Structural Engineers Association of California

Any opinions, findings, conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

PROCEEDINGS

1976 CONVENTION

YOSEMITE

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-761761	2.	3. Recipient's Accession No. PB299749

4. Title and Subtitle
Structural Engineers Association of California, 1976 Proceedings, 45th Annual Convention

5. Report Date
1976

6.

7. Author(s)

8. Performing Organization Rept. No.

9. Performing Organization Name and Address
Structural Engineers Association of California
171 Second Street
San Francisco, Cal. 94105

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C)
(G)

12. Sponsoring Organization Name and Address
Engineering and Applied Science (EAS)
National Science Foundation
1800 G Street, N.W.
Washington, D.C. 20550

13. Type of Report & Period Covered
Proceedings
09/30/76/-10/02/76

14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

These proceedings contain papers presented at the 1976 Convention of the Structural Engineers Association of California (SEAOC) in Yosemite. Topics include the role of research in future engineering, the Sears Tower, concrete yesterday and today and ACI 318, improved seismic design, trends and requirements of concrete structures in the next 15 years, and the Guatamala Earthquake of February 1976. Also included are texts of a panel discussion on the present and future of engineering, SEAOC business meeting reports and a three part presentation, "Rehabilitation," which concerns seismic rehabilitation of the state capitol, VA hospitals, and a case study of medium scale seismic upgrading. A considerable number of photographs and charts accompany the reports.

17. Document Analysis a. Descriptors
Structural engineering
Meetings
Technical reports
Concrete construction
Earthquake resistant structures

b. Identifiers/Open-Ended Terms
Sears Tower
Guatamala Earthquake of February 1976
Seismic design

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		141
	20. Security Class (This Page)	22. Price PCA07/A01

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77)
(Formerly NTIS-35)
Department of Commerce

PROCEEDINGS

45th ANNUAL CONVENTION – 1976
SEPTEMBER 30 – OCTOBER 2, YOSEMITE

Structural Engineers Association of California

Officers and Directors

OFFICERS

H. S. Kellam, President
Albert J. Blaylock, President-Elect
James R. Libby, Vice President
Sol E. Cooper, Secretary
Robert J. Toft, Treasurer

DIRECTORS

Central California	Northern California	Southern California	San Diego
Gerald H. Haines	S. E. Cooper	Donald K. Jephcott	Thomas L. Cook
Kenneth A. Luttrell	Richard Gould	John C. Loevenguth	J. R. Libby
Kenneth V. Venolia	H. S. Kellam	Clarkson W. Pinkham	James A. Willis
	R. J. Toft	Donald R. Strand	

COMMITTEE CHAIRMEN

Architect-Engineer
C. W. Pinkham

Board Policy
J. A. Willis

Bylaws
K. A. Luttrell

Code
Philip G. Griffin

Convention Advisory
S. E. Cooper

Finance and Budget
R. J. Toft

Legislative
J. C. Loevenguth

Private Practice
J. A. Willis

Professional Development
G. H. Haines

Public Relations
K. V. Venolia

Research
J. R. Libby

Resolutions
C. W. Pinkham

Seismology
Edwin G. Zacher

Steering
H. S. Kellam

Structural Examination
D. K. Jephcott

Technical Activities
J. A. Willis

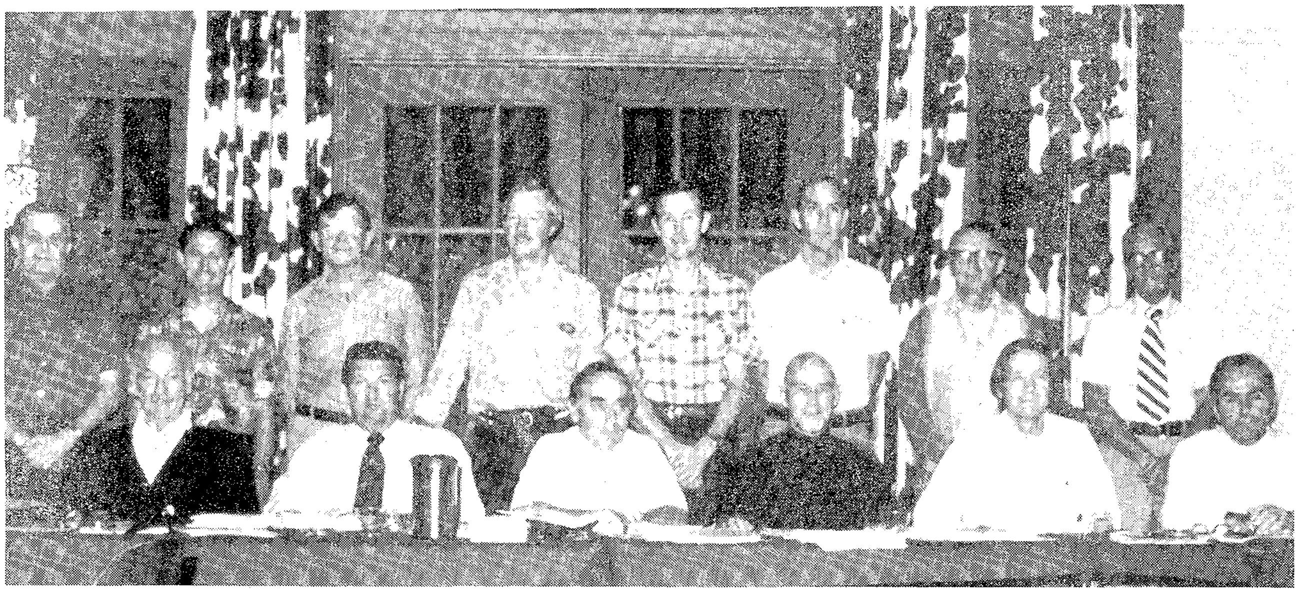
ASSOCIATION PRESIDENTS

Central California	Northern California	Southern California	San Diego
John M. Shaffer	David L. Messinger	Ben L. Schmid	A. J. Blaylock

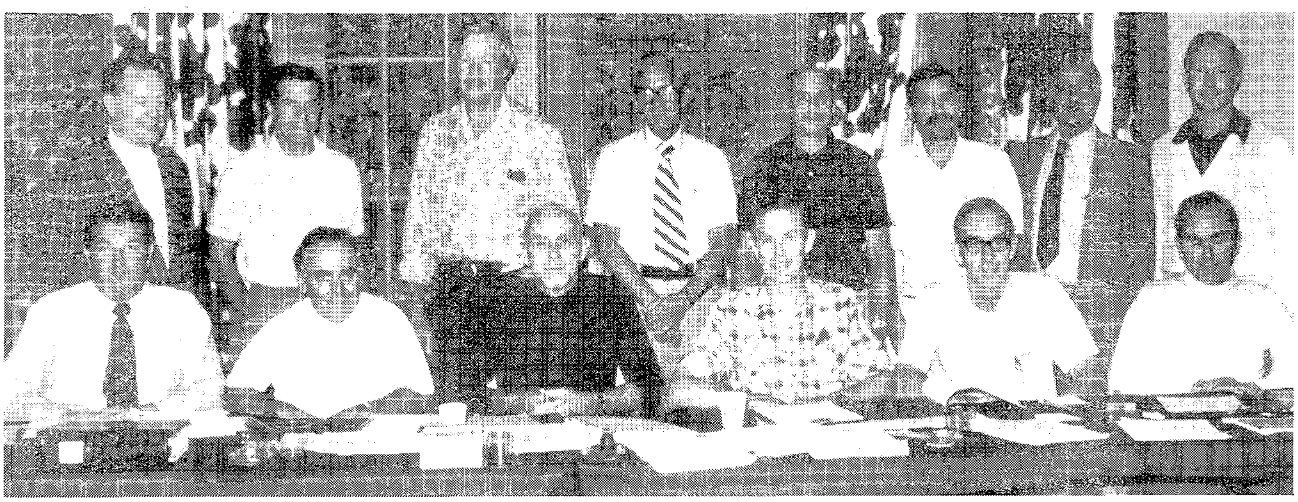
171 SECOND STREET, SAN FRANCISCO 94105 (415) 362-1721

Table of Contents

*Papers not available for publication.



Some of the 1976 Structural Engineers Association of California Board of Directors (standing, from left): Clarkson W. Pinkham; Kenneth A. Luttrell; Donald K. Jephcott; John C. Loevenguth; David L. Messinger; Thomas L. Cook; James R. Libby; Gerald H. Haines. Seated: Don Wiltse, SC Executive Secretary; Richard Gould; President H. S. (Pete) Kellam; President-Elect Albert J. Blaylock; James A. Willis; Robert J. Toft.



Among the Directors and Observers for the 1977 SEAOC Board: Standing— John A. Martin, Robert K. Burkett, John C. Loevenguth, Gerald H. Haines, Stanley E. Teixeira, Richard L. Miller, Ajit S. Virdee, Ben L. Schmid. Seated— Richard Gould, H. S. Kellam, President Albert J. Blaylock, David L. Messinger, Thomas L. Cook, Robert J. Toft.

Convention Committees

CHAIRMAN: HENRY R. LUTGE

Assistant: Robert J. Toft

Technical: Stephen E. Johnston

Linwood Brightbill, Michael Pregnoff, Eric Elsesser, Loring Wyllie, Sigmund Freeman

Operations: H. D. Frankel

Kenneth Brayton, Loren Salladay, Wayne Ferree, E. W. Timmons, Robert Kenefick

Accommodations: Arthur R. Weatherbe

Finance: Ted J. Canon

Registration & Publicity: Bill Giles

Kelly Giles, Margaret Randolph

Sports: Mr and Mrs William Whitcher

Mr and Mrs

Charles De Maria, Robert Toft, Jerome Kluball, Charles Vickers, Chris Poland, Robert Wildman, Ross Rudolph, Mr. E. W. Timmons, Roger Tansley

Social: Paul F. Fratessa

Lawrence Daniels, Loring Wyllie

Thursday Evening

Remember When....We Were in Yosemite in the Past

Mr and Mrs –

Howard Schirmer

Lawrence Daniels, George Nelson, Leo Gosliner, Robert Preece, Thomas Hallyburton, Arthur Weatherbe, Frank Killinger

Friday Evening

Remeber When....There Were Natives in the Valley

Mr and Mrs –

Larry McLean

Richard Hogan, James McLean, Joe Koopman, Mr. Roger Erickson

Saturday Evening

Remember When.... We Last 'Put on the Ritz'

Mr and Mrs –

Phillip Koeber

Fred Cavin, John Paquette, William Enkeboll, Robert Schwein, Oliver Gilbert, Samuel Seifert, Richard Gould, Daniel Shapiro, Philip Griffin, Sanford Tandowsky, George Hervert

Ladies' Activities: Evelyn Daniels
Assistant: Carol Vickers

Madelyne Boardman, Jan Canon, Beverly Cavin, Marge Gosliner, Loretta Gould, Betty Harrington, Margaret Hervert, Joan Hogan, Ursula Holmes, Barbara Kellam, Esther Koopman, Bebe Lawson, Mary Lee, Millie Lutge, Inez McLean, Lena Nelson, Ann Preece, Kathy Ritchie, Lee Robinson, Georgia Scordelis, Carol Singer, Pat Sulser, Lois Teixeira, Donna Toft, Odette Weatherbe, Jan Wosser, Beverly Wyllie

The Super Prize went to SEAOSC's Irv Mendenhall, portraying "Yosemite Valley Floor," a costume created starting at 3 o'clock that afternoon by his wife, Peggy.

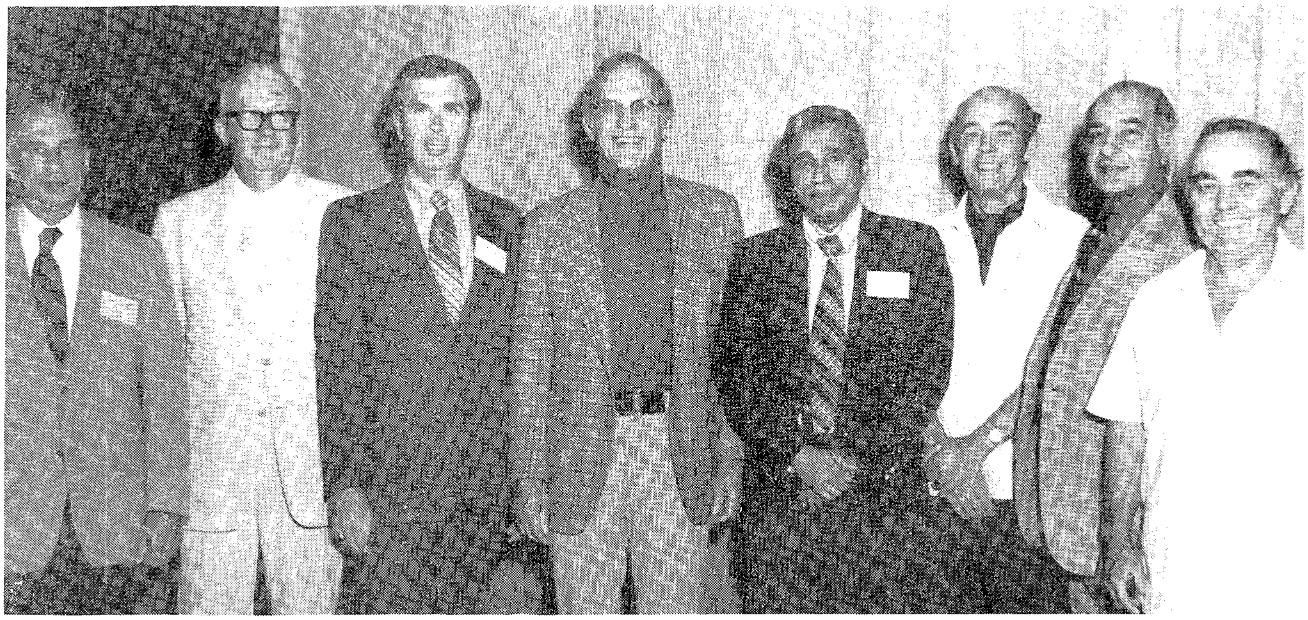
REMEMBER WHEN

........"There Were Natives in the Valley" was the general theme of the Friday Evening Costume Party.

Funniest Getup – Carlos and Mimy Munoz-Flores from SEAOSC.

Six couples from SEAONC won the costume skit competition with a dramatic bit about Teddy and Johnny – Roosevelt and Muir, that is. Barbara and Chris (Johnny) Poland with Ted and Jan Canon are shown at right. The others in the cast were the Carl Basores, the Hal Davises, the Joe Koopmans and the Bob Schweins

The Winning Group – The Big Foot Tribe from Los Angeles: Jean and Al Dambros, Thea and Bob Haussler, Alicia and John Loevenguth, and Jean and Jack Meadville.



Participants in the Thursday afternoon Technical Session (from left): Stanley E. Teixeira; Eivind Hognestad; Gene Corley; Ray W. Clough; Fazlur R. Khan; Stephen E. Johnston, Technical Program Chairman; Ajit S. Virdee, SEAOCC, presiding; and, H. S. (Pete) Kellam, SEAOC President.

THE ROLE OF RESEARCH IN THE FUTURE OF ENGINEERING

by

Ray W. Clough*

Introduction

From the title that was specified for this talk, it might appear that it should be presented from a rather philosophical point of view. However, I am much too pragmatic to prepare that type of paper. In my opinion, the role of research in the future of engineering is to seek answers to engineering problems which will arise in the future, just as the role of research in current engineering practice is to find answers to present problems. The title seems to imply that research may play a greater role in the future than at present; but there is no evident reason to assume that future engineering problems will demand more research effort than those which face us at present.

Because I have no special talent for predicting the future, this talk will be focussed on recent and current research; and it will be restrictied to the field of earthquake engineering with which I have some familiarity. Research has contributed significantly to the rapid advance of the state of the art of earthquake engineering during the recent past. Thirty years ago, structural design practice treated the earthquake as a static horizontal force equal to ten percent of the weight of the structure. The concept of an earthquake response spectrum and the notion that the seismic force should depend on the period of vibration of the structure were rather well kept secrets. It was the pioneering research effort of the Joint Committee of the San Francisco Section, ASCE and the Structural Engineers Association of Northern California, in their paper on "Lateral Forces of Earthquake and Wind"(1) which brought these ideas out of the closet and made them acceptable to respectable design engineers.

The next major research effort in earthquake engineering was in the development of improved methods of analysis for predicting the effects of earthquakes on structures. This effort began in the early 1950's, in conjuction with the introduction of automatic digital computers. These devices amplified the engineer's computational capacity by several orders of magnitude, and it was necessary to devise completely new analytical procedures to take full advantage of this computational capability. During the decade 1955-1965, the field of structural analysis was revolutionized--first by the matrix formulation of structural theory, and then by the generalized and integrated approach which goes under the name of the finite element method. During this same period, it was recognized that static structural behavior is only a special case of dynamic response, and dynamic analysis procedures were added to many computer programs as extensions of methods of static analysis.

By 1965, earthquake engineering research had provided the design profession with the capability of evaluating the dynamic response of essentially any prescribed structural system when subjected to any specified earthquake input. Despite the generality of this statement, however, the earthquake

*Professor of Civil Engineering, University of California, Berkeley.

response problem was far from solved. The analysis techniques could treat any specified mathematical model, but it was difficult to provide an accurate mathematical description of the actual structure in the field; and this difficulty became most obvious in trying to predict nonlinear dynamic earthquake response. The need for nonlinear analyses was recognized when detailed dynamic analyses were made of the response of code-designed structures to actual recorded earthquake motions. Elastic analyses demonstrated that the proportional limit of the structural materials generally would be exceeded during severe earthquakes, so it was necessary to develop analysis procedures which could deal with inelastic material properties. But practically no information was available on the cyclic damage mechanisms of structural components, and consequently it was impossible to formulate realistic mathematical models of the actual earthquake damage mechanisms.

During the past 10 years a major segment of the earthquake engineering research effort at the University of California has been directed toward this problem: the definition of adequate mathematical models to represent earthquake damage mechanisms in typical structural components and systems. With financial support from the National Science Foundation, the Earthquake Engineering Research Center at Berkeley designed and developed extensive experimental research facilities with which to measure the appropriate structural properties during simulated earthquake excitation; and the purpose of this talk is to discuss the research being done with one of those facilities--the earthquake simulator or shaking table.

The EERC earthquake simulator, shown in Fig. 1[2] can produce the vertical component and one horizontal component of any desired earthquake motion, with maximum accelerations in these directions of about 1/2g and 3/4g respectively. Thus the earthquake response behavior of any structure mounted on the shaking table can be observed directly, and the accuracy of any proposed analysis procedure can be tested by correlating the analytically predicted structural response with the observed performance.

In this talk, the interaction between research workers and the design profession, which characterizes experimental earthquake engineering research, will be emphasized. In general, the basic problem which

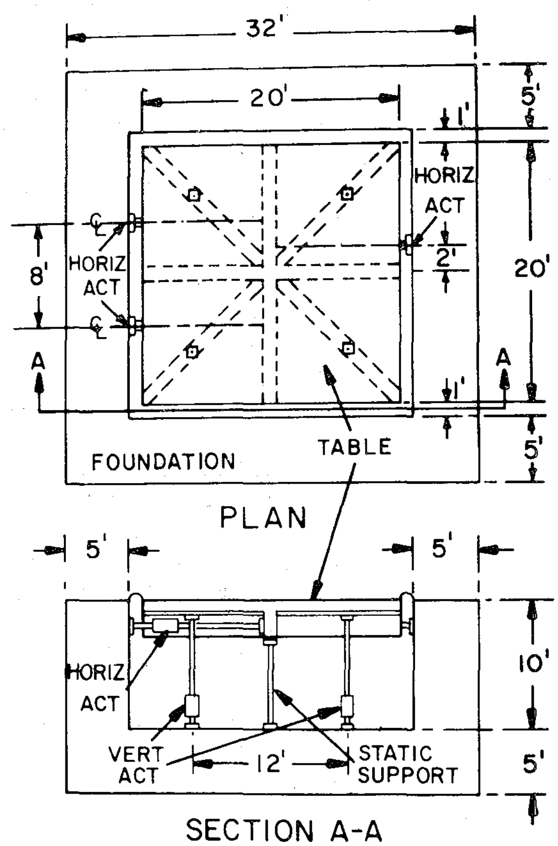


Fig. 1 EERC Shaking Table

identifies the need for research comes from the profession. The research team then plans and carries out suitable experiments to provide the desired information; and ideally the research results are put to their required use by the design engineers as the final step in the sequence. In this connection, it is interesting to note that initial impetus which led to the design and construction of the EERC shaking table came from the profession. Jack Meehan of the Office of the State Architect asked members of the Berkeley faculty to study the feasibility of building a really large shaking table--100 x 100 ft in plan. Our research indicated that such a facility would indeed be feasible, but because of the magnitude of the project it seemed desirable to check out the design concept in a smaller pilot model. The present 20 x 20 ft. EERC shaking table was originally built as the pilot model--but we do not now have ambitions toward developing the "super table" because of the enormous costs and management problems that it would entail.

In this talk, two shaking table projects will be described which demonstrate how some of the EERC research is motivated directly by the profession. These are: (1) an investigation of building uplift due to seismic overturning moments, and (2) a study of the earthquake response of cylindrical liquid storage tanks.

Building Frame Uplift

The suggestion for this investigation came directly from one of the SEAOC members. The basic problem arose as a consequence of the new hospital design code[3], which increased the horizontal earthquake forces used in design. For many moderate to high-rise structures, these increased static lateral loads induce such large overturning moments that some of the outer columns tend to lift off their foundations. The code requires that the columns be anchored to the foundation rock so as to constrain any tendency to uplift, frequently at a significant cost. However, it is evident that the actual dynamic earthquake motions induce only momentary tendencies toward uplift--there is no real possibility of the building actually overturning. So what is the value of the expensive anchoring system which is provided to resist uplift?

This problem is one which lends itself readily to a shaking table study, and an existing three-story steel frame was available (Fig. 2) which would be suitable for a preliminary investigation[4]. The diagonal braces shown attached at mid-height of the first story columns were added to the frame for this test in order to compensate for the increased flexibility introduced by the introduction of hinges at the column bases. These hinges are part of the column base connections, shown in Fig. 3, which were designed to permit uplift of the columns, but to prevent horizontal motions. Roller bearings control the vertical motions of the column bases, a rubber pad cushions the impact as the column comes down after uplifting. Instrumentation attached to the first story columns served to measure the column axial forces and moments during the simulated earthquake test; other gages measured the accelerations and displacements of the three floors of the frame.

Uplift tests of the frame were conducted by the standard shaking table procedure. The selected earthquake motion was applied to the structure with successively increasing intensity, and the response behavior was observed as a function of the intensity. The El Centro earthquake (1940 N-S), was used as one of the input earthquakes in this study. The displacement response of the uplifting frame to this earthquake is shown in Fig. 4. The upper graph shows the displacement of the top of the frame relative to the base--about 6 inches maximum. The other two traces show the amount of vertical footing motion --uplift or compression of the rubber base pad. The experimental results are depicted in these graphs by the dashed lines; the solid lines indicate the corresponding response results computed analytically. The good agreement between analysis and experiment demonstrates the validity of the nonlinear mathematical models used in these calculations. It is evident in this figure that the rubber cushions used in these tests were quite soft; however, pads which were an order of magnitude stiffer were used in another test series, and produced essentially the same type of behavior

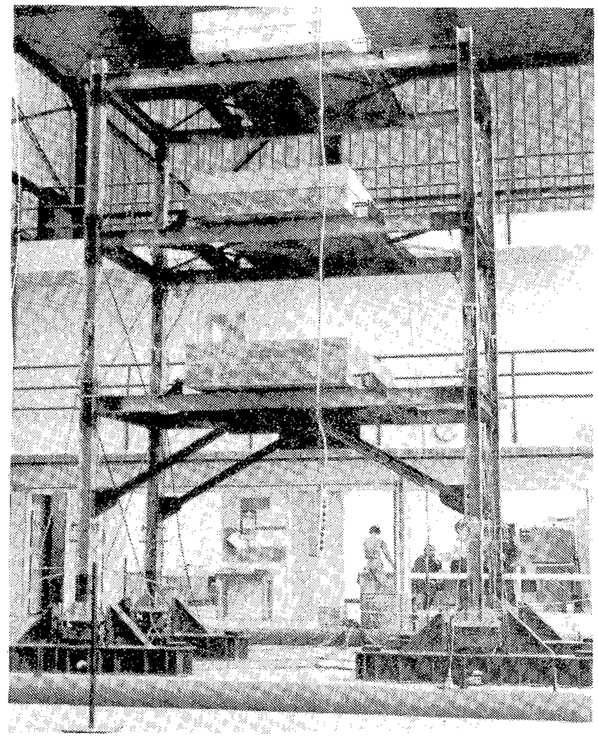


Fig. 2 Steel Frame Modified to Uplift

In order to obtain a direct measure of the influence of the uplifting mechanism on the dynamic response behavior, an additional sequence of tests was made using the same input but with the frame rigidly constrained against uplift. Fig. 5 presents a comparison of the top story displacement response with and without the uplift mechanism; it is not surprising to note that the displacement is greatly increased by the uplift. Fig. 6 presents a similar comparison of the first story column shear forces; where it is apparent that permitting uplift significantly reduces the seismic forces in the frame. The

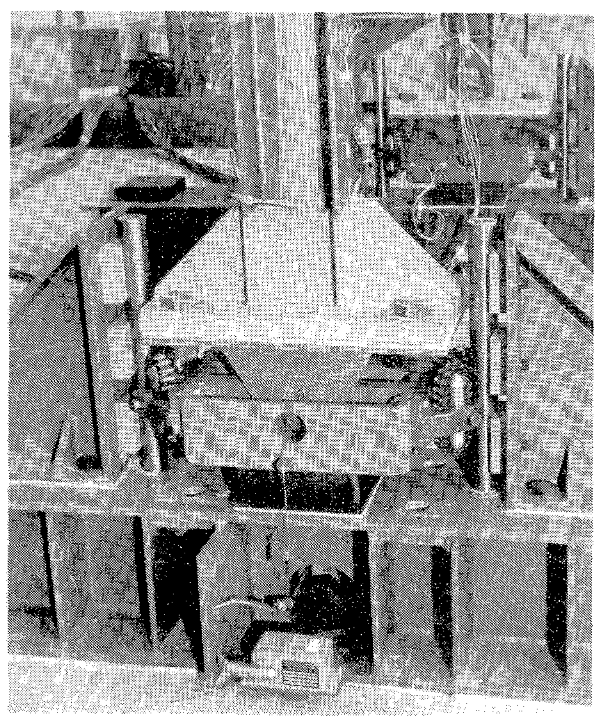
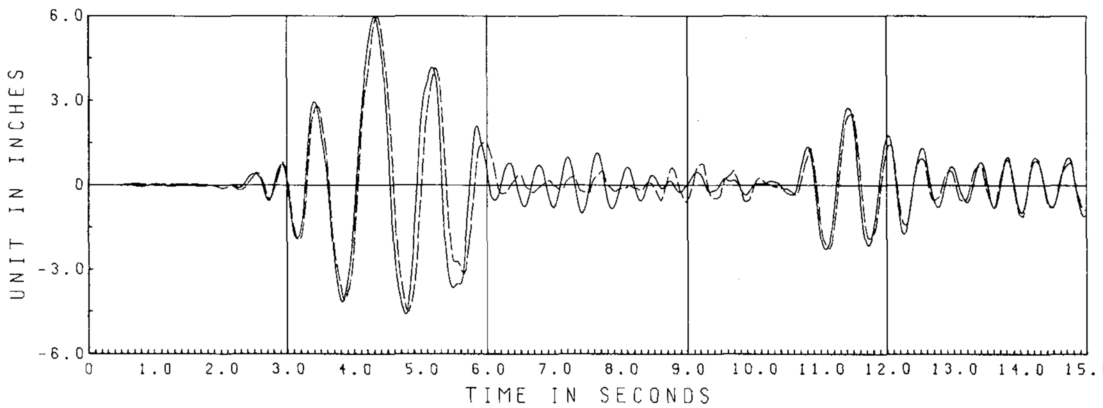
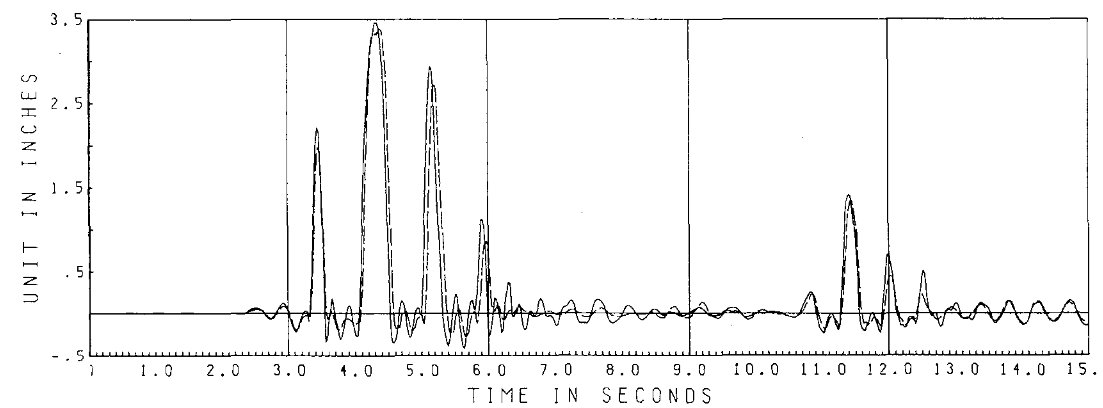


