response needs, some have worked toward linkage of real-time notification with automated shutdown or other procedures which would ensure continued service after a major earthquake. Telecommunications companies, for example, have developed methods of rerouting network traffic away from areas likely to have sustained earthquake damage based on real-time broadcasts. Lifeline organizations have also been advocates of expanded real-time information including strong ground motion, estimates of displacement and early warning systems.

Earthquake loss estimation studies and scenarios such as those developed by the California Division of Mines and Geology have been used by lifeline organizations for planning and hazard mitigation purposes. Most, if not all, of these planning scenarios have included relevant vulnerability information which specifically address transportation and utility lifelines. Although new rapid loss estimation techniques which are linked to real-time earthquake monitoring systems have been developed for state and federal agencies, lifeline organizations, both in the United States and Japan, have expressed considerable interest in these new systems which provide a range of geotechnical, economic (e.g. damage, dollar loss) and population impact (e.g. casualties, displaced persons) data within minutes of the occurrence of a major earthquake.

In our earlier discussion of the application of GIS technology, we cited two examples of the use of GIS systems by lifeline operators in the Northridge earthquake. The experience of the two Los Angeles city agencies suggests that GIS-based systems in lifeline organizations are more fully integrated with operational processes than is the case in other organizations. Recall that the Department of Transportation used their GIS-based system to assess traffic conditions, implement control mechanisms and provide public information in one operation. This level of system integration suggests that lifeline organizations are likely to provide key innovations in the application of GIS technology to the management of emergencies.

Aerial surveillance and satellite imagery could be potentially beneficial sources of rapid information on above ground lifeline systems. Although high altitude aerial reconnaissance was attempted in the Northridge earthquake, neither the mission nor the results of this effort are known and to date have not appeared in after action assessments or other media known to the authors. The value of aerial photography and remote imaging are questionable for underground lifelines. The potential benefits of GPS data for pipelines and other underground systems, however, is quite significant.

The convergence of these new technologies after the Northridge earthquake mark an important development for emergency management in general, and for the performance of lifelines in particular. Of particular significance is the integration of real-time monitoring of seismic activity, loss estimation methodologies and GIS systems. The potential benefit lies in the ability of lifeline operators to obtain early warning of potentially damaging ground motion and implement procedures to lessen impacts and to respond rapidly to an earthquake, based on both empirical information and model estimates.

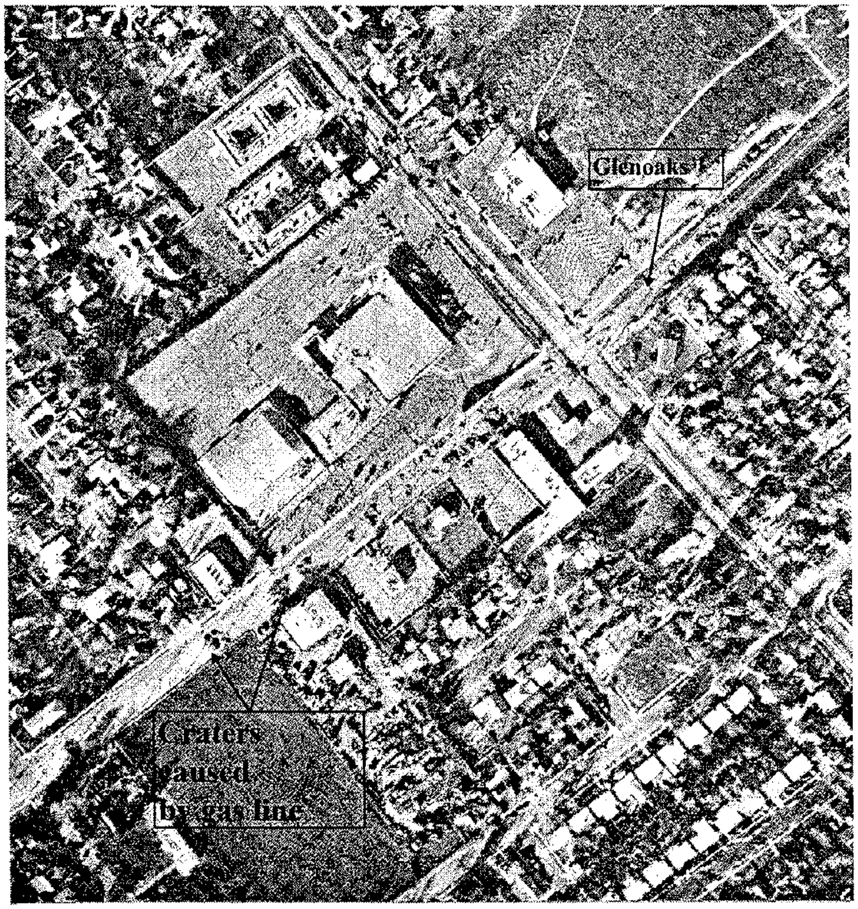

Figure 1. Cratering caused by gas leakage and explosions along Glenoaks Blvd. After the 1971 San Fernando Earthquake

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RESPONSE OF REINFORCED CONCRETE BUILDING WITH SOFT FIRST STORY SUBJECTED TO 1995 KOBE EARTHQUAKE

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ABSTRACT

A very new RC building with soft first story was collapsed by the 1995 Kobe earthquake. Nonlinear dynamic response analysis, where strength deterioration was considered in representing member nonlinearity, was conducted to simulate how the building behaved and eventually collapsed during the earthquake. The analysis was found to well reproduce the observed response such as residual displacement, mechanism and damages to members. The analysis has also revealed the existence of nonstructural walls above the second floor level had a significant effect on the overall behavior of the building.

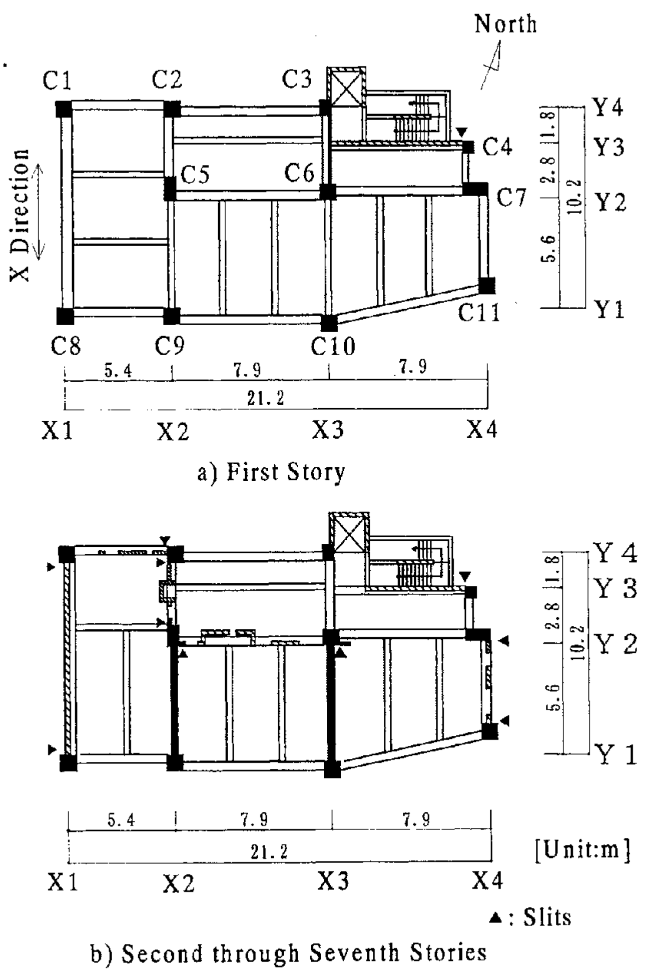


Fig. 1. Plan Views

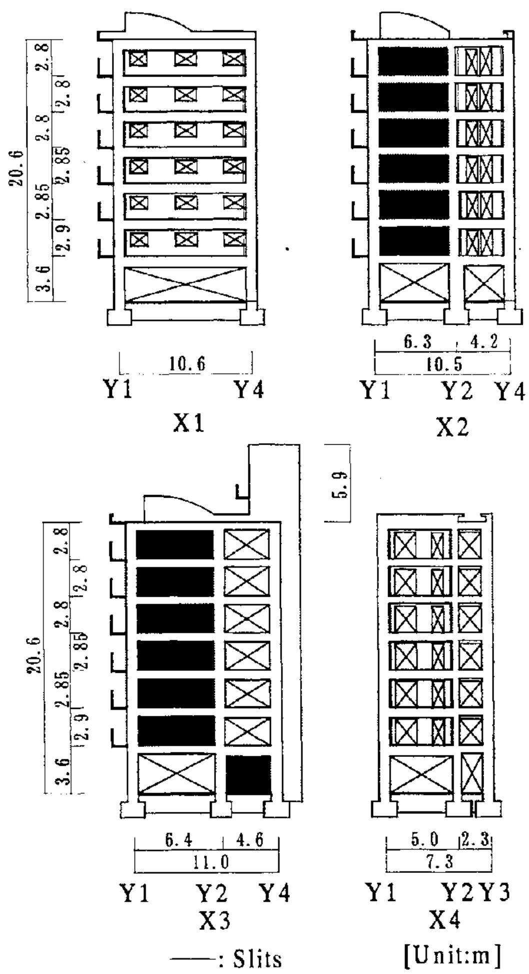


Fig. 2. Section Views (X Direction)

This paper was originally prepared for the presentation at Seventh International Conference on Computing in Civil and Building Engineering, 19-21 August 1997, Seoul, Korea.

INTRODUCTION

Many of RC buildings damaged by the 1995 Kobe earthquake were those constructed before 1981, when the Japanese building code provisions were extensively revised. However, there were found some RC buildings suffering severe damages among those constructed after 1981, and most of them were buildings with soft first story. In this paper, an example of the damaged soft-first-story buildings is taken up, and inelastic dynamic response analysis is conducted to simulate the building behavior during the earthquake. The building encountered the earthquake only a month later than the completion and collapsed.

Strength deterioration in member hysteresis is usually not considered in the nonlinear dynamic response analysis because of the problem in numerical solution. However, it is considered in this analysis in order to reproduce the process of building collapse as realistically as possible.

OUTLINE OF BUILDING AND DAMAGE

Building

The building was located in Nada Ward of Kobe City, which was one of the most severely affected areas. The building was RC seven story apartment houses without basement. Figures 1 and 2 show plan views and section views of the north-south (X) direction, respectively. The X direction, for which major damages occurred, was a frame-wall structure with structural walls in Frame X3 at the first story and in Frames X2 and X3 at the second through seventh stories. There were also nonstructural walls in Frame X1 at the second through seventh stories, which was designed to be separated from the edge columns by slits. Total weight of the building was 2240tf.

Figure 3 shows sections of the first story columns. D25 bar (deformed bar with 25mm diameter) was used as main bars, and D16 bar was used as hoop with 100mm spacing. Two ties of D16 bar were provided for C5 for the X direction and for all columns for the Y direction. Details of a slit are shown in Fig. 4.

Name	C1	C4	C5	C8
Size	850×850	600×600	600×1200	850×850
X Dir.	9 / 9	7 / 7	7 / 7	8 / 8
Main Bar	32-D25	24-D25	24-D25	28-D25
Hoop	⊟-D16@100	⊟-D16@100	⊞-D16@100	⊟-D16@100

[Unit:mm]

Fig. 3. Sections of First Story Columns

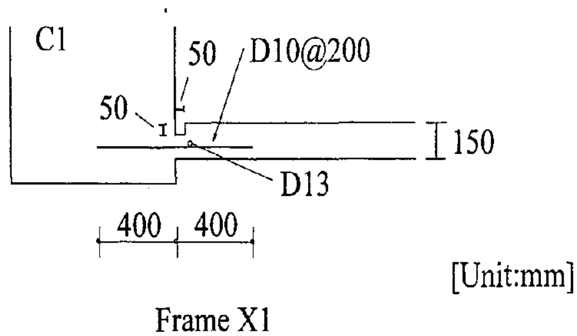


Fig. 4. Details of Slit

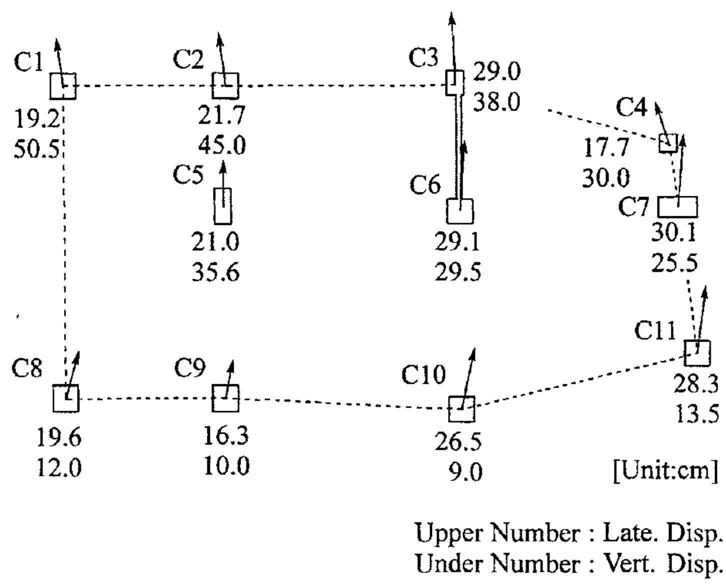
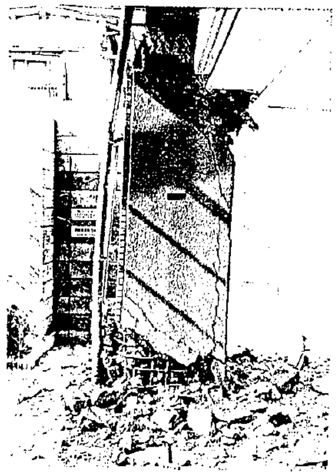
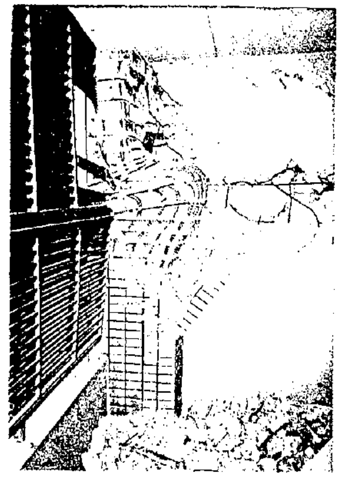


Fig. 5. Residual Displacement of First Story Columns

a) C1 (East Face)



b) C8 (East Face)



c) First Story Wall and C3 (West Face)

Photo 1. Typical Damages

The slit, although called so, did not perfectly separate the nonstructural wall from the edge column in reality. The slit thickness (50mm) was only one third of the wall thickness (150mm).

Damage

Damages were observed to concentrate upon the first story of the X direction, and those to the other stories of this direction and to the Y direction remained slight, indicating typical first story mechanism formed for the X direction. Figure 5 shows lateral and vertical residual displacement of the first story columns. The arrow in the figure indicates amplitude and direction of lateral residual displacement. Lateral displacement was extremely large, ranging from 16.3cm to 30.1cm, 23cm on average. All columns were observed to move nearly to the north: clear evidence of torsion was not found. Vertical displacement (column shortening) was much larger for the north columns (C1 through C4) with 41cm on average than for the south columns (C8 through C11) with 11cm on average.

Photo 1 shows typical damages to the first story members. C1 and C8 were selected as representatives of the north and south columns. The damage was severer for the north columns than for the south columns: C1 was completely destroyed at the top, while C8 escaped from complete collapse although severe flexure or flexure-shear failure occurred at both ends and pronounced bond-splitting cracks appeared along the member length. Such difference in damages to the north and south columns apparently corresponds to the fact that vertical displacement was larger for the former than for the latter. All columns are believed to have failed after experiencing flexure yielding. The first story wall in Frame X3 completely collapsed along with the edge columns at the midheight due to sliding shear failure.

METHOD OF NONLINEAR ANALYSIS

For the X direction, inelastic dynamic response analysis was performed. The analysis methods are as follows.

1) The building was represented by a two-dimensional plane frame consisting of Frames X1 through X4 arranged in a line. The building was assumed to be fixed at the base.

2) Columns and beams were represented as a line member and a rigid zone was considered for beam-column joints. Walls were represented as a deep column at wall center line with wall properties and a rigid beam at each floor level.

3) Frame X1 above the second floor level was idealized in two ways as: walls (Model 1), where the slits were considered ineffective, and frames (Model 2), where the slits were considered effective.

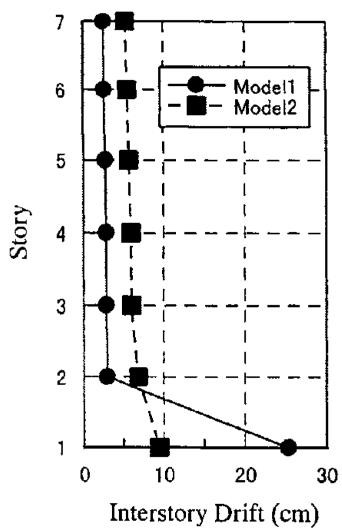


Fig. 6. Maximum Interstory Drift

4) For columns and beams, flexure nonlinearity was considered by two springs placed at both ends, the hysteresis of which was represented by Takeda Model (Takeda 1970). For walls, flexure and shear nonlinearity were considered separately: the former was represented by Takeda Model and the latter was considered by a spring, the hysteresis of which was represented by Origin-Oriented Model (Hisada 1962). Member flexure and shear properties were computed according to the equations available in Japan based on the specified material properties.
5) For the outer (north and south) columns, strength deterioration after yielding was considered for the direction where varying axial load was in compression. Strength deterioration was also considered for the wall shear hysteresis after shear strength. Negative stiffness defining the extent of strength deterioration was arbitrarily determined as -2% of the elastic stiffness. In this case, load would reduce to about 60% of the shear strength at 2% shear strain for the wall shear hysteresis (Fig. 9) and reduce to 80% of the yield strength at 2% rotation for the column flexure hysteresis (Fig. 10).
6) The building was subjected to the NS and UD components of the ground motions recorded at Kobe Ocean Meteorological Observatory, Japan Meteorological Agency, the maximum acceleration of which were 818gal and 332gal respectively. Damping was assumed to be of a viscous type and proportionate to the instantaneous stiffness. A damping factor was assumed 3% with respect to the elastic fundamental periods (0.393s. for Model 1 and 0.435s. for Model 2).

ANALYSIS RESULTS

Maximum interstory drift along the building height are plotted in Fig. 6 for Models 1 and 2. Almost all displacement concentrated on the first story for Model 1 with first story drift (FSD) of 25cm (drift angle of 7.9%), while such extreme displacement concentration did not occur for Model 2 with FSD of 10cm (drift

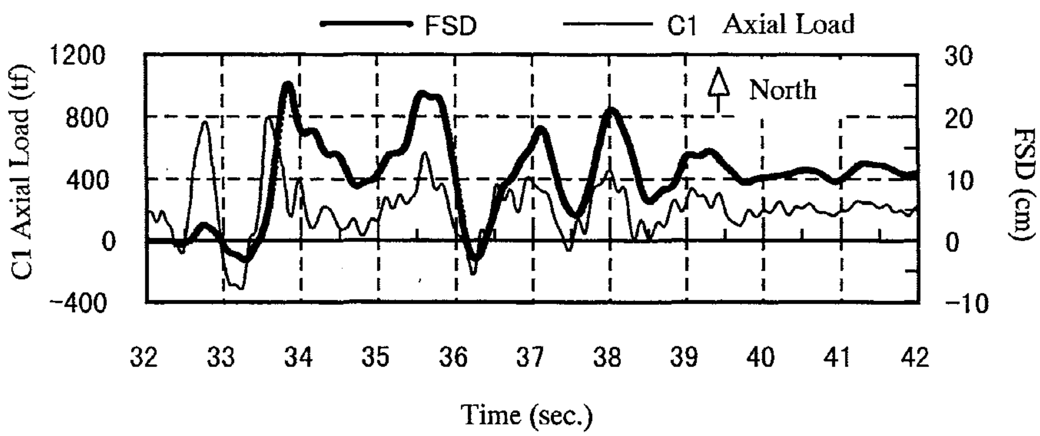


Fig. 7. Time Histories of FSD and C1 Axial Load

angle of 3.2%). Comparison of the analysis results with the observations clearly indicates Model 1 well reproduced the observed results while Model 2 did not, and that if the slits had been effective, the collapse of the first story might possibly have been avoided. Problems on nonstructural walls are often discussed from a view point of their failure or their effect on adjoining structural members. However, it should be noted there are cases like this building, where the existence of nonstructural walls may govern the overall behavior of buildings such as mechanism. Model 1 alone is considered in the subsequent discussions.

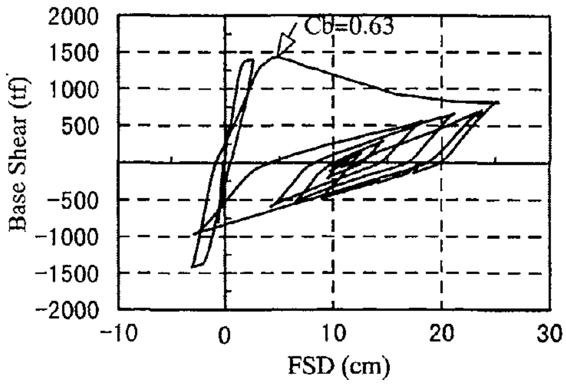


Fig. 8. FSD - Base Shear Relations

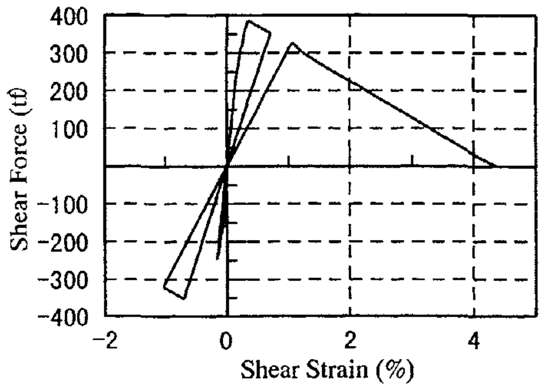


Fig. 9. Shear Force - Shear Strain Relations
(First Story Wall)

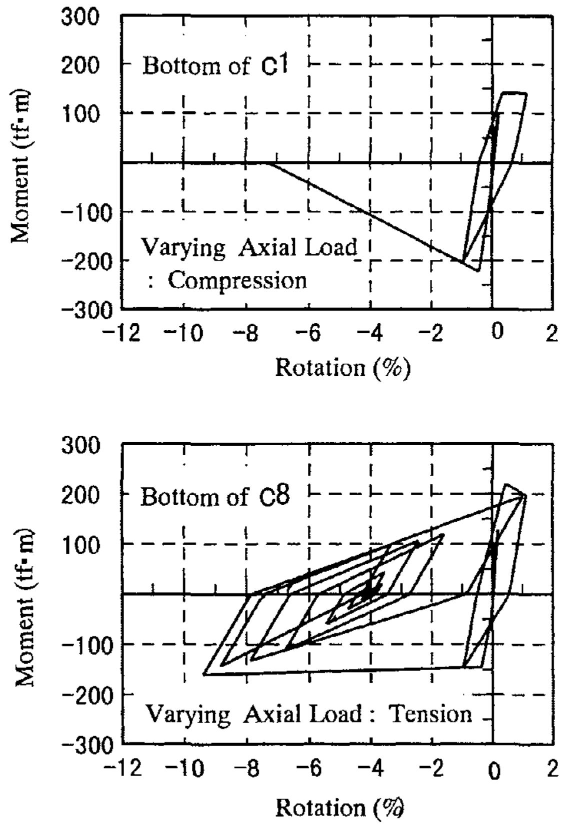


Fig. 10. Moment - Rotation Relations
(First Story Column)

Figures 7 and 8 show time histories of FSD and FSD-base shear relations. Shear force-shear strain relations of the first story wall and moment-rotation relations of the first story columns are also plotted in Figs. 9 and 10. In Fig. 7, after large amplitude vibration to the north at 33.8s., FSD was observed to shift to this direction, causing residual displacement of 12cm to the north at the end of the earthquake. Such result coincides with the observed residual displacement of 23 cm to the north, though the amplitude was a little different. The first story wall completely lost shear resistance, and C1 (north column), which was subjected to axial compression when displaced to the north, also completely collapsed. However, C8 (south column), which was subjected to axial tension when displaced to the north, did not lose flexure resistance much. These analysis results on member behavior also agrees with the observations. As shown in Fig. 8, base shear reduced to 60% of the maximum value due to the member strength deterioration.

Maximum axial compression of the first story columns including permanent load are listed in Table 1. The amplitudes of axial stress divided by specified concrete strength (Axial Stress Index) reached as much as 0.47-0.61 for C1, C8, C9 and C10, which were outer columns supporting walls. These amplitudes were

Table 1. Maximum Column Axial Compression (tf)

	C1	C2	C4	C5	C7	C8	C9	C10	C11
Maximum Compression	800 (0.53)*	547 (0.36)	406 (0.54)	680 (0.45)	298 (0.17)	716 (0.47)	921 (0.61)	827 (0.55)	394 (0.26)
Permanent Load Only	184 (0.12)	181 (0.12)	89 (0.12)	222 (0.15)	191 (0.11)	207 (0.14)	267 (0.18)	285 (0.19)	175 (0.12)

* Axial Stress Index

much greater than those due to permanent load only. The time history of C1 axial load is shown in Fig. 7. C1 was computed to have collapsed at 33.7s., when this column underwent axial compression of 800tf (Axial Stress Index of 0.53) and displacement of near 20cm. Note although almost same axial compression occurred at 32.7s., displacement at this time was very small. The north columns including C1 are understood to have collapsed at around 33.7s. due to large lateral displacement combined with high axial compression.

CONCLUSION

The nonlinear dynamic response analysis of the building with soft first story collapsed by the 1995 Kobe earthquake was conducted. The major findings from the study are as follows:

1) The observed response of the building, such as residual displacement, mechanism and damages to members were well simulated by the analysis using the recorded ground-motions. The first story columns collapsed due to large lateral displacement combined with high axial compression.

2) Nonstructural walls occasionally affect overall behavior of buildings. For this building, since the slits provided for the nonstructural walls above the second floor level were ineffective, these nonstructural walls behaved as structural walls , causing the first story mechanism. If the slit had been effective , the collapse of the first story might have been avoided.

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HIGH PERFORMANCE AND SMART CONSTRUCTION MATERIALS

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ABSTRACT

The last two decades have been marked by significant improvements of existing construction materials and the availability and application of new materials. Improvements of existing materials have come in the form of durability, strength, constructability, and toughness. The new materials are being used not only as superior substitutes of technologically older materials but in new applications such as repair and rehabilitation of existing structures. This paper gives an overview of recent advances made with construction materials. In addition, some test methods required for advanced characterization of these materials are described and recommendations to help expedite the process for application and wide acceptance of new materials is presented.

INTRODUCTION

Advances in materials available for construction have grown exponentially since the beginning of history. Few, if any, of the recent advances are as landmark as the invention of portland cement in 1824 or the advent of steel in the early 20th century; however, the selection of application specific materials is growing faster than ever before. While no single new material in the last 20 years, or perhaps ever in the future, will revolutionize the construction industry the way the emergence of steel or reinforced concrete construction has, the combined impact of all recent advances has the possibility of advancing the construction industry more.

In order for a new material to be used as a construction product, it must have improved mechanical properties (strength, constructability, toughness, stiffness, etc.) with little to no additional cost, provide significant improvement in long term life cycle costs, or have an application which cannot be filled by existing materials. In order to gain wide scale use of a new material, the confidence of engineers must be won before they will specify a new product. Complete confidence can only be gained if the following are available:

- demonstration of the material performance in the form of a full scale application
- full engineering characteristics of the material
- engineering design requirements and methods for analysis
- measures for quality control
- construction personal who can properly implement the design
- and, of course, the material itself must be readily available.

While these requirements may seem straightforward to implement, they actually require considerable research, development, and promotion and take decades to implement. On a smaller scale, materials from a single source supplier who can demonstrate and guarantee material performance, aid in design standards, and supply the expertise and tools during construction can be implemented faster and be developed cheaper.

New materials already provide cost effective solutions for the rehabilitation and retrofitting of our aging infrastructure. These materials, in general, fall into the category of small scale implementation for specialty applications. On the other hand, improvements of materials already widely utilized enjoy ready acceptance by the construction industry, although, in some cases the mechanical properties are so radically changed, that the improved material has only slight resemblance to the original material and the codes must be altered to reflect the new materials.

SMART MATERIALS

Smart materials are defined as having properties of self-adaptability, memory, multiple functionality, or self-sensing[1]. While these materials are normally considered esoteric for application as a construction material, they are a rapidly growing group of technological innovation. Smart materials are finding applications as self-correcting and self-compensating components of mechanical systems. Current and proposed applications include valves, active/passive vibration control, deformation control and recovery, and sensors. Smart materials are often grouped with smart structures which couple sensing transducers with active controls. Smart materials on the other hand combine sensors and the active control integrally within the material, not as individual components.

Shape memory alloys (SMAs) are one such smart material. SMAs can be deformed under low or room temperatures and be restored to their original shape by heating them above a transition temperature. There are even two way SMAs which can have two stored shapes in their memory, one of which will be recalled by heating the material above a critical temperature and a second which can be recalled by lowering the temperature below a second critical temperature[2]. By recalling a memory by changing the SMA's temperature, thermal energy can be converted into mechanical work. SMAs can be applied to structures to control low frequency vibrations by using their thermally induced hysteretic stress-strain behavior.

Piezoelectric materials strain under the influence of electric fields and produce electric fields when under strain[1]. With these properties, piezoelectrics can be used as sensors or actuators. Piezoelectric materials are already widely in use as transducers and have seen some limited applications as control mechanisms in the form of active engine mounts, shock absorbers, and valve controls. Closed-loop digital control of cantilever plates has been proposed by Tani et al[3] demonstrating the potential application to full scale structures.

Magnetostrictives are similar to piezoelectrics, however, they instead strain under the influence of a magnetic field and generate magnetic fields proportional to applied strain rates[1]. Magnetostrictives can be applied as either a sensing device or as an actuator. Due to power requirements, they are not as efficient as piezoelectric materials.

The smart materials described in this section have only been applied on extremely small scales, not yet suitable for full structural control; however, they do have the ability to be used as sensors and controllers. As development of these materials continue, they may emerge as full structural control components in the not too distant future. In applying these materials, entirely new design and implementation schemes will have to be derived, and the potential of the materials to be utilized as combined sensors and controllers is yet to be fully realized.

HIGH PERFORMANCE MATERIALS

In this section, a number of relatively new advances in materials engineering will be described. This section concentrates on advances made with concrete and cementitious materials and on fiber reinforced composites which was originally developed and applied to the aerospace industry but has found its way into the construction industry and its potential for impact is greater than any other recent advances in materials engineering. Not included in this section is a discussion of advances with geosynthetics or steel (for example, stainless steel for concrete reinforcement has recently been defined under ASTM A955 and is being used in test bridge decks in Michigan, Ontario, and Oregon).

Composite Materials

In this sub-section, composites are defined as being composed of a polymer, metal, or ceramic matrix strengthened (or stiffened) with continuous or chopped fibers such as E-glass, S-glass, carbon, or Aramid, to name a few. Fiber reinforced composites discussed in this sub-section should not be confused with reinforced concretes whether it's ordinary reinforced concrete, fiber reinforced concrete, or fiber reinforced cement based materials which will be discussed in the following sub-section.

In the last two decades, fiber-reinforced composites have advanced rapidly. Initially, their applications concentrated on the aerospace industry where their lightness, strength, and stiffness were critical and outweighed the high expense. In the last 15 years, composites have been used in the automotive industry, electronics, bicycles, tennis rackets, and skis. More recently, the construction industry has been examining applications of these advanced composites.

Numerous examples of pedestrian footbridges constructed of composites can be cited: the Westminister Cathedral glass reinforced polyester in London; the Virginia Pedestrian Bridge with glass fiber reinforced truss girders and a GFRP cover plate; the Chongqing Pedestrian Cable-Stayed Bridge in China; in La Jolla, California, a graceful cable stayed pedestrian bridge; in Aberfeldy, Scotland a 113 meter long all composite cable stayed footbridge[4]. Recently, a 100 meter long by 56 meter wide E-glass/polyester composite bridge section was recently built by Lockheed Martin Research Laboratories[5]. As can be seen, the examples presented are all non-critical structures, but this is the proper starting point for the application of advanced materials; these non-critical structures provide vehicles from which to learn, gain hands on experience, and provide living examples which give engineers the confidence to try these applications.

For repair and rehabilitation of existing structures, composites are proving their worth. Fiber reinforced polymer sheets have been used extensively in Japan and Germany; however, the technology has only recently reached the United States. External bonding of these sheets can be used to repair and rehabilitate structures in cases where a structure was improperly designed or constructed, potential for earthquake damage has been reassessed, additional load carrying capacity of the member is necessary, or the member has lost strength or stiffness due to deterioration[6,7]. A carbon fiber composite jacketing system has been developed and used for seismic retrofit of bridge columns in California; this system wraps the column in place forming to the column. These jackets have been experimentally verified to be as effective as their steel jacket counterpart.

Concrete cover is often insufficient to protect steel reinforcing and prestressing from corrosion produced by ingress of chlorides from salt water and deicing salts. Fiber reinforced plastic (FRP) rebars and tendons have been proposed and used in place of rebar and prestressing wire since they are not susceptible to corrosion as conventional reinforcement is[8,9]. There are still significant issues which need contending with. First is the cost issue: glass fiber reinforced plastics (GFRP) are the least expensive and have potential to function as reinforcing bars except they have low modulus of elasticity, poor bond with the concrete, and lack ductility. Structures or elements which do not take large tensile loads such as bridge decks and pavements can still be effectively designed using GFRP bars. One additional issue is the existence of alkali silica reactivity (ASR) problem between the glass fibers in the GFRP and the cement resulting in degradation of the GFRP. The ASR problem must be addressed by adequate protection of the glass fibers by the epoxy resin or by using ASR resistant glass, otherwise glass cannot be used in the FRPs, leaving only more expensive fiber types such as carbon and Aramid. Prestressing strands are an excellent use of FRPs as they can achieve the ultra high tensile stresses required and the cost is not significantly more than that of high strength steel prestressing strand. The issue of poor bond between FRPs and concrete still exists; however, through the development of special anchors, this problem can be solved[10].

Fiber reinforced plastics are also being used to enhance strength, stiffness, and ductility of glue laminated wood beams[11]. Contrary to reinforced concrete, composite layer is placed on the compression side of the composite beam, which is the weakness for glulams.

The composites described in this sub-section show the potential for large market application of composites in the civil infrastructure definitely exists. These composites are still more expensive to implement as complete structures due to the material costs, but in some cases, the lower construction costs and applications which cannot be filled by conventional construction materials make composites viable construction materials, and these materials will only become relatively cheaper in the future. There are still some major hurdles which need to be overcome before they are commonly specified. Issues which still need to be addresses and fundamental research areas include:

- repairability
- fire protection
- environmental impact of production and disposal
- long term durability.

Advances In Portland Cement-Based Materials

More than a century and a half after the invention of portland cement, there is still no construction material which can match the cost performance of concrete for ordinary construction purposes which is why it still remains the most widely used construction material over the world. The simple principle of having a material with the unique property of being castable into any shape or form and then have it self harden four to six hours later into a strong and durable material is remarkable. By modification of mix designs, the introduction of fiber reinforcement, the inventions of superplasticizers, shrinkage reducing admixtures, air-entrainment admixtures, and other mineral admixtures, concrete has the ability to be stronger, stiffer, more ductile, and more durable. Specially designed mixes can modify concrete to give it nearly any property desired. Some significant recent advances made with concrete and other cementitious materials which show promise for future applications will be described in this sub-section.

With the use of superplasticizer and careful proportioning of cement, water, fine and coarse aggregates, self-compacting concretes have been made and used[12]. Self-compacting concrete has low water to cement ratio and fairly high cement content and exhibit extremely good flowability without segregation.

The major drawbacks of concrete have always been its lack of strain capacity, low tensile strength and low fracture toughness. Fibers have been introduced into concrete in order to improve its ductility in compression, resistance to tensile cracking, and toughness. Reduced cracking also helps improve the concrete durability by reducing the penetration of water and deleterious chemicals. Small volume fractions of polypropylene fibers (0.2% to 0.5%) have been shown to improve resistance to shrinkage cracking[13]. Slurry infiltrated fiber concrete (SIFCON) provides extremely good strength (compression, tension, bending, shear), stiffness, and energy absorption under cyclic loading by using up to 12% steel fibers by volume[14]. Significant advances have been made in the characterization of fiber reinforced concrete; however, they have not been widely used in the construction industry. While an engineer may be able to specify a particular volume fraction of steel fibers, there is no guarantee that the desired engineering properties will be obtained, and without adequate quality control test methods, it is unlikely that they will be widely used.

Freeze thaw durability of concrete can be better controlled through the use of air entraining admixtures, and chloride attack and ASR problems can be controlled through the use of mineral admixtures such as silica fume (micro silica), metakaolin, fly ash, and granulated blast furnace slag. Being able to control material durability is critical if we are to design structures to have long life span. Without durability, the value of improved mechanical performance diminishes.

Higher concrete compressive strengths have long been the desire of engineers so more efficient reinforced concrete structures can be designed. Available strengths have been rising exponentially. Water reducers and subsequently superplasticizers were initially responsible for the strength gains; as the water demands of the concrete decreases with the use of water reducers, the matrix becomes denser and the bond between the matrix and aggregates is increased until the aggregates become the weak component in the concrete instead of the matrix and the interface between the aggregates and matrix. Use of silica fume in the concrete further increased the strength by densifying the matrix and making it stronger through pozzolonic reactivity. With the use of superplasticizers to reduce the water to cement ratio below 0.3 and silica fume, concrete with strengths as high as 140 MPa are available for use in construction. High strength concrete is often called high performance concrete since other properties such as porosity and abrasion resistance are greatly improve. Many examples of high strength concrete use in construction are available from columns in high rise buildings, bridge decks, off-shore oil platforms, and bridge piers.

On a higher performance scale, there are also other categories of high performance cementitious composites such as macro defect free (MDF) cement, densified small particle (DSP) concrete, and reactive powder concrete (RPC). MDF is a cement-polymer composite of water soluble polymer, cement and a small amount of water combined under high shear producing a material with flexural strengths in excess of 200 MPa[15]. DSPs are made by careful particle sizing, use of microsilica, and reduction of the water to cementitious ratio below 0.13. RPC is a variation of DSP with the exception that small (~10 mm) steel fibers are also incorporated into the mix. RPC is also often cured under pressure and heat treated. RPCs can achieve strengths ranging from 200 to 800 MPa depending on compaction and heat treatment[16]. A recent sample project used RPC in the construction of a footbridge in Canada[17], and it is also being used in the fabrication of pipe products[18].

Only a brief overview of the possibilities which can be obtained using cement-based materials has been presented. The variety of products which can be produced and the widely varying properties which can be engineered contrasts the ordinary concrete which comprises the majority of concrete applications. Before these specialty concretes can be property-selected by engineers, much work needs to be done to provide mechanisms by which the necessary properties can be assured.

ADVANCED TEST METHODS FOR MATERIAL CHARACTERIZATION

In order to derive engineering properties of new materials, new test methods must be created. New test methods also can provide better characterization of materials which leads to more efficient design. In addition, standardized tests for quality assurance will have to be implemented before materials can be readily implemented. Having advanced test methods available will also allow engineers to design more efficiently. For example, if a concrete with high compressive ductility is needed, then the engineer could specify a minimum stress-strain curve which can be obtained by addition of steel fibers. Without a standardized test method, the engineer will not have confidence that the required ductility will be supplied, subsequently the steel fiber reinforced concrete won't be specified.

Over the last two decades, closed loop test machines and digital data acquisition systems have become readily available. With these machines it is possible to obtain complete compressive and tensile stress-strain behavior in addition to fracture properties. Having access to complete mechanical performance allows for more efficient plasticity and fracture analysis of structures.

Redefinition of current standards may also be required. For example, with self-compacting concrete, standard slump tests are not adequate to assure that proper flowability and lack of segregation is obtained. Several test methods have been proposed to evaluate self-compactibility, and with time one or two will emerge as standards[12]. As a second example, as concrete has reached strengths in excess of 120 MPa, ASTM C39, the standard test for compressive strength, has been found to provide inconsistent results, so this standard needs modification.

ADOPTION OF NEW MATERIALS

Figure 1 gives a flow chart summarizing the steps necessary to go from a material with an application to actually have the material used in construction. The joining of a material with an application can come from either having a new, special requirement and adopting a new material for the application or creating a new material and applying it to an existing application. The mechanical and chemical properties must be characterized, and if necessary, new test methods must be created to characterize the critical properties. In order for a material to be widely used, assurances for quality control must be in place, real life example applications must be built, the building codes must be adapted to allow for the use of the new material, the engineer and contractor must be educated in the proper design and construction techniques, application, and finally the material must be readily available. If the application is a specialty one where it won't be for general specification by engineers, then a single company can supply the material (hence the quality assurance), expertise for design and construction, and application experience. Materials such as composites, which have the potential for a large share of the construction market, must take the longer path otherwise, the material simply won't live up to its potential.

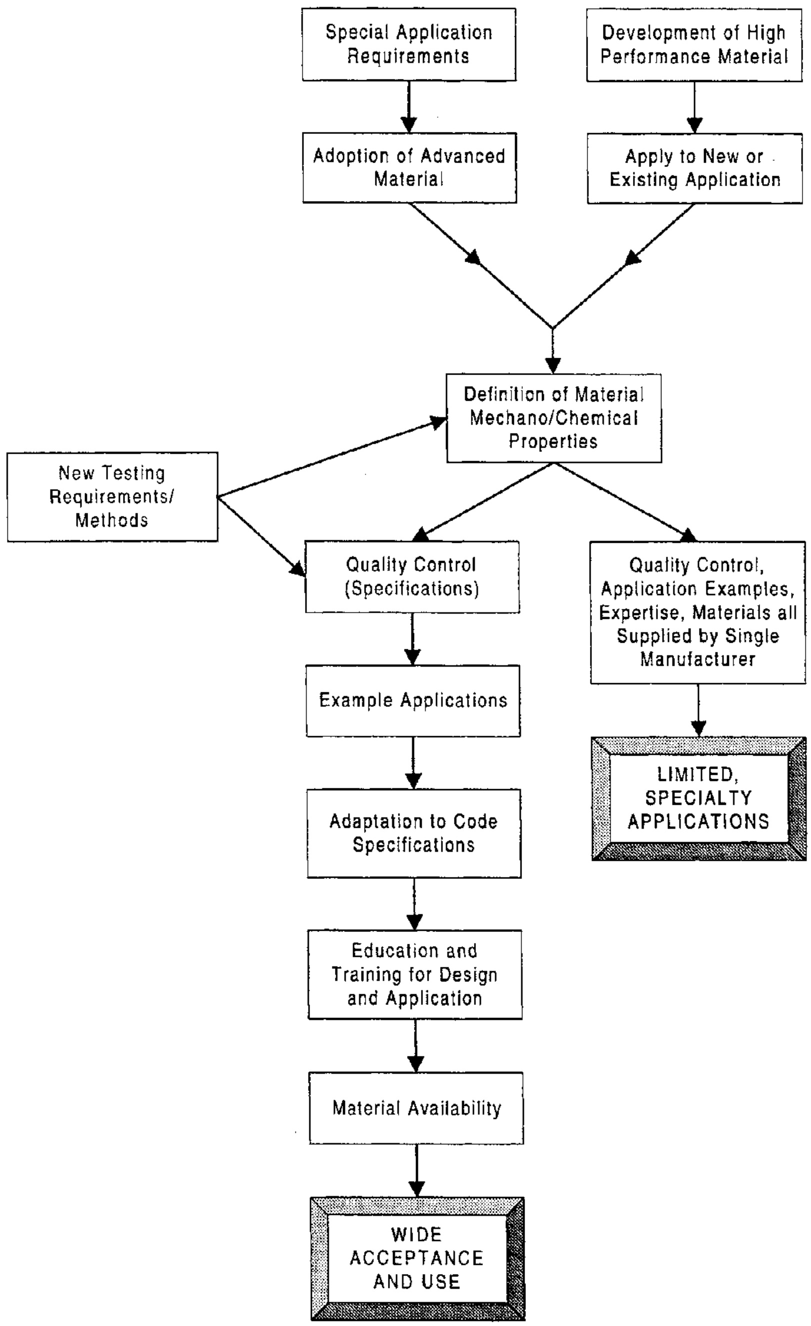

Figure 1. Flow chart of necessary steps to bring new materials from concept to application

The expensive and time consuming components of the longer path in Figure 1 are development of quality control standards and the production of real life sample applications. These components can be made less expensive if there is better cooperation and coordination between countries in their development; instead of several countries independently developing these components and reliving the same mistakes.

CONCLUSIONS

As engineers, we are entering an extremely exciting era with all the potential advances in materials. It also promises to be a very challenging era as application and acceptance of new materials is difficult to implement. Already, there are many high performance construction materials which have been developed and mechanically characterized; however, it seems they won't be applied any time in the near future because there are vital links missing in the chain which extends from material to acceptance and use of the material. Each new material must go through a process of matching material to application, characterization, quality control development, and real life sample application. And, of course, the material must be readily available from reliable suppliers, engineers must be educated how to design with the material, and the contractor must also be trained to proper install or build with the material. Mechanisms for the exchange of information of newly developed materials is currently in place; however, little exchange of hands on experience and development of quality control testing is practiced. By arranging exchanges of contractors and constructors, not only academicians, during new applications of innovative materials, the cycle of learning could be reduced. Finally, quality control methods should be shared between countries; this saves effort in two ways: test methods do not have to be developed multiple times and materials can be universally used and don't have to satisfy different requirements in different countries.

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THE DEVELOPMENT OF THE HIGH BENDING STRENGTH MORTAR AND THE RHEOLOGICAL TEST APPARATUS FOR HIGH PERFORMANCE AND SMART CONSTRUCTION MATERIALS

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ABSTRACT

In the recent Japanese construction industry,it has been urgentry necessary to automate the concrete construction works because of the lack of the labors,especially the carpenter and of their high wages. Therefore,the two construction methods are payed attention to,one of which is the precast concrete form works insterd of the wooden or metal form works. And another method is the high performance concrete which has the high fluidity and is easy to selfcompact to the forms. When the precast concrete forms are used, the works of removing forms become unnecessary,so the carpenter can be put to another works.And the precast concrete forms can be made in the factory,delivered to the sites, hanged and fixed to the designed position of the building by the crane.As the results,the factory automation and the site robot can be easily put practical use by the precast concrete forms. And being able to manufacture the thin and light weight precast concrete, the form works became very economical. Therefore, development of the high bending strength mortar or concrete has been expected in the construction industry.

On the other hand,the rheological properties of high fluid concrete has been considered to be very difficult to grasp numerically. We have proposed the J・shaped flow test and have been able to gain the rheological properties of high fluid conorete.

Part1 DEVELOPMENT OF THE HIGH BENDING STRENGTH MORTAR

1. Introduction

This paper is intended to study the new method to grow the high bending strength of cement past or mortar. Up to date,the hydraulic cement product is said to have low bending strength notwithstanding its high compressive strength. This paper clarify the new method to gain the high bending strength of cement mortar which has been mixed with silica-fume and cured in 60℃ moisture.

And these curing mèthods are studied in detail and the composition of the hydraulic cement product is studied in order to elucidate their high bending strength.

2. Manufacturing of mortar and its test pieces.

2.1 Material of mortar and its test pieces.

The mortar and its test pieces have been made of materials shown in table1,such as ordinary portland cement,fine aggregate. some superplasticizer and some silica-fume.

2.2 Mixing and Proportion of the mortars.

The Proportions of the mortars are shown in table 2 and these mortars are mixed with little water just enough to enable hydraulic material to hydrate,preferably the water/cement ratio is between 18 ~30%,in order to be less air content and to get a structure of high density and homogeneity.

The flow viscocity of the mortars are measured by the table flow test and the results of these test are shown in table 2.

2.3 Curing of the mortars.

The mortars are moulded for test pieces,4×4 ×16cm size,and the test pieces have been demoulded at 24 hours after moulding and then they have been cured for about 3 ~7 days,which is called two stages curing. The primary curing is in the moisture of the normal temperature about 20 ℃ and the secondary curing is in the water or moisture of 40~90℃ temperature,as shown in table 3.

2.4 The bending strength test and the compressive strength test

The bending strength test and the compressive strength tests have

Table 1 Materials

Cement	Portland Cement
Sand	Old riversand
Admix	Silica-fume Superplasticizer

Table 2 Proportions

No.	W/B (%)	S/B (%)	Si/B (%)	flow (mm)
1	20%	1.25	10%	140
2	25%	1.5	10%	160
3	27%	1.5	10%	160
4	30%	1.5	10%	160

been excuted using 4×4 ×16cm mortar specimens,according to the ASTM . The bending strength test specimens have been loaded at the center point and the splited specimens of the bending strength tests have been used for the compressive strength test.

Table 3 Curing Method

Curings	Primary curing	Secondary curing	After curing
Two stages curing	20~30℃ moisture	40~90℃ moisture or hot water	20~30℃ water
Normal curing	20℃ moisture	20℃ water	

3 . The bending strength of the mortars.

3.1 The bending strength of the mortars and the temperature of the secondary curing.

The relationship between the bending strength of the mortars with 10 % silica -fume and the temperature of the secondary curing are shown in fig.1. As the result,it has been found that the most suitable temperature of secondary curing is about 60℃.

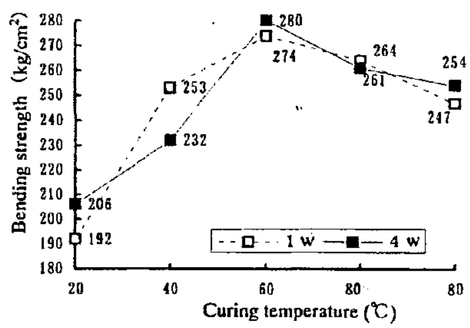


Fig.1 The relationship between the bending strength of the mortars with silica-fume and the temperature of the secondary curing

3.2 The bending strength of the mortars and the length of the period of the secondary curing.

The relationship between the bending strength of the mortars and the length of the period of the secondary curing are shown in fig.2 and the most suitable length of the period of the secondary curing have been found to be about 4 ~7 days.

3.3 The bending strength of the mortars and the length of the period of the primary curing.

The relationship between the bending strength of the mortars and the length of the periods of the primary curing are shown in fig.3 and it have been found that the most suitable length of the period of the primary curing is about 15 ~24 hours.

3.4 Effect of the quantity of silica-fume to the bending strength of the mortars.

Fig.4 shows the bending strength of mortars which are mixed with the several quantities of fine silica-fume and with the several water/cement ratios.In fig.4,the real line represent the bending strength of mortars with two stages curing (temperature of the secondary curing was 60℃) and the dotted line shows the bending strength of the mortar with standard curing of 20 ℃ temperature for 4 weeks. As the result,the bending strength of mortar with water/cement ratio of 20% and with two stages curing, have been found to become 220~280 kgf/cm². And the dynamic elasticity modulus have been found to be 4.9~5.5 kgf/cm².

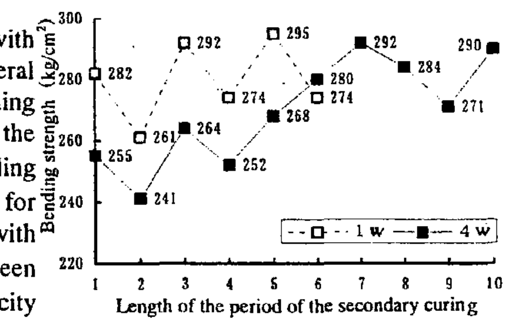


Fig.2 The relationship between the bending strength of the mortars and the length of the period of the secondary curing

4 . Another properties of the mortars

4.2 The aborbtion of the mortars

The absorbtion of mortars with two stages curing is remarkably small as compared with that of the normal curing. Fig.5 shows the result of water absorption test,in which the test pieces of mortar (4 ×4 ×16cm,being dried at 80℃ after 4 weeks curing)were laid in the water by 2 cm depth for 24 hours and were measured of their weight at every 5 minute. The result of the absorption of mortar with two stage curing have been found to be about 0.2% which is one third of that of mortar with normal curing.

4.2 The shrinkage of the mortars

Fig.6 shows the results of the shrinkage test of the high bending strength mortar with two stages curing. And the shrinkage ratio of these mortars were less than 0.02%, which were one fourth of that of the mortars with normal curing .

4.3 The impact resistance

Fig.7 shows the results of the impact resistance test,in which 1 kg steel weight is dropped from some height to the mortar specimen of 4×16×13.2cm size. And the heights of the weight when the mortars have been split,are measured to evaluate the impact resistance.As the results ,the impact resistance of the mortar with two stages curing has been found to be 1.7 times higher than the normal cured mortar.

5 . The composition of the mortar with stage curing.

. The composition of the mortar with two stages curing containing silica-fume is studied by analyzing the trimethylsilyl silicate derivation with gel-permeation chromatography(GPC) and by measuring the relative aboundance of the molecular weight of the hydration of the mortar.

Fig.8 is the result of analyzation and they shows that there are many trimers or polymers in the composition of mortar of two stages curing and many dimers or monomers in the composition of the mortar of normal curing.These results show that the silica-fume have been polymerlyzed in the mortar with two stages curing.They are the kind of silica polymer and they bind the hydrated calcium silicate each other,that is siloxane binding as shown in fig.9. Furthermore, they bind with the silica in the surface of the sand in mortar untill they make some kind of the briges of silica polymer.

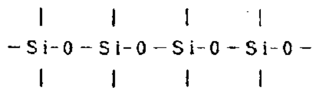


Fig.9 The model of the silica polymer,siloxane binding

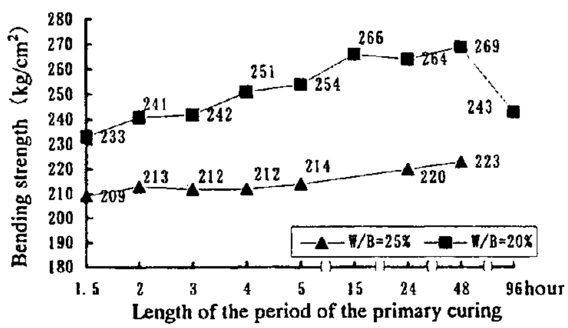


Fig.3 The relationship between the bending strength of the mortars and the length of the period of the primary curing

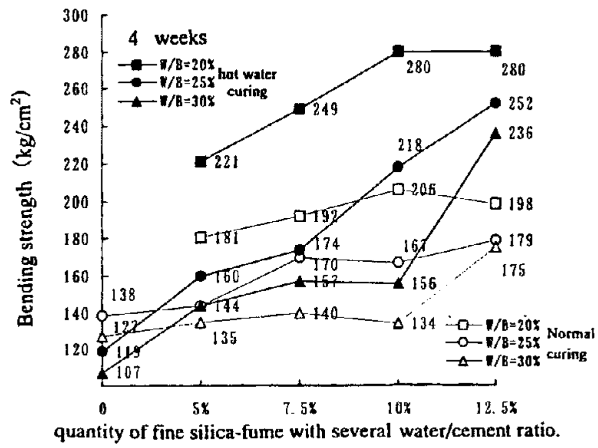


Fig.4 The relationship between the bending strength of the mortars and quantity of fine silica-fume with several water/cement ratio.

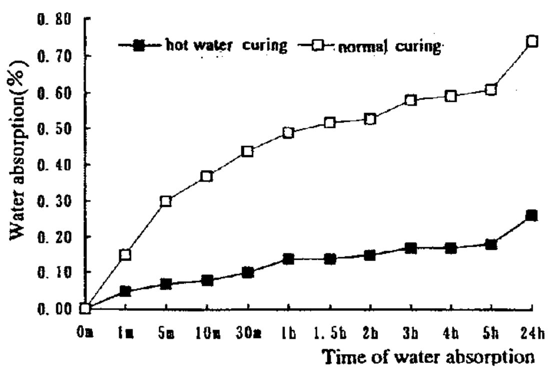


Fig.5 The test results of the water absorption of mortar

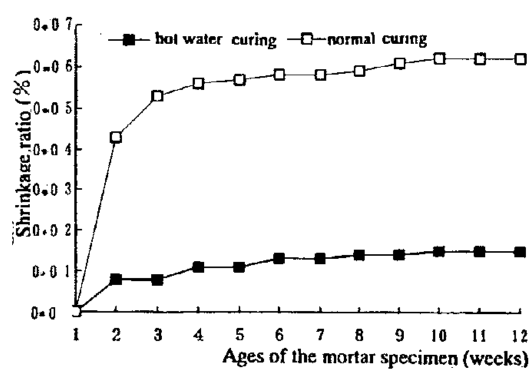


Fig.6 The results of the shrinkage test

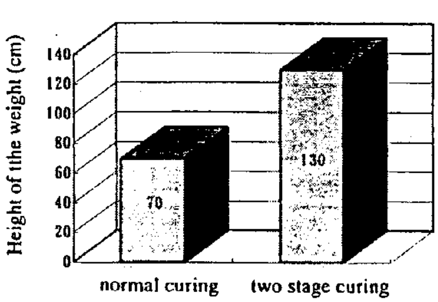


Fig.7 The results of the impact resistance test

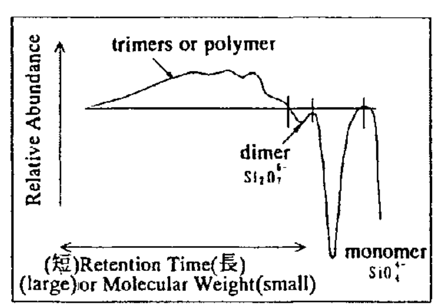


Fig.8 The results of analyzation of the composition of the mortar

Part2 TESTING METHOD OF THE RHEOLOGICAL CHARACTERISTIC OF HIGH FLUID CONCRETE

1. Introduction

In recent years, the utilization of concrete with high fluid and high consistency has been progressed for the purpose of labor saving and maintaining a high quality of concrete. However, it is difficult to grasp the basic numerical properties of high fluid concrete and to know the real workability of these concrete.

For the purpose of obtaining the rheological characteristics in order to analyze the fluidity of the high fluid concrete, we have investigated many testing methods and apparatuses.

Although, the testing method of the consistency and fluidity of concrete by the slump flow test, L-shaped flow test and cylinder penetration test (Torum test) have been used from the past, it has been difficult to measure and obtain the reliable values of the flow viscosity coefficient and shear stress yielding value. There are precise methods such as the rotary viscometer[1)2)3)] , but these methods are very complex and need the large equipment, so a simpler method is expected.

In this paper, we proposed the new J-shaped flow test, which was evaluated with many kinds of high fluid concrete and compared the results with those of the conventional slump and slump flow test, and rotary viscometer for concrete.

2.Testing Methods

In the experiments, several testing methods of high fluid concrete have been used, such as the conventional slump and slump flow test. To examine the segregation of concrete, we compared the weight volume of the aggregate at the inner part of the concrete with the weight volume of the aggregate at the outer part of the concrete after the slump flow test (In/Out aggregate weight ratio test). And then, to elucidate the rheological property of concrete, the concrete rotary viscometer test and the J-shaped flow test, which we proposed here, were carried out. Details of each testing method are described below.

The J-shaped flow test is the test method to obtain the data measuring the flow velocity of concrete which is caused by deferential pressure of concrete after packing concrete into the J-shaped tube shown in Fig.11. In our experiments, the tube was made of vinyl chloride. Its inner diameter was 10.9cm as large as enough to prevent the locking of coarse aggregate and the length of the tube from the upper opening to the lower opening is 75cm.

The concept of this test is shown in Fig.11. Testing was begun by packing concrete into the tube, and preventing the concrete from flowing out of the lower opening (here after called flow gate), the flow gate is sealed with a 6kg weight and concrete is packed to fill the upper side of the tube.

Then being removed the seal from the flow gate and concrete began to flow out of the gate. The time of concrete falling down from the upper side of the tube by every 5cm height was measured. After stopping of flow, the height of the concrete face in the tube from the flow gate are measured.

3. Analysis of Testing Methods

When driving pressure ΔP acts on the concrete in a tube and concrete flows laminarly, it can be modeled as shown in Fig.12. Then, Eq.(1) can be derived.

$$\pi r^2 \Delta P = 2\pi r l P_t, \quad \therefore P_t = \Delta P r / 2l$$

Assume $r = R$, $p = \Delta P R / 2l$ (1)

Because the shear stress P_t is proportioned to the shear velocity,

$$P_t = \lambda \cdot dV / dr \text{ , or } P_t = \Delta P r / 2l$$

$$\therefore dV / dr = \Delta P r / 2l / \lambda \text{ and } \therefore dV / dr = V$$

The distribution of the velocity in Fig.12 is parabolic. Because the flow Q is equal to the integration of distribution of velocity Eq.(2) can be derived using driving pressure ΔP. This equation is Hagen-Poiseuille's law:

$$Q = \pi R^4 \Delta P \cdot t \ / \ 8 l \lambda \quad (2)$$

with assuming $v = 4Q / \pi R^3 t$ (3)

Substituting Eq.(1) into Eq.(2)

$$p = V\lambda \quad (4)$$

and V is equivalent to shear velocity (1/sec), p is equivalent to the shear stress (g/cm^2) and λ is equivalent to the viscosity coefficient (g·sec/cm^2).

When the concrete face is at $(h_0 + h_1)/2$ height in the tube with diameter R, the driving pressure ΔP can be calculated from potential energy, so the shear stress p is derived from Eq.(5).

$$p = \frac{\Delta P R}{2l} = \frac{\rho \cdot (h_0 + h_1)/2}{L + (h_0 + h_1)/2} \cdot \frac{R}{2} \quad (5)$$

$$V = \frac{4Q_i}{\pi R^3 t_i} = \frac{4\pi R^2 (h_{i-1} - h_i) / t_i}{\pi R^3} = \frac{h_{i-1} - h_i}{t_i} \cdot \frac{4}{R} \quad (6)$$

Then, from the falling time t_i (sec) of concrete at the height i in the J·shaped flow test, shear velocity V(1/sec)can be calculated from Eq.(6) with Eq.(3).

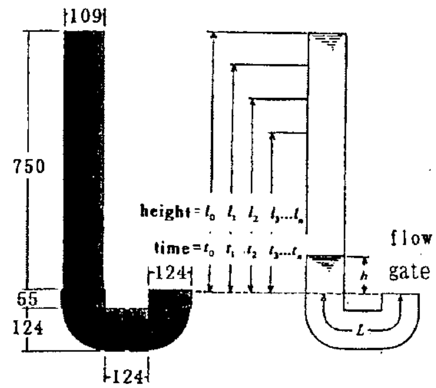
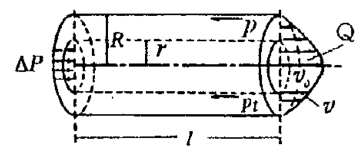


Fig.11 Details of J•shaped flow test apparatus



ΔP : driving pressure (g/cm^2)
R : diameter of tube (cm)
v_0 : velocity in center (cm/sec),
v : velocity in position (cm/sec)
P : shear stress of position (g/cm^2),
Pt : shear stress of position (g/cm^2)
λ : viscosity coefficient (g•sec/cm^2)
V : shear velocity (1/sec)
Q : flow (cm^3/sec)
r : radius of a position (cm)
l : length of tube (cm)

Fig.12 Model of body of laminar flow in tube

Fig.13 shows the consistency curves of a Newtonian fluid and a Bingham fluid obtained using a tube viscometer. The shear stress and the shear velocity can be calculated using the equations indicated in Fig.13, Eq.(5) and Eq.(6).

When we test a Newtonian flow by J·shaped flow test apparatus, the relationship between the shear stress p and the shear velocity V is represented by Eq.(4). In contrast, because there is an intercept F_0 in the shear stress for a Bingham fluid flow, the relationship between the shear stress p and the shear velocity V is represented by Eq.(7).

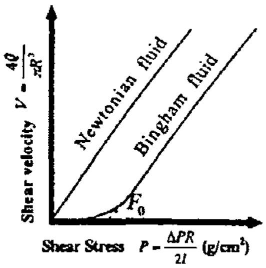
In Eq.(7), F_0 is a constant related to the initial shear stress yielding value, which we call the shear stress yielding value. In the J·shaped flow test, assuming its hight to be h (cm), and the resist length being $l = h + L$ ($v = 0$), when flowing of concrete is completed, the shear stress yielding value Fo can

$$p = \lambda \cdot V + F_0 \tag{7}$$

$$F_0 = \frac{\rho \cdot h}{L+h} \cdot \frac{R}{2} \tag{8}$$

$$\lambda = \frac{p - F_0}{V} \tag{9}$$

Fig.13 Tube viscometer



be calculated using Eq.(8). In this case, the distribution of velocity in Fig.12 indicates Plug flow . Once the value of Fo(g/cm^2) is obtained, λ of the Bingham fluid can calculated using Eq.(9).

4 . Test Results

Fig.14 shows the results of the concrete rotary viscometer test, and Fig.15, which shows results of the J·shaped flow test. These two groups of high fluid concrete shows almost the same tendency. Although the curves in Fig.14 show a larger range of the relationships between the shear stress and the shear velocity than those of Fig.15. However, lower part of shear velocity of the concrete of K-25's curve, in Fig.14 which seems to show the slipping between concrete and rotating inner cylinder by compulsory driving force of motor, making it difficult to measure the low shear velocity range accurately. On the other hand, because the J·shaped flow test uses the potential energy of gravity, the low range of shear velocity can be measured accurately. Then, in this study we will compared and investigated about the viscosity coefficient and the shear stress yielding value calculated from the results of the J·shaped flow test.

The compressive strength of each high fluid concrete after 4 weeks and the that of concrete after 13 weeks are tested, and the ratio of 13 weeks compressive strength to 4 weeks compressive strength (here after it called 13 weeks strength ratio) are shown in Table 4. Additionally, results of a general evaluation of the segregation of high fluid concrete from the results of In/Out aggregate weight ratio and 13 weeks strength ratio, are shown in the rightmost column of Table 4.

5 . Discuss

5.1 Discuss of consistency.

We discussed the test results of the fluidity of the rheological property of each type of high fluid concrete. N-40, N-35, N-30 and B-30 showed segregation, as shown in Table 4, which indicated by the high slopes of the consistency curve in Fig.14 and Fig.15. K-20, B-20 and B-23 tended to have worse workability due to their excessive viscosity, as shown in Table 4, this fact indicated by the low slope of the consistency curve. Therefore, from the viewpoint of workability and quality control, the proper range of rheological properties of high fluid concrete can be determined as the white area enclosed by a dot-dash-lines (viscosity coefficient 0.75～3.75) in Fig.14 and Fig.15. It was found that concrete above this area is segregated and the concrete below of the white area is to have be excessive viscose. But the slump flows of these concrete had the slump flow value about 60cm, so their segregation and excessive viscosity properties could not be detected by these conventional method. Fig.14 shows the concrete rotary viscometer test results and Fig.15 shows the J·shaped flow test results, all consistency curves show almost the same tendency, but the concrete rotary viscometer test covers a wide range of relationships between the shear stress and the shear velocity. The J·shaped flow test covers a narrow range of practical use. In Fig.14 the K-25's consistency curve has an extremely high slope. Slippage was observed between this concrete and the inner cylinder, which prevented accurate measurement. On the contrary, because the J·shaped flow test uses potential energy of gravity, it can measure without failure and consistency curves can be obtained.

5-2 Comparison with slump flow test

The relationships between slump flow values and the viscosity coefficients are shown in Fig.16. On the basis of the general evaluation in Table 4, the concrete with the viscosity coefficient in the range of 0.75～3.75 g·sec/cm^2, within the white area in Fig.16, have good fluidity. Concrete with viscosity coefficient over 3.75 g·sec/cm^2 (excessive viscosity) or under 0.75 g·sec/cm^2 (segregation) is not appropriate for the high fluid concrete. These concrete are considered to be inadequate fluidity, their slump flow values are 50～70cm. Therefore the inadequency of fluidity cannot be detected using slump flow values. The relationships between slump flow values and shear stress yielding values are shown in Fig.17. The white area indicates the range in which concrete is apt to have good fluidity, and shaded area, that in which concrete is not to be inadequate by their too big force to make concrete flow. As shown in Table 4, concrete B-20 and B-23, considered to have inadequate fluidity, have very high shear stress yielding value and show poor of workability. Their slump flow values are about 60cm, therefore defects of fluidity could not be detected by the slump flow test.

5-3. Results of J·shaped flow test and compressive strength of concrete

Up to now, we have compared the J·shaped flow test with the conventional test methods and have found that it can be able to evaluate the fluidity of concrete by using the viscosity coefficient and the shear stress yielding value derived from the J·shaped flow test. Furthermore, we are going to investigate the relationship between the results of J·shaped flow test and the compressive strength of concrete. When the ratios of 13 weeks compressive strength to 4 weeks compressive strength is under 1.0, it shows that the long term strength become lower than that of the 4 weeks strength. And when the ratio of 13 weeks compressive strength to 4 weeks compressive strength is in the range of 1.0～1.1, it shows that the long term strength does not developed so much. It is found for the concrete N-40, N-35 and N-30, which evaluated to be segregated or nealy segregated in previous judgement of fluidity, that the 13 weeks strength ratio are under 1.1. Although the concrete N-25 being evaluated to have good fluidity, having a high shear stress yielding value, indicating a 13 weeks strength ratio within the range of 1.0～1.1, and its long term strength did not greatly increase.

The range of the proper fluidity of high fluid concrete are shown with white area in Fig.18 with the shear stresss yielding values on the abscissa and the viscosity coefficients on the ordinate. The concrete with 13 weeks strength ratio under 1.0 is marked by ↓ , and that with 13 weeks strength ratio within the range of 1.0～1.1 is marked by ←. The concrete with the viscosity coefficient over 3.75 g·sec/cm^2 has inadequate fluidity due to excess viscosity. On contrary, concrete with the viscosity coefficient within 0.75～3.75 g·sec/cm^2 and the shear stress yielding value under 1.20 g/cm^2 had adequate fluidity as a high fluid concrete. Most of the concrete with the shear stress yielding value over 1.20 g/cm^2 had inadequate fluidity and 13 weeks strength ratio within the range of 1.0～1.1 and thus the long term strength of concrete did not develop. Additionally, when its viscosity coefficient was under 0.75 g·sec/cm^2, the concrete tended to segregate and long term strength decreased or did not develop so much.

On the basis of the results above, the best properties of fluidity for high fluid concrete are in the area enclosed by lines, where viscosity coefficient is in the range of 0.75 g·sec/cm² to 3.75 g·sec/cm² and shear stress yielding value is under 1.20 g/cm² (white area in Fig.18). Concrete in this area has less tendency to segregate and less strength loss in the long term.

Table 4 Results of test of fluidity of high fluid concrete

	Cement	W/C %	Slump cm	Slump flow value cm	L shapedflow test value cm	Falling length of L-shapedflow test value cm	Cylinder penetration test value cm	In/Out aggregate weight ratio	Shear stress yielding value g/cm²	Viscosity coefficient g·sec/cm²	4 weeks compressive strength (4W) kgf/cm²	13 weeks compressive strength (13W) kgf/cm²	13 weeks strength ratio (13W/4W)	General Evaluation of fluidity
N-40	OPC	40	25.1	52.5	-	—	-	-	0.82	0.22	519.2	401.4	0.773	Segregation
N-35	OPC	35	23.7	45.5	-	—	-	-	0.87	0.30	567.0	580.8	1.024	Segregation
N-30	OPC	30	26.2	68	66.5	33	21	1.18	1.28	0.42	810.3	736.8	0.9091	Nearly Segregated
N-25	OPC	25	23.7	51.5	49	25.5	12	-	1.27	2.55	783.8	835.2	1.066	Nearly Adequate
K-30	BFSC	30	24.8	59.5	60.5	32	24.5	1.28	0.68	0.75	683.0	814.6	1.193	Adequate
K-25	BFSC	25	26.1	65.5	85	35	15.5	0.99	0.40	1.27	744.8	847.2	1.137	Adequate
K-23	BFSC	23	28.0	69.5	82.5	34	-	1.08	0.39	1.37	800.0	1054.8	1.319	Adequate
K-20	BFSC	20	26.5	60	72	33.5	24	0.97	0.38	4.38	695.6	939.1	1.350	Inadequate fluidity
B-30	Belite	30	25.3	59.5	58	33	26	1.12	0.62	0.47	704.8	811.1	1.151	Nearly Segregated
B-25	Belite	25	23.0	59.5	56	32.5	22	1.19	0.61	0.96	725.0	936.1	1.291	Adequate
B-23	Belite	23	25.4	59	50	30.8	13	1.32	1.30	5.33	800.9	1051.6	1.313	Inadequate fluidity
B-20	Belite	20	25.9	59.5	52.8	31.4	11	1.21	1.50	7.59	866.2	1146.6	1.324	Inadequate fluidity

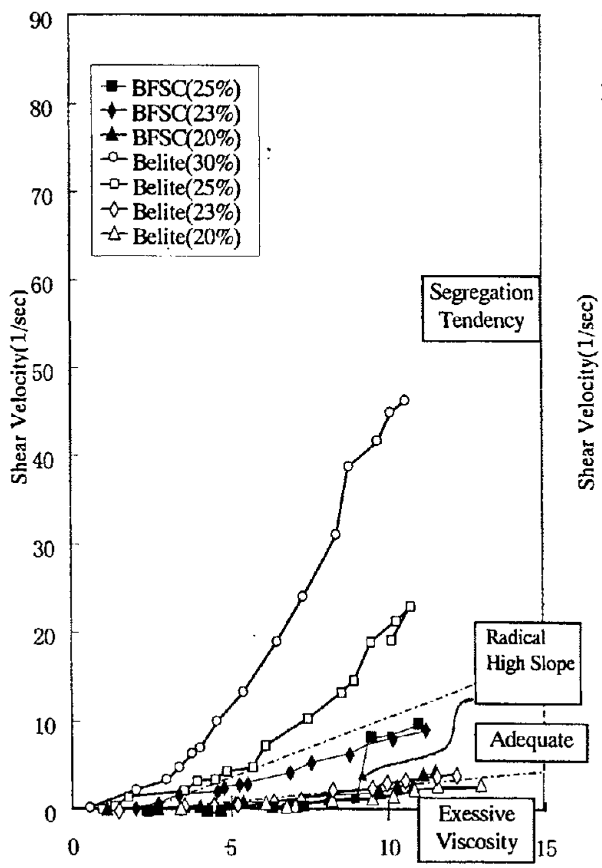


Fig.14 Results of rotary viscometer method

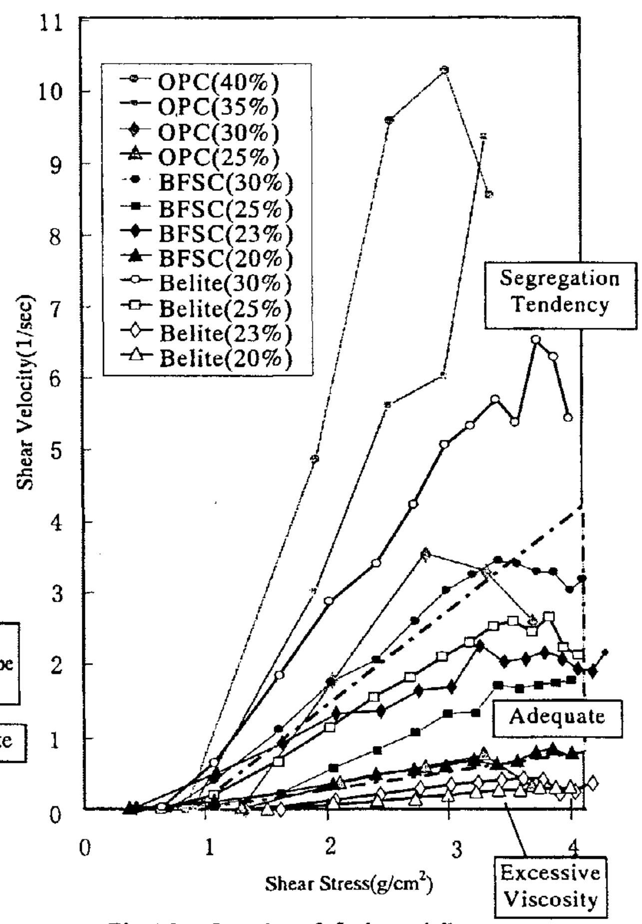


Fig.15 Results of J-shaped flow test

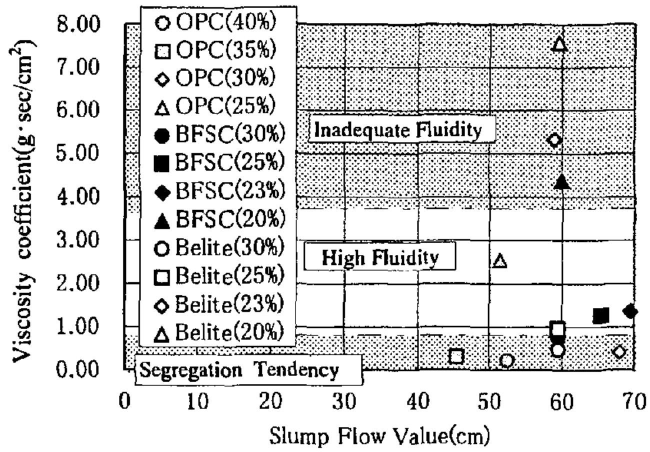


Fig.16 Slump flow value and viscosity coefficient

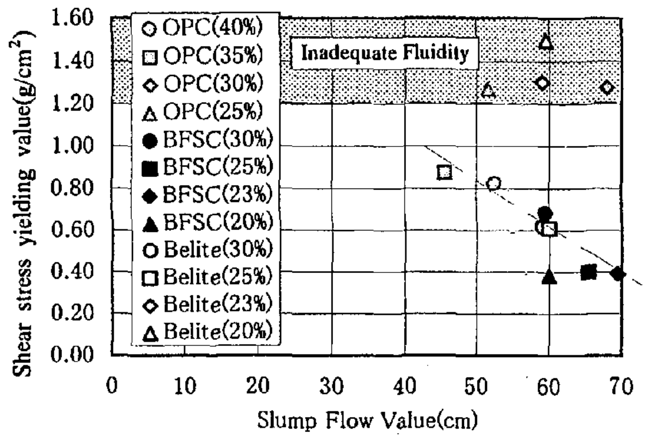


Fig.17 Slump flow value and shear stress yielding value

Marked ↓ 13 weeks strength ratio under 1.0, Marked← within 1.0~

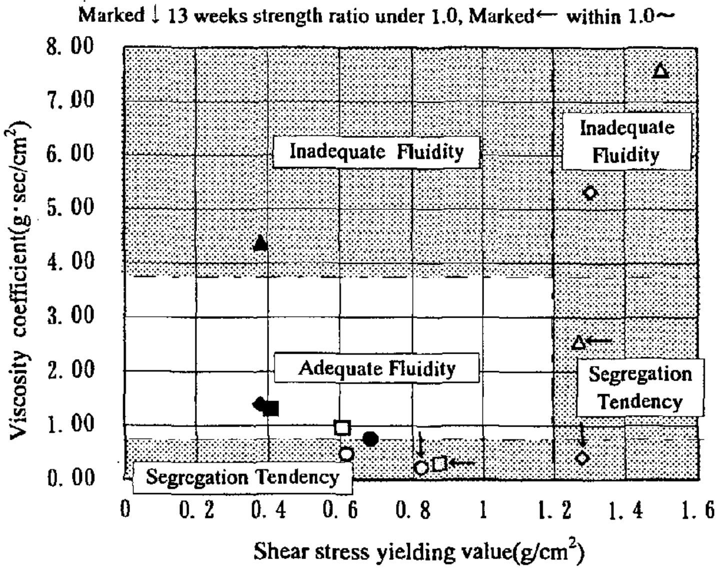


Fig.18 Shear stress yielding value and viscosity coefficient

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INNOVATIVE DESIGN, CONSTRUCTION AND REHABILITATION METHODS FOR CIVIL INFRASTRUCTURE RENEWAL

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ABSTRACT

Following the 1994 Norhtridge earthquake many innovative procedures were implemented to accelerate the recovery of the civil infrastructure systems. One of the areas where these innovative procedures were most notable was the highway transportation network. However, innovative procedures were also utilized by several of the public utilities to accelerate service restoration. In the state highway and transportation area the California Department of Transportation (Caltrans) incorporated new design features which were tested and used in the bridge reconstruction, used accelerated contracting procedures for twelve major projects, and adopted nnovative advanced composite material applications to speed recovery and provide an improved infrastructure. Most major damaged routes were reopened in three months after the earthquake and the major interchange of two highways wes reopened in six months. These accelerated design and construction procedures were also utilized for reconstruction of three damaged bridges after the 1995 flooding in Central California.

INTRODUCTION

The Northridge earthquake of January 17, 1994 caused approximately \$ 18 billion in property damage to the Los Angeles area, including loss of most of the hospitals in the northwest portions of the city close to the epicenter. This area, also known as the San Fernando Valley, suffered the loss of four major freeways due to approach embankment settlement and bridge damage. Two other freeways in Los Angeles were closed due to bridge damage. While the freeway and street damage accounted for only \$ 0.5 billion of the total property losses, the loss of these major freeways and arterials was a crippling blow to the transportation system. The transportation system is a vital infrastructure element and critically required for disaster recovery. The cities in the northern San Fernando valley could not recover without the transportation system to provide mobility and site access for other public utilities and construction equipment, and to facilitate return of normal commerce. Twelve bridges on the six freeway routes were damaged severely and required replacement. Ten of the twelve bridges were older structures that had been designed and built prior to the 1971 San Fernando earthquake. These structures were included in the Department's bridge seismic retrofit strengthening program but the retrofit work had not been completed when the Northridge earthquake struck. Approximately 190 other bridges suffered minor to moderate damage which did not require closure to repair. Another 115 bridges in the hardest hit areas, which had been recently retrofitted or had been built to the post 1973 seismic design and performance criteria, had no appreciable damage. Therefore, most of the Los Angeles Freeway and road network was in full operation immediately after the earthquake. The most heavily traveled freeway in California, the Santa Monica Freeway, traverses the downtown Los Angeles city center from East to West and carries 330,000 vehicles per day. Severe damage to two large bridges on this route closed it immediately, causing major traffic congestion. Innovative design and construction procedures were implemented by

ACCELERATED DESIGN AND CONSTRUCTION

The California Department of Transportation (Caltrans) working with the Federal Highway Adminsitration (FHWA) immediately embarked on a program of accelerated cleanup, recovery and reconstruction to reopen the damaged routes as rapidly as possible. In the first few days "Force Account" or "Day Labor" procedures were used to provide shoring, demolition and debris removal, and emergency repairs. Contractors were simply called in to begin work under the supervision of the State Engineers. Within hours damaged structures were being shored up until repairs could be completed, demolition of the twelve bridges with major damage began and detours were put into service, utilizing local streets and older highways that paralled the freeways. After the first two to three days emergency contracts were utilized, wherein three contractors are given a sketch and some limited details upon which to base an estimate. The lowest of the three bids is accepted and the contractor begins work immediately. Much of the emergency minor bridge repair was accomplished using this informal contract procedure.

For the major reconstruction of freeway pavements and bridges the normal contracting process was utilized, but on an accelerated time schedule. The bridge design office began designing the replacement structures on the day of the earthquake and worked around the clock and weekends. Consultants were used to check the designs and peer review panels of outside experts were also engaged to insure that the latest seismic design technology was incorporated in the new designs. Since the Gavin Canyon bridges were vital to the recovery effort because of the heavy truck traffic, it was the first emergency contract to be attempted. Eight days after the earthquake the plans and bid documents were completed for the twin 785 foot structures. At the same time state highway maintenance crews and contractors were repaving the old highway to be used as a detour while the bridges were being rebuilt.

On the twelfth day after the earthquake the detour was opened and a contract was awarded to rebuild the two bridges. The bridges were completed and the freeway reopened to all public traffic less than five months after the earthquake. All the damaged bridges and freeways were completed by the same accelerated processes and reopened to traffic within six months of the earthquake. The record for completion was made at the two sites on the Santa Monica freeway, Interstate Highway 10. The contractor on that project completed the work in just ten weeks after being awarded the contract. The earthquake occured on January 17, 1994 and the freeway was reopened on April 11, 1994. This remarkable achievement was due to cooperation between the design consultant, the state engineers who checked the designs, the Peer Review Panel whose members worked rapidly to stay on schedule with the design effort, the local city agencies who had to issue permits and cooperate with development of the detour plans, and the contractors, sub-contractors, and material suppliers who accomplished the work. Other damaged bridges were also replaced in record times and the entire freeway system was back in operation in November, ten months after the earthquake. There were critics who questioned how the work could be completed so fast and not jeopardize the quality that is necessary for future longevity of the completed structures. It was, therefore, important to make extraordinary efforts to insure that quality was maintained from the design to the final construction. Most contractors who worked on these projects have stated that the Caltrans engineers were tougher on them on these projects than on our normal work, probably due to the very large financial incentives.

Following the 1995 Central California winter storms, which caused heavy flooding and loss of bridges at three sites, Caltrans again utilized these same accelerated design and consstruction procedures to reconstruct bridges on Interstate 5, the major north south truck route, and at two sites in Monterey County on the coast. In these instances temproary bridges were also used to provide reasonable detours while the accelerated reconstruction was underway. The comabination of these rapidly constructed temporary detour structures and the accelarated reconstruction resulted in Interstate 5 being fully restored in 33 days and the Carmel River bridge on the scenic coast highway being replaced by a new permanent bridge in eight weeks.

THE ACCELERATED PROCESS

The accelarated reconstruction process requires some emergency powers through a proclamation of emergency by the Governor, but most of the acceleration can be accomplished without such emergency powers. Success depends on the cooperation and diligence of all involved parties as stated above. Engineering and support staff, consultants, bond/surety houses, attorneys, contractors, sub-contractors, and material suppliers had to work together to get the city back on its feet.

Design

The design office was busy working on new designs while the rubble was being removed. All but one of the projects were designed in-house and checked by consultants. One project was designed by the consultant who had just completed seismic retrofit plans and had most of the site and geometric data and computer models on hand. The Caltrans bridge design office checked the consultant's work. For the design of four of the long viaduct structures in the major interchange at State Sign Route 14 and Interstate 5 Caltrans engineers flew to Pittsburg, Pennsylvania to use the AISC steel bridge optimization computer program. In ten days they returned with four optimized bridge designs and completed contract drawings in the Caltrans offices. These four large bridges were bid with two alternatives, Steel girders and Cast-in-place pre-stressed concrete to provide for maximum flexibility and speed in construction. The typical accelerated contract bid documents consisted of a General Plan, Foundation Plans, estimated quantities for each bid item and the Standard Detail Sheets. Contract Special Provisions provided for a specific date for delivery of the superstructure plans and provided for adjustment of pay if quantities varied from the estimated amount. Quality control of the plans and specifications was assured by the use of the same peer review panel for all the bridges. The very latest seismic technology and seismic details were utilized.

Bidding Process

There were eight steps in the accelerated bidding process. All these steps were designed to provide the best contractors to complete the work, provide a level playing field at bid time, accelerate all steps to shorten the processing time from the normal routine contracting, and to get the work completed in as short a time as possible.

1. Preselection of Qualified Contractors-The field construction staff, Resident Engineers and Inspectors were queried to obtain a list of the contractors who had worked in the Los Angeles area, had a reputation for fast and high quality work, had experience with the minority/disadvantaged (MBE/DBE/WBE) sub-contractors in the area, and had a history of few claims against the state. Using this list, the Chief Engineer selected five or six contractors who were allowed to bid the project. We could not allow more than this number and get the bids opened, reviewed and awarded in one day. Once a contractor was successful low bidder, the firm was removed from the list. This was done to spread the work and not allow one contractor to load up and potentially delay some work.

2. Bid Package and Conditions Discussed with Selected Contractors- Caltrans Office Engineer called each of the potential bidders by phone to discuss all the conditions for bidding. They were required to work 24 hours a day, seven days a week. They were required to comply with a Minority/Disadvantaged (MBE/DBE/WBE) sub-contracting requirement of 40% (an effort to provide work for people in the area who had been hard hit by the earthquake and to pump the local economy). They were to be ready to begin work at midnight on the day of contract award. They had only two and one half days to prepare a bid after receiving the documents. The contracts were all bid with an A+B process which included the construction bid and proposed time to complete. Incentive/disincentives were incorporated to encourage fast construction.

3. Information Sharing-All questions raised by any one contractor and the answers to those questions were immediately telephonically discussed with all other selected bidders.

4. Bid Package Hand Delivered-All plans and specifications and other bid documents were hand delivered by Office Engineer staff to the nearest airport to the contractor's office. This typically was during the late evening hours.

5. Bids Opened 2 1/2 days later-Sealed bids are traditionally opened at 10:00 AM on bid opening day. This gave the contractors just 2 1/2 days to analyze the project and prepare bids. There were no complaints from the contracting industry on this issue because they were aware of the emergency nature of the work and the urgency to get moving. This was one feature of the process that did require the Governor's emergency declaration because the normal bidding process requires advertising in a prescribed number of publications and at least two weeks for contractors to prepare bids.

6. All Normal Bid Processing Completed and Award Made in One Day-This is a process that normally requires three weeks. To make it happen the Office Engineer took her entire Bid opening and processing staff to Los Angeles for the bid openings. Review of bids for errors, confirmation of sub-contracted items with the subs, DBE/MBE/WBE compliance review, good faith hearing, contract award, review of performance bonds, and final approval of the contract were all completed in one day. That required the sub-contractors and bond/surety houses to be available by phone. It required the attorneys for both contractors and the state to be available for signatures. By the time we had done several of these accelerated processes the Office Engineer could process three contracts in one day.

7. Contract Time Begins Next Day-Work began and the time clock began at midnight on the next day. All project contractors were there and working at the prescribed time. All of them completed their projects ahead of their own established schedule.

8. Incentive/Disincentive Bonuses Based on Loss of Use-The amount of the incentive/disincentive payment or penalty was determined by the traffic engineers as a function of loss of use of the facility by the traveling public.

A+B Bidding Process

Contractors were required to bid two items; the "A" bid was for the payment to construct the project, the "B" bid was for the number of calendar days to complete the work. The number of days in the "B" bid was multiplied by the incentive/ disincentive amount previously determined for the project, depending on traffic volume on the route. Caltrans estimated the number of days to complete the project using a 24 hour day, 7 day work week and would not accept bids with more than the designated maximum days. Individual contractors could bid the maximum number of days and lower construction costs if they were conservative, or they could bid higher construction costs and fewer days to complete. In the bid analysis the lowest sum of the A and B elements combined determined the successful bidder. That was not the final payment, however. If a contractor completed the contract earlier than the days they bid an incentive was paid for each day completed earlier. This was the payment for the "B" portion and was added to the "A" portion to determine the final payment. Conversely, if the contractor completed later than his estimated time a penalty of the same amount would have been charged for each day overrun. None of the contractors completed late. Typically it cost the state about 50% more for the accelerated completion than it would have cost for a normal work week contract.

Figure 1 shows the comparison of an accelerated schedule versus a normal schedule for a typical Northridge earthquake emergency bridge reconstruction project. The normal design time which is 52 weeks was completed in two weeks; the Plans, Specifications and Estimates (PS&E) processing time from completion to advertising for bids, which is normally 12 weeks was reduced to two days; the normal advertising time of six weeks was reduced to three days; the bid opening to contract award time of 5 weeks was reduced to one day; the normal construction time of 52 weeks was reduced to 20 weeks. The typical total time was reduced from 127 weeks to 23 weeks. For the Santa Monica freeway bridges that time was reduced to 12 weeks. The incentive/ disincentive clauses ranged from $20 thousand to $200 thousand per day. The higher number may seem exhorbitant but the loss of use on that route (Santa Monica freeway) was calculated at $1 million per day. Certainly the citizens who used the freeway each day considered the incentive payment worthwhile because their delay during reconstruction was approximately one hour each way every day.

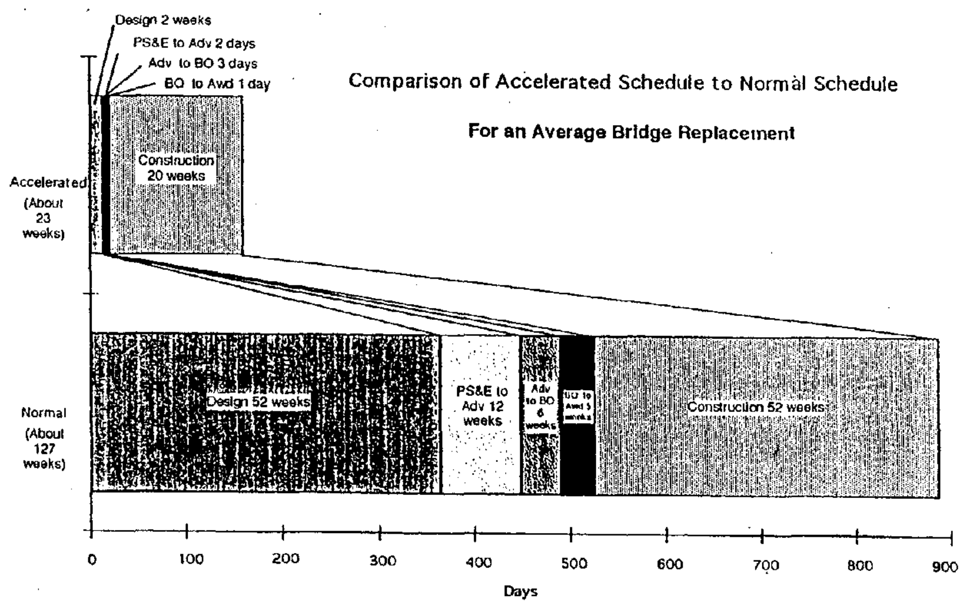

Figure 1 Accelerated Schedule

Construction

Contractors began work at midnight on the day of contract award and worked continuously until completion and opening to traffic. After opening , additional working days were allowed for cleanup and minor detail work which did not affect the flow of normal traffic. After the additional time, if the contractor orerran the contract days designated by the state, liquidated damages were assessed in the same manner as on our normal contracts. Most contractors worked two 12 hour shifts, seven days a week. Caltrans Resident Engineers and inspection staff personnel were divided into two groups so that 24 hour inspection and contract administration could be maintained. Field office trailers were moved to the worksites so the engineers could be on the jobsite at all times. This is not the normal practice in a congested urban area because of the scarcity of office space or land on which to park office trailers. Our field engineers in most large urban areas work out of remote office complexes and travel to the actual work sites in pickups and passenger cars.

Designers and consultants were on call by phone 24 hours a day to resolve any design problems that might occur. The Caltrans headquarters staff who needed to be contacted were available by cellular phones and pagers throughout the accelerated construction period. The staff engineering geologists and geotechnical engineers were available from the Los Angeles branch office to inspect the foundation work and immediately resolve any differences caused by changed site conditions. On at least one of the projects the pile tips were lowered after the contractor had begun work and a contract change order was written to compensate the contractor for the additional work. The contractor was also given additional time because of the unforseen delay so he was still eligible for the incentives for early completion. This was standard practice when necessary changes were made or when actual item quantities substantially exceeded the original estimated quantity. Figure 2 shows the damage and collapsed spans of a Connector Ramp at the Interstate Route 5 and State Route 14 Interchange. This location was 10 kilometers from the epicenter.

Figure 2 Collapse of Connector Ramp at Interstate Route 5 and State Route 14 Interchange

Because of the accelerated construction schedules, many of the normal construction operations were condensed in time and there were multiple contractor/sub-contractor operations underway simultaneously. This fact required close coordination between all the contractors, sub-contractors, state field engineering staff, material suppliers, the headquarters designers, and consultants to insure that nothing delayed any part of the work. Figure 3 shows the congestion and multiple operations on a typical work day during the accelerated construction on the more difficult site at Gavin Canyon on the Interstate Route 5 truck route. Due to the acceleration of design the final foundation exploration was not complete and there were both state Geology boring crews and contractor drilling operations being conducted simultaneously. On this bridge the pile tips were lowered from the tip shown on the preliminary plans after the exploratory drilling was completed. This change was handled as an increase in bid quantities at bid prices. Both Drilling and furnishing are bid at linera foot prices and the Caltrans specifications allow increases up to 25% at bid price. A contract change order was written to cover the additional cost and a time extension was granted because the pile reinforcing had already been ordered and had to be adjusted. Figure 4 shows the site after only two weeks work. Note that columns are being cured with blankets, column foundation pile reinforcing is beiang placed, concrete is being poured in abutments, and falsework for the deck is being set in the upper left hand corner. The site consists of two parallel bridges over 80 feet (26 meters) above the existing county road. That lower roadway was being used as a four lane detour during the reconstruction.

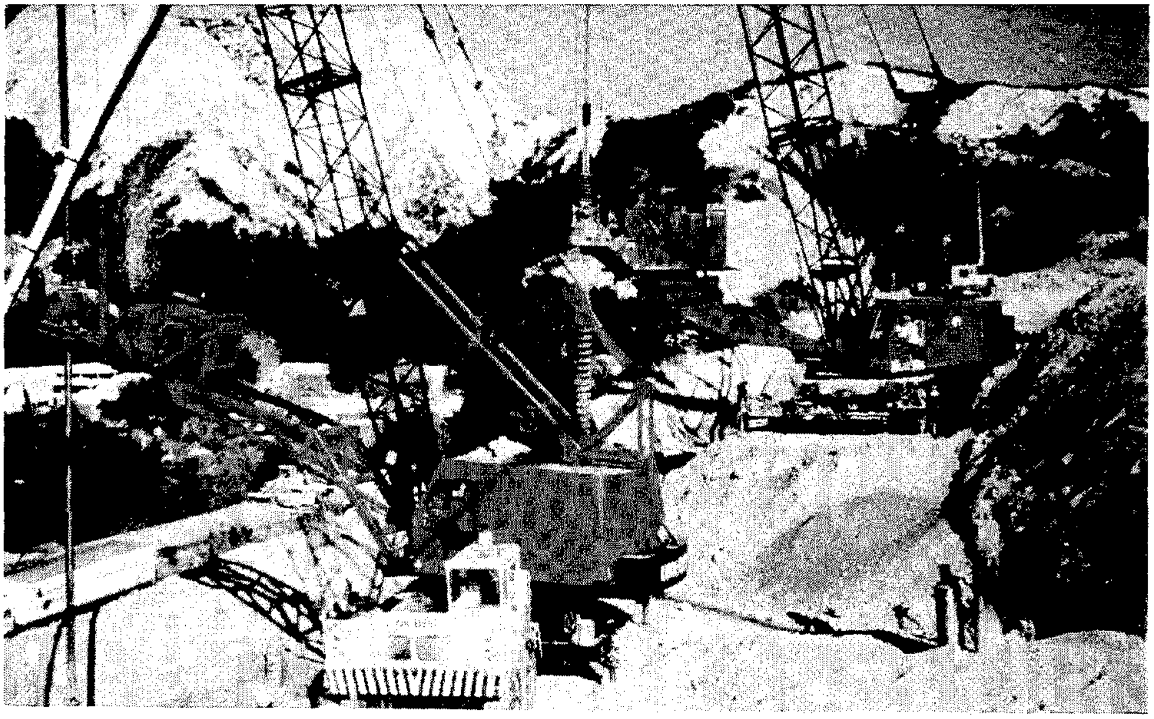

Figure 3 Congestion at Abutment Site

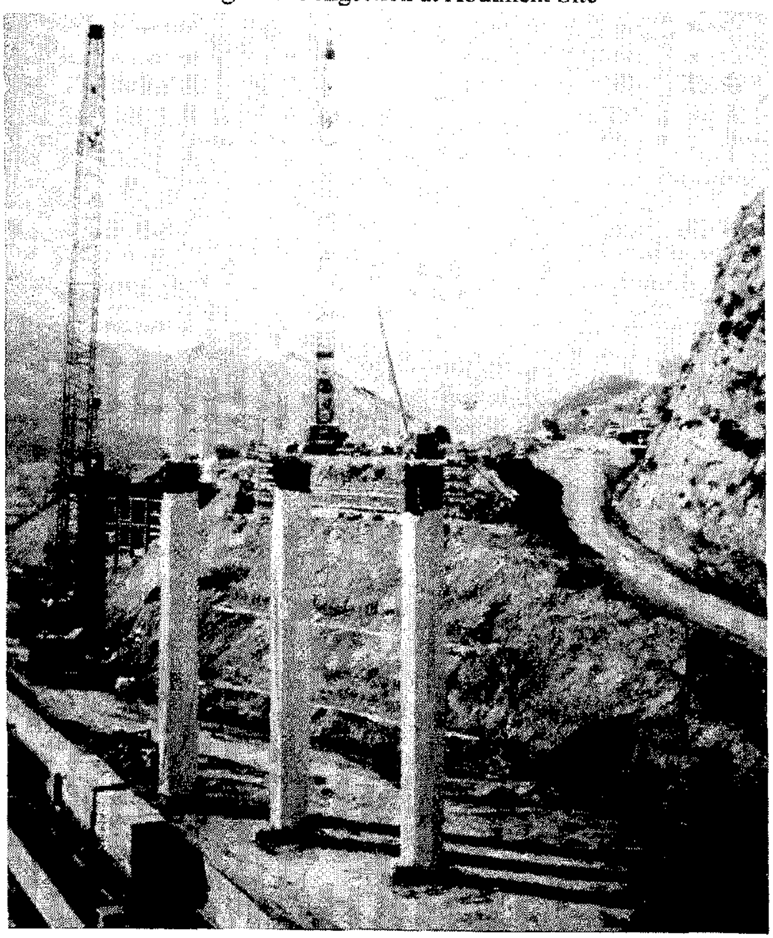

Figure 4 Multiple, Simultaneous Operations-Two Weeks

Figure 5 shows the completed Gavin Canyon Bridges. Note lack of joints because all expansion is accomodated at the abutments (up to 5 feet at each end). Also note that all columns are of equal length for flexural compatibility.

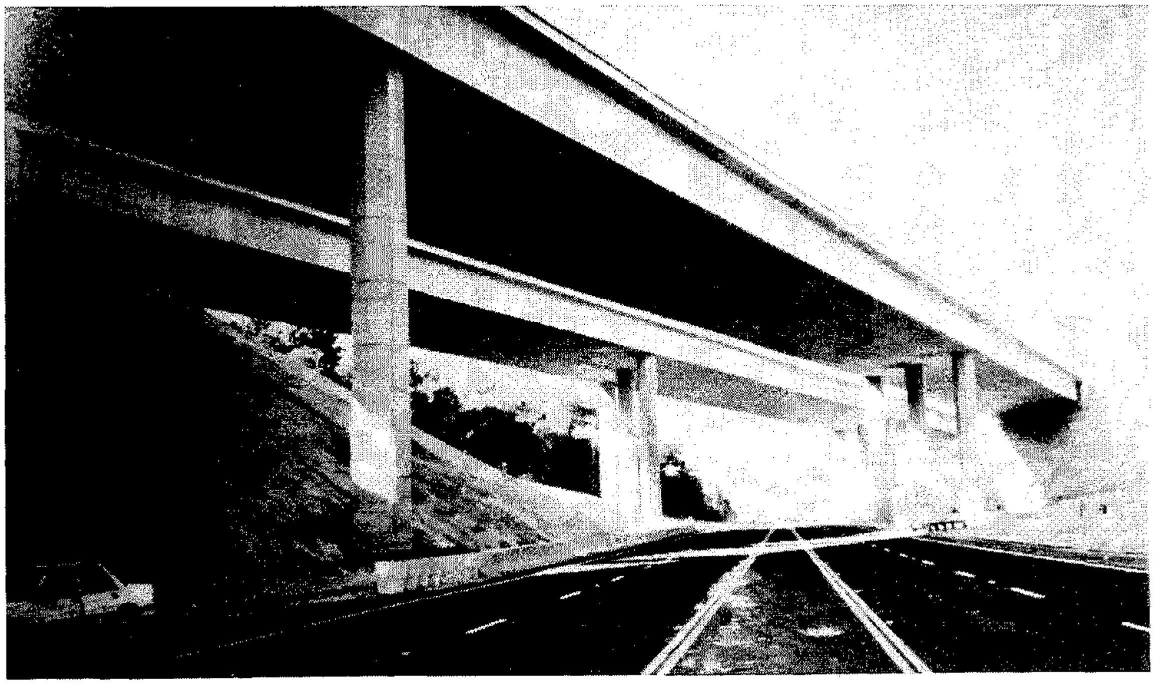

Figure 5 Completed Gavin Canyon Bridges

Figure 6 shows the congestion at the worksite for reconstruction of bridges on the Santa Monica Freeway (Interstate Route 10) in Downtown Los Angeles. This site is in a residential area so noise abatement was critical and caused additional work for the contractor. Residents were moved into Motels for the first few weeks until the foundation work was completed. Figure 7 shows the completed bridges, which were completed in ten weeks and the freeway was reopened less than three months after the earthquake.

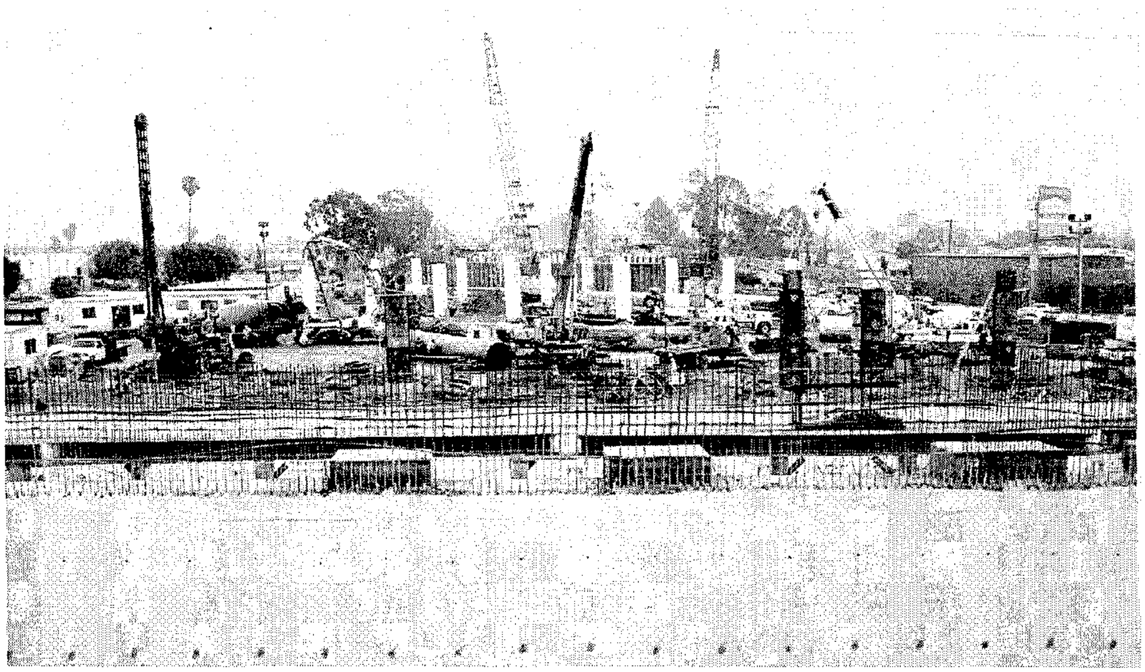

Figure 6 Congestion at Santa Monica Freeway Site

Figure 7 Completed Santa Monica Freeway Structures

Two improved design features were incorporated into the accelerated reconstruction of the Interstate Route 5 and State Route 14 Freeway to Freeway Interchange just two miles south of the Gavin Canyon Bridge site. Both features were used to provide for large movements during earthquakes. The first detail provided for large movements at necessary deck joints while eliminating the possibility of a deck collapse as shown in Figure 2. The detail is shown in Figure 8 which is typical of three similar details used at this interchange on three separate ramps. The columns are more closely spaced than normal so the decks can be cantilevered out to the movement joint a\with neither side supporting the other. Large movements from ground motion or structure deflection can be accomodated with these details. A closeup of this detail is shown in Figure 9.

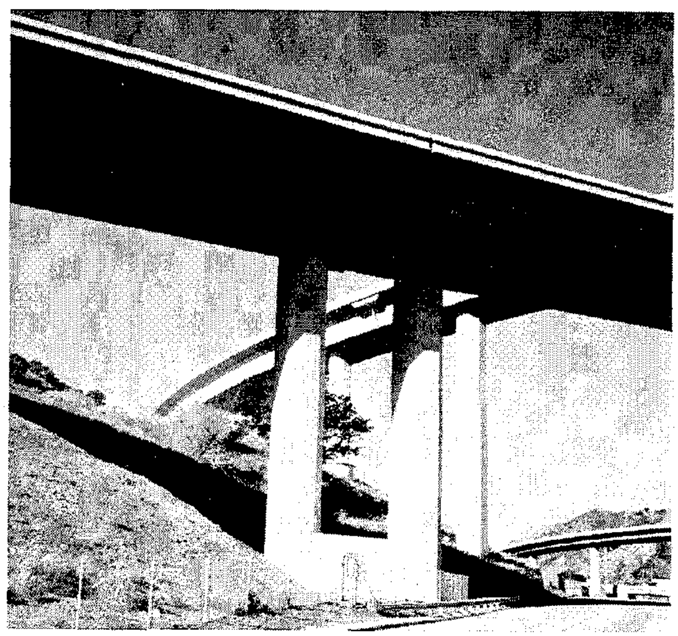

Figure 8 Large Movement Joint for Long bridges

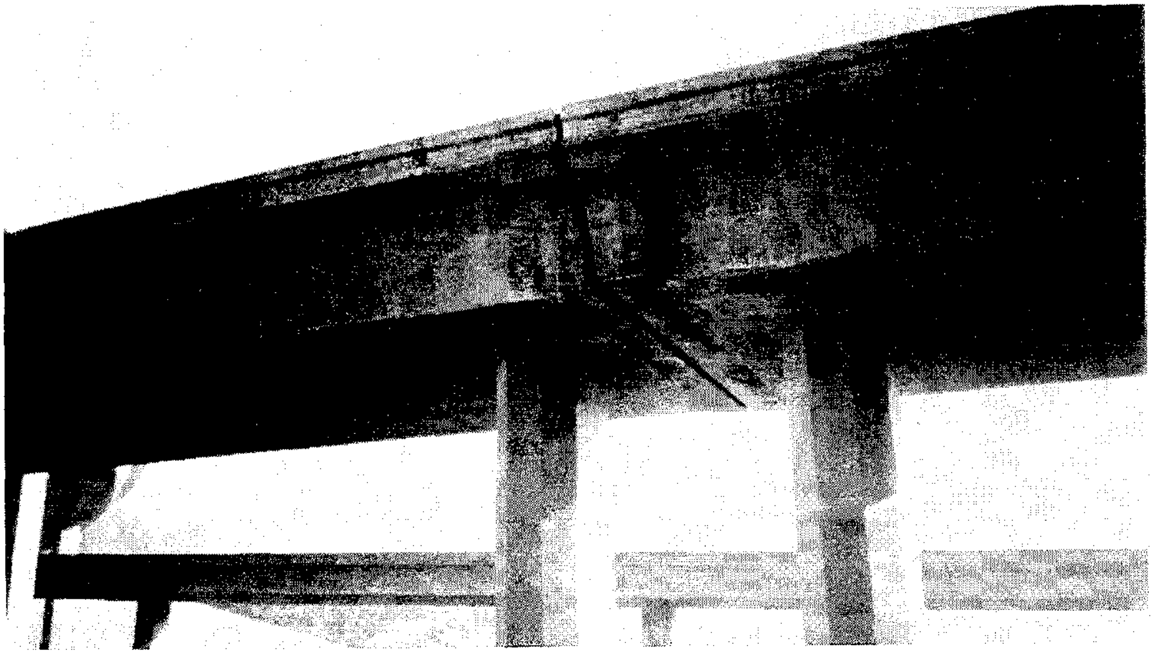

Figure 9 Closeup of Joint Detail

A second detail is shown in figure 10. The steel shell surrounding the column is a seismic isolation silo, used to extend the column length so the end columns are nearly as long as the middle columns and can deflect without taking all the seismic loads. This detail is used extensively on the shorter end columns of large bridges with variable height columns. Figure 11 shows the completed interchange. Utilizing both innovative new design features to prevent a repeat of the damage and innovative construction procedures this interchange was completely demolished and reconstructed in nine months. The main connector ramps for mainline Route 14 traffic, in the center of the picture, were opened to traffic six months after the Northridge earthquake.