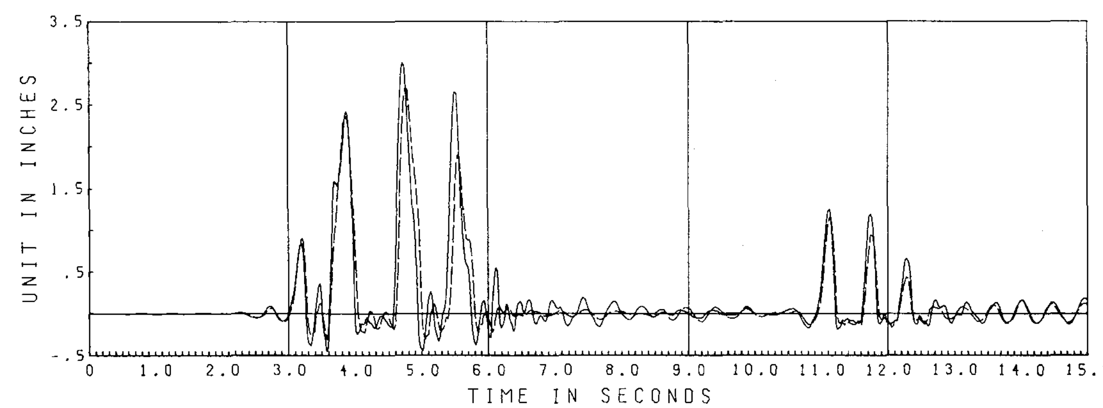
Fig. 3 Uplifting Column Base Device



a. Horizontal Displacement, Third Floor

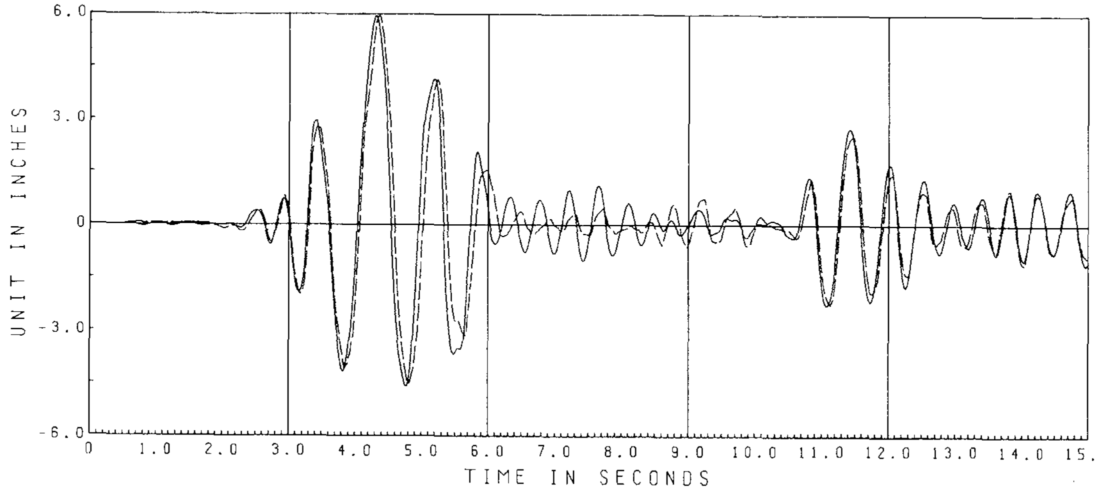


b. Vertical Displacement, North Footing

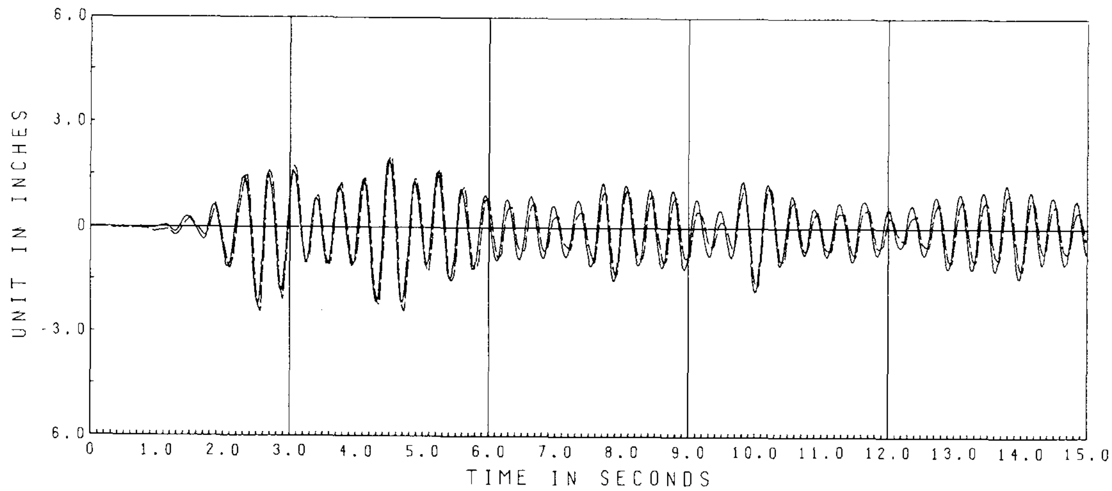


c. Vertical Displacement, South Footing

Fig. 4 Frame Displacement Response to Earthquake



a. Uplift Permitted

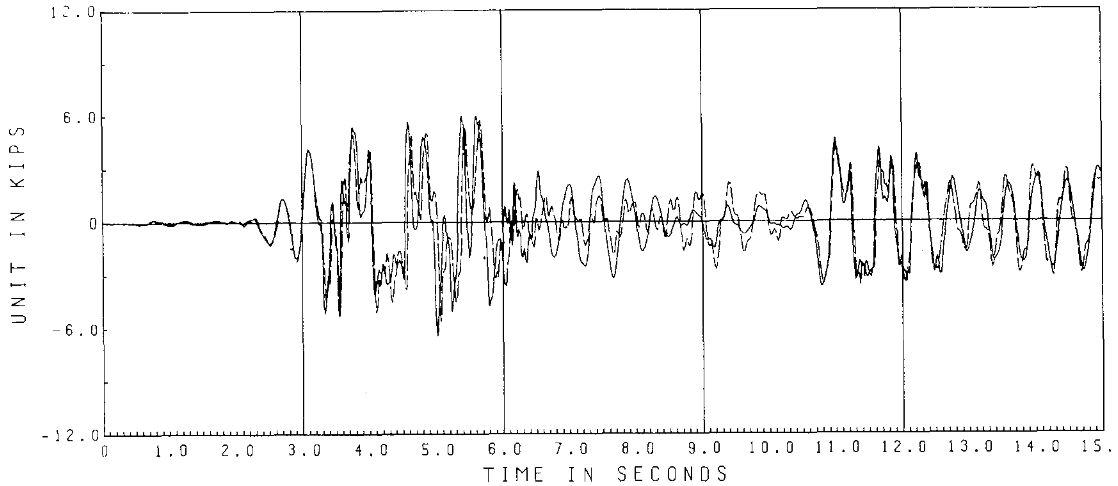


b. Without Uplift

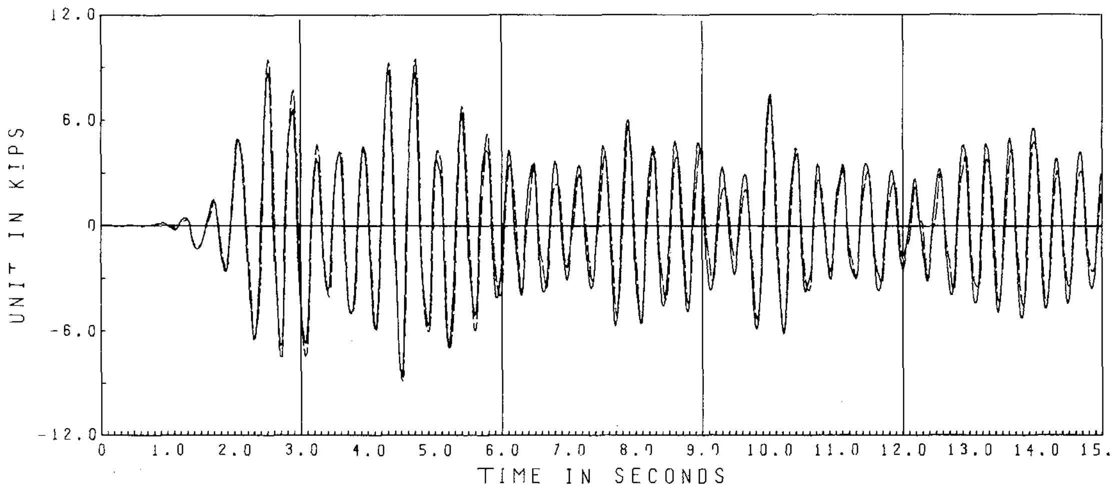
Fig. 5 Effect of Uplift on Displacement Response

column axial forces shown in Fig. 7 demonstrate a similar reduction in the average forces, but no important change in the peak compressive force.

Because of the favorable performance demonstrated by this simple uplifting system, a more extensive investigation of the general building frame uplift problem has been planned; and now funding has been received from the American Iron and Steel Institute to fabricate an appropriate test structure. This will be a nine story, three bay steel frame, constructed at about 1/3 scale. This new model will demonstrate the type of uplift which takes place in multibay frames, in which uplift of a single column must be accompanied by frame deformation; this is in contrast with the rigid-body rotation mechanism which occurs in the single bay frame tests. This new model also will have greatly simplified uplift attachments at the column bases--devices which can be adapted to installation in real buildings. It is expected that this



a. Uplift Permitted



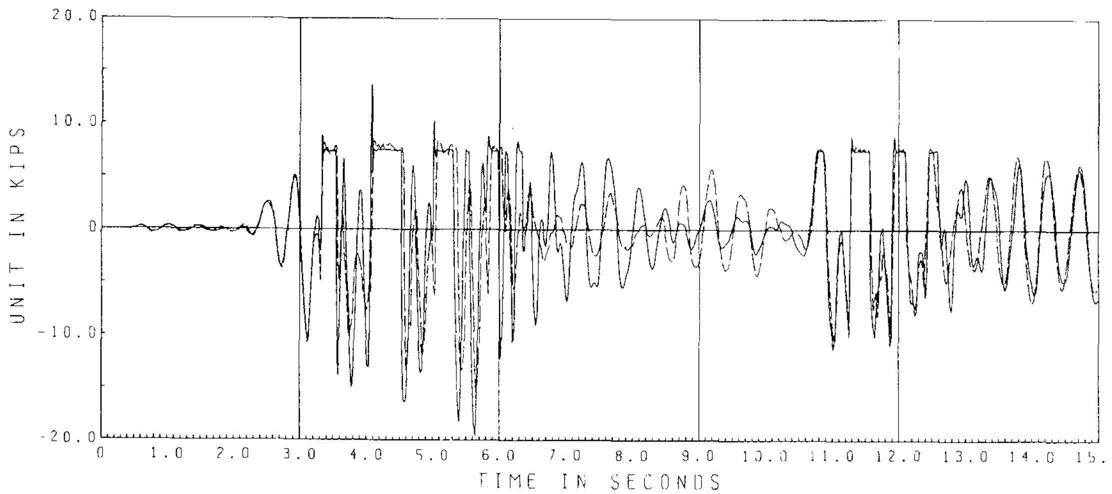
b. Without Uplift

Fig. 6 Effect of Uplift on Column Shear Response

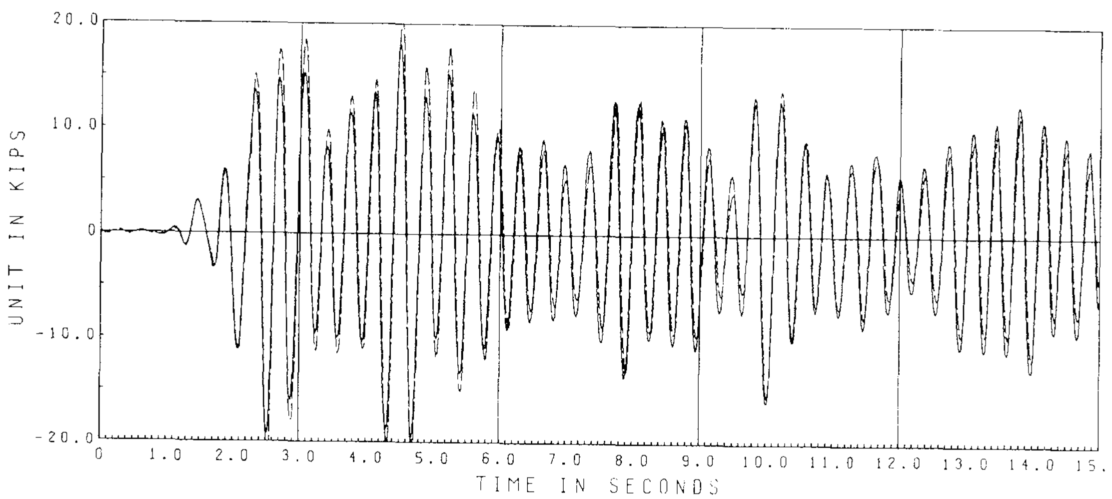
investigation will demonstrate conclusively that uplift constraint is unnecessary from a structural design point of view, and that earthquake stresses actually are reduced if uplift is permitted.

Cylindrical Liquid Storage Tanks

The second test program to be described also was suggested directly by the design profession. The subject of investigation is the earthquake behavior of cylindrical liquid storage tanks, such as are used in great numbers of petroleum refineries. Many such tanks have been damaged in many earthquakes aroung the world[5]. Fig. 8 shows typical "elephant foot" buckling damage at the base of a tank caused by the Managua earthquake.



a. Uplift Permitted



b. Without Uplift

Fig. 7 Effect of Uplift on Column Axial Force

The seismic design of cylindrical liquid storage tanks is based on Housner's approximate analysis of the hydrodynamic pressures developed in a rigid tank under earthquake motions[6]. However, these thin steel shells obviously are not rigid and the complex dynamic fluid-structure interaction mechanism is not well understood at present. So a group of petroleum producing companies, tank manufacturers, and engineering design firms collaborated under the leadership of Chevron Oil Field Research Company to sponsor a three year shaking table study of the problem. Four tank models are included in the research program, but results of only one of these will be mentioned here. This is a 6 ft high by 12 ft diameter cylinder (Fig. 9), fabricated from 0.05 in and 0.08 in sheet aluminum to simulate a steel tank at three times larger scale. Basic testing parameters included the type and intensity of earthquake excitation, the top boundary condition (open or with two different types of roofs) and the depth of water in the tank; but the most important test parameter was the base support condition--either fully clamped or free to uplift.

Response measurements included the shaking table accelerations and displacements, radial and tangential displacements of the tank relative to the table, vertical surface wave motions, hydrodynamic pressures, and internal stresses of the tank shell. The external reference frame for displacement measurements and the internal wave height gage frame are visible in Fig. 9. The dynamic response behavior of this tank is extremely complex, and only the case with open top and base free to uplift will be discussed here. These results are for a water depth of 5 ft. and for the El Centro (1940 N-S) earthquake input. Fig. 10 indicates many significant features of the response to the applied table acceleration history which is shown in graph 10a. The hydrodynamic pressure record in Fig. 10b indicates a direct correlation with the input accelerations; but in the later stages of the response a two second period oscillation clearly is superimposed on the high frequency acceleration effect. This record demonstrates that the hydrodynamic pressures in the tank result from two essentially independent mechanisms: impulsive pressures which correlate directly with the input accelerations, and convective pressures which are produced by wave sloshing effects. The wave height record of Fig. 10c demonstrates the correlation with this component of the hydrodynamic pressures of Fig. 10b. It is of interest to note that Housner's approximate analysis procedure takes account of these two independent pressure mechanisms.

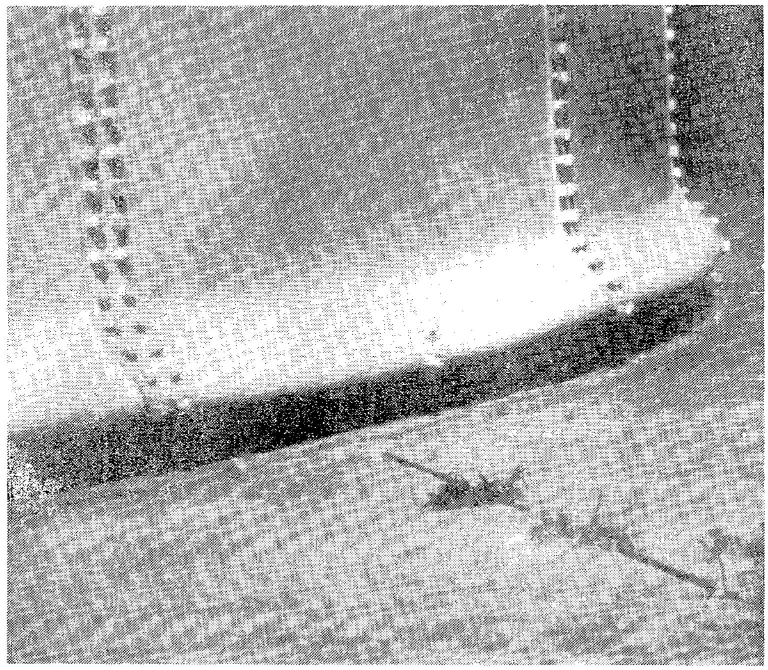


Fig. 8 Tank Buckled during Managua Earthquake

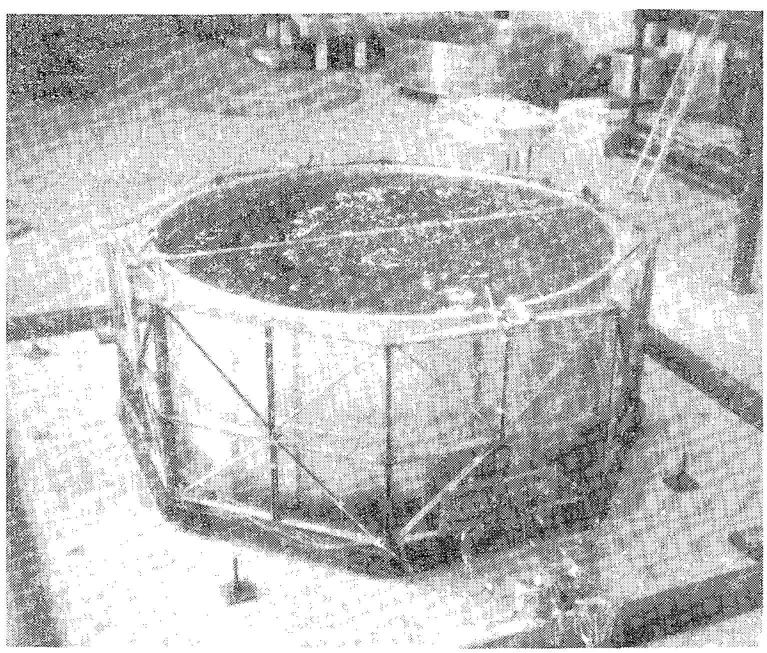
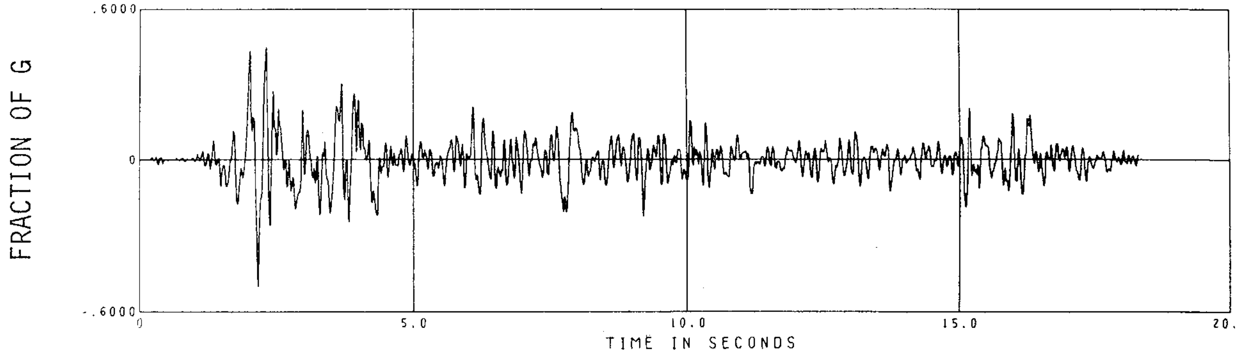
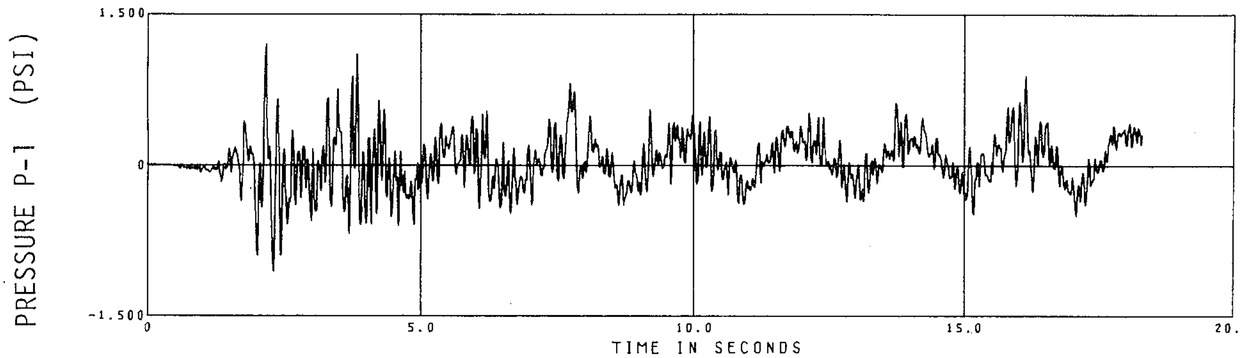


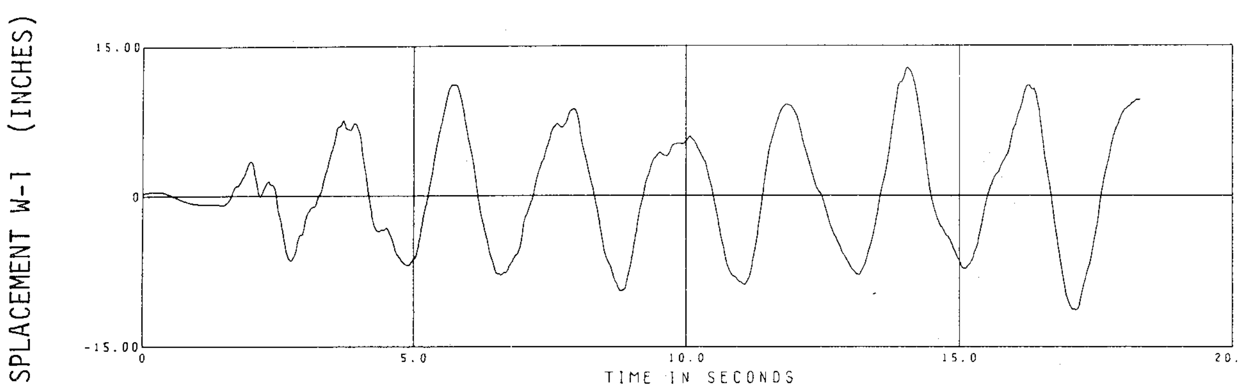
Fig. 9 6 x 12 ft. Tank on Shaking Table



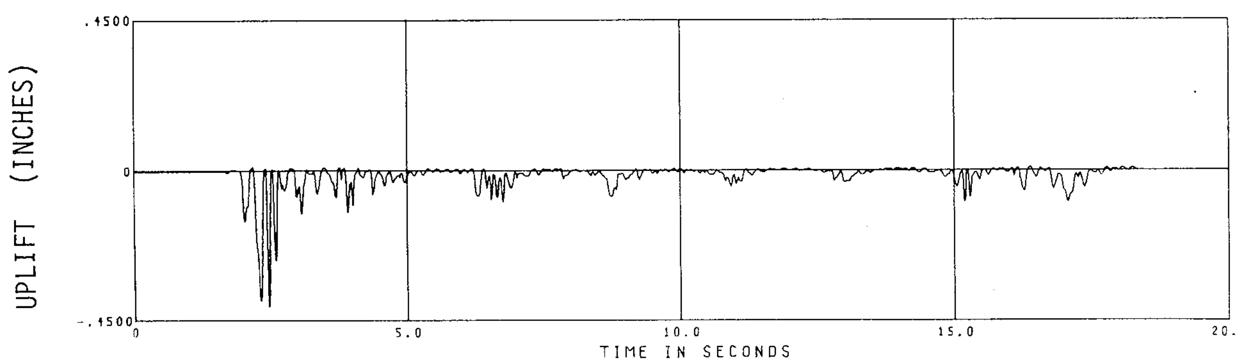
a. INPUT ACCELERATION DUE TO EL CENTRO 1940 (N-S)



b. PRESSURE RESPONSE AT GAGE P-1 (BASE FIXED)



c. WATER SURFACE RESPONSE AT GAGE W-1 (BASE FIXED)



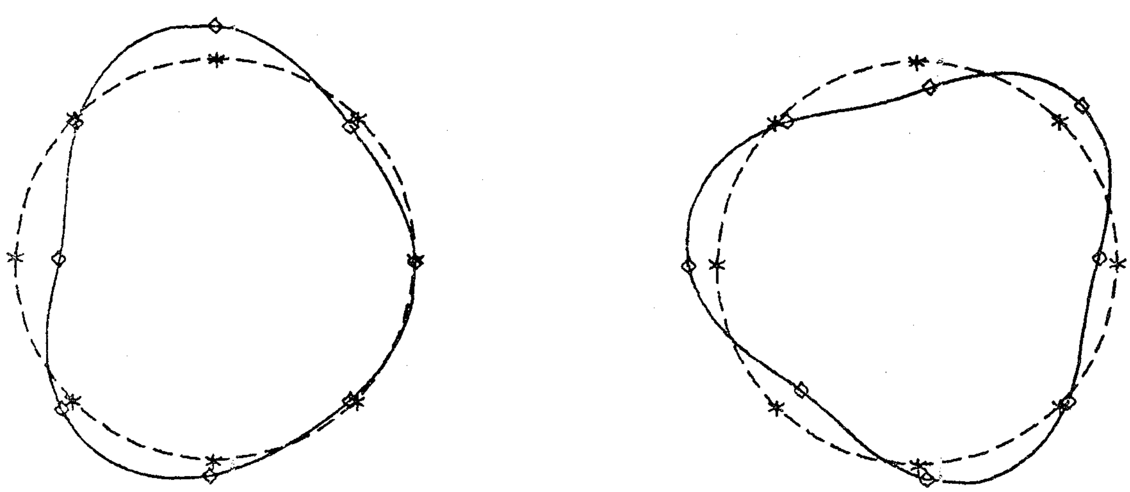
d. UPLIFT RESPONSE AT POINT B (BASE FREE)

Fig. 10 Measured Tank Performance during Test

The other major feature of the tank response is its displacement behavior; in particular, when the tank is not anchored at its base, uplift results which has a major influence on the displacements. The uplift induced at one edge of the tank in this test, shown in Fig. 10d, clearly correlates both with the wave sloshing and the base acceleration mechanisms. The uplift occurs only around a limited portion of the base, and is associated with a significant out-of-round distortion of the top rim. Fig. 11a shows how the top rim deflects inward as the base uplifts beneath that region; Fig. 11b shows the cos 3θ vibration mode which results from this type of deformation, and which continues until it is modified by another strong base acceleration impulse.

An attempt to portray the interrelationships between the various types of response measurements is presented in the isometric view plots of Fig. 12. Time variations of table acceleration, hydrodynamic pressures, wave height, tank wall stress, base uplift, and radial displacements are shown by the horizontal traces; and concurrent distributions of related quantities at specified times are shown by the vertical traces. Detailed study of such plots reveals a great deal about the actual earthquake behavior of cylindrical tanks, and it is hoped that mathematical models can be developed which will simulate accurately this complex fluid-structure interaction mechanism.

From the practical design point of view, the most crucial question raised by these test results is whether design procedures based on Housner's approximate theory lead to designs which are satisfactory with respect both to economy and safety. Clearly the response is not that of a rigid tank, but comparisons of the hydrodynamic pressures predicted by Housner's theory with the pressures observed for a tank with fixed base, shown in Fig. 13a, demonstrate that for this case, the essential response behavior is predicted quite well. On the other hand, where the tank is free to uplift, and therefore incurs much greater dynamic distortions, the actual hydrodynamic pressures deviate greatly from the predictions, as shown in Fig. 13b. Thus it is not



a. 2.5536 SECONDS　　　　b. 2.7264 SECONDS

Fig. 11 Deformations of Tank Top Rim, Base Free

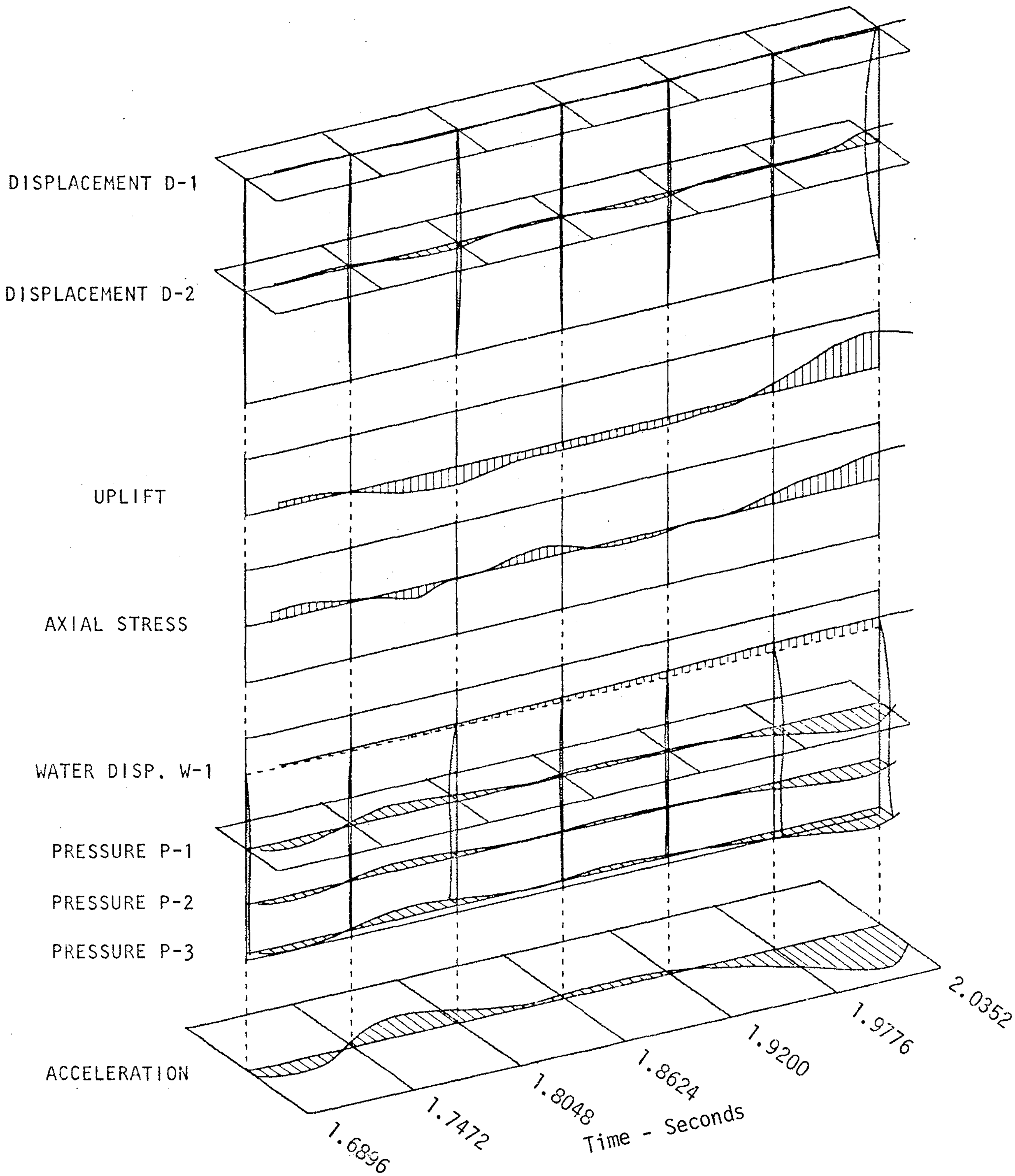
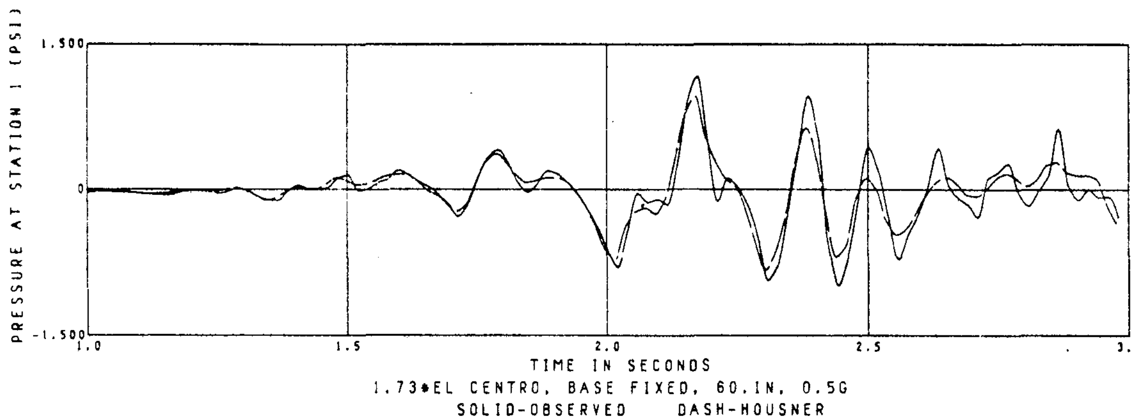
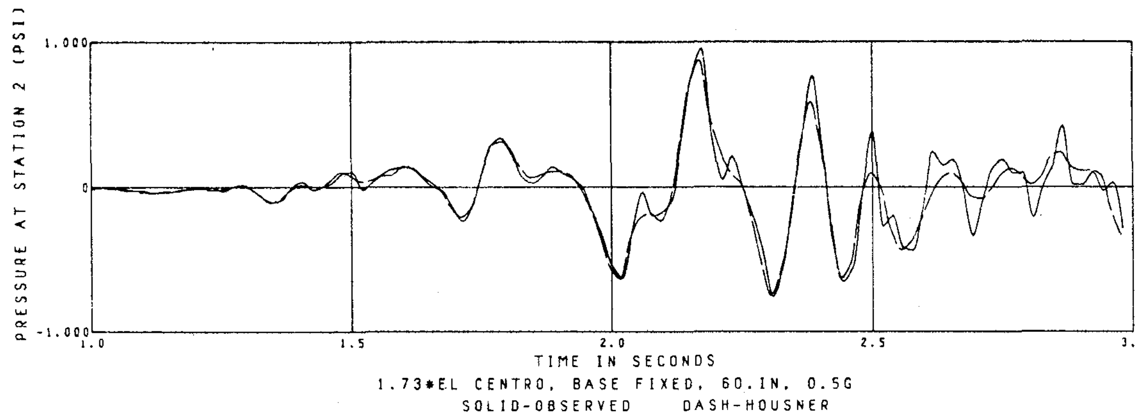
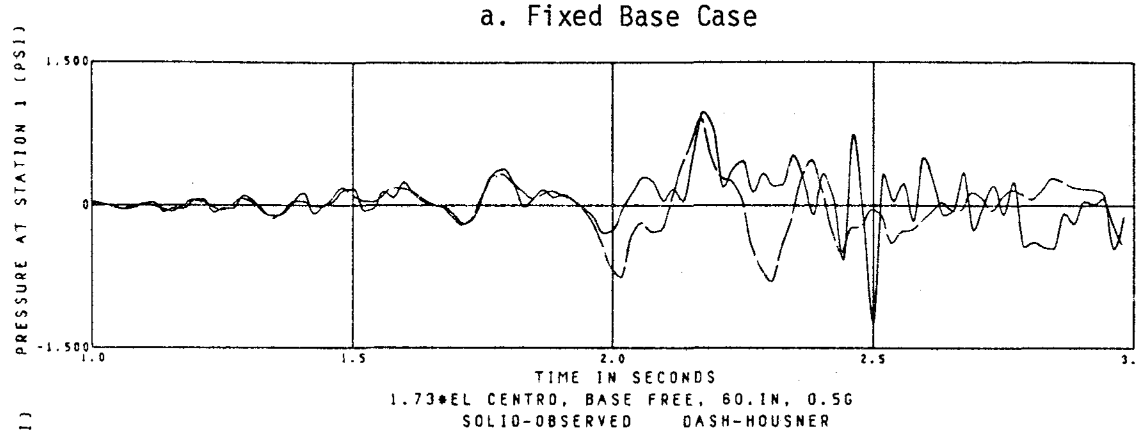