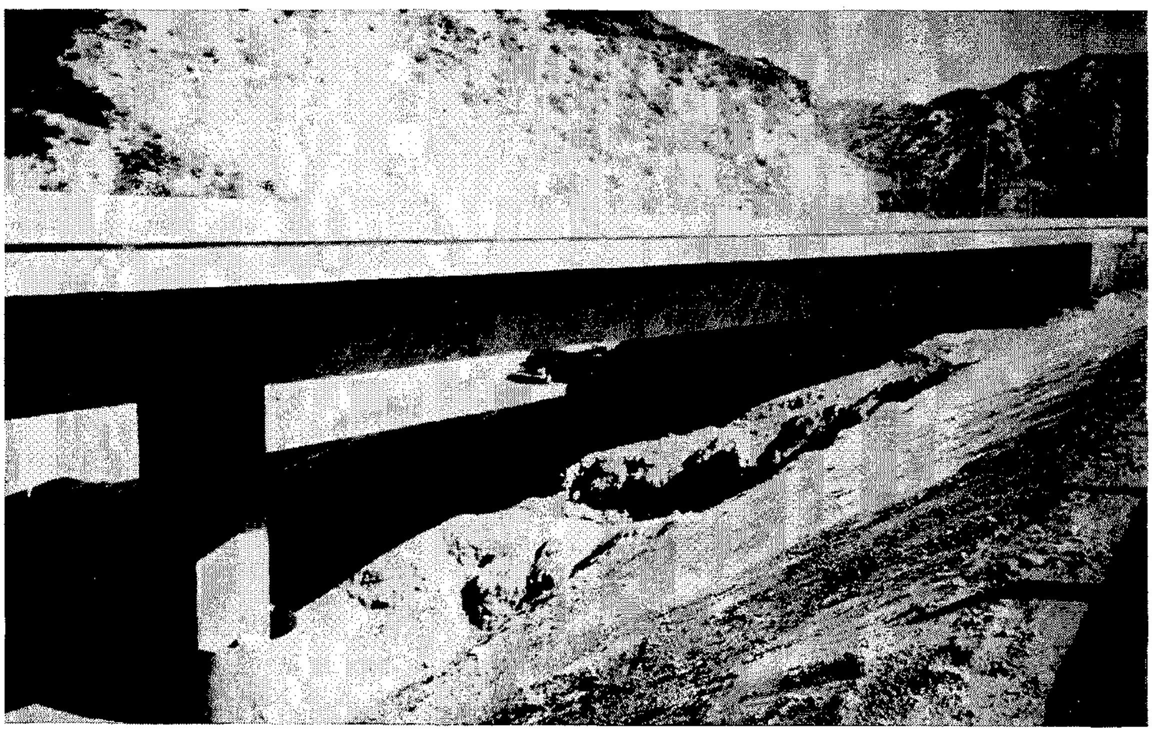

Figure 10 Column Seismic Isolation Silo

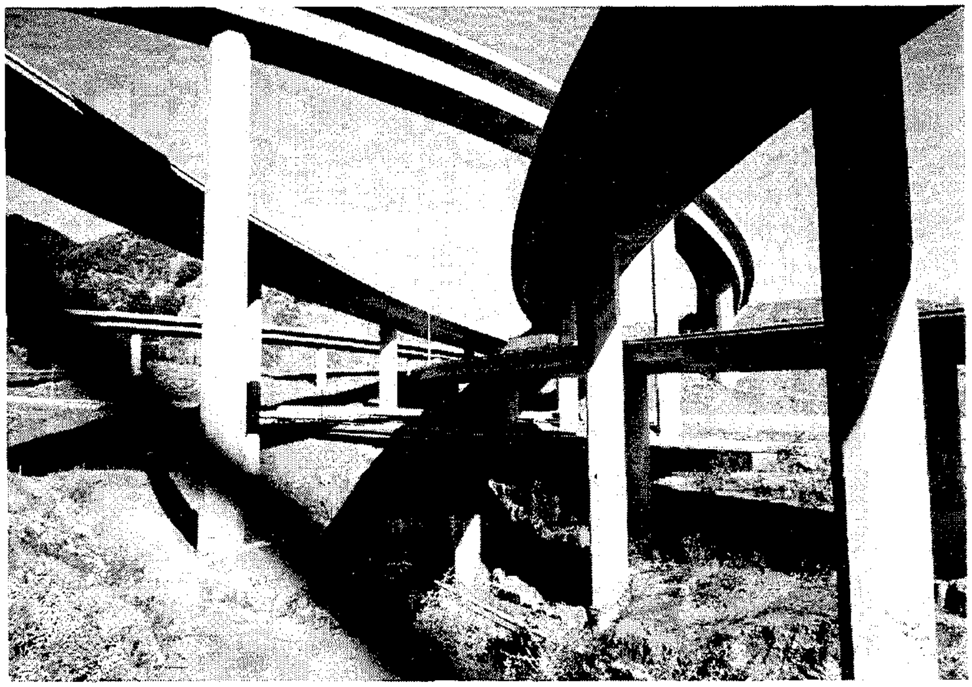

Figure 11 Completed Interchange at Interstate 5 and Route 14 Freeways

REHABILITATION/RETROFITTING

Caltrans has funded research at the University of California at San Diego for the past six years to develop field applications of advanced composite materials for both repair of older structures and construction of new bridges. The most highly developed application to date is the use of advanced composites in repair of bridge columns and other supporting elements to improve their ductility for seismic resistance. Epoxy impregnated fiberglass and various resin impregnated carbon fiber materials have been tested in the laboratory on half-scale models of bridge columns to determine the ductility that can be achieved in an older, non-ductile concrete column. The tests have confirmed the viability of these materials for strengthening existing structures and for replacement components of rebuilt or new bridges in infrastructure renewal. Material Quality Control Specifications and procedures have been well established by the Aeorspace and Defense industry but civil infrastructure field application quality control specifications were not originally available and have been developed along with the testing program.

Column Strengthening

Following the October, 1989 Loma Prieta earthquake Caltrans began a research program, in cooperation with the University of California at San Diego (UCSD), to develop techniques for utilizing epoxy impregnated fiberglass sheets to wrap around older, non-ductile concrete bridge columns as an alternative to the already proven steel jacket technique. The jackets provide sufficient confinement in the concrete to allow them to perform in a ductile manner under seismic loading. It was known that the Japanese had used high strength carbon strands to similarly reinforce industrial stacks and chimneys but the use of glass fiber sheeting had not been used. The major unknown was the durability of the fiberglass materials under cyclic loading and to what level of ductility the columns could be designed. The testing program was conducted under the same conditions that were used in the testing of steel plate jackets. Half scale models of the prototype bridge columns were constructed, wrapped with the desired layers of glass fiber sheets and tested through several cycles of loading at various levels of ductility until the column failed due to degradation of its hysteretic performance. These laboratory tests proved that the epoxy impregnated fiber glass column wraps could develop nearly the same ductile performance as the steel plate jackets. Figure 12 shows a typical testing laboratory setup for the fiberglass column wrap. Figure 13 Shows the completed field installation on bridge columns in the Interstate Route 5/State Route 2 interchange near Griffith Park in Los Angeles. This installation was completed in 1990 and performed well during the Northridge earthquake.

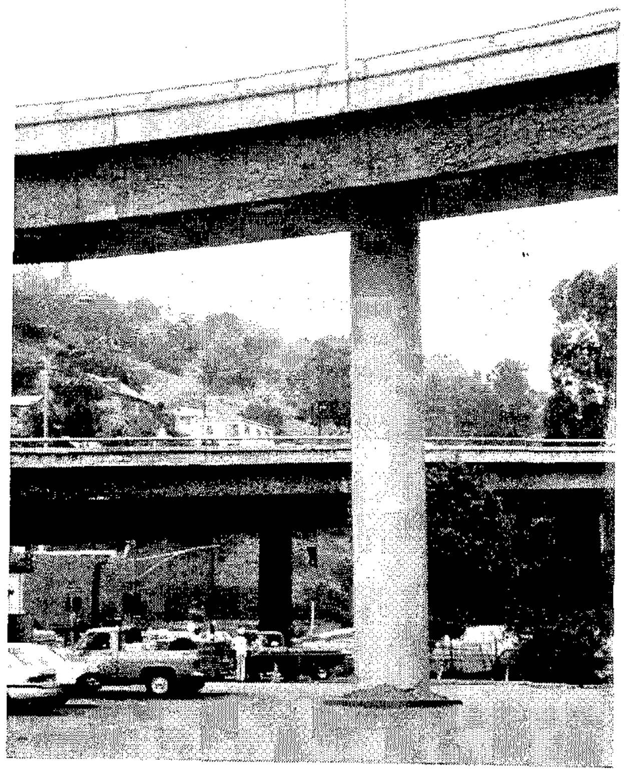

Figure 12 Laboratory Testing of Fiberglass Column Wrap

Figure 13 Field Application of Fiberglass Column Wrap

Material properties are readily available from the manufacturers but there remained the issue of adequate quality control specifications for the field application. These early applications were rather crude, being hand laid in a similar manner as hanging wallpaper. It required some months to fully develop adequate quality control (QC) specifications so the materials tested in the laboratory could be replicated with confidence in the field. The application using epoxy impregnated fiber glass has been approved for two systems and field applications have been in place for over seven years.

There is a larger market for this technology in the simple repair and strengthening of columns which have deteriorated from corrosion. It is relatively easy to clean and repair these columns and encase them with the non-corrosive composite materials. This application will undoubtedly increase the life of the columns or piers. Three manufacturers have developed prefabricated resin impregnated-fiberglass shells which can also be used as the form for concrete in the repair process. Because the prefabricated shells have been heat cured in the manufacturing plant they can be used under water as forms for the repair concrete. Figures 14 and 15 illustrate two of the three prefabricated fiberglass column shell systems which have been approved and can be used for repair of both columns and piling. Figure 16 shows one of the jackets being installed on the test column at the UCSD laboratory. These systems have been utilized by the US Navy for repair of pier columns in Southern California, where both underwater and splash zone applications are necessary. These systems have the advantage of low labor costs and are manufactured in various lengths to facilitate easy application. The critical issue with these applications is the curing of the adhesives used to fasten the shells to the in place concrete members. Good field quality specifications and testing, followed by fffadequate inspection fo the work are the key to success for these applications.

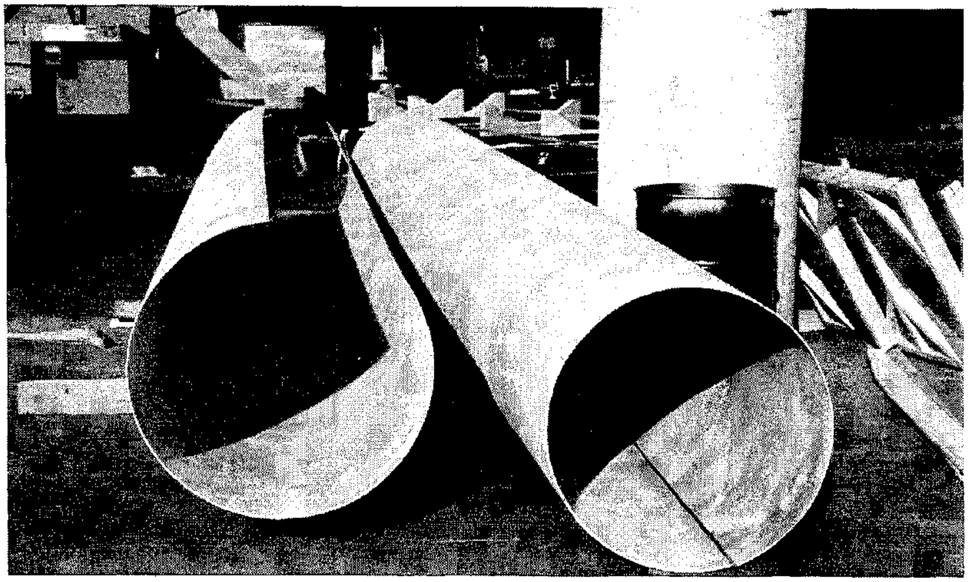

Figure 14 Prefabricated Epoxy-Fiberglass Shell (Snaptite System)

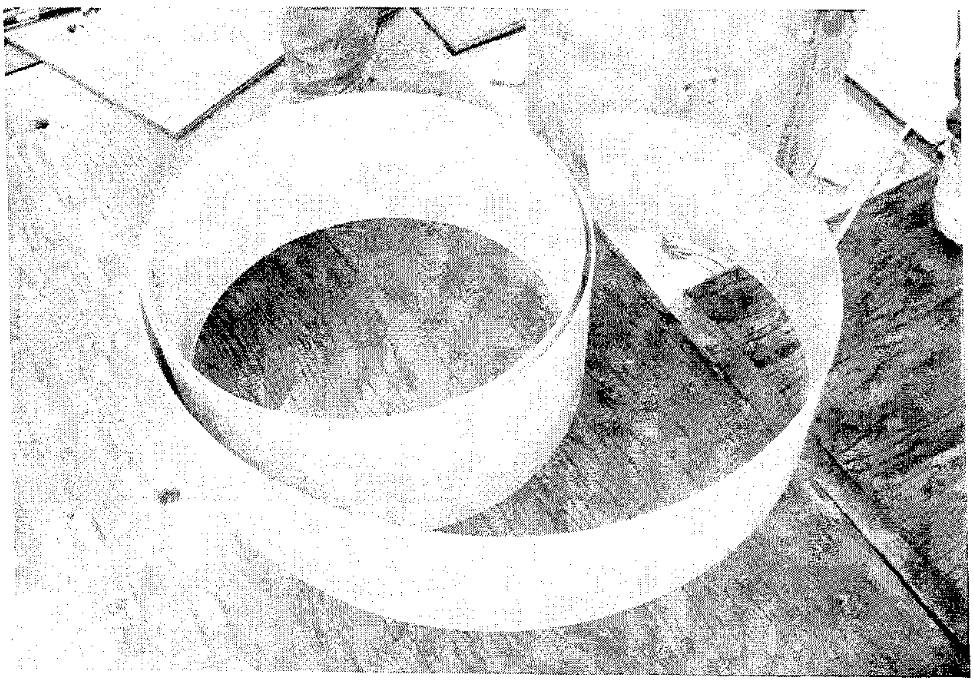

Figure 15 Clockspring System

In 1993, following the end of the cold war and reduction of major aerospace and defense applications, the advanced composites industry began looking for applications of advanced composites in the civil infrastructure. The Caltrans-UCSD testing program was expanded to develop similar applications for the higher strength carbon fibers. This testing program has continued as more manufacturers submit their materials for approval and there are at least five systems approved for field application in California at this time. The carbon fibers are applied by automatic wrapping machines which wrap several 1/4 inch strands simultaneously and can fully wrap a typical four to six foot diameter, 20 foot long bridge column in two hours. Because of the higher strength to weight ratio these materials are very competitive with the steel shell retrofit technique, and they can be applied with much lighter lifting equipment. The materials are more resistant to corrosion than the steel jackets and they will require very little maintenance. They do not degrade with time as some of the less expensive system do and the durability strength reductions factors are not necessary for the carbon wraps.

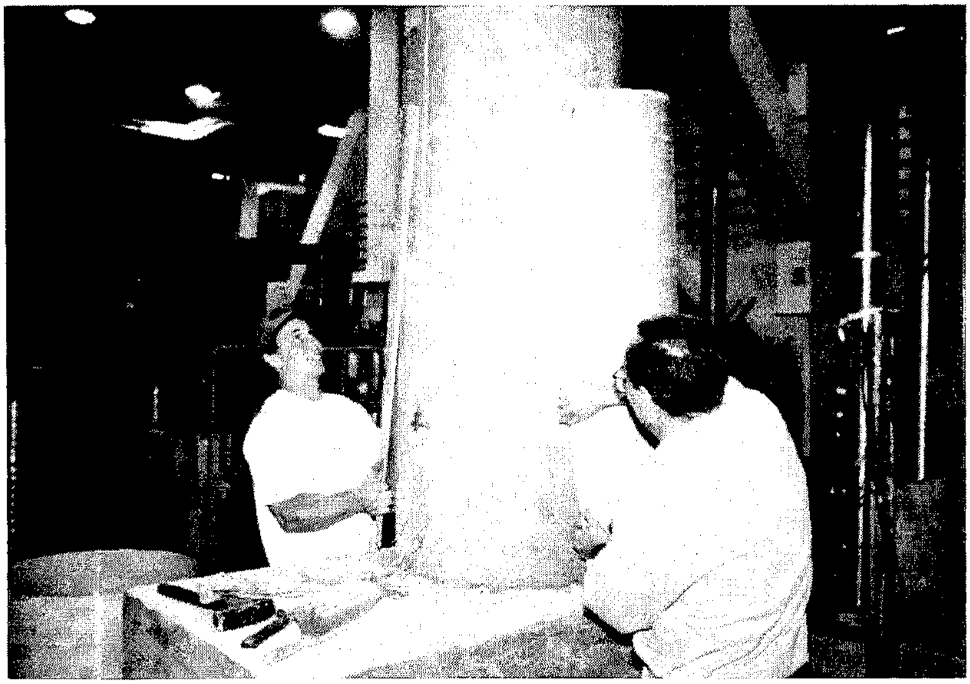

Figure 16 Snaptite System Being Prepared For Testing

Figure 17 illustrates the application of pre-preg carbon fiber wrapping on bridge columns at the field test site. The columns are heat cured under controlled conditions by electrically heated blankets or enclosures. The columns are painted concrete color for aesthetic purposes, but the coating does provide protection against the elements. The thickness can be varied as the ductility requirements dictate. In the field applications on the Santa Monica Freeway the white paint was also used for the same purposes as on the resin impregnated fiberglass wraps. This material has the potential of becoming the most cost effective column wrapping system because of its high strength to weight ratio. The system does not require heavy lifting equipment and can generally compete favorably against the steel shell retrofit systems. Since it is not as labor intensive as some of the other systems being approved, it will ultimately be the system of choice for most contractors. The controlled heat curing system that is used by XXSys provides a material that is very reliable and has the best chance of guaranteeing the same properties as those of the laboratory samples. This relaiability is more difficult to achieve with many of the other systems.

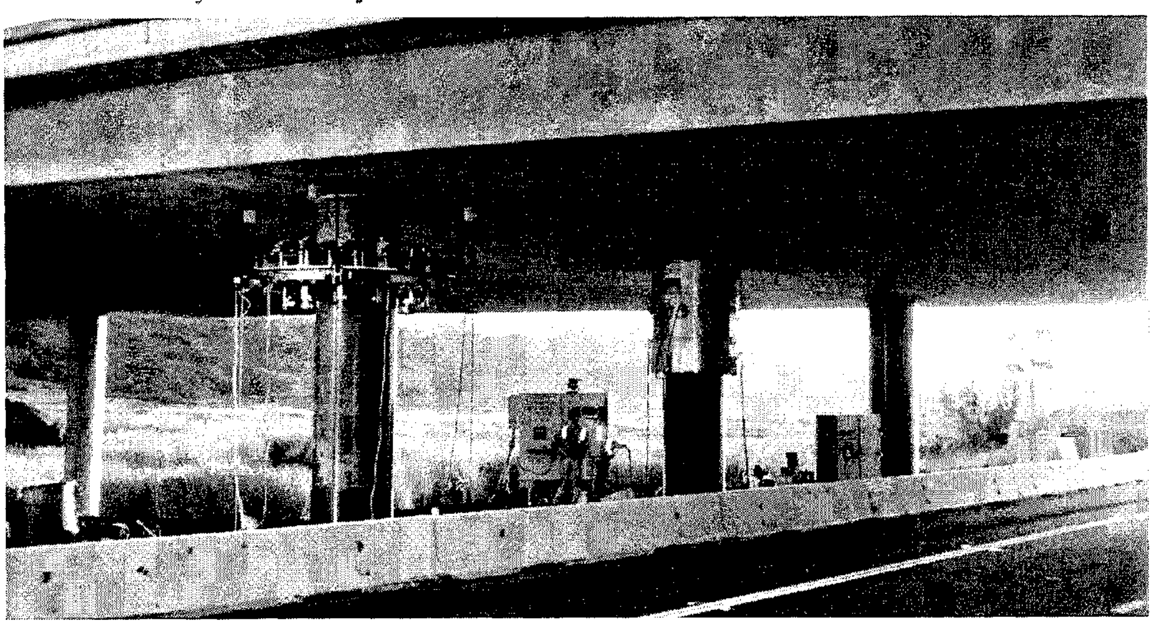

Figure 17 Carbon Fiber Wrapping Machine and Heating Jacket

Strengthening Arch Ribs And Spandrel Columns

Analysis has shown that the typical arch bridge is extremely vulnerable to seismic forces in the transverse direction and, therefore, it is necessary to retrofit strengthen the main ribs and spandrel columns. Getting any type of erection and application equipment into these locations is very costly and difficult so Caltrans has developed retrofit solutions using carbon or fiberglass sheets. Portions of the arch ribs and the joint where spandrel columns frame into the ribs are being reinforced by wrapping with these sheets. Testing has also been conducted at UC San Diego over the past three years to evaluate the various sheet systems. Results show that the confinement and shear capacity increase can be achieved to the levels required. Full scale tests are being conducted during this summer (1997) on the column-rib joint to determine the material thickness needed to provide the required performance in a seismic event. Use of these materials on older arches is also important from a historical preservation perspective because the sheets do not significantly alter the appearance of the bridges. Figure 18 Shows the "Robo Wrapper" applying carbon filaments to a spandrel column mockup in the testing laboratory. At the lower portion where the slope of the main arch prevents the machine from wrapping the sheets will be applied by hand methods.

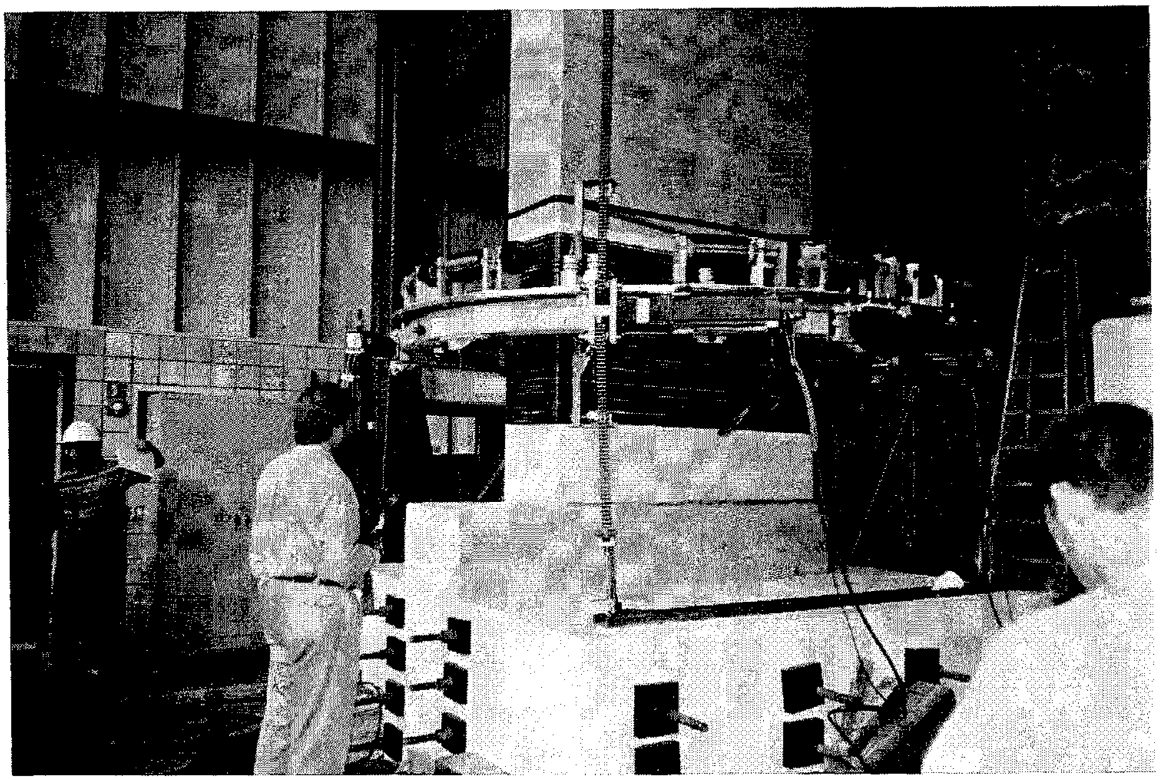

Figure 18 "Robo Wrapper" Applying Carbon Filaments to a Spandrel Column Mockup

Composite Bridge Components

Working in cooperation with the UC San Diego research team and the ARPA and FHWA technology transfer programs Caltrans has been testing other applications of advanced composites in the seismic reinforcing of older bridges and in the construction of major bridge components and ultimately, a complete highway bridge designed for AASHTO loads. The first applications involve resin impregnated fiberglass or carbon sheets on non circular bridge members. These include the use of sheets to wrap and confine the spandrel columns and rib members on several arch bridges and sheet strengthening of bridge deck soffits The second application involves the use of small diameter carbon fiber tubes, constructed by the same technology as rocket bodies, for bridge girders. This application has been tested at the laboratory and design details are being developed for a bridge on the state highway system in southern California. The bridge will include deck units which are composed entirely of advanced composite materials and construction is scheduled for late Fall of 1997. The testing program for these bridge components has been underway at UC San Diego for over three years, under the ARPA grant.

SUMMARY

Following the Loma Prieta and Northridge earthquakes Caltrans engineering staff developed numerous improved seismic design details, based on a comprehensive "problem focused" seismic research and testing program. These design details will appreciably improve the performance of new and retrofitted bridges in future seismic events. After the Northridge event accelerated design and construction procedures were introduced which materially shortened the time to reconstruct the damaged highway system. The use of large incentives for early completion was a key element in that procedure.

Additionally, Caltrans has been working with the composites industry and UC San Diego researchers to develop strengthening, repair and rehabilitation techniques utilizing advanced composite materials which have been previously developed for high performance defense and aerospace applications. These materials are being incorporated into the civil infrastructure repair and rehabilitation projects and will one day become competitive with the traditional and commonly used engineering construction materials. All components and details have been extensively laboratory tested under simulated seismic loading conditions to assure adequate ductility, toughness and shear strength under seismic induced cyclic loading. Ultimately, lightweight and durable bridges will be constructed of these advanced materials to reduce weight and the damaging effects of seismic forces.

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INNOVATIVE DESIGN, CONSTRUCTION AND REHABILITATION METHODS

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ABSTRACT

Since the stock of civil infrastructures has been increasing in volume, it is strongly required to improve quality of public works facilities, maintain them in good conditions effectively and minimize the life cycle cost. This paper mainly describes innovative technologies which are helpful in supporting design, construction and maintenance of civil construction projects related to the Ministry of Construction, namely management supporting system for public facilities, application of computer and network technology in construction industry and performance-based design method. Finally, introduced are the seismic design code of highway bridges revised last year and seismic retrofit of the highways considering the damage caused by the Hyogoken-Nanbu Earthquake, as examples of the recent practices in Japan.

1.INTRODUCTION

Various kinds of infrastructures, namely dams, river and coastal management structures, countermeasures for sediment-related disasters, highways, tunnels and bridges are constructed and maintained by the Ministry of Construction and public works departments in local governments in Japan. Consequently, the Ministry of Construction has developed design criteria and maintenance systems and promoted research and development of advanced construction technology for the related infrastructures. The Public Works Research Institute (PWRI) has been carrying out researches on design, construction and maintenance of the above-mentioned infrastructures. Research on buildings, however, has been conducted by the Building Research Institute (BRI) which also belongs to the Ministry of Construction.

In recent years, public needs such as the preservation of nature and the creation of qualified environment are becoming of great importance for infrastructure construction. On the other hand, gradual changes in socioeconomic organization are resulting in serious restrictions, such as the decrease of technical workers due to the aging society. Furthermore, effectiveness is also strongly required; that is, to reduce cost of construction and maintenance of public facilities as well as the number of staff in public sectors and also to assure and improve quality of public works facilities. These circumstances require innovative technologies in design, construction and rehabilitation methods.

2.PUBLIC FACILITY MANAGEMENT SYSTEM

Developed countries have built up a large scale of their infrastructures, and governmental engineers have become aware of the importance of maintenance and management of them. It is difficult to expect high economic growth in future, a substantial increase of budget and an increase in the number of engineers in public sectors. It is necessary to carry out cost-effective and systematic management with a limited amount of budget and a limited number of engineers.

The concept of life cycle cost has been applied only to management of road pavement and some others. A few years ago, the Ministry of Construction conducted preliminary study on estimation of life cycle cost of the public facilities which the Ministry of Construction manages to construct and maintain, namely bridges, tunnels, water gates, coastal embankments and sewage pipelines. There were some difficulties in the study as follows;

1) There are only few data available on the behavior of long-term deterioration of the structures and also improvement of healthiness by repair work.

2) There are many causes of deterioration and also many kinds of repair methods so that it is difficult to make suitable and precise simulation models.

In case of road bridges in Japan, the stock has been increasing in volume. There are 650,000 bridges longer than 2m, 120,000 of which are longer than 15m. Fig.-1 shows the number of these bridges constructed in each 5 year period. The number of road bridges constructed immediately before and after World War II was significantly small, but the number increased rapidly for the period from the middle 1950's. The number of bridges constructed, which reached the peak of 4,500 bridges in 1975, has decreased since then and average number of new bridges per year is approximately 2,000. Of many existing bridges, those that have been in service more than 50 years, generally considered to be the service life of a road bridge, number approximately 7,300, or nearly 6% of the total number of bridges. If new bridges are constructed and old bridges are replaced at this pace, it can be estimated that this proportion will change significantly before 2001, but the proportion will increase to 12%, when bridges constructed after 1950's reach their service life, and will reach nearly 30% in 2011, as aging rapidly accelerates. It is considered necessary to prolong the life of existing bridges with proper and full maintenance, and also to make increased efforts to ensure highly reliable and durable designs for bridges to be constructed in the future.

On the other hand, the number of staff of the Ministry of Construction, concerned with infrastructure management, has been reduced from 33,000 persons in 1977 to 23,700 persons in 1997. In addition, increase of traffic volume and enlargement of vehicle size require frequent maintenance of highway bridges.

Inspection techniques, evaluation techniques and maintenance techniques have been improved and manuals for these have been provided by each highway administration organization. Then, systematic and cost-effective management requires computerized support system. Several countries already developed and have been using their own bridge management systems[BMS].

The PWRI started to develop a bridge management system in 1990[1]. It is necessary to maintain bridges in good service conditions, to allow for free flow of road users and goods; and, as a consequence, to take part in the economic activity of the region concerned. Knowing their exact conditions, recognizing forewarning signs of distress, periodical bridge inspection is essential. All distresses identified in due time can avoid severe accidents and high repair costs. In addition, it has to be noted that bridges represent an expensive asset; it is therefore necessary to optimize bridge related investments, in particular, through a sound maintenance. The ultimate purpose of the BMS is to minimize the life cycle cost of bridges.

The BMS seeks the most effective rehabilitation plan taking account of financial constraints. The BMS consists of two program modules, which are the bridge condition evaluating module and the bridge rehabilitation planning module. The flow chart of the BMS is shown in Fig.-2.

In developing a BMS, there are several "gray" area, e.g. cause of distress, remaining life, sensitivity of visible distresses, environmental effects, inspection techniques, rehabilitation and retrofitting techniques and their efficiencies etc. which require special research efforts.

3.ADVANCED CONSTRUCTION TECHNOLOGIES USING CALS

The idea of CALS has been changing over the last ten years due to industry advancement in technology and society's increased use. Now it is recognized that CALS means "Computer-aided Acquisition and Life-cycle Support" or "Commerce At Light Speed". CALS was started for logistic support in U.S.A. in 1980's and this idea can be introduced in construction industry. The Ministry of Construction organized a task committee to study application of CALS in public construction.

CALS is a system for integration and sharing via intercompany networks of computer-based information of various kinds on the entire life cycle of products, from design to manufacture, distribution, and maintenance. CALS can realize easy exchange, holding and utilization of standardized information and data in related organizations.

In case of public works, the following features are pointed out from a viewpoint of information-oriented construction technology.

a) There are many organizations, such as owner, designer, contractor, material provider and so on, who join a construction or maintenance project and they need frequent exchange of information.

b) The exchanged information features a variety of information, massive in volume, containing reference documents, drawings, photographs and calculations.

c) The life of the structures is generally so long that they require long-term maintenance and repair and this means that the information on structures is quite important to support management of the structures.

It is, therefore, expected that CALS can realize the following when it is introduced in construction and maintenance of public works.

1) sound and fast procurement and transaction

2) exchange of information without any restriction of time and place

3) speedup of office work
4) smooth execution of public construction projects
5) support life cycle of public construction projects and infrastructures

Then these improvement may result in ensuring and improving productivity, economic efficiency and quality of public works.

The Ministry of Construction formulated "Basic plan for improvement of execution procedure of public works projects based on CALS". This plan proposed short-, medium- and long-term objectives in the coming 15 years.

The Public Works Research Institute and the Building Research Institute in cooperation with construction companies and consulting firms have been conducting a research project for four year period extending from 1995 to 1998[2]. The purpose of this project is to conduct ways of using CALS to upgrade construction projects in public works by integrating of data base, sharing data, automating operations, improving productivity and reducing the time and costs for construction. This research deals with four themes as shown in Fig.-3.

4.PERFORMANCE-BASED DESIGN

Recently it is indicated that design standard of structures and the criteria for quality control of construction should be shifted to performance-based standards that define the performance required for a structure[3,4].

The current design codes specify design procedures and rules definitely and prescriptively as code provisions.

There are some merits in this type of design codes. Namely, designer may not make mistakes in designing when he follows the procedures and rules specified in the design codes and owner can rely on the quality of the design and the structure as far as the codes are concerned. But this may cause unadaptable design. For example, because the current structural codes of buildings in Japan are regulated for conventional materials and structural methods, the codes cannot match the new materials and new structural methods. In addition, the current design codes do not require to define target structural performance.

The current design methods are based on assessment of either strength or ductility, or otherwise the both of them in order to mainly assure safety of structure itself and human lives. The performance of a structure is not limited to structural safety. The performance should be defined in terms of safety, reliability, durability, aesthetics and so on. After the Kobe Earthquake, people realized clearly that there must be several levels of performance of structures. For instance, it is not easily accepted for some residents that nonstructural walls of their houses could be damaged by severe earthquakes, even though it is a matter of course for designer. Some are interested in the cost and duration of repair work of damage.

It is suggested that performance-based design will be able to improve the current design codes as follows;

1) clarify the performance of a structure to be designed and make a design by mutual consent between owner and designer
2) enlarge possible selection of materials, types of structure and construction methods, and it results in cost reduction.
3) adopt new technology easily and promote progress of new construction technology
4) enable to correspond to new movement in other countries and international organizations such as ISO (International Organization for Standardization).

Some of the seismic design codes revised after the Hyogoken-Nanbu Earthquake have introduced partially the concept of performance-based design. However it seems the concept has not been fixed yet.

Three year research project on performance-based design/engineering system for building structures was initiated in 1995 by BRI in cooperation with many professional communities[3,4]. In this research project, the following two technical subjects will be studied.

1) Develop performance based structural design system
2) Convert specification codes into performance codes in Building Standard Law.

In addition, a social subject will be also studied.

3) Propose social system such as qualification of structural engineers, building confirmation procedure in Building Standard Law and so on for performance-based design system.

The PWRI started in 1997 four year research project on developing a total quality control system covering everything from setting new criteria for performance definitions, design and construction, to inspections and maintenance.

In order to adopt the performance-based seismic design method, technical issues as follows must be solved.

1) Basic conditions for performance-based seismic design

Most fundamental concept of the performance-based seismic design should be determined, such as:

a) Required (target) [seismic] performance. (What should be the ultimate purpose of seismic design? Is it to save human lives, to guarantee the proper service of structures after an earthquake, or something else?)

b) Seismic design forces and effects(ie. input motions. What kind and level of input motions and earthquakes should be considered?)

c) Seismic performance. / Performance during an earthquake (What condition should be satisfied to realize the fundamental concept mentioned above? It should not be destroyed, it should be easy to restore, its essential facilities should function properly, it should be highly durable, etc.)

d) Coordination of performance level of different kinds of structures. (Different kinds of structures are designed according to their own design codes which usually have been developed separately. Their performance should be coordinated and harmonized. i.e. seismic performance of highways, railways and lifelines in urban areas.)

= In case of buildings most of which are privately owned, an owner and a designer can come to an agreement on the performance of a building to be designed and constructed. On the contrary, in case of public facilities such as infrastructures concerned, it is not an easy task to make an agreement about these conditions among the wide spectrum of people.

2) Determination of the detail of the required seismic conditions.

In order to specify the required seismic performance, it is essential to seize the social demand for the seismic design level of various structures by conducting research on:

a) Comparison of the required seismic performance with that of a structure designed according to the current design codes.

b) Comparison of hazard due to earthquake, with hazards due to other causes.

c) How the durability and life cycle cost should be prioritized in seismic design.

3) Conditions required for the society

In order for the society to accept the new concept, that is the performance-based technologies, following issues should be considered.

a) Methods and systems to control the quality of both design and construction processes.

b) Who should take responsibility, when insufficiency of the quality of a structure is found or some accident due to such insufficient quality occurs.

c) How to cope with the current shortage of engineers.

d) How to cope with the problem which is beyond the current [seismic design] technology.

e) Promotion of legislation.

5.SEISMIC DESIGN CRITERIA OF HIGHWAY BRIDGES[5]

The Hyogoken-Nanbu Earthquake occurred in January 1995 caused destructive damage to highway bridges. The Earthquake revealed that there are a number of critical issues to be revised in the seismic design and seismic strengthening of bridges in urban areas.

Highway bridges are generally designed in accordance with the Specification for Design of Highway Bridges issued by the Ministry of Construction in Japan. Basically the specification has been using allowable stress method. Although the checking method of the ductility, namely the check of the horizontal bearing capacity, was introduced in the specifications of 1990, it was applied only for the single-column concrete column.

The Ministry of Construction released the new Design Specifications of Highway Bridges in November, 1996. There are various major revisions in the new Specifications, for example, seismic design forces were upgraded and design procedures were changed from allowable stress method to ductility design method (lateral capacity method).

(1) Seismic Performance

Table-l shows a classification of the seismic performance levels and design methods. Bridges are categorized into two groups depending on their importance; standard bridges (Type-A bridges) and important bridges (Type-B bridges). Seismic performance levels depend on the importance of bridges. For moderate ground motions induced by earthquakes with high probability to occur, both A and B bridges should behave in an elastic manner without any essential structural damage. For extreme ground motions induced by earthquakes with low probability to occur, the Type-A bridges should prevent critical failure, while the Type-B bridges should perform with limited damage .

(2) Design Methods

Bridges are designed by both the Seismic Coefficient Method and the Ductility Design Method. In the Seismic Coefficient Method, a lateral force coefficient ranging from 0.2 to 0.3 has been used based on the allowable stress design approach. In the Ductility Design Method, assuming a principle plastic hinge formed at the bottom of piers as shown in Fig.-4(a) and the equal energy assumption, lateral capacity of a pier should be clarified to be larger than the equivalent lateral seismic forces obtained by multiplying equivalent lateral force coefficient by equivalent weight, considering allowable displacement ductility factor of a pier. In the Type-B bridges, residual displacement developed at a pier after an earthquake must be checked. In a bridge with complex dynamic response, the dynamic response analysis is required to check the safety of a bridge after it is designed by the Seismic Coefficient Method and the Ductility Design Method. In the seismic design of a foundation, a lateral force equivalent to the ultimate lateral capacity of a pier is generally assumed to be a design force, because damage at a foundation is undesirable from a viewpoint of repair. If a pier has large lateral capacity compared with the lateral seismic force, the foundation is designed assuming a plastic hinge at the foundation and surrounding soils as shown in Fig.-4(c).

(3) Seismic Ground Motions

In the Ductility Design Method, two types of ground motions must be considered. The first is the ground motion which could be induced by inter-plate earthquakes with magnitude of about 8 at a distance of about 50 km from the faults in the ocean area. Ground motions which developed at Tokyo during the 1923 Kanto Earthquake are typical of this type of ground motions. The second is the ground motion developed in intra-plate earthquakes with magnitude of about 7-7.2 at very short distances from their epicenters in the inland area. Obviously the ground motions developed at Kobe during the Hyogoken-Nanbu Earthquake are typical target of this type of ground motions. The first and the second ground motions are called as Type-I and Type-II ground motions, respectively(Fig.-5). The recurrence time of the Type-II ground motion is much longer that that of the Type-I ground motion, although the estimation is very difficult.

6.SEISMIC RETROFIT OF HIGHWAY BRIDGES[5]

(1) Seismic Retrofit prior to the Hyogoken-Nanbu Earthquake

The Ministry of Construction conducted seismic evaluation of highway bridges 5 times throughout the country since 1971. The latest evaluation was made in 1991. Approximately 60,000 bridges in total were evaluated and approximately 18,000 bridges of which were found to require retrofit. Through a series of seismic retrofit works, approximately 32,000 bridges were retrofitted by the end of 1994. Because collapse of bridges tends to be developed due to the excessive relative movement between the superstructure and the substructure, and failure of substructures associated with inadequate strength, the evaluation was made based on both the relative movement and the strength of substructure. Emphasis had been placed to install the unseating prevention devices. That is because it becomes important to promote the strengthening of substructures with in adequate strength and lateral stiffness after installation of the unseating prevention devices.

(2) Seismic Retrofit after the Hyogoken-Nanbu Earthquake

Because the damage concentrated to the single reinforced concrete piers/columns with small concrete section, the seismic retrofit program has initiated for those columns, which were designed according to the pre-1980 Design Specifications at important bridges such as bridges on expressways, urban expressways, and designated highway bridges, and also double-deckers and overcrossing and so on which significantly affect highway function once damaged. Unseating prevention devices also should be installed for these extremely important bridges.

Main purpose of the seismic retrofit of reinforced concrete columns is to increase their shear strength, in particular the pier with termination of longitudinal reinforcements without enough anchoring length. This increases ductility of the column so that premature shear failure could be avoided. Furthermore, the flexural strength should be also adjusted in order to restrict residual displacement of the pier and not to increase the seismic forces transferred from piers to foundations. For such requirements, seismic strengthening by Steel Jacketing with Flexural Strength Controlled was suggested. This uses steel jacket surrounding the existing column as shown in Fig.-6.

(3) Prioritization

The 3 year retrofit program will be completed in 1997 fiscal year. In the program, the single reinforced concrete piers/columns with small concrete section which were designed according to the pre-1980 Design Specifications on important highways have been evaluated and retrofitted and the other bridges with wall-type piers, steel piers, frame piers and so on as well as the bridges on the other highways should be evaluated and retrofitted if required in the next retrofit program. Since there are approximately 200,000 piers, it is required to develop the prioritization methods and the evaluation methods of the vulnerability for the intentional retrofit program.

Fig.-7 shows the simple flow chart to give the prioritization of the retrofit works to bridges. The importance of highway, structural factor, members vulnerability (reinforced concrete piers, steel piers, unseating prevention devices, foundation) are the factors to be considered for the prioritization.

(4) Seismic Retrofit using New Materials

The retrofit work is often restricted because of the limited construction space under the condition to open the public traffic in particular for the seismic retrofit of highway bridges in urban areas. Therefore, there are sites where the conventional steel jacketing and reinforced concrete jacketing methods is difficult to apply. New materials such as carbon fiber sheets and aramid fiber sheets are attractive to be applied for the seismic retrofit of such bridges with construction restriction as shown in Fig.-8. Since new materials such as fiber sheets are very light, workability is excellent with no machines using epoxy resin as glue bond.

7.CONCLUDING REMARKS

There are many kinds of civil infrastructures. In addition, there are many issues related to design, construction and rehabilitation methods. This paper mainly describes the technical subjects which are discussed in the MOC. The stock of public facilities has been increasing. It is required to maintain them in good conditions effectively under the restricted situations such as the limited budget and the limited number of staff. Further research efforts are needed .

1) Estimation of life cycle cost of infrastructures
   cause of distress, environmental effects and durability, remaining life
2) Management supporting systems
   inspection and evaluation techniques, prioritization, rehabilitation and retrofitting techniques
3) Construction technology using computer and network technology
   information integration
4) Performance-based design method.
   clarification of performance of infrastructure and system, methods and system to control the quality
5) Seismic design and construction
   seismic design forces and effects, structural control, seismic retrofit using new materials,
   sensor technologies, self-diagnosis materials, self-repairble materials

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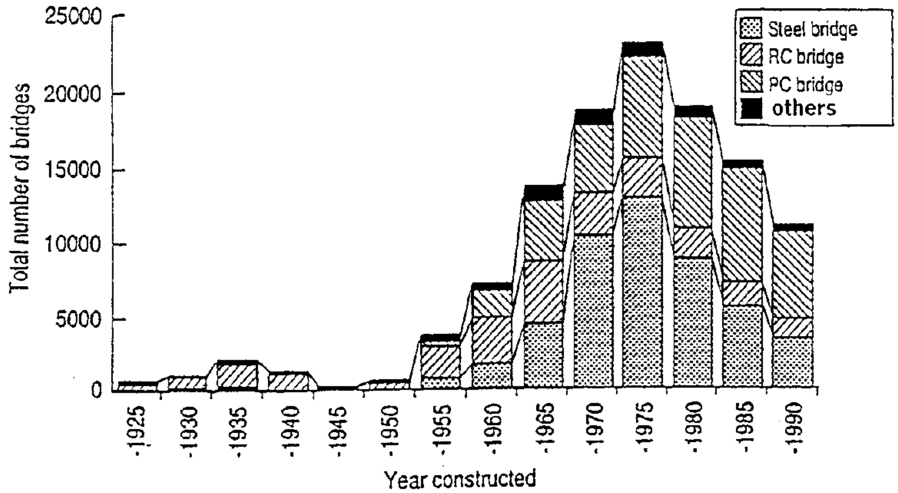


Fig.-1 Number of Bridges by Year Constructed(as of April 1, 1991)

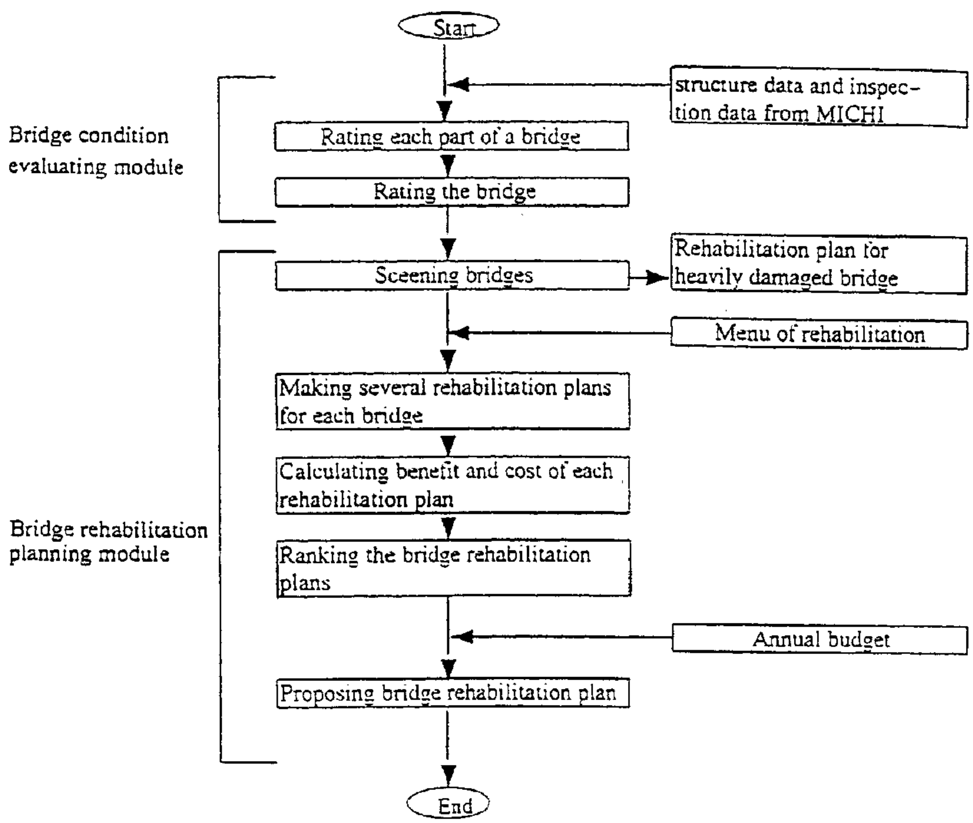


Fig.-2 Flow Chart of the BMS[1]

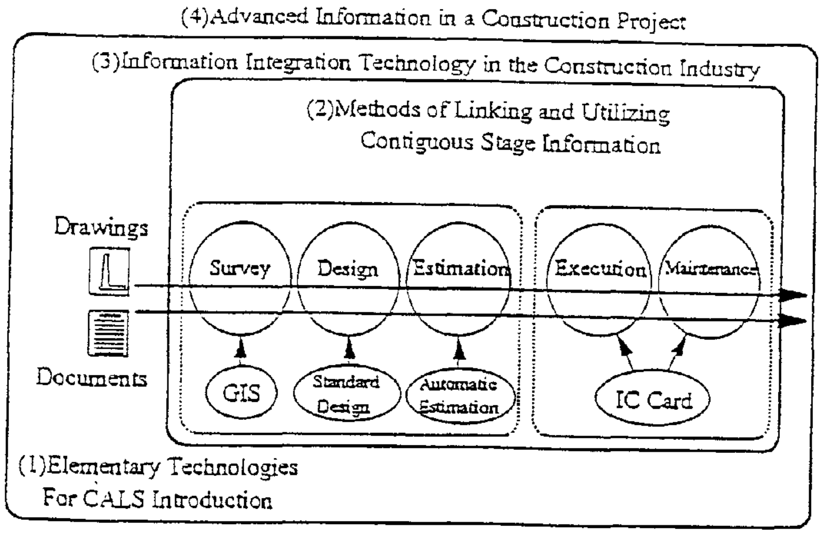
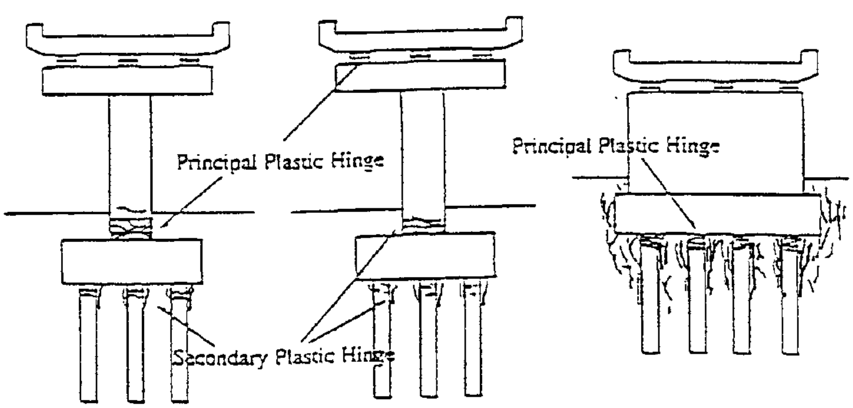


Fig.-3 Project Research Items[2]

Table 1 Seismic Performance Levels

Type of Design Ground Motions		Importance of Bridges: Type-A (Standard Bridges)	Importance of Bridges: Type-B (Important Bridges)	Design Methods: Equivalent Static Lateral Force Methods	Design Methods: Dynamic Analysis
Ground Motions with High Probability to Occur		Prevent Damage		Seismic Coefficient Method	Step by Step Analysis or Response Spectrum Analysis
Ground Motions with Low Probability to Occur	Type-I (Plate Boundary Earthquakes)	Prevent Critical Damage	Limited Damage	Ductility Design Method	
	Type-II (Inland Earthquakes)				



(a) Conventional Design (b) Menshin Design (c) Bridge Supported by A Wall-type Pier

Fig.-4 Location of Primary Plastic Hinge

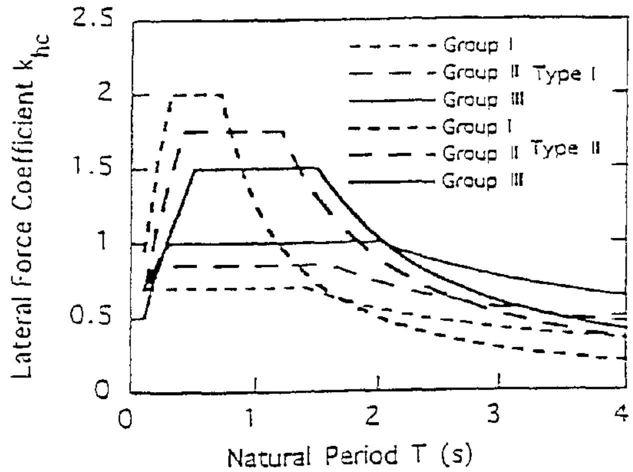


Fig.-5 Type I and Type II Lateral Force Coefficients in the Ductility Design Method

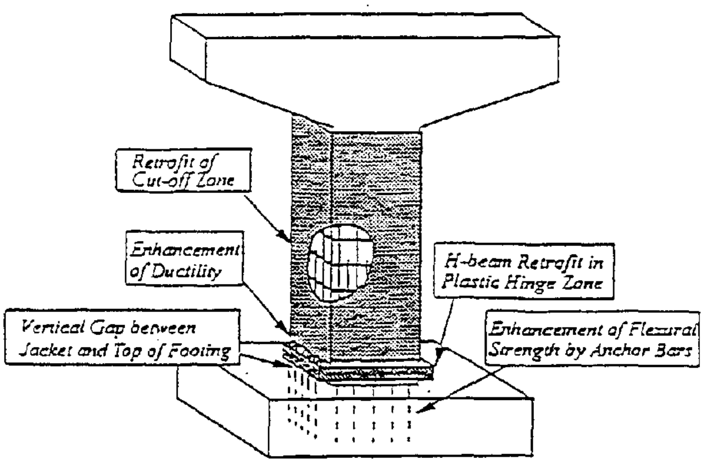


Fig.-6 Seismic Retrofit of Reinforced Concrete Piers by Steel Jacket with Controlled Increase of Flexural Strength

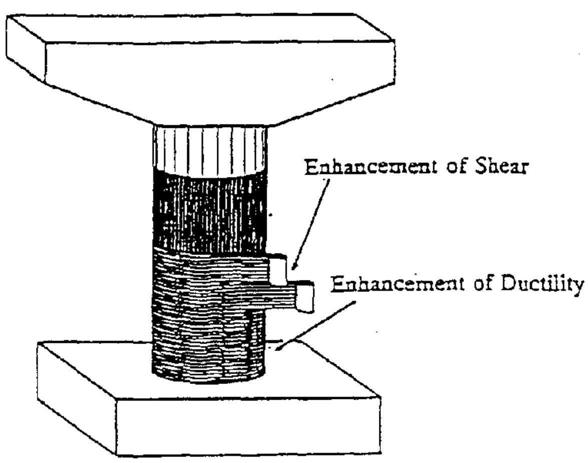


Fig.-8 Application of New Materilas for Seismic Retrofit of Reinforced Column

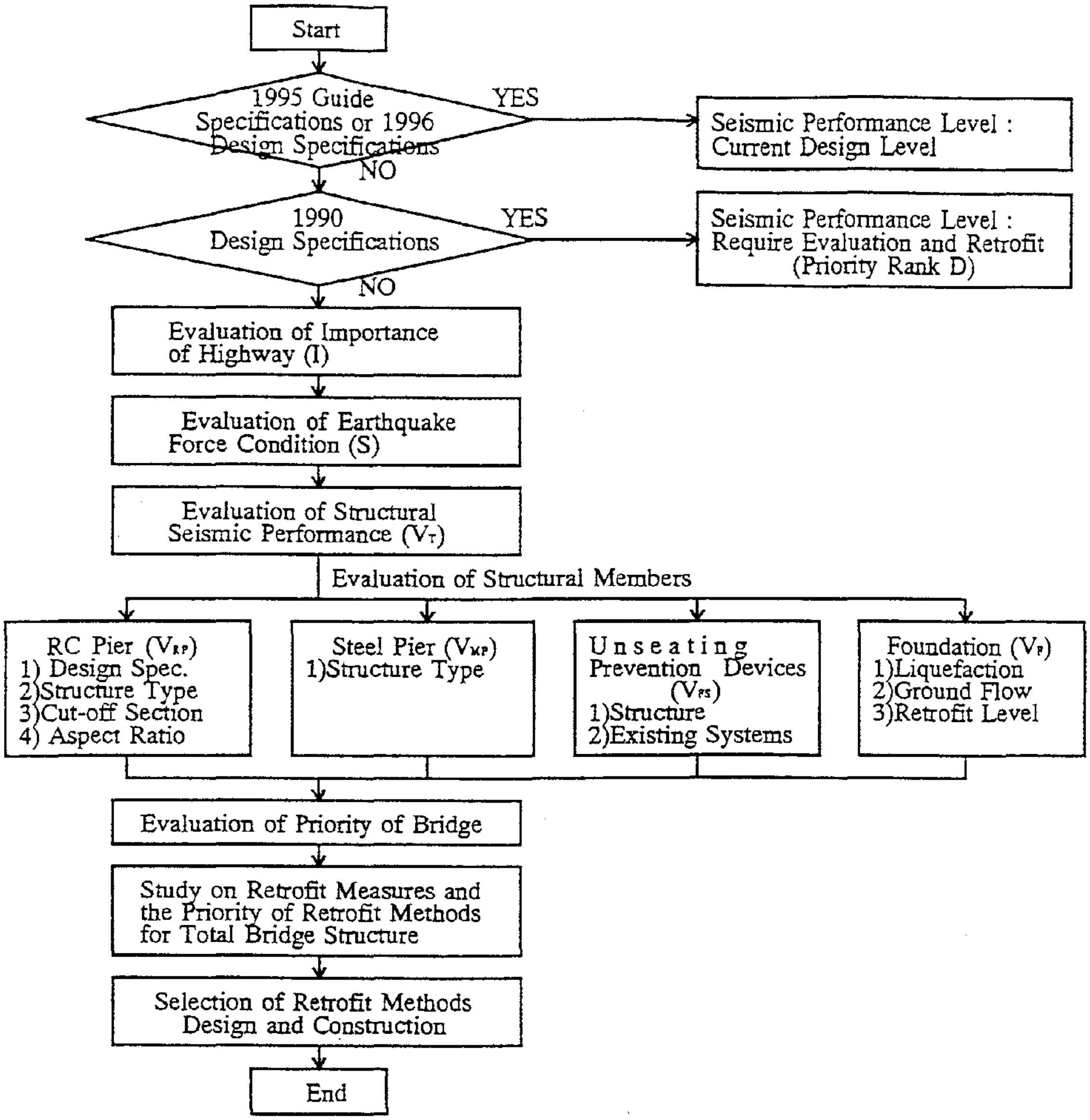


Fig.-7 Prioritization Concept of Seismic Retrofit of Highway Bridges

INFRASTRUCTURE LIFE CYCLE COST ANALYSIS: DIRECT AND INDIRECT USER COSTS OF NATURAL HAZARDS

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INTRODUCTION

In recent years, the concept of life cycle cost has been gaining attention in the civil infrastructure systems field, particularly in the areas of bridge and highway management. That is, it is becoming increasingly recognized that infrastructure investment alternatives should consider not just initial construction costs but also all other expected costs (discounted to the present value) following construction to the end of the structure's life span. The proper accounting of expected costs, however, remains controversial.

In particular, the concept of *infrastructure user costs* deserves further study. For an individual structure such as a building, it can be argued that *owner (or investor) costs* are relevant, including such items as construction, maintenance, and eventual demolition. The situation for infrastructure systems differs, in that highway, electric power, water delivery, and other such systems are primarily public investments that provide vital services to entire urban areas. Thus the costs accruing to the users of the infrastructure systems, i.e., the public, should also be accounted for. Special care, however, should be exercised when considering infrastructure investments for reduction of natural hazard risk, where mitigation expenditures represent not only an additional initial owner and user cost but also may lead to substantial future savings in terms of human, property, and economic losses avoided.

This paper explains how infrastructure user costs may be incorporated into life cycle cost analysis methodologies for infrastructure management. Section II presents a basic framework and illustrative example. Section III discusses how the user cost concept can be further extended and enumerates possible approaches and considerations. Section IV identifies a number of areas for further research.

LIFE CYCLE COST ANALYSIS FRAMEWORK

Chang and Shinozuka (1996) propose that total life cycle costs associated with an infrastructure system can be classified into four principal types: planned and unplanned costs accruing to the system owner (C_1 and C_3, respectively), and associated user costs for the two categories of owner costs (C_2 and C_4, respectively). For example, planned owner costs include expenditures for system initial construction (C_I) and maintenance (C_M), as well as seismic retrofit costs (C_S). For analysis involving an existing system, construction costs would be omitted (since they are "sunk" costs). Unplanned costs here refer to the consequences of natural disaster, specifically earthquakes. User costs pertain to the economic loss imposed on users from disruption of infrastructure services. Total life cycle cost C is the sum of costs in each time period t discounted to the present value by a factor z that is based on a discount rate i indicating the expected market rate of return on an investment. Annual maintenance costs m at time t depend upon the physical properties x_p of the system element (e.g., deterioration rates and associated maintenance needs may vary according to pipe material, corrosiveness of local soil conditions, etc.). Thus total life cycle costs can be summarized as:

$$C = C_1 + C_2 + C_3 + C_4 \tag{1}$$

and planned costs as:

$$C_1 = C_I + C_M + C_S \tag{2}$$

$$C_{M\acute{E}} = \Sigma_t \Sigma_p\, m_p(t,x_p)\cdot z(t) \tag{3}$$

$$\text{where} \quad z(t) = (1+i)^{-t} \tag{4}$$

$$C_S = \Sigma_t \Sigma_p\, s_p(t,x_p)\cdot z(t) \tag{5}$$

Note that this differs from traditional life cycle cost analysis, where only planned owner costs (C_1) would be considered in equation (1) and seismic retrofit costs would be omitted in equation (2).

Unplanned costs accruing to infrastructure owners (C_3) associated with damage from earthquakes is expressed in probabilistic terms to reflect the local seismic hazard. Expected repair costs associated with damage in an earthquake derived from a probabilistic condition/performance index G_p evaluated for and summed over each element p, damage state k, and time period t, multiplied by the associated unit repair cost r for the damage state and discounted to the present value.

$$C_3 = \Sigma_t \Sigma_p \Sigma_k\, G_p(x_p, d_k, t)\cdot r_k \cdot z(t) \tag{6}$$

The element condition/performance index G_p is specified as a probabilistic factor that changes over time due to natural deterioration, as well as the mitigative (condition-improving) effects of maintenance and seismic retrofit activity. That is, in the initial time period, the index represents a convolution of the local hazard curve ($Pr(h)$) with the element's fragility curve ($Pr(d_k|h)$) for each damage state d_k over all levels of the hazard h, measured for instance in terms of peak ground acceleration (PGA). In subsequent time periods, this index increases or decreases through a natural deterioration factor (w_n), maintenance factor (w_m), and seismic retrofit factor (w_s) that represent changes in the element's physical condition.

$$\begin{aligned} G_p(x_p, d_k, t) &= Pr(d_k | x_p, t) \\ &= G_p(x_p, d_k, t-1)\cdot\{1 + w_n(x_p,t) - w_m(m_p(x_p,t)) - w_s(s_p(x_p,t))\} \end{aligned} \tag{7}$$

where

$$\begin{aligned} G_p(x_p, d_k, 0) &= Pr(d_k | x_p, t=0) \\ &= \textstyle\int_h Pr(d_k | x_p, h)\cdot Pr(h) \end{aligned} \tag{8}$$

The formulation of user costs associated with planned and unplanned owner costs (C_2 and C_4, respectively), depends to some extent on the type of infrastructure system being considered, as discussed further in Section III. For systems such as transportation, where an individual system element such a bridge can be related in a meaningful fashion to its users, i.e. vehicles/occupants crossing it, the user cost formulation can be related to a unit user cost for the bridge (u_p) which is specific to that bridge and represents the maximum user cost associated with temporary closure of the facility. The actual user costs associated with regularly recurring maintenance work as well as one-time seismic retrofit respectively will depend on the extent of service disruption (b_m, b_s) and the duration of this disruption (t_m, t_s):

$$C_2 = \Sigma_t\, [t_m\cdot b_m\cdot u_p(t)\cdot z(t) + t_s\cdot b_s\cdot u_p(t)\cdot z(t)] \tag{9a}$$

For other systems such as water delivery, an individual element such as a pipe cannot be meaningfully related to a specific group of users. Here, user costs can be related to a unit user cost for a specific *service area* (u_a) where level of service is determined by conditions in the entire system, rather than an individual pipe. In the case of certain lifelines such as electric power, service areas can be naturally defined in terms of the customers served by a particular substation. For water delivery systems, census tracts or larger agglomerations may represent meaningful service areas. In addition, functional disruption of an individual pipe (e.g., a transmission conduit) can affect users in many different service areas. Thus,

$$C_2 = \sum_t \sum_a [t_m \cdot b_m \cdot u_a(t) \cdot z(t) + t_s \cdot b_s \cdot u_a(t) \cdot z(t)] \tag{9b}$$

In contrast to the owner costs associated with earthquake damage, the user costs C_4 are related to the resulting infrastructure service disruption, rather than the amount of repairs that need to be made. User costs are modeled in relation to a performance index at the system element level (G_p) or service area level (G_a) depending on the type of system as mentioned earlier. For a bridge, the associated unplanned user costs may be formulated as:

$$C_4 = \sum_t \sum_k G_p(x, d_k, t) \cdot [b_d(d_k) \cdot t_d(d_k) \cdot u_p(t)] \cdot z(t) \tag{10a}$$

where b_d ($0 \leq b_d \leq 1$) is the disruption factor associated with the kth damage state for the bridge.

For a water delivery system, areal performance levels L_j analogous to structural damage states d_k can be defined in terms of the level of service provided in each service area; for example, the percentage of customers with electric power supply.

$$C_4 = \sum_t \sum_j [G_a(L_j, t) \cdot b_d(j) \cdot t_d(j) \cdot u_a(t)] \cdot z(t) \tag{10b}$$

where

$$G_a(L_j, t) = Pr(L_j \,/\, t) \tag{11}$$

Equations (1) through (11) above describe the estimation framework for life cycle costs. They enable us to optimize total life cycle costs C over a specific set of parameter values.

As an illustration, Chang and Shinozuka (1996) evaluate a hypothetical bridge in an area of high seismicity. Results are reproduced in Table 1 for the case with and without seismic retrofit:

Table 1. Life Cycle Costs for a Hypothetical Bridge

Type of Cost	Costs Without Retrofit ($ units)	Costs with Retrofit ($ units)
C_1 (owner, planned)	1,024	1,038
Initial	*1,000*	*1,000*
Maintenance	*24*	*24*
Retrofit	-	*14*
C_2 (user, planned)	100	100
C_3 (owner, unplanned)	165	82
C_4 (user, unplanned)	82	41
Total costs	1,371	1,261

These results illustrate that while retrofit adds to the planned owner costs, they also substantially reduce unplanned costs to both the bridge owner and users. For this particular example, user costs included only traffic delay costs to vehicles crossing the bridge and are meant to be representative of a heavily used urban bridge.

EXTENDING USER COST

The Concept of Indirect Effects

Analysis of user cost in the literature thus far pertain only to those directly affected. However, modern economies are characterized by a high degree of interdependence arising from the need for highly-processed inputs, which require a large number of suppliers; fine-tuned production techniques, which limit substitution possibilities; and a market orientation, which means little of what we consume is homemade. Thus, an impact on one part of the economy ripples through the rest of it. A factory shutdown, arising from say a bridge outage, means orders from its direct suppliers are terminated, which in turn reduces "upstream" orders from successive rounds of other suppliers. The factory shutdown also reduces the supply to direct customers, who then will reduce "downstream" shipments to successive rounds of other customers. The sum of these "indirect" effects are some multiple of the original factory production loss, hence the term "multiplier" effect. Moreover, the multiplier effects are not limited to the interindustry interactions just mentioned, but also extend to household business interactions. When a factory closes, its employees no longer receive paychecks, and its owners no longer receive profits (or dividends). This translates into direct cutbacks in consumption, which means still further rounds of ripple effects of lost production and even more lost income "induced" by the original loss of income from the factory shutdown.[1]

Moreover, some infrastructure disruptions do not lead to direct losses of production *per se*, but to increased costs, typically arising from lower productivity. Examples are delays due to a damaged transportation system and reduction in product quality due to overheating of a process as electricity or water used for cooling is curtailed. A multiplier effect arises here as well in terms of interindustry cost-push inflation, i.e., an increase in the cost of one product raises the cost of all products for which it is an input, these products then raise the costs of all products for which they are inputs, etc. High-priced goods have the effect of lowering consumer purchasing power, which then sets off a chain of output (synonymous with production or sales) multiplier effects as discussed previously.[2]

Estimating Indirect Effects

These successive rounds of multiplier effects would appear to be rather tedious to compute, but several types of economic models can readily do so. The most basic approach is input-output (I-O) analysis, which refers to a linear model of all purchases and sales between sectors of an economy, based on the physical relations of production. A more advanced approach is computable general equilibrium (CGE) modeling, which captures the optimizing behavior of individual decision-making units in response to price changes and subject to resource constraints. I-O and CGE models have been widely used to measure the economy-wide impacts of natural hazards in general (see, e.g., Cochrane, 1974; Brookshire and McKee, 1992) and with respect to infrastructure systems in particular (see, e.g., Gordon and Richardson, 1996; Rose et al., 1997).[3]

The basic input into the calculation of indirect effects is a measure of direct cost increases, production increases, or consumption decreases. However, care must be taken to include only appropriate portions of direct effects and to avoid double-counting. The reason is that some direct effects do not have multiplier consequences, while, in some cases, counting both consumption and production losses results in a duplication.

These considerations can be clarified with the aid of Table 2, which enumerates direct and indirect effects for two categories of infrastructure: transportation, such as bridges and highways, and utilities, such as water and electricity. The table readily reveals that some categories of impacts may not be applicable for all types of infrastructure. Moreover, the relative magnitudes of impact components will vary as well.

For example, commuters suffer damage from transportation outages through loss of time (economists believe that even leisure time has a value); however, there are no multiplier effects associated with this activity. On the other hand, the loss of productivity to producers or transportation companies results in cost increases that do have price multiplier effects first and then output multiplier effects subsequently.[4]

For the case of utility disruptions, the direct effects on consumers are labeled an "inconvenience" as opposed to a delay, but here also there are no multiplier effects. On the production side, there are likely to be productivity losses due to down-time or declines in product quality.

Table 2. User Costs Associated With Infrastructure Disruptions

Bridges/Highways	Water/Electricity
<u>Direct User Costs</u>	<u>Direct User Costs</u>
Delays (congestion costs)	Inconvenience
consumers/commuters (loss of time)	consumers (inconvenience)
Producers/transporters (loss of productivity)	producers (loss of productivity)
Reduced Activity	Reduced Activity
consumers (loss of consumption)	--
producers (loss of production/sales)	producers (loss of production)
Increased Risk	Increased Risk
property damage	property damage
morbidity/mortality	morbidity/mortality
<u>Indirect User Costs</u>	<u>Indirect User Costs</u>
Price multiplier effects from lower productivity	Price multiplier effects from lower productivity
Output multiplier effects	Output multiplier effects
lower consumption	--
lower production	lower production

Consumers may also curtail their shopping trips due to bridge or highway outages. These decreases in direct consumption then feed into an economy-wide models to determine the indirect effects. Note the magnitude of the direct shopping losses are likely to be small in relation to congestion costs. Also, some shopping is not so much curtailed as simply delayed to another date, a case for which the cost is only waiting time, also not subject to multiplier effects.

Businesses will also have a decrease in economic activity due to the inability of employees to get to the job site or the inability to transport goods to direct customers. Strictly speaking, the inability to operate because inputs cannot be transported to the factory are a type of indirect effect.

For the case of utility lifelines, production losses are likely to be in the major loss category. We have specified consumer activity losses as not applicable ("n.a.") in that decreases in household activity (reduced showers, reading time, cooking) are not part of economic indices, and they should simply be counted in the "inconvenience" category above. There is no loss of consumption listed here either in that utility lifeline disruptions will have little effect on shopping over and above that attributable to business operation itself. For example, a power outage causing the closure of a department store is listed as a direct output (sales) loss for the producer, and it would be double-counting if counted as a consumption loss as well. The only instance of reduced shopping associated with utility lifelines would be the case of electric rail operation, which is very limited beyond a few major cities, at least in the U.S.

Finally, there are direct user costs associated with increased risk resulting in property damage or physical injury. For example, high congestion can spawn an increase in traffic accidents, and both transportation and utility damage can make fire prevention less effective. Two caveats are in order here. The first is mixing stock and flow concepts. Property damage and injury to life and limb are asset values, or stock measures, referring to the total value at a point in time. Economic indices we have been using to measure losses are typically flow concepts, pertaining to a duration of time. The two concepts are connected in that the value of an asset is the discounted flow of future profits (similar to a measure of value-added but excluding the labor component). Thus fire damage, or ground-shaking damage for that matter, should not be added as is, but rather converted into measures of lost production.

Personal injuries do not have multiplier effects and only some property damage will have multiplier effects. Factory building damage from fires would result in multiplier effects associated with upstream and downstream impacts to suppliers and customers, respectively.

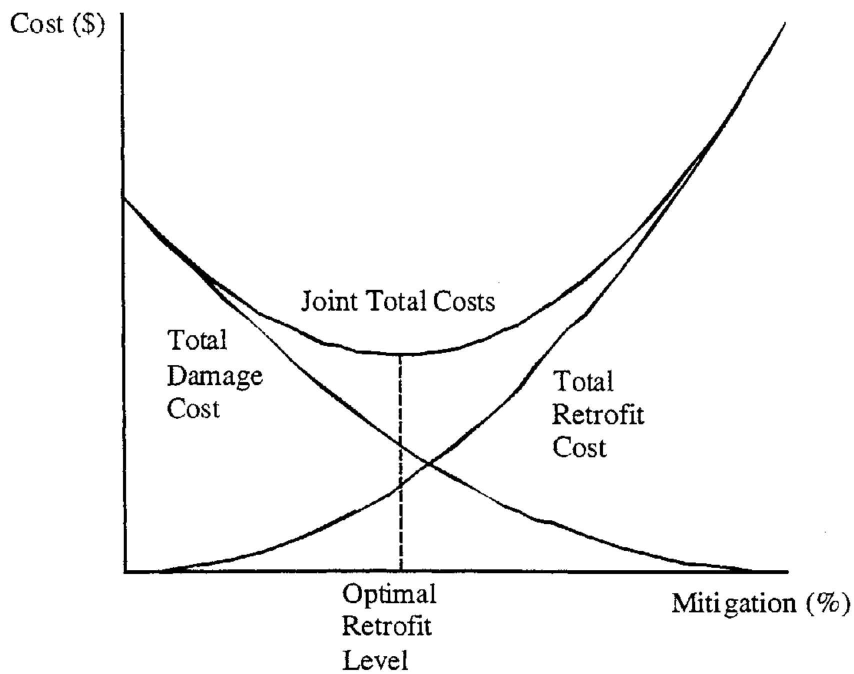
Interestingly, some property damage results in a positive stimulus to the economy from repair and reconstruction, but this is really just a reallocation of resources from one activity to another. Typically, the increase in repair/reconstruction would be matched by a decrease in other uses of the funds, often other investment.[5] Two exceptions may arise but their so-called positive impacts are illusory. First, is the case where victims (businesses or consumers) draw from their savings; however, as they replenish their savings, this reduces the spending stream by an equivalent amount in subsequent years. Second, is the case of a regional economy in which outside aid or insurance payments represent a net infusion of dollars (see, e.g., Lahr, 1997); however, this infusion is a drain on the remainder of the country, and the net economic impact of a natural hazard on society as a whole is still negative.

Fine-Tuning the Analysis

Cost categories C_3 and C_4 are difficult to forecast because of the uncertainty of the occurrence and magnitude of the natural hazard event. Complicating the picture further is the fact that these costs categories are highly variable depending upon recovery and reconstruction decisions with respect to timing and nonstructural options. For example, two similar factories may sustain the same amount of physical damage, but direct output decreases in the factory themselves, as well as the multiplier or general equilibrium effects, will differ if the first factory is repaired within a week and the second factory is not repaired for any other length of time. Sophisticated interdisciplinary damage models exist to trace ground-shaking through to structural damage, we have much less of a grasp of the speed of the recovery and reconstruction processes (see e.g., Chang et al., 1996; Rose et al., 1997). Moreover these may be as much a matter of politics than economics. In any case, we recommend against using extreme values of either the shortest or the longest recovery times.

Second, while the emphasis of this paper is on life cycle cost analysis of capital projects and the associated structural mitigation options, an increasing number of non-structural measures are becoming available. These range from rerouting traffic to reallocating utility services so as to minimize direct and indirect losses. For example, in the case of electricity disruptions, one can minimize employment losses by reallocating electricity toward sectors with the lowest direct and indirect electricity input intensity per dollar of output (see Rose et al., 1997).[6] This can turn out to be a rather costless way of reducing damages, and need not increase C_3 at all in order to lower C_4 (including indirect effects). Again, it is difficult to predict the sum total of economy-wide impacts a priori. However, we again offer the recommendation of avoiding extreme estimates.

One final point about the juxtaposition of C_3 and C_4. There are a range of retrofit (mitigation) options and of intensities at which they can be applied. The question is: which should we include in C_3? Again, the maximum level is not the ideal number, but for reasons different than noted above. The retrofit decision should be made by comparing its implementation costs (C_3) against its damage costs (essentially, the avoided damage cost represents benefits of lowering C_4). Thus, an optimum is reached at the retrofit intensity that minimizes the joint total costs (C_3 plus C_4), as depicted in Figure 1. For those more familiar with the standard terminology of benefit-cost analysis, this is also the point at which net benefits are maximized, or, equivalently, the point at which the marginal cost of retrofitting equals the marginal benefit of lower damages. Implicitly, retrofit optimization sub-model would be part of the overall model framework. Conceptually, this is straightforward as long as the four cost categories are distinct. However, as first pointed out by Chang and Shinozuka (1996), routine maintenance and hazard prevention can overlap. In such instances, one could ideally use an overall optimizing framework to minimize the sum of all costs simultaneously.[7]



AREAS FOR FURTHER RESEARCH

This paper has discussed a life cycle cost framework for evaluation of infrastructure investments, including those related to retrofitting or upgrading structures for natural hazard risk. It has focused on the significance of considering user costs in such a framework and suggested how this might be accomplished. Areas for further research include modeling of uncertainty in data and parameters, calibration of key parameters such as deterioration factors, and application to an existing infrastructure system using local cost data.

With regard to user costs, moreover, the discussion suggests several key areas for further study:

- Modeling direct user costs. Improvements would include more sophisticated modeling of direct user costs that differentiates between costs and benefits for different types of users (e.g., commuters, shoppers, transportation companies, etc.). An important advance would be to incorporate demand analysis and forecasting in evaluating future user costs, rather than assuming that such costs remain invariant over time. User costs may increase, for example, from urban growth.
- Modeling indirect user costs. A methodology should be developed for incorporating indirect user costs into the analysis. This includes distinguishing between the sources of different types of indirect impacts: output change, productivity change, safety considerations. It also requires an optimization analysis of minimizing the joint costs of mitigation and of damage, so as to avoid inflating mitigation damage costs.
- Consideration of non-engineering mitigation alternatives. In the life cycle cost examples outlined in this paper, it is assumed that risk reduction measures consist of structural retrofits or upgrades. These measures reduce total costs through decreasing potential hazard-related repair and user costs. However, non-engineering types of mitigation alternatives should be explored that are capable of reducing user costs alone (and at possibly lower cost), without necessarily changing either investment costs or expected physical damage patterns. Examples include emergency preparedness or emergency directed rationing of scarce electricity.

- Link to concept of "acceptable risk". The life cycle cost concept should be applied to current debates on questions of "acceptable risk". One approach is to minimize costs subject to a system performance constraint. An alternative approach would be to consider the probability of attaining specified performance levels through, for instance, methods of chance-constrained programming. Addressing "acceptable risk" could also involve reconciling performance objectives for different owner/user groups, i.e., optimization with reference to multiple performance criteria.

ENDNOTES

[1] Multipliers that include only indirect (interindustry) effects are referred to as Type I. Multipliers that include both indirect and induced (household-industry) effects are referred to as Type II. We will use the term "indirect" effect in this paper more loosely to include both categories.

[2] Note that the ability of a firm to pass along a cost increase depends on the competitiveness of the industry in which it operates. Thus costs that cannot be passed on to customers must be absorbed by the firm in the form of decreased profits or wages, thereby setting off a multiplier process through another channel.

[3] Most CGE models use I-O tables as a database for interindustry transactions (and hence the production requirements of material inputs), but they improve on I-O by allowing for input substitution and other nonlinear aspects of production and consumption, a greater role for prices, and an explicit incorporation of constraints. The National Institute of Building Sciences (1997) recently sponsored the development of an expert system to perform hazard impact analyses using a hybrid I-O/CGE approach to estimate indirect effects.

[4] Total impacts are measured in terms of gross product (gross output or sales) or net product (value-added, or returns to primary factors of production—capital, labor, and natural resources). The price multiplier effect is simply an intermediate step to calculate output effects. Note also that the two step process of price and output calculations is necessary only for the case of I-O analysis. In a CGE model, these are calculated simultaneously. Also, indirect effects in a CGE model are referred to as "general equilibrium" effects, in contrast to "partial equilibrium" effects, referring to a single sector or consumer category.

[5] Actually, the overall economic impact of the reallocation is likely to be negative if the use of the funds were optimally allocated before the earthquake. After the event, they are in effect transferred to a pressing but less overall efficient use.

[6] Indirect electricity intensity refers to whether a sector's upstream rounds of suppliers utilize large amounts of energy in their production. The optimal reallocation can be evaluated with an I-O model extended to a linear programming format intended to achieve some objective subject to production requirements and resource constraints, including electricity (see Rose et al., 1997; Rose and Benavides, 1998).

[7] Such a framework has been suggested by Chang et al., (1997); however, it differs in one major respect from that presented here by stipulating a given performance level. This is taken as exogenous, and usually stems from the interplay of engineering and political considerations relating to acceptable risk. The performance level itself is unlikely to pass a benefit-cost test as such. The overall framework offered in this paper, however, calculates the optimal level of retrofitting endogenously on the basis of a comparison of economic considerations alone. These are not likely to coincide with some more absolute measures of safety. Given the stochastic nature of decision-making in the context of natural hazards, and the steeply rising marginal costs as one approaches high performance levels, acceptable risk can be inserted in the problem as the minimum probability of attaining a target level of mitigation. In these contexts, the problem is best couched in an extension of linear programming, referred to as "chance-constrained" programming (see, e.g., Kunreuther, 1994; Rose and Benavides, 1998).

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INFRASTRUCTURE CAPITAL AND PRODUCTIVITY ANALYSIS

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I. INTRODUCTION

Several studies have been undertaken by economists to clarify the relationship between productivity growth and public infrastructure capital. These studies can broadly be classified as those which estimate a neoclassical production function augmented to include the publicly financed infrastructure capital stock as an input of production, and those which utilize the dual approach by estimating a cost or profit function. The second approach utilizes market data about the prices of private inputs and output, and treats public infrastructure capital as an unpaid factor of production. The nature of the data which has been used to estimate the production or cost functions is very diverse. Some studies use highly aggregate national and international data; others use regional and state level data. Some studies have used cross-section time series of metropolitan SMSA's, while others have employed industry-level data. The studies in each of the two categories often differ in their coverage of industries, geographic region, methodology and use of econometric estimation techniques, and the reported results often reflect these differences.

In this paper we attempt to briefly:

1. Review the findings reported in the literature on the effect of infrastructure capital productivity using the production function framework;
2. Contrast these findings with those reported using the cost and profit function framework;
3. To report the result of a recent study which discusses the effects of highway capital on costs of 35 US sectors and two digit industries.

The paper is organized as follows: In Section II, we briefly summarize the main results based on production function estimation about the effect of public infrastructure capital on productivity growth. Section III is devoted to a brief survey of the literature based on cost and profit function estimates and to report the major findings of a number of studies. In Section IV, we provide the results based on an analytical framework that takes into account *both* demand and supply factors that affect growth of output and total factor productivity. Within this general framework, relative contribution of public infrastructure capital to productivity growth is evaluated. It is possible to trace the effect of aggregate demand, population growth, rise in real factor prices, technical change, and types of infrastructure capital on productivity growth of different industries. Aggregating over the industries, it is possible to estimate the productivity effects of the infrastructure capital on the aggregate economy and demand for capital labor and materials for the total private sector as well as calculating the social rate of return on investment in infrastructure capital. A brief summary concludes the paper.

II. WHAT HAS BEEN LEARNED FROM THE PRODUCTION FUNCTION APPROACH?

There is extensive literature on the effect of public infrastructure capital on growth of output and productivity using the production function framework. The analysis falls into two categories: (a) aggregate production studies, and (b) regional or state level production function analysis. Table 1 presents some of the characteristic features of a number of studies that are based on production function estimation. This literature began with a paper by David Aschauer (1989) which stimulated an extensive discussion of the kind and magnitude of the impact of infrastructure capital on output and productivity growth.[1] He used an aggregate production function to argue that infrastructure capital financed by the public sector increased the capacity of the private sector to be more productive, and that public infrastructure investment stimulates private sector investment by enhancing the rate of return to private sector investment. Munnell

[1] For a review of the literature, see David A. Aschauer, "Infrastructure and Productive Efficiency: A Literature Review," mimeo, 1993.

Table 1: Production Function Estimates

PAPER	EQUATION	DATA	COEFFICIENT*	COMMENTS
ASCHAUER (1989)	Cobb-Douglas prod. func. and TFP regres. on t, g, cu	time series 1949-1985 private business economy	0.39-0.36 0.37-0.41 Significant	CRS in all inputs, including public capital input
MUNNELL (1990a)	Cobb-Douglas prod. func. reproduces Aschauer	time series 1948-1987 private non- farm sector	0.34-0.41 Significant	CRS in all inputs; also priv. and publ. cap. coef. Equal
MUNNELL (1990b)	Cobb-Douglas prod. func.??	cross-sect. time series 48 states 1970-1986	0.15	see Munnell 1991 and other references
MUNNELL (1991)	Cobb-Douglas prod. func.	cross-sect. Average 1970-1986 states values 12 high endowm. 26 mid. endowm. 10 low endowm.	 0.14 0.11 0.22 Significant	Returns to Scale 1.01 1.03 1.04
GARCIA-MILA AND McGUIRE (1988)	Cobb-Douglas prod. func.	cross-sect. time series 14 annual obs. of 48 states gross state prod. labor, capital expenditures on education and highways	Highways: 0.045-0.044 Education: 0.16-0.072 Significant	Returns to Scale 1.04 Cannot reject increasing RTS
EBERTS (1988)	Translog prod. func.	cross-sect. manufacturing 1958-1978 38 Metropolitan areas	0.04 Significant	CRTS public and private capit. substitutes public and labor complem.
HULTEN AND SCHWAB (1991a)	Cobb-Douglas prod. func. First differ.	time series 1949-1985 same as Aschauer	0.42 Significant 0.028 Insignificant	(-) coeff. for labor?
TATOM (1991)	Cobb-Douglas prod. func. including energy price First differ.	time series 1974-1987 Business Sect.	 0.146 Insignificant	CRTS
MERA (1972)	Cobb-Douglas prod. func.	Japan pooled data of regions and time 3 sectors primary secondary tertiary 4 classifications of social overhead capital	 0.22 0.20 (.50) 0.12-0.18 Significant	
FORD AND PORET (1991)	TFP regressions	USA and 11 OECD Countries time series and country cross-section	Half of countries significant effect after 1960 Mixed support of Aschauer results	
HULTEN AND SCHWAB (1991b)	TFP regression on g, t, and k	Cross section time Series regional study of Snow-Sun Belt 1970-1986 Gross output value added	Public capital insignif. in all regressions private capital insignif. in gross output regres. signif. in value added implying scale .88	

*Coefficient of infrastructure capital

(1990a) extended this line of argument, and her results generally support the proposition that there is a strong and significant effect of public capital on productivity.

Aschauer and Munnell employ aggregate time-series data on the United States to estimate the relationship between private output and the stock of nonmilitary public capital, which consists of the following equipment and structures: highways, streets, educational buildings, hospital buildings, sewer and water facilities, conservation and development facilities, gas, electric, and transit facilities, and other miscellaneous but nonmilitary structures and equipment. Aschauer finds that the elasticity of output with respect to public capital is 0.39. Munnell finds an elasticity of 0.33 for output per hour with respect to public capital. She uses the estimated coefficients of the aggregate production function to calculate annual percentage changes in multifactor productivity and concludes that the "drop in labor productivity has not been due to a decline in the growth rate of multifactor productivity or technical progress. Rather it has been due to a decline in the growth of public infrastructure."

Several criticisms of these results have been raised. Some have argued that the estimated elasticities and the implied marginal productivity of the public capital are extremely high. The marginal productivity of public infrastructure capital based on Aschauer's estimates will exceed that of the private capital stock by several times, which seems highly implausible (Aaron (1990)). Others have argued that the aggregate time series correlation may not reflect a causal relation, but may reflect a spurious correlation, i.e., both labor productivity and public infrastructure spending have declined over the same period due to other forces (Aaron (1990) and Tatom (1991a)). A third issue is that a reverse causation may exist between public infrastructure capital and productivity growth; it is argued that the positive coefficient estimated in various studies may result from the effect of productivity growth on infrastructure capital rather than the reverse. Also there is some evidence of a lack of robustness when more recent data are used to estimate the aggregate production function of Aschauer and Munnell.

There are a number of production-function studies that utilize data at the state level. The data consist of time-series cross-section of data on 48 states for several years. This richer body of data has certain advantages which mitigate the possibility of spurious correlation over time. As a whole, the studies based on state-level data support a small but positive relationship between public infrastructure and productivity.

Munnell's (1990b) elasticity coefficients: public capital, 0.15; private capital, 0.31; and labor, 0.59; show that although public capital has a positive effect on output productivity, it is only half the size of the effect of private capital. For example, a 1 percent increase in public capital causes a 0.15 percent increase in output productivity, whereas a 1 percent increase in private capital causes a 0.31 percent increase in output productivity. However, calculating the marginal products shows that one more unit of public capital will increase output by the same amount as an additional unit of private capital. The results remain plausible when public capital is split into its three components—highways, water and sewer systems, and other. The first two, which are the largest part of core infrastructure, have much larger effects than does the "other" category. The coefficients are labor, 0.55; private capital, 0.31; highway stock, 0.06; water and sewer facilities, 0.12; and other public capital stock, 0.01.

Using Munnell's data, Eisner (1991) reports that for all functions considered, the significance of public capital holds up when the data are arranged to reflect cross-sectional variation, but disappears when the data are arranged to allow for time-series variation. The data tell us that states with more public capital per capita have more output per capita, but that a state that increases its public capital in some year does not as a result get more output in that year. Therefore, Eisner regards the direction of causation between output and public capital as undecided and postulates that a lag structure will be needed to get a true time-series relationship between output and public capital.

Calculating manufacturing productivity growth rates for the years 1951 to 1978 for major regions of the United States, Hulten and Schwab (1984) test whether different rates of public capital growth correspond to different rates of productivity growth. They find that the differences in output growth are not due to differences of public infrastructures, but instead to variation in the rates of growth of capital and labor. In a later paper, Hulten and Schwab (1991) expand the analysis to include the years 1978 to 1986, but their conclusion remains the same: public infrastructure has had little impact on regional economic growth.

These disparate results are likely due to whether the unobserved state-specific characteristics are controlled in the estimation process. Holtz-Eakin (1992) has tested the hypothesis that the positive and strong effect of infrastructure will diminish or disappear if the state-specific effects are accounted for. An interesting recent study which provides a feel of the range of estimates is performed by McGuire (1992). McGuire estimates four specifications of a state-level production function with public capital as an input: Cobb-Douglas with no control for State effects; Cobb-Douglas controlling for State fixed and random effects; and translog with no control for State effects. The drawback of a Cobb-Douglas production function is that it restricts the elasticity of substitution between each pair of input variables to equal 1. The four specifications of the model yield broadly similar results, with public capital having a positive and statistically significant effect on GSP. The elasticity ranges from 0.035 to 0.394 in McGuire's new estimates.

When public capital is split into its three component parts (highways, water and sewers, and other), highways has the strongest impact, with elasticities ranging from 0.121 to 0.370. Water and sewers has a much smaller but usually significant effect, and other public capital is not statistically significant or has a negative effect on private output. Indeed, some economists hypothesize that state-level data may systematically underestimate the productivity value of public capital, because such data cannot capture the aggregate effects of public capital as a system.

Similar findings have been reported by a number of production function studies which utilize even more disaggregated data. Studies by Eberts (1986), Eberts and Fogarty (1987) and Duffy-Deno and Eberts (1991) use data at the metropolitan level. They test the direction of causation between infrastructure capital and output and estimate the magnitude of the elasticity of output with respect to infrastructure capital. Their findings suggest that the causation runs mostly from infrastructure capital to output growth and there is a positive but considerably smaller elasticity of output with respect to public capital than those based on aggregate production functions, relationship between infrastructure and growth of output and productivity.

From a reading of the evidence so far, based on production function studies it is possible to draw the following conclusions: (1) The early estimates based on aggregate production function analyses considerably overstated the magnitude of the effects of public infrastructure capital on growth of output and productivity. (2) The estimates on state level data indicate a much smaller contribution of infrastructure and that the composition of infrastructure capital matters; some types of infrastructure capital's contribution are larger than others. (3) There are serious estimation problems in both aggregate national time series studies and state and regional level studies that lead to highly disparate results. (4) On the whole, it seems that the evidence points to a positive but small elasticity of output with respect to public infrastructure capital of about 0.10 to 0.20 at the national level and possibly a lower range at the regional level.

One reason for the wide range of estimates of the elasticity of output with respect to infrastructure capital based on production function estimates may be due to minimal modelling structure imposed on the data. If enough structure is not imposed on the data, provided that the underlying data are not subject to serious or major measurement problems, the parameter estimates of the underlying production structure are likely to be biased and the estimates are not likely to be robust. In estimating production functions, whether using national aggregates or state level data, the production function is treated as a purely technological relationship between output and inputs, and firms optimization decisions with respect to how much output to produce and what mix of inputs to use in the production process is not considered specifically. In reality, inputs and output are simultaneously determined when firms optimize (minimize) their profit (costs). When firms' optimization is explicitly considered, the marginal productivity conditions for the inputs should be estimated jointly with the production function. If the marginal conditions are not explicitly considered, the estimated production function parameters are likely to be seriously mismeasured.

III. THE COST (PROFIT) FUNCTION APPROACH

The cost- or profit-function approach takes explicit account of the optimizing behavior of firms with respect to inputs, with prices as the only exogenous variables. In addition, in most studies on infrastructure production is assumed to be of a Cobb-Douglas specification, which a priori imposes restrictive conditions of the substitutability of inputs. There is a need for more flexible functional forms.

This approach also yields direct estimates of the various Allen-Uzawa elasticities of substitution. These parameters are the key to describing the pattern and degree of substitutability and complementarity among the factors of production. Furthermore, the effect of public capital on the demand for inputs can be directly estimated. If the effect is positive, the public capital and the private inputs are complements; if it is negative, the public capital and private inputs are substitutes. Finally, it is possible to easily derive the marginal benefit of infrastructure capital from a cost function; the first derivative of cost with respect to public capital gives the marginal benefit, in terms of cost reductions, of public capital services. The sign of the cost derivative with respect to infrastructure capital will indicate the direction of the effect. If it is negative, an additional unit of public capital stock will make the firm better off; if it is positive, worse off. Under general conditions of convexity of the technology of the firms (see Diewert (1986)), the equation indicated in the footnote can be considered the demand for publicly-financed capital. Furthermore, we can test the notion of whether the amount of infrastructure is optimal. Summing the marginal benefits of infrastructure over all firms, and equating the sum of marginal benefits with the marginal cost of infrastructure k, we can derive the optimal amount of infrastructure services. By comparing the optimal amount of infrastructure capital services determined by the model and its actual level, it is possible to conclude whether there is an undersupply of infrastructure capital services in a particular geographical unit.

In general, the dual approach (i.e., cost and profit function) provides a richer framework to explicitly address such questions as: What is the effect of public capital on inputs of production? Does public capital have a crowding-out or crowding-in effect on private capital? What is the marginal benefit or willingness of the private sector to pay for an additional increase in public infrastructure? Is the level of publicly-provided capital optimal from the perspective of the private production sector?

The number of studies using a cost function to analyze contribution of infrastructure capital and other types of publicly financed capital are relatively few compared to the number of studies based on the production function. Some of the important features of these studies are presented in table 2. The dual approach has been applied in a set of diverse studies, using different types of data at the national and international level, state level data, and industry data, using different assumptions about the optimizing behavior of firms, and specifying different functional forms with special preference to the translog and generalized Leontief. In addition, different authors use different notions of public infrastructure. Some use the core infrastructures, others the total stock, and others adjust these stocks to reflect the collective nature of public capital investment.

For the United States, the cost function approach has been applied by Keeler and Ying (1988) to the trucking industry, Lynde and Richmond (1992) to the corporate business sector, and Nadiri and Mamuneas (1994) to disaggregate two-digit manufacturing industries. Also, Morrison and Schwartz (1991) estimate a variable cost function for the manufacturing sector by state, while Deno (1988) estimates a profit function for 36 metropolitan areas. For outside the US, Shah (1992) estimates a variable cost function for the Mexican manufacturing sector, Conrad and Seitz (1992) and Seitz (1992a, 1992b) estimate a cost function for West Germany's manufacturing, construction and trade sector, and finally Berndt and Hansson (1992) estimate a variable cost function for the private sector of Sweden. Even though a single estimate cannot be provided for the effect of public infrastructure on the cost and consequently on its contribution to the productivity, all studies reach the conclusion that publicly financed capital contributes positively to productivity in terms of cost savings.

(a) Cost Elasticities

At the aggregate US level, Lynde and Richmond (1992) estimate a translog cost function using aggregate US nonfinancial corporate business sector data for the period 1958 to 1989. By imposing constant returns to scale in all inputs, public capital included, and by assuming that firms behave competitively, they estimate two cost-share equations: one for labor, and one for public capital. Due to the constant returns to scale assumption, they are able to define the cost share of public capital as one minus the output cost share. Their findings suggest that publicly financed infrastructures reduces the cost of the nonfinancial corporate business sector.

Table 2: Cost or Profit Function Estimates

DESCRIPTION				DIRECT EFFECT	INDIRECT EFFECT		
Author	**Unit of Analysis**	**Specification**	**Public Capital**	**Cost**	**Labor**	**Capital**	**Intermediate**
BERNDT AND HANSSON (1991)	Sweden Private Sector 1960-1988	Variable Cost Labor Requirement Function	Core Public Capital	Cost Savings? Unclear	Short-run Complements	.....	.....
DENO (1988)	USA 36 SMSA Manufacturing Industries 1970-78 Pooled	Profit Truncated Translog	Highway, Water and Sewer Adjusted with the proportion of population employed by the sector	Profit increase Elasticity = .08 to .5	Gross complements Elasticity = 0.1 to .4	Gross complements Elasticity = 0.11 to .4	.....
CONRAD AND SEITZ (1992)	West Germany Manufacturing Construction, Trade and Transport 1960-1988 Time-Series	Cost Translog and MR=MC	Total Adjusted with capacity utilization rate	Cost Savings	Substitutes	Complements	Substitutes
KEELER AND YING (1988)	USA Trucking Industry 1960-1988 Regional Pooled	Cost Translog	Highway Stock	Cost Savings	.....	.....	.....
LYNDE AND RICHMOND (1992)	USA Nonfinancial Corporate Business sector 1958-1989 Time-Series	Cost Translog P = MC and CRTS	Total Federal and State	Cost Savings	Substitutes Elasticity = -.45 to -.49	Complements Elasticity = .71 to .90	.....
LYNDE AND RICHMOND (1993)	U.K. Manufacturing sector 1966:1 to 1992:2 value added	Cost translog	P_1 Total	Cost Savings	.....	Substitutes	.....
MORRISON AND SCHWARTZ (1991)	USA Manufacturing by State 1971-1987 Pooled by Region State specific Effects	Variable Cost Generalized Leontief P=MC	Core	Cost Savings Elasticity = -.10 to -.27	.....	.....	.....
NADIRI AND MAMUNEAS (1991)	USA Manufacturing 12 2-digit industries 1955-1986 Pooled Industry Specific Effects	Cost Translog CRTS for Private Inputs	Total public Stock Adjusted with Capacity Utilization Rate	Cost Savings Elasticity = 0 to -.21	Substitutes Elasticity = 0 to -1.4	Substitutes Elasticity = -.02 to -1.4	Complements Elasticity = .12 to .76
SEITZ (1992a)	West Germany 31 2-digit Industries 1970-1989 Pooled Industry Specific Effects	Cost Generalized Leontief	Public Roads Length of Motorway System	Cost Savings	Substitutes Elasticity = -.0004	Complements Elasticity = .03 to .04	.....
SEITZ (1992b)	West Germany 31 2-digit Industries 1970-1989 Pooled Industry Specific Effects	Cost Generalized Leontief	Total Core	Cost Savings	Substitutes Elasticity = -.15 to -.13	Complements Elasticity = .34 to .86	.....
SHAH (1992)	Mexican Manufacturing Sector 26 3-digit Industries Pooled	Variable Cost Translog	Total Adjusted with industries' output proportion	Cost Savings	Complements Elasticity = -.006	Complements Elasticity = -.002	Substitutes Elasticity = .005

Nadiri and Mamuneas (1994) estimate a translog cost function for 12 industries of the manufacturing sector at the two-digit level for the period 1995 to 1986. They pool the data across the industries, but allow for industry-specific effects and estimate the cost function together with the share equations for labor and capital inputs. Their findings indicate that an increase of public infrastructures as well as publicly financed R&D reduces the cost of the industries in their sample. They estimate that the cost elasticity of public infrastructure varies from industry to industry within the range .05 to -.21.

Morrison and Schwartz (1991) estimate a variable cost function using US state level data for the total manufacturing sector over the period 1971 to 1987. They specify a generalized Leontief cost function, treating private and infrastructure capital as exogenous. They estimate a system of input-output equations for production labor, non-production labor and energy, and a short-run output price equation (p = mc) to incorporate profit maximization. The estimation is carried out for the regions—Northeast, North Central, South and West —, with pooling parameters for each state added as intercept terms on the estimating equations. Their results suggest that an increase of 1% of public capital reduces the cost from .15% in the Northeast to .25% in the West. In addition, the authors calculate the contribution of infrastructures to productivity growth for each region and state.

Deno (1988) estimates a translog profit function at the regional level of 36 standard metropolitan statistical areas for the manufacturing industries from 1970 to 1978. He estimates the effects of highway, sewer and water capital on output supply, and on unconditional demands of capital and labor. In order to take into account the collective nature of public capital, he multiplies the public capital stocks by the percentage of the metropolitan population that is employed in the manufacturing sector. His findings suggest that all types of public capital contribute positively to output growth, but that highway and sewer capital contribute the most to output growth, capital formation and employment. He finds that output supply responds strongly to total public capital with an elasticity of 0.69. The corresponding elasticities for specific types of capital are 0.31 for highway capital, 0.30 for sewer capital, and 0.07 for water capital.

In particular, for the US road freight transport industry, Keeler and Ying (1988) estimate a translog cost function of regional truck firms for the period 1950 to 1973. They find that the highway infrastructure has a significant effect on the productivity growth of the trucking industry, with substantial benefits of this investment, justifying about half of the cost of the Federal aid highway system. Dalenberg (1987) estimates cost functions across metropolitan areas and finds public capital and private capital to be complementary inputs. He finds that public capital lowers costs; however, he also finds, based on his estimates of the shadow price of public capital, that many SMSA's have overinvested in public capital from the manufacturing sector's point of view.

At the international level, Berndt and Hansson (1992) estimate a short-run (variable) cost function by specifying a labor requirement function and using aggregate data from the Swedish private sector, assuming that private capital as well as public capital are fixed in the short run. They find that public infrastructure and labor input are complements for the 1960's and 1980's, while they were substitutes for the 1970's. The authors conclude that the increase of public infrastructures reduces private costs. In addition, the authors estimate the ratio of the optimal amount of infrastructure capital to the existing capital stock by equating the marginal benefits of private and public capital with their corresponding rental prices and solving simultaneously for the optimal amounts of private and public capital. They find that for the period 1970 to 1988, there were excess amount of infrastructures for the private production sector of the Swedish economy.

Lynde and Richmond (1993) estimate a translog cost for U.K. manufacturing for the period 1966-1990 using quarterly data. They control for nonstationary effects in the time-series and classify changes in productivity according to four parts: (i) changes in the public capital to labor ratio; (ii) changes in economies of scale; (iii) changes in prices of intermediate inputs, including energy; and (iv) changes in technology. They find an average elasticity of output with respect to public capital of 0.20 and they attribute approximately 40 percent of the productivity slowdown to the decline in the public capital to labor ratio. Their estimates indicate that there is a significant role for public capital in the production of value-added output of the manufacturing sector.

Shah (1992) estimates a translog variable cost function treating labor and materials as variable inputs and private capital and public capital as fixed inputs. Shah uses data from 1970 to 1987 for twenty-six Mexican three-digit manufacturing industries and takes into account, as do Nadiri and Mamuneas (1994) and Deno (1988), the usage of public infrastructures. Thus, he constructs the industry usage of public infrastructures to be proportional to public capital where the degree of proportionality is defined as the ratio of the industry's output to the output of all industries. He finds that the short run effect of infrastructures is to reduce variable cost, implying that there is underinvestment in public capital.

Conrad and Seitz (1992) estimate a translog cost function together with marginal revenue equal to marginal cost condition for the manufacturing, construction and trade and transport sector of West Germany for the period 1960 to 1988. They find that the estimate of the shadow price of infrastructures is .06, 0.03 and .06 respectively, implying that there is substantial reduction of cost. Similar results are reported by Seitz (1992a, b) for the effect of core and total public capital on the cost of 31 two-digit industries of the West German Manufacturing sector for the period 1970 to 1987. These results are generated by estimating a generalized Leontief cost function.

(b) Effects of Public Capital on Employment and Private Capital

The public capital hypothesis asserts that the public capital has both a direct effect and an indirect effect on the productivity of private sector (see Tatom (1991b)). The direct effect arises under the assumption that marginal product of public capital is positive, i.e., an increase of public capital services increase the private sector output. The indirect effect arises under the assumption that the private and public capital are complements in production, i.e., the partial derivative of marginal product of private capital with respect to public capital is positive. If private and public capital are complements, this hypothesis asserts that an increase of public capital raises the marginal productivity of private capital, and given the rental price of capital, private capital formation increases, further raising private sector output.

In the cost function framework, the direct effect of infrastructure capital is measured by the magnitude of cost reduction, due to an increase of public capital. Its indirect effect is given by the magnitude of its effect on demand for public sector factors of production. This sign of this effect will determine whether infrastructure capital is biased toward one or another of the private inputs.[2] A priori no sign can be assigned to the indirect effect of public capital on the inputs of production. The sign and magnitude of the effect is an empirical question. It seems that the cost function literature supports the hypothesis that labor and public capital are substitutes. Lynde and Richmond (1992) find that the public capital elasticity of labor is about .45, Nadiri and Mamuneas (1994) estimate labor elasticities from 0 to 1.4, Seitz (1992a) estimates an elasticity of .0004 for public roads, Seitz (1992b) estimates .15 for public capital and finally the same substitutability relationship is found by Conrad and Seitz (1992). Exceptions are Berndt and Hansson (1992) who find a short-run weak complementarily between labor and public capital and Shah (1992) who finds labor and public capital to be short and long-run weak complements. For the relationship between public capital and private capital there is no clear cut evidence. The studies are divided between Conrad and Seitz (1992), Seitz (1992a, b) and Lynde and Richmond (1992), who find that public capital and private capital are complements and Shah (1992) and Nadiri and Mamuneas (1994), who find that there are substitutes with elasticities from 0.005, and 0.02 to 1.4, respectively. Also, Nadiri and Mamuneas (1994) have estimated that public capital and intermediate inputs are complements for the US manufacturing sector, while

[2] To see the linkage between the direct and indirect effects note that at the optimum, for a given output, Y, the cost function is:

$$C(Y,P,S,T) = \sum_i P_i X_i^*$$

where *P*, *S* and *T* are vectors of relative price, vector of infrastructure capital and rate of disembodied technology. X_i^* is given by $\partial C / \partial P_i$. Differentiating the cost function with respect to *S* yields

$$\partial C / \partial S_k = \sum_i P_i \cdot \partial X_i^* / \partial S_k$$

which decomposes the cost change associated with an increase of public capital services into adjustment effects of private inputs. If now all private inputs are substitutes with public capital, then an increase of public capital is always cost saving. The inverse, of course, is not true. As has been shown so far, the literature of the cost function framework reviewed supports the hypothesis that cost savings are associated with an increase of public capital. Hence, if one of the private inputs is a complement of public capital, then cost savings can arise only if the substitution effects of the other private inputs outweigh the complementary effect (see also Seitz (1992b)).

Conrad and Seitz (1992) report that they are substitutes for West Germany. Finally, Deno (1988) has estimated that labor and capital are both gross complements of public capital.[3]

(c) Optimal Provision of Public Capital and Its Rate of Return

One question which has been raised in the literature and has important public policy implications is whether or not public capital is at its optimal level. In other words, is public capital under-supplied? Public capital can be considered as not only a public good, used as an intermediate input for the production of private goods, but also as providing services directly to the consumer. The optimal provision of public capital services in such cases will be given by the well-known Samuelson condition, as modified by Kaizuka (1965). This condition requires that public capital be provided at the point where the sum of marginal benefits of producers and consumers is equal to the marginal cost of providing one additional unit of public capital. However, the literature has so far emphasized only the producer benefits arising from infrastructures.

Ignoring the consumption sector, an alternative means of determining whether public capital is provided optimally is to consider the rate of return of public capital and compare it with the rate of return of public capital for the whole economy. The optimal provision of public capital requires that the rates of publicly provided and private capital be equalized. Thus, if the rate of return of public capital is higher than that of private capital, public capital is undersupplied and an increase of public investment is necessary. Nadiri and Mamuneas (1994) find that the rate of return of public infrastructures implied by the industries of the manufacturing sector is about 7%, while the rate of return of private capital is about 9%. If, however, one considers that the industries of their sample is a small fraction of the economy's production sector, then the implied rate of return of infrastructure will exceed the rate of return of private capital.

Morrison and Schwartz (1991) take another approach. They compare the shadow price of public capital with the "user cost" of public capital, and find that the Tobin's q ratio of public investment exceeds one, suggesting that infrastructure investment has been too low for social optimization for the manufacturing sector of all regions in their sample. Similarly, Shah (1992) estimates a Tobin's q equal to 1.04 for the Mexican manufacturing sector, and concludes that there is indication of underinvestment of public capital.

Finally, Berndt and Hansson (1992), by equating the marginal benefit of public infrastructures with its ex-ante rental price, solve for the optimal capital stock and then calculate the ratio of the optimal capital stock to the actual public capital. They find that this ratio is above one for the period 1960 to 1970, and below one for the period 1970-1990, suggesting overinvestment, although the ratio is rising for the late 1980s. Thus, they conclude that the "roads and highways were not as well maintained as had been the case in the 1970s and early 1980s" in Sweden.

(d) Comparison between production function and cost function estimates

One important question which arises from these studies is if one is able to derive a "single" estimate of the effect of public infrastructure, in terms of magnitude, on the productivity of the private sector. Another important question is how the estimates from the cost function approach compare with the estimates of the production function approach. Both questions are difficult to answer because of the diverse data, assumptions employed, level of aggregation and available information provided by the authors.

First, the direct magnitude of the productivity effect in terms of the elasticity of cost with respect to public infrastructures is unfortunately not reported in many studies except in Morrison and Schwartz (1991) and Nadiri and Mamuneas (1994) who report an elasticity of -.1 to -.3 and 0 to -.2, respectively. Therefore, if any comparison should be made it must be based only on those two studies. Second, in comparing these elasticities which are based on a disaggregated level with the output elasticity generated from an aggregate production function, reported by Aschauer (1989), there are two problems involved. First is the problem of proper aggregation and second is that the elasticity of cost with respect to output has to be known. It is easy to show that the public capital output elasticity is

[3] Note that substitutability and gross substitutability are different notions. Gross substitutability allows for the adjustment of output (see Chambers (1988)). One input can be a substitute for another input, as well as a gross complement, as long as both are non-regressive, and the induced output effect overcomes the substitution effect.

equivalent to the negative of the ratio of the elasticity of cost with respect to public capital over the cost elasticity of output.

Based on the public capital cost elasticities of Nadiri and Manuneas (1994) and noting that the output of the twelve industries of their sample corresponds to about one-sixth of private sector output, we can deduce from (9) that Nadiri and Mamuneas' estimates will imply a public capital output elasticity of about -.12. Similarly, the corresponding public capital output elasticity based on the cost elasticities of Morrison and Schwartz (1991) will be a weighted average from -.15 to -.27. Both these estimates are much lower that the elasticities based on the production function approach and reported by Aschauer (1989) and Munnell (1990a). Note, however, that since Nadiri and Mamuneas' estimation is based on gross output rather than value added, the output elasticities are not directly compatible with the other elasticities and is likely to be downwards biased.