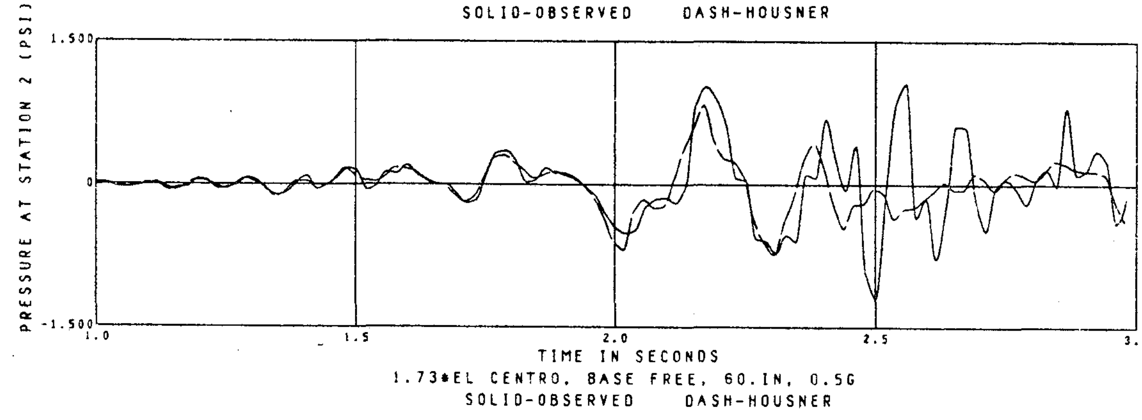
Fig. 12 Time Relationships of Table Accelerations and Tank Response





a. Fixed Base Case





b. Free Base Case

Fig. 13 Comparison of Observed and Calculated Hydrodynamic Pressures likely that tanks which are free to uplift can be designed adequately by means of the Housner theory. A report on the seismic performance of the 6 x 12 ft. tank model will be submitted to the project sponsors in the near future.

Conclusions

These two examples demonstrate how needs expressed by the structural design profession can lead to major research efforts undertaken by the research community. Many of the other research projects being conducted by my colleagues in the Earthquake Engineering Research Center also originated in response to suggestions from the profession, and there is no doubt that such practical motivation creates a useful sense of interest and concern among the students and faculty participating in the research.

It must be emphasized, however, that a major problem in the development of an effective engineering research program is establishing suitable channels of communication between the profession and the research organization so that the needs of the profession may be made known. Moreover, these must be two-way channels: first, informing the research workers of the design problems most urgently in need of study, and then when the research group has compiled sufficient data to formulate a solution, transmitting these results back to the engineers to incorporate into actual designs. Unfortunately, the latter step often seems to introduce the greatest difficulty in the entire research cycle; results of research frequently end up in the files of the researchers, or perhaps on the bookshelves of the practicing engineers, which is an equally useless outcome. It is imperative that the entire profession be concerned with this problem of research utilization; unless the results are put to use, the research process is merely a costly academic exercise.

Acknowledgements

The EERC Earthquake Simulator Facility was financed by the National Science Foundation, as was the test program on the uplifting frame earthquake response. Additional funds for this test program, and for the second phase of the uplifting frame tests have been provided by the American Iron and Steel Institute. The cylindrical liquid storage tank test program is financed by a group of companies under the leadership of the Chevron Oil Field Research Company. All participants in these EERC projects gratefully acknowledge this support.

The studies on the uplifting frame are being carried out by Ph.D candidate Arthur A. Huckelbridge, and the cylindrical tank studies by Ph.D. candidate Douglas P. Clough, both under supervision of the writer.

References

1. Joint Committee, San Francisco Section ASCE and Structural Engineers Association of Northern California, "Lateral Forces of Earthquake and Wind," Trans. ASCE, Vol. 117, 1952, pp. 716-754.

2. Rea, D. "Structural Research Using an Earthquake Simulator," Structural Engineers Association of California, Annual Convention Proceedings, October 1972.

3. Hospital Seismic Safety Act, California State Senate Bill 519, Chapter 1130, Statutes of 1972.

4. Clough, R. W. and Tang, D. P. "Earthquake Simulator Study of a Steel Frame Structure, Vol. 1: Experimental Results," University of California, Earthquake Engineering Research Center Report No. EERC 75-6, 1975.

5. Rinne, J. E. "Oil Storage Tanks," The Prince William Sound, Alaska, Earthquake of 1964, U. S. Department of Commerce, Coast and Geodetic Survey, Vol. II-A, p. 76, 1967.

6. Housner, G. W. "Dynamic Analysis of Fluids in Containers Subjected to Acceleration," Nuclear Reactors and Earthquakes. TID 7024, U. S. Atomic Energy Commission, 1969.

SEARS TOWER: SPECIAL STRUCTURAL DESIGN AND CONSTRUCTION CONSIDERATIONS

by
Fazlur R. Khan
Partner and Chief Structural Engineer
Skidmore, Owings & Merrill

Introduction

Chicago's Sears Tower project (Fig. 1) is the largest single private office complex in the world, since its completion early in 1974. The total development contains a gross floor area of 4.4 million square feet, and when fully occupied will involve a daily user population of 16,500 people. Sears Tower is now the world's tallest building with 110 stories for a height of 1,450 feet above ground and will enclose 3.9 million gross square feet of office space. The building is being occupied since Fall of 1973.

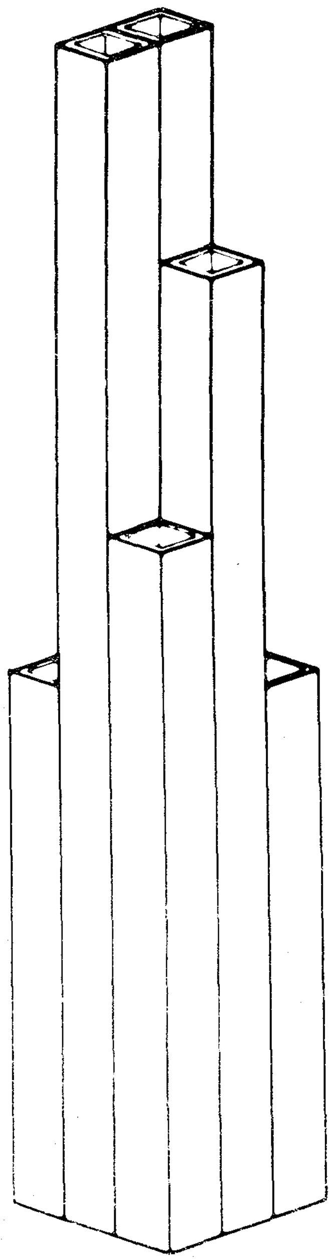
The project occupies a full city block of approximately 129,000 square feet on the southwest side of Chicago's Loop. The tower occupies about 41% of the site area and the rest is designed to be an open plaza. The concept of a single tower system in lieu of multiple towers was to create a large plaza at the street level with human activities and a greater feeling of open space which is so needed in urban centers. This urban environment could be created because of the use of a very efficient and economic structural system.

The search for a new structural concept was therefore central to the overall project development. The evolution of the "Bundled Tube" system represents a logical integration of office space requirements and structural efficiency for an effective economic solution. The geometric reformulation of a single perimeter framed tube into the Bundled Tube system caused higher efficiency for the building as an upright cantilever, with the consequent reduction in structural steel quantity.

This report presents in detail the development of the structural system and studies for optimum combination of subsystems and key parameters. The structural analysis and design methodologies, including dynamic aspects, are also presented together with development of significant structural details. It should be noted, however, that while descriptions pertain to Sears Tower, the design considerations are applicable to any ultra high-rise project.

Space Criteria

The non-prismatic building profile is a direct result of space requirements and represents an integration of two different criteria. One pertained to space to be occupied by Sears, Roebuck and Company (approximately 60% of the Tower) who required large floor areas for their operation and the other to the rental portion, which for maximum flexibility and efficiency, was required to be smaller and if possible in varying sizes. The concept was to create a three dimensional, modular, spatial arrangement which would permit large floor areas desired by Sears in the lower portion of the building, while the upper portion would consist of floors of different sizes and shapes. The eventual architectural-structural solution was a modulized "drop off" system consisting of an assemblage of nine 75 ft. square shaped megamodule "framed tubes" which were terminated at various heights to create the drop-off, which interestingly appears somewhat similar to the earlier high-rise building with step backs (Fig. 2). The overall shape is composed of nine such square tubes for a square floor dimension of 225 ft., which continued up to the 50th floor. These larger floors of 52,000 sq. ft. are suitable for Sears' occupancy. Drop-offs then occur at floors 50, 66, and 90 creating a variety of floor configurations ranging from 41,000 to 12,000 sq. ft. in floor area as shown in Figure 2a. An interesting aspect of floors above the 50th floor was that more perimeter space was created and the non-prime space was, therefore, practically eliminated. The consequent increase of prime rental space was extremely beneficial in maximizing the rental revenue. In addition, the monotony



MULTIPLE OR
BUNDLED TUBE

FIGURE 2

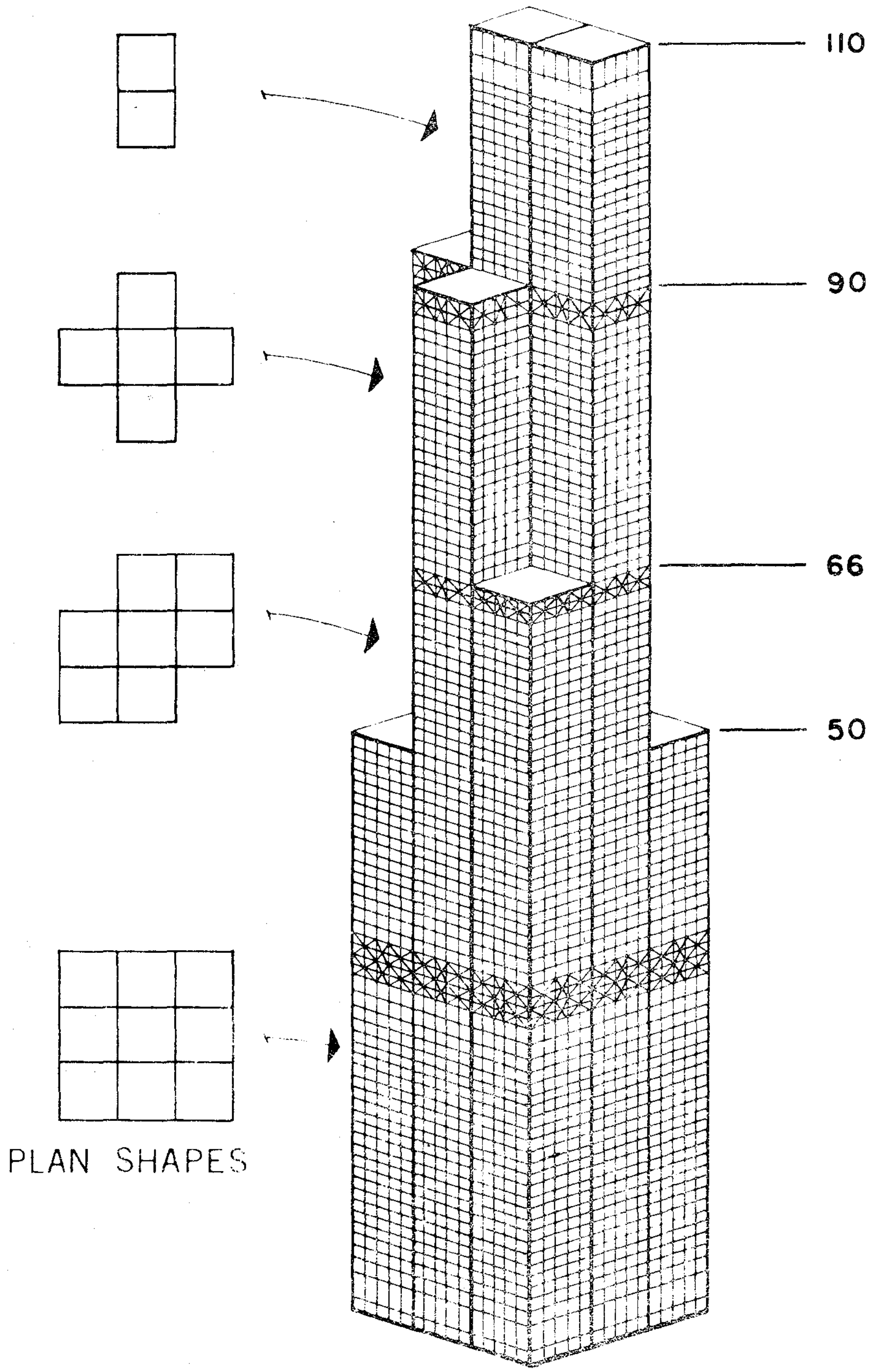


FIGURE 2A

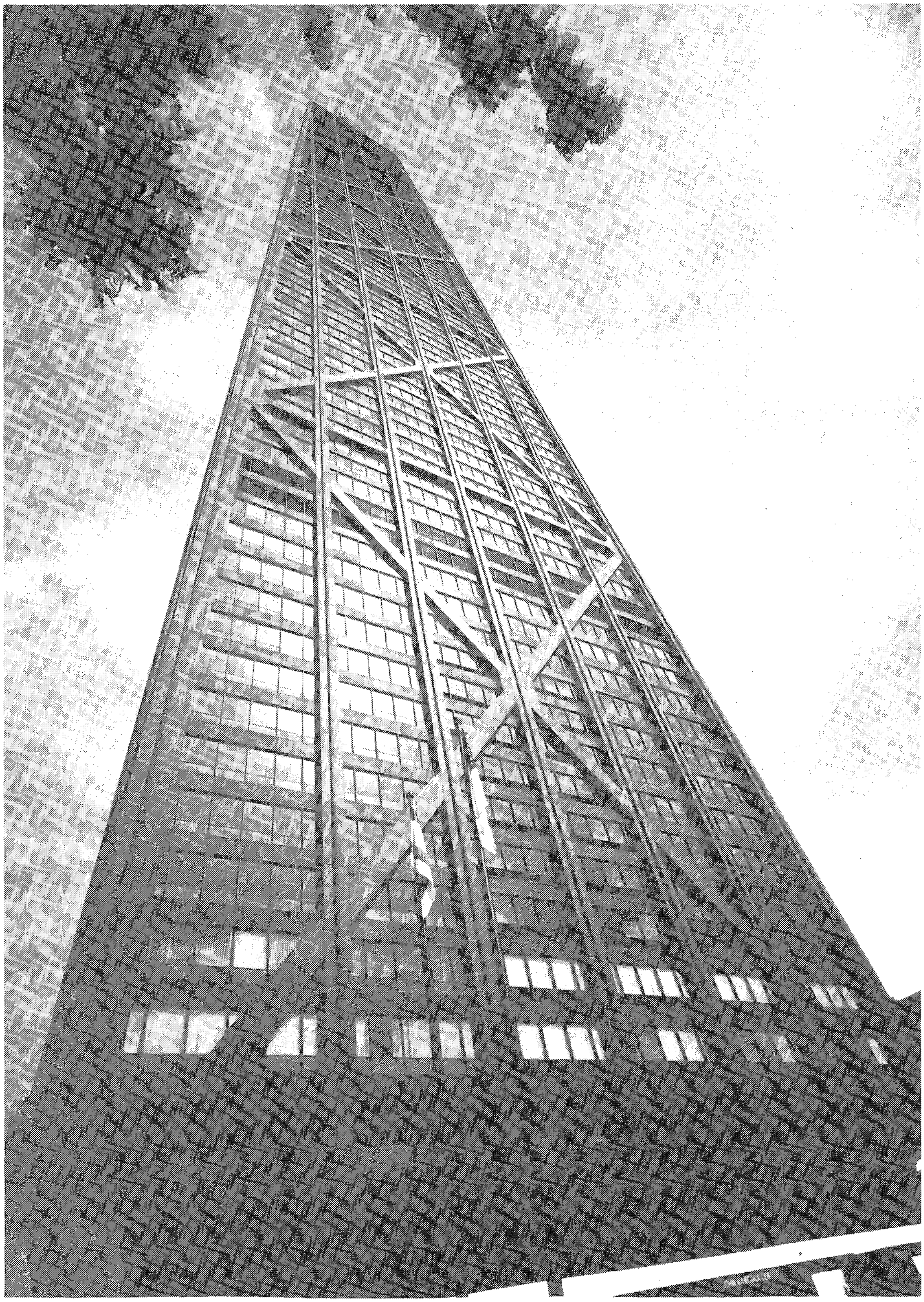


FIGURE 3

of a constant prismatic "shoe-box" shape was also avoided.

Bundled Tube Concept

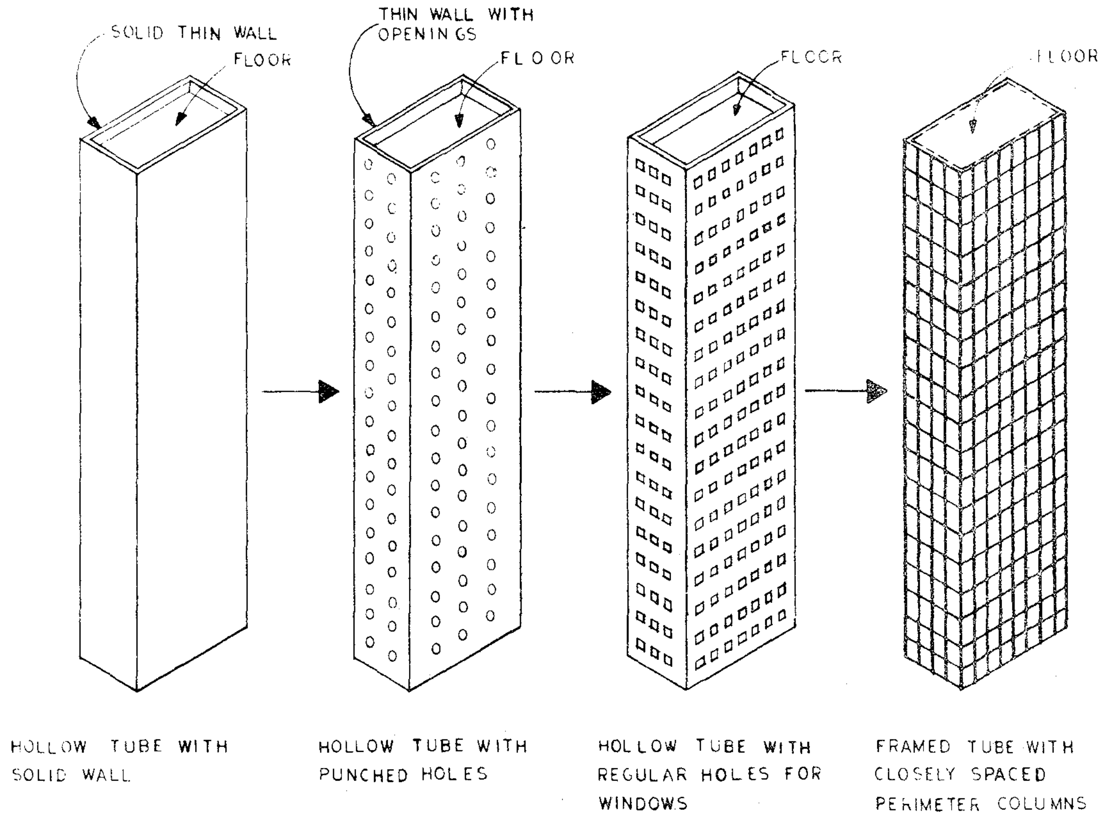
The first step in the design process was the selection of an efficient structural system which would have the highest inherent lateral stiffness. Historically, high-rise steel buildings have most often used plane-frames with 25 ft. to 40 ft. bays. These structures have extremely low efficiency in resisting lateral loads, and therefore, involve very high structural steel premiums over what is required to carry only the gravity loads. For example, a 30 ft. span frame structure for a 110-story building would require structural steel quantities of the order of 60 to 65 psf of average floor area as contrasted to 33 psf required in the final design of the Bundled Tube System. Although a number of other systems had been developed earlier by the writer in an attempt to reduce the steel premium for tall buildings, such as the Truss-tube system used for the 100-story John Hancock Center in Chicago (Fig. 3), they were not suitable for Sears space requirement program.

Another system also developed earlier by the author included the introduction of vertical truss elements in the core of the building and introduction of Belt trusses and Outrigger trusses connected to core trusses. These systems were very effective for 40 to 60 story buildings, but were not adequate for extremely tall buildings. A new class of structures termed "Tubular Structures" were developed which exhibited a considerate increase in cantilever efficiency by making the perimeter frame to act as a vertical cantilever tube.

The concept of a tubular structure is to create an equivalent of a solid thin-walled hollow tube of the configuration of the perimeter of a building. Its development was a logical outcome of the search for the most efficient use of the vertical load carrying members to resist the effects of horizontal loads on the building. Ideally, the most efficient structural form would be a thin walled tube designed to carry the vertical loads of the interior floors. However, in a real building openings must be provided for exterior exposure.

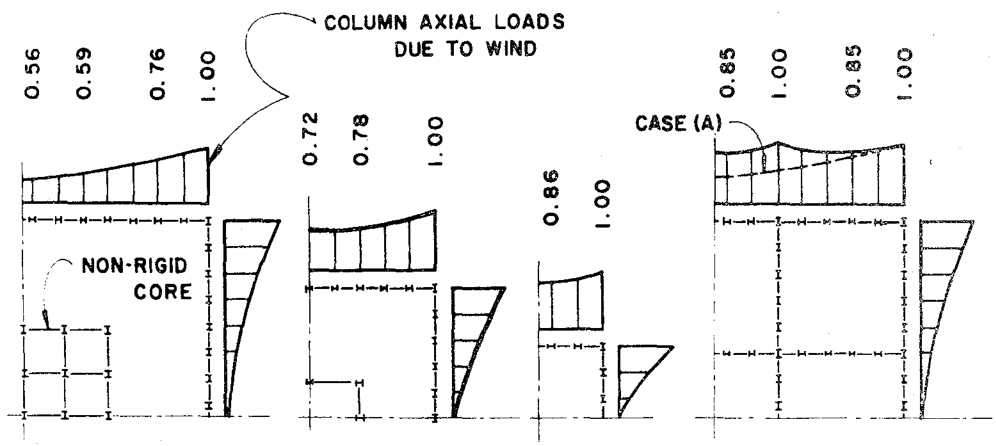
This led to the evolution of the framed tube system. (Fig. 1a) The framed tube system in its simplest form consists of closely spaced exterior columns tied at each floor with deep spandrel beams, thereby creating the effect of a hollow vertical tube with perforated openings for the windows. As the overall plan size of such a framed tube increases, the effectiveness of such a tube in acting as a pure vertical cantilever reduces, partly because of the effect of the shear lag of exterior walled elements and partly because of the non-participation of the many interior columns required to support large floor area. The next logical evolution of the tubular system was then to divide a larger tube into smaller cells which can be looked upon as a series of smaller sized framed tubes bundled together to create a large size tubular structure. The shape of the component modular tube can be square, rectangle, or any other polygonal form that can fit together with each other. However, for practical reasons and simple achievement of higher efficiency, the square shape tends to be always the first choice. The bundled tube concept used for the Sears Tower was for the same reason based on square dimension 75 ft. by 75 ft. for the unit modular tube. The column spacing along the wall of each tube was chosen to be 15'-0 on centers based on its effect on tube efficiency on one hand and the effective architectural planning of the entire building floor on the other.

The transformation of the solid walled tube into a perforated tube, consisting of discreet elements, adds one more component to the lateral cantilever displacements due to Pure Cantilever behavior normally referred to as the column shortening effect. The additional component is termed"Shear-Frame component" and is caused by double curvature bending deformations of columns and beams of the tube. The effectiveness of a tubular system can be assessed by studying the Cantilever component (Column Shortening) relative to total lateral displacement. The inter-relationship between shear frame (D_{sf}) and column shortening (D_{cs}) components defines the relative nature of the structure; for a true cantilever $D_{sf} = 0$ and for a pure shear-beam frame $D_{cs} = 0$. Therefore, the basic premise of developing a highly efficient cantilever tube would be to reduce the D_{sf} component



EVOLUTION OF THE FRAMED TUBE CONCEPT

FIGURE 1A



(A)	(B)	(C)	(D)	
EXT. TUBE	EXT. TUBE	EXT. TUBE	BUNDLED TUBE	- TYPE
225' x 225'	150' x 150'	75' x 75'	225' x 225'	- SIZE
6.65	9.50	19.00	6.65	- H/W
0.61	0.75	0.88	0.78	- Δ_{cs}/Δ_{T}

FIGURE 4

to a realistic minimum. The cantilever efficiency could be defined by the ratio of D_{cs}/D_t where D_t = total lateral displacement = D_{sf} + D_{cs}. A tubular system with an efficiency of about 0.80 is a desired target value for avoiding the premium for height in an ultra-high rise building, and thereby keep the structural steel quantity to a minimum.

The size of the unit modular tube is influenced by the efficiency of the floor system and efficiency toward cantilever behavior. Fig. 4 shows studies of three sizes for cantilever efficiency under lateral loads. They are 225 ft., 150 ft. and 75 ft. The respective cantilever efficiencies are 0.61, 0.75 and 0.88. The cantilever efficiency of the selected bundled tube at the base of the building is .78. For a large structure with large percentage of exterior openings such high efficiency is rarely achieved. The cantilever efficiency is also influenced by the height (H)-to-width (W) ratio of the tube. A larger H/W ratio results in larger cantilever efficiency for a particular frame stiffness. The maximum size of each modular tube would normally be limited by the floor system. However, because of the high efficiency of the composite truss system, spans up to 80 ft. can be economically feasible.

The studies of modular tube sizes resulted in the selection of 75 ft. for the modular tube unit. The bundled square shape represents a H/W ratio of 6.65 at the base although the average H/W ratio is in fact higher than 7.5. A three dimensional plot of the column axial load distribution under the lateral load is shown in Fig. 5. The net effect due to bundling the module tubes, has been the same as to provide two additional web frames in each direction engaged to perimeter flange frames. As a consequence, the transverse wind shears were transferred at four points on each flange face, thus lifting the sagging axial load distribution line of the exterior framed-tube into peak points at the intermediate frame locations as shown in Fig. 5.

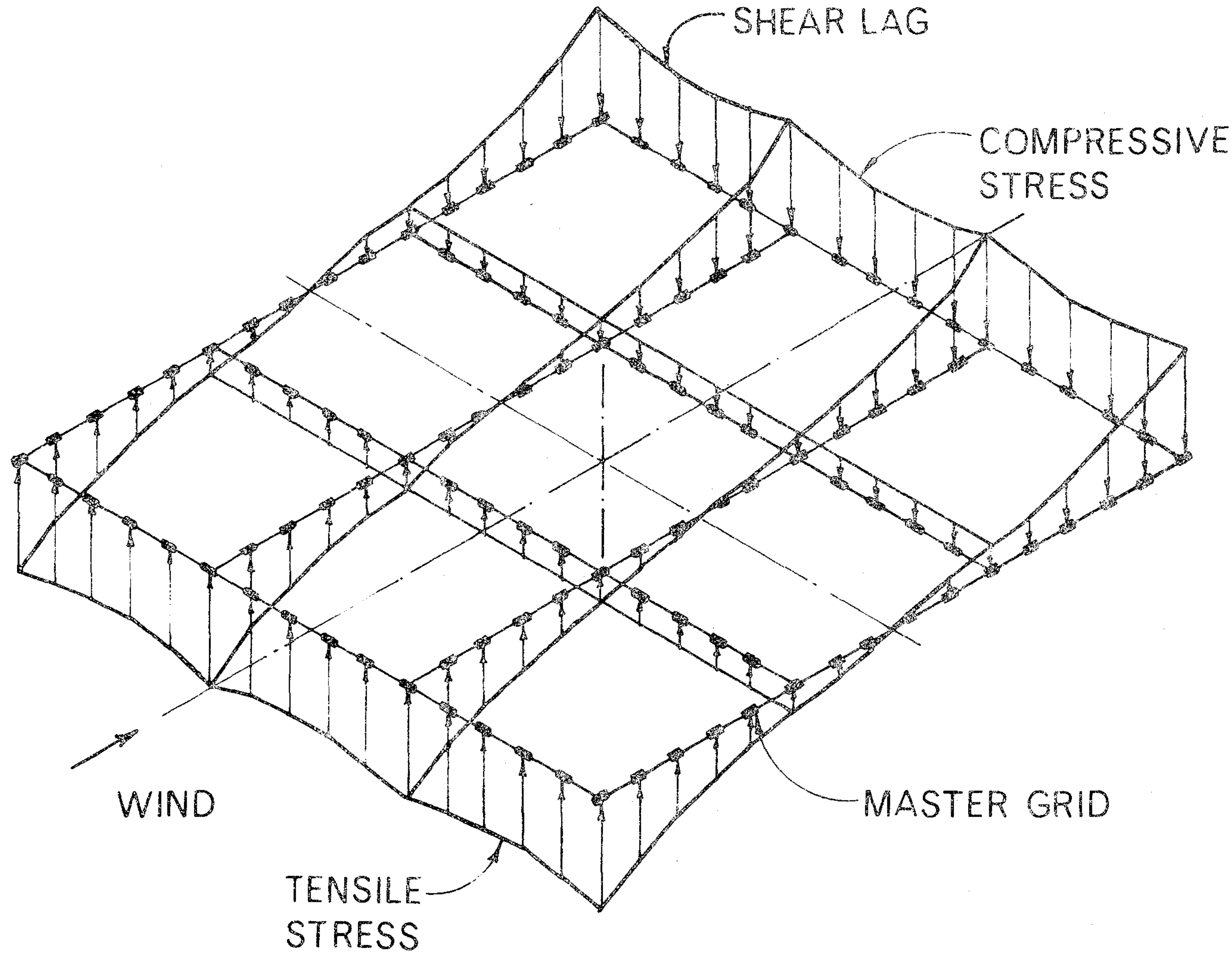

SHEAR LAG
COMPRESSIVE
STRESS
WIND
MASTER GRID
TENSILE
STRESS

FIGURE 5

Description of Structure

The framed-tube consists of wide steel columns at 15 ft. on centers and deep steel beams at each floor as shown in Fig. 3. All beam-to-column intersections are fully welded connections. The drop-off is accomplished by termination of columns on any particular tube while other columns in the remaining tubes continue. Belt trusses around each tube, consisting of diagonal members between columns, are provided at several mechanical levels and their locations were planned to occur immediately below the drop-offs at the 66th and 90th floors. Other Belt truss levels occur at the 29th and 31st mechanical floors. The Belt Trusses acted as vertical shear diaphragms and were instrumental in reducing the general dishing effect brought about by differential column shortening drop-offs, while at the same time providing an efficient means of absorbing the large member shears in the immediate vicinity of the termination. The Belt Trusses also contributes to lateral stiffness by eliminating shear-frame displacements over mechanical levels. The overall tubular efficiency was also improved due to vertical shear diaphragm effect at those levels. Fig. 6 shows schematically the improvement in lateral displacement distribution due to Belt Trusses.