IV. A STRUCTURAL DEMAND AND SUPPLY FRAMEWORK

In a major study, Nadiri and Mamuneas (1996) have developed a general structural model that includes demand and cost functions to capture the interplay of both supply and demand forces in determining total factor productivity. The contribution of infrastructure capital is assessed within this general framework. The model is estimated using 35 sectors and two digit industry data for the period 1950-1991 and highway capital stock is considered as a measure of infrastructure capital in the model. The approach accounts for effect of aggregate demand increase in real factor prices and autonomous technical change in the industry output and productivity growth. The effect of public infrastructure (highway) capital on demand for private sector inputs such as labor and capital and its productivity effect on industry costs are estimated. The main results of this study are:

(i) Total highway capital contributes significantly to economic growth and productivity at the industry and national economy levels. Its contribution varies across industries and over time. The magnitude of the elasticity of output with respect to total highway capital at the aggregate level is about 0.05, which is much smaller than comparable estimates reported in previous literature.

(ii) The contribution of highway capital to TFP growth is positive in almost all industries. The magnitudes of the contribution varies among industries, although the most significant contribution of highway capital is to the productivity of manufacturing industries. At the aggregate level, highway capitals contribution to TFP growth is about .17.

(iii) There is some evidence of increasing returns to scale in most industries and at the national level. Both at the industry and national levels, the contribution of private capital to economic output dominates that of total highway capital by almost four times. This is in sharp contrast to the results reported in the literature.

(iv) Total highway capital has a significant effect on employment, private capital formation and demand for materials inputs in all industries. For a given level of output, an increase in highway capital can lead to a reduction in demand for all inputs in manufacturing, while in non-manufacturing industries the pattern is mixed. The magnitude of these effects varies among the three inputs in a given industry and among the industries, and does not consider output expansion aspects of lower costs.

(v) The marginal benefits of total highway capital at the industry level were calculated by using the estimated cost elasticities. Demand for highway capital services varies across industries as do the marginal benefits. The marginal benefits are positive for all the industries, and their magnitudes vary greatly.

(vi) The results indicate that net social rate of return on total highway capital was high (about 30 percent) in the 1950s and 1960s, then declined considerably until the 1980s to about 10 percent. In the 1980s, the rates of return on total highway capital and private sector capital seem to have converged, and are basically equal to the long term rate of interest.

(vii) The main contributor to productivity both at the industry and aggregate level is aggregate demand. Relative prices, capacity utilization rate and technical change also contribute to the growth of TFP, but their contributions are generally smaller and vary across industries. Highway capital contributes mainly to long run trends in TFP growth and only minimally to its acceleration or deceleration over different periods such as the period 1973-76.

V. CONCLUDING REMARKS

The economic studies mentioned in this paper suggest that a positive relationship exists between investment in infrastructure and output and productivity growth. What is in dispute is the magnitude of the effects of changes in infrastructure capital on output and productivity growth. The initial few studies based on aggregate production function suggested an extremely powerful effect and attributed an excessively high rate of return to infrastructure investment. Subsequent studies and particularly those employing the dual framework, i.e., cost or profit functions, point to much more modest but still significant impact of infrastructure capital on output and productivity growth.

Most recent studies that examine the impact of infrastructure capital, especially the highway capital on the cost structure of 35 sectors and two digit industries, suggest that all industries benefit from such investment though the degree of benefits vary among the industries. Highway capital has an important effect on employment, investment in plant and equipment, and demand for materials in each industry. Highway capital is a public good and the estimate suggests a fairly high social rate of return to this investment. The rate was particularly high at the beginning when the US Interstate Highway System was being built. As the system matured, the social rate of return has also declined. At the economy level, the impact of highway capital on growth of output and productivity are much more modest than the original production function estimates. It is also clear that investment in highway capital (for that matter, other types of infrastructure capital) is to raise productivity growth in the long run but not to affect short-term fluctuation in rate of productivity.

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Infrastructure and Sustainability: Building Sustainable Cities and Regions

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Cities and Infrastructure

In cities and metropolitan regions throughout the world, citizens routinely base their daily economic, personal and social activities upon publicly and privately owned and operated infrastructure systems. Infrastructure can be categorized into two broad forms: linear networks and site-based systems.

Linear networks include subsurface systems such as water, gas, sewer, oil, and gas lines, and surface systems such as power lines, flood control channels, storm drains, freeways, streets, bridges, and railroads. Associated with linear networks are supporting facilities such as railroad stations, power generation plants and substations, water and sewer treatment plants, and debris basins and dams. Site-based systems include separated facilities such as civic buildings, police stations, fire stations, hospitals, schools, parks, libraries, waste recycling centers, and waste disposal facilities.

Also to be considered as part of both types of systems are the institutions, including organizations, financing, and decision processes which determine all of the preceding. Normally, the socio-political or "soft" institutional systems related to "hard" physical infrastructure are overlooked as a critical factor influencing system development, maintenance and renewal. Yet in order to address infrastructure sustainability issues, institutional aspects are a crucial component.

Except in new towns, physical infrastructure systems have been constructed at different points in history, designed to function at varying capacities, and financed from a variety of capital sources. Similarly, they are decaying or becoming obsolete at differential rates. Consequently, for any city which has existed for a half century or more a process of renewal and redefinition of infrastructure need is continuously taking place. This leads to issues of system efficiency, impacts, costs, and utility having complex ramifications for city planning. While themes may be very similar from city to city, the issues are manifested quite differently in relation to the specific experiences of local history.

Purpose

The purpose of this paper is to explore the topic of sustainable development as it relates to infrastructure and focus on aspects of potential mutual interest between the U.S. and Japan in ongoing joint research. The concept of sustainability has emerged as part of the environmental movement in recent decades. Initially the term referred primarily to minimizing negative impacts of development on environmental resources. In recent years, it has been broadened to also encompass aspects such as development of long-term economic self-sufficiency, reversal of blight and urban decay, retention of historic and cultural values, and other quality of life factors related to development and the environment..

Central and Related Questions

In this exploration, the evolving concept of sustainable development is examined for its potential value and relevance to infrastructure design, development, financing, and management. To focus on issues worthy of joint research, a central question is raised: how and to what extent can we build efficient and convenient infrastructure systems providing optimal service at least cost with minimal negative impacts on the economy and environment for the longest appropriate life spans?

To answer this central question most completely in the long run, a series of corollary questions must be addressed.

- What lessons have we learned from the history of existing systems?
- What socio-economic and technological changes will influence their design in the future?
- How can we finance such systems most equitably as well as effectively?
- How can we minimize the impacts of natural disasters on these systems?
- How can we fit these systems more compatibly into the complex fabric of growing metropolitan regions and threatened natural environments?

What is Sustainable Development?

One of the greatest difficulties in applying sustainability as a concept to any aspect of development or the environment is its widely varying meanings. In the popular literature sustainability has been characterized as "not borrowing against the future." Thus ***sustainable development*** might be characterized as that which by design, development, financing, and long-term management ***avoids irreversible effects*** such as 1) accelerated or total ***depletion of a natural resource,*** 2) ***elimination of options*** for future development or conservation, 3) ***escalation of costs to prohibitive levels,*** and 4) significantly ***increasing the probability of a catastrophic disaster,*** either natural or technological.

However, in seeking models of sustainable development it is easier to identify infrastructure related decisions which have raised sustainability challenges either because of incomplete knowledge of possible outcomes at the time they were made or because of unforeseeable changes in circumstance. For the purpose of discussion, two examples have been selected from the immediate post-World War II era, one in the U.S., the other in Japan. Sustainability challenges raised in these examples are associated largely with unexpected problems which subsequently emerged from these early decisions made a half century ago.

Abandonment of Red Car System

In the decade following World War II, public and private decisions were made to abandon the extensive rights-of-way for the Red Cars, a basic regional trolley network founded by entrepreneur Henry Huntington in the early part of the 20th Century prior to the proliferation of automobiles (see Figure 1). The Red Car lines spread over a vast area, connecting hundreds of Southern California neighborhoods and communities. One major reason for the system's abandonment was the conflict between trolleys and the growing number of automobiles at street crossings.

That former system is now being virtually recreated in the form of the new MetroRail mass transit system which has been developed and is under construction along many of the same corridors -- in some instances using the same rights-of-way (see Figure 2). The cost for developing the new MetroRail system, in the hundreds of billions of dollars, is probably much greater than costs required to grade-separate former street crossings had the Red Car system been retained.

The huge budgets for this mega-project are controversial now because of competition with financing needed to run the bus system on surface streets. At this point, there is a question whether the value gained from rebuilding a regional rail system can approach the value lost when the original system was abandoned. During the intervening half-century, Southern California grew in a more diffused pattern than might have emerged had the Red Car rights-of-way been kept.

Keihin "Industrial Land Tsunami"

An example emerging from the same era in Japan might be found in the massive post-World War II expansion of the Keihin industrial belt into Tokyo Bay. The image of a tsunami is of a great wave of water sweeping over the land. The Keihin development was like a tsunami in reverse, extending the land over the sea, though more slowly.

Figure 1. Pacific Electric Railway Company Lines in the early 1900's

From Ride the Big Red Cars- The Pacific Electric Story by Spencer Crump, 1983 Trans-Anglo Books

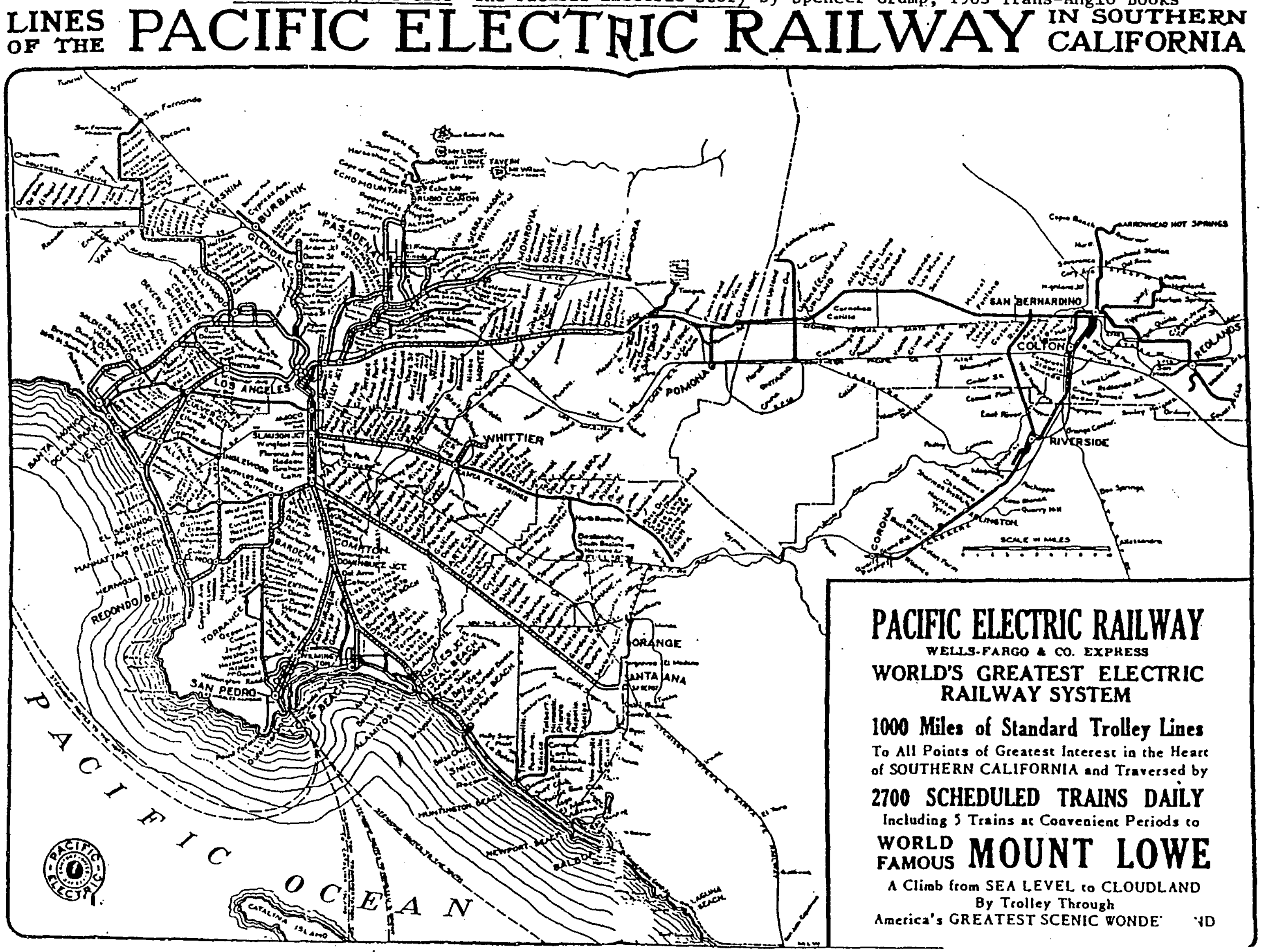
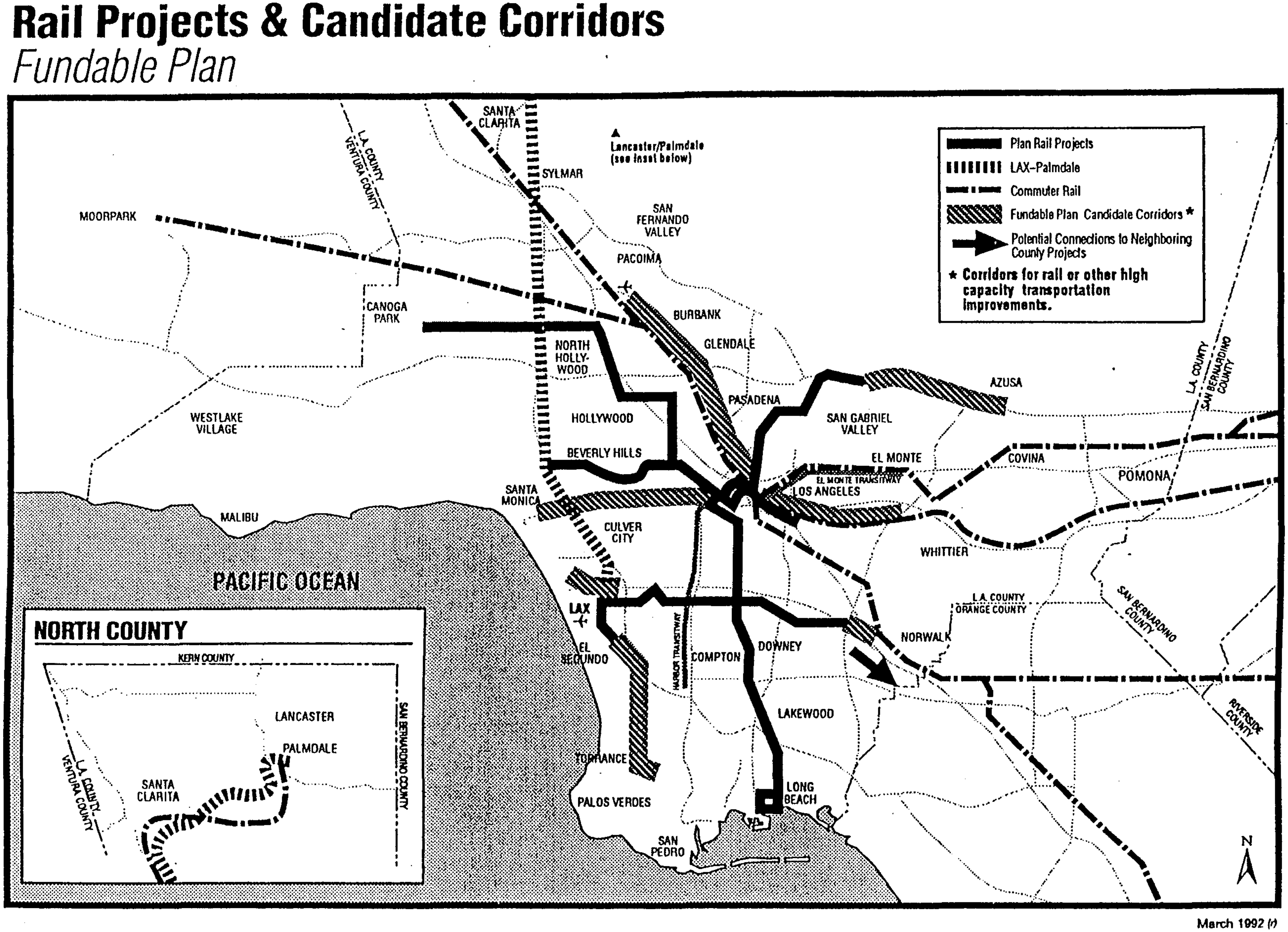


Figure 2. Rail Transit Projects

Source: Los Angeles County Transportation Commission



Part of a larger land reclamation pattern on the perimeter of Tokyo Bay, the Keihin industrial development swept into the sea, extending the shoreline with landfills from Tokyo to Yokosuka. This massive construction of factories on extended land dislocated the local marine-based economy typified by fishing villages and the seaweed industry. It also severed a historical connection between the people and the sea in cities such as Kawasaki, Yokohama, and Yokosuka.

For over three decades, the new Keihin development represented an essential foundation for Japan's emergence as a world economic power through creation of products sent around the world. Then, within the past decade, heavy manufacturing began to decline and move away from the area to other countries. Like a tsunami, the great wave of industrialization began to recede, leaving large abandoned and underutilized areas in its wake.

Now, substantial sums are being spent by national, prefectural, and city authorities on renovation and redevelopment of the Keihin area to give it new economic, functional and social utility. This is demonstrated by the addition of expressways and bridges connecting reclaimed land masses and recreating of marine recreational areas as well as introduction of mixed use communities along the waterfront to reconnect people to the sea.

Due to the variable quality of dredge and fill reclamation techniques of the previous decades, however, some landfill areas are settling and would suffer severe deformation and sea wall failures in the next great earthquake in the Kanto region which is expected someday. Therefore, infrastructure systems and redevelopment projects built in this area are likely to bear additional costs for soil stabilization, strengthened foundations, and other earthquake hazard reduction measures in order to avoid future human and economic losses in particularly vulnerable portions of the area.

Sustainable Infrastructure

In assessing the meaning of these preceding examples, it might be reasonably argued that it is impossible to fully anticipate future costs and problems related to current infrastructure decisions. Rapid and extensive technological and socio-economic changes make perception of long-term future consequences particularly difficult. Aside from the numbers which may have justified a direction being taken, development of certain major public works (such as the Panama Canal) have often been based on a <u>blend of visionary as well as practical thinking</u>.

From the discussion so far, it can be seen that planning and engineering professionals and the scientific community do not yet have a clear model of sustainable development. Nor do we have specific understandings of the meanings of this relatively new concept in relation to infrastructure systems design, development, finance, and management.

What are elements of sustainable infrastructure development? From more recent thinking, the concept of sustainability as applied to civil infrastructure can be seen as connoting a mixture of 1) life cycle engineering and costing, 2) market-oriented strategic planning, 3) place-based analysis, 4) information technology development, 5) linkages to land use and development planning, 6) long-term system financing, 7) project management and implementation, 8) operations and maintenance, 9) risk management, and 10) training and institution building.

Life Cycle Considerations

Issues considered might include traditional tradeoffs between ultimate design requirements for maximum benefit vs. short-term costs, desirable vs. practical service levels, optimum coordination with other infrastructure systems vs. the need to proceed expeditiously, long-term vs. incremental capital financing, full accounting in advance of maintenance and operations requirements including repair costs vs. expedient budgeting which ignores such factors. In its broadest expression, however, the concept of sustainable development implies balancing of such issues in favor of long-term considerations, benefits, and user interests.

Infrastructure planning often does not fully take into account such considerations, This is especially true when considering the long-term operations and maintenance costs which might influence project design were such costs known and fully taken into account at an early stage. Short-term constraints, such as available budget, tend to discourage consideration of long-term functionality and maintenance factors. This results in accelerated decline or obsolescence of facilities at an unnecessarily early stage. This is especially a problem where technological changes require flexibility of thinking with regard to aspects of future system design which existing professional standards do not necessarily take into account.

Emphasis on short-term thinking leading to suboptimization also does not include sufficient awareness of relationships between infrastructure systems on the part of the organizations building the systems. An example of both good and poor coordination of inter-system facility design in Los Angeles is the fact that on the positive side, the fairly new MetroRail Green Line light rail system operates within the median of the 105 Freeway. On the negative side, however, the Green Line goes near but not directly into Los Angeles International Airport.

Evolution of Master Plans

During the 20th century, infrastructure system design has increasingly been linked to master plans of various kinds. These include master plans for single systems such as water, sewers, streets, parks, and so on as well as master plans for cities and regions. The latter are compiled in a comprehensive format which identifies relationships between separate systems and also their relationships with land use. In both kinds of master plans, the systems are generally shown in their ultimate capacity for a point in the future representing the maximum buildout of a city or region.

In both the U.S. and Japan, such end state concepts of system development have begun to be supplemented by mid-range plans which identify more detail for intermediate phases of system development or city and regional growth. Now, on one side of planning thought is the traditional master plan approach which emphasizes system design providing capacity for ultimate demand and anticipated contingencies. On the other side is the recognized need to realistically assess the near-term demand and financing in order to avoid building a system which because of technological or societal changes is used only to partial capacity.

An example also from Los Angeles of use only to partial capacity is Union Station, a central railroad station built in 1939 as a prototype of the ideal station. Although used heavily during World War II, during an era of severe gasoline rationing, Union Station was rendered obsolete in subsequent decades with the growth of air travel. Since World War II it has never reached capacity use although it now functions at a somewhat higher level with the expansion of regional commuter rail service to outlying counties.

Influence of Strategic Planning

Evolution of the master plan concept to include intermediate as well as ultimate time horizons has more recently blended with the private sector inspired idea of "strategic" planning. Rapid change of nations throughout the world from socialized to more market-oriented economies has brought strategic planning into greater prominence. Although strategic planning can fit within the framework of traditional master plans, it is more near-term oriented and fundamentally more flexible. Strategic planning identifies long-term functional goals in a manner somewhat similar to traditional master plans. However, it places more emphasis on clear articulation of specific intermediate system development and economic objectives. Based on strategic thinking with which private businesses enhance their competitive position, strategic planning examines obstacles, opportunities, and options offered by changes in the market place and social fabric of a city or region and then focuses on defining a series of detailed actions for expediting fulfillment of near-term objectives, together with assignments of responsibility for those actions.

Place-Based Analysis

Additionally, strategic planning is much more "place-oriented." It emphasizes system development which takes maximum account of the unique competitive advantage which a city or region may possess in comparison with other areas. Rather than possibly diminishing a specific advantage through imposition of uniform standards devised for typical situations, place-based strategic planning emphasizes adaptation of system design to enhance an area's special qualities.

For a hypothetical Japan example, were a new freeway to be built through a region having unique beauty and cultural value near Mt. Fuji, a place-based strategic planning approach might result in a design for the facility with somewhat different horizontal or vertical cross-sections harmonizing the visual impact of the facility with its surroundings than for a facility being developed in an area lacking these special qualities. While this might be done in any case for aesthetic reasons, a place-based strategic planning approach would emphasize the competitive economic advantage of avoiding damage to such amenities.

Information Technology and GIS

Strengthening the effectiveness of place-based thinking in both strategic planning and facilities systems design, development, and management is the evolution in information technology including geographic information systems (GIS). A geographic information system (GIS) combines automated mapping with relational databases to make it possible to develop accurate maps, measurements, statistics, and comparative analyses regarding a variety of development-related subjects.

For example, mapping which might be useful to an infrastructure project might include such related features as parcel and administrative boundaries, roads, topography, and rivers as well as the project's own design characteristics. Related statistics and records might include parcel acreage, land ownership status, land prices, traffic volumes, landscape and environmental conditions, master plan and zoning acreages, grading volumes, and parking ratios.

GIS can conveniently generate spatial and interpretive analyses about relationships between map features and corresponding numbers, records, and statistical projections in a manner which better informs the user about their meaning. It can also be used as a foundation to host computer aided drafting and design (CADD) and other related digital information tools using geography as the integrating factor.

Although the evolution and development of practical applications of GIS has been somewhat faster in the U.S., there are now many sources in Japan of digital data which are available for the metropolitan areas. These are maintained by various central government, prefectural and local authorities. Examples include the substantial land databases which have been assembled by various Japanese government agencies for the Tokyo metropolitan area with the support of private sector firms.

Benefits of GIS

There are many benefits to developing a GIS database for infrastructure design, development, and management. Given an initial investment in database development and integration, a GIS can be an effective tool for providing, updating and maintaining timely and accurate information for project management to guide action. It represents a valuable resource for coordinating contributions of multiple consultants to project design and development, and it provides the spatial data manipulation and graphical product tools to support communications with government officials and the public.

Preparation of a GIS database for an infrastructure project enhances development management and operations throughout the life of the project. Information needs evolve with each phase of a project life cycle. Within this longer term frame of reference, it is possible to derive a variety of interpretive spatial and statistical analyses supporting each phase of project design, development and management decisions.

Analysis at each phase involves different map scales which determine database development and management requirements. Database content tends to be simpler in the early phases, becoming more detailed and complex in succeeding phases. For example, a map scale of 1:25,000 might be suitable for initial feasibility studies and a scale of 1:10,000 may be suitable for preliminary design analyses, whereas a map scale of 1:500 is needed for detailed pre-construction drawings (see Figure 3).

Linkage to Land Use Planning

Another important component of sustainable infrastructure development is linkage to land use planning. Aspects of this component include development monitoring and trends analysis, modeling and options evaluation, development planning, and investment programming.

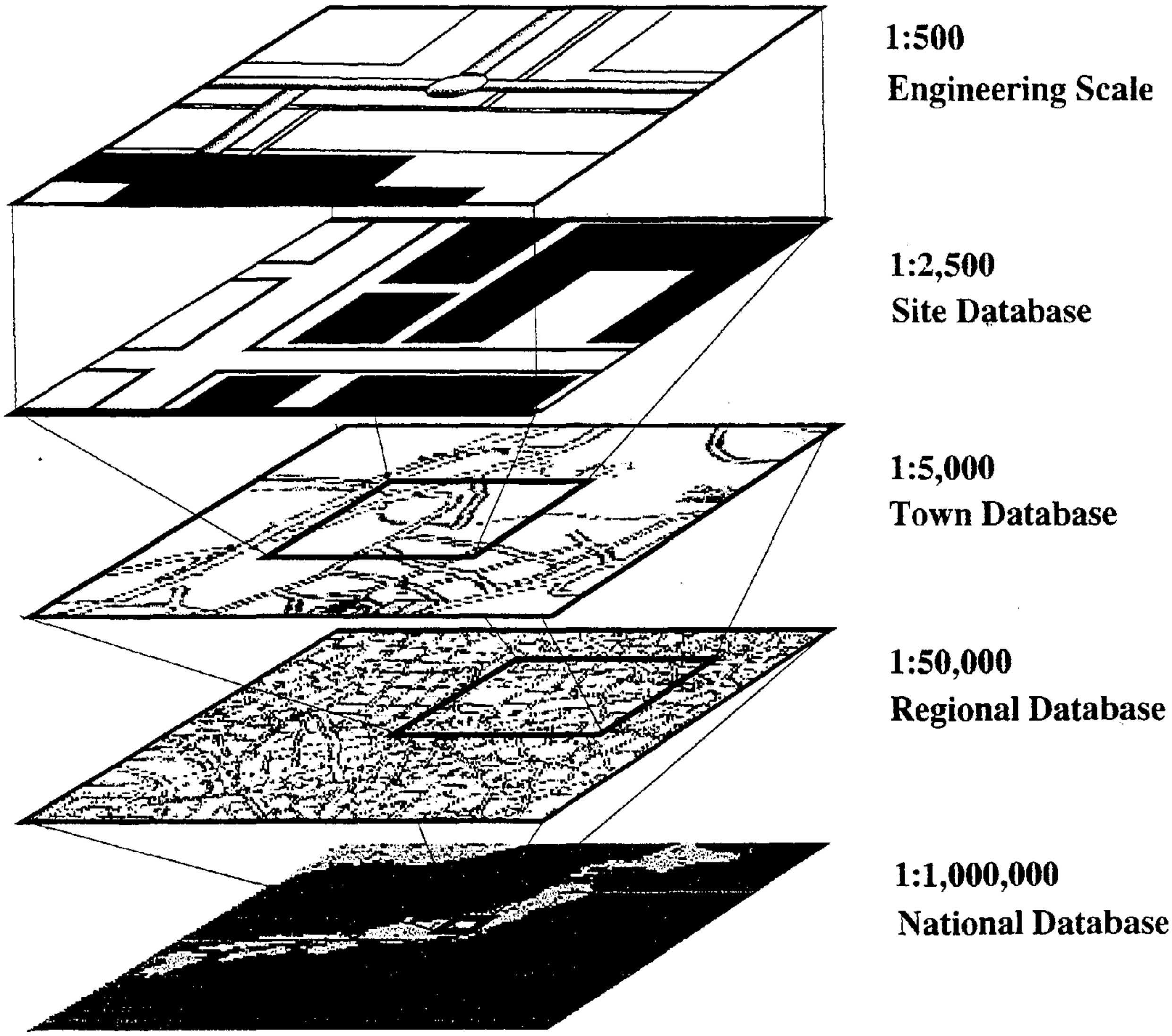


Figure 3
Mapping Scales

1. Development monitoring and trends analysis. This involves monitoring and evaluating market demand for infrastructure caused by quantitative, qualitative and locational shifts in urban development. It also involves determining the capacity of the infrastructure system to meet market demand, projected life cycle of this capacity, and possible social, economic, financial and environmental impacts of deficiencies in meeting this capacity.

2. Modeling, options generation and options evaluation. This involves use and integration of available automated models, where available. Attention should be given to formulating reliable relationships between transport and land use modeling, together with the possible relationship of water demand modeling.

3. Development planning. This involves spatial planning and analysis for land use changes in areas experiencing rapid economic and urban growth as well as for areas which are undergoing public or private redevelopment. Examples of such analysis include:

a) Zoning regulations which govern the height, bulk, density, land use intensity, and placement on development sites of buildings, and related activities;

b) Urban design controls which specify the architectural treatment, relationship between public and private spaces, vistas, landscaping, signage, and general texture of development;

c) Special districts placed on selected areas which impose specialized land use regulations, development standards, or design review procedures due to identified historical, symbolic, and strategic values associated with an area;

d) Transferable development rights involving creation of systems and procedures for transferring the right to develop a given area to another area in order to protect unique agricultural, environmental, historical or other special values;

e) Developer agreements by which developers are permitted to enter into an agreement with the local government to develop in a certain way provided certain conditions are met, protecting the development from later changes at the policy level which might destroy its feasibility;

f) Environmental standards which form the basis for such regulations as floodplain zoning, air pollution controls, and effluent discharge controls imposed on the entire city or those portions affected by the presence of certain existing environmental conditions or anticipated adverse impacts from development;

g) Fiscal incentives such as tax reduction agreements, lease concessions, and other joint development arrangements intended to stimulate private development in exchange for long-term job, income, and revenue generating benefits of such development;

h) Infrastructure financing districts which assess specific properties for benefits gained through agreement of property owners to contribute a certain annual service fee in return for improvements such as roads, drainage, public landscaping, or street lighting;

i) Impact analyses and mitigation consisting of cost recovery through fees of costs to the municipality of infrastructure development or environmental mitigation.

4. Investment Programming. This involves development of financial policy instruments and analytical, programming and management tools including: techniques to appraise the viability of infrastructure projects, determine their prioritization and schedule their implementation, and develop tools to estimate costs of capital investment programs; ways to identify, evaluate and select financing and cost recovery options for construction and operations and maintenance (O &M), with an emphasis on various partnership arrangements with nongovernmental entities; and techniques for multi-year budgeting which allows for revenue forecasting, taking into account non-traditional sources of funds.

Project Management

Also important to sustainable infrastructure development are certain project management considerations. Project management is generally a well known engineering and administrative discipline with certain phases in common for both new capital projects and O & M projects. These are classified essentially into two parts: project planning, and project implementation, with each having five common steps. Project planning steps include:

1. Project identification. Ideally, this should be done within a strategic planning framework which weighs options regarding capital projects and investments in terms of their relative near-term physical, social, economic, and environmental benefits in their relationship to a long-range comprehensive master plan. In practical terms, however, project identification often proceeds along a separate path, not always congruent with the outcome of a strategic planning approach. Sometimes the outcome is widely divergent, with negative consequences in terms of the timing, impact, and value of the investment.

2. Project feasibility analysis. Ideally, project feasibility should be assessed in terms of longer term economic, financial, technical, and environmental feasibility elements associated with strategic planning process models. In practical terms, feasibility determinations are sometimes made on the basis of expediency, either from an overly narrow engineering or construction cost perspective or because of the pressing influence of other projects and policies which may not have been subject to scrutiny for their relative benefits and consequences.

3. Project definition. Once determined feasible and having received formal approval, attention quickly turns to the detailed definition of user requirements, timelines, costing, and financing. Ideally, these considerations are linked to strategic planning processes, particularly in terms of prior identification of appropriate project financing which harnesses some of the entrepreneurial energy and resources in a municipality. In practice, however, project financing becomes an end in itself after so much effort is expended on the details of the project. Often the weight of early commitments to these details leads to a tendency to "force fit" project financing back onto capital budgets and improvement programs in a manner which diverts funding from other, perhaps more justifiable, projects.

4. Project design. At this point in project planning, plans are entering the nearly final phase prior to implementation. Ideally, this and preceding phases should be informed by existing GIS data which provides a smooth transition from 1:10,000 contextual mapping to the 1:500 detailed pre-construction drawings. Practically speaking, in many cities such mapping and/or GIS databases are not consistently developed, raising the cost of this phase for special surveys and related field observations necessary to tie down projected facility locations, alignments, configurations, and costs.

5. Project construction documentation. This is characteristically the final step in project planning prior to project implementation. Ideally, this phase is informed by a consistently accurate and detailed digital database. It often suffers, however, from the same practical database problems experienced in phase four. The consequences of insufficient field data upon which to base such drawings include subsequent delays, resurveys, and expensive change orders which tend to escalate project costs after construction is well under way.

Project Implementation

Project implementation includes steps common to both new capital projects and to O & M projects. These are similarly affected by the growing need for better strategic planning and Information technology applications.

1. Project procurement. In the absence of a suitable mapping and/or a detailed, multi-functional GIS database this phase also can lead to downstream problems. Bidders not wishing to place themselves in jeopardy may not reveal important questions about field conditions bearing on costs until after selection has taken place or construction is underway, leading to costly change orders and delays.

2. Project construction. Ideally, construction should take place on time and within budget. However, in many cities construction is often accompanied by inadequate processes and procedures for content, cost, time and management planning, monitoring and control. This inevitably leads to unanticipated cost escalations and other negative ramifications. Extremely large, multi-year projects are especially susceptible to these problems.

3. Project commissioning. By the time a project is commissioned in formal ribbon-cutting ceremonies, problems originating in the project planning phase or associated with its selection, planning, and construction are often pushed aside due to sensitivities of those involved with the decision or development process. The greatest concern of policy level and technical level decisionmakers may be the commissioning of a project which , when built, is criticized for cost overruns or quickly proves not to be cost-justified due to overestimation of need. To the extent that such problems are avoidable through more adroit strategic planning in the early phases, resources can be saved for deployment on other infrastructure projects having merit and such concerns can be avoided.

4. Project operations and maintenance. Similarly, unexpected and unwelcome O & M costs can be reduced in the long run by more careful early linkage to strategic planning and database development.

5. Post evaluation. With adequate strategic planning and information systems management, post evaluations will more readily justify the project planning, selection, construction, and O & M processes.

Operations and Maintenance

While O & M appears to be a routine aspect of infrastructure development and management, it is nevertheless a crucial component. The ongoing costs of maintaining existing systems in various phases of their lifecycles and the need to replace some systems can be in direct competition with the need to develop new facilities. O & M can be unexpectedly complex, depending upon such factors as the age and condition of existing improvements, demands placed upon their use by new development, differential rates of deterioration, deferred maintenance, vulnerability to natural disasters, and availability of restoration and replacement financing.

1. Age and condition. Cities grow in cycles, sometime with very rapid development during a particular era, and sometimes very slowly during another. Infrastructure improvements installed during periods of rapid growth tend to be given less attention from an O & M standpoint than older improvements more obviously in need of repair due to other demands on capital investment.

2. Demands from new development. Infrastructure installed in one era may not have been sized for demand conditions stimulated by a new wave of development many years later. Demand levels may far exceed those which could have been foreseen when first installed. This problem can emerge for large areas of a city or for individual districts undergoing new entrepreneurial pressures.

3. Differential rates of deterioration. Similarly, infrastructure deteriorates at different rates over time depending upon a variety of influences including initial design standards, adequacy of construction, weather, or damage due to some unexpected event such as a natural disaster.

4. Deferred maintenance. Absence of systematic monitoring of routine O & M processes can result in accumulation of deferred maintenance requirements having multiple implications. Most critically, continuous deferral of normal maintenance can accelerate the deterioration process, increasing repair and rehabilitation costs when the work is finally done. These costs are compounded geometrically in proportion to the number of facilities for which normal, routine maintenance is deferred, resulting at some point in a decision crisis when systemwide deterioration reaches critical proportions.

5. Replacement financing. Unless carefully scheduled on a sustained basis through annual capital budgeting and multi-year capital programming, financing deficiencies for ongoing repairs of a city's infrastructure can accumulate, confronting decision-makers with overwhelming no-win dilemmas.

Infrastructure Financing

The purposes of creating a fiscal mechanism for long range infrastructure financing is to ensure that the needs generated by new growth can be met in a timely manner, that deficiencies can be corrected over time, and that funds for maintenance will be sufficient to avoid deterioration and decay. Inadequate and deteriorating infrastructure can undermine a nation's economic viability in world commerce.

Although a normal function of government, infrastructure financing is sometimes dependent upon contributions from the private sector. The history of infrastructure financing in the U.S. reflects various stages in this regard. During the initial stages of national development, large sums were spent by private sector interests to build roads, canals and railroads for which tolls were charged to recover investments and make profits. Over time the burden of infrastructure costs was shifted to the public sector. Federal programs initiated before and after World War II culminated in massive freeway, highway and mass transit projects which were financed by gas taxes and other means.

The pendulum has now begun to swing back away from primary reliance on public financing due in large part to two related influences. The first is the popular anti-tax movement, started in the 1970s, which resulted in tax limitation initiatives passed by voters in states like California. The second is the recent effort at the national level to reduce federal expenditures.

The lack of sufficient infrastructure or an unintended delay in its availability can be a negative factor in a city's ability to accommodate growth and can lead to deterioration of existing infrastructure from overuse. Various types of funding mechanisms such as bond sales and special assessments are needed in addition to property and sales tax revenues to address existing deficiencies and attain the desired level of service for all citizens of a city at projected buildout.

If no action is taken, increasing infrastructure needs go unmet. This in turn expands deficiencies and reduces alternative financing options as unmet needs increase. In short, the fundamental consequence of inaction is accrual of overwhelming long-term infrastructure deficiency problems, in turn reducing the desirability of a city as a place to live and do business.

Infrastructure development fees can be useful as a tool for funding certain types of infrastructure. However, because of difficulties in public finance created by passage by voters in 1978 of Proposition 13, the tax limitation initiative, cities are now facing a dilemma. Even if they pass developer fees, they still must find other means of providing funds for existing infrastructure deficiencies because development fees by law can only be used to offset the costs of future growth. Other long-term infrastructure financing measures such as bonds require a two-thirds vote of the people which is hard to obtain. Cities are also reluctant to impose development fees at 100% of cost recovery because of the negative economic effects on development projects.

An important lesson for other states and nations from California's experience may be that even though development fees are used to offset the costs of new facilities, the short-term economic benefits to taxpayers of tax limitation initiatives may have an opposite effect than intended in the long-term by greatly increasing infrastructure deficiencies. If other financing means are not found to correct existing and future deficiencies, the cumulative effects may be the creation of seriously deteriorated infrastructure conditions which may drive growth away as well as the escalation of future remediation costs to unmanageable proportions.

Vulnerability to Disasters

A final component of sustainable infrastructure development is the issue of risk management related to natural and technological hazards. The concept of sustainability is now being expanded to encompass pre-disaster mitigation of technological and natural hazards.

Infrastructure is a vital lifeline during natural disasters such as floods and earthquakes. Yet insufficient information about the character of hazards or unwise decisions ignoring such information can unnecessarily lead to damage and destruction, extensive delays in post-disaster recovery, and unnecessarily expensive reconstruction of facilities over extended periods of time. Under post-disaster circumstances, issues of project finance are exacerbated by unusually high repair and replacement costs which must absorb the additional mitigation costs at a time when the local economy is already seriously dislocated. In such circumstances, external sources of financing such as from national, state, or prefectural governments are often found to be insufficient to cover the full range of disaster generated costs.

In the U.S. in recent years, it has been realized that the billions of dollars in disaster reimbursements paid out under the Stafford Act unintentionally served as rewards to localities and governments for not mitigating known hazards such as floods, landslides, and earthquakes. This realization has helped motivate the Federal Emergency Management Agency (FEMA) to initiate a National Mitigation Strategy which will place new incentives and obligations on states and local governments to commit to encouraging local pre-disaster mitigation. This National Mitigation Strategy program encourages the new concept of disaster-resistant communities.

Disaster-Resistant Community Design

The broad purpose of the disaster-resistant community concept is to create new communities and infrastructure systems in an overall pattern which is more resistant to the effects of natural disasters and to mitigate existing hazards to avoid such future destruction and costs. Disaster-resistant community design represents a rational approach to reduce life and property losses and avoid the time, costs, and barriers to productive living associated with post-disaster recovery and rebuilding.

The main strategy for disaster-resistant community design is to apply lessons learned from disasters in other cities around the world through intelligent layout of streets, utility systems, open spaces, and urban development. Examples of disaster-resistant design techniques include: setting buildings back from flood, landslide and fault hazard zones; requiring adequate minimum paved street widths; limiting street grades to assure fire truck access; requiring second access points into each development in case of blockage of the primary access during an emergency; restricting lengths of dead-end streets as well as the number of dwelling units on them; developing adequate water supply and fire flow and redundant storage locations; and using open space easements for fire breaks, equipment staging, and evacuation areas.

A major challenge for infrastructure development with regard to disaster-resistant community design is to catalog and refine such mitigation techniques, introduce them into city planning and engineering practice, and train engineers and planners in their use. This brings up the final issue of training and institution building.

Institution Building and Training

It is important to recognize that all of these sustainable development practices require strengthened management information systems (MIS), management, and training. Sophisticated methods of analysis, comparison, decision-making, monitoring, problem solving, and negotiating are required to develop and manage complex infrastructure systems. Development of GIS and other information technology applications will help but not be sufficient unless matched with training and institution building components.

For long-term progress in sustainable infrastructure design, development, financing, and management, it will be necessary to assess and develop new skills and institutional competence in public-private partnerships, contracting, capital financing, and market-oriented management. These assessments should define mandates of various stakeholders, examine organizational structures and reporting relationships, and identify upgrading of human resources, information, and facilities that will be required to undertake strategic planning on a sustained basis.

We need to promote training which highlights linkages between strategic planning, investment programming, budgeting, project development, O & M, and risk management knowledge, as well as training which promotes attention to inter-institutional relations. In both institutional and training needs assessments, attention should be focused on real world problems and challenges faced by specific municipal organizations with attention to solutions being developed elsewhere. This appears to be true not just for cities in the U.S. and Japan but for cities all over the world.

Research Agenda

From this preceding review, the essential components of sustainable infrastructure development are all appropriate candidates for future joint U.S.-Japan research:

1. Life cycle engineering and costing;

2. Market-oriented strategic planning;

3. Place-based analysis;

4. Information technology development;

5. Linkages to land use and development planning;

6. Long-term system financing;

7. Project management and implementation;

8. Operations and maintenance;

9. Risk management;

10. Training and institution building.

Among these research areas, additional attention is especially needed to applicability of the sustainability concept to the following aspects of infrastructure development and financing:

a) information technology development, particularly GIS,

b) land use planning,

c) risk management, especially disaster resistant community infrastructure design.

It is hoped that the preceding concepts will contribute to a meaningful research program as a result of the U.S.-Japan Joint Seminar on Civil Infrastructure Systems Research.

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ECONOMIST-ENGINEERING FRAMEWORK FOR SUSTAINABLE DEVELOPMENT: PRACTICAL ASSESSMENT OF INFRASTRUCTURE INVESTMENT IN TERMS OF PRODUCTIVITY AND SUSTAINABILITY

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ABSTRACT

To attain optimal investments in infrastructure in terms of sustainability and productivity, engineers must account for socioeconomic features in their design framework. The engineering approach to solving problems as a closed system must change to an open system approach, which includes unknown factors.

The environmental impact assessment of private developments and public works are sometimes expected to be too idealistically intensive to be realistically applicable. Assessment procedures must account for the environment and, at the same time, be affordable for the developers. Assessments should be conducted according to economist and engineering practical and objective procedures rather than political motivations.

With respect to the fairly complex characteristics of this problem, a simplified framework and some proposals for a different engineering approach will be discussed in this paper, including:

1. Recent criticism of public works in Japan
2. Need for proper methods of assessment
3. Proposal for developing a system to share assessment costs
4. Other proposals on theoretical problems and data acquisition

INTRODUCTION

Problem 1: Criticisms of Public Works in Japan

In Japan, the public investment for infrastructure has recently been criticized frequently and severely concerning inefficiencies and neglecting to account for their impacts on the surrounding environment. Development in general, including in the private sector, has been continuously criticized over a long period of time for their destruction of the surrounding natural environment.

However, generally speaking, public investment for infrastructure was believed to be a positive mechanism to shift the depressed economy into a better condition. The public believed that these investments were necessary for the sake of their welfare and for economic prosperity.

Nowadays, journalists or other mass media cannot be expected to support development or public investment. The public in general is also no longer as generous regarding public investment in infrastructure. The cause of this shift in public opinion is due to several factors, including ineffective public works projects on the long-depressed economy, relatively high cost of public works, widely reported scandals involving corrupt government officials, and many cases of wasteful spending on infrastructure.

Sometimes, infrastructure construction faces strong protest, reasonably or unreasonably. There are many examples of public investments which have had problems during development, including accusations of exterminating rare species, destroying the natural environment, emission of nuisances, cost/benefit inefficiencies, overlapping investments, and so on. Apparently, new infrastructure development must be planned and constructed much more carefully today than in the past. Projects must be clearly shown to be necessary, efficient, have positive environmental impacts, and have global sustainability.

ENGINEERING APPROACH TO SUSTAINABLE DEVELOPMENT

Problem 2: Need for Proper Methods of Assessment

Methods to evaluate public works projects objectively in engineering terms rather than as political terms need to be developed. Both the planning (promoting) side and the protesting side must be able to discuss the issues together.

Concerning direct environmental impacts, the interests of both sides are very different and each side has valid reasons for pursuing their interests. However, they can negotiate with each other to develop mutually beneficial compromises.

Global problems are more difficult for the opposing parties to discuss objective data and reliable theories. At present, solutions to deal with global sustainability are not easy to find, and there is a lack of realistic theory and reliable data which clearly explain the relationship between individual development and global sustainability. There are no established systems which can treat these assessment procedures quickly and practically.

Engineers are usually involved in the planning (promoting) side. They naturally concentrate their efforts on solving the details of a project — safety problems, efficient construction, minimizing the costs of construction, etc. They usually are not as concerned with the global environment. Most engineers were not trained to deal with the environmental impacts of their projects, especially on issues of global sustainability. I believe that most engineers understand that they must account for more global issues when working on infrastructure projects; however, they do not have the means or access to proper information to effectively change their approach.

At the same time, establishing effective control against environmentally doubtful developments is a political issue. It includes the regulation of free economic activity and adjustment of conflicting interests of different groups of people. In Japan, environmental impact assessment law has not yet been successfully enacted. The organic law on environment impact was recently enacted in 1993. Some local bylaws are requesting environment impacts assessment.

PROPOSAL FOR DEVELOPING A SYSTEM TO SHARE ASSESSMENT COSTS

Comments on Developing a System to Share Assessment Costs

Environmental impact assessments must be intensive enough to determine all aspects of the influence of the development, including direct and indirect effects, and socioeconomic impacts. The following items should be included in the assessment:

1. All relative assessment documents should be submitted before any decision to proceed with a particular project is made.
2. Alternative methods of development should be investigated and compared.
3. These documents should be reviewed by an independent specialist.
4. Cost/benefit analyses of the project should be prepared.
5. An evaluation of the influence to the surrounding environment, global environment, species protection, etc. must be addressed.
6. More infrastructure projects should be required to prepare an assessment.

Probably, such an intensive assessment would be meaningful only when it is assessed in the context of a total system of global sustainability or total system efficiency. Even though some public works have been intensively assessed, these assessments have less meaning if the project is not appropriate in terms of the total efficiency or global sustainability. It is not easy to assess all individual public investment projects so rigidly, because to do so requires substantial labor, time and cost. If intensive assessments were compulsory for all public investment projects, executing any public works project could become inefficient.

Thus, how should projects be categorized in terms of assessment? Should only doubtful projects be selected, or should only the big projects be required to have such assessments? Doubtful or big projects are of course important, however smaller projects are also important because there are many of them and their combined impact on the environment can be fairly large. I suspect that there must be a more reasonable way to determine a fair and reasonable method of cost sharing.

As one possible solution, I would like to propose a realistic system of assessing development or public investment projects. Public works and private developments have a kind of hierarchy structure. Each individual project would be an element of a tree-shaped structure. The assessment of each public works project should be evaluated in the context of this structure. For example, road construction can be viewed as layers of assessment, from the higher level to the lower level, as follows:

Transportation systems	Upper total system alternatives
Network of road	Subsystem
Road construction	Individual works
Design of road	Lower alternatives

To obtain a realistic assessment of the impact of some of the lower works in this structure, all levels of assessments must be performed for this relatively small project. However, all the individual works need not be assessed so intensively as the first sample. Probably, the top levels of this structure should be assessed independently from the individual projects. At the same time, all the projects need not be assessed at the same level in the same way.

Each project should be assessed at a reasonable level, when it is considered to be necessary. The first assessment in some category or in some big projects must be checked and evaluated very carefully and intensively including alternative approaches, in terms of efficiency, direct environmental impacts, cost/benefit, global environment, etc. The assessment of the following similar works could be simpler than the first ones.

There are problems sharing expenses among projects of different intensities. An averaging system under NPO (Non Profit Organization) could be applied to provide a reasonable cost distribution. By applying such a system of assessment, the relationship between total systems and individual small projects or public works can be better understood in terms of productivity and global sustainability. Costs caused by wasteful assessment procedures can be avoided.

THEORETICAL PROBLEMS OF SUSTAINABLE DEVELOPMENT

Comments on Theoretical Problems as Tools of Engineering

Theoretical problems concerning assessment are substantial. The following is a list of some of the problems:

1. Range of assessed items

   Not only direct impacts of the development, but indirect influences must be assessed. An effective assessment should cover the following items.

   a. Direct demand for labor and materials due to construction
   b. Indirect demands, which could be constructed by input-output analyses
   c. Direct impacts to the surrounding environment
   d. Impacts to the global environment
   e. Same items concerning the maintenance and abolition of the infrastructure
   f. Necessity or effectiveness of the projects

2. How to allocate each partial effectiveness into the total system

3. Principles of adjusting international differences concerning global environment

4. How to treat uncertainty factors which must be forecasted. For example, future usage of some infrastructure is a crucial factor in evaluating the effectiveness of its construction. It is also crucial to assess the indirect impacts of the infrastructure.

5. Development of a reasonable and acceptable system of cost sharing to perform the assessment among the projects.

6. How to measure the productivity or efficiency of infrastructures. They are very different from other economic activity or products.

7. What kind of data are necessary and how can it be accumulated?

MEASUREMENT OF PRODUCTIVITY

Measuring the Productivity of Infrastructures

One of the most difficult questions is how to measure the productivity or efficiency of infrastructure. They are very different from other economic activity or products and their productivity can only be measured by more flexible definitions.

The unique features of infrastructure can be thought of as follows:

1. There are no perfect private markets for usage.
2. Each infrastructure is not independent from other infrastructures.
3. Like roads or railways, there are no definite boundaries between them.
4. The users are public and productivity is decided by usage by the people.
5. The public sector owns them.
6. Usually, they are free to the public.

Efficiency in terms of productivity and effectiveness in terms of sustainability are sometimes not proportionate with each other.

For example, the consumption of energy and cost of travelling by different route can be proportionate and the effect of a new route can be measured on each user's merit. However, at the same time, the number of the users are decided by other factors. In this case, the effect of a new bridge is decided according to public attitude or population distribution, etc.

DATA PROBLEMS

Data Acquisition and Expected Roles of the Economist-Engineer

To precisely evaluate the effect of public investment, statistical data and other technical coefficients must be developed which can be accumulated by national organizations or another independent body. To consider how such useful data can be obtained, we can refer to two existing databases. These data were accumulated and arranged by the Ministry of Construction, supported by many specialists including economists, engineers and other specialists. Both databases will provide us with information on how construction activities will influence other economic activities:

1. Disaggregated input-output table edited for the analysis of construction activities.
2. GENTANI (material and labor requirement per unit amount of work) data of construction works.

The I-O table provides both the direct effect of construction works and indirect demands caused by the various materials used in construction. However, it provides only the monetary coefficient and does not provide information on physical amounts. GENTANI provides the physical amounts of materials and labor per square meter for a building project and the same data per 1 million yen for other engineering projects.

However, both data sets are not accumulated to analyze global sustainability or measure productivity of the construction works. The direct consumption of energy can be estimated for each construction category, but information on maintenance or the influence of abolition works cannot be obtained. The emission of nuisances can also not be obtained.

The accumulation of such data needs the expertise of engineers, surveyors, or others who are familiar with the economic activities in general. The same difficulty may occur when the government tries to develop a new survey on the influences of constructed works to environmental problems or their productivities.

SUMMARY

When dealing with infrastructure assessment, it is easy to assume that this is the object of the activities of NGO, scientists and politicians.

However, ideas concerning efficiency or sustainability of infrastructure are much more difficult to obtain a clear conclusion than those of ordinary economic activity in the private sector. At the same time, it must be realized that engineers can best contribute to providing solutions for these serious problems.

Engineers need to change their attitudes so that they can provide realistic solutions to problems not only of traditional closed systems but also of open systems like global sustainability. In cooperation with socioeconomic specialists, engineers need to research theories that are more flexible and open-system oriented, based on reliable data. Engineers now have to search for acceptable methods to measure productivity or sustainability of investment to infrastructures, from the planning stages to the end of a given infrastructure's lifetime.

Public Policy Issues in Civil Infrastructure Systems: A Focus on Natural Hazard Risk Reduction

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ABSTRACT

Civil engineers find themselves involved in systems controlled by increasingly complex social, technical, administrative, political, legal and economic forces that simultaneously stimulate and constrain their actions, thereby having a direct impact on plans to improve the infrastructure. Controversies surrounding approaches employed by traditional engineering stem from differing ultimate goals and differing perspectives on increasingly complex urban problems rooted in fundamentally different methods of conceptualizing the nature and behavior of urban systems. These differences are significant because they influence policies, programs and procedures for achieving urban design. Effective engineering management requires that consideration be given to each of the constraints in the process of working to achieve an acceptable balance between quality of the human environment and the many conflicting demands for the use of scarce resources. To illustrate the complexity of the public policy process in which engineers must work, the following discussion will focus specifically on the issues associated with the development, adoption and implementation of natural hazard risk reduction policies, which in general are the same as those affecting civil infrastructure systems. Finally, the application of risk management approaches to facilitate the policy process will be discussed.

INTRODUCTION

Civil engineering is generally viewed as the application of technological-based solutions to infrastructure system needs or problems. Engineering is seen simply as a problem solving device based upon tactical and situation oriented decision making. The result is often an emphasis on changing the physical systems without concern for the larger social-political system. The question frequently asked by the engineer is, *Can* we *do it?* Thus, the approach to design and decision making depends on technologically based problem-solving in the narrow sense; the primary focus is on the end state where productivity is the key. This approach has generally been based upon an advocacy position that tends to promote competitive advantage, narrow interests, and limited scope, which generally fails to view the urban system as a set of complex interdependent systems/subsystems.

A total systems approach, important in achieving desired public policy, attempts to examine the relationships of system wide objectives and develop alternatives to guide society toward a coordinated set of goals. This approach builds on a process that does not provide the values, but provides a mechanism to help integrate the various stakeholder inputs and value sets. Decision making processes are to be open to allow for all inputs. This process addresses the question, *Should we do it?* This consistent pursuit of long-range goals may, however, give rise to serious conflicts between future interests and the interests that have motivated an infrastructure plan. It is important to note, in periods of rapid change, civil infrastructure engineers are locking future citizens into forms dictated by present values, not necessarily future interests.