The floor within each modular tube was typically framed by one-way 75 ft. span trusses at 15 ft. centers. Each truss frames directly into a column by means of a high strength friction bolts (A490) designed for shear only. The span direction of these trusses was alternated over groups of six floors to equalize the gravity loading on the walls of the modular tube. The trusses are 40 inches deep and their design was based on composite action with the floor slab. The floor slab consisted of 3-inch composite blended metal deck with 2-1/2 inches of lightweight concrete above the deck for a total thickness of 5-1/2 inches. The composite deck spans the 15 ft distance between the trusses. The composite assembly is shown in Fig. 7.

The blend includes a 28-inch cellular portion for electrical and telephone services combined

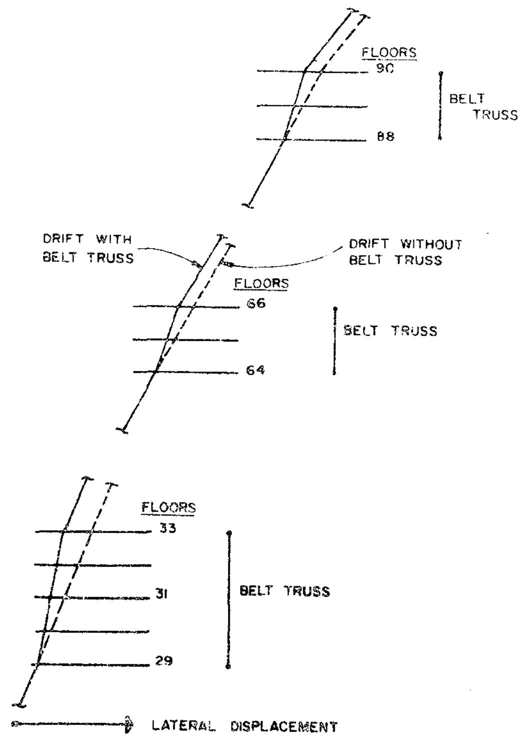


FIGURE 6

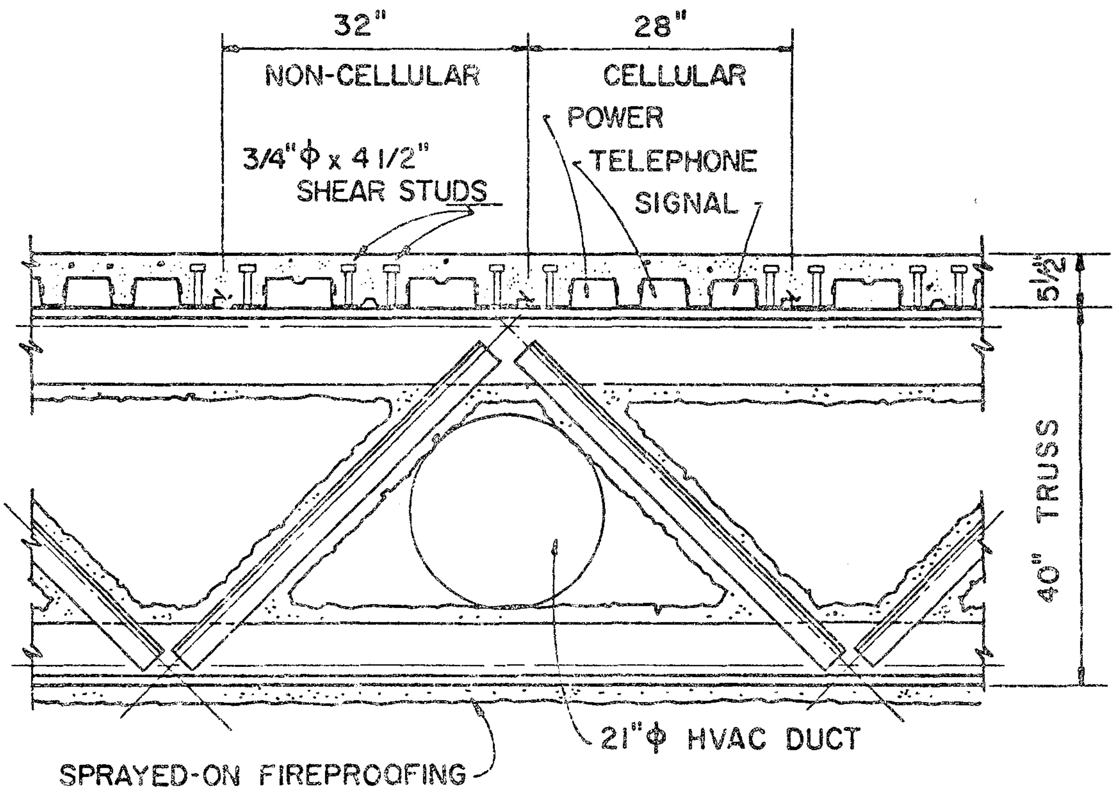


FIGURE 7

with a 32-inch noncellular portion for each 5 ft. module. The composite action of the truss was established by 3/4 inch x 4-1/2 inch shear studs welded through the metal deck in the noncellular portions. The interaction of the metal deck and the rib shaped concrete above the deck produced shear cone type failure of concrete above each group of studs. The shear stud capacities were therefore, related to the dimensions of the shear cone. Extensive experimental verifications were undertaken by means of push-off and beam tests to establish maximum stud capacity. Other tests included single and multiple span slab tests and full scale verification of the truss-slab assembly. The development of the total composite assembly also resulted in an efficient floor diaphragm. Plenum continuity and space for passage of mechanical and plumbing service systems were provided through the triangulations in the trusses. The mechanical-structural integration allowed all the space above the ceiling to be utilized for the truss depth.

The members of the Framed-tube were proportioned to develop maximum tube-cantilever efficiency with simple member shapes. Open three plated built-up I-sections were used for both column and beam elements which simplified fabrication. The three plated open section also reduced residual stress problems due to welding as compared to other shapes. The column and beam sections were made as deep as practicable, thus allowing 39 inches for column and 42 inches for the beams throughout the entire height of the building. Larger depths would have involved cost premiums due to web stiffening or thicker web plates. These depths were held constant for all typical levels and frame lines to standardize prefabrication of the frame segments in a jig. Column flanges varied from 24 in. x 4 in. at the bottom to 12 in. x 3/4 in. at the top and that of the beams from 16 in. x 2-3/4 in. at the bottom to 10 in. x 1 in. at the top.

The proportion and size of the corner column at the intersection of two tubular lines was modified to respond to biaxial force conditions in two orthogonal directions. The depth and width of these columns were both made equal to 39 inches, still retaining the simple I-shape. The flange thicknesses varied from 3-1/2 in. at the bottom to 1 in. at the top.

A total of 76,000 tons of structural steel is involved in the project, consisting of ASTM-A588 and A572 grades for columns and A36 for beams and floor framing. The unit structural steel quantity for the tower portion amounted to 33 psf.

Analysis and Design Approach

The preliminary design was performed in two phases. The first phase was concerned with the framed-tube behavior and optimization of key parameters. The overall geometry including the various drop-offs was determined in this phase together with optimum column spacing and member proportions. Two dimensional frame analyses were performed on the flange and web subsystems. Efficiency was judged on the basis of the ability of the frame to distribute loads to produce the least shear lag. The study was logically extended to equivalent three-dimensional tubes of different overall plan dimensions to correlate the size effect. Gravity and wind load distributions were derived at various heights in the building for preliminary member designs. The shear-frame and column shortening components of the lateral displacements for the wind loads were separately computed and superimposed.

The second phase of the preliminary analysis and design was performed on a more refined basis using three dimensional frame analysis to verify the overall behavior of the Bundled Tube. The unsymmetrical geometry required a division of the building into two vertical segments as shown in Fig. 8. The lower part (Part I) was based on two diagonal symmetry lines and was applicable only up to the 90th floor. The upper part (Part II) had only one axis of symmetry and was used only above the 66th floor. The number of joints in each part was reduced by formation of a coarse grid equivalent frame, whereby several prototype stories were grouped together to represent one story in the model. The equivalent frame levels are also shown in Fig. 8. Each column was represented in its true geometric

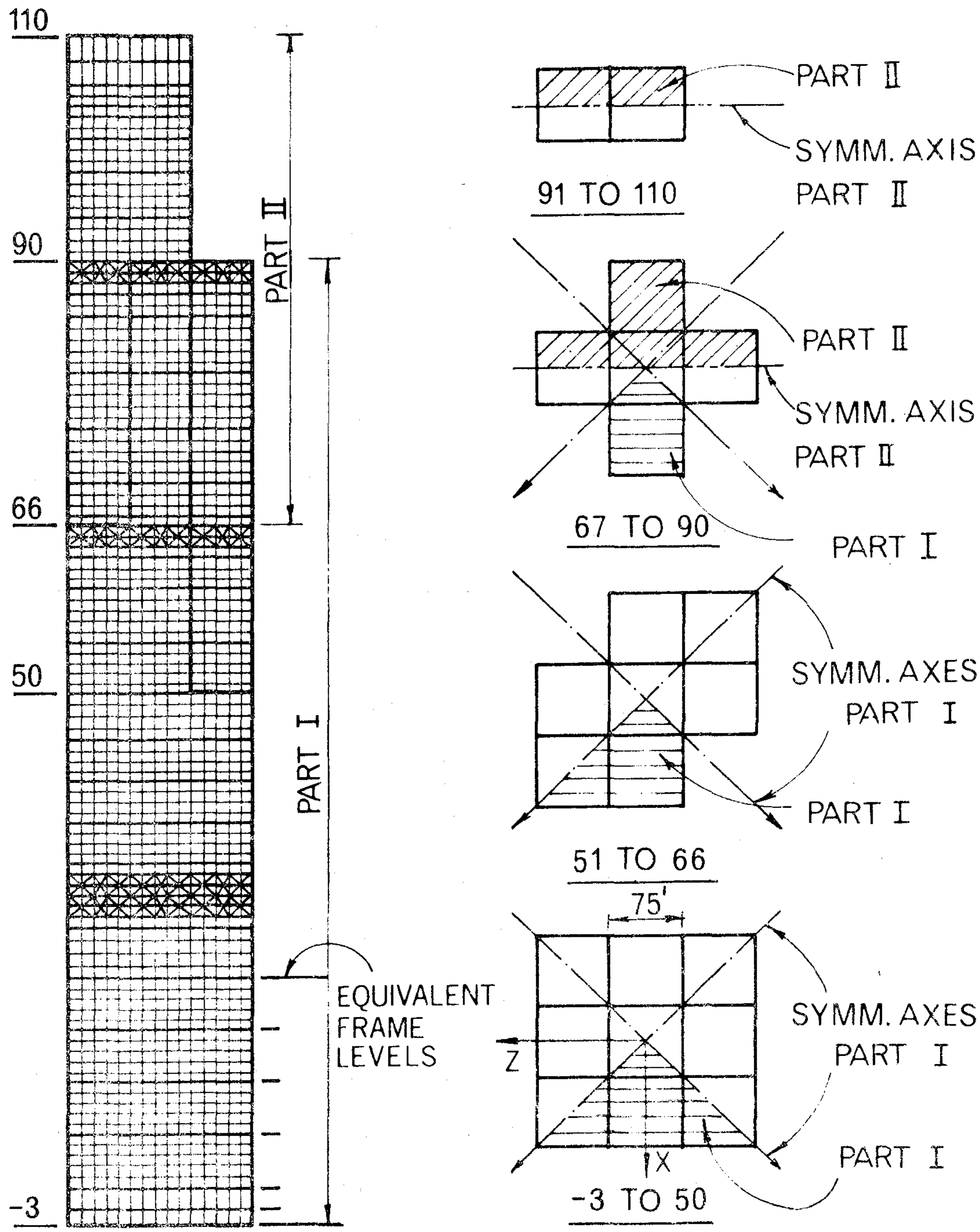

110
90
66
50
-3
PART II
PART I
EQUIVALENT
FRAME
LEVELS
PART II
SYMM. AXIS
PART II
91 TO 110
PART II
SYMM. AXIS
PART II
67 TO 90
PART I
SYMM. AXES
PART I
PART I
51 TO 66
75'
Z
X
SYMM. AXES
PART I
PART I
-3 TO 50

FIGURE 8

location to accurately simulate the column shortening component. The equivalent stiffnesses were derived on the basis of equivalent lateral displacement characteristics. The frame analysis involved 968 joints and 2,094 members in Part I and a smaller number in Part II.

The final verification was performed on the 3-dimensional full building frame. The full building was modeled by a vertical combination of several floors to formulate the equivalent frame. The total structure of 10,000 joints and 21,300 members was reduced to a 4,000 joint and 10,000 member space frame simply because it was evident at that time that its solution would be difficult, if not impossible, with the available computer hardware storage capacity and software capability. The reduced frame was analyzed for gravity and four wind loading cases. Two wind loadings corresponded to the Chicago Code wind distributions and the other two to pressure distributions obtained as a result of the statistical wind study and wind tunnel tests.

The results of the full building analysis were used to perform member and connection designs. Separate computer programs applicable for the design of columns, column splices, beams, beam splices, beam-column moment connections and beam column panel zones were used.

Evaluation of Dynamic Properties

The dynamic properties consisting of the modes, frequencies and damping factors were used to assess the dynamic behavior under the wind load and also for the construction of the Aero-elastic mode for the wind tunnel test. The modes and frequencies were derived for an equivalent seven mass mode. The fundamental period was computed as 7.8 seconds which compared favorably with other buildings in this general height range. A separate analysis for the torsional period of 3.3. seconds. The frequency of the translatory modes in the two principal directions was similar.

The assessment of the damping factor was, however, more complicated. A review of

loads in two principal directions. While these methods appear to be adequate for high-available experimental data on several steel structures revealed that damping generally ranges from 1 to 2 percent of the critical damping. Recent field measurements on the 100-story John Hancock Center in Chicago, also designed by the writer, resulted in damping factors of the order of 0.6 percent because of the involvement of a large number of members

directional loading controlled the design. Similarly, beams are also controlled by uni-directional loading.

Joint action factors representing the degree of participation of each of the responses corresponding to the principal directions were developed for each junction column at one of the lower floors based on wind tunnel results and detailed statistical analysis. It was assumed that junction columns at other levels would also be affected by a similar combination. The Joint action factors ranged from about 44 percent from each principal direction to 73 percent from each principal direction for different junction columns at the first floor. A constant Joint action factor of 75 percent from each principal direction was established for all levels in the building.

Column Design

Column designs for each loading combination included axial forces and biaxial bending moments for each load component. An effective length of one was typically used for all columns. In a tubular structure, the instability behavior of individual columns in a sway mode cannot be treated independent of the total system. Because of the large in-plane shear stiffness of the slab diaphragm, the buckling mode of any column must necessarily involve all columns in the same story in a similar mode. Since the predominant component of side sway is a result of column shortening, buckling evaluation of a story involves the entire cantilever system. An approximate buckling analysis of a vertical cantilever was performed using equivalent cantilever properties of the Bundled Tube. Column designs at the design level were performed according to AISC methods with all the attendant provisions. An increase of 33 percent in allowable stress was used.

loadings. The bending stresses were verified against allowable and yield stresses for the two loadings respectively. Similarly, web shear stresses were also verified recognizing that the shear yield approximately corresponds to $F_Y/\sqrt{3}$ where F_Y is the Yield stress of steel.

Panel Zone Stresses

Stresses in the unreinforced column panel zone of the typical moment connection were computed for verification against allowable and yield stresses. Member forces from the full building solution were reduced to a set of axial and shear loading on the panel plate. Principal normal and shear stresses were computed for a typical element in the panel zone. Octahedral normal and shear stresses were also computed. In general, the principal stresses were higher than the octahedral stresses.

Joint Details and Erection

The systematic regularity of the modular framed-tube combined with simple shapes and uniform member depths permitted efficient use of prefabrication concepts. The concept is based on the formation of a two-story erection unit consisting of the column and half-length beams on either side. A typical shop fabricated unit is shown in Fig. 9. The splices typically occur at mid-spans for beams and midstory heights for columns and generally correspond to the natural points of contraflexure in the two elements. The modular frame units were transported to the site by truck and field bolted to neighboring units. It is obvious that the elimination of practically all field welding, smaller number of pieces and simpler field bolted joints, contributed to fabrication-erection economy. A particular advantage of prefabrication related to the use of automated welding procedures which were performed under well controlled shop facilities and in desirable positions. The modular frame units were fabricated in a jig in a horizontal position for tight dimensional control (Fig. 10). Prefabrication further facilitated shop verification of weld quality and shop corrections of non-compliant welds under better controlled conditions.

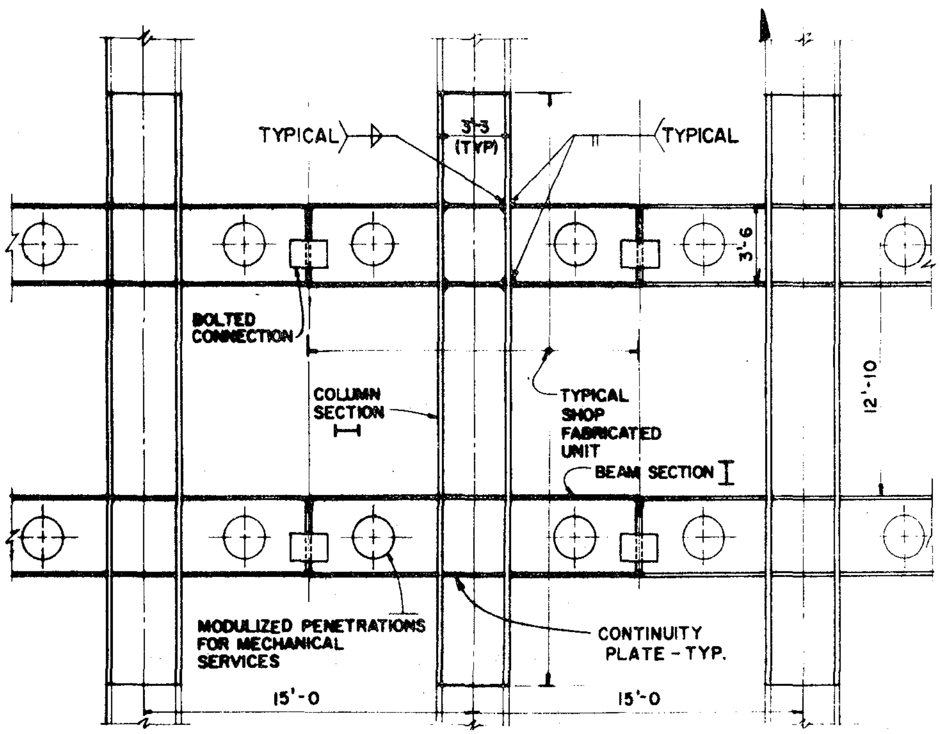


FIGURE 9

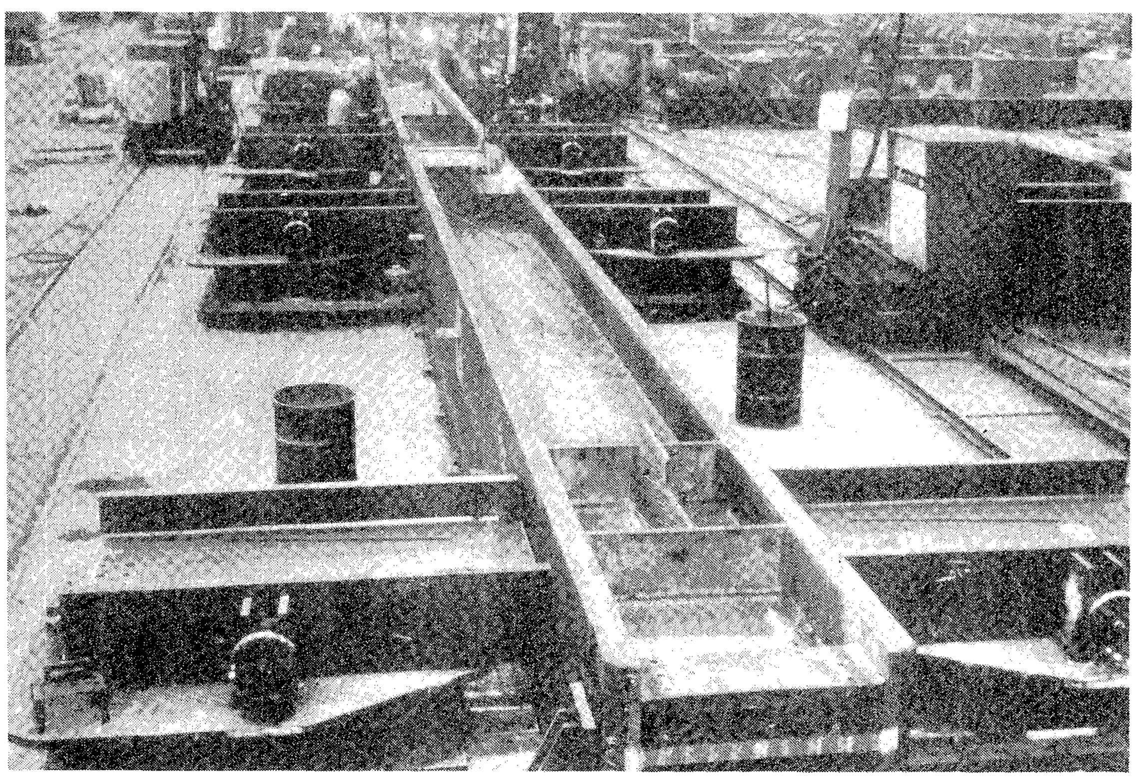


FIGURE 10

Since the butt welding of beam flanges was performed with longitudinally free half-length beams, the residual stress due to weld shrinkage was practically eliminated in this direction. An automated electroslag welding process was used for the butt welds of beam flanges to columns. This was performed in the vertical beam flange position. At the column-beam joints the continuity plates inside the column flanges were fillet welded by the innershield process.

Beam splices typically used high strength A490 friction bolts in double shear with a splice plate on each side of the web. Oversized holes (1-7/16 dia. for 1-1/8 dia. bolts) were provided in both beam web and splice plates to allow for erection tolerances. Several bold slip tests indicated a relatively high factor of safety against slip in excess of 1.8. A positive engagement with the concrete slab was provided by means of shear studs as shown in Fig. 11. Nominal bending moments that occur at the splice location were resisted by the composite assembly by formation of a force couple. The resistant arm for the couple was increased by lowering the center of gravity of the bolt group. Axial forces in the concrete slab were resisted by reinforcements in the slab and the transfer mechanism was established by shear studs.

Column splices consist of field bolted web connections with minimum partial penetration bevel welds for flanges as shown in Fig. 12. The particular bolted web detail was designed to satisfy two requirements: (1) to provide enough wind resistance before flange welding over an arm A during construction, and (2) To resist permanent column shears parallel to the web. Larger flange bevel welds were provided at isolated locations for column bending at the splices. In general, the predominant splice force including the wind effects was compression. Isolated occurrences of axial tension were verified using 75 percent of the dead load for strength purposes. Fig. 13 shows a field view of column splice plates.

The diagonal members of the Belt Trusses typically consist of standard T-Sections used back to back. The connection is by means of A490 friction bolts with one gusset plate

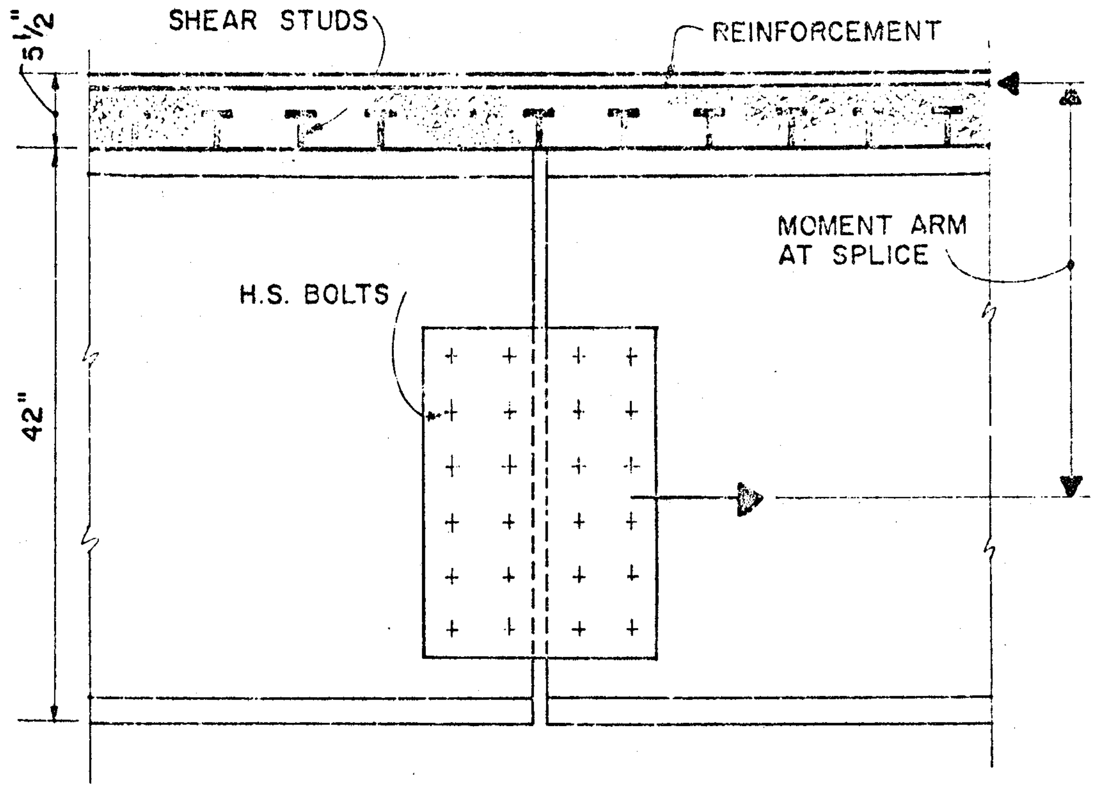


FIGURE 11

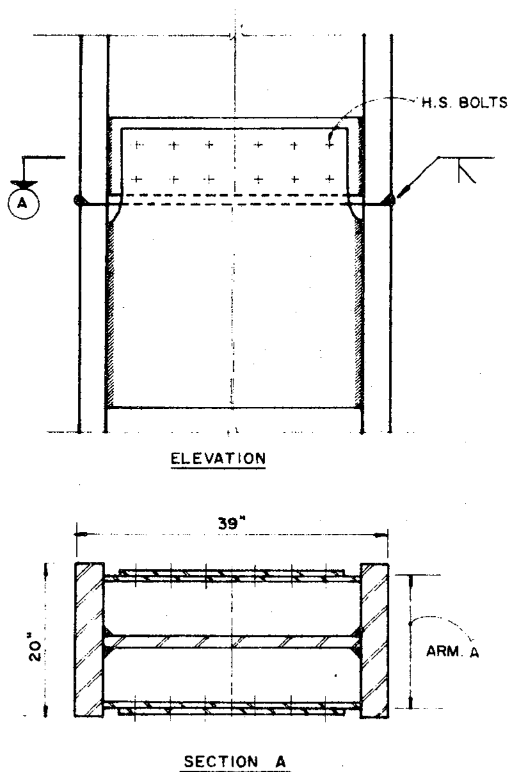


FIGURE 12

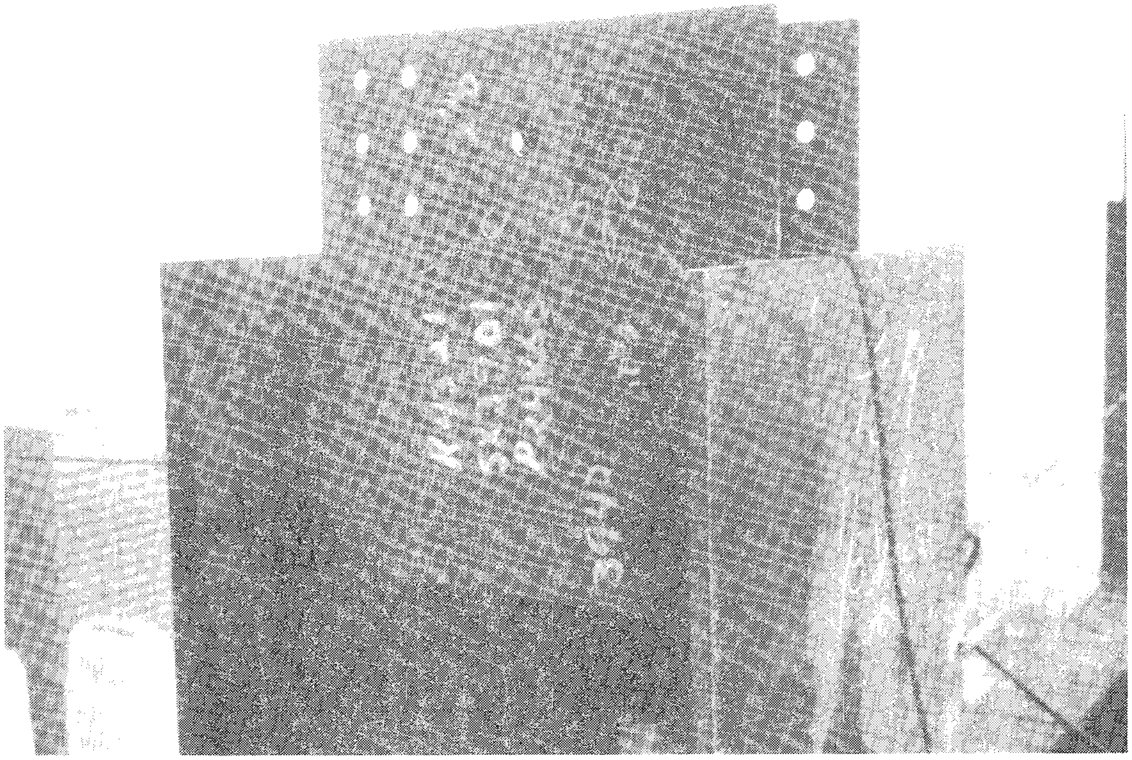


FIGURE 13

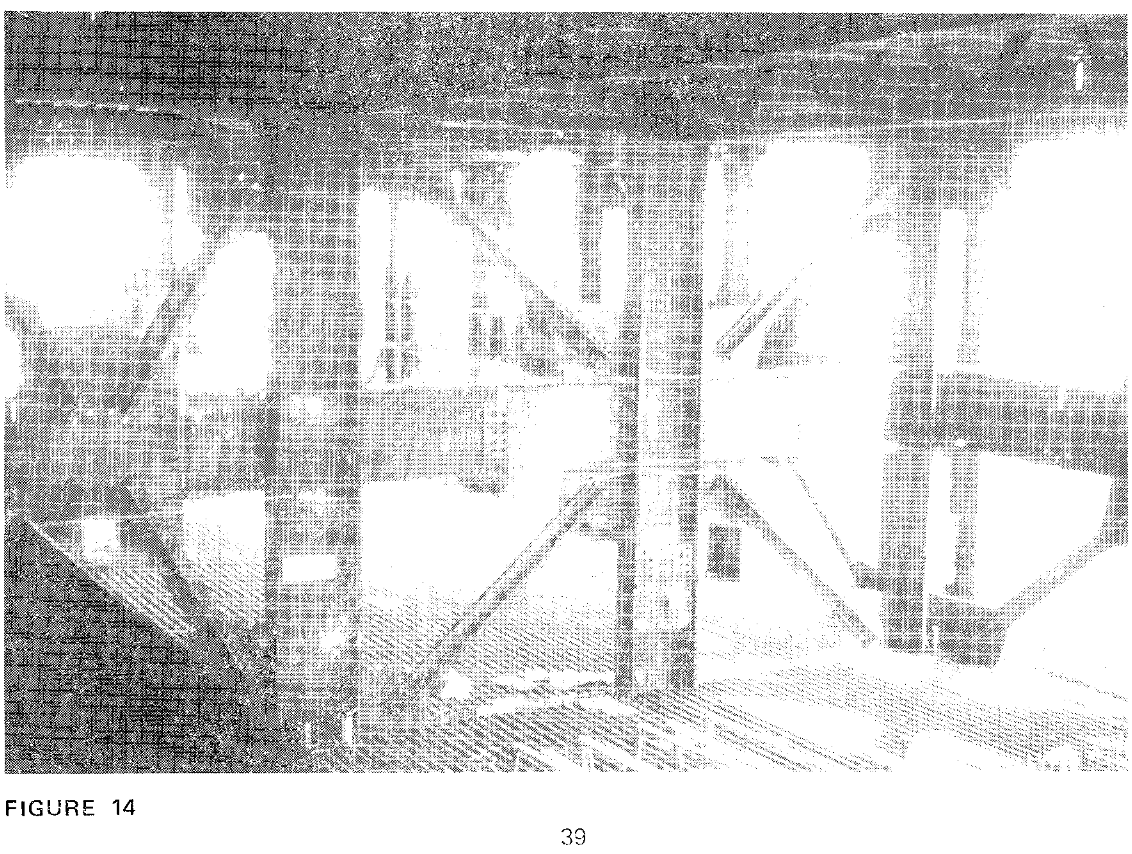


FIGURE 14

between the T-Sections. The gusset plates are typically fillet welded to beams and columns. Fig. 14 shows a field view of the joint.

Structural steel was erected by standard S2 type stiffleg derricks capable of lifting modular units up to 45 tons. Four such derricks were used up to the 90th floor and erection from the 90th to 110th floors took place by means of a guy derrick which was added at the 90th floor. Each stiffleg derrick was supported on a 65 ft. high derrick tower which was located in a megamodular area. The derricks were lifted to the next position after two tiers of steel construction. The lifting was accomplished by use of slow speed electric winches operating from cathead beams supported on the highest level of the steel just erected. The overall steel erection was about 8 stories a month and the total erection was anticipated to take 15 months.

Conclusions

A short discussion of the evolution of the bundled tube concept used for the Sears Tower Building is presented in this report. Although by sheer size and proportions of the building, a number of sophisticated analysis and design methods were developed, the system itself was kept relatively simple to design and construct. In the evolution of construction systems for buildings in this century Sears Tower underlines a distinct point of progressive departure from the previous tradition of tall buildings. By using framed tubes in a bundled form, a wide variety of proportions and massing can now be achieved both for functional as well as visual impacts and yet avoid any significant premium for height. The bundled tube concept perhaps opens up a new vocabulary in structural-architectural design of tall buildings for the future.

CONCRETE YESTERDAY AND TODAY – and – ACI-318

by Stanley E. Teixeira
H. J. Brunnier Associates, San Francisco

When comparing the concrete of yesterday and today, it is impossible to draw a sharp line that chronologically separates yesterday from today. I believe that a logical comparison can be made by reviewing the more significant developments in concrete during the past several years, then commenting on the advantages, as well as, in some cases, the adversities that these developments have also brought to concrete construction.

When I speak of the concrete of yesterday, I am speaking of my personal experience that goes back only a modest 35 years.

As we review concrete over that time period let's glance at concrete in four related areas: materials, concrete production, structure types and design codes.

We have seen the development of newer materials for concrete, although we see in concrete today the same adverse characteristics, like undue shrinkage and creep; but, in general, it is certainly a more sophisticated product.

We have seen various types of cements developed for specific uses - like high early strength cements, low heat cements, sulphate resistant cements for sea water resistance and in more recent years the development of cement with expansive properties to counteract the shrinkage. I have experienced use of this particular cement in slabs on grade resulting in an almost completely crack-free floor, but at the present time availability is questionable. Slabs with expansive cement presented some uncommon problems in finishing.

During the past 3 or 4 decades we have seen established the reliability of higher strength concretes. Whereas in the 1940's concrete with compressive strengths of 2,500 psi was the usual norm of structural concrete, now strengths of 5,000 psi and even higher are not uncommon, especially with the quality control normally common to plant case concrete, and high strengths now are achieved with good reliability.

There have been developed various admixtures for specific purposes; some to accelerate the set of concrete, some to retard the set, and others such as air entraining agents to increase the workability of the mix. The use of air entraining agents now gives concrete a remarkably higher resistance to the effects of freezing and thawing cycles.

During the past few decades we have seen the emergence of lightweight concrete as a workable and reliable structural concrete.

Concrete using lightweight aggregates was used as far back as World War I in the construction of concrete ships for the Emergency Fleet, a project by the way, in which the late "Bru" Brunnier played a leading role; but it wasn't until after World War II that lightweight concrete began to gain a general acceptance in building construction. The observations of some of the hulks of the concrete ships of World War I fully established confidence in the durability and weathering resistance of lightweight concrete.

Prior to extensive testing in the 1950's there was misconception that because lightweight concrete has a lower modulus of elasticity than stone concrete, engineers tended to associate this lower modulus with higher creep and shrinkage characteristics.

Eventually tests proved that lightweight concrete, particularly using the expanded shale aggregate available to us in Northern California, has excellent shrinkage and creep characteristics, but, on the other hand, it was found to have a lower tensile strength.

The lower tensile strength of lightweight concrete was recognized in the shear and diagonal tension aspects of the 1963 ACI Code.

Today, reinforcing bars remain somewhat the same unsophisticated product that they were 35 years ago; that is, they are a product without a maximum specified yield point and with indefinite chemical properties, thus it still is a steel of uncertain welding characteristics. There have been, however, changes in rebar specifications to account for increases in minimum yield strengths. While some years ago the prevailing minimum specified yield strengths was 33,000 psi, bars in common use today have minimum specified yield strengths of 40,000 psi and 60,000 psi; and in limited use, even up to 75,000 psi.

In the middle 60's when the ductile concrete provisions were propounded, the SEAOC Seismology Committees believed that in the ductile concrete requirements there was a need for a reinforcing bar with certain definite properties, that is, a bar with properties that approached A-36 structural steel, both in ductility and in weldability.

About 8 years ago ASTM was presented with the challenge by SEAOC to produce a specification for such a rebar. After countless meetings among SEAOC, ASTM and the steel industry, there finally recently emerged ASTM A-706.

The specification is now available; whether the bar itself is available is a matter that should be ascertained before embarking on a design that is based on A-706. In any case, the requirement is yet to be introduced into UBC. SEAOC is now working on that aspect.

This brings up the technique of "ductile concrete", a technique developed in the 1960's to make moment resisting concrete frames effective to resist earthquake stresses beyond the elastic range primarily through a complex system of reinforcing. Since being introduced to building codes nearly 10 years ago ductile concrete moment resisting frames have seen only limited use. A common criticism of the framing system of ductile concrete is that the reinforcing requirements, particularly at column-beam joints, border on the impractical in design, detailing and field execution.

Adverse deflections resulting from creep have plagued many flat slab floors and long span slabs and beams. Minimum depth-span ratios as required by ACI 318 can result in adverse long term deflections for flat slabs.

As a result of some experiences with adverse deflections in flat slab floors, I can offer some rules of thumb to minimize deflections. The first rule is to recognize that these minimum depth-span ratios may lead to deflection troubles; and secondly, in the pattern of reinforcement shown in ACI 318 for flat slabs no bottom bars are shown continuous across column centerlines and no top bars are shown in the middle of the spans of column strips and middle strips. While this pattern can theoretically satisfy the moment requirements, past experiences have shown that running a certain percentage of bottom bars across the supports and running a certain percentage of top bars continuously across the top of the middle and column strips will contribute very effectively to increasing the stiffness of the slabs thus reducing the deflections, especially in reducing deflections due to long term creep. As higher stresses come into use for tensile steel, it becomes even more important to use counter measures to control deflections.

It is disturbing to me that while ACI 318 makes a point of requiring analyses that recognize unbalanced live loads, no mention is ever made of this in connection with flat slabs, especially when considering that flat slab construction is a frequent choice of framing for heavy live loads.

The past 35 years have brought the development of many innovative techniques in reinforced concrete. Precast concrete, for instance, prestressed concrete, and thin shell structures.

As innovative as the techniques may be, some have been accompanied by a myriad of problems.

Precast concrete panels have become a popular method for cladding the exterior of high rise moment resisting frame buildings.

The drift of the frame under seismic forces requires that the rigid concrete panels be connected to the frame in a manner that will permit the frame to drift sideways without shearing off the panels. This requires some very ingenious connection details and careful execution in the field. Just how successful the design professions as a whole, have been at accomplishing this, we may some day find out.

Prestressed concrete has made possible some spectacular structures, but some prestressed, precast structures have performed unsatisfactorily because of an apparent lack of recognition of the magnified effects of shrinkage, creep and temperature changes of the prestressed elements on other more rigid elements of the completed structure.

In view of lessons learned yesterday from this general type of structure, we must also recognize today in designing, that buildings of this type generally possess an inherent lack of continuity that adversely affects earthquake resistance unless all details are given unusual attention in the design, both in design and construction.

on concrete of tomorrow. I ask, on behalf of all practicing structural engineers, that our next two speakers use their influence with ACI to simplify design code provision, so that the structural engineer can devote more time and direct more effort toward the significant aspects of engineering; i.e.; creativity, ingenuity, and a thorough workable presentation of his design; let the structural engineer be liberated from the shackles of needlessly complex mathematical enigmas.

IMPROVED SEISMIC DESIGN - INFLUENCE OF CURRENT STRUCTURAL CONCRETE RESEARCH

by

W. G. Corley*

INTRODUCTION

In the early part of this century, structural design of buildings was accomplished with rather simple mathematics and easy to understand structural configurations. In many cases, acceptance of a building system or even a single building itself was based on load tests. Though lateral loads due to wind were considered, earthquake was either ignored or considered in a simple way.

Within the last 20 to 30 years, engineering sophistication has greatly advanced. Computers have become available to carry out extremely complex analysis. In the experimental field, the use of electronic sensors and recorders has made it possible to greatly improve our understanding of structural performance.

In recent years, model building codes, and particularly those dealing with seismic design, have reflected the progress and technology with evermore complex requirements. In many

*Director, Engineering Development Department, Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

cases, engineers have found the requirements time consuming to apply and sometimes confusing.

PREVIOUS RESEARCH

About 10 years ago, the Portland Cement Association began a major experimental investigation of structural walls for lateral load resistance in buildings. When these tests started, the best experimental data available was that produced by Professor Benjamin at Stanford University(1-5). His work, plus some tests done at MIT(6) and in Japan(7), formed the basis for the limited design information then available.

The first experimental investigation started at the Portland Cement Association was a rather basic study to determine the shear strength of thin deep members having proportions similar to those of structural walls. Only monotonic loads were applied(8). As seen in Fig. 1, the walls were tested on their side to permit the use of very large specimens. Some of the information obtained from these tests was described at the 1969 Structural Engineers Association of California Annual Meeting(9).

In the next phase of the PCA experimental work, very short walls were tested(11). These walls, shown in Fig. 2, had a horizontal length equal to or less than their height. Results of these tests have just been published in a Portland Cement Association Research and Development Bulletin RD043(12).

Tests carried out in the first three phases of the Portland Cement Association work have contributed to a basic understanding of resistance of structural walls to lateral loads.

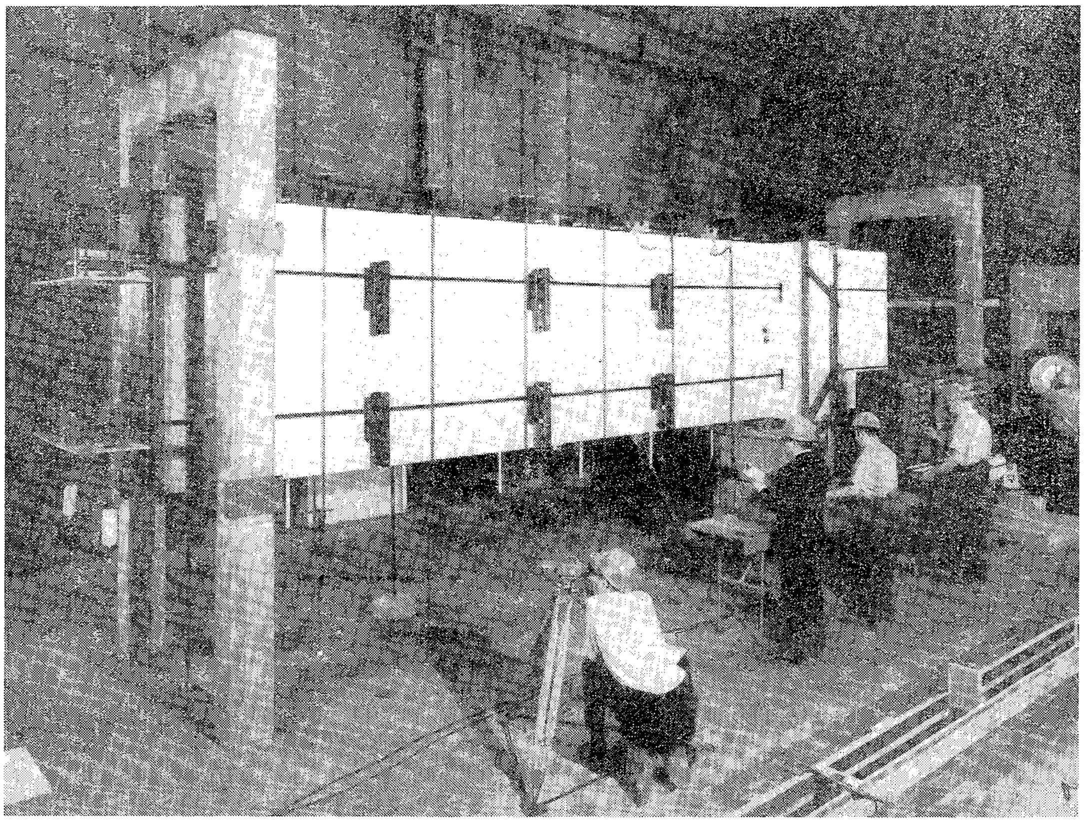


Fig. 1 Structural Wall Subjected to Monotonic Load

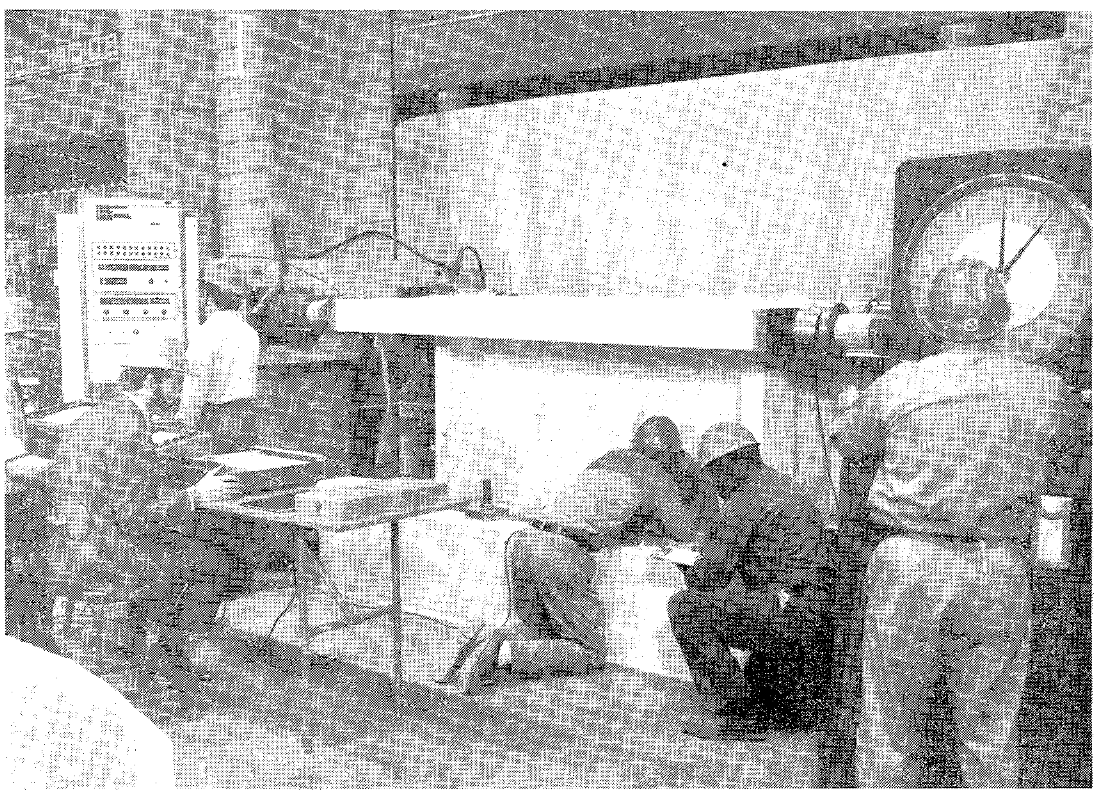


Fig. 2 Low Rise Structural Wall

Some of these results have been used to develop building code requirements. Both the UBC and ACI 318 have incorporated some of the results of these tests[9,10].

CURRENT TEST PROGRAM

Currently, the Portland Cement Association is carrying out a major analytical and experimental investigation to develop improved design criteria for reinforced concrete walls used as lateral bracing in earthquake resistant buildings. This project is sponsored jointly by the National Science Foundation and the Portland Cement Association.

The experimental program is divided into three parts. Part 1 includes tests of isolated walls, Part 2 includes tests of wall systems, Part 3 includes tests of coupling beams and of confined concrete. This paper discusses only Part 1, Tests on Isolated Walls. The test program and some of the results obtained to date are described.

One of the goals for the isolated wall test program is to determine the load and deformation capacity of walls. This portion of the investigation is intended to find a suitable way of determining the load deformation history under repeated loads.

Emphasis in the investigation is concentrated on determination of ductility. From this part of the program, the energy dissipation capability of the structures and their total deflection or rotational capacity are being determined. In addition to deformation, strength of the walls both in flexure and shear are being measured.

The primary goal in this program is to develop design criteria that will provide walls with adequate strength and ductility to resist the design earthquake. Part 1 of the experimental program uses relatively large isolated structural walls subjected to reversing loads. As shown in Fig. 3, the wall is 15-ft high and has a horizontal length of 6-ft 3-in. Walls are 4-in. thick to accommodate two layers of reinforcement. Reversing loads are applied to the specimen through a top slab. Post-tensioning forces clamp the base block to the laboratory floor.

Variables considered in the first phase of the program include shape of the wall cross section, percentage of longitudinal reinforcement, and amount of confinement reinforcement in the boundary elements.

Cross sections of the walls that are being evaluated include rectangular, barbell, and flanged shapes. The barbell section represents a wall with integral columns at each end. In this test program, the columns are 12-in. square. The flanged section represents a system of inner connecting walls. Flanges are 36-in. wide and 4-in. thick.

Main vertical flexural reinforcement in the specimens is either 1% or 4% of the area of the boundary element. These percentages were chosen to give section moment capacities corresponding to both low and high nominal shear stresses on the web of the test specimen. Vertical web reinforcement equal to 0.25% of the gross concrete area is provided. This is minimum reinforcement permitted by building codes[9,10].

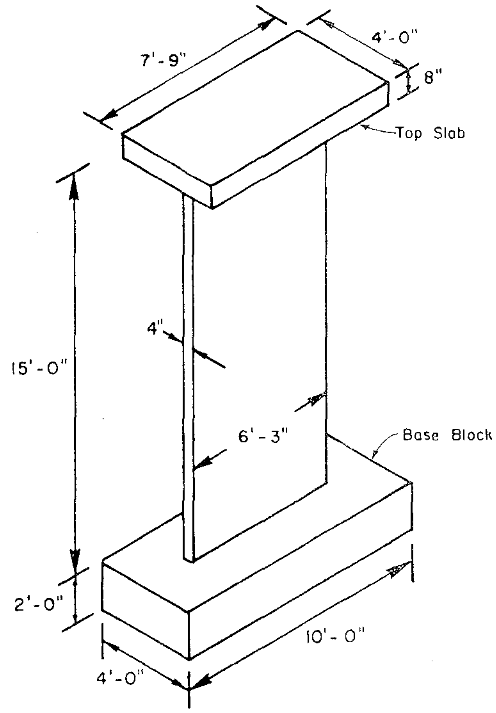


Fig. 3 Nominal Dimensions of Test Specimen With Rectangular Cross Section (1 in. = 25.4mm)

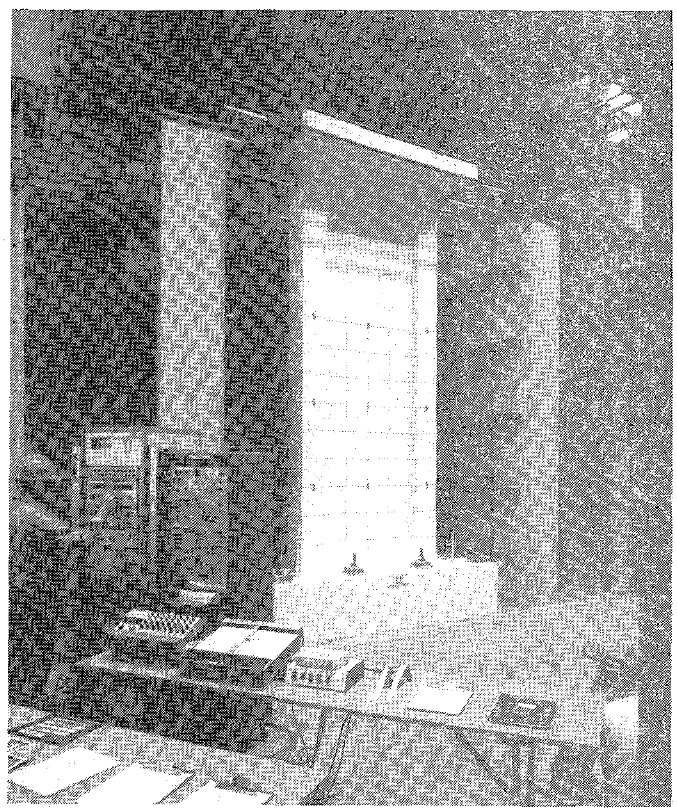


Fig. 4 Isolated Structural Wall

Where confinement hoops are provided, they are designed according to Appendix A of the 1971 ACI Building Code[10]. Hoops are provided only in the hinging region of the test specimen, normally the lower 6 ft. Walls are fabricated in a vertical position with six 3-ft lifts.

After a specimen has been completed, it is painted white and a 1-ft grid is marked on the surface as shown in Fig. 4. Hydraulic rams are used to apply the static reversing loads to the top of the slab.

Independent reference planes on each side of the wall are used to support instrumentation. During the test, measurements are made to determine applied loads, deflections, rotations, shear distortions, steel strains, and slip at construction joints. In addition, a complete photographic record including time lapse motion pictures is obtained.

Fully reversed loading cycles following the predetermined pattern shown in Fig. 5 are applied to each specimen. Prior to first yield, loading is controlled by increments of force. Beyond yield, increased increments of deflection are applied till the wall has been destroyed. Note that three complete cycles are applied at each new load or deflection increment.

TEST RESULTS

During each test, measured load versus deflection relationships are recorded. An envelope or boundary for these curves can be obtained by passing a curve through the peaks of the load deflection cycles. As shown in Fig. 6, load is plotted

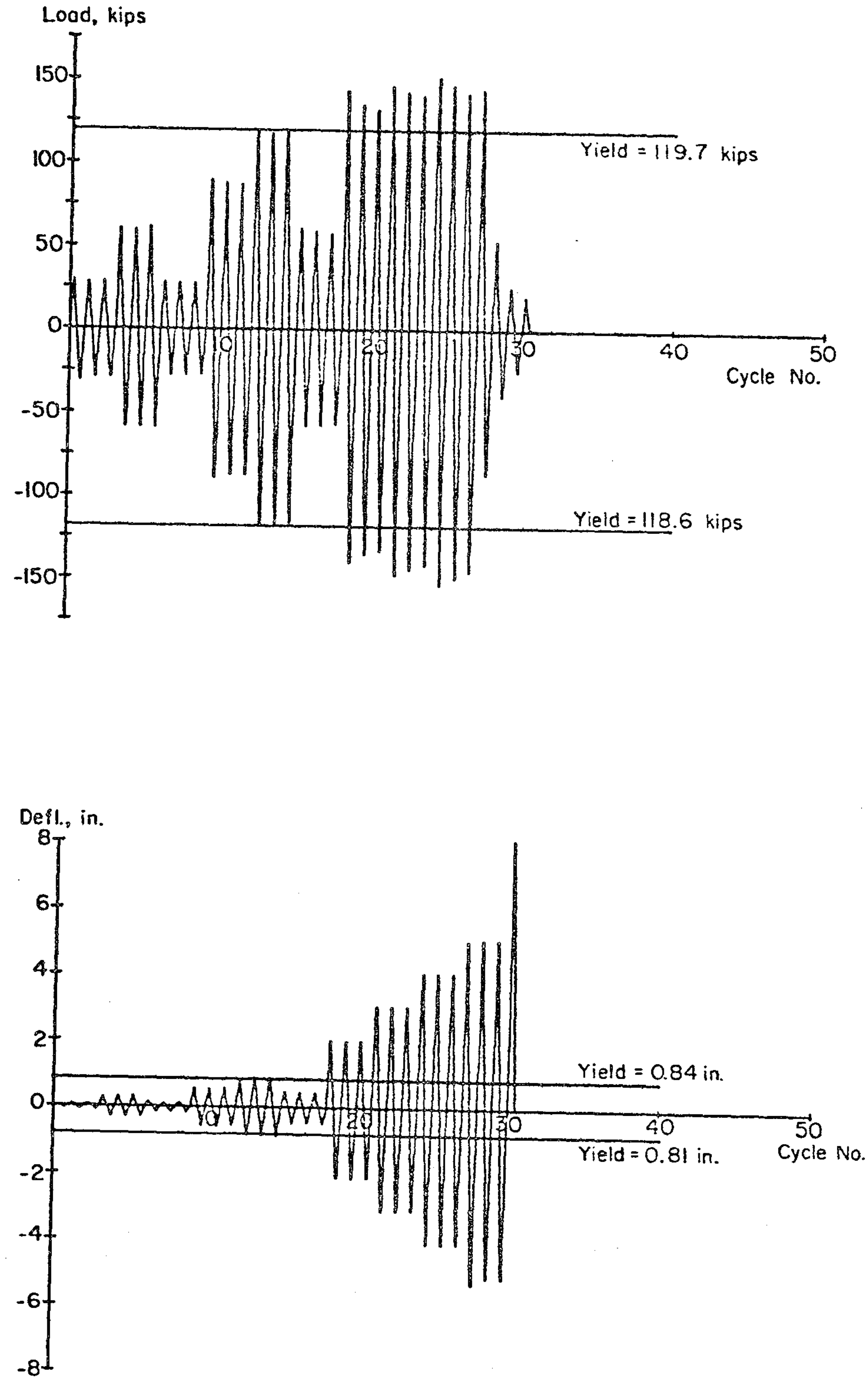


Fig. 5 Loading History for Specimen B2

on the vertical scale and deflection on the horizontal scale. Load deflection envelopes for the first five test specimens are shown in Fig. 7. The load scale is in terms of nominal shear stress divided by $\sqrt{f'_c}$. Deflection is that measured at the top of the wall.

Specimens indicated as F1 and B2 were reinforced so that loading produced relatively high nominal shear stress. These tests ended by crushing of the web.

Specimens R1, B1, and B3 were reinforced so that loading produced relatively low nominal shear stress. These tests ended with fracture of reinforcing bars caused by alternate tensile yielding and compressive buckling of the main flexural reinforcement.

SUMMARY

For walls with strength controlled by flexure, confinement hoops improved ductility but not strength. For walls governed by web crushing, confinement hoops improved strength, but not ductility. It should be noted that significant ductility was obtained even without confinement.