Engineering planning, design and construction is dependent upon resource allocation and policy decisions made by a governing body, and must, therefore, be sensitive to evolving and changing public interest patterns, functional requirements and structural vulnerabilities of the built environment, legal and constitutional constraints, and the regulations that govern the design and construction process. In this context, the development or improvement of a civil infrastructure system should be the product of a succession of decisions among alternative courses of action. If the alternatives are precisely enough defined and thoroughly enough understood to be well differentiated in terms of risks-costs-benefits and cost-effectiveness, then choices can be made with confidence. Development of a systems approach, that includes system architecture and system engineering processes, can be thought of as the pursuit of definition and understanding of customer needs,

clarification of mission objectives, definition of design alternatives, assessment of the consequences of the alternatives (i.e., political, economic and social), and tradeoff analysis. Thus, engineering and management of infrastructure systems requires working in an environment characterized by a high degree of uncertainty and ambiguity not considered under a more narrow economic rationality/technological model. To facilitate an improved recognition and understanding of the complex relationships, a framework for conceptualizing and modeling the system is needed. Such a framework must include the second tier systems (subsystems) if it is to facilitate development of an understanding of the interactions between and among the subsystems. The total systems approach requires active participation in the policy process and integration of engineering, planning, and policy analysis. Bringing the various disciplines together into a coherent team is the fundamental task of the system program manager. Figure 1 is an illustration of the interrelationship among the major system elements (subsystems) and the management
decision making subsystem.

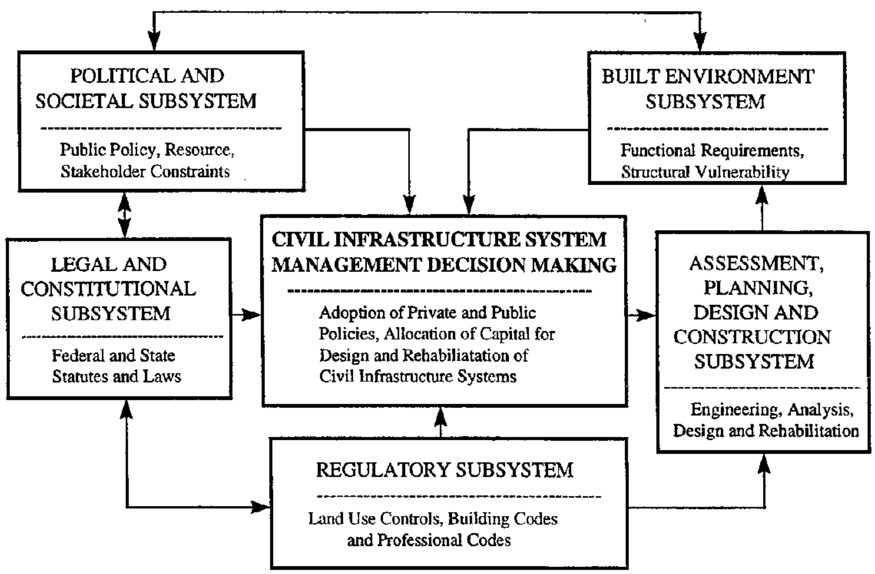

Figure 1. System Framework for Civil Infrastructure Management Decision Making

Given the changes taking place in the regulatory environment and the competing demands for scarce resources (i.e., public capital) it appears to be increasingly difficult to develop and implement new and retrofit civil infrastructure programs. To overcome these difficulties, it is necessary that the engineering community develop the knowledge and skills necessary to effectively compete for scarce resources in the deregulated market place and the public policy arena. With regard to public civil infrastructure systems, the knowledge and skills required can only be developed if there is an appreciation for the issues associated with public policy process that allocates the scarce resources among competing demands. Since effective functioning civil infrastructure systems is critical to urban communities, a reduction in the risks posed by natural hazards is a major area of concern. However, implementation of risk reduction programs for these low probability events must compete with resource demands for immediate needs.

To illustrate the difficulty in obtaining the necessary public capital to address pressing civil infrastructure problems, the following discussion will focus specifically on the public policy issues associated with development, adoption and implementation of natural hazard risk reduction policies. This will be followed by a discussion of how the development of risk management approaches can help overcome the barriers posed by the issues and facilitate the policy process. A research program to develop tools to aid the civil infrastructure engineering community in competing in this complex environment is suggested. The program will involve the performance of a high level systems analysis and development of an integrated decision support system.

PUBLIC POLICY ISSUES IN NATURAL HAZARD RISK REDUCTION

Before discussing the public policy issues associated with natural hazard risk reduction, it is necessary to note that most successful public policies are distinguished by having a narrow focus, a high probability of success, and short term payoffs. They are easily understood, have widely derived benefits, and are perceived as affordable. Natural hazard mitigation is not easily addressed by policies that have these characteristics. However, success in the design and implementation of civil infrastructure systems will be significantly improved if these characteristics are given consideration when policy proposals are being developed. The primary issues that affect the ability of government to formulate, adopt, and implement hazard mitigation policies are summarized below:

1. Organization of government to manage risk associated with hazards works against cooperation.

The fragmentation of the United States governmental structure and associated legislation, makes efforts to manage an already complex situation more complicated. Differences in approach required by the various policy tools (i.e., land use controls) and conflicting mitigation requirements for different hazards results in an administrative system with many discontinuities. In addition, government officials involved includes elected officials, land planners, engineers, code officials, and financial planners/managers. In small communities, many people work only part time, further complicating the need for intergovernmental coordination. All of these factors affect the management of risk within an intergovernmental system. In general, the dynamics of the system often result in discouraging the adoption of appropriate hazard mitigation programs [1,2].

2. Other problems appear to be more important than natural hazard and technological reduction.

Policy makers cannot effectively deal with all the possible problems that they confront. Thus, it is difficult for many policy issues to make it to the action agendas, and few of these get the support that is necessary to translate problem concerns into effective problem solving actions. Although many factors may influence whether a potential problem makes it to the policy agenda, one of the more important is the perceived magnitude of the problem, as compared with all other problems competing for policy maker attention, and the level of political salience as demonstrated by the actions of important stakeholders. Weighed against these criteria, the effects of low probability future events often appear to be comparatively unimportant in the short term.

3. There is generally an absence of hazard mitigation oriented political constituencies.

It is important to recognize that policy agendas do not always include what many stakeholders would consider the most important concerns. Some of the potentially major problems of a community may go unattended while community officials expend their energies on what others might view as comparatively minor subjects. Contrary to the belief of most people, the typical elected official is not the principal designer of major policies. In most major areas, stakeholders (i.e., professionals, lawyers, bureaucrats, citizens, and lobbyists) play a critical role by explaining its merits to both the elected officials and the public. It is not enough that a problem is identified. A substantial portion of the community must be convinced that the problem exists, must be sufficiently concerned to mobilize their political energies, and to raise their voices loud enough to be heard by the relevant policy makers. As in the economic marketplace, political constituencies compete with each other to get their respective issues and concerns on the government's action agenda. A large allocation of human resources, money, energy, and talent may be necessary to shape the political environment, identify the relevant issues, articulate the appropriate mitigation alternatives, and build support for the preferred alternative. A major factor affecting constituency support is the political culture in the community. A conservative political orientation often results in opposition to government policies that are viewed as interfering with private property rights, decreasing the ability to adopt land use controls as hazard mitigation measures. Real estate development and economic development interests tend to be the most active opponents [3,4,5]. Special interest and varying values and perception among other stakeholders also makes it difficult to reach a consensus about appropriate hazard mitigation policies [6].

4. There is often an absence of "inside" advocates and an inadequate institutional capacity.

The behavior of policy making bodies is not much different from that of any other human group. Interest groups must inevitably establish linkages with a comparatively small number of policy makers who will devote substantial time and energy to promotion of the constituencies' cause(s). Therefore, mitigation policy proposals must become a matter of concern to specific elected officials, committees, or institutional advocates. Issues, problems, and policy proposals that are not owned by responsible and attentive parties tend to drop to a low level on the political agenda, and may quickly disappear. Influential policy advocates, or champions, who have access to policy makers and a high degree of legitimacy, political power, or the prospects of a longevity in office are best able to promote adoption of hazard mitigation measures [3,6,7].

In addition to the need for inside advocates, there is a need for sufficient institutional capacity to support the management of policy proposals. Although capacity is generally determined by the availability of professional engineering support, it also includes legislative staff and committees, political caucuses, and appointed boards and commissions. The lack of sufficient trained and experienced personnel reduce the amount of attention given to the implementation of hazard mitigation practices, while staff professionalism increases the attention [4]. Thus, local government's are more likely to adopt mitigation measures when the professional staffs regularly participate in professional meetings where mitigation approaches to hazards are discussed [6]. Larger jurisdictions are more likely to be able to satisfy the institutional capacity requirements than are smaller jurisdictions [4,8]. The federal government is likely to have the most sophisticated institutional capacity, whereas local government that is not likely to have much capacity, is the level at which most civil infrastructure and hazard policies are expected to be developed and implemented.

5. Cost frequently mitigates against adoption of problem-solving policies.

Many public problems are never placed on public policy agendas, and many that are placed on an agenda are either ignored or lead to decisions that nothing should be done. A variety of factors may lead to such outcomes, including uncertainties concerning the causes of the problem, the efficacy and impacts of alternative solutions, the relative importance of the problem as compared with others competing for the policy maker's attention, the practical politics of decision making, etc. A major factor, however, is the perceived costs of developing and implementing a policy. Costs may be viewed by legislators in such terms as money, time, difficulty in acquiring information, political consequences, inconvenience, and conflict. When the mixes of these costs appear to be large, doubt may be expressed concerning the immediate wisdom of tackling the problem. In short, policy makers ask whether or not the development of a solution to any specified problem is worth the mix of costs associated with the formulation and implementation of a solution.

Policy makers know quite well that public policies frequently involve the conferring of benefits on one group while imposing costs on another group. Even under situations where the aggregate benefits of public activity may far outweigh the aggregate costs, the disproportionate allocation of these costs and benefits may deter legislators from acting to resolve a problem. In circumstances where such disproportionate allocation of benefits and costs occurs, the pain produced by the resulting intergroup conflicts may be further exacerbated if the elemental issues of fact associated with the situation are also too numerous and appear to be too difficult to resolve. Costs also occur as unintended consequences from implementation of a policy solution to a problem. An example in this area is in the application of structural mitigation for flood mitigation. The building of infrastructures, such as dams, sea walls, levees and flood channels, was the application of engineering technology to reduce the flood risks. The effect was to reduce the return frequency and intensity of the flood hazard. However, these actions changed the characteristics of the risk resulting in a flood hazard changed from a high-probability/low-consequence event to a low-probability/high-consequence event. People either ignored the objective flood risk, had insufficient knowledge about the flood risk, or considered the risk to be acceptable and moved into the flood plain in large numbers [8]. It is in these contexts that risk estimation and risk assessment can be of assistance in the public policy formulation process.

6. Issues of fact and value and differences in perceived risk and objective risk constrain the policy process.

Important policy questions arise from interrelated sets of fact and value issues. The resolution of the factual issue may lead to the subsequent revision of a stakeholders' value commitment. Alternatively, however, commitments to ideology may result in such unshakable support of and commitment to value propositions that regardless of the facts the person or group will not move from their value positions. As a result, many public policy decisions are made under conditions of factual uncertainty, even though numerous issues of fact may be associated with various perceptions of reality concerning alternative approaches to solutions of problems. Although scientific inquiry may ultimately resolve such issues, the current situation often requires policy makers to act before the efforts of science can reduce or resolve the uncertainties. Thus, the policy maker is required to decide in the absence of the kind of information required in a typical scientific analysis. In the area of hazard mitigation, numerous issues of fact and value are present in the policy making process [1]. Uncertainties concerning the frequencies of future events and intensities may freeze the policy maker into inactivity; conflicts over mitigating methods may lead to similar results. Conflicts concerning the cost escalation that will result from increases in building standards or from the adoption of building retrofit policies may produce similar results.

7. Problems of complexity and uncertainty constrain the policy making process.

Complexity may be the equal to uncertainty in its effect on the policy making process. Even under circumstances where uncertainties can be resolved and the issues of fact reduced to manageable proportions, policy makers may be reluctant to act if overly complex patterns of problem solutions are demanded of them in a single sitting. When the solutions of problems require complex patterns of problem solving activity, legislative bodies seem to prefer that the outside constituencies resolve the priority questions: Which corrective action should be initiated first, and which can be ignored for at least a reasonable period of time? In the policy formulation system, simplicity is a key to success. Conventional wisdom indicates that attention will first be allocated to those problems that can be most easily understood and economically eliminated or reduced. Accordingly, big problems that can be simply understood, or which are perceived as having simple solutions may receive first attention. Similarly, smaller problems that can readily be solved may also be given a higher priority than bigger problems whose solution seems more elusive. The availability of a policy option that is viewed as practical and efficacious is an important factor in the adoption of natural hazard mitigation policies [6,7].

8. Lack of planning hinders implementation of appropriate mitigation during post disaster reconstruction.

A disaster event often provides the opportunity to make improvements in urban systems, usually as a result of system failures; however, mitigation strategies that a community should pursue following a disaster are rarely defined in advance. Also, the recovery phase is complicated by conflicting sets of values and needs. In the post-disaster period, there is the pressing need for (1) the community to return to normal as soon as possible, (2) reducing risk from future events, and (3) improving efficiency, equity and system effectiveness. The process of decision making is complicated because many stakeholders find themselves in multiple groups with differing expectations and sets of values. This results in the development of different coalitions, which may or may not continue throughout the recovery period, and issues about managing future risks, such as appropriateness of land use practices, adequacy of building and construction codes, and overall improvement of the community. Many of these same issues arise during pre-disaster periods.

In summary, to make progress in improving the productivity of the infrastructure and in reducing risk to the infrastructure, engineers must learn how to effectively work within this complex decision making system. This requires knowledge about the policy and decision making processes, as well as an understanding of the issues. Improved innovation and technology transfer programs, which take into consideration of these issues are needed to make progress in risk reduction [10]. Research has shown that formulation, adoption and implementation of hazard mitigation/risk reduction policies are complex tasks, and even when accomplished may produce at best limited success. Success is most likely to be achieved in areas of new development and when designing new civil infrastructure systems. There have been opportunities where it has been possible to adopt and implement hazard mitigation policies in developed communities, such as the case of bridge retrofit programs in the State of California following earthquake caused bridge failures. Although these opportunities occur after a disaster when a policy window opens, it is likely to take a number of years before the process has concluded with the implementation of infrastructure improvement programs.

RISK MANAGEMENT AND THE PUBLIC POLICY PROCESS

As noted above, the civil infrastructure engineering community needs to develop an understanding of the system that produces policies and allocates capital resources if they are to be able to strategically intervene in the system to bring about change. To aid in improving this understanding, the following is a presentation of a model intended to provide insight to the policy making process, and serve as a point of departure for describing the role engineers can play in facilitating the policy making process. The model is based on the assumption that organizations function as open systems and that (1) there is always imperfect information for decision making, (2) there is never adequate time to deal with all the items seeking a position on the policy agenda and (3) not everyone values the array of issues and options equally, in fact they may not even view them similarly. That is, there is likely to be a lack of clear definition of problems, it is often unclear as to which stakeholders have an obvious right, obligation, or mandate to deal with the situation, and finally, decision makers are confronted with numerous simple and complex problems, multiple solution alternatives, and choice opportunities.

The basis of the model is that problems and solutions exist independently from one another, and that people frequently have solutions looking for problems. Also, there are often problems for which there are no known solutions, and there are simple problems with easily identifiable solutions. It is the easy problems that are solved first and frequently at the expense of considering the more difficult problems. Further, a matching problem and solution will have little impact if there is no opportunity for implementation. Identified problems and solutions need the support of the various participants or stakeholders in the policy/decision making system. If there is no way to move a problem solution to the policy agenda, there will be no way to formally adopt the policy, thus no ability to implement a problem solution.

An informed decision process is expected to lead to better decisions, which will in turn provide the opportunity for implementation of improvements to the civil infrastructure system. However, as Kloman [11] has noted, managing risks with "unprecedented uncertainty" will require society to (1) "adopt a more holistic approach to problems, looking at the whole and avoiding the trap of dealing with specialties with which we feel most comfortable"; (2) "adopt a more systemic approach, understanding the complex interrelationships that existing every thing we do"; (3) "accept the thesis that risk is probability and that probabilities are always changing"; and (4) "become more humanistic, reflecting the multiplicity of values, viewpoints, cultures and perspectives that make the human race as interesting and disturbing as it is" (p. 204).

What is the role for the engineering community in managing risks with unprecedented uncertainty? Since there is a disconnect between problem recognition and definition, formation and refinement of problem solutions (policies), and participation by interest groups, the professional engineering community (i.e., acting as inside advocates or as professionals external to the decision making system) has an important role to play. This role can best be defined as addressing the requirements articulated by Kloman. Valid and reliable input to the decision making process based on performance of high quality risk-cost-benefit analysis with full disclosure of assumptions and uncertainties is an important step in the process of education. Success requires the development of an understanding of policy issues and the communication skills necessary for effective knowledge and/or technology transfer. Figure 2 provides an illustration the relationship between risk analysis, policy formulation and policy (program) implementation.

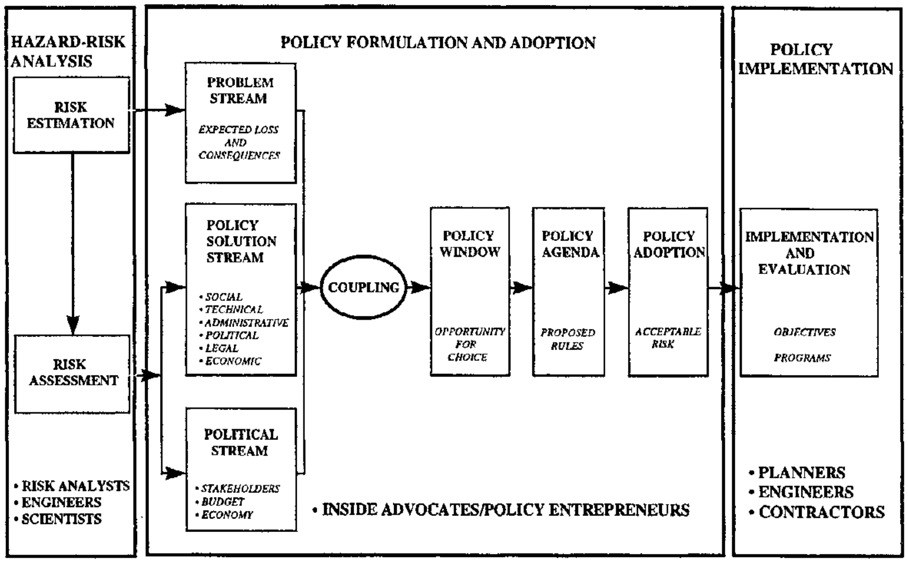

Figure 2. Risk Management-Policy Formulation-Implementation Model

In summary, engineers must engage in, not only risk analysis, but also the process of formulating appropriate policies and strategies. Thus, engineers need to educate policy makers, listen to policy advocates, and incorporate the views of participant stakeholders in the design process before implementation is possible. Engineers who participate in the policy process must (1) understand the how ambiguity caused by uncertainty and complexity affects decisions, (2) aid in incorporating the principles of risk management into the culture of the policy formulation and implementation system, (3) understand the conflicts inherent in a system of multiple levels of government and stakeholder groups, and (4) incorporate the social, technical, administrative, political, legal and economic factors into system design. Failures in adopting and implementing effective risk management policies are generally a function of factors having more to do with the lack of understanding the risks, costs and benefits and the dynamics of the public policy process rather than the need for better engineering knowledge.

To facilitate the policy process engineers need to help inform policy makers, as well as stakeholder groups, about the significance of the problems and effectiveness of alternative policies. Inside advocates who understand the work of the experts, and who can communicate the significance of the risk, are important to bringing about stakeholder agreement on the problems and proposed solutions. Outside professionals who understand the policy issues and who are effective communicators are also important to achievement of the coupling. Although policy makers may have been successful in moving a policy to the legislative agenda, the process of policy adoption remains to be accomplished. The selection and clear explanation of appropriate engineering technologies for the retrofit of existing or development of new civil infrastructure systems, the application of risk management methods of analysis can help to facilitate the policy process.

For natural hazards, risk management requires implementation of mitigation policies that have been based upon a careful consideration of the risks and reasoned decision making with respect to a set of risk reduction alternatives. Thus, management of the risk requires the performance of a risk assessment, which integrates quantitative estimation of (1) the likelihood of a hazardous event occurring at some location, (2) the intensity of the event, (3) the vulnerability of people and systems exposed to the event, and (4) a quantitative and qualitative assessment of the consequences of the event in terms of subsystem/system failures and the impacts on the greater societal systems. The selection of mitigation policies that will result in "acceptable risk" requires consideration of the social, administrative, political, legal and economic factors, along with results of a technical risk analysis. Figure 3 provides a system framework for understanding the relationship between hazard reduction policies, expected loss analysis, assessment of consequences, cost-benefit-efficacy assessment and acceptable risk.

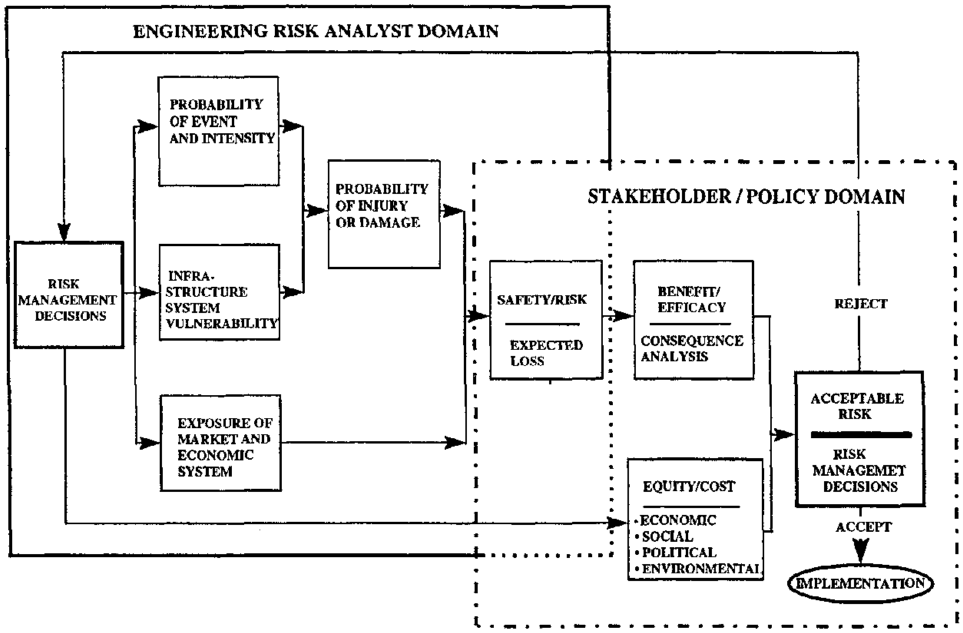

Figure 3. Risk Management Decision Process Model

What distinguishes *risk assessment* from the limited analytical process of *risk identification* is the addition of value judgment. Risk identification is an objective analytical process, whereas risk assessment attempts to evaluate the relative importance of consequences. Acceptance of the results of a risk assessment and related policies, from the stakeholders' perspective, will not be achieved until private and public interests have evaluated the assessment and the mitigation benefits and costs, and argued their case before political decision makers. What emerges from the political arena as acceptable may not be optimal in the rational sense, but rather tends to be some negotiated compromise meeting minimum requirements for the majority, if not all, of the concerned stakeholders.

An underlying premise on the utilization of risk assessments is the need to merge rigorous, systematic, value neutral, empirical results of scientific investigation with the problematic, high pressure, political, ideological approaches to public policy making. The degree of acceptance by stakeholders is determined (1) by whether the results are perceived to be helpful or detrimental to their interests, (2) by their level of knowledge about risk assessment, and (3) by the amount of new information provided by the analysis. Although the analysis of risks is a task for the professionally trained, analysts must be careful not to understate or overstate the risk, but provide the best possible estimate of loss with a complete explanation of the assumptions and range of uncertainty. Those involved in deciding the level of acceptable risk will assign the amount of conservatism they consider appropriate in setting hazard mitigation policies [12].

Stakeholders in the process will most likely develop conclusions based upon their perceptions, whereas the risk assessment is based on data and empirical analysis. Mitigation policies will most likely be accepted and supported by stakeholders when the policy process includes empirically defensible estimates of risk, an assessment of the efficacy and cost of the mitigation policy, and when the policy makers and stakeholders are capable of understanding the nature of probabilistic risk assessments when making judgments about safety, benefit and equity.

Risk assessment methods have and will continue to contribute to a better understanding of the risks posed by construction of civil infrastructure systems. There remains, however, the need to better understand how society evaluates, distributes and accepts risks, and a need to improve the capacity of government to incorporate risk assessments into the risk management process. While the analytical methods are being continuously improved, there are many constraints that affect the performance and utilization of risk assessments in the management of risks, most notably the lack of available data, high degrees of uncertainty in the results, insufficient capacity in knowledge and expertise, and insufficient fiscal resources.

Questions that must be addressed when considering risk assessments are: (1) How good are the methods and domain knowledge required to estimate the risks? (2) How are the expected loss estimates to be entered into the decision making processes and what will be the significance of the uncertainties? (3) How will the bureaucracy and/or the political structures affect the political process and decision making about risk reduction? (4) What are significant factors affecting the perception of risk and how will the perceptions be incorporated into the public policy debate? (5) How will the process determine when a risk level is considered acceptable and balanced against the issues of equity and social justice? (6) How will the trade off between the demands for factual accuracy in the risk analysis and the demand for immediate action to meet the needs of society be made? In addition to questions related to the efficacy of risk assessment, critical questions related to specific policy considerations are: (7) To what extent should economic and other social costs be weighed against the benefits of immediate implementation of risk reduction policies? (8) Can improvements in the adoption and implementation of hazard mitigation practices reduce hazard losses? (9) How can risk mitigation be more effectively accomplished?

Debates about who pays and who benefits are matters of significant concern to selected stakeholder groups where much of the debate is value based and not easily subject to analytical analysis, thus, making it possible for perceptions more than reality to influence the outcome. Numerous issues of fact confront the technically competent, but complex issues of value confront the policy maker who is expected to act on the facts. Only after completing the socially sensitive process of dealing with these subjects, will the policy process end with desired results, and only then is it possible to consider the policy implementation process.

It is important to remember that, although risk management models are important in developing an understanding of the significance of the issues, they tend to define policy in terms of winners and losers, which make's adoption more difficult. Thus, in problem areas where the science and data are limited resulting in a high degree of uncertainty in the modeled results, models for public policy should be used heuristically and not as predictions of the future. This is particularly true when the models are utilized to assess the short term risk and policy for specific communities and/or locations, such as in the case of the earthquake risk. In the assessment of long term risk for a larger geographical area where the uncertainty, models can often provide predictions that will be useful in guiding public policy. Both modelers and public officials need to be aware of when it is appropriate to use models for heuristic or predictive purposes.

In addition too inside advocates, there is a need for sufficient institutional capacity to support the assessment and management of policy proposals. Although technical support is required, legislators, legislative committees and their staffs, political caucuses, and boards and commissions must also have the capability to understand the issues. Larger jurisdictions are more likely to have the institutional capacity than are smaller jurisdictions. The federal government is likely to have the most sophisticated institutional capacity, whereas local government that is not likely to have much capacity, is the level at which most hazard policies are expected to be developed and implemented.

As noted, there are many issues associated with the use of risk assessments in the policy formulation and implementation process. Following are several areas of concern.

1. Complexity and Data Requirements - In order for risk assessments to be useful in the policy process they must be doable and practical. Conventional risk analysis methods are generally too complex and data intensive for most local and state governments to readily use as an aid to decision making. Most local governments need simple, easy-to-use risk assessment tools.

To increase the application of risk assessment methods by local and state governments, a consensus process needs to be developed to establish uniform guidelines and standardized approaches to the performance of each step in the risk assessment process. Standards for data collection should be developed and assistance provided local governments in developing approaches to the collection of data sufficient to perform the risk assessments. Identifying and coupling data requirements for other activities with data requirements for natural hazard risk assessments will improve efficiency in this critical area. Standardization and refinement of appropriate models and guidelines are essential to making the risk assessment process universally more acceptable.

2. Uncertainty - It is important to recognize that the utility of a risk assessment in the policy setting process is a function of the perceived reliability of the results. The assessment of risk is highly dependent on both data and expert judgment. In the case of low probability events, the lack of factual data and need for expert judgment contributes to a high degree of uncertainty in the results of the analysis. Further, people regularly underestimate the risks associated with low probability events. As a result, it is difficult for those involved in the process of determining appropriate policies to readily accept probabilistic-based estimates and for stakeholders to internalize the meaning of the results. The uncertainty and concern over reliability contribute to an already existing tension between the elements of citizen participation in a democratic system and the utilization of scientific and technical expertise in assessing risk and framing the problem [13].

All risk assessments should be presented in a standardized written form with a complete discussion of the uncertainty and the assumptions made to arrive at the conclusions. Guidelines should be developed to provide a standard approach and consistency in presentation that will facilitate communication and result and contribute to educating and informing stakeholders about probability, thereby developing a greater confidence in the results and the need for proposed mitigation policies.

3. Credibility - An assessment of natural hazard risks is not likely to be accepted unless the stakeholders find the methodologies employed to be credible. There is always concern about the possible abuse of the process by those who are oriented to client needs. The possibility for manipulating the results of a risk assessment is well understood, and as a result a risk assessment may not carry the authority and objective weight necessary to support policy decisions. Information sufficient to assess the completeness and underlying assumptions is necessary for evaluating the quality and completeness of the analysis. Using quantitative criteria for normative decisions when the decisions are the outcome of a social process is a difficult undertaking, especially when the combined probability of an occurrence and the magnitude of its consequences is too complex a form of calculation to be helpful to ordinary persons.

Policy makers need to be able to view risk assessment as a tool to aid in developing and refining policy proposals that inform stakeholders involved in the political process. Risk assessments help to educate participants about the significance of problems and the benefits and costs of solutions. It is, however, necessary to develop an understanding of the context in which risks are to be presented to the public, and to overcome the concerns about ethical considerations [14]. The complexity of the issues makes it very difficult to communicate with stakeholder groups who tend to mistrust both public officials and scientific experts [14, 15], so improvements in communication between policy makers and scientific experts regarding risks and future consequences are critical to effective risk management.

4. Capacity - A major factor that has direct impact on the adoption and implementation of hazard mitigation policies is the ability of the public body (i.e., capacity) to understand and utilize the results of risk assessment. Although risk assessment methods are considered by many to be well defined, the major constraints that affect the utilization of the analysis are insufficient staff knowledge and expertise, and insufficient fiscal resources. Increased capacity is most important at the time of debate about the level of risk acceptable to the community and the consequences of policy implementation. Debates about who pays and who benefits will always be of significant concern to selected stakeholders and their elected representatives, but achieving agreement on acceptable levels of risk is the most difficult task in the policy process. It is at this point in the process that limited ability to understand the outcome of a risk assessment has the most. Much of the debate is value based, qualitative and thereby not empirically verifiable, and is not easily subject to analytical or rational analysis. The inability to be precise and absolute makes it possible for perceptions more than reality to influence the outcome.

Research has shown that those who understand the work of technical experts and who can communicate the significance of risk are better able to bring about a coupling of problems, policy options, and stakeholder interests. Thus, a research program needs to be started which will lead to the development of tools to help the engineering community effectively participate in the complex social-political system. The tools will help to improve the capacity of the experts, policy makers and interest groups to recognize the need for appropriate civil infrastructure system policies. The research program should have a long term perspective and a focus on (1) development of agreement on a common philosophy, language and process, (2) development of standard models, data requirements and risk assessment methods, (3) and development of standardized training materials, programs, and community-based workshops. A first step in the research program would be the development of a high level systems analysis to define the major system elements, their relationship to system improvement, and the characterization of system problems that impede progress in improving of civil infrastructure productivity. Following the systems analysis, a research program leading to the development of an Integrated Decision Support System (IDSS) as illustrated in Figure 4 should be started.

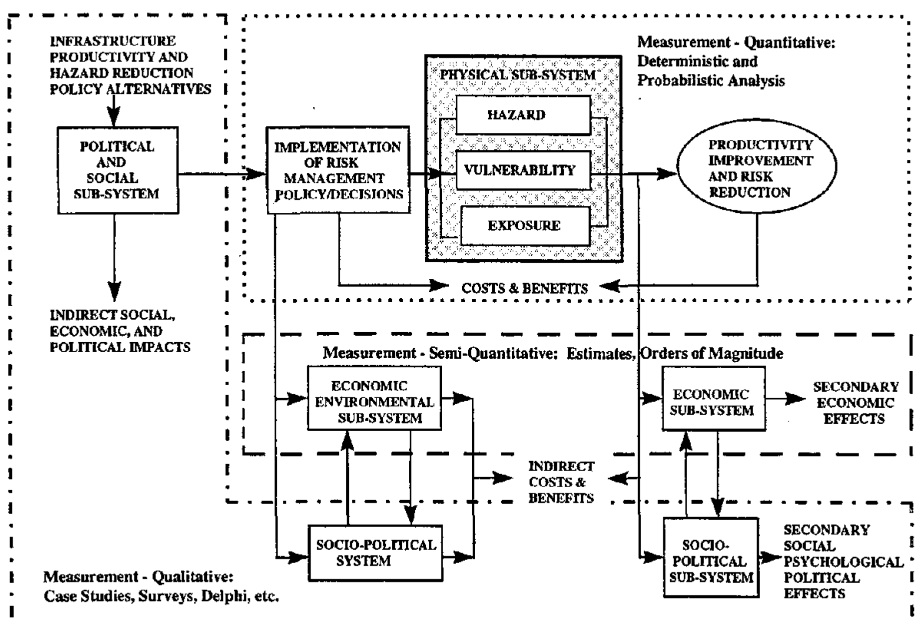

Figure 4. Integrated Decision Support System Components

The research should incorporate incorporating the following major elements.

1. Development of a system architecture that incorporates the political and economic factors affecting infrastructure system engineering is critical in facilitating the public policy decision process. The rules and variables must be understood as clearly as stress analysis and structural design, however, the rules differ profoundly from those of engineering design. From a system perspective, it is important that information be accurate and relevant to the risks, costs and benefits of a proposed infrastructure project. The development of national and state level norms and standards will aid in managing the system so as to achieve results acceptable to a broad community.

2. Power is distributed widely in a political system. There is no single authority, and the interests of the stakeholders within the system most often diverge sharply from one another. In order to understand these relationships, simulation models and tools that simulate system interactions and processes in dynamic terms

need to be developed. Simulation tools are very useful in educating and informing the many participants of the consequences of alternative infrastructure decisions.

3. Understanding the dynamics of the urban system is critical to the implementation of any program for improvement. Barriers that impede the adoption and implementation of civil infrastructure programs need to be characterized so that they can be incorporated into the IDSS in order to aid in making appropriate tradeoffs in system design, location, and construction that will lead to acceptable levels of service at an acceptable level of risk.

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TRENDS IN JAPANESE POLICIES TOWARD INFRASTRUCTURE

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1. IMPROVEMENT TO INFRASTRUCTURE AND ECONOMIC SOCIETY

1.1 Roles and Functions of Infrastructure

- Basic Roles:
  - Improve quality of life: life facilities (such as hospitals), parks, environmental access roads, water supply and sewage systems, etc.
  - Maintain economic development: information, communication, trunk roads, airports, harbors, etc.
  - Increase safety and security of life: flood control systems, disaster prevention facilities, etc.
- Aimed at effective allocation of economic resources, infrastructure as purely public goods: non-exclusive and non-competitive. In addition to Administration of Justice, Defense, Police, these situations are characteristic of infrastructure (e.g., roads, parks, embankment systems).
- Economic gains and social value of its scale, infrastructure as semipublic goods has specified users, but large scale is advantageous. Private companies can undertake improvements to sewage systems or toll roads, but usually the public is in charge of them.

1.2 Roles of Government and Private Sector in Improving Infrastructure

- Infrastructure supplied by the government in cooperation with the private sector includes electric power, railways, telephone, etc. A tradeoff may exist between profitability or the public interest (e.g., academic facilities, welfare facilities) and the balance between profitability and the public interest has improved. Historically, the public sector was the main decision-maker, but more recently, the private sector has begun to play an important role.

1.3 Effects of Infrastructure Improvement

- It is essential to grasp the effects of infrastructure improvement from various viewpoints such as the effect of monetary evaluation, effect of intangibles, influence on environment, etc.
- Infrastructure affects the vitality of the entire economy (e.g., increases productivity, increases GDP, supports industrial and productive activities in general).
- Flow effects and stock effects both need to be considered (e.g., effects of the existence of infrastructure; effects of investment in infrastructure, its influence on economy, its multiple effects on GDP).

Table 1. Effects of Infrastructure Improvement

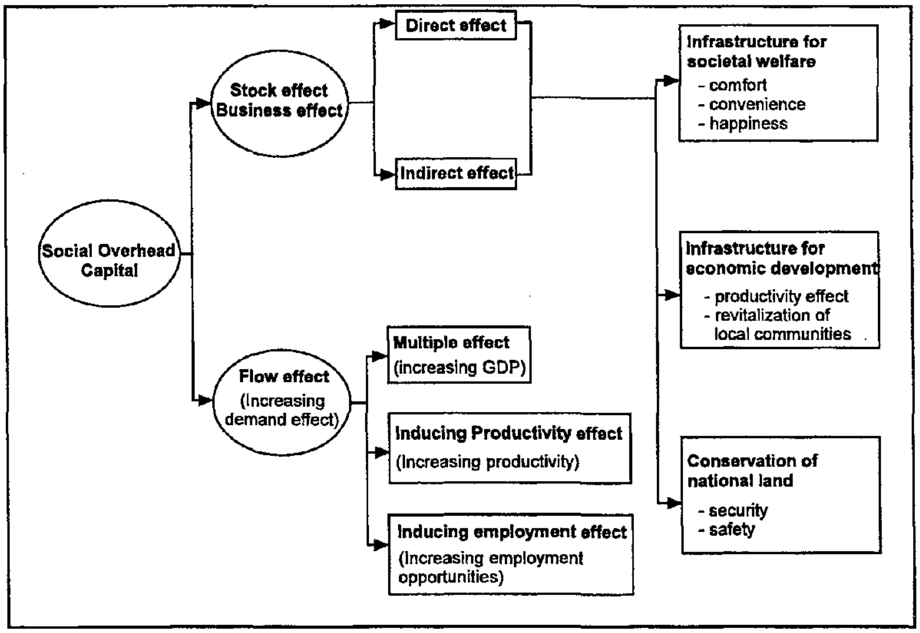

Table 2. Various Effects of Infrastructure

Field	Direct Effect		Indirect Effect	
	Monetary	Non-monetary	Monetary	Non-monetary
Trunk roads	Save fuel cost Save time cost	Comfort, safety, traffic, widening interchange circle	Lower commodity prices, location of industry, increase in sightseeing demand	Improve environment by decreasing emission gas
Environmental roads	Save fuel cost Save time cost	Comfort, safety, traffic, disaster prevention	Advanced utilization of land	Life environment, sights, traffic safety
Airports	Save time cost	Comfort, widening interchange circle	Location of industry, increase in sightseeing demand	—
Harbors	Save time cost	—	Lower commodity prices, location of industry	Improve environment by decreasing overland transportation
Parks	—	Mental satisfaction, health, disaster prevention safety, nursing of children and education	—	Life environment, relieving heating problems
Flood control facilities	Decreased flood damages	Security (mental satisfaction)	Location of industry, decrease in disaster restoration expenses	Increased population (life environment)
Water resources, water supply system	Stable water supply to inhabitants, business, agriculture	Convenience, health (mental satisfaction)	Location of industry	Increased population
Sewage, water cleaning system	Decreased flood damages, no need to set up and maintain sanitation facilities	Convenience, health, security	Increased land value	Sanitation, conservation of water quality and ecology, mental satisfaction

2. HISTORICAL BACKGROUND OF INFRASTRUCTURE IMPROVEMENT

2.1 Development of Economic Society and Infrastructure

- In the 1950's, the focus was on restoration of national land (e.g., conservation of national land, investment on agriculture)

- From the 1960's, provided a way to greater economic growth, supported industrial activities, focused on improving industrial facilities (e.g., roads, harbors, railways, water for industrial use, industrial areas)

- From the late 1970's, provided quality of life and psychic satisfaction, focused on life-related investment (e.g., life facilities, environmental roads, parks, sewer systems, welfare facilities)

- From the late 1980's, provided information communication infrastructure in response to structural transition of economy, restructuring of urban areas, etc.

2.2 From Quantity to Quality, From Construction to Repair and Maintenance

- Catching up with the standard of European countries and the U.S. Development of economy, focus on improvement of harbors and expanding the length of roads, balanced development of national land has been completed to some extent. Emphasis now aimed at improving public quality of life and environment.

2.3 Organization of Infrastructure Improvement

- Public sector and private sector, improvement of nationwide network need to catch up with the standards of European countries and the U.S. Formerly, the public sector was dominant because of high technology and sufficient funds (e.g., the public should bear the responsibility of railways, telephone and telegraph). As the management of the Japan National Railway Corporation and the Telegraph and Telephone Corporation got increasingly complex, and the private sector developed, these two corporations were privatized. These days, there is more of a need to use the private sector for public works.

3. OVERVIEW OF IMPROVEMENTS TO INFRASTRUCTURE

3.1 Short History of Japan's Infrastructure

- Japan's GDP per capita was the world's highest in 1993, but housing and housing environment were in very poor condition. In spite of the construction of three transbay bridges and an undersea tunnel, there is still a poor diffusion rate for sewage facilities. Improvement of life environment facilities has just started.

3.2 Still the Halfway to the 50th Generation of Infrastructure Improvement

Table 3. Progress of Infrastructure Improvement: 1960 - 1990

Function	Indicator	1960	1970	1980	1990	Present (Year)	Target Figure in the Beginning of the 21st Century[1]
National roads, traffic	Length of expressways[2]	0 km	649 km	2,874 km	5,288 km	(as of 1996) 6,768 km	14,000 km
	Improved ratio of national roads, metropolitan and prefectural roads (width over 5.5 m)	18.9%	37.7%	54.4%	64.1%	(Beginning of 1995) 68.0%	—
Urban environment	Improved ratio of well developed urban area[3]	—	—	28.9%	36.7%	(as of 1994) 38.9%	—
Life environment	Improved ratio of urban planning roads	—	27%	36%	45%	(as of 1994) 47%	—
	Area of urban parks per person (National) (Tokyo)	2.1 m^2 –	2.7 m^2 1.2 m^2	4.1 m^2 1.9 m^2	6.25 m^2 2.6 m^2	7.1 m^2 2.9 m^2	About 20 m^2 —
	Diffusion rate of sewage systems	6%	16%	30%	44%	(as of 1995) 54%	About 90%
Flood Control	Flood prevention rate[4]	—	—	—	—	(as of 1996)	—
	(All rivers) (Large rivers) (Small and medium-sized rivers)	— 40% 5%	24% 46% 9%	32% 57% 17%	43% 57% 28%	52% 67% 43%	

Source: Ministry of Construction

[1] According to the basic plan of public investment and the long term planning of national land construction.
[2] Total length of high-standard trunk roads (expressway and general national roads and highways).
[3] The ratio of well developed urban area to improved facilities (e.g., the road network which separates transit traffic adequately) by land partition projects, etc. in urban areas within urban planning districts and non-ruled districts.
[4] Flood prevention rate is for the 50 mm per hour rainfall. Before 1980, for large sized rivers, the rate was for the largest flood after World War II. The present target is for the rainfall which occurs every 30-40 years.

3.3 Basic Plan of the Public Investment: 630 Trillion Yen

- Japan-U.S. economic friction (e.g., Japan's trade surplus, the U.S. trade deficit). Problem of increasing Japan's domestic demand, because life environment and social investment have been underdeveloped, the public investment 10-year basic plan of 630 trillion yen was announced to increase public works. But the downturn of economy, owing to the burst of "Bubble Economy", the financial deficit, need for financial reform, the term of the plan was prolonged to three years, and the budget was cut to 470 trillion yen. Cuts to public works — 7% in 1998 and —15% in 2000. The ratio of construction investment and public works against GDP has become smaller and finally to the level of European countries and the U.S.

3.4 International Comparison of Infrastructure Improvement

Table 4. International Comparison of Infrastructure Improvement

	Japan	U.K.	Germany	France	U.S.
Roads (length of expressways per 1000 passenger cars)	98 m (End of 1995)	138 m (as of 1993)	264 m (as of 1994)	301 m (as of 1994)	378 m (as of 1993)
Parks (park area per person)	4.3 m² Tokyo 23 Ward (1995)[1]	25.6 m² London (1982)	37.4 m² Bonn (1984)	11.6 m² Paris (1989)	23.0 m² New York (1989)
Underground cable facilities (Ratio of cable underground)	35.3% Tokyo 23 Ward (1990)	100% London (1977)	100% Bonn (1977)	100% Paris (1977)	77% New York (1977)
Sewage systems (Sewage diffusion rate)	54% (End of 1995)	96% (1993)	90% (1993)	78% (1987)	71% (1992)
Flood control (All rivers) (Large rivers) (Small and medium rivers)	52% (End of 1996) 67% 43%	Prevention of damage from flood occurring once every 1000 years. Improvement of the Thames completed (1983)	Prevention of damage from flood occurring once every 500 years. Improvement of the lower Rhine completed (1995)	Prevention of damage from flood occurring once every 100 years. Improvement of the Seine completed (1988)	Prevention of damage from flood occurring once every 500 years. Improvement of the Mississippi embankment 79% complete (1992)
Water resources (Reserved water per person[2]	30 m³ Metropolitan area (1991)	35 m³ London (1986)	—	74 m³ Paris Metropolitan area (1981)	285 m³ New York (1991)

Source: Ministry of Construction, "White Paper on Tokyo Metropolis", Metropolitan Office, "World Statistics," Management and Coordination Agency, Federation of Electric Power Companies, "Register of the World Dams," "Chronological Table of Science"

[1] Includes national parks and ward, city, town and village parks for children in addition to urban parks.

[2] Reserved water is the entire volume of water in the dams, whose purpose is to supply city water.

Table 5. Construction Market in 1995 (in trillions of yen)

	Japan	U.S.	Western Europe	Eastern Europe	Asia
GDP	488.5 (100)	681.5 (140)	822.8 (168)	21.2 (4)	201.0 (41)
Construction market	92.3 (100)	—	87.5 (95)	2.6 (3)	—
Against GDP ratio (%)	18.9	—	10.6	12.3	—
Construction investment	79.8 (100)	51.3 (64)	53.3 (67)	1.7 (2)	26.0 (33)
Against GDP ratio (%)	16.3	7.5	6.5	8.0	12.9

Nominal figures, exchange rates: Average in 1995, Japan: fiscal year

Source: Euro Construct Conference (96.12), Asia Construct Conference (96.10), "Estimate of Construction Economy," Research Institute of Construction Economy, "Statistical Abstract of the United States," the U.S. Commerce Department, "Annual Report on Overseas Economy" (95).

In Japan, there is a need to improve infrastructure and increase investment due to the lower level of its improvement compared with Europe and the U.S. The reason is not popular yet. Construction of the three long distance bridges connecting an island in Shikoku (southwestern part of Japan) with a population of 4 million, completion of the huge undersea tunnel larger than the Dover Channel Tunnel, and connecting an island in Hokkaido (northern part of Japan) with the population of 5 million.

4. PRESENT PROBLEMS OF INFRASTRUCTURE IMPROVEMENT

4.1 Priority and Efficiency of Investment

Difficulty in reconstructing finance and reserving resources, the end of persistent economic growth, criticism of non-priority investment, need to reform categorical and regional allocation of resources, categorical and regional allocation of resources has been fixed. Group interest structure, political discussion, conflicts between urban districts and non-urban districts, owing to the lack of a reasonable way to allocate resources and clear standards, political discussion and group of special interests are prioritized.

4.2 Global Viewpoint

Need to develop a global viewpoint for planning and implementation of projects, sectionarism, categorical allocation of resources, lack of totality, meaningless double investment, criticism of unnecessary investment.

4.3 Adequate Evaluation of Investment Effects

Lack of appropriate, reasonable evaluation standard and method, priority on political judgement, improvement in cost-benefit analysis manual of road operations is now introduced. Cut in public construction cost, and public works cost in Japan is 1.5 times higher than that of the U.S. The target is to reduce the public works cost by 10%.

4.4 Using the Private Sector

Using privatization, withdrawal of public bodies in the competitive field (e.g., The Japan National Railways Corporation was privatized and split into seven JR companies, the Telegraph and Telephone Corporation into the NTT Inc., Trans Tokyobay Highway is constructed by joint-stock companies, Housing and Urban Development Corporation is in charge of supplying new towns and housing will be privatized.

4.5 Necessity of Adequate Use of Stock

Progress of aging and maturing, decrease in new investment ability, completion of a nationwide network of roads and railways, efficient use of existing stock, multiple use.

4.6 Adequate Maintenance and Repair

Not only the cost of new investment but also the total cost including that of maintenance, rise in cost of maintenance and repair, increasing necessity of renovating investment, increasing necessity of new technology and systems.

4.7 Internationalization, Globalization

Internationalization of construction market (e.g., internationalization of public works market by the World Trade Organization, European unification), construction market friction between Japan and the U.S., foreign companies involved in Japanese market. Need to have an international standard of material and equipment, human resources, production system and technology.

4.8 Change of Bidding and Contract System and Construction Production System

Bidding and contract systems differ from country to country. In Japan, selective tendering system governed by bureaucracy is common. It has been criticized at home and abroad. Need to introduce open tendering system to counter existence of "Dango" (bid rigging) in construction business.

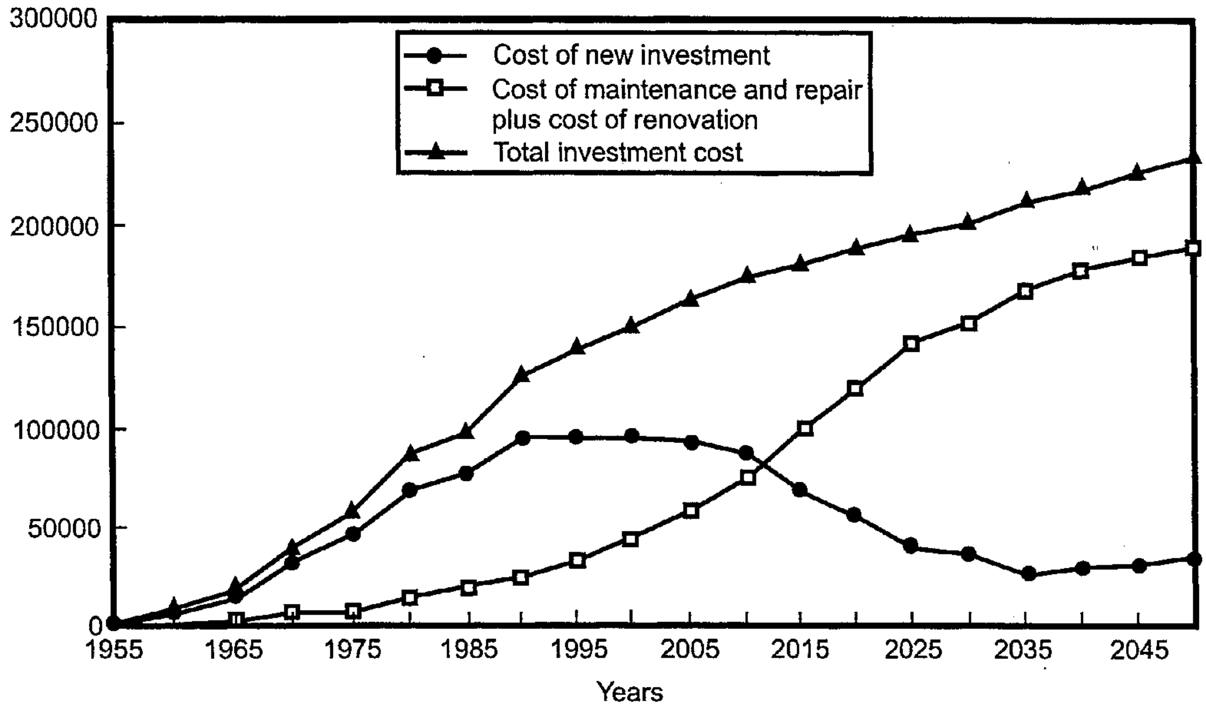

Figure 1. Estimate of Future New Investment Cost and Maintenance and Repair Costs (in billion yen, cost in 1985)
Source: "White Paper on Construction 1994"

Note: Increase rate of total investment cost for 13% av. from 1955 to 1960, 18% av. from 1961 to 1970, 10% av. from 1971 to 1980, 4% av. from 1981 to 1990, 1.6% av. from 1991 to 1994, 1.75% av. from 1994 to 2000, 1.50% av. from 2001 to 2010, 0.75% av. from 2011.

CIVIL INFRASTRUCTURE SYSTEMS RESEARCH
Academic Perspective on Research Coordination Issues

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ABSTRACT

In this paper, the author discusses his views on the challenges currently facing the U.S. academic community with respect to carrying out multidisciplinary CIS research initiatives. Qualitative changes to the purpose and approach of the academic community, particularly those in research intensive universities, are discussed. It is articulated that one important change is to establish multidisciplinary capacities for research, education and public service as a prerequisite to effectively addressing civil infrastructure systems research.

The concept of developing multidisciplinary, university-based "Center of centers" in collaboration with funding agencies, industry and government agencies, is advanced with a number of specific recommendations.

BACKGROUND

Civil engineers consider themselves as the builders of civilization. This is a fully justified statement, because the progress of civilization is significantly dependent on the physical infrastructure systems which civil engineers are responsible for designing. Today, the planning, design, construction, maintenance, and environmental industries represent the nation's largest manufacturing sector and the second largest economic activity. According to a National Science Foundation estimate, the U.S. today has approximately $20 trillion investment in civil infrastructure systems. Because of aging and other reasons, many of these systems are deteriorating and require intelligent renewal. This has led to the NSF initiative on Civil Infrastructure Systems (CIS), and indeed this U.S.-Japan Joint Seminar on CIS Research.

The importance of CIS to the economic health, quality of life and sustainable development of the nation is widely recognized. In a recent study by Batelle, the top ten technological challenges for U.S. industry during the next decade include two CIS areas: Renewed Infrastructure and Environmental Protection. These technological areas also suggest future manpower needs during the next decade.

The National Science Foundation, in recent years, has taken a leadership role to encourage fundamental CIS research. Since 1992, there have been three NSF CIS research workshops and many follow-up conferences and seminars. The actual number of research activities, however, has not been reflected in the amount of workshops/conferences that were carried out. There are two basic reasons for this lack of sufficient research activities. The first is a shortage of research funding. Each year, only a small amount of research money has been allocated for CIS basic research (say, tens of millions), which is supposed to develop renewal strategies for the $20 trillion capital investment in CIS. The second reason is the lack of an efficient approach or method to carry out CIS research, which is typically complex and involves many different disciplines and professions. This second concern, CIS research coordination and integration, from an academic perspective, is the central theme of this paper.

THE NSF CIS INITIATIVE

It is reasonable to review the NSF Initiative on CIS research, which led to this U.S.-Japan seminar. Since 1992, NSF has carried out three consecutive workshops (1992, 1993 and 1996). It is useful to examine the main conclusions of each of these workshops.

1992 WORKSHOP

Participants: Architects and planners, structural and geotechnical engineers, construction management, material and computer scientists, and others.

Outcome: Support basic CIS research in Deterioration Science, Assessment Technology, Renewal Engineering, and Institutional Effectiveness and Productivity.

This first workshop was initiated by the Engineering Directorate at NSF and the participants invited were primarily researchers and practicing engineers as well as some physical scientists. The outcome was recommended research areas that can be categorized by deterioration science, assessment technologies and renewal engineering. The category of institutional effectiveness and productivity was added after the 1993 workshop (see below) by NSF in the 1994 Program Announcement.

1993 WORKSHOP

Participants: All participants were SBE scientists with only one engineer.

Outcome: Recommend basic research needs in Human, Institutional and Physical Infrastructure Systems, with a note that "the solutions to infrastructure problems are minimally technical and overwhelmingly social, political, environmental, and economic."

This outcome is revealing about the complexity of the infrastructure renewal task. It is prohibitive from the cost viewpoint to replace all the deteriorated civil infrastructure systems. They can only be renewed intelligently by prudent and effective use of our limited financial, material and human resources, and by emphasizing the optimal performance of the CIS.

1996 WORKSHOP

Participants: A reasonable mixture of engineers, planners, managers and social scientists.

Outcome: Recommend multidisciplinary efforts, university-industry-government cooperation and systems integration.

These recommendations are on target for CIS research. They are broadly stated with few specific ideas. This outcome again clearly reflects the complexity of CIS research.

As pointed out earlier, one of the major difficulties for the academic community in pursuing CIS research is the lack of an effective approach. In the following, the current condition of U.S. research universities is briefly reviewed and a possible approach is suggested for some of these universities to pursue multidisciplinary research.

CHALLENGES FACING THE U.S. ACADEMIC COMMUNITY

The most effective way to engage in team research efforts is to have a team of different experts, each with sufficient knowledge of the expertise of the other team members. However, we rarely can have such a situation. The issue then is how to develop such capabilities to carry out multidisciplinary research. This requires that the universities take a leadership role. A university usually has many disciplinary components as represented by the various departments, programs and/or schools, etc. Further, universities are primarily responsible for the development and preparation of human resources. They are in a unique position to face the challenge of developing the necessary human capital for CIS research. It requires a long-term commitment.

Today, U.S. research-intensive universities are in the midst of a transformation. For the research mission, they want to add new capacities in multidisciplinary research in addition to maintaining their mission to do basic research and to develop new knowledge by individual investigators. In education, they must engage in curricula modification and change because of rapidly developing educational technologies and the vast amount of new knowledge that has become available. This demands a careful review of the role of a professor, from classroom lecturing and grading of homework and exams to developing course materials, and managing the course and the progress of the students.

The changes demanded from research-intensive universities today are both qualitative and quantitative. These changes can only be made effectively by strong and sustained academic leadership in terms of personnel policy, resource allocation and intellectual persuasion. The following are some important steps in developing additional capacity in multidisciplinary research, education and public service activities:

- Establish different faculty performance measures (individuals vs. team members)
- Establish partnership policies among disciplines (incentives for different schools and departments to work together)
- Establish partnership programs with other universities and with industry and government agencies (all levels)
- Emphasize education and research integration.

THE "CENTER OF CENTERS" APPROACH

A mechanism to address CIS research coordination and integration by the academic community is to establish a "Center of centers" jointly with industry and government agencies.

In this "Center of centers," the participants are departments, programs, projects, etc. rather than individual membership of faculty members, consulting engineers or planners. This requires that at least some of the subdisciplines in a department communicate and agree to a collaborative approach. Then, these departments can communicate and agree to joint efforts through the "Center of centers." This represents two levels of coordination/integration. Eventually these cooperating departments will interface with cooperating industries and cooperating government agencies at the third (top) level. At this top level, coordination is again desirable nationally or internationally.

There should be many "Center of centers," each with a unique theme in multidisciplinary education and research.

As an example, to join a university-wide multidisciplinary "Center of centers," a civil engineering department may engage in some of the following activities:

- A faculty team to join the Center representing the civil engineering component on campus. These faculty members have already agreed to cooperate and have already established understanding with the department chairs, the engineering dean and the provost for their role in the team efforts and reward system.

- The civil engineering team joins faculty teams from social science departments to develop a sequence of integrated social science courses concerning CIS for civil engineering students, and a sequence of integrated technology courses for non-engineering, non-science majors.

- Develop a M. Eng. degree program in CIS for civil engineering students by working with industry and the government agency components, requiring all students to work on real-world problems as a degree requirement.

- Ph.D. students in CIS must be supervised by a multidisciplinary team of faculty members. All civil engineering students with a CIS major concentration should have at least one minor from the other categories of disciplinary department/program (category 2A, 2B, 3A, or 3B) below.

Disciplinary Components of a CIS Center

Category 1.	All subfields in civil engineering
Category 2A.	All other engineering fields and applied physical and mathematical sciences programs
Category 2B.	Basic natural sciences and math
Category 3A.	All fields of applied social sciences (law, business, management, urban planning, public policy, etc.)
Category 3B.	Basic social science disciplines

The bottom line is: the "Center of centers" offers an environment for mutual learning and collaboration among current faculty members and participating professionals. At the same time, a new generation of students can be prepared to address CIS problems more effectively.

SOME RECOMMENDATIONS

1. General

   - Individual department/program/schools should first be convinced to take part in the university-wide Center activities

   - At the university level, members of the CIS center should be departments or programs or schools (i.e. "Center of centers") rather than individuals

   - At the national or international level, some organizational effort is necessary to coordinate the activities of all the Centers and to share results (though cyberspace?)

2. Funding Agencies

   - Develop programs to establish a multidisciplinary Center for CIS Education and Research with senior university-wide academic officer(s) as PI(s)

- Funding primarily for support of basic research components and graduate students

- Coordinate the themes of various CIS centers

3. Universities

    - Commitment to establish two different faculty reward schemes for research accomplishments on campus to encourage multidisciplinary education and research

    - Provide significant resources (both cash and in-kind) for curricula and degree program development by multidisciplinary teams as matching funds to industry and government agency support

    - Choose a center theme based on existing strength, regional need, and take part in regional consortium development

4. Industry and Governmental Agencies

    - Join the partnership of a center by providing projects, manpower and problems

    - Contribute in research tasks, teaching and student supervision

SUMMARY

- The academic community is in a good position to establish systems-integrated, multidisciplinary efforts for CIS research.

- The efforts should emphasize mutual learning and mutual appreciation among different disciplines and professions to develop future CIS researchers and educators.

- One approach is to establish "Center of centers," jointly funded and participated in by university departments, government agencies and industry

- Because of the vast number of possible multidisciplinary CIS themes that can be developed, it is desirable to have a national or international committee to coordinate their development and to enhance cooperation and sharing.

ACKNOWLEDGEMENT

The opinion and ideas of the author on the academic approach to multidisciplinary education and research were conceived and developed over the last decade when he served as dean of engineering at the University of Buffalo, State University of New York and director of the Multidisciplinary Center for Earthquake Engineering Research established by the National Science Foundation.

RESEARCH COORDINATION BETWEEN THE U.S. AND JAPAN USING FULL-SCALE MODELS

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Keio University

On January 17, 1995, Japan suffered a very strong earthquake called the Hyogoken-nanbu earthquake or Kobe-Awaji earthquake. Generally, earthquake ground motions in Japan are caused by subducting plates, but the Kobe-Awaji earthquake was a near field earthquake similar to the Northridge earthquake in the U.S. The magnitudes of these two earthquakes were also similar.

We undertook research on the Kobe-Awaji earthquake, including nonlinear analyses. The duration of the earthquake ground motions were very short (about 10 seconds), and vertical ground motions were as strong as the horizontal ones. An exception was the reclaimed areas, where horizontal ground motions (S waves) could not be amplified because of nonlinear effects, but the vertical ground motions (P waves) could be amplified as much as possible. Ground motions of the Kobe-Awaji earthquake were not as strong in linear analyses compared with the Kushiro and Northridge earthquake ground motions, which occurred two years ago and exactly one year before the Kobe-Awaji earthquake, respectively. However, nonlinear earthquake ground motion analysis of the Kobe-Awaji earthquake indicated that they were much larger than either of these earthquakes. This may have occurred because latter parts of ground motion frequencies recorded during the Kobe-Awaji earthquake were slower in the beginning of the earthquake, so structures continued to be damaged by lower frequencies.

In this type of near field earthquake, Dopler effects were very large and north-northeast direction areas were heavily damaged at a distance of about 20 km. Earthquake ground motions of the Kobe-Awaji earthquake were the kinds of shocks that caused ductile steel structures to be brittlely broken by tension force in the Ashiyahama area. Three conditions were necessary for such brittle damages:

1. Size: length of more than 50 cm and thickness of more than 5 cm.
2. Shape: must be closed types like pipes, rather than open types like H shapes.
3. Speed: must be the ground motions of shocks (very quick).

Most collapsed buildings and houses were constructed under old seismic building codes, before 1981, when the present earthquake building codes were initiated. In 1978, the Miyagi-ken-Oki earthquake ground motions damaged so many building structures that the Ministry of Construction became especially sensitive about earthquake codes for buildings. Other civil infrastructures like bridges and roads were not as badly damaged, so the Ministry of Construction was not as eager to change the earthquake codes pertaining to them.

As a result of this event, I would like to propose that earthquake ground motion codes be changed for all infrastructure in Japan and perhaps in the U.S. To test the new codes, we would like to use the existing *shaking table* in Tadotu in Shikoku Island owned by the Nupec. Test structures should be as large as possible, perhaps even full-scale, and include monitoring systems for maximum effectiveness. The tests will be performed in Year 1999. I expect to test steel frame building structures with monitoring systems. I propose that all costs for test specimens be paid by the U.S. side, while costs concerning shaking table tests be paid by the Japanese government.

CLPSI - A Model for Research Coordination for Loss Prevention and Structural Integrity

by

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Mohammad Noori, Worcester Polytechnic Institute

ABSTRACT

The main purpose of this paper is to briefly look at the status of research funding in the area of CIS, why CIS research needs to be coordinated and how a recently organized cooperative effort between academia and industry can serve as a model for coordinated research in loss prevention and structural integrity.

PERSPECTIVE

The perspective presented in this paper in general reflects (though is not totally based upon) the loss prevention perspective of Factory Mutual Research Corporation (FMRC), which is a non-profit organization devoted to advancing loss prevention and control technology oriented toward the reduction of risk. This technology often is used to protect utilities, telephone facilities, refineries, distribution networks, buildings and other components of our national and international civil infrastructure (CIS).

FMRC is sponsored by three large property insurance companies who collectively have among their customers about 35% of the top Fortune 500 companies. It is worth noting that they underwrite all peril coverage for both property damage and business interruption, and have operated for over 160 years on a philosophy that couples sound underwriting with proactive risk management which includes the application of proven loss prevention and control technology.

FMRC combines capabilities in fundamental and applied research, the development of installation and occupancy standards and independent testing for system, product and material certification and/or approval. Our principal areas of expertise include protection systems, materials, structures, process risks, risk assessment methodologies, and large scale testing. Our research results are delivered to insured customers through a field force of about 1000 engineers worldwide in a sister organization. Our work therefore covers the range from fundamental research to practical field application, and our results have direct profound financial implications.

No other company in the world has this focus on loss prevention with this combination of capabilities. Recently, FMRC became the first internationally recognized independent testing laboratory to require dynamic load testing with the issuance of its draft approval test standard for pipe hangars and seismic sway bracing components [1].

BACKGROUND

It is with this perspective, coupled with our civil and mechanical engineering backgrounds and experience, that we examined the status of the source and use of research funds, as it pertains to our CIS, and arrived at the following conclusions.

- Over the last fifteen years, the source of research funds has shifted dramatically from the federal government to industry [2], who conducts research primarily to improve commercial product lines.

- Even with its diminishing role, the federal government remains the largest provider of research funds to independent researchers (mostly universities and other non-profit organizations).

- Of this amount provided by the federal government only a small piece supports CIS research, and that is mostly on or for new technology development with very little for loss prevention.

Yet there are indications that CIS research is effective [3], including the technology for loss prevention, which has resulted in about a threefold decrease in the cost of losses per $1000 insurance in force every 40 years.

Clearly, however, more could be done. The total amount available for CIS research was estimated by CERF at about $1 billion in 1992, which is minuscule when compared to our total investment in CIS, estimated at $20 trillion [3]. Further, it is extremely difficult for a single organization to address large CIS issues because of the variety of disciplines required. Hence, coordinated thrusts are needed for

- effectively using our economic, material and human resources,

- optimizing research performance through careful evaluations of relevance and effectiveness, and

- addressing the complexity of issues, because CIS solutions often involve - and require resolution of - social, economic, environmental and political issues, as well as technical issues.

Multidisciplinary coordinated research efforts are not new or unique; however, not all such efforts have been effective. Some current examples involving industry and addressing CIS issues include:

- Center for Chemical Process Safety - This very successful collaboration has been spearheaded by industry through the American Institute of Chemical Engineers, focusing on safety and risk reduction in the chemical industry. Its success has prompted its work to be broadened and applied to process industries in general.

- NSF Centers for Engineering/Scientific Research, such as the National Center for Earthquake Engineering Research and the Center for Advanced Cement-Based Materials - These Centers have covered a broad range of research topics and have been successful in launching some very strong university-industry partnerships.

- Program for Architectural Surety - A new multidisciplinary effort promoted by Sandia National Laboratory is focused on transferring surety technology (covering safety, security, reliability) from military to private and general public sectors. In addition to Sandia, it involves a potential collaboration among industry, professional societies, universities and other national labs.

- International Multi-Hazard Mitigation Program - The Idaho National Engineering and Environmental Laboratory is currently promoting an aggressive proposal to construct and operate a number of "very large scale" test facilities in order to better understand, and therefore better mitigate, the effects of natural hazard phenomena on structures.

Recently, Prof. Noori, Prof. Shinozuka of the University of Southern California and Dr. Croce authored a presentation [3] at the public hearing of President Clinton's Commission on Critical Infrastructure Protection in Boston on June 6, 1997. In it, we made the following recommendations in order to highlight the direction for future CIS efforts:

- recognizing the limited amount of research funds available for CIS, to increase the NSF budget, especially to the Engineering Directorate, with a careful evaluation of the relevancy of its funded work;

- to use an integrated approach with enhanced connectivity for best informed decisions, encouraging partnerships among university research and professional practitioners with industries that are owners, developers, users and protectors of the CIS;

- to support global partnerships and international cooperation;

- to support emerging and advanced technologies for the development and protection of existing and future CIS; and

- to foster and promote an education paradigm for CIS that starts with primary and secondary schools, continues through undergraduate and graduate schools and includes educators, future practitioners and the general public.

CLPSI

Most of the other papers presented at this joint US-Japan Seminar on CIS are examining and discussing ways to improve, retrofit, upgrade, replace and expand our current CIS. But, in our minds, it is equally, and in some cases more important to protect our current CIS, especially when it is recognized that improving, retrofitting, upgrading, replacing and expanding our current CIS will take decades to accomplish. In the meantime, our societies simply cannot tolerate huge, catastrophic losses; they are traumatic to society, they are expensive, they are inhumane, they result in an inefficient use of resources and they are - for the most part - avoidable with appropriate planning and protection.

Prof. Noori, Prof. Shinozuka and I are therefore also collaborating in the formation of the Center for Loss Prevention and Structural Integrity - CLPSI. The framework has been established as an industry-university partnership, and we continue to build its structure and membership base.

The founding participants are:

University

- Worcester Polytechnic Institute
- University of Southern California
- University of California at Irvine
- University of Puerto Rico at Mayaguez
- Cornell University
- John Hopkins University
- Clarkson University

Industry

- Factory Mutual Research Corporation
- General Electric (Corporate Research and Development Division)
- Xerox Corporation
- United Technologies
- Computing Technologies International
- Neles-Jamesbury

In addition, discussions continue with potential university and industry partners (including some in Japan). Clearly, we want also to have participants who can address social, environmental and other related issues.

The mission of CLPSI is to

- conduct fundamental and applied research that assures major advances in the area of loss prevention and structural integrity, and

- establish relevant academic programs for preparing professionals to meet future challenges.

With this mission, the vision of CLPSI is to become the premier international university-industry research center of excellence for loss prevention and structural integrity.

The goals for CLPSI are to

- generate relevant fundamental and applied knowledge for loss prevention and structural integrity,

- provide a forum for joint industry-academic discussions of multidisciplinary issues, and

- serve as an education center for
  - developing programs, projects
  - training professionals
  - disseminating results.

The capabilities of the Center will (at least) include the following areas

- probabilistic/stochastic methods for design and analysis
- hybrid solution methodologies
- loss prevention and control technology, including hazard mitigation
- thermo-fluid/structure interaction
- control and diagnosis of dynamic systems
- sensors/actuators and smart structures
- laser technologies and micro-mechatronics
- "health monitoring" of structural/mechanical systems,

and we are planning activities to focus on

- workshops, seminars and conferences
- sponsored projects for undergraduates, graduate students
- internships and fellowships for students, industry participants
- technical education programs
- summer work for faculty
- newsletters

The management structure for the CLPSI is shown in Figure 1.

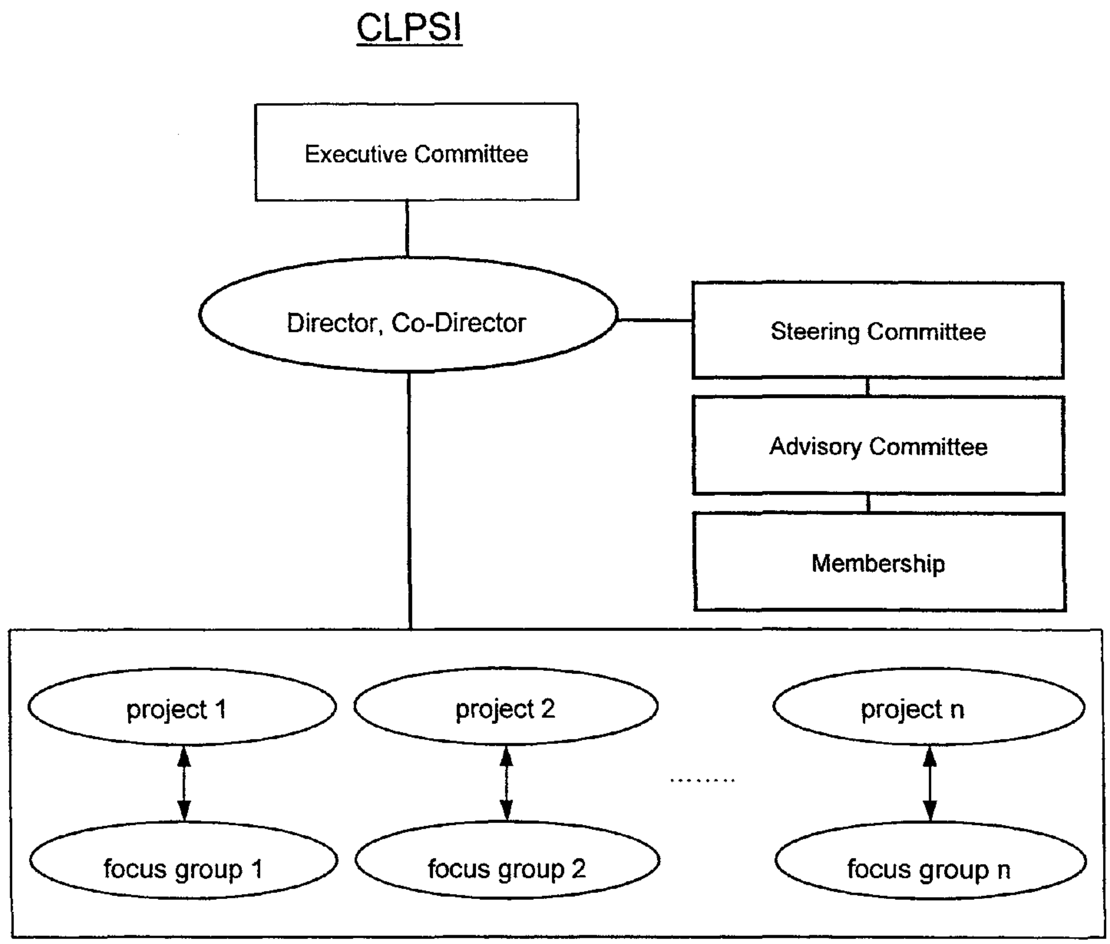

Figure 1. Management Structure of the Center for Loss Prevention and Structural Integrity

The primary structure of the Center calls for a Director and Co-Director to oversee the full range of projects that the Center sponsors. More significantly, a focus group of topical specialists from among members (both industry and academia) is formed and assigned to each project. The purpose of this focus group is to interact closely with the project research team in order to provide timely feedback and guidance as the work progresses. The Director's responsibility is to assure the communication between project team and focus group is happening in a timely and effective manner, reports are prepared and distributed appropriately and the research work is reviewed periodically. The Director also has the responsibility to see that other Center activities (see list above) are implemented and functioning properly.

The Executive Committee is composed of the Director, Co-Director, Chair of the Steering Committee and one other at large industry representative. The primary purpose of this committee is to set policy for the Center. It also has final decision-making authority for dispute resolution.

The Steering Committee, chaired by an industry member on an elected three-year rotating basis, is responsible for strategic planning and direction, evaluation of research work for relevancy and priority setting.

The Advisory Committee, composed of technical representatives for both university and industry members, is responsible for the technical evaluation of Center research activities, suggesting new research topics and providing input to the Steering Committee for strategic planning.

Other aspects of the CLPSI include member fees and benefits, active pursuit of federal research funds coupled with member in-kind contributions, a formal proposal submission and review process, ongoing evaluation of Center membership, involvement, programs, activities and results, and an annual report for members.

CONCLUSION

In conclusion, we believe that societal needs worldwide require an improved and protected CIS, and that research funding trends dictate the need for effective cooperative efforts for protecting new and existing CIS and for planning and implementing future CIS. Several consortia are functioning effectively with collaborative efforts among academia, industry and government, and others are forming. We believe the CLPSI to be a new and effective model for a university-industry collaboration on loss prevention and structural integrity, which are key components for the renewal, assessment, development and expansion of our current and future CIS.

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INDUSTRY PERSPECTIVE IN JAPAN ON CIVIL INFRASTRUCTURE SYSTEM -- ENGINEERING TREND OF JAPANESE CONSTRUCTION COMPANY -

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ABSTRACT

Industry perspective in Japan is summarized and presented. The focus of the contents is at engineering trend of construction company, especially on earthquake engineering problems of : 1) strategies against earthquake disaster and seismic loads for design, 2) application aspect of structural response control techniques, and 3) development of rehabilitation techniques of existing structures.

INTRODUCTION

The social and industrial trends associated with civil infrastructure system have been currently as follows.

1) The tight money policy in the government has been decreasing the investment for public project works, that intensifies industrial competition among private companies.
2) The constructed buildings has shifted from the era of flow to the era of stock, in which buildings are not under scrap-and-build but have characteristics of high durability and long life.
3) The recycling technology of construction by-products and scraped materials such as concrete fragment must have been developed and established because of the cost-up of their disposal and of course saving natural resources for the global environment.
4) Since estimating the Life Cycle Cost (LCC) has commonly carried out, a little high initial cost might be accepted for highly durable buildings.
5) The construction technology directs toward automated construction to save labors and ensure quality, on the one side aims at sharing of overall information through at every stage of design, computation and construction by using CALS (Continuous Acquisition and Life-cycle Support).
6) The importance of earthquake disaster prevention and countermeasure techniques had been made more clear from the 1995 Hyogo-ken Nanbu earthquake.

This paper presents perspective of a few problems on earthquake engineering based on the consideration of the sixth trends expressed above. The contents are consisted by three topics ;

1) Strategies against earthquake disaster and evaluation of seismic loads for design
2) Application aspect of structural response control techniques
3) Development of rehabilitation techniques of existing structure

STRATEGIES AGAINST EARTHQUAKE DISASTER AND EVALUATION OF SEISMIC LOADINGS

It is well known that Japan is one of the countries in Pan Pacific around which earthquakes frequently occur and severe ones sometimes hit. Considering the natural phenomenon, it is important and necessary to take countermeasures against earthquake disaster. The strategies for the program might be set up as shown in Fig.1 : 1) pre-disaster measures, 2) emergency measures, and 3) post-disaster measures. Some example

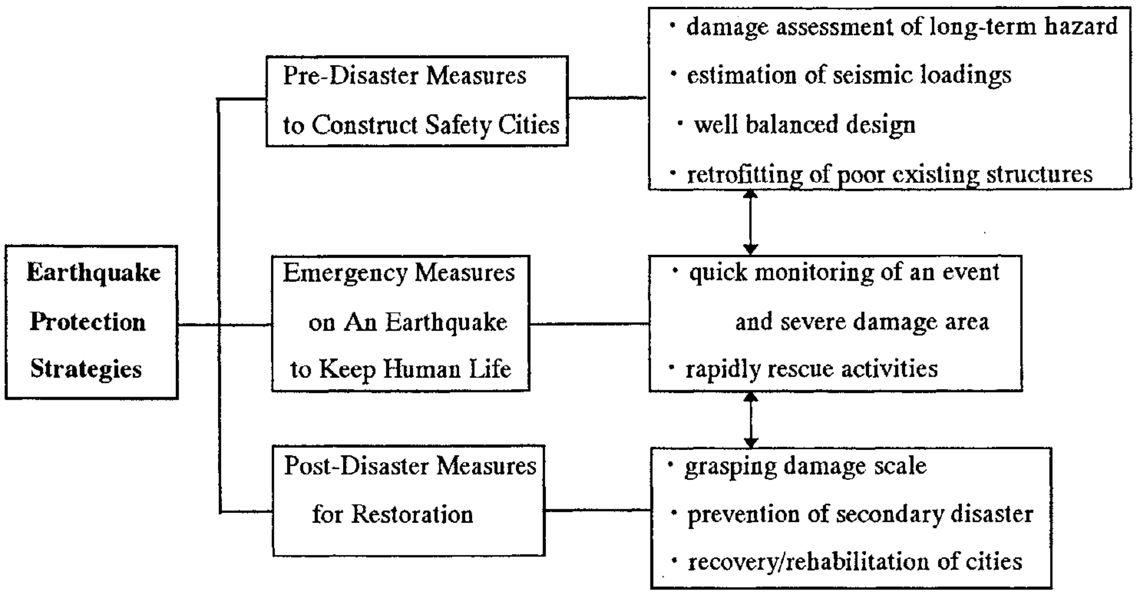

Fig.1 Strategy contents for earthquake disaster mitigation

measures of these items are introduced as follows.

One of the important actions for well balanced design of structures to seismic events which leads to seismic damage reduction is to evaluate earthquake loads for design under sufficient consideration of regional seismicity. For this purpose, earthquake hazard analyses are carried out, the main points of which are 1) set of seismic sources with location, magnitude and occurrence rate in time, 2) determination of earthquake ground motion intensity at a site from an earthquake occurrence using attenuation formulae, and 3) execution of hazard analysis for excitation levels having exceedance probability [1]. In Japan, most of such analyses had been based on the historical earthquake data [eg.2] which are limited to earthquakes detected since 1500 years ago. However the 1995 Hyogo-ken Nanbu earthquake might be caused by an active fault with a few thousands years of occurrence interval. From this essential lesson, a movement arises that active faults must be considered as source data. About 1,000 active faults on Japanese land have been found by field and aerial photographic surveys. For one example, active faults around Tokyo are plotted in Fig.2 as well as a fault of 1923 Kanto earthquake-like occurring on the plate boundary in the Sagami bay with 200 years interval of occurrence. Using these source data, a seismic hazard analysis are conducted for seismic loads of structures at Tokyo.. The resulted base shear coefficients in terms of return period are shown in Fig.3 , which suggests that the base shear coefficient is over 1.0 for long return period and seems to have a upper-limit.

Dense observation net-work system of earthquake ground motions must be an effective tool for quick monitoring of an earthquake occurrence and severe damage area. The K-Net (Kyosin = strong motion) operated by National Research Institute for Earth Science and Disaster Prevention is one such system. The observation system is consisted with 1,000 stations over Japan as shown in Fig.4. The observation data should be quickly opened through internet after an event. By applying this system, we can get effective information where an event occurs, how severe damage suffers and so on. Figure 5 in which contour lines are drawn as the same earthquake ground motion intensity is one schematic instance for us to useful apply to rapidly achieve rescue activities.

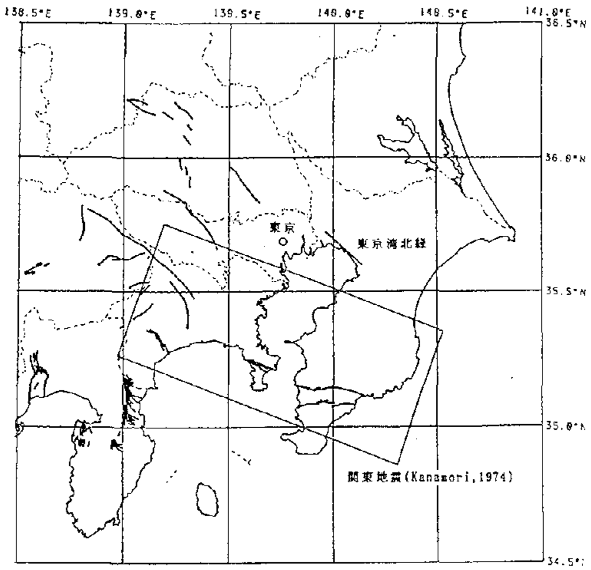

Fig.2 Active faults and a fault on plate boundary around Tokyo

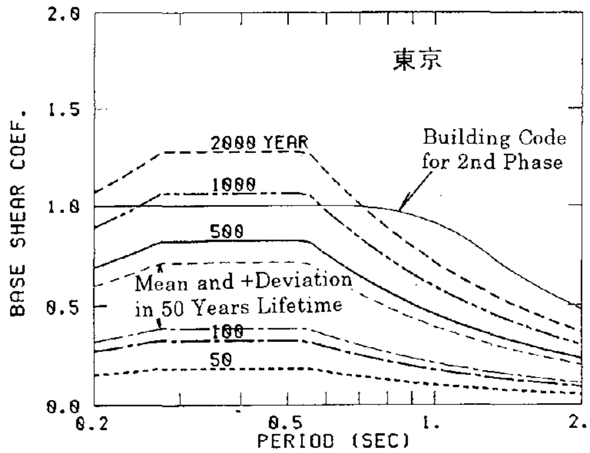

Fig.3 Design loads for building design with return periods

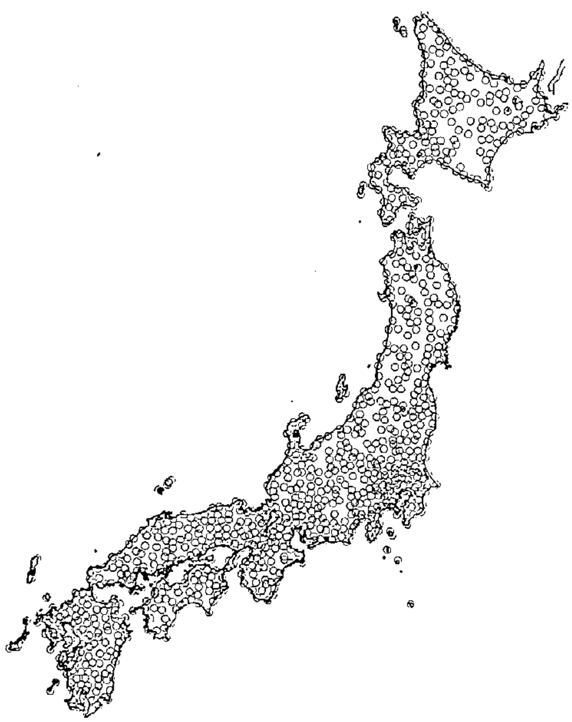

Fig.4 Stations of K-Net (earthquake observation system)

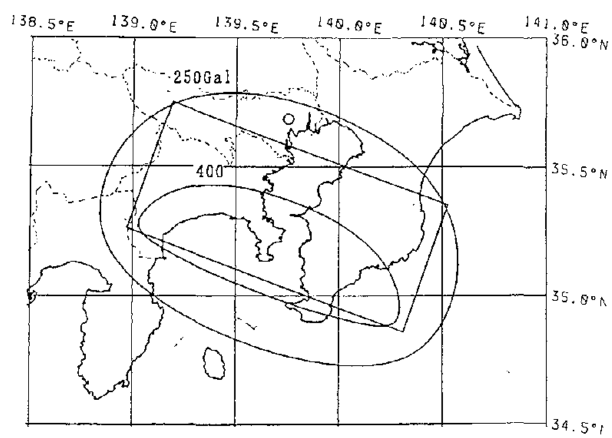

Fig.5 An example of applications of dense observation system
(Schematic contour plot of the same acceleration intensity modeled under an assumption of 1923 Kanto earthquake-like hit)

APPLICATION OF STRUCTURAL RESPONSE CONTROL TECHNIQUES

In the last decade, buildings applied structural (vibration) response control techniques have been constructed in Japan. The aim using the techniques are at 1) elimination of uncomfortable vibration for human life during slight or moderate wind or earthquake motions, and 2) release of human damage from collapse of buildings and especially maintenance of functions and valuable articles against severe excitations by strong wind and/or earthquake ground motions. The techniques can be classified into two main categories, passive and active response control and the types of them are shown in Fig.6. Schematic figures of buildings providing base-isolation, energy absorption or mass damper (include AMD) are illustrated in Fig.7.

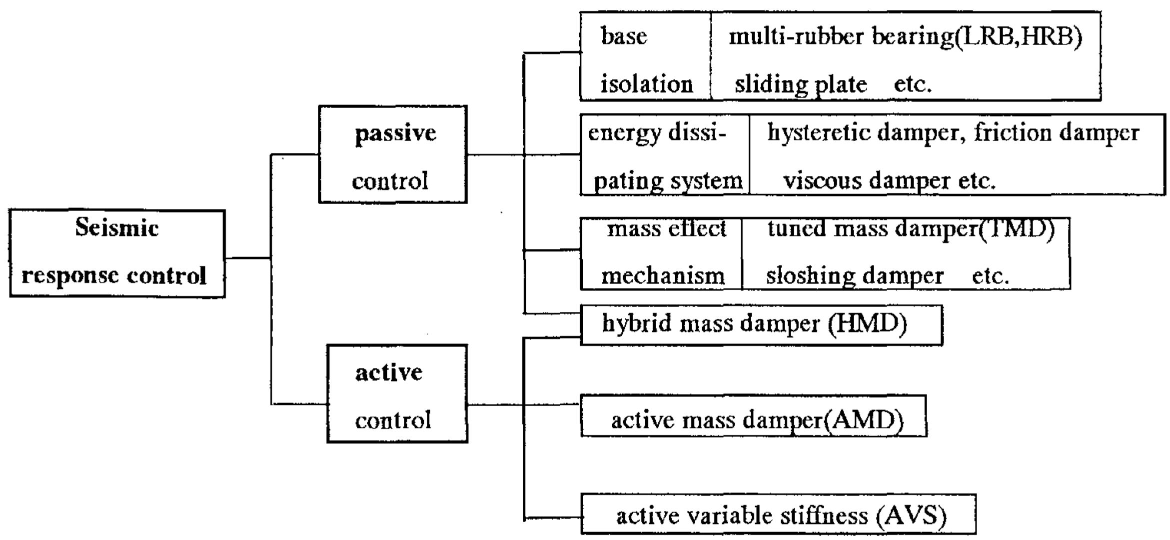

Fig.6 Classification of structural response control techniques

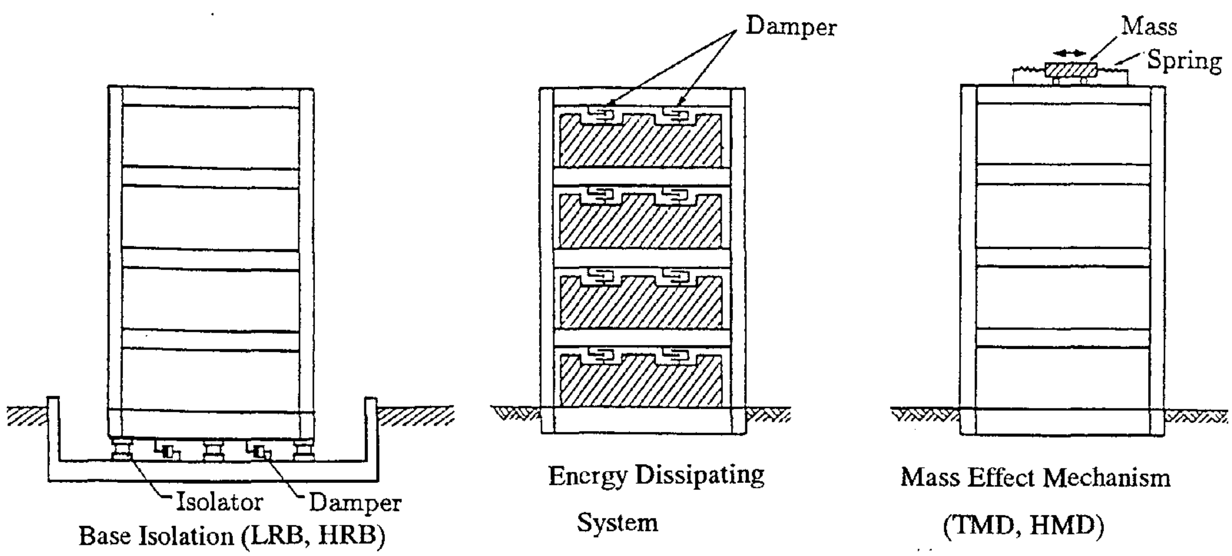

Fig.7 Schematic views of structural response control

The representative technique for buildings constructed by response control is the base isolation put multi-rubber bearings and/or sliding plate between super-structure and foundation-base. Base isolation system has, in general, energy absorption elements also to control excess super-structure drift. Buildings with base-isolation have rapidly increased since 1995 Hyogo-ken Nanbu earthquake. As of July in 1996, 252 base-isolated buildings have been constructed in Japan, of which apartment housings, offices and laboratory facilities account for 40%, 20% and 15%, respectively [3]. Hospitals to function in the emergency after a severe earthquake and art museums to keep valuable articles have been constructed by base-isolation system, as well. This phenomenon suggests that owners and users have been aware of the importance to keep human life, properties and functions even by paying higher initial cost than the ordinary(generally 10% up of structural cost).

Active response control system is composed of the parts of sensor, computer for command and control devices driven by external energy. Twenty tall buildings and towers installed the system have been constructed in Japan, in which HMD installed buildings are 14. , The HMD control system which generally use mass driver at top floor works as active system during slight and moderate vibration of wind or earthquake and as passive system during severe excitation.

Many of these response controlled buildings have conducted vibration observation due to wind or earthquake excitation. Behavior simulated from these records have shown the good results of the system, however we must pay more attention for reliability of their system because of the poor experience against severe excitation.

REHABILITATUION OF EXISTING STRUCTURES

Rehabilitation which includes repair, retrofitting and strengthening is necessary for existing structures to elongate the utility life of buildings, i.e. durability, and to ensure the safety during severe earthquakes. The first one of rehabilitation is related to repair against neutralization and deterioration of materials of concrete structures due to alkaline aggregate reaction and/or salt attack, consequently occurrence of crack of concrete elements and corrosion of reinforcing bars. If the faults having capability of earthquake occurrence are detected by survey, the repair as to remove deteriorated materials and re-construct must be conducted. The second one of rehabilitation is concerned with improvement of structural performance of buildings. The technologies on improvement of seismic performance , i.e. strengthening of existing structures are presented as follows.

The Japanese guideline recommends using the next seismic index, Is, to estimate seismic performance of existing concrete buildings [4].

$$Is = Eo \cdot S_D \cdot T \tag{1}$$

where S_D and T are sub-indexes for configuration and aging of structures, respectively. The sub-index Eo indicates basic seismic performance of a structure, that is expressed by

$$Eo = k \cdot C \cdot F \tag{2}$$

where k is some coefficient for floor level, C and F are the indexes for strength and ductility capacities of structures, respectively. The seismic index is required to be larger than 0.6 not to suffer severe damage on the basis of damaging experience in the past earthquakes , the validity of which had been assured by the 1995 Hyogo-ken Nanbu earthquake.

The relationship of Eqs.(1) and (2) indicates that as shown in Fig.8 strengthening of structures with poor seismic performance may aim at providing more strength and/or ductility or a proper combination of them so as to satisfy required seismic performance. Typical techniques to provide strength are to install walls, braces and side walls, and to add buttress shown in Fig.9 [5]. The ductility demand techniques for columns, especially ones without sufficient shear capacity which might have shear failure mode, are jacketing with steel, carbon fiber etc. shown in Fig.10. The behavior of structural elements rehabilitated by means of their techniques has been investigated and improved in detail.

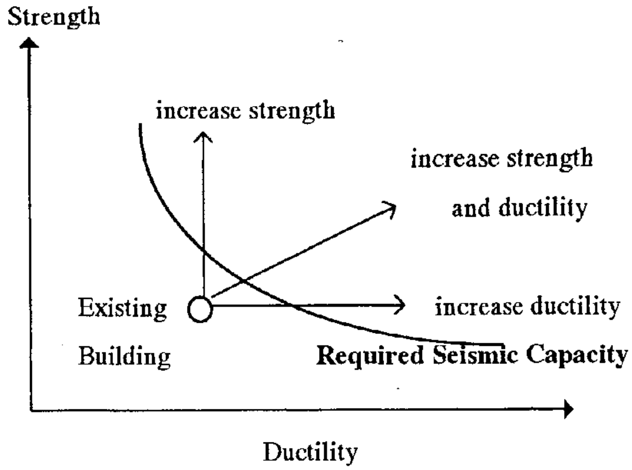

Fig.8 Concept to strength existing structures

The overall techniques for existing structures including the above to mitigate the structural vulnerability and ensure the safety are summarized in Fig.11. Base isolation technique for retrofitting has mainly applied to preserve buildings installed with expensive articles or historical buildings.

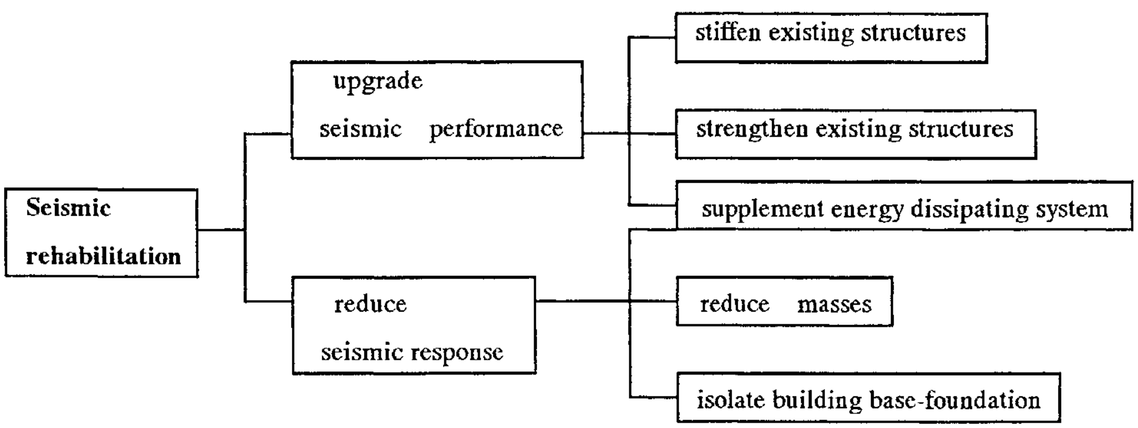

Fig.11 Classification of rehabilitation techniques for existing structures

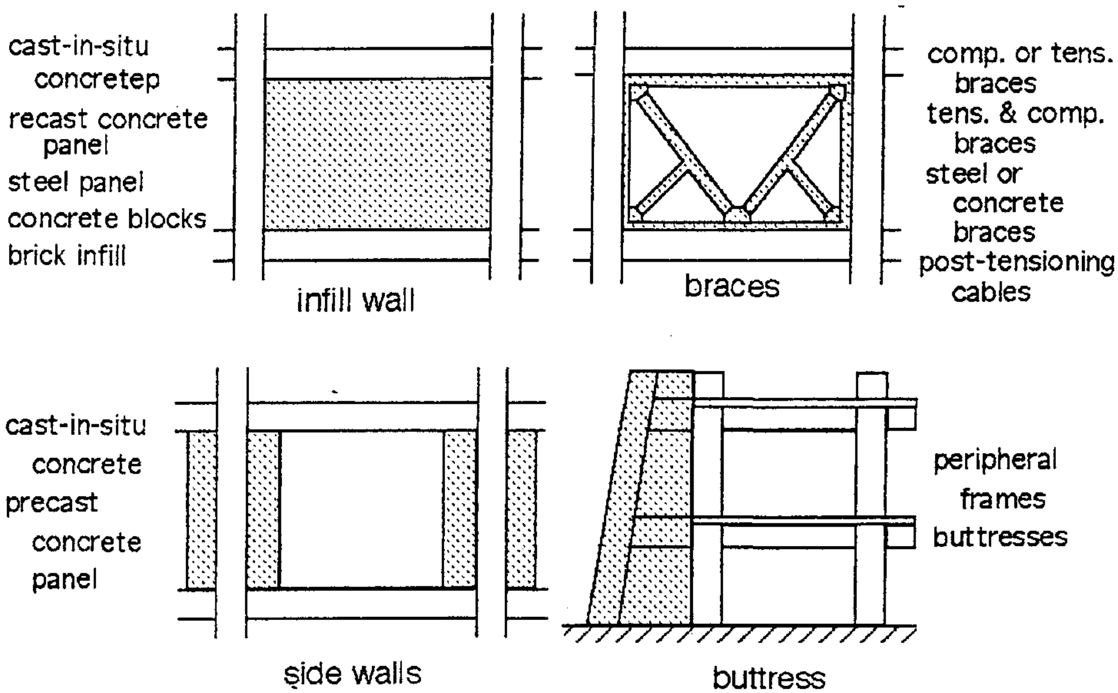

Fig.9 Strength improvement techniques of RC-frames

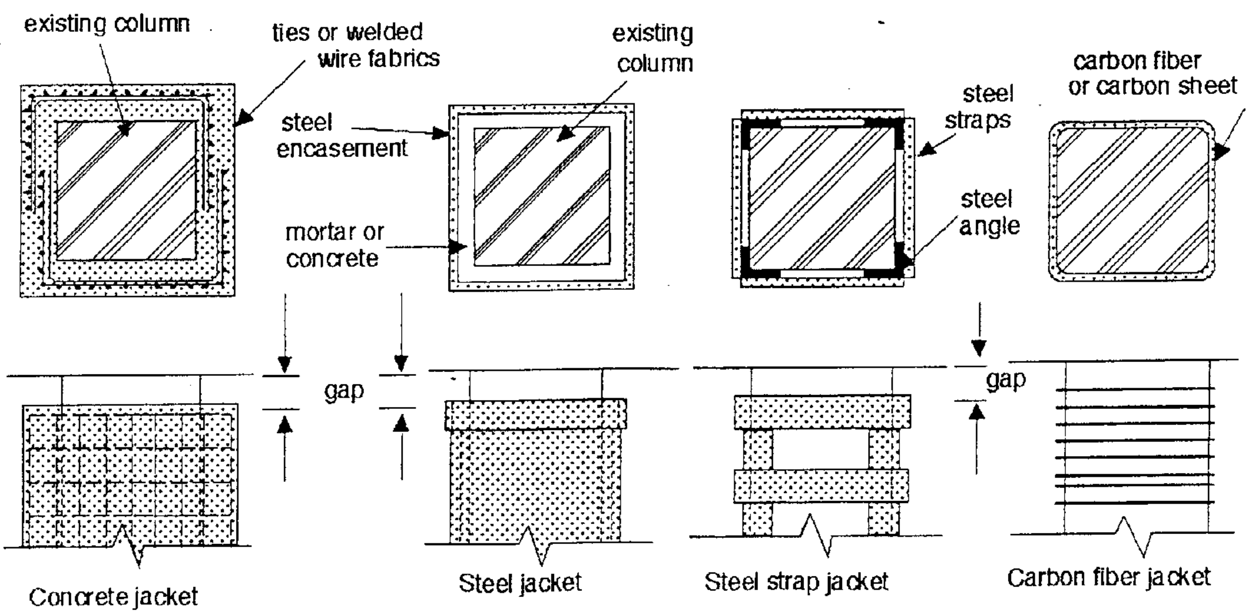

Fig.10 Ductility improvement techniques of RC-columns

CONCLUSIONS

From 1995 Hyogo-ken Nanbu earthquake we have learned the importance to ensure against the human loss and urban collapse. The important points for this purpose are quoted from Ref.6 as follows and keeping in our mind we concludes this paper.

1) Urban Planning for Disaster Prevention
 a. Ensure the provision of open space for community use
  · community recovery efforts
  · post-earthquake mental health care
 b. Disaster management bases
  · recovering functional and operational integrity
  · arrangement of urban infrastructure
 c. Development of techniques for the fire prevention of buildings
  · fire prevention of wooden housings
  · countermeasure against earthquake induced fire
2) Social System for Disaster Mitigation
 a. Risk assessment and risk management
  · organization of disaster prevention network and planning of action program
  · adjustment of life-line and infrastructure
  · strengthening of degraded buildings (especially wooden houses)
  · disaster prevention drill in a community
 b. Crisis management (post-earthquake recovery)
  · rescue operation
  · building inspection to know its performance
  · recovering and reconstruction technologies

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CIVIL INFRASTRUCTURE SYSTEMS RESEARCH: GOVERNMENT PERSPECTIVE

U.S.-JAPAN JOINT SEMINAR ON CIVIL INFRASTRUCTURE SYSTEMS RESEARCH

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ABSTRACT

The U.S. federal agencies' strategy for research, development and deployment in support of the industries of construction has been developed with these industries by the Subcommittee on Construction and Building of the National Science and Technology Council's Committee on Technological Innovation. The vision is a competitive U.S. industry producing high quality, economical, sustainable and hazard resistant constructed facilities. Goals have been established for better constructed facilities and improved health and safety of construction workers. Numerous cooperative activities addressing the goals are underway.

INTRODUCTION

Construction is one of the Nation's largest industries. In 1996, new construction was $569 billion [1]; with renovation added, the total volume of construction was about $900 billion, 12 percent of the GDP. Since constructed facilities shelter and support most human activities, their quality and economy is vital to the competitiveness of all industries and everyone's quality of life.

The industries of construction include: designers, builders, and manufacturers of materials and equipment installed in constructed facilities or used in construction. Because of specializations among owners, designers and builders, the industry is divided into sectors: residential, commercial/institutional, industrial and public works, which amount respectively to 43, 20, 12, and 25 percent of new construction. Civil infrastructure systems include the public works sector and utilities in the industrial sector.

VISION FOR CONSTRUCTION

The vision for the industries of construction, gleaned from numerous industry, government and academia fora is:

- High quality constructed facilities support the competitiveness of all U.S. industries and everyone's quality of life.
- U.S. industry leads in quality and economy in the global marketplace for construction products and services.
- The industries of construction and constructed facilities are safe and healthful, energy efficient, and sustainable in use of resources.
- Natural and man-made hazards do not cause disasters.

CONSTRUCTION GOALS

In spite of the economic importance of constructed facilities and the industries of construction, relatively low emphasis is given to construction-related research by the private and public sectors in the U.S. U.S. private and pubic sector annual investment in research in construction is less than 0.5% of sales, whereas the all-industry average is 3.7% of sales. The picture appears to be similar in most other developed countries; Japan [2] and Sweden [3] are exceptions.

This is representative of the conditions that inspired the President of the United States, in 1993, to establish the National Science and Technology Council. Among the principal objectives of its Committee on Technological Innovation [4] are:

- Set national priorities for technologies that enhance U.S. industrial competitiveness.
- Coordinate federal R&D activities to minimize duplication.
- Involve the private sector in setting federal R&D priorities.
- Identify technology needs of industries particularly important to the U.S. economy.
- Encourage coordination with industry and academia.
- Monitor foreign technology advances.

The fundamental objective is <u>not</u> to increase government R&D, but to facilitate and increase industries' investments in highly competitive, innovative products and services.

In recognition of the importance of the industries of construction, the Subcommittee on Construction and Building (C&B) was formed in March 1994.

C&B, in order to demonstrate to policy makers in the private and public sectors the values of investments in innovation, and in consultation with leaders of the industries of construction [5], formulated goals for the life cycle performance of constructed facilities:

- 50% reduction in project delivery time.
- 50% reduction in operation and maintenance.
- 30% increase in productivity and comfort.
- 50% fewer occupancy-related illnesses and injuries.
- 50% less waste and pollution.
- 50% more durability and flexibility.
- 50% fewer construction illnesses and injuries.

The baselines are average performance of the industries in 1994. The targets are to have verified practices, capable of meeting the goals with reduced life cycle costs for owners, available for construction projects in 2003.

The construction goals are expressed in performance terms. An important challenge is to characterize and quantify the baselines and define measures for progress. C&B and its participating federal agencies are working with industry to establish these baselines and measures.

The construction goals are intended to express the desires of leaders of the industries of construction; they are not a government mandate to the industry. A focused workshop [6] with industry leaders defined the priorities of industry sectors and described the goals as National Construction Goals. C&B is encouraging industry to inform the federal agencies of the federal or federally-funded R&D needed to achieve the goals.

ADDRESSING CONSTRUCTION GOALS

C&B, drawing on many research fora and interactions with industry, has identified the following technical opportunities to obtain progress toward the goals:

- Information and decision technologies. Advanced information technologies can improve the flow of information to decision makers, and improve understanding through multi-media information processing.

- Automation in design, construction and operation. Attention is needed to exploiting the potential of automation while avoiding unwanted side effects.

- High performance materials and systems. There are great opportunities for "designed" materials and systems fit to the specific project needs.

- Environmental quality. Objectives include energy conservation, air quality, and environmental sustainability. Needs and opportunities for remediation and use of contaminated sites provide particular challenges.

- Risk reduction technologies. Natural (fire, earthquake, wind, flood) and manmade (fire, terrorism, toxic discharges) hazards with increasing urbanization pose growing risks of losses requiring advanced, performance-based technologies for mitigation.

- Human factors knowledge. Performance ultimately is human functionality, safety and comfort; much research is needed to make human performance, as affected by the built environment, measurable and predictable. While the importance of human factors knowledge is evident, at present, many industry leaders are doubtful about our abilities to get valid and useful results from research. We should, at least, invest modestly and systematically in human factors research. When successful techniques are evident, this will be a most fruitful area for performance-oriented research.

- Performance standards system. The system of performance standards and conformity assessment is essential to acceptance of innovations and for focus of resources on effectively achieving construction goals.

Industry leaders, in all of our interactions, have emphasized that private investment in innovations is severely inhibited by the existence of barriers to profit from investments. The barriers most often cited are:

- Lack of leadership in private and public sectors. The current U.S. Administration, with its establishment of the National Science and Technology Council and its assignment of unprecedented high priority to federal R&D for construction [7], has shown leadership. This is not yet reflected in Congressional responses to Administration requests for appropriations. The construction goals approach is attracting extraordinary support of leaders of industry for improved construction technologies and for removal of barriers to innovation.

- Regulatory barriers result from the myriad, uncoordinated approvals required for a project from local, state and federal regulatory authorities. These generate long and costly delays for conventional projects; fear of yet longer delays inhibit innovations. Our Administration encourages federal agencies to work together and with state and local agencies to streamline regulatory processes. This work is beginning.

- Liability from real or imputed failure of products to perform over the service life as the customer or other injured party expected discourages investment in innovations. Federal agencies, and other organizations, can provide low risk test beds to reduce liabilities for innovators.

- Adversarial relations in construction projects often discourage each participant from innovating. Partnerships among project participants are needed.

- Financial disincentives for innovation exist because most construction organizations are too small to invest substantially in research, and because of the difficulty of protecting intellectual property when innovations are evident to observers and the workforce often changes employers. Some of these barriers can be overcome by consortia for innovation involving industry and researchers from academia and government.

- Lack of skilled workforce extends from shortages of replacements for retiring skilled tradesmen to the loss of knowledgeable corporate real estate executives from corporate downsizing. Solutions include education and training in conventional and innovative technologies, improvement of the work environment, automation and knowledge systems and the development of innovative professional and business services.

Knowledge from performance-oriented research must be transferred to practice so that it may be used to produce private and public benefits. Principal deployment activities include:

- Standards and practices are formulated to integrate new knowledge into the various contexts for its practical use.

- Education and training are required for the implementers of innovations. Collaborations between employers, employee organizations (such as professional societies and trade unions) and innovators are needed to fund and develop the educational syllabi.

- Demonstration projects, as noted above, can provide low risk test beds for innovations and show practical people the benefits to be obtained from and the techniques required in implementation.

COLLABORATIONS AND ACCOMPLISHMENTS

Leading U.S. private sector organizations have taken initiative to convene sector leaders to define: the goals most important to the sectors of the industries of construction, and the practices and research results that will be needed to achieve them. For each sector, many other organizations are participating with the lead organization. The sectors and their lead organizations are:

- Residential, the National Association of Homebuilders Research Foundation.

- Commercial/Institutional, the National Institute of Building Sciences.

- Industrial, the Construction Industry Institute.

- Public Works, the American Public Works Association.

The Civil Engineering Research Foundation is coordinating these private sector efforts and preparing a synthesis of their recommendations for guidance of the federal agencies participating in C&B. The findings are expected to include activities of:

- The industries of construction to:

  - Remove barriers to innovation.
  - Develop baselines and measures of progress toward goals
  - Invest in improved products and services.
  - Lead the world in quality and economy.

- Government Agencies and Researchers to:

  - Conduct R&D enabling private investments.
  - Help remove barriers to innovation.
  - Support deployment of innovations.

While the Subcommittee on Construction and Building is efficient in convening the federal agencies for planning with industry groups, actual cooperations in research will be accomplished by consortia involving one or more companies and one or more agencies. To guide industry and agencies in establishing such collaborations, a Collaborations Workshop [8] was held with leaders of industry and representatives of federal agencies. The workshop report describes: cooperative mechanisms protecting industries' intellectual property, research needs and opportunities, and the capabilities of the federal laboratories.