The goal of the experimental and analytical work in this investigation is to use the results along with those from the University of California, the University of Illinois and other institutions currently working on structural walls, and develop a design procedure that will take full advantage of the favorable performance provided by structural walls. These design

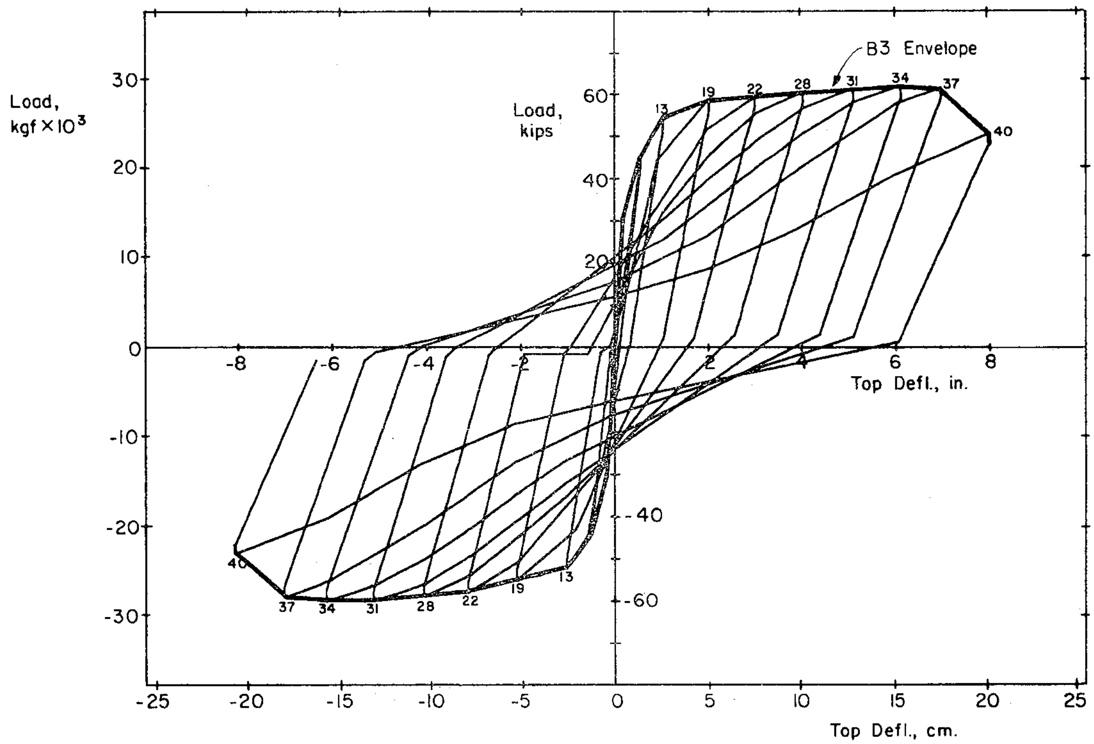


Fig. 6 Load Versus Top Deflection for Specimen B3

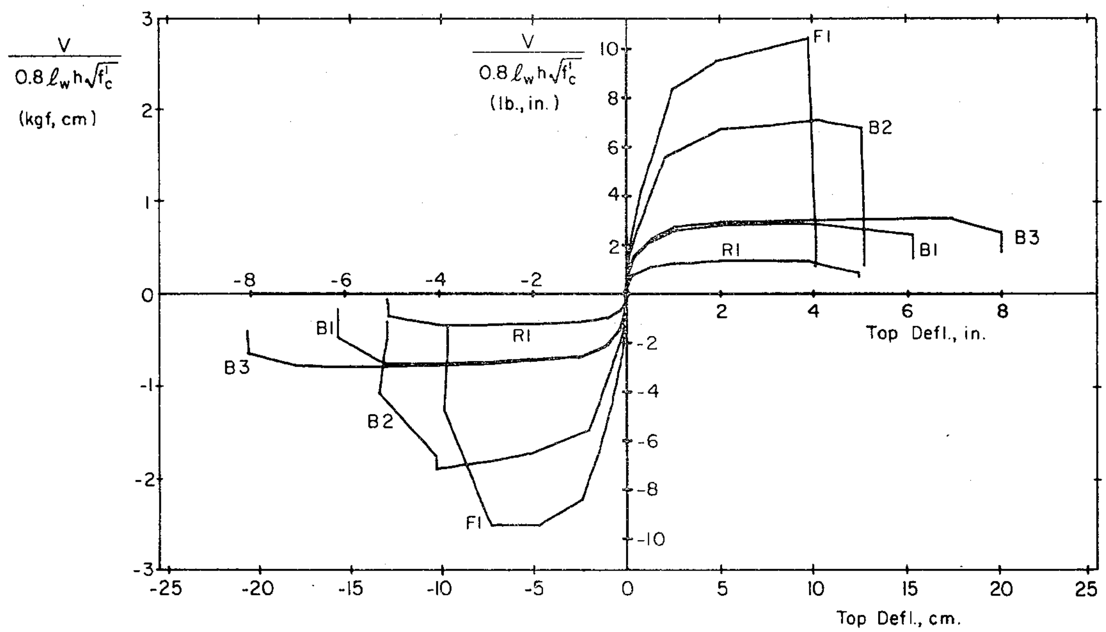


Fig. 7 Load Versus Deflection Envelopes

procedures should provide structures that are economical, have excellent resistance to lateral forces caused by earthquakes, and, an important consideration, provide excellent damage control during an earthquake.

References

1. Williams, H.A., and Benjamin, J.R., Investigation of Shear Walls, Part3-Experimental and Mathematical Studies of the Behavior of Plain and Reinforced Concrete Walled Bents Under Static Shear Loading, Department of Civil Engineering, Stanford University, Stanford, Calif., July 1, 1953.

2. Benjamin, Jack R., and Williams, H. A., Investigation of Shear Walls, Part 6-Continued Experimental and Mathematical Studies of Reinforced Concrete Walled Bents Under Static Shear Loading, Department of Civil Engineering, Stanford University, Stanford, Calif., August 1, 1954.

3. Stivers, R.H., Benjamin, J.R., and Williams, H.A., Investigation of Shear Walls, Part 8-Stresses and Deflections in Reinforced Concrete Shear Walls Containing Rectangular Openings, Department of Civil Engineering, Stanford University, Stanford, Calif., August 1954.

4. Benjamin, Jack R., and Williams, Harry A., Investigation of Shear Walls, Part 9-Continued Experimental and Mathematical Studies of Reinforced Concrete Walled Bents Under Static Shear Loading, Department of Civil Engineering, Stanford University, Stanford, Calif., September 1, 1955.

5. Benjamin, Jack R. and Williams, Harry A. Investigation of Shear Walls Part 12-Studies of Reinforced Concrete Shear Wall Assemblies, Department of Civil Engineering, Stanford University, Stanford, Calif., December 1956.

6. Galletly, G. D., Behavior of Reinforced Concrete Shear Walls Under Static Load, MIT Department of Civil and Sanitary Engineering, Cambridge, Mass., August 1952.

7. Antebi, J., Utku, S., and Hansen, R. J. The Response of Shear Walls to Dynamic Loads, MIT Department of Civil and Sanitary Engineering, Cambridge, Mass., August 1960

8. Cardenas, A. E., and Magura, D. D. "Strength of High-Rise Shear Walls-Rectangular Cross Sections," Response of Multistory Concrete Structures to Lateral Forces, American Concrete Institute Publication SP-36, Detroit, Mich., 1973, pp. 119-150.

9. Corley, W. Gene, "1969 Portland Cement Association Research on Shear Walls," Proceedings, 38th Annual Meeting of the Structural Engineers of California.

10. Corley, W. Gene, "Ductile Shear Walls in Multi-Story Buildings - Laboratory Tests," Proceedings, 42nd Annual Convention of SEAOC, October 1973.

11. Barda, F., "Shear Strength of Low-Rise Walls with Boundary Elements," Ph.D. Thesis, Lehigh University, Bethlehem, Pa., 1972.

12. Barda, F., Hanson, J.M., and Corley, W. G., "Shear Strength of Low-Rise Walls with Boundary Elements, Portland Cement Association, Research and Development Bulletin, RD043,01D, 1976.

13. Uniform Building Code, 1976 Edition, International Conference of Building Officials, Whittier, Calif., 1976, 728 pp.

14. ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, Detroit, Mich., February 1971, 78 pp.

Among the speakers for the Friday morning Technical Session: Elmer Botsai; John A. Martin, President of SEAOSC, presiding; Howard Eberhart; and John A. Blume......

......visiting engineer from Guatemala City, Mario Roberto Monterroso V.; Technical Program Chairman Stephen E. Johnston; and Thomas D. Wosser.

ENGINEERING EDUCATION - PAST, PRESENT AND FUTURE

Howard D. Eberhart
Professor of Civil Engineering, Emeritus
University of California, Berkeley

Nearly everyone who has completed an educational program becomes an expert on the good and, particularly, the short-comings of that program. This is especially true of engineers who enter the profession and attempt to apply what they have learned to making a living and performing a service. If they have difficulty it could be blamed on the curriculum, the course content, or poor instructors. If successful, it was due to their own ability, initiative, and sound business judgment. These two extremes may be exaggerated, but various elements of both may enter into each individuals feeling about professional preparation.

The fact that engineering education has been reviewed, analyzed, discussed and argued about so much for so many years is a result of: 1) members of the profession thinking they know what the short-comings are and sincerely wanting to help bring about improvements for future generations, and 2) members of engineering faculties continually seeking ways to improve the educational process and the resulting product. As a result there is almost continual change being made in engineering programs--a situation that can be expected to continue. Evaluation of the result of innovation or other changes that are made is very difficult and generally subject to the personal opinion of the evaluator.

Let's take a look at what has happened to Civil Engineering education in the last forty years. At the beginning of that period, the B.S. program was very rigid, with few electives and very little consideration of the humanities. There was broad coverage of the technology of the day with great attention to details, including ink drawings on vellum with accurate location of innumerable rivet heads. Courses were demanding and more generally were required than for other programs of study. There was very little graduate work, possibly because the undergraduate program was adequate preparation for earning a salary on the first job out of school.

As more and more research took place leading to developments in materials, methods of analysis, and design, courses were modified by eliminating some of the "make work" parts to provide time for new developments. Time devoted to drafting was reduced, as was that for surveying. Then came the Grinter Report and other critical reviews that emphasized the need for socio-humanistic courses, and even some essential technical courses had to give way. The baccalaureate degree then became scarcely enough for entry into the profession at a reasonable level and graduate work began to grow rapidly. The undergraduate workload was reduced to the same unit requirement as in other fields, the number of required courses was reduced, increasing the flexibility and giving the student a greater choice in developing a program to fit his particular needs. Encouragement was given to those students who demonstrated the ability and motivation to continue their formal training through at least the master's level with some degree of specialization.

So we have come, in big jumps, to the present time with a considerable amount of new technology not available 40 or even 30 years ago, with very involved and sophisticated material being offered in graduate courses--even some in undergraduate courses--that was not available fifteen or even ten years ago. Not only is a large number of students in Structural Engineering taking master degree programs of one or two years, but many are continuing toward the Ph.D. or the professional D.Eng.

It has been claimed that there is a lack of communication between the professional engineer and the academician. Recent national conferences on engineering education organized by ASCE (March 1974) and the annual meetings of ASEE have demonstrated the need for better understanding of all concerned.

Statements are made that "the trend in engineering education seems to be away from the needs of the practicing profession". Many changes have been made by faculty groups because, in their opinion, continuing developments in analysis, design and materials are needed by the profession. Some faculty members go so far as to say that some members of the profession do not use the new technology because they don't understand it and have made no attempt to study and learn it. The faculty groups could be wrong, but it is evident that greater mutual understanding is needed.

Also, it has been stated that "emphasis in our most highly regarded engineering schools has been directed toward research, and that this is a detriment to their main function, which is to teach." Research in universities has been an important activity for a good many years and accounts for a great deal of the technological progress of the past. This must continue and at an accelerated pace if one of the most important functions of first-rate education is to be fulfilled. In itself it is good and not bad. It is a detrimemt to the "main function" only when the teaching side of the picture is slighted, and that is the responsibility of each individual faculty member who appears before a class. No course outline, curriculum, plan, or orders from the top can bring out the desired qualities of a good instructor if there is no interest in teaching. But certainly, a general climate where good teaching is encouraged, particularly at the undergraduate level, should prevail and be greatly expanded over what it is today in many institutions.

Orientation toward professional practice is part of the whole picture and is an area where the profession could be especially helpful. Faculty members with professional experiences who are familiar with current problems in practice have a real opportunity and a duty to inject the flavor of practical application into each course they teach. But greater emphasis on the teaching function should not result in a reduction of research activity. Certainly no good graduate program could exist without research and a faculty active in research, and there are positive advantages to the undergraduate program as well. Motivation can be developed, creativity encouraged, curiosity aroused, and a recognition arrived at that new developments are continuously being made in a dynamic profession and requiring continuing education. In the same way that a professor's consulting keeps him up to date in the profession, so research keeps him up to date in his teaching.

The last few years have seen the introduction and use of computers and sophisticated methods that they have made possible. With a library full of prepared programs it is conceivable that the technical work of the structural engineer of the future could be reduced to selecting the right program and feeding the known conditions into the machine. On this basis much of the basic course material would not be needed and major revisions of educational programs could result. Certainly the man-hours of analysis have been reduced which makes possible more attention to obtaining optimum designs.

However, the computer does not lessen the requirement that basic principles and fundamental relationships be well understood and applied. Numerical operations can be reduced but necessary decisions made by design professionals are based on many more considerations. The advent of computers because of the time saved makes possible the involvement of the engineer to a greater extent in multi-disciplinary teams that consider all aspects of major projects. Active participation in public affairs, planning in all its varied applications, and project management are areas where engineers, working with other design professionls, could render great service. Some interaction with these disciplines in the educational process should receive greater consideration in the future.

What should be the function of an engineering educational program? Is it to train individuals for employment by consulting offices or engineering organizations? Or should it be to educate people to be useful to society with the knowledge, understanding, and competence in an area to make contributions to improve or to produce. Should we be bound by current ways of doing things or should we encourage independent thinking that may result in creative developments of value?

Isn't it true that very little of what you learned in school is used in your present work? You have developed, with time and experience, with study and self development. So you didn't learn about computers in school but dug into the subject yourself and found it not too difficult. Now every freshman engineering student has to take a course in computer science. Is this progress or shouldn't it be treated like the slide rule with each student learning on his own?

Looking back on your formal educational experience, has it made much difference what courses you took as long as you were motivated and challenged to do your best and really had to put out? Dividing knowledge into little units for transfer from instructor or books to students does not necessarily produce an educated person. What is important is development of the ability to think independently: to define a problem or comprehend a situation, to determine what information is pertinent, to locate it, understand it, apply it, evaluate the result and make a decision. A curriculum and its courses is only a start. An instructor can have a great influence, not in teaching a student to think but in forcing him to learn to think for himself. Certainly there have been individuals that developed this ability by themselves. A great deal can be said about variation in individual abilities, particularly in the field of learning. Some need to be led and some pushed. A few don't respond to any kind of treatment and drop out. But not many drop by the wayside that have an interest and are motivated to become engineers. It can be said that the prime function of an educational program is to teach the student to teach himself those things that have not been taught in school.

So what can be predicted for the future development of engineering education, particularly for the structural engineer?

> There will continue to be change as in the past and as new developments occur. Changes should be evolutionary and not revolutionary. There should be a continuing increase in emphasis on societal and environmental considerations, and on participation in interdisciplinary activities. Ideally, a sense of professionalism and ethics should be impressed. This must come from attitudes of instructors and from the conduct of individuals who make up the profession.
>
> Emphasis must be on the "non-changing" fundamentals - statics, strength of materials, basic structural analysis, behavior of reinforced concrete and steel structures -- rather than the details of codes, methods, etc. that become outdated.
>
> The future should see more emphasis on conceptual design and the decision making process, undoubtedly aided by greater interaction with computer capabilities.
>
> The ability to model structures for analysis in a realistic and correct way will remain a basic and important function.

Computers cannot replace judgment, but if used properly can be used as real physical models have been in the past to develop a better understanding of structural behavior.

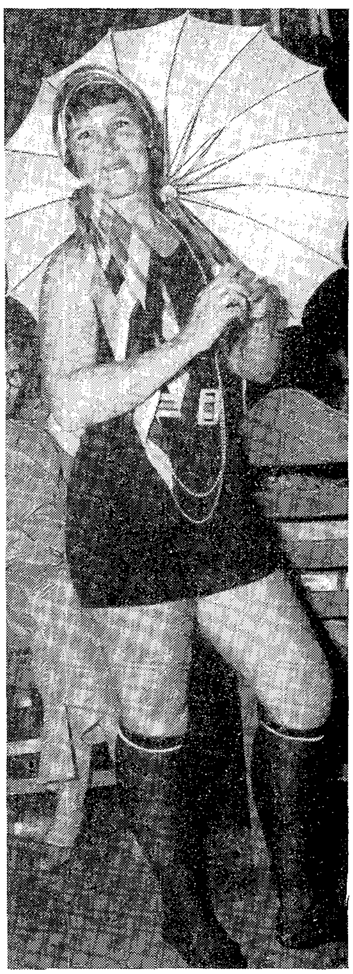
Laboratory experience working with real engineering materials and structures must continue to be an important part of an educational program.

Remember When

Meanwhile, as Ladies Social Chairperson Evelyn Daniels watched admiringly, some of SEAONC's classiest beauties modeled historic gowns and other attire in tune with the Convention Theme – Remember When – at the Friday Luncheon. A few of the models are shown here.

Jan Wosser

Jo Fratessa



Esther Koopman

Jimmie Wildman

Ann Preece

Carol Singer

Three of the foxiest flappers you'll find anywhere.

ON FORCES THAT INFLUENCE STRUCTURAL ENGINEERING PRACTICE

by

John A. Blume, President
URS/John A. Blume & Associates, Engineers, San Francisco

There was a time when I could have responded to the title of this presentation by simply noting axial forces, perhaps resolved into the X, Y and Z directions, together with moments and shears. Although such items are basic in the practice of structural engineering, I am afraid they alone would no longer be responsive to this assignment.

All sorts of forces influence the practice of structural engineering today. The engineer who has a sound theoretical background, who has had a great deal of technical experience, and who keeps up to date in his field has a start but he must also contend with many other factors and influences to practice structural engineering successfully. Whether this trend is good or bad I shall leave for your decision. Many of these things are brought about by changes in society in which structural engineering is merely being swept along, many by engineers themselves and their competitors, and many by the clients.

All I can hope to do here is to list and comment briefly about some of the many forces that influence practice, and to compare "now" with "yesterday" and perhaps guess as to "tomorrow". For this purpose I have divided the forces that influence into three categories: (I) Those brought about mainly by society (in which engineers unfortunately exercise a relatively small influence); (II) those caused by changes in technology; and (III) those caused by engineers, competitors, and/or clients. In several cases, the division is difficult, and perhaps some items belong in two or three categories. There is one thing in common with these various forces -- they take time and money to cope with and thus they detract from your technical activities or add to your costs if you have others do the coping for you.

We all recognize that the practice of structural engineering takes many forms which are generally responsive to the types of clients. Some engineers do almost all of their work for architects, some do little if any work for architects, and there are those who have a mixed practice in that regard. Some do essentially all work as subcontractors in the design team, some joint venture, some are prime contractors, and there are combinations. Some do all government work, some do none, many do some. In this discussion I shall imagine a hypothetical composite engineer whose work has any or all of these aspects at various times.

Tables I, II and III provide listings of the three categories of forces outlined above. It is clear that there is much more to private structural practice than determining bending moments and shears and selecting the members to resist them. A good engineering firm today not only has to have a high level of technical competency and seasoned experience but operate successfully in view of the factors in Tables I, II and III, as well as others not listed.

TABLE I - SOCIETAL FORCES THAT INFLUENCE PRIVATE PRACTICE

Item	Yesterday	Today	Tomorrow(?)
Insurance, various kinds	Nominal	Increasing in cost	Look at the MD's!
Malpractice claims, awards	Almost none	Many, costly	Growing, serious
Inflation	Some	Bad, allow for in estimates	Hope rate doesn't increase, but it could
Fringe benefits, various	Nominal	Heavy, and growing	Will increase more
Decisions on suggested fees	None	Adverse	Bidding increasing
Safety regulations	State	Also Federal, complex, costly	Increase
Environmental impact	Informal	Reports, delays, costly	Increase
Energy and conservation	Informal	Important	Very important
Government research funds	Very little	Considerable	Increase
Military work	Considerable	Nominal	Needed
Work volume	Up and down	Fair	More needed
Taxes	Bad	Worse	Socialism?
Media power (TV)	Considerable	Great	A greater influence
Civil Service	California	Increasing	Who pays the bills?
Foreign competition	Minor	Strong	Could be serious
Public hearings, boards, commissions	Some	Many	More
License requirements	Some	More	More
Limits on fees	Yes	Some, but out of date	Should be removed or be raised
Quality assurance	Informal	Often formal	Probably more formal

TABLE II - TECHNOLOGICAL FORCES THAT INFLUENCE PRIVATE PRACTICE

Item	Yesterday	Today	Tomorrow(?)
High speed computers	None	Good, fast, costly	Faster, smaller, more powerful
Electronic calculators	None	Excellent	Can't be much better
Software services	None	Some good, some dangerous in wrong hands	Will stabilize
University research	Nominal	Increased, much output	More detail
State of technology	Basics known	Advancing	More science/less art
Design	Art/mechanics	Art/science	Science/art
Drafting	Hand	Hand/machine	Machine
Para-engineers	Very few	Few	More
Technology transfer	In some offices	Seminars/some offices	Seminars/TV
Duplication procedures	Ink on linen, pencil, blue-prints	Copies to everyone	Better
Filing and retrieval	Cumbersome	Fair, but costly	Better, less costly
Technical papers	Few, most good	Lots; many rein-venting the wheel with new name, or in more detail	More
Education	Good	Better	Better yet
Seminars, meetings	Few	Many	Many
TV courses	None	Few	More
Nuclear energy	None	Here, advancing the state of the art	Hopefully stabilize in the public mind
Earthquakes	Public apathy; mostly static design	Public interest; funds for research; new techniques in design	Public action to reduce risk
M/E elements	Nominal	Increased amount	Energy conservation
Architectural elements	Dominate buildings	Integrate with structural and M/E	Earthquake and energy related

TABLE III - ENGINEER, COMPETITORS, AND/OR CLIENT INFLUENCES ON PRIVATE PRACTICE

Item	Yesterday	Today	Tomorrow(?)
Engineers work long & hard	All	Many	Some
Competition for work	Yes!	Yes!!	Yes!!!
Formal proposals	Few	Many, costly	More complex
Costs of proposals	Nominal	Considerable	Considerable
Research work	None	Some	Some
Client use of consultants	Little	Considerable	Considerable
Client use of panels	Little	Considerable	Considerable
Client does his own work	Some	More	More
Client merger problems	None	Some	Some
"Buzz words" in sales	None	Yes	Probably more
Specialization	Little	Considerable	More
The "retired" consultant	None	Several	Many
Ethics	Good	Fair	Probably poor
Costs of doing work	Low	High	Higher
Fees	Low	Low	Low
Codes and regulations	Some; general	Many; detailed	More, and more detailed
Available personnel	Few	Many	Too many?
Moonlighting	Very little	Some	Less
Advertising	None	Some	Some
Law, management, accounting	Minor	Considerable	More
Personnel raiding	Very little	Yes	Yes
Construction management	None	Some	Some
Turnkey operations	Some	Some	Some
Joint ventures	Few	Some	More
Multiple offices	Very few	Some	Some
Corporate practice	Few	Many	Many
Acquisitions & mergers	None	Some	Some
Professional societies	Few	Many	More
SEAOC	Yes!	Yes!!	Yes!!!

The fact that this isn't easy makes it a challenge and more interesting. There are easier ways to live that pay more and involve less responsibility but most engineers have a dedication that means more than ease or money. It is unfortunate that the professions that have great responsibilities are compensated poorly as compared to successful entertainers or professional athletes.

Although the notes in Tables I, II and III are necessarily brief, they do, I hope, convey a message. By no means would everyone agree with all the notations. Additional comments related to some of the items in Table III follow.

Engineers, along with everyone in the construction industry, seem to work hard and have much endurance and energy. Many have athletic backgrounds and bring that type of drive and competitive spirit to their work. This is great, but it does make the competition keen. However, I believe this particular influence is decreasing -- even engineers don't work as hard as they used to. Unfortunately, jobs seem to be fewer, perhaps due to a combination of less work for private practice and more engineers in private practice, and thus the overall competition for work is increasing.

Some clients or potential clients seem to do much work in-house by means of hiring specialized consultants as advisors, or perhaps engaging panels of experts to make the big decisions and share responsibility. Such clients seek experience, expertise, and responsibility-bearers but retain the "bread-and-butter" work that can be delegated. Needless to say, there are many such consultants available, especially with early retirements, longevity and with inflation affecting retirement funds. This trend has reduced the market for "private practice" considered in the context of overall design responsibility. Some of these consultants also hold other jobs, full or part time, but they do constitute a force. They will say they are in private practice, and they are, but it is of a type that didn't exist to such a degree "yesterday".

The merger of a client corporation with another corporation, or a client's acquisition, may result in loss of that client. The new company may have its own engineers either in-house or as long-time favorite consultants. This is perhaps most apt to happen to a western engineering firm because the eastern firm in a merger often has most financial weight.

A "buzz word" or term is a new name for something that may be new or -- much more likely -- has been around a long time. Perhaps a paper is written in which the buzz word is coined. Then the paper is referred to or perhaps an underlined reprint is included with a proposal. The prospective client is (a) impressed with the term, and (b) convinced that any proposer who doesn't use it is way behind the times. In close decisions, jobs have been won or lost on buzz words, as sad as that situation is. You can check your calculations and drawings in detail, as you have for years or decades, but unless you call this "quality assurance" or "QA", you may be out of phase with some proposal reviewers. (This discussion should not be confused with the formal, documented QA program for nuclear work.) In another case, an engineer was turned down for a job because he did not propose to solve his client's problem by "iteration" as did the successful applicant. The problem was not a mathematical one such as closing to a numerical value, but consisted simply of two or three alternative (trial) preliminary layouts or schemes, from which the best would be selected. We have all been doing this for decades, as has the engineer who (properly) didn't call the process "iteration".

Specialization is good up to a point but it is often overdone today, especially in sales or promotion. Not only are architects being called "theatre architects" or "office building architects" but structural engineers are being so typed. Unless you have done a dozen or more research laboratories, you aren't considered seriously for a research laboratory in spite of the fact that the laws of mechanics apply to research laboratories as well as to hospitals! One of these days we shall have different barbers for blondes, brunettes and redheads! This super specialization is, of course, a sign that there are too many architects and engineers for the work volume. I hope this situation changes. By way of contrast, when I first went to work as a structural designer for a large corporation during the depression, I was assigned the job of laying out and detailing the transition section between a boiler and a stack. I told my supervisor that it wasn't really "my thing". He told me right back that I was an engineer and should be able to do it. I did. Later, I also did pipelines, pumps, valves, metallurgy, and even some electrical work in addition to structural matters. They liked adaptability and operated on the theory that a good man should do a lot of things, and well, and fast, or else!

Conclusion

There are many new forces affecting engineering practice. There will be more. Things are changing, and they always will change. The changes may be considered in two categories -- those you have control over and those you can't control. With the former it is essential to do what can be done in a timely manner to prevent or alter the change if you don't like it. In this group there might be listed such things as ethics, costs, fees, unfair competition, professional practices, personnel policies, and quality of work.

For the changes that you can't stop or alter to your satisfaction, I refer you to the dinosaurs who have been termed the most powerful, best organized, most dynamic creatures on Earth. However, they lacked a very important ingredient -- they couldn't adapt to change -- so they are now being pumped into gas tanks!

THE GUATEMALA EARTHQUAKE OF FEBRUARY, 1976

Mario Roberto Monterroso V.
CORPA - Arquitectos-Ingenieros, Guatemala City

Mr. Chairman, distinguished guests, members, ladies and gentlemen,

On behalf of the people and Government of the Republic of Guatemala and myself, I would like to thank the SEAOC for their kind invitation to attend the 1976 Convention, and for the opportunity to say a few words along with my friend Tom Wosser on Guatemala and the recent February 4th. earthquake.

My colleague Ing. Juan José Hermosilla had been scheduled to make this presentation but an unfortunate accident prevented him from attending. What I will now read to you is the paper he had prepared for this occation, which coupled with Tom's comments and my own comments at the end, we hope will give you a fairly good idea of the experience we lived through, and the lessons we have learned which we feel the world can benefit from.

STRUCTURAL DAMAGE CAUSED BY THE 1976 GUATEMALAN EARTHQUAKE

by

Ing. Juan Jose Hermosilla

The Republic of Guatemala is a country located in Central America, bounded to the Northwest and North by Mexico, to the East by El Salvador, Honduras and the Atlantic Ocean, and to the South by the Pacific Ocean. It covers an area of approximately 43,000 square miles with a population of 5,000,000 inhabitants.

Guatemala City, the capital of the country, is located approximately 1,000 miles directly south of the City of New Orleans at an elevation of 5,000 feet, in a valley called "La Ermita."

The city was founded in 1773 when an earthquake destroyed Antigua, the previous capital.

The city in its actual location has experienced about six strong motions per century. The main two events, before the one of this year, were on January 3, 1918, as a group of shocks that started in December 25, 1917, which destroyed the city. The other was the event of October 6, 1942, with a magnitude of 7.75 and an epicentral distance of 96 miles.

The events of February 4 and 6, 1976, were caused by the local Motagua fault located in the

2. Buildings designed with seismic design codes prior to SEAOC 1966 that did not include the latest recommendations, confinement and other provisions.

   Some of these buildings performed badly as the lack of strength and ductility combined with interaction with architectural elements caused different types of structural failures.

3. Buildings designed with the latest seismic recommendations of strength and ductility as confinement, shear walls, etc., that in general performed satisfactorily.

In my office since we started designing structures, we have followed the ACI, SEAOC, and UBC specifications. Before the latest provisions on seismic designs were included in the SEAOC codes, we used the recommendations given by Messrs. Blume, Newmark and Corning in the book called "The Designs of Multistory Reinforced Concrete Buildings for Earthquake Motions", Chapters 6 and 7.

We have used different types of structural systems as follows:

1. Ductile moment resisting frames. In general, we tend to use large size columns and beam depths from 1/10 to 1/12 of the span. Due to the size of the columns, the percentage of reinforcement required is usually the minimum. These buildings were not taller than 8 stories and in general they performed well, in most cases the non-structural damage was small. There were other taller buildings with moment resisting frames, where the size of columns were as large as $3\frac{1}{2}$ feet by $3\frac{1}{2}$ feet, in general with large percentage of reinforcement. The floor system was either formed with regular beams, haunched beams or ribbed slabs in one direction. Some of these buildings performed structurally well with large damage to non-structural elements. Although the structural behavior of this building was satisfactory, we are introducing shear walls in new projects of this type.

2. Buildings structured with ductile moment resisting frames combined with shear walls that in general perform well.

   I personally think this should be the most accepted system in our country, as we have examples of their excellent behavior.

3. Buildings structured with columns, shear walls with a floor system formed by flat slabs that also in general perform satisfactorily. As we all know, the actual codes do not recommend the use of flat slabs in strong seismic zones.

   Unfortunately this seems to be a very efficient architectural solution, specially true in Guatemala City, where due to aeronautical regulations, the height of buildings is limited and the architects tend to obtain the maximum number of stories possible in a give height for economic reasons. I think that this system should be studied more thoroughly to see the possibility of considering it as an accepted practice.

4. Buildings structured with columns and flat slabs up to 4 stories that performed well; for taller buildings that require the use of flat slabs, we introduced shear walls in our structural system.

   Taller buildings in the city with this structural system performed generally bad with not only structural damage, but also with a considerable damage to non-structural elements.

   Today these buildings are being studied to see the possibility of introducing shear walls or other structural elements to transform them into better buildings.

5. Post tensioned buildings: There are only two post tensioned buildings in Guatemala.

   The criterion used in designing these buildings has been that vertical loads are taken by the cables or strands and to add reinforcing bars at the joints to take the seismic effects. The requirements for shear reinforcement in post-tensioned beams are not as severe as for plain reinforced concrete beams, but we use the specifications for regular reinforced concrete beams.

   a) Edificio Tívoli Plaza ductile moment resisting frame.

   b) Plaza del Sol ductile moment resisting frame with shear walls and beams with a span up to 100 feet.

It is important to point out that in Guatemala we usually have very good materials, as we can manufacture concrete with strength up to 6,000 psi. There are different criteria among designers and engineers on the strengths of concrete to be used in buildings. Personally, I favor the use of 5,000 psi in columns and shear walls, and 4,000 psi in the rest of the structure (foundations, slabs, beams, and stairs).

Before the price of steel went up about three years ago, we used 33,000 and 40,000 psi reinforcing bars in our structures, but due to the increase in cost, we decided to use higher strength steel and several of the actual buildings have 50,000 and 60,000 psi bars.

Another point that I want to emphasize is that our steel workers usually are well trained by practicing engineers teaching them all the requirements and specifications of overlaps, splices, embedment lengths, etc., and they become very skilled and can almost do the ideal details required by the codes. We can invest more time trying to accomplish these details, because the wages are low in comparison to the ones paid in the U. S. A.

I am aware that credit should be given to these skilled workers for the good behavior of the structures.

(Tomas Wosser gave his presentation and slide show).

(Ing. Hermosilla's conclusion 1 was read).