Example success stories are cited. Owners, designers, builders and suppliers for process plants are working with the National Institute of Standards and Technologies to develop international information exchange standards supporting automatic transfer of data among project participants throughout the life cycle. Manufacturers of seismic base isolation devices have worked with universities sponsored by the National Science Foundation, the Veterans Administration and the General Services Administration to develop systems for cost-effective retrofit of federal buildings. Five asphalt paving equipment manufacturers, the National Asphalt Paving Association, the Federal Highway Administration and the National Institute for Occupational Safety and Health have developed pavers that significantly reduce the exposure of workers to asphalt fumes. Global positioning system software developed by the Army has been integrated with controls developed by Caterpillar to provide autonomous construction vehicle navigation.

The delays and other costs of obtaining regulatory approvals for new products are great barriers to private investment in new product development. With support from federal agencies and industry groups, the Civil Engineering Research Foundation has organized evaluation centers to assist manufacturers to obtain a single, authoritative evaluation that will be recognized by regulators and users nationally and internationally. Centers include: the Highway Innovative Technology Evaluation Center, the Environmental Technology Evaluation Center, the Civil Engineering Innovative Technology Evaluation Center, and the National Evaluation Service Building Innovation Center.

Delays and costs of obtaining regulatory approvals are both a barrier to use of innovative products and practices in construction projects and an important factor in increasing costs and obsolescence of constructed facilities. As an element of the U.S. Innovation Partnership between federal and state governments, and with support from the federal agencies, the National Conference of States on Building Codes and Standards has organized a program Streamlining the Nation's Building Regulatory Process to develop and gain the adoption of a package of model reforms which when adopted by federal, state, regional or local governments will enhance public safety, economic development, and environmental quality while reducing by as much as 60% the amount of regulatory processing time it takes to move projects from the initial step of zoning approval through to the last step of issuance of the certificate of occupancy.

The High-Performance Construction Materials and Systems Program (CONMAT) has been organized by twelve materials trade associations to develop and commercialize the high performance materials and systems needed for 21st century infrastructure. CONMAT members have underway about $250 million in R&D shared roughly evenly between the private sector and federal agencies. It will be desirable to create similar focal points for federal and industry cooperation in the areas of mechanical systems and furnishings.

The National Institute of Standards and Technology has established the Manufacturing Extension Partnership - a nationwide network of extension centers and experts co-funded by states and local governments that provides hands-on technical assistance to smaller manufacturers. The Manufacturing Extension Partnership is studying with industry the potential for establishing a Construction Technology Extension Program.

The residential sector has expressed two major goals: reducing housing costs and improving durability. Their Action Plans for Achieving High Priority Construction Goals in the Residential Sector [9] cites seven strategic approaches: information infrastructure, methods for assessing and increasing durability, improving production efficiency, streamlining regulatory processes, understanding the performance of light frame structures - particularly in high winds and earthquakes, fostering commercialization of innovations, and expanding markets and marketability. The Subcommittee on Construction and Building is working with the residential sector to create a focused program for advancing housing technology.

The Construction Industry Institute is comprised of major owners of industrial and infrastructure facilities and the major contractors and suppliers serving these markets. It conducts research and implementation on best practices to improve the capital investment process and is exploring collaborations to achieve breakthrough improvements in practices. In a recent workshop with federal agencies [10], eleven priority topics were identified: business leaders' needs, technology transfer and implementation, global standards and conformity assessment, model based design, integrated project information to the field, information exchange protocols, production line techniques in construction, remediation of hazardous materials and sites, aids/ tools/robotics for the productivity and safety of construction workers, safety practices, and behavior modification for safety.

The American Institute of Architects, the International Association of Corporate Real Estate Executives, the International Facility Management Association, and Johnson Controls, Inc. have organized the Building Performance Consortium to address the goal for improvement of the productivity of building occupants. The Consortium is holding annual National Summits on Building Performance involving leaders of industry and government and cosponsoring with the federal agencies studies to better define the effects of building environments on their occupants' productivity.

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CURRENT RESEARCH ACTIVITIES RELATED TO THE PERFORMANCE-BASED DESIGN METHODOLOGY FOR BUILDINGS IN BRI/MOC, JAPAN

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ABSTRACT

This paper describes the current research activities related with the performance-based structural design and engineering system that is to function in the near future in Japan. These are under the initiative of the Building Research Institute (BRI), involving many experts and professional communities related to the design of building structures.

INTRODUCTION

The current design codes do not clearly require target structural performance, whereas design procedures and rules are definitely and prescriptively specified as code provisions. Therefore, under this inflexible system, the performance of a designed structure cannot be clearly predicted and described even by a structural engineer who designed the structure. Furthermore, structural design itself has become "calculation" just to keep restrictively the prescriptive specifications under code provisions.

In the future design system, on the contrary, the target or objective structural performance should firstly be defined as a clear picture, and then in order to attain the performance, a structural engineer may choose an appropriate design method and procedure. Thus, the most important thing toward the future design system is to facilitate a new design system including codes, where a structural engineer can determine target performance and realize it by choosing an appropriate design procedure. It is not until this is realized that a structural engineer can clearly explain the objective and expected performance of a structure designed by him/ her to a client.

STRUCTURAL PERFORMANCE

The performance of a structure is not limited to structural safety; the word of structural performance has even wider concepts. That is, the concepts includes every structure-related performance; safety, reparability, durability, human comfort, maintainability, constructibility, aesthetics and so on. Furthermore, performance against various actions such as loads and forces should be taken into account. The basic definition and description materials of structural performance have already been proposed by the author elsewhere (1).

PERFORMANCE-BASED CODES AND SPECIFICATION CODES

In recent years, structural technologies of building structures are making a rapid progress. For instance, various technologies such as new structural materials, base isolation and response control devices/systems have been developed. However, the current codes for building structures in Japan cannot match new structural materials and systems, since they are regulated for conventional structural materials and methods and most of them are of specification types that specify, in detail, materials to be used, sizes and spacing etc., in most cases with numerals. Therefore, it becomes necessary to convert structural codes from current specification types into performance-based types that prescribe objective performance to be accomplished in designed structures, and to allow structural engineers broad ways in selecting new structural materials and methods for achieving the required performance.

BRI/MOC ACTION TOWARD NEW SYSTEM

Considering significant needs for a new design and engineering system on the basis of performance, thc Building Research Institute, Ministry of Construction, has initiated a three-year National Comprehensive R&D Program entitled " New Structural Engineering System" since the fiscal year of 1995, in coordination with structure-related communities such as those of structural engineers, contractors, industries, academy and so on.

The most critical mission of the Program is to show possible ways to a new structural design and engineering system where a structural engineer can clearly explain the objective and expected performance of a building structure designed by him or her. For this, a lot of issues must be studied, discussed and solved; as a result of preliminary studies for formulating the Program, the major subjects to be dealt with in the Program have been identified, and the committees for the action have been organized.

In this Project, we are examining the following two technical subjects:

1. Develop performance based structural design system. Here, we examine not only seismic performance but also other performances related to building structures,
2. Convert specification codes into performance codes in the Building Standard Law, and furthermore, social issues related to structural engineering are being dealt with as follows:

Propose a menu of social systems which can effectively support performance-based design and engineering in terms of qualification of structural engineers, building confirmation procedures, quality assurance of design and so on.

We have organized a Synthesizing Committee. This Committee steers and manages this comprehensive project. Under the Committee, we settled three Technical Committees. The tasks of the Technical Committees are as follows:

Technical Committee 1: Evaluation of Performance : The Committee is examining the concepts of performance of structures, definitions of performance, measures of performance, loads and forces and their design levels. Under this committee, working groups are settled.

Technical Committee 2: Target Performance Level : The main purpose of the Committee is to clarify the recognition and the requirements of owners and users for building structures, and to investigate desirable performance items and levels of building structures from the side of the society. At present (in 1996-1997), as the important subjects, the Committee are collecting and arranging the research data to clarify the performance levels and investigating the methodology to determine the performance levels.

Technical Committee 3: Social System : The purpose of this Committee is to propose a menu of ideal social systems matching the purpose of performance-based design and engineering. Thus, the Committee deals with the problems on design review system, qualification of structural engineers, role of building officials and so on.

In 1995, we began with gathering information on past surveys to gain a better understanding on all elements related to the social system issues, including structural design, construction supervision, building confirmation and inspections, the "kentikushi" system (the qualification system of building engineers in Japan), design codes and regulations. At the next step, we put in order the findings and prioritized problems and issues which must be studied.

On the other hand, in April of 1996 it was officially decided by MOC that the Building Standard Law of Japan should be converted into that based on performance. Considering this change of policy, the Committee has also begun to deal with social systems preferable under the "new performance-based building standard law" that will be realized in a few years, in order that the future law will function smoothly and efficiently

NEED FOR INTERNATIONAL COORDINATION/COOPERATION

In accomplishing a new structural design world in each country, it is hoped that international harmony and consistency in the conceptual framework on the new system will be attained considering the progressing borderless world. For this aim, an "international committee" is under consideration to be formed in the BRI National Program.

Partly for an initial step of the above action, a Japan-U.S. Workshop on Seismic Building Codes (Performance Based Seismic Engineering of Buildings) was held at the Earthquake Engineering Research Center, University of California, Richmond, CA. The main objectives of the workshop were:

1) to discuss the adequacy and efficiency of present Japanese and U.S. seismic codes in light of the experience derived from recent (1985-1995) significant earthquakes (particularly the 1994 Northridge and the 1995 Kobe earthquakes), and from studies conducted in the last ten years, and to identify problems whose solutions need improvement,
2) to discuss what can be done to improve present seismic code approaches, with particular attention to the interim recommendations for engineering procedures that the Vision 2000 Committee of the SEAOC has recommended for obtaining buildings with predictable and defined performance; and
3) to discuss the development of a work plan and the organization that will be needed to formulate the basic concepts and framework that can be used in Japan and the U.S. to develop practical seismic code regulations whose applications will result in facilities whose seismic performance will be predictable with a reasonable degree of confidence under the different types of earthquake ground motions that can occur during their service life.

The recommendations of the Workshop should be thought very significant to break through current blocked situations in the structural design/engineering world and to make progress in international harmony related to a future structural engineering system based on performance.

RELATED SEISMIC RESEARCH ISSUES

Among each step in general structural design procedures, even though it is not performance-based, the followings might be the main parts, i.e., (1) Evaluation of design external loads (or disturbances), (2) Confirmation of required structural performance. When we restrict ourselves to discuss only seismic design related issues. The followings might be our on-going activity in BRI.

Earthquake Observation in Buildings and Surrounding Ground

After 1995 Hyogoken-Nanbu earthquake, considerable number of strong motion seismographs were installed nationwide, by especially STA (Knows as Kyoshin-Net) or JMA. Most of these installations were in the ground.

We have installed and renewed strong motions seismographs in 47 sites nationwide since 1957. Most of the seismographs are set in building structure to record the behaviors during severe earthquakes. With these seismographs, many strong motion records during damaging earthquakes such as the 1964 Niigata earthquake, the 1978 Miyagiken-oki earthquake, and 1993 Kushiro-oki earthquake etc. have been collected and these are sometimes used for the dynamic analyses of new buildings for seismic design.

We also installed seismographs in grounds. Eleven recording stations are deployed around Sendai, Japan. Each station has three seismographs with three component sensors. They are installed from surface to so-called Engineering Bedrock. These records provides data on effect of surface geology on seismic motions. (3)

In addition, we also installed seismographs around Tokyo, in both buildings and grounds. These are expected to examine the difference between building and ground response and to serve as the material for evaluating the soil-structure interaction, or effective input earthquake motion to buildings.

Evaluation of Design Seismic Force on Buildings

Investigation on structural damage during the Hyogo-ken Nanbu (Kobe) Earthquake is still continued, however, our survey on overall building damage shows that most of the building damage occurred to ones designed with older codes, and the buildings designed under the current code mostly survived the earthquake without collapse. Therefore, it was concluded that the urgent reevaluation of the level of the current design seismic force was not necessary in the building design. On one hand, the strong motion records during the 1995 Hyogoken-Nanbu earthquake shows that distinct difference from the large scale interplate earthquake occurring in ocean region of Japan. The influence of active fault was not considered in the evaluation of seismic design force to date since the destructive motion from active fault was not recognized with the recorded motions and its recursive characteristics is not clearly understood.

The active fault existing near densely populated area causes near field earthquake ground motion to the area, when dislocation of the fault takes place very rapidly. The spatial distribution of the ground motion around the fault is strongly dependent on the rupture mechanism and process of the fault, i.e., the velocity and direction of the rupture, sizes of asperity, etc. The fault rupture process of the recorded earthquake can be detected by a post-(pre)diction. However, the uncertainty of the future motions is so large since we cannot predict the rupture process of the earthquake to occur. At present, the synthetic technique using the Green's function becomes popular in strong ground motion evaluation. In this case, the empirical rule for rupture process, if any, may be necessary to minimize the uncertainty.

Another issue is the relationship of the intensity of recorded motion with the extent of structural damage. In 1994 Norhtridge earthquake, high peak acceleration reaching 2G was recorded. However, a significant damage was not observed nearby the recording site. We can find similar examples in 1995 Kobe earthquake. For these issues, we should consider the mechanism of propagation of seismic motion in the ground and also the concept of effective input motion to buildings with soil-structure interaction.

For these research issues, Japan and US can collaborate in experimental and analytical activities, by making efficient use of the experiences with considerable sacrifices in each country.

A few years ago, we have established a guideline for composing design earthquake motion for design of high-rise buildings and base-isolated buildings in cooperation with the Building Center of Japan (BCJ). The guideline is officially used as such in the design approving process in BCJ.

The followings are the basic concept of our proposal for prospective design seismic force in performance-based design code (BRI proposal).

(1) Basic design spectrum defined at Engineering bedrock
Engineering bedrock : soil layer with shear wave velocity of lager than approx. 400m/s
The spectra with return periods of 500, 50 5 years are used to examine safety, restorability, serviceability, respectively.

(2) Evaluation of Site response from the surface geotechnical data
Key parameter may be the fundamental period of ground above the engineering bedrock.
Non-linearity of soft soil is taken into consideration for safety examination
Topographical effect in surface geology is taken into consideration in case of slope and sediment-filled ground. The method was developed in the MOC general research project introduced in the next section.

(3) Effect of Soil-Structure Interaction
Allow analysis assuming the fixed based model.

(4) Near Source Effect on seismic motion
The near source site is much hazardous especially in 1 to 2 second period component due to the rupture directivity and dimension of asperity on the seismic fault.
The effect will be considered by the length of fault, slip rate, source-to-site distance.

Seismic Disaster Mitigation Techniques in Urban Soft Soil Area

This is an MOC general research project aiming to develop the evaluation techniques for effect of surface topographical irregularities on ground motion (2), for evaluate the response of underground structure such as piles including the liquefaction condition, for design guidelines for soil improvement. these three research topics have finished at the end of 1996 Japan fiscal year. Another topic on rehabilitation of existing buildings with new technologies is still underway until 1998. The new technology includes strengthening, response control using base isolation or energy dissipation dampers, and consideration of avoiding excessive force on superstructure and foundation by preceding yielding in pile caps.

International Cooperative Research Project

Cooperative research activities on the earthquake resistant technology for civil infrastructure and buildings have been implemented under the auspices of the US-Japan Joint Panel on Wind and Seismic Effects. Research activities introduced here can be addressed by collaborative efforts of existing scheme

Development of Upgrade Technology of Structural Safety with New Steel

During 1995 Hyogoken-Nanbu earthquake, brittle fracture was found in the super high-rise steel residential buildings. This failure is supposed to be caused without energy dissipation due to plastic deformation. This behavior is beyond our intention. Hence, we started a project for the cause of such phenomenon, and countermeasures. The research includes characterization of the materials and the influence of fabrication, design etc. The welded steel connection failure was also found during 1994 Northridge earthquake

Japan-US Cooperative Research on Structural Experiment - Phase 5-

After the 5-year research project on hybrid structures, Japan side are planning its 5th phase as research on smart structure. The concept of smart structure is a self-reliant building structural system that fully exhibits its intrinsically equipped performance, and can adapt to the general environment and also minimize its lifecycle.

CONCLUSION

We have shown the recently finished and currently ongoing research project in BRI/MOC. The 1995 Kobe earthquake strongly influenced the seismic research activity in Japan. Instrumentation and new construction of research facilities as well as research project for the purpose of reducing the seismic disasters have been unprecedentedly funded as kind of research infrastructures. The Japan-US research coordination/cooperation will benefit both sides by sharing the experience and/or new technology.

REFERENCES

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(2) I. Okawa, 1996, "An Evaluation of Local Site Effects on Strong Motion Considering Micro-landform and Deep Subsurface Structure", Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, June No.1788 (CD-ROM)

(3) Y. Kitagawa, et al. 1988, "Dense Strong Motion Earthquake Seismometer Array at Site with Different Topographic and Geologic Conditions in Sendai", Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo & Kyoto, August

(4) I. Okawa, et al., 1994 "A Guideline for Composing Design Earthquake Ground Motion for Dynamic Analysis of Buildings", Proceedings of the 5th National Conference on Earthquake Engineering, Vol.2, pp.273-282, Chicago

Table 1 Basic Structural Performances (required performance)

Safety	
Purpose	to avoid risk endangering human lives inside and outside the buildings
Contents	to prevent fracture appropriately regarding safety on structural frames, members, interior and exterior finishing materials
Restoration	
Purpose	to secure the repairing ability to the damage caused by outer stimuli to the buildings
Contents	to control deterioration and damage degrees on structure frames and members, interior and exterior finishing materials, utensils and foundation in the light of repairing ability
Serviceability	
Purpose	to secure serviceability (function) and habitability of the buildings
Contents	to eliminate harmful deformation or vibration on structural frames and members, interior and exterior finishing materials, facilities and foundation in the light of serviceability

Table 2 Evaluated Objects and Fundamental Performance

Performance Objects	Safety Limit (keep of Human Life)	Damage Control Limit (keep of Properties)	Serviceability Limit (keep of Function)
Structural Frame	Non-destruction*1 to human life	with Assigned Damage	Non-Harmful*2 Defect or Vibration for normal use
(Structural Members)	Non-destruction*1 to human life	with Assigned Damage	Non-Harmful*2 Defect or Vibration for normal use
Non-structure Members	Non-Destruction*1, Drop, Scatter to human life	with Assigned Damage	Non-Harmful*2 Defect or Vibration for normal use
Equipments / Machines	Non-Overturn, Drop, Movement by deformation or vibration of structural frames or members	with Assigned Damage by deformation or vibration of structural frames or members	Non-Harmful*2 Defect or Vibration of structural frames or members for normal use of equipments / machines
Fixture	Non-Overturn, Drop, Movement by deformation or vibration of structural frames or members	with Assigned Damage by deformation or vibration of structural frames or members	--
Soil	Non-destruction*1 (decrease of support ability or change*3 of soil)	with Assigned Damage (decrease of support ability or change*3 of soil)	Non-Harmful*2 Change*3 for normal use of buildings or traffic

<Supplement>
(*1) destruction : unbalanced state of energy, force or deformation in static or dynamic response
(*2) harmful : available without interferance in the usual usage
(*3) change : landslide, movement, deformation, decrease of stiffness (ex. by liquilifaction), gap, crack of soil

Table 3-1 Basic Structural Performance : Safety

Principal of Performance Evaluation : Failure that affects human life is evaluated in the term of force, deflection and energy.

Action Performance item	Permanent (P) D. L. etc	Snow (S)	Wind (W)	Earthquake (E)	Others temp. etc
1) Failure of structural frame	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\underline{\delta r < \delta c}$ $Er < Ec$ $Ir < Ic$	$\underline{Qr < Qc}$ $\underline{\delta r < \delta c}$ $Er < Ec$ $Ir < Ic$	$Qr < Qc$ $\delta r < \delta c$
2) Failure of structural members (column, beam, floor, wall, roof, foundation, pile etc)	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$	$Qr < Qc$ $\delta r < \delta c$
3) Failure or drop off of non-structural member	$\underline{Qr < Qc}$	$Qr < Qc$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{Qr < Qc}$ $\underline{oc\ \delta r < \delta c}$
4) Drop off, overturn of equipment	$\underline{Qr < Qc}$	--	$\underline{Ar < Ac}$ $\underline{Vr < Vc}$	$\underline{Ar < Ac}$ $\underline{Vr < Vc}$	-- --
5) Drop off, overturn of contents	$\underline{Qr < Qc}$	--	$\underline{Ar < Ac}$ (fix) $\underline{Vr < Vc}$ (loose)	$\underline{Ar < Ac}$ (fix) $\underline{Vr < Vc}$ (loose)	-- --
6) Failure of soil	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$	$\underline{Qr < Qc}$ $\delta r < \delta c$

Q : froce, stress, δ : deflection, E : energy, I : index, A : acceleration, V : velocity
suffix c : criteria (limit value), suffix r : response
_____ (underline) : evaluation method is already given.

Table 3-2 Basic Structural Performance : Damage Control

Principal of Performance Evaluation : Damage control limit determined by the economic and technical point of view is evaluated in the term of force, deflection and energy.

Action Performance item	Permanent (P) D. L. etc	Snow (S)	Wind (W)	Earthquake (E)	Others temp. etc
1) Damage of structural frame	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$ $Ir < Ic$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$ $Ir < Ic$	$Qr < Qc$ $\delta r < \delta c$
2) Damage of structural members (column, beam, floor, wall, roof, foundation, pile etc)	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$ $Ir < Ic$	$Qr < Qc$ $\delta r < \delta c$ $Er < Ec$ $Ir < Ic$	$Qr < Qc$ $\delta r < \delta c$
3) Damage of non-structural member	$\underline{Qr < Qc}$	$\underline{Qr < Qc}$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{Qr < Qc}$ or $\underline{\delta r < \delta c}$
4) Damage of equipment	$\underline{Qr < Qc}$	--	$Ar < Ac$ $Vr < Vc$	$\underline{Ar < Ac}$ $\underline{Vr < Vc}$	-- --
5) Damage of contents	$\underline{Qr < Qc}$	--	$\underline{Ar < Ac}$ (fix) $\underline{Vr < Vc}$ (loose)	$\underline{Ar < Ac}$ (fix) $\underline{Vr < Vc}$ (loose)	-- --
6) Damage of soil	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$

Q : froce, stress, δ : deflection, E : energy, I : index, A : acceleration, V : velocity
suffix c : criteria (limit value), suffix r : response

Table 3-3 Basic Structural Performance : Serviceability

Principal of Performance Evaluation : Deflection or vibration that affects normal usage is evaluated in the term of force, deflection, vibration and velocity.

Action Performance item	Permanent (P) D. L. etc	Snow (S)	Wind (W)	Earthquake (E)	Others temp. etc
1) Deflection or vibration of structural frame	$\underline{\delta r < \delta c}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{fr < fc}$
2) Deflection or vibration of structural members (column, beam, floor, wall, roof, foundation, pile etc)	$\underline{\delta r < \delta c}$ $\underline{Qr < Qc}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{Qr < Qc}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{Qr < Qc}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{Qr < Qc}$ $\underline{fr < fc}$	$\underline{\delta r < \delta c}$ $\underline{Qr < Qc}$ $\underline{fr < fc}$
3) Deflection or vibration of non-structural member	$\underline{Qr < Qc}$	$\underline{Qr < Qc}$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{\delta r < \delta c}$ $\underline{Ar < Ac}$	$\underline{Qr < Qc}$ oc $\underline{\delta r < \delta c}$
4) Deflection or vibration of equipment	$\underline{Qr < Qc}$	--	$\underline{Ar < Ac}$ $\underline{Vr < Vc}$	$\underline{Ar < Ac}$ $\underline{Vr < Vc}$	-- --
5) Deflection or vibration of soil	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$	$Qr < Qc$ $\delta r < \delta c$

Q : froce, stress, δ : deflection, f : vibration, A : acceleration, V : velocity
suffix c : criteria (limit value), suffix r : response

Table 4 Maximum Amplitudes of Major Earthquake Records

Earthquake	Date	Obs. Site	Install	Comp.	Acc. (cm/s/s)	Vel. (cm/s)	Disp. (cm)	Inc.V (cm/s)
Interplate Eq.								
1968 Tokachioki	1968.5.16	Hachinohe	Grnd	NS	230	34	8.3	
				EW	180	38	12.4	
1978 Miyagiken-oki	1978.6.12	Tohoku-univ	1F	NS	255	36	14.5	65.4
				EW	203	28	9.1	39.8
				UD	153	12	3.2	16.1
1994 Sanriku-Haruka	1994.12.28	Hac. CtyHall	B1	N164E	415	44	6.7	76.1
				UD	118	8	2.2	14.2
				N254E	319	29	3.6	56.6
1985 Chile	1985.3.3	Llolleo	B	N100E	437	23	4.2	39.3
		Near		UD	849	20	6.8	34.9
				N010E	654	40	10.5	52.6
1985 Mexico	1985.9.19	Zacatula	Grnd	NS	271	30	18.2	
		Near		UD	145	11	9.6	
				EW	182	14	8.6	
		SCT1	Grnd	NS	98	39	19.1	
		Mexico City		UD	37	9	7.6	
				EW	168	61	21.9	
Intraplate (ocean)								
1993 Kushiro-oki	1993.1.15	Kushiro JMA	Grnd	N063E	711	33	4.8	
				UD	363	14	1.2	
				N153E	637	41	8.0	
1994 Hokkaido-Toho	1994.10.4	Kushiro JMA	Grnd	N063E	314	27	4.9	
				UD	190	10	4.2	
				N153E	392	21	4.3	
Intraplate (inland)								
1995 Hyogoken-Nanbu	1995.1.17	Kobe JMA	Grnd	NS	818	90	20.2	154.8
				UD	333	40	10.2	56.8
		Fukiai	Grnd	N030W	802	121	43.9	215.8
		Takatori	Grnd	NS	661	138	42.0	243
		Kobe univ	Grnd	NS	270	55		
		NTT	B3	N309E	342	84	27.5	142.1
				UD	169	20	7.5	38.84
1940 Imperial Valley	1940.5.18	ElCentro	1F	NS	342	33	10.9	52
				UD	206	11	5.6	10
1952 Kern County	1952.7.21	Taft	Tunnel	EW	176	18	9.2	27.14
				UD	103	7	5.0	8.96
1971 San Fernando	1971.2.9	Pacoima Dam	Grnd	S016E	1148	113	37.7	138.9
				UD	696	58	19.3	63.2
	1979.10.15	Array 6	Grnd	N230E	408	112	69.1	133.6
				UD	1655	57	35.8	88.9
1992 Landers	1992.6.28	Lucerne	Grnd	S080W	740	146	261*	
1994 Northridge	1994.1.17	Rinaldi	Grnd	Hor.	826	170	33.3	240.6
				UD	830	49	9.2	56.2
		Sylmar	Grnd	Hor.	827	129	32.5	147.8
				UD	525	19	7.6	26.3
		Newhall	Grnd	Hor.	578	95	30.5	152.6
				UD	537	31	12.8	52.95
		Tarzana	Grnd	Hor.	1888	110	29.0	166.01
				UD	1130	72	17.0	80.3

* Iwan (1994)

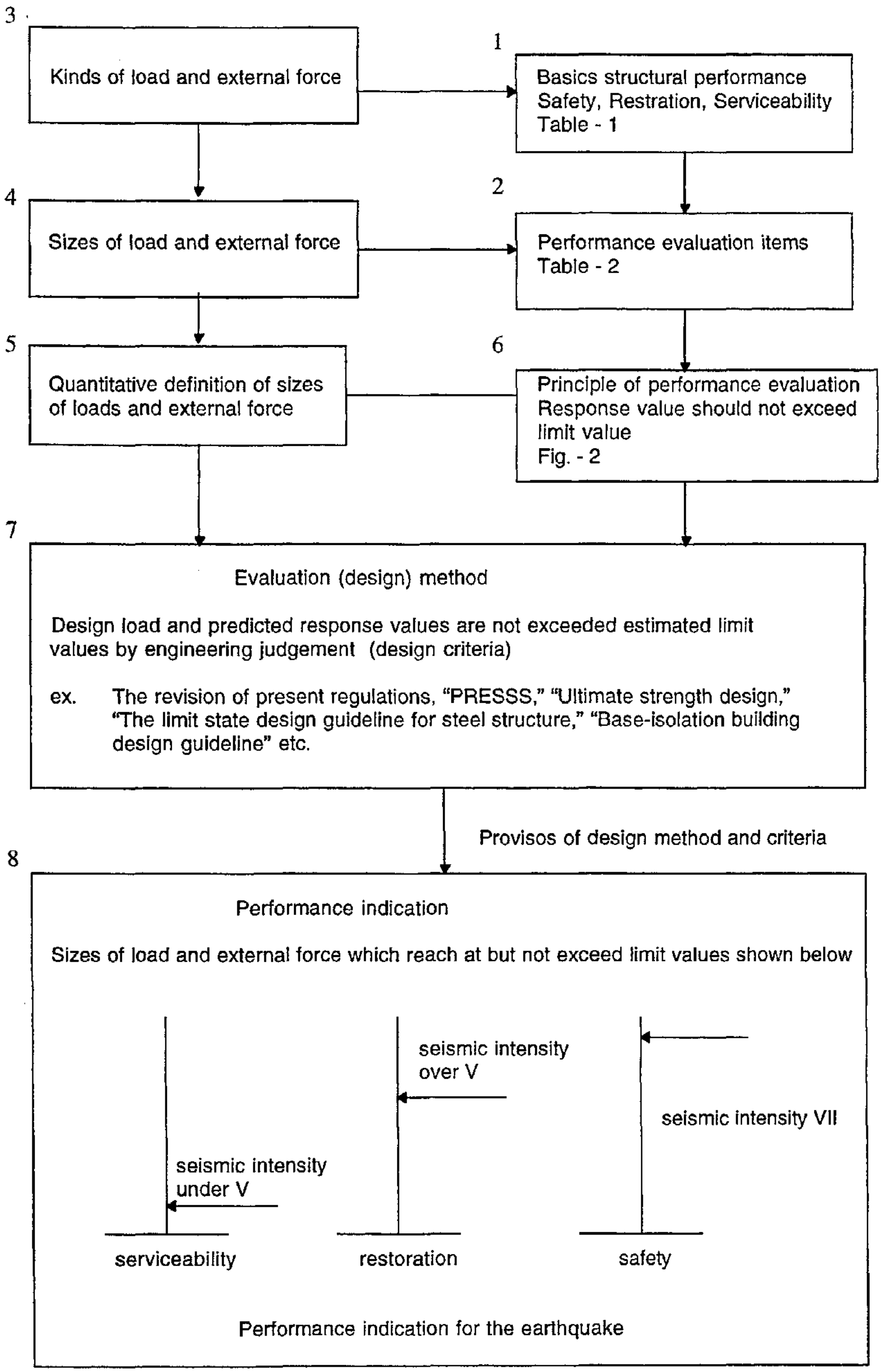

Fig. 1 EVALUATION SYSTEM OF STRUCTURE PERFORMANCE

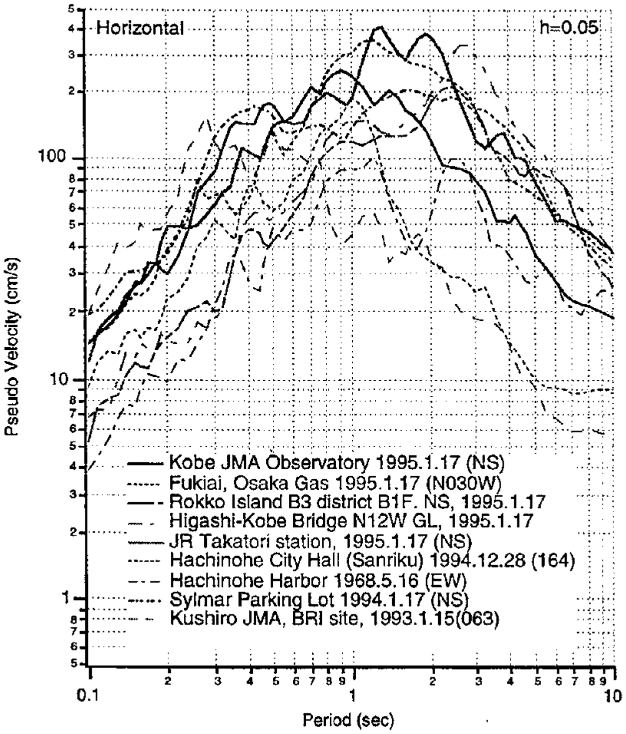

Fig. 2 Compariosn of pseudo spectra from large earthquakes
Data Source : JMA, Osaka Gas, Sekisui, CDMG , PWRI, JR, BRI, PHRI

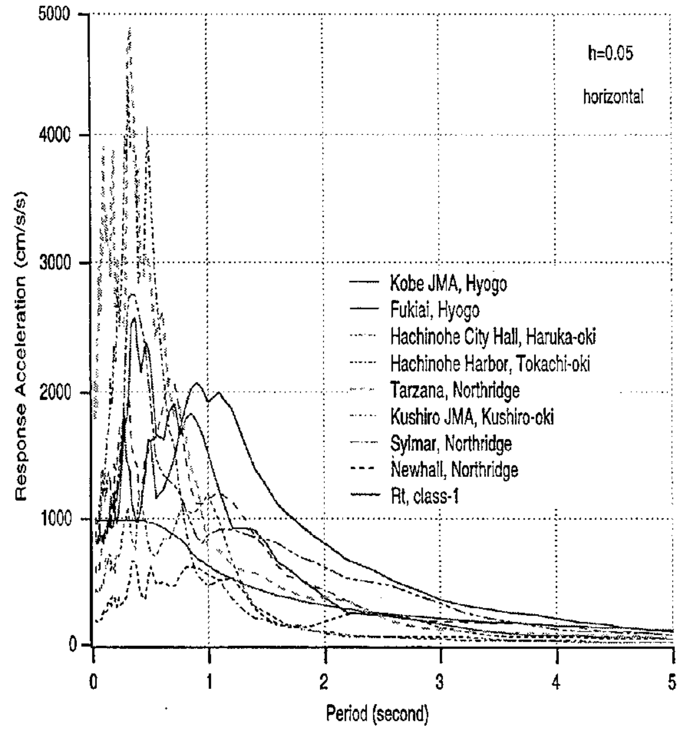

Fig. 3 Comparison of Acceleration spectra for large earthquakes
Data Source : JMA, Osaka Gas, CDMG, PHRI, BRI

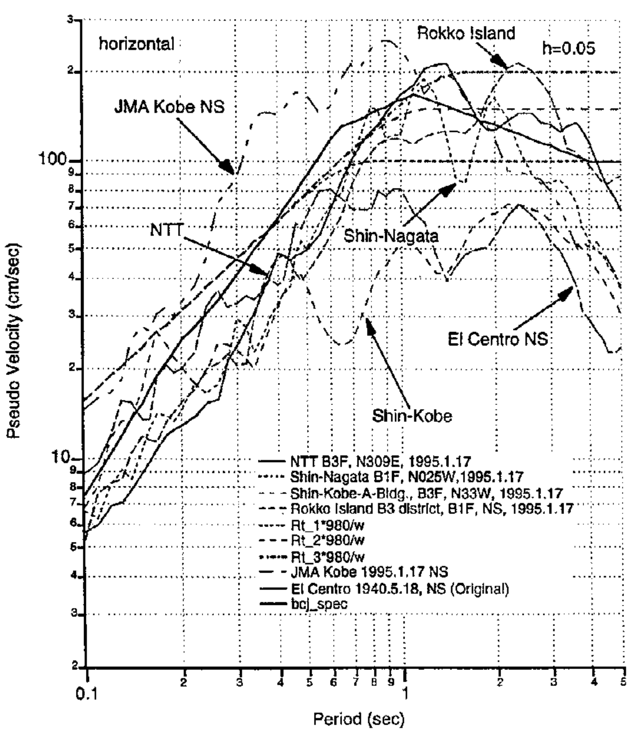

Fig.4 Comparison of pseudo spectra for building responses
Data Source : Sekisui, NTT, HUD, Takenaka, USGS, JMA

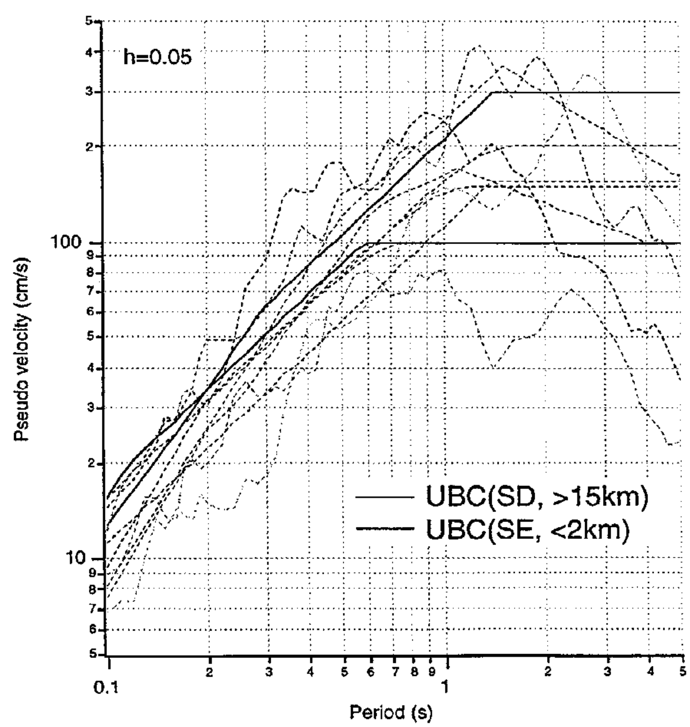

Fig. 5 Comparison of design spectra for UBC, Roadway Bridge, BCJ, BSL

Part III: Appendices

Appendix A
Workshop Program
Seminar Program

August 28	7:30-8:30am	Executive Session (EXCOM Members)	
	8:00-8:30am	Registration and Continental Breakfast	
	8:30-9:30am	Opening Session	
	Coffee Break		
	10:30-12:00 noon	Technical Session 1	Science of Aging & Deterioration
	Lunch		
	1:30-3:00pm	Technical Session 2	Health Monitoring & Condition Assessment
	Coffee Break		
	3:30-5:00pm	Technical Session 3	Civil Infrastructure Renewal
	5:30-6:30pm	Executive Session	
	Evening	Free	
August 29	8:00-8:30	Continental Breakfast	
	8:30-10:00am	Technical Session 4	Socioeconomic and Productivity
	Coffee Break		
	10:30-12:00noon	Technical Session 5	Research Coordination
	Lunch		
	1:30-3:00pm	Working Group Session I	
	3:30-5:00pm	Working Group Session II	
	5:30-6:30pm	Plenary Session	Working Group Reports
	8:00pm	Dinner Party	
August 30	8:00-8:30am	Continental Breakfast	
	8:30-9:00am	Executive Session	
	9:00-10:00am	Plenary Session	Development of Resolution
	Coffee Break		
	10:30-11:30am	Plenary Session	Adoption of Resolution
	11:30-12:00pm	Closing Session	
	12:00noon	Adjournment	

Notes:

1. All technical sessions are plenary.
2. Working group sessions are break-up parallel sessions for five groups of researchers respectively on the five themes to identify and recommend joint research projects.
3. Each technical session consists of three themes.
4. Executive sessions deal with administrative needs, and are attended by seminar and working group chairs.

Technical Sessions

TECHNICAL SESSION 1: SCIENCE OF AGING AND DETERIORATION

Papers (1) and (2) Damage/Deterioration processes **(Area 1)**
US delegate: S. P. Shah
Japanese delegate: T. Hasegawa

Papers (3) and (4) Mathematical models and simulation of material deterioration **(Area 2)**
US delegate: V. Karbhari (Presented by M. Shinozuka)
Japanese delegate: T. Kishi

Papers (5) and (6) Crack, failure, fatigue, corrosion **(Area 3)**
US delegate: K. H. Frank
Japanese delegate: K. Takanashi

TECHNICAL SESSION 2: HEALTH MONITORING & CONDITION ASSESSMENT

Papers (7) and (8) Advanced sensors and implementation **(Area 4)**
US delegate: E. Landis
Japanese delegate: A. Mita

Papers (9) and (10) Advanced non-destructive evaluation techniques **(Area 5)**
US delegate: G. Washer
Japanese delegate: Y. Adachi

Papers (11) Integrated real-time disaster information systems **(Area 6)**
US delegate: R. Eguchi

TECHNICAL SESSION 3: RENEWAL ENGINEERING

Papers (12) and (13) High-performance and smart construction materials **(Area 7)**
US delegate: D. Jansen
Japanese delegate: Y. Yamamoto

Papers (14) and (15) Innovative design, construction and rehabilitation methods **(Area 8)**
US delegate: J. Roberts
Japanese delegate: K. Yokoyama

Papers (16) Maintenance and life cycle cost **(Area 9)**
US delegate: A. Rose

TECHNICAL SESSION 4: SOCIOECONOMIC ISSUES AND INSTITUTIONAL EFFECTIVENESS AND PRODUCTIVITY

Papers (17) Infrastructure productivity **(Area 10)**
US delegate: I. Nadiri (Participation by paper contribution)

Papers (18) and (19) Sustainable development **(Area 11)**
US delegate: K. C. Topping
Japanese delegate: R. Shimada

Papers (20) and (21) Policy issues **(Area 12)**
US delegate: W. Petak
Japanese delegate: T. Hasegawa

TECHNICAL SESSION 5: RESEARCH COORDINATION

Papers (22) and (23)	CIS Research: Academic perspective **(Area 13)** US delegate: G. Lee Japanese delegate: M. Watabe
Papers (24) and (25)	CIS Research: Industry perspective **(Area 14)** US delegate: P. Croce Japanese delegate: M. Todo
Papers (26) and (27)	CIS Research: Government perspective **(Area 15)** US delegate: R. N. Wright Japanese delegate: I. Okawa

Appendix B
Workshop Participants and Observers
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Appendix C
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Executive Committee/Seminar Organizing Committee

M. Shinozuka, University of Southern California, Seminar Chair
M. Watabe, Keio University, Seminar Chair
A. Rose, Pennsylvania State University, Seminar Co-Chair
M. Yoshimura, Tokyo Metropolitan University, Seminar Co-Chair

Scientific Committee

Japan	U.S.
M. Yoshimura	Joanne Nigg
M. Watabe	Adam Rose
	S.P. Shah
	M. Shinozuka

Working Group Chairmen

S. P. Shah, K. Takanashi	Chairs of Working Group 1	Science of Aging and Deterioration
R. Eguchi, A. Mita	Chairs of Working Group 2	Health Monitoring and Condition Assessment
J. Roberts, K. Yokoyama	Chairs of Working Group 3	Renewal Engineering
W. Petak, R. Shimada	Chairs of Working Group 4	Socioeconomic Issues, and Institutional Effectiveness and Productivity
R. N. Wright, I. Okawa	Chairs of Working Group 5	Research Coordination

Appendix D
Working Group Resolutions

Working Group 1: Science of Aging and Deterioration

Working Group Resolution

Acknowledging the importance of aging and deterioration of infrastructure systems, we propose the pursuit of life-cycle prediction of performance. To achieve this objective, we propose the following research objectives:

1. Integration of physics, chemistry, and micromechanics of degradation processes
2. Reliability analysis of deterioration
3. Development of models synthesizing material science, structural engineering and system performance (micro- meso- and macro-scale) and integrate them into models predicting life cycle performance of infrastructure systems

High Priority Research Themes

Steel

- Coating
  - New coatings with low VOC
  - Encapsulate lead-based coating
  - Risk assessment of Z_n coating
- 3-D modeling of distortion-induced stresses and fracture
- Predicting future vehicle loading
- Tools for weld inspection
- New steels with improved fracture toughness
- Study of science of welding

Concrete

- Corrosion of reinforcing and prestressing steel
- Cost effective repair materials: FRP, FRC, polymer modified
- Cracking of HPC due to temperature and shrinkage stresses
- Understanding ASR, freeze-thaw, carbonation sulfate attack (mechanisms, accelerated tests, prediction, uncertainty)
- FRP reinforcements
- Integrate micro- and meso-structure (welding, chemo mechanics)

Nature of Cooperation

Share costly experimental facilities
Exchange information
Organize focused workshops
Establish industrial partnerships

Participants

K. H. Frank
T. Hasegawa
T. Kishi
S. P. Shah
K. Takanashi

Working Group 2: Health Monitoring and Condition Assessment

Working Group Resolution

Acknowledging the importance of damage and condition assessment and health monitoring of civil infrastructures, we propose the following research objectives:

1. Clear definition of damage in engineering terms for using health monitoring studies
2. Quantitative method of measuring the degree of damage in a broad set of structures
3. Correlation of physical indexes to more general indexes such as repair cost
4. Prototypes of health monitoring systems to demonstrate feasibility of the method
5. Quantitative method of measuring the degree of damage in a broad set of structures

Background Themes

- Motivation
  - Cut life-cycle cost
  - Assess safety of structures more accurately
  - Allow for damage tolerant design
- Application and Integration
  - Can be used in a post-disaster situation to prioritize repairs
  - Can be used to assess the in-situ conditions of a bridge/building for repair/maintenance, etc.
  - Could be used for the health monitoring of "common" bridges

High Priority Research Themes

- Quantitative NDE techniques
  - Substructure investigation
  - Structural capacity (remaining strength)
  - Fatigue status
  - Internal properties (e.g. size and location of rebars)
  - Material properties/characterization
  - Reliability of NDE technologies
  - Monitoring during manufacturing
  - Indexes to correlate with damages
- System ID
  - Correlate sensing data to damages
  - Real-time ID for during and post-earthquake damages
- Networking
  - Parallel ID configuration
  - Optimal large-scale sensor network
  - Standardized sensor units
  - Standardized software interface
  - Wireless data acquisition system

- Integration
  - GIS
  - Bridge management, facilities management, pavement management, etc.
  - Post-disaster information system

Nature of Cooperation

1. Identify flagship projects (long bridge, super tall building, etc.)
2. Exchange information through home page
3. Standardize numerical simulation platform (e.g., Matlab)
4. Establish partnership with experts in other field (e.g., structural control)
5. Establish a validation center

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R. Eguchi
E. Landis
A. Mita
G. Washer

Working Group 3: Renewal Engineering

Working Group Resolution

Acknowledging the importance of effective and economical CIS renewal, we propose the following research objectives:

1. Improve the current methodologies for vulnerability analysis of the CIS for all natural hazards (wind, earthquakes, fire, floods) to provide planners and political leaders with information to make realistic renewal decisions. Methodologies are available in both the U.S. and Japan for many of these systems but much more cooperative research is needed in this area so scarce funds can be utilized effectively
2. Develop methodologies for evaluation of renewal costs versus potential life-cycle cost reduction
3. Engage in joint research to improve design and constructive methods for CIS renewal

High Priority Research Themes

- Vulnerability analysis (improve algorithms)
- Performance based design for CIS renewal
- Management systems for optimum renewal
- Develop methodology to evaluate costs of renewal versus potential life-cycle cost reduction
- Large-scale testing of:
    - Joint-shear reinforcement (to improve design details)
    - Soil-structure interaction
    - Real-time dynamic shake table tests
- Evaluate advanced materials
    - Composites
        - New construction
        - Retrofitting existing structures
        - Durable alternative to rebar and prestressing strand
        - Issues
    - High performance concretes
        - Issues
        - Seismic performance (and other dynamic situations)
        - Repair of normal concrete with high performance concrete
- Quality control in construction
    - Education/training
    - Test methods
    - Speed of material evaluation
    - Faster inspection and approval
- Design loads and detailing for wind loading
- Flooding-scour critical

- Improved instruments for scour detection
- Improved condition assessment
- Development of remedial measures

- Fire Hazards
  - More fire resistance in renewal efforts
  - Develop advanced fire detection and suppression systems
  - Disaster management (evacuation and access for fire suppression equipment)

Participants

D. Jansen
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Working Group 4: Socioeconomic Issues and Institutional Effectiveness/Productivity

Working Group Resolution 1

Acknowledging the importance of socioeconomic issues and institutional effectiveness/ productivity, we propose the following research objectives:

1. Sustainability is an area of considerable interest and needs to be further defined as it relates to infrastructure design, development, financing, and management.
2. The research agenda should include reference to sustainability issues, perspective, and assessment of methodology as applicable to infrastructure.
3. Additional attention should be provided to processes by which management decisions are reached on major civil infrastructure system development and rehabilitation with regard to sustainability, hazard reduction, and productivity.

Working Group Resolution 2

Acknowledging the importance of socioeconomic issues and institutional effectiveness/ productivity, we propose the following research objectives:

1. The research community needs to develop a CIS architecture that will provide a framework for engineering.
2. Experience in retrofit and reconstruction needs to be documented for all major projects in order to provide the benefit of learning from both successes and failures.
3. There is a need for a high level systems analysis in order to develop the necessary perspective for understanding the socioeconomic issues in CIS.

Background Theme—Sustainability

- Basic definition: not borrowing against the future
- Alternative definition: harmonious pursuit of economic growth and environment quality
- Avoids irreversible effects
  - Depletion of resources
  - Elimination of future options for development or conservation
  - Escalation of future costs to prohibitive levels
  - Catastrophic damage

High Priority Research Themes

General

- Estimate direct and indirect effects, both monetary and nonmonetary, of civil infrastructure
- Integrate expenditures on safety, including hazard mitigation, into infrastructure productivity analysis

- Explore application of strategic planning principles
- Include direct and indirect user cost in life-cycle cost analysis
- Give greater consideration to non-engineering approaches to reducing user costs (e.g., utility, lifeline rationing)
- Link design investment, and O&M to acceptable risks
- Define sustainable development with respect to infrastructure; develop guiding principles and decision criteria
- Identify values and perspectives of "stakeholders," i.e., groups having an interest in the outcome (owners, government, industry, citizens, etc.)
- Evaluate current standards for environmental/infrastructure assessments
- Assess opportunities for including "place-based" analysis
- Need for improvement of cross-cultural and cross-disciplinary communication regarding infrastructure
- Need for improved knowledge and technology transfer process
- Development of understanding of total systems as a context for project and local system level assessment
- Need to understand the effects of shifts in public vs. private roles and responsibilities
- Need to consider all ramifications of infrastructure on sustainability including contributions to and protection against global warming, inclusions on biodiversity, and impacts on oceanic deterioration

Assessing Methodologies For Infrastructure: Analysis/Evaluation

- Apply land-based classification systems
- Improve input-output (I-O) databases
- Explore I-O refinements and extensions, e.g., computable general equilibrium models
- Improve life cycle engineering and costing
- Link infrastructure and land use development
- Explore and identify improved return on investment (ROI) for low probability/high consequence events
- Improve risk management decision methods
- Establish methods for applying CIS and information technology to appropriate levels of analysis
- Develop methods for streamlining assessment — to simplify, reduce cost, and make assessment procedures more efficient/effective
- Identify approaches to communication between CIS professionals and non-specialist managers, elected officials, and members of the public
- Apply advancements in ROI methodologies to alternate long-term financing methods
- Develop improved methods for knowledge and technology transfer to infrastructure professionals (training)

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Working Group 5: Research Coordination

Working Group Resolution 1

Acknowledging the importance of research coordination to achieve the objectives of civil infrastructure systems research, we propose the following objectives:

1. Use partnerships of government, industry, and academia to respond in a timely way to issues related to CIS performance.
2. Build on successful U.S.-Japan partnerships such as, UJNR Panel on Wind and Seismic Effects, to address the recommendations of this seminar. In many instances, successful U.S.-Japan government agency collaborations should be opened to industry and university participants.
3. Reinvent educational and research programs in universities to respond to capabilities of information technologies, and need for interdisciplinary and international perspectives in research and decision-making in industry and government.

Working Group Resolution 2

Acknowledging the importance of sustainability as an area of considerable interest that needs further definition as it relates to CIS design, development, financing, and management, we propose the following objectives:

1. Improve knowledge about sustainability in CIS development and rehabilitation.
2. Define a policy of actively pursuing new knowledge on the elements of sustainability and their relationships to CIS.

Working Group Resolution 3

Acknowledging the importance of socioeconomic dimensions of CIS development and rehabilitations including communication with the public, we propose the following objectives:

1. Improve knowledge about total systems design including socioeconomic aspects.
2. Define a policy of integrating knowledge of, and showing responsibility for, socioeconomic considerations in CIS design, development, finance, and management.

Working Group Resolution 4

Acknowledging the importance of learning from experience in CIS retrofit and rehabilitation, both successes and failures, also including the impacts of natural disasters on infrastructure systems, we propose the following objectives:

1. Improve knowledge gained from system rehabilitation experience about specific lessons learned including about damage from natural disasters for improved infrastructure recovery and reconstruction.

2. Define a policy of pursuing additional knowledge regarding system rehabilitation including recovery, retrofit, rehabilitation and reconstruction from natural disasters.

Working Group Resolution 5

Acknowledging the importance of informed and participatory management determinations and decisions on design, development, performance finance, maintenance and protection by CIS, management professionals should:

1. Improve knowledge about CIS management decision processes.
2. Define a policy of expanding inquiry into the factors and processes influencing decision-making by various levels of CIS management professionals.

Background Themes

- Reinvent university programs
- Create Institute for Civil Infrastructure Systems Research and institutes at leading universities
- Conduct U.S.-Japan full-scale seismic testing using facilities in Japan
- Identify models for industry/academic consortia
- Determine whether Japanese private R&D for construction can be duplicated in U.S.
- Identify goals meaningful to private and public policy makers to gain support for and implement civil infrastructure systems research
- Create programs that address the removal of barriers to innovation and implementation as well as research
- Build on existing U.S.-Japan collaborations
- Address CISR issue with urgency

Research Coordination

1. Partnerships on major issues
   - Examples: U.S.-Japan Earthquake Disaster Mitigation Partnership Construction and Building Subcommittee of the U.S. National Science and Technology Council
   - Issues: Major issues important to government, industry and academia
   - Participation: Partnership between agencies, industries and universities
   - U.S.-Japan Potential: High for public purposes such as, earthquake disaster mitigation. Low for competitive areas such as, Partnership for New Generation Vehicle

2. Agency-Led Programs
   - Examples: U.S.-Japan Cooperative Structural Testing
Japanese Performance-based Seismic Design
U.S. Engineering Research Centers
   - Issues: National issues fit to an agency mission

- Participation: Lead and participating agencies. Industry and universities as participants/ consultants with cost sharing
- U.S.-Japan Potential: Many good examples where related programs exist:
  US/Japan Wind and Seismic Effects
  US/Japan Fire Research and Safety
  NSF/JSPS Center and Center Cooperation
  NSF Institute for Civil Infrastructure Systems Research

3. Cooperative R&D agreements
   - Examples: Factory Mutual/U.S. Army for development of a new flammability apparatus.
     Enidine/U.S. Navy/SUNY Buffalo to develop semi-active vibration control technology for earthquake and ship vibrations
   - Issues: New products or services with commercial potential
   - Participation: Industry and government and/or university laboratories
   - U.S.-Japan Potential: U.S. and Japanese companies, universities and government laboratories can collaborate (but legal issues involving both governments may be difficult)

4. Issue focused consortia
   - Examples: Center for Chemical Process Safety
     Institute for Building and Home Safety
     Japan Building Contractors Society Criteria for Pile Foundation Design
     SEAOC/JSCA Seismic Performance Criteria
   - Issues: Addressing a broad industry issue
   - Participation: Private sector led with government and academic participation/support
   - U.S.-Japan Participation: Many examples where competitiveness is not an issue. For instance, the BRI/NIST/US timber industry study of post-earthquake fire safety of frame houses.

5. Interdisciplinary educational programs
   - Examples: NCEER education and research
     NSF educational coalition
     Sloan Foundation university centers on industries
   - Issues: Long-term improvements in knowledge and education
     Creation of next generation leaders in industry, government and education
     Support and be supported by other mechanisms
     Initiate coordination and follow through
   - Participation: Universities, industry and government
   - U.S.-Japan Participation: U.S.-Japan seminars and workshops
     NSF/JSPS

Research Coordination Mechanisms

What should the U.S. do?

- Develop partnership to address important issues
  - National Science and Technology Council involving agencies with industry and academia to address this seminar's recommendations
  - NSF Institute for Civil Infrastructure Systems Research to guide response to this seminar and other studies

What should the U.S. and Japan do together?

- Involve industry and universities in effective ongoing collaborations such as:
  - UJNR Wind and Seismic Effects
  - UJNR Fire Research and Safety
  - U.S.-Japan Earthquake Disaster Mitigation Partnership

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Multidisciplinary Center for Earthquake Engineering Research
List of Technical Reports

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through MCEER. These reports are available from both MCEER Publications and the National Technical Information Service (NTIS). Requests for reports should be directed to MCEER Publications, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275, A04, MF-A01).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341, A04, MF-A01).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

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