CONCLUSIONS

From all the previous observations there are some important facts to be noticed. Most of them have been pointed out in prior earthquakes, but our experience in the Guatemalan earthquake confirms most of these lessons.

1. As it has been repeatedly observed, the interaction of non-structural elements with the structure itself was extremely important.

   In certain cases it helped absorbing energy and in others it was an important factor in structural failures.

   It is necessary to give more attention to these non-structural elements not only to prevent structural damage, but to minimize the cost of repairs.

1\. ... It can be generalized that our heavy mesonry infill walls are responsible for creating accidental torsional problems, shear concentrations in "captive" columns, dead weight concentrations in undesirable locations such as cantilevers, etc. The non structural elements are accounting for by far the greatest portion of the cost of repairing the engineered buildings and residences.

2. Stiffness is not only a desirable quality in a structure, but it is necessary to protect the non-structural elements of a building. Similar to cases in other earthquakes, in Guatemala the cost of repairs in stiff and flexible structures was very different. According to our experience, the use of shear walls seems to be the most efficient way to stiffen structures.

   There were cases of good behavior of ductile moment resisting frames, but we are aware that in an earthquake of higher intensity even these buildings would need a stiffer structure.

3. Buildings designed according to the latest seismic recommendations including provisions to ductilize concrete behavied very well in contrast with older buildings.

2, 3. ... We have learned though, that the cost of introducing a shear wall system as a remedial process to improve a weakened older structure is also extremely expensive and the concept of evaluating the potential risk of not improving the structure, versus the prohibitive cost of substantially modifying it has to be seriously considered. A wise decision is not only the engineering adequate one but the overall adequate one, having placed recurrence probabilities, usable building life, economic and human costs as well as the owner's financial possibilities in the right perspective.

4. The importance of good structural concept and good detailing was shown again. Although, we are aware of the importance of an exact analysis, I think that in seismic areas, it is of greater importance the initial and final stages of a project, design, structuration and detailing.

4. ... In our nation the engineering profession is far older than the architectural profession, with my apologies to my friend Elmer Botsai, Vice-President of AIA, present at this session, yet in recent years, we have been relegated to the job of wizards or fools, expected to put together a project that often times began with an erroneous structural concept. In order to avoid the regrets of life and economic loss after an earthquake, we have learned that the three phases:

a) good structural concept,

b) good design and detailings, and,

c) careful construction supervision must go in hand throughout the stages of a project.

If any one is missing , then the professionals involved in the other two aspects have spent their time on something that has a much lesser chance of survival.

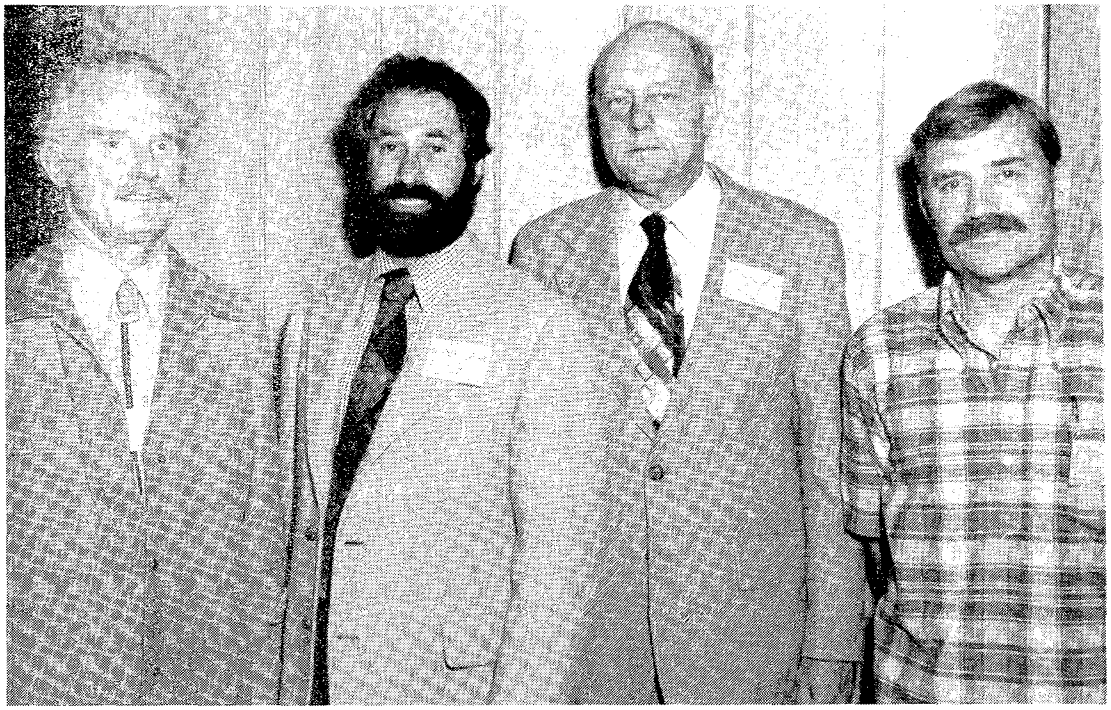
5. In our case in Guatemala, as it is true for many other countries, careful attention should be given to non-engineered constructions. Although, I think we can be satisfied with the behavior of most of our engineered projects. Provisions should be taken to apply the technic and knowledge that we have to improve the quality of non-engineered constructions.

5. ... In our country this involves the careful consideration of the possible cultural shock to the low income native people. Even though we are faced with the problem of providing shelter for a full one fifth of the population exposed to the subtropical climate and diseases, the process has to be adequately planned and carried out. This effort will tax our economic and technical resources for the next two generations, and consequently the assistance of the rest of the world is most welcome.

In keeping with the theme of Engineering Past, Present and Future, and your Bicentennial Celebration, I would like to give you an outsiders point of view. The technological and research capacity of the United States has placed it in a position of world leadership in all fields and you the Structural Engineers are certainly among the groups responsible for the preservation of this leadership. I would like to urge you to keep in mind the responsibility of leadership to the rest of the world in your advances in engineering. If such advances produce only complicated codes and computer programs that only few of you understand and the concept of sound engineering judgement is lost, then I feel your position of leadership will suffer and the less developed nations of the rest of the world will abandon your guidance.

My country's ambassadors have expressed our thanks to the world for the aid we received promptly after the nightmare of February 4th., 1976. I would like to part with a warm and personal word of thanks to the people and Government of the United States of America for reassuring their commitment of good neighbours and brothers, extending to you an invitation to visit our country so that we can partly repay your help with hospitality.

Thank you.



Among the participants in the Friday afternoon Technical Session: Roland L. Sharpe; Eric Elsesser; Robert Henderson; and, Richard L. Miller, President of SEAOSD, presiding.

INTENTIONALLY BLANK

76

Progress Report

SEAOC AD HOC COMMITTEE ON PROFESSIONAL LIABILITY INSURANCE

by J. F. Ruthroff, Chairman

The Ad Hoc Committee was formed and charged in March of 1976 by direction of the State Board of Directors. At this time a Committee of the Southern Section of the Structural Engineers Association was already formed and functioning, making an independent study at that level. I was Chairman of that Committee, and it was therefore my decision to merge the Committees into one functioning Committee. The Committee Members were selected not only from the Structural Engineers Association and representative of all sections, but from other Professional Disciplines. Our Committee then, is represented as follows:

Mr. Diekmann and Mr. Paul Fratessa, Northern Section
Mr. Jack Barrish, Central Section
Mr. John Ruskin, San Diego
Mr. John Day, Mr. George Gray, and Mr. Sam Schultz, Southern Section.

In addition to these we have representatives from the AIA, including Mr. Clint Turnstrom, Mr. Art O'Leary, Mr. Bob Carli - from the South, and Mr. Bill Reiner from San Francisco. The Mechanical Engineers are represented by Mr. Don Nack, Electrical, Mr. Leo Press, Civil and Soils Engineers by Mr. LeRoy Crandall, and Mr. Leo Hirshfeldt. The Medical Association is represented by Mr. Frank Clark, Executive Director, the Dental Association is represented by Alan Bucco, Executive Director, the Bar Association is represented by Mr. Leonard Marangi, Mr. Paul M. Guyer, and Mr. Jim Acret. Hospital Administers are Mr. Calvin Hegarty, Dr. Robert L. Evans.

Although the title of the Committee by name would indicate an exploration into Insurance problems, it was your Chairman's opinion that Insurance was only a result of the more continuing general problem of Professional Liability. Although there has been deep concerns, by engineers, for some period of time in our own Liability Problems, the Medical Associations' current Insurance crisis surfaced the requirement for an indepth study.

The first task of the Committee, then, is to define the general problem as to cause and solution so that it can affectively recommend procedures to the Association for adoption. At this point your Committee is still in the process of definition. We are receiving testimony from all disciplines of the various professions including communication with Professional, Assemblymen and Insurance Company Representatives. The cause of our current crisis can be broken down into several categories. The first we shall call Public Attitude. I wish to point out at this point, these comments given to you are in many cases direct quotes taken from testimony.

PUBLIC ATTITUDE

These are attitudes which have developed over a number of years with both the public and the courts for which the professional, himself, is largely responsible. The attitudes have developed into a source of money, to be tapped by the public. The Professional has contributed to this attitude, by his holding himself forth, as being expert and faultless, and having specific knowledge and being free from mistakes. He is responsible for his own lack of communication in conveying to his client, his real function. The attitudes of the courts have shown their misunderstanding as to the true functions of the Professional, and Society hold the Professional to be spotless and mistakeless. When consultation can be on a one-to-one basis, and a true understanding can develope between the client and the consultant, the problems then can be resolved to a degree of understanding and not by the courts. In order for the professional therefore, on a short term basis, reduce his losses in the courts, he must adopt business attitudes including the study and understanding of loss prevention, construction contracts, execution of documents, and client communication.

LEGISLATION

The Trial Lawyers Association greatly aggrivates the potential exposure to law suit through their system of contigency fees. The Trial Lawyers Association has very openly gone after the Professional as a cource of claim. The contigency, fee basis, for court settlement is obviously not equitable because the attorney obviously wishes to press for maximum recoveries to maximize his return.

INSURANCE

Insurance is a last resort to the settlement of a dispute. Insurance Claims can be paid out of arbitration, or settlements in or out of court. Insurance companies, like anyone else, are primarily concerned with profit motives and they develope premium schedules solely with profit motive in mind. Mr. Ed Howell, President of Design Professionals Insurance Company, visited London and, the Insurance Carriers there, to determine their methods of premium structure. He was advised that he did not understand the insurance business and that an insurance premium, was not based on exposure, but was based upon the amount that the industry is willing to pay. If insurance premiums double, and there is only two or three percent lost in customer, there is obviously a much higher gross return to the insurance company. The phenomenon of layering of insurance has been discussed. This is where many individuals, insure for liability losses, on the same project. Here we have the architect, structural engineer, civil engineer, electrical engineer, mechanical engineer, contractor, and possibly the owner insuring a single project for liability losses caused by design. Along with this layering, is the large pool of deductible money which is available for tapping by attornies and their clients and is many times uncontested. Today many professional policies are written with very large deductibles, from $100 to $100,000, where companies are trying to protect themselves from disaster and not the usual losses in conduct of business. In most cases, it becomes more convenient to pay off a case, where the loss is less than a deductible amount of the policy, than to take the trouble to fight. Any single project may have more than a $100,000 available in the deductible pools, jointly, amongest all of the consultants involved. Many consultants fail to realize that the real premium of their insurance policy is the deductible amount plus the premium. In other words, on a $20,000 deductible policy with a $20,000 premium, would require a $40,000 loss before the insurance company would be interested beyond, the assignment of an attorney, who himself is working on fees paid for out of the deducted amount. So much for the general problems, and most of you are already more familiar than I with what they are, let's discuss some possible solutions.

LEGISLATION

The medical profession unleafed a great perpondance of possible legislation upon the assembly and the senate in late 1975 and 1976. Our interest, of course, in these bills would be in amendment, to aid all professionals by the legislation. It is obvious that there are two basic reasons why such limiting legislation is not popular, and may be even not possible.

1. Our assemblymen and senators are generally attorneys who do not, for their own personal reasons, wish to relinquish sources of income dollars.
2. The bills may be held as counter society. It can be stated and has been stated, by many, that the design professionals are special interest groups seeking legislation for their own benefit and not in the interest of society as a whole, and further, efforts to limit liability detract from the Citizens Constitutional Right for his day in court. Out of many bills, Assembly Bill #1, 1975 was the most significant. To my knowledge, there is no other effective legislation. The design professions are not included in Assembly Bill #1 but it has some interesting basic features. Some of the items that the bill considers is a maximum loss of $250,000 including pain, suffering, inconvenience, compensation may be paid in periodic payments rather than lump sum, provides for arbitration, provides for limitation of contigency fees, provides for three year statue of limitation, together with a 90 day notice of intent to file suit. I do not believe that there is any likelyhood of relief in the near future by legislative action. There are things that can be done, however, which would include: further definition of construction terms, further work on statute of limitations, and work on the possible extension of workman's compensation to cover employees from loss due to design deficiencies.

In the long run, it is hoped that significant basic Tort law changes may be made. The California Medical Association has formed a

Commission on Tort Reform and have appropriated three quarters of a million dollars to finance and staff the Commission. The Commission consists of 18 persons, has been organized and is starting its' work. Our Ad Hoc Committee of the Structural Engineers Association, has been recommended as an auxiliary function to that Commission. There has been however, no assignment of tasks to date. Additional work has been done with the procedure of Arbitration under the concept of Mediation Arbitration. The general concept is simply this; that as part of the general contract agreement, and in the architects, engineers, and owner agreements, that a Mediation Arbitration Director be predetermined for that project and that individual then functions as a Mediator during the construction of the project. Each dispute is arbitrated as it occurs. There should be a net result of lower costs of litigation.

INSURANCE

In order to stop-gap, run-away insurance premium amounts, several independent insurance companies have been formed by the medics and and design professions. Design Professional Insurance Company with its' President, Mr. Ed Howell, has been operating for several years and is available to our Industry. Mr. Howell has developed several concepts of insurance, some you are familiar with, includes a general policy, which provides coverage on a deductible basis for all events, and a secondly a policy which is based on the concept of limited liability. I estimate that the premium of the second policy is approximately 75% of the premium costs of the base policy. The limited liability policy limits the engineers liability to $50,000 or the amount of his fee which either is greater. It is obviously necessary to sell this concept to your client. There has been reasonable success with some engineers and architects who have carefully promoted the concept. The coming on the line of DPIC in my opinion did slow down the run-away on premium costs and actually did reduce premium costs some years back. The basic carriers already on board then, even tho they claimed losses, had new competition, and had to take a closer look at their premium structure. D.P.I.C. has implemented programs, which include, a concentration on loss prevention in an effort in assemblying a battery of competent adjustors and attorneys who are familiar with our practice. They have written several publications, which are of high value to each one of us as a practicing engineer, and should be obtained and studied and followed. These publications deal with simple things from shop detail stamps to, how to write a bill, how to follow-up a bill, write a spec, sue a client. They are what I call nitty-gritty grass roots publications and are well executed. Mr. Howell's next thrust will include the packaging of insurance for individual projects. In other words, one policy per job in effort to lower the total premium costs due to the effect of layering, which we mentioned earlier. There are obviously many problems to be solved but Mr. Howell expresses encouragement that such a procedure is possible.

The Soils Engineers have formed their own insurance company, which is an off-shore company, and although they are currently writing only a modest number of policies, they have solved some of their problem of being uninsurable. The company is concentrating on the education of their participants in loss-prevention, expect to make large expenditures on legislation, and promote continuing education. It is the company's opinion that Peer Review is a necessary function for the reduction to losses to claims.

PEER REVIEW

It is interesting that both Insurance Companies are giving great credence to Peer Review. Ed Howell advises that Peer Review will be necessary for the professional to avail himself of the concept of packaged insurance. As a matter of function, the plans would be submitted to a Peer Review Committee for comment and recommendation prior to the construction of a project. The practioner will also be required to engage himself and his staff in a program of continuing education. There is no question in the Committee's mind as to the value of continuing education, so that the professional and his company may remain current and viable in his work, but there is question as to whether litigation losses are primarily generated from mal-practice technically, or mal-practice as a business function.

Your Committee will continue to take testimony and to strive to define the problem and establish two or three significant obtainable goals with recommendations to the Board for certain measures and procedures that must be taken. As we begin to pursue legislation heavily it will be necessary for the Association to underwrite some rather large expenses. I believe that we are arriving at a point in time where we either have to fish or cut-bait and put our money where our mouth is, if we really intend and are really interested, in bringing the problem of Professional Liability into a reasonable halter.

STRUCTURAL ENGINEERING CONSULTING SERVICES FOR BUILDINGS

Guidelines for Scope and Compensation

Prepared By

Structural Engineers Association of Northern California

Office and Professional Practice Committee

Ephraim G. Hirsch, Chairman 1975-76

TABLE OF CONTENTS

STRUCTURAL ENGINEERING CONSULTING SERVICES FOR BUILDINGS

Guidelines for Scope and Compensation

INTRODUCTION

Changes in the nature of the practice have created the need for new guidelines for the furnishing of structural engineering consulting services for buildings and compensation thereof. Among these changes are more complex requirements for the analysis and design of buildings than have previously been the case and greater demands made upon the structural engineering consultant to provide design input for items previously not required or excluded from his scope of services. Additionally, recent court decisions have held that recommended minimum fee schedules are illegal in that they represent restraint of trade.

These guidelines are primarily intended for use in the situation where the structural engineer acts as a consultant to the design professional, usually, but not necessarily, an architect, who holds the primary building design contract with the owner/client. In cases where the structural engineer is the Prime Design Professional or contracts to work directly for the owner/client, modifications to these guidelines, particularly in the areas of contractual obligations and fee payment scheduling, may have to be made to suit the particular circumstances in question. However, the basic philosophy still applies.

These guidelines are general in nature and seek to provide a basis for establishing scope of services within several specifically designated categories. By defining agreed upon demarcations of responsibility and limits within each category, it is the intent that these guidelines be used by the individual consultant as an aid in negotiating conditions of contract and in establishing method and amount of compensation commensurate with the services provided.

Three categories of services - Basic, Special, Extra - are defined, followed by an exposition of the scope encompassed by each.

I. DEFINITION AND SCOPE OF SERVICES

A.1. Basic Services

The Basic Services responsibility of the structural engineer is limited to the analysis, design, detailing and specification of the Primary Structural System of the Building. The Primary Structural System is defined to mean that basic system which furnishes the required stiffness, strength and stability to support all structural and non-structural elements and to resist within acceptable or codified limits the loads imposed upon the building by gravity, wind, earthquake, machinery inertia, earth or hydro-static pressure, or any other designated design force.

The traditional role of the structural engineering consultant, and for which traditional fees were structured, has been to provide solely these Basic Services for the Primary Structural System.

The Primary Structural System comprises the assembly of decking, slabs, joists, beams, girders, trusses, columns, cables, shells, vaults, domes, piers, walls and foundations, etc. necessary and sufficient for support. Non-structural elements are those architectural, mechanical, electrical and other components, for which specific design information must be furnished by the architect, by the mechanical, electrical, other consultants, and/or by the owner and which make no direct contribution to the Primary Structural System other than by imposing loads upon it.

2. Basic Services Scope:

The following structural engineering services are those which are Basic to any project. They are considered to be the minimum, but not necessarily the maximum, which must be provided to insure a complete and thorough structural engineering analysis and design from concept through construction. Elimination of any portion should be the subject of specific discussion with the client regarding the potential consequences of that omission.

a. Schematic Phase

1) General consultation with Prime Design Professional, his client, and his other consultants.

2) Study and advise on selection of systems and materials, including foundations if adequate soils information is available.

3) Sketches, calculations and other assistance to Prime Design Professional for preparation of his conceptual drawings.

4) Assistance in establishing criteria for soils report.

b. Preliminary Phase

1) Preliminary calculations

2) Review of soils report and selection of foundation system.

3) Preliminary structural foundation and framing drawings showing Primary Structural System materials, gross sizes, and critical details.

4) Assist Primary Design Professional in preparing preliminary cost estimate and outline specifications.

5) Coordination with Prime Design Professional and other consultants.

c. Working Drawings Phase

Following written approval and acceptance of the preliminary design by all interested parties to the project, preparation and furnishing in a mutually agreeable format:

1) Final structural calculations.

2) Final structural drawings showing:
   a) Typical details and general notes
   b) Foundation and framing plans, elevations, and sections dimensioned, detailed, and identified sufficiently to define and establish The Primary Structural System.
   c) Indication of non-structural items which affect the basic structure, or appropriate cross reference to drawings by others for such items.
   d) Any other information required to ensure the adequate design performance of the structure.

3) Preparation or editing of structural specifications.

4) Final checking of structural drawings and clearing them with the appropriate reviewing agencies.

5) Assistance to cost estimator in preparing final estimate.

d. Bidding and Negotiation Phase

1) Issuance of structural clarifications and/or addenda.

2) Assistance to the Prime Design Professional in the matter of analyzing bids and letting contracts.

e. Construction Phase

1) Attendance at the preconstruction conference and at construction conferences as required.

2) Review of structural shop drawings furnished by the building contractor to check for general compliance with and understanding of the intent and requirements of the structural aspects of the construction documents. This review is in no way to be construed as an assumption of responsibility by the structural engineer for actual compliance by the building contractor with the contract documents, or for his methods of work and safety procedures.

3) Interpreting the intent of the structural contract documents should conflicts arise or ambiguities be revealed during construction and answering questions and issuing clarification drawings or addenda as they pertain to these.

4) Review of reports submitted by testing and inspection agencies.

5) Site visits during the course of construction to observe and be familiar with the progress of the structural work, with, at the minimum, visits occuring at the foundation stage; the framing stage while the Primary Structural System is still visible; and at the final stage before the Primary Structural System is hidden from view by collateral materials.

B.1. Special Services

Expanding code, reviewing agency, or client requirements have brought many items heretofore undefined, randomly treated, or entirely ignored, within the purview of the structural engineering consultant. These include building elements not necessary to the Primary Structural System, as defined under Basic Services above, but for which structural design, detailing and specification are now required. Such consultation is outside the scope of Basic Services but may be furnished as a Special Service. Any or all of such Special Services may, upon appropriate adjustment of the Basic Services compensation, be included within the Basic Services if required and requested by the Prime Design Professional.

2. Special Services Scope

a. Analysis, design, and drafting of items not part of, or necessary to, the Primary Structural System, together with such calculations, meetings and consultations as may be required. Items in this category include, but are not necessarily limited to:

1) Secondary or auxiliary members, struts, angles, pipe, battens, etc., or any patented systems whose sole purpose is to serve as carrying members for non-structural elements.

2) Exterior cladding not part of the Primary Structural System.

3) Glazing, window wall and door systems.

4) Partitions and partitioning systems not part of Primary Structural System.

5) Ceiling and lighting systems and related bracing and support systems.

6) Mechanisms and guide systems for elevators, escalators, other conveyor systems and associated operating equipment.

7) Casework and furniture and their installation and attachment.

8) Installation and attachment of mechanical, HVAC, plumbing equipment and fixtures, including, but not limited to boilers, heat exchangers, chillers, cooling towers, tanks and vessels, motors, pumps, furnaces, general piping systems, air distribution systems and fire sprinkler piping systems.

9) Installation and attachment of electrical elements including, but not limited to transformers, emergency generators, conduits and cables, cable trays, panel-boards, lighting fixtures and switch gear.

10) Special equipment including stage equipment, acoustical fixtures, etc.

11) Landscape furnishings such as flag poles, lighting poles, benches, fountains, etc.

12) Decorative work such as sculpture, screens, murals, etc.

13) Site work elements exterior to and non-contiguous with the buildings such as retaining walls, culverts, bridges, etc.

b. The review of architectural/landscape/mechanical/ electrical or other design drawings or specifications prepared by others with respect to their adequacy for system, for anchorage of non-structural elements, and for conformance to codes.

c. Studies of schemes alternate to the one developed and approved during the Basic Services preliminary design phase.

d. Special dynamic analyses such as spectrum or time history response for seismic, or floor response for foot-fall or vibratory equipment.

e. Special wind analyses.

f. Risk analyses

g. Non-structural civil engineering services for the project.

h. Soils and site evaluations, selection and reports.

i. Building surveys, reviews, and inspections.

j. Work connected with the letting of alternate bids and of segregated contracts for phased construction.

k. Continuous and detailed inspections of construction.

l. Design or inspection of shoring, earthwork, excavation, or formwork.

m. Design for future expansion.

n. Filing application for and taking out of building permit.

o. Preparation as "as-built" drawings after completion of the project.

p. Administration of construction contract.

C.1. Extra Services:

These are services which result from changes or corrections due to, or instigated by, others over which the structural engineer has no control and which are not included or anticipated at time of initial Agreement.

2. Extra Services Scope:

a. Work resulting from changes in scope or magnitude of the project as described and agreed to under the Basic Services Agreement.

b. Work resulting from changes required due to a construction cost guarantee by the Prime Design Professional.

c. Work resulting from changes in design or from construction method of the project over which the structural engineer has no control after acceptance of agreed design as described under Basic Services.

d. Work resulting from corrections or revisions required due or errors or omissions in construction by the building contractor or his failure to comply with the construction documents.

e. Work resulting from shut-down of the project for protracted periods of time.

f. Services after final payment to the building contractor.

g. Providing services as an expert witness in connection with any public hearing, arbitration, or proceedings of a court of record with respect to the project.

II. COMPENSATION

A. Basic Services

Since most contractual agreements between the Consultant and the Prime Design Professional specify that the Consultant perform his services in character, sequence and timing in the same manner and to the same extent as those of the Prime Design Professional, it is recommended that the method of computing fee also be commensurate with and identical to that of the Prime Design Professional with the owner/client.

If the full scope of Basic Services as set forth above is not to be performed, then perhaps there is justification for a differing method of fee computation to reflect reduced scope and responsibility. However, when full services are to be furnished, this recommendation is intended to insure equity between Consultant and Prime Design Professional for services rendered and fees received.

Among methods of computing compensation, the following are most common and merit discussion.

1. Percentage: Fee computed by this method is usually expressed as a percentage of the total cost of construction for the project as paid by the owner. The actual percentage will depend both upon the complexity of the project - more complex or unusual structures requiring a higher percentage - and its cost - the percentage usually declining with increasing construction cost.

   It is very important to define accurately at the initiation of negotiations the cost of construction to be used as a basis for fee computations, since often items such as landscape work, site development casework, etc., are deducted from the construction cost for purposes of fee computation. In such cases any consultation for omitted portions would obviously have to be paid for as Special Services. In order to avoid complexities of re-phrasing any sliding percentage fee scales as a function of the cost of construction, it may be advantageous to express the Consultant's fee as a percentage of the Prime Design Professional's fee. For full Basic Services, the structrual engineering consultant's fee thusly computed is usually in the range of 17 to 25% of the Prime Design Professional's fee, with 20% being the usual median.

   Although this method of fee computation has been popular in the past and is still in mixed general use, it has fallen in disfavor among design professionals in recent years. Among reasons for this is the problem that investing time and effort to reduce the cost of construction or to accomodate "fast-tracking" construction procedures also results in a reduction of the fee received despite increased production costs to achieve lowered overall project cost. Perhaps a way to combat this would be to set a minimum fee regardless of the cost of construction.

2. Hourly: In this method of compensation, charges are based on multiplying salary costs on an hourly basis by a factor appropriate enough to cover benefits, taxes, holidays, sick leave and vacations, general overhead, insurance surcharges (if any), and profit. Principal's time may be computed as a flat hourly rate of an appropriate amount. It is recommended that computer service charges be treated as an "employee" rather than as a reimbursable expense, but multiplied by a factor that recognizes that no 'vacation, sick leave, holiday, benefits, etc.' need to be covered for the computer, only general overhead and profit.

   This method of compensation is particularly appropriate where consultation is only done for portions of a project or when the total scope of project or services cannot be determined in advance. The total fee to be charged may be open-ended or a limit set at a guaranteed maximum depending upon contractual negotiations and ability to define the scope precisely.

3. Cost Plus Professional Fee: This is similar to the previous method, save that the multiplier for salaries and computer time would only cover costs and no profit. To this direct cost sum, then, must be added a negotiated professional fee to cover profit and principal's time and consultation.

4. Lump Sum: If the scope of the project can be adequately defined to everyone's satisfaction, then a lump sum fee may be appropriate. It is most important in this type of compensation that the parameters and limits of the scope of services and project be precisely defined so that work outside of the original intended scope will be compensated. Obviously, the lump sum must be sufficient to cover all costs and anticipated profit, allowing sufficient time for thorough professional services in accordance with generally accepted standards of professional engineering practice.

B. Special Services Fee and Extra Services Fee:

These services may be furnished under any of the methods of compensation listed above, though, especially for Extra Services, hourly charges times an appropriate multiplier is usually the most suitable.

C. Reimbursable Expenses:

1. Travel
2. Lodging and meals
3. Long distance telephone and telegraph
4. Printing costs above and beyond progress check prints
5. Sub-contracting for other professional services
6. Photography

Reimbursement: It is suggested that a service charge be added for bookkeeping expense entailed.

D. Schedule of Payments:

It is recommended that upon signing of a contract for furnishing of professional services, a retainer of 5% of the anticipated total fee be paid. It is further recommended that monthly billings be made to reflect progress of the work. In the cases of compensation by the hourly, cost plus or lump sum method, these billings become due and payable at the end of each month. In the case of compensation on a percentage basis, the more traditional schedule of payments is to receive 15% of the estimated total fee upon completion of schematic design, an additional 20% (to a total of 35%) on completion of a design development phase, an additional 40% (to a total of 75%) on completion of construction documents, and an additional 5% (to a total of 80% upon award of construction contract. The remaining 20% is to be received in monthly installments as the construction proceeds. However, it is still recommended that monthly billings be made as the work progresses to reflect work completed.

Regardless of the scheduling method used, it is recommended that all billings more than 90 days in arrears from the agreed upon due date of billing be subject to a service charge at a legal rate of interest retroactive to the original date of billing and that provision be made for costs of collection should that prove necessary.

E. Other Considerations:

The legalities and precise wording of contractual obligations and mutual responsibilities are beyond the scope of these Guidelines. Reference for these matters should be made to legal counsel and to Standard Contracts and Agreements such as those published by CCAIA/CEAC and by the A.I.A. Consideration in negotiations and in setting of contractual terms should also be given to the question of limitation of liability, as recommended by some insurance carriers, and to possible financial consequences should it not be obtained, depending upon the terms of the professional liability insurance carried.

III. RELATIONSHIP OF STRUCTURAL ENGINEER TO OTHER DESIGN PROFESSIONALS.

In order to facilitate cooperation between design professionals engaged in providing consulting services in common for a building project, this section is intended to define divisions of responsibilities with regard to structural elements and to the support and anchorage of non-structural elements. The need for cooperative effort and interdisciplinary solutions is obvious in order effectively to ensure a quality of performance for non-structural elements which will be consistent with the overall design aims for a project.

A. Included within Basic Services of the Structural Engineer:

1. The Primary Structural System shall be designed, detailed and specified for support of all non-structural elements for which specific design information has been furnished by the architect and/or, by the mechanical, electrical other consultants and/or by the owner. Since the validity of the structural design or analysis will be largely dependant upon the accuracy of the available data, it is essential that the various disciplines transmit the following basic information to the structural engineer as early as possible in the design sequence:

    a. Location: Location of all elements by dimensions from identifiable reference points.

    b. Operating or Installed Weight: Maximum operating weight of elements, including all accessories, and based on the most critical of the proprietary models potentially capable of satisfying functional requirements.

    c. Dimensions and Configurations: Physical description of element including height, width, depth, etc., or a copy of the manufacturer's data sheet.

    d. Anchorage: Indications as to whether element will be solidly anchored or flexibly mounted.

    e. Mounting Details: Description of support configuration, including forces to be transmitted through it to the Primary Structural System; whether element has individual legs, continuous perimeter frame, integral support rails, etc.

    f. Special Support & Mounting Pads: Information regarding the size, thickness or weight of inertia or housekeeping pads which may be required.

    g. Special Requirements: Unusual conditions which may affect or limit the choice of structural materials or design, such as flexible (i.e., changeable) mountings, electrical/mechanical/acoustical/impact isolation, accessibility, pre-planned future modifications, etc.

Based on these data, the structural engineer will provide the Primary Structural System design necessary to resist forces generated by these non-structural elements.

2. When a dynamic analysis of the Primary Structural System is necessary, inform the other members of the design team of any extrordinary seismic design requirements (coupling, amplification, etc.) which may result in modification of standard or conventional details.

3. The structural engineer will provide the other members of the design team with information regarding the supporting capability and physical attachment limitations of the particular types of framing systems to be utilized on a given project.

B. Responsibility of Other Design Professionals:

In general, each design discipline will be responsible for specifying, detailing or otherwise selecting its own specific anchorage mechanism for all non-structural elements included within its area of design responsibility. This will include anchorage of elements for gravity, inertial, and seismic forces.

C. Additional Considerations:

1. The structural engineer will assume responsibility only, as is required by law, for those items for which he has actually done the design and will so signify by stamping and signing his own design drawings and calculations.

2. The review of design drawings for the project prepared by other design professionals will be undertaken solely to ascertain whether or not the designs contained therein comply with the general requirements of the contract documents, as is the situation with shop drawings previously described. However, beyond such review, the structural engineer will not, and indeed cannot, stamp and sign drawings prepared by others. This is the responsibility of the design professional who actually supervised the preparation of such drawings.

3. Performance clauses which contain references to "certification", "verification" or "design" by a structural engineer of non-structural elements or their anchorage should distinguish clearly between the individual who is to be retained by the contractor, sub-contractor or supplier to perform this service, and the responsible structural engineer of record for a given project.

INTENTIONALLY BLANK

INTRODUCTION TO SEISMIC REHABILITATION OF THE STATE CAPITOL

by

Henry H. Lee and Sigmund A. Freeman

URS/John A. Blume & Associates, Engineers, San Francisco

INTRODUCTION

In 1971, a study[1] by the State of California, Office of Architecture, reported that, for the century-old State Capitol Building in Sacramento to meet modern standards of earthquake safety for its present use, it would be necessary either to undertake a complete structural rehabilitation or to demolish the building and build a new one. Although the cost of either scheme would be approximately equal, a new structure was never seriously considered. Architects and politicians agreed that the old building is an architectural gem as well as an irreplaceable landmark of great historic significance. After a report[2] by Welton Becket & Associates (WBA) of Los Angeles and URS/John A. Blume & Associates, Engineers (URS/Blume), of San Francisco showed that rehabilitation was feasible, legislation was enacted in 1975 providing funds for that purpose.

Figure 1 is a photograph of the capitol taken shortly after its completion in 1874. All the walls are unreinforced brick. The main portion of the

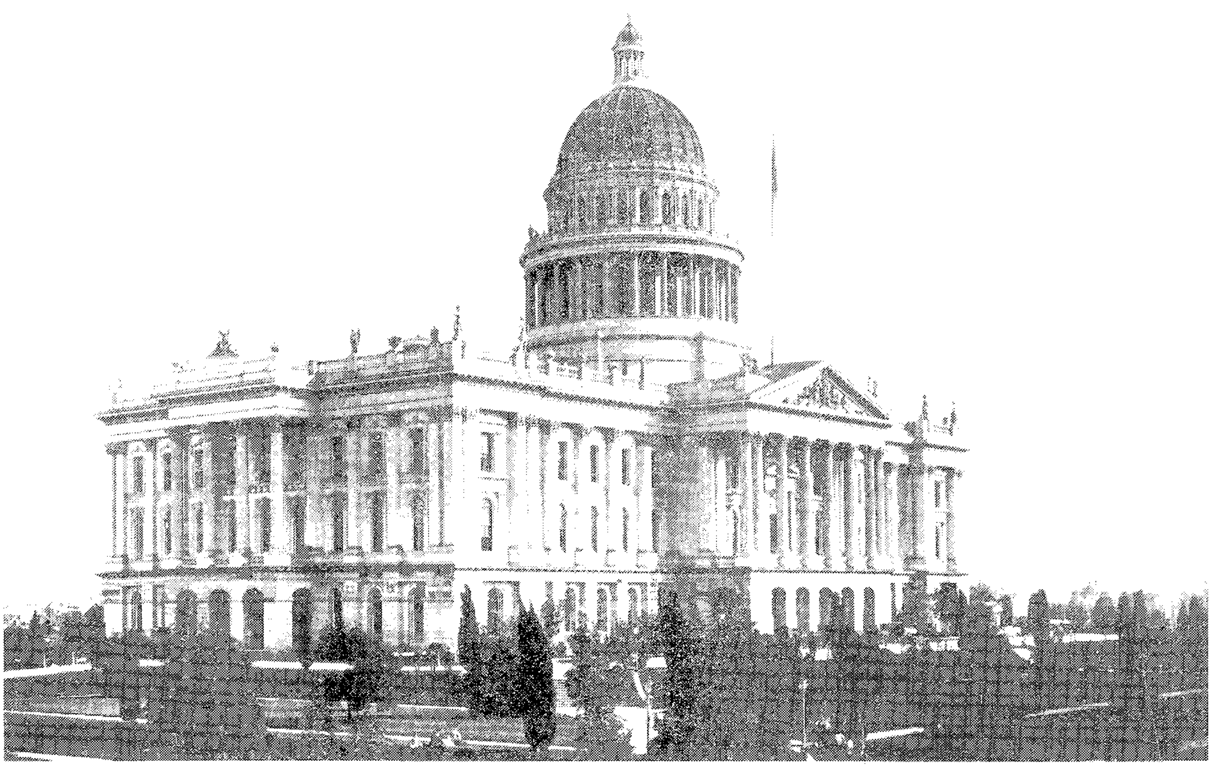


FIGURE 1 THE STATE CAPITOL IN 1874

Preceding page blank

building has four stories and is 84 ft high; the rotunda walls extend 82 ft higher, supporting the outer dome, which is framed with wrought-iron trusses and covered with wood sheathing on wood purlins. The cupola on top has a wood roof supported by cast-iron columns. The typical floor system consists of shallow brick arches (Figure 2) spanning 5 to 7 ft between wrought-iron beams supported without anchors on the brick walls. The finish floor is 1-1/4 in. tongue and groove over wood joists resting on the brick arches.

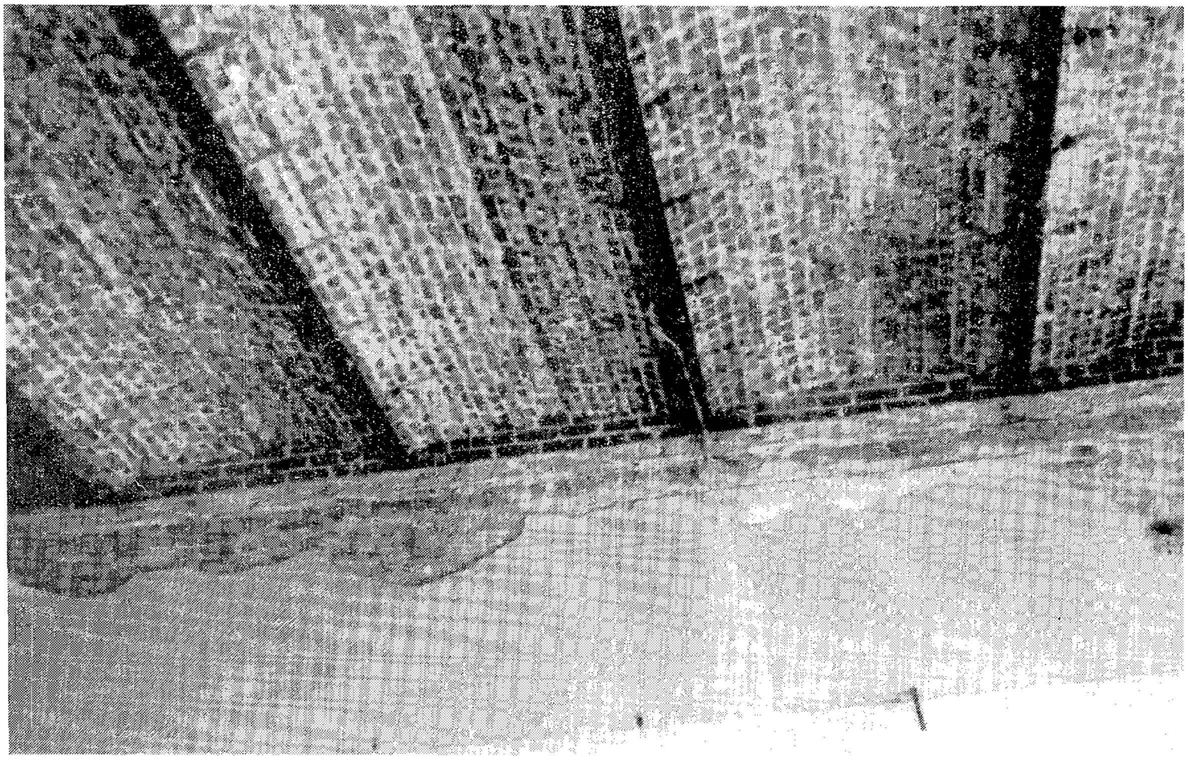


FIGURE 2 TYPICAL FLOOR SYSTEM

A large quantity of cast iron is used as ornamentation in addition to the columns that support the porticos and the colonnades. Window frames, roof cornices, balustrades, etc., are in evidence over the entire exterior of the building (see Figures 3, 4, and 5). Granite facing is used up to the second floor, and above that, the brick is covered with cement plaster. The need for structural rehabilitation is apparent.

Some of the historic background of the capitol is as interesting as the technical aspects. Before the present capitol building in Sacramento was built, California had a "roaming" capitol. The state legislature had met in four other cities -- San Jose, San Francisco, Vallejo, and Benicia. Prior to that, before the admission of California to the Union (1850), the California Constitutional Convention had met in Monterey. Each of the moves involved intensive debate and, in some cases, litigation.

FIGURE 3 CAST IRON WINDOW FRAMES AND ROOF CORNICES

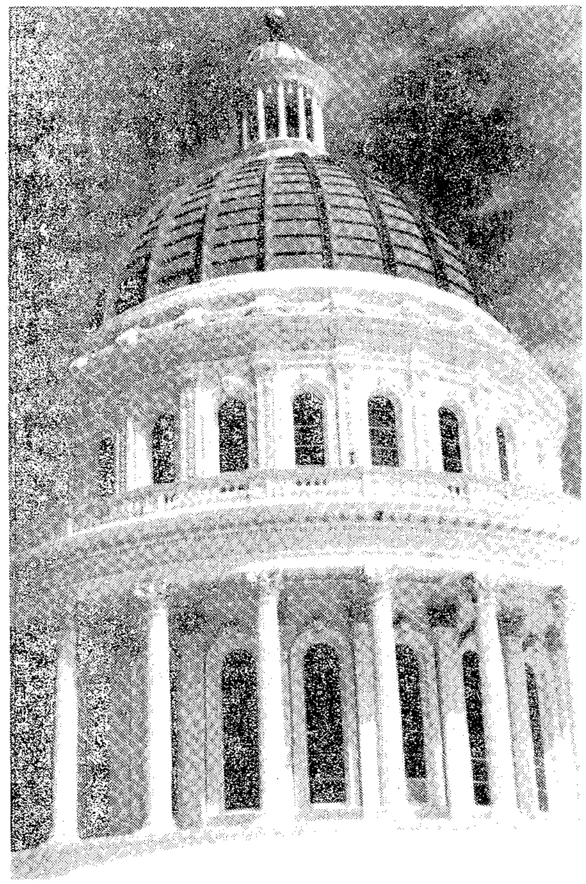


FIGURE 4 COLONNADE, DOME, AND CUPOLA

In 1856, legislation was formed to build a permanent capitol building on the square near the present site. A plan by architect Reuben Clark was adopted, a low bid of $200,000 was obtained, and bonds were sold; ground was broken in December 1856. Shortly thereafter, a supreme court decision declared the bonds had exceeded the state's legal indebtedness and rendered the contract void. The contractor sued and settled for his costs of $5,400. The project was then abandoned.

In May 1860, after several more years of debate, a bill was passed designating a new site, the present site, for construction of a new capitol. The Board of Capitol Commissioners selected a plan by architect M. F. Butler and appointed Reuben Clark, the architect of the previous plan, superintending architect. There was never a general contractor on the job, nor were there complete working drawings. Each superintending architect over the next 14 years contributed drawings and influenced the course of the design.

The first contract was awarded to Michael Fennell in September 1860 for $80,000 for the foundation and basement walls. Seven months later, Fennell withdrew due to financial difficulties. A new contract was awarded to Blake and Conner, who carried on the work until the winter of 1861-62, when several great floods devastated the work. Blake and Connor quit in April 1862. Figure 6 is a typical page from their specifications.

FIGURE 5 PEDIMENT OVER WEST PORTICO, CAST IRON CORNICES, AND COLUMN CAPITALS

At this time, a decision was made to raise the level of the first floor by 6 ft. Construction continued, but funds were running out. In March 1863, the legislature passed a bill to levy a property tax to finance the construction. Another act directed the commissioners to let contracts for material purchases and to employ labor by the day. These two acts forced the progress of the work to continue at a snail's pace for the next 11 years of construction. In January 1866, Gordon Cummings was appointed to replace Reuben Clark, who became mentally ill and died shortly thereafter.

In November 1866, a crack appeared in the northwest area of the basement wall. Work was halted, and Captain Elliot of the Corps of Engineers was called in to investigate. The west wall had been carried up faster than the other walls to give the appearance of faster progress and had apparently caused a differential settlement of 2-3/4 in. at the northwest corner. A board of inquiry of the legislature also heard testimony from many architects and engineers and contractors, and, as a result, brick buttresses were added at the area of greatest settlement, and no further damage occurred.

Digging and Grading

Excavations are to be made for the reception of all the various foundations, to a level of the bottom of those now laid, and the foundations now dug are to be trimmed to the required forms, where required. The earth is to be filled in around all the foundations and solidly rammed, and all the surplus earth to be graded under the basement floor and around the building, as may be directed.

Foundations

All the various foundations are to be formed of concrete three (3) feet in thickness, including the layer of cobble stones, and all to be of the forms and widths shown and figured on the sections marked "AA" "BB" "CC" etc. The bottoms of all the trenches are to be made perfectly level and covered with a layer of cobble stones, solidly bedded and rammed down; the spaces between the stones to be clear from all earth, rubbish or small stones, for the reception of the concrete. The stones now in the trenches to be replaced and the loose earth to be removed. The concrete to be formed as follows: To one (1) measure of the best Rosendale Cement; well manipulated with one and one quarter (1¼) measures of clean, sharp, river sand; add one and three quarter (1¾) measures of granite chips broken in small pieces (i.e. the average size not to exceed a cubic inch) and one and one half (1½) measures of fine gravel, clear from all foreign substances; and all to be thoroughly manipulated and rammed into the trenches immediately after mixing. No layer to exceed ten (10) inches in thickness. The sides of the concrete to be formed by means of plank boxes, which can be removed for the formation of other sections after the concrete has sufficient consistency

FIGURE 6 TYPICAL PAGE FROM BLAKE AND CONNOR SPECIFICATIONS

Figure 7 shows a scene during construction. While the construction dragged on and costs increased, various bills were introduced in the legislature to move the capital to other cities, including San Jose, Oakland, San Francisco, and Benicia. They all failed, and construction continued. The building was first occupied in November 1869, although the upper dome and the porticos were not yet completed. It was another five years, 1874, before an official completion date could be established.

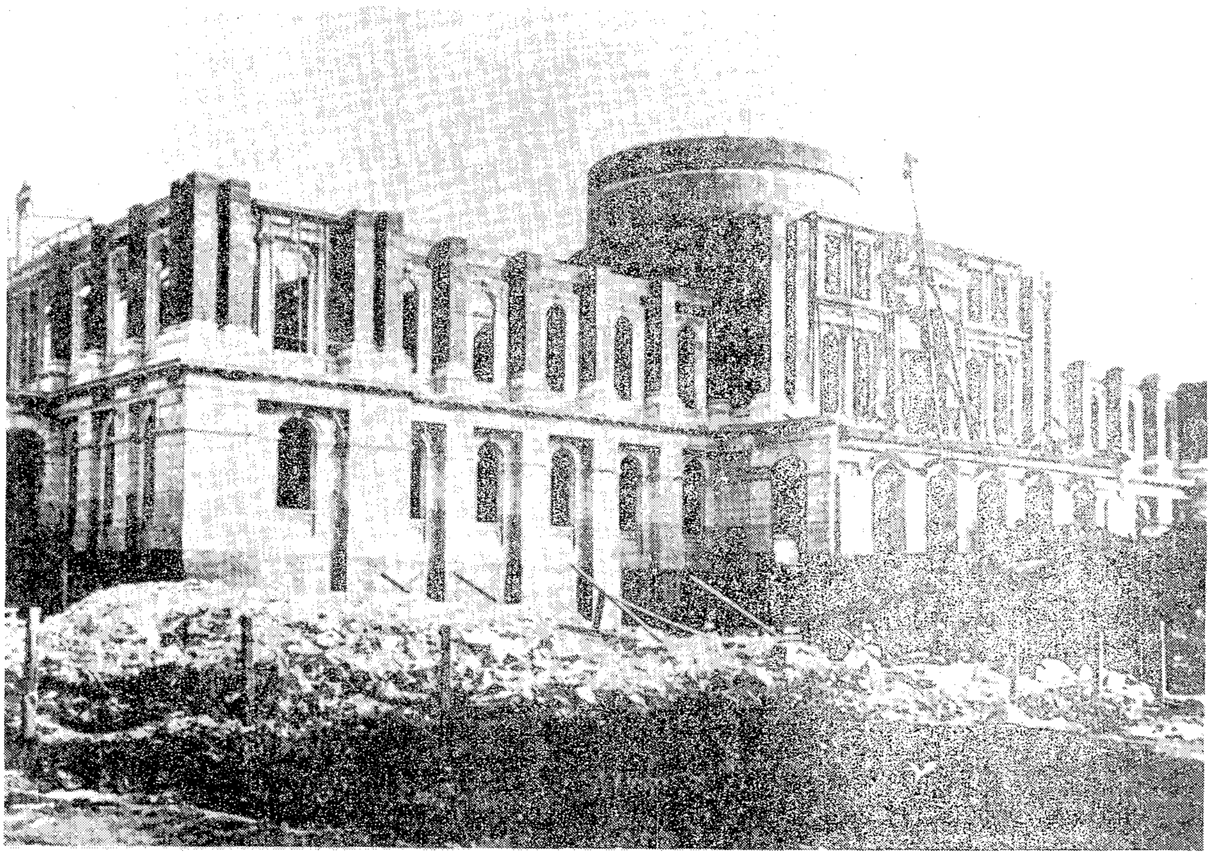


FIGURE 7 CONSTRUCTION SCENE, CIRCA 1867

However, construction never really stopped. Various interior alterations continued throughout the life of the building. In 1906, a major remodeling took place: the wood roof trusses were replaced by steel trusses, and an additional floor, the fourth, was suspended from the trusses. The statues on the roof balustrade were also removed in 1906.

When WBA and URS/Blume started work, the state archives had been searched, but no original drawings other than the foundation plan were found. Numerous other drawings were found, but these were for the subsequent alterations. It was therefore necessary to go through the building to obtain dimensions and details of the existing construction. This was not an easy task because the structure had been obscured by new partitions, mezzanines, ceilings, and stairs.

The structural design was also not a straightforward procedure. Design decisions often required additional consideration for purposes of historical preservation. The project required the services of a special historical consultant, architect Raymond Girvigian, who has done an enormous amount of research and investigation into the history of the capitol. His recommendations on which architectural features should be preserved were generally followed. It is from his report[2] that much of the historical data was obtained.

Figures 8 through 12 show the building interior as it looked before work started.

FIGURE 8 CORRIDOR

LATERAL FORCE DESIGN CRITERIA

Lateral force design criteria from several different sources were considered during the preliminary evaluation of the building. These sources included the 1973 *Uniform Building Code* (*UBC*),[3] the 1974 SEAOC Blue Book[4] (since adopted with minor revisions for the 1976 *UBC*), the California Code for Hospital Facilities[5] (*Title 24*), recommendations prepared for the U.S. Atomic Energy Commission[6] (AEC, now ERDA), and the California Department of Transportation[7,8] (CALTRANS). Recommendations from all these sources were compared to each other to establish a single criterion that would satisfy the intent of the static seismic force recommendations (e.g., *UBC*, SEAOC, and

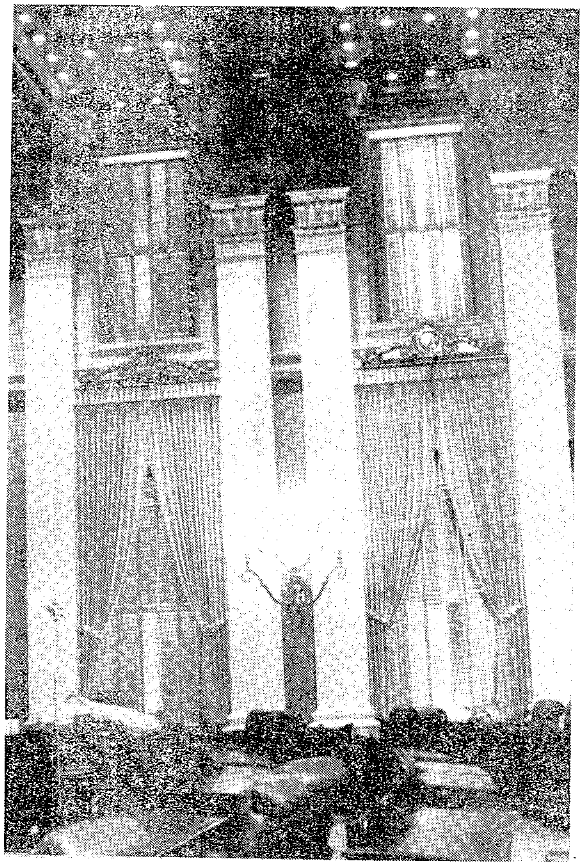


FIGURE 9 SENATE CHAMBER

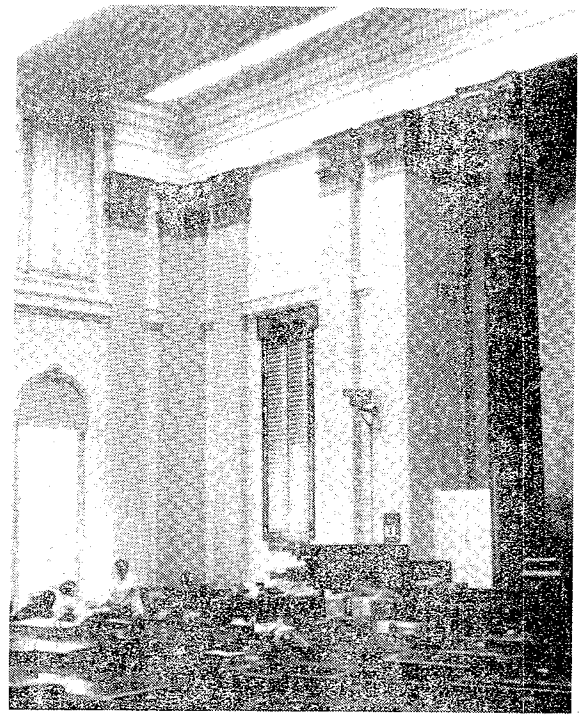


FIGURE 10 ASSEMBLY CHAMBER

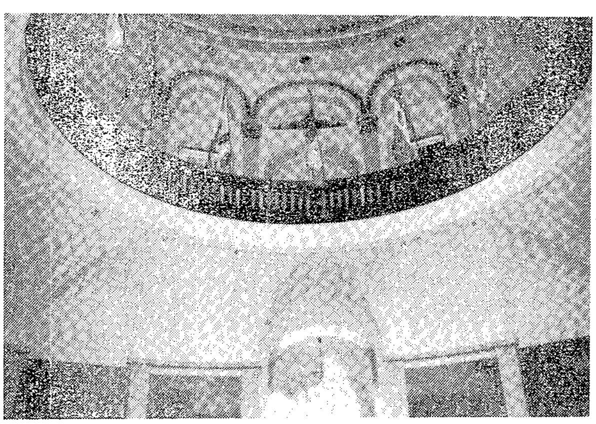


FIGURE 11 ROTUNDA

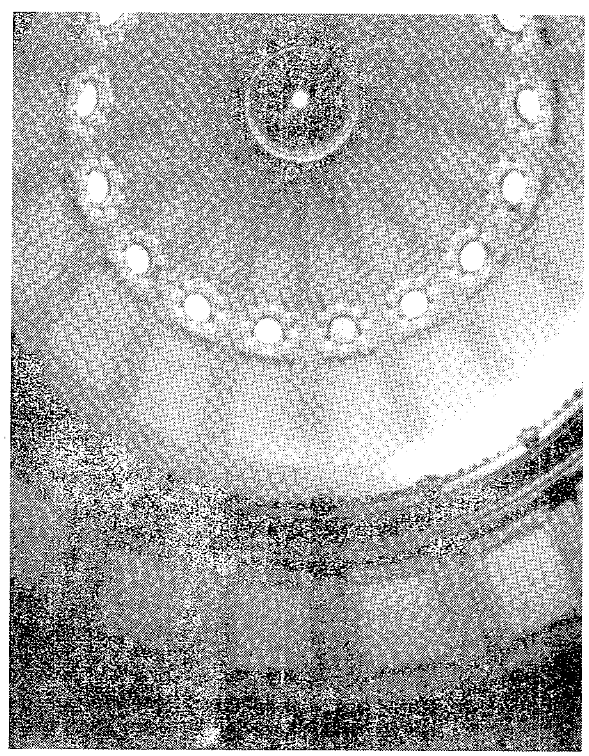


FIGURE 12 INNER DOME

Title 24) and to provide by means of dynamic analysis reasonable safety against a postulated maximum credible earthquake that may occur in 100 years.

CALTRANS provided the results of seismicity studies that postulated design earthquakes on either the San Andreas or the Midland faults. The resulting acceleration response spectrum shape appeared to be unrealistically low in the short period range (i.e., < 0.4 sec). With CALTRANS' concurrence, it was decided that the design spectrum would be modified by adjusting the short period range of the spectrum to conform to the AEC standard spectrum shape.[6] The result of this modification was designated the modified free-field spectrum (i.e., a spectrum produced by surface ground motion not influenced by the effects of nearby structures).[9]

Further studies considered the effects of soil-structure interaction. A representative acceleration time history was developed to reproduce the modified free-field spectrum. A corresponding time history was developed at the bedrock level. Dynamic analyses were then performed on idealized mathematical models of the soil and structure. The subsurface soils were represented in the models by physical properties obtained from in-situ and laboratory tests.[7] The building was represented by lumped masses and stiffnesses obtained from preliminary design. Parametric studies were performed to simulate soil-structure interaction under seismic motion. These studies included investigation of the effect of the adjacent East Wing, the variation of depth to rock, the change in shear modulus and damping at various strain levels for each soil layer, the effects of the size of the model, and the effects of the boundary conditions in the analyses. The results of this study[9] resulted in a design response spectrum for the foundation level of the building as shown in Figure 13.

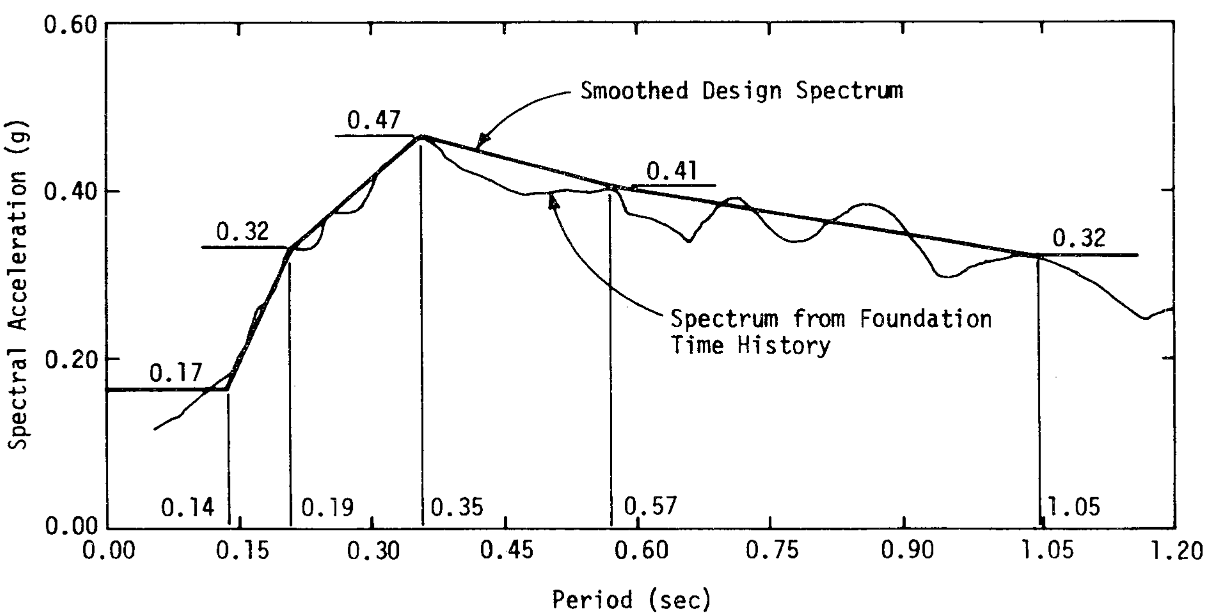


FIGURE 13 DESIGN RESPONSE SPECTRUM AT FOUNDATION LEVEL

Because of the presence of the domes and rotunda, and due to its unusual features, the building was considered to be a structure having highly irregular shapes or unusual structural features; therefore, the distribution of lateral forces was determined considering the dynamic characteristics of the structure.[4] Several mathematical models were developed to represent various assumptions relating to building elements, soil-structure interaction, and computer programs. After studying the results of the various assumptions, one model was selected as the best representation of the building for design purposes. The distribution of force determined by considering the dynamic characteristics of the building was compared to the distribution of lateral forces determined by conventional static seismic code procedures. Relative distribution of forces to the upper portions of the building was substantially higher when the dynamic characteristics were considered, thereby emphasizing the importance of considering the dynamic characteristics of structures with unusual shapes and setbacks.

The proposed lateral force design criteria are felt to provide sufficient conservatism to provide for life safety of the occupants and to preserve the historical importance of the building. In spite of the apparent conservatism, comparison with code requirements indicates that some provisions of current codes would be more stringent than the criteria being implemented in the design. Because the codes permit a rational analysis instead of adherence to the specified provisions, it is considered that the proposed analyses and design procedures comply with the intent of these codes. The intent of the codes, with regard to earthquakes, is stated in the 1975 commentary to the fourth edition of the recommendations of the Structural Engineers Association of California[4] as follows:

1. Resist minor earthquakes without damage;
2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;
3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, but with some structural as well as nonstructural damage.

GENERAL SCHEME OF REHABILITATION

In view of the criteria requiring resistance to large lateral forces, the most feasible scheme would be to replace as many of the structural components as possible. Which parts were to be replaced and which were to be preserved was not decided until after extensive investigation. Figure 14 shows a typical floor plan of the building.

The roof and the entire interior of the building will be removed and replaced with new construction in the same layout as the original system. New 12-in. concrete walls will replace most of the old interior brick walls in essentially the same locations, and new concrete pan-joist or waffle-slab floors will replace the old brick arch floors at the same elevations. The exterior walls will remain to preserve the architectural character of the building.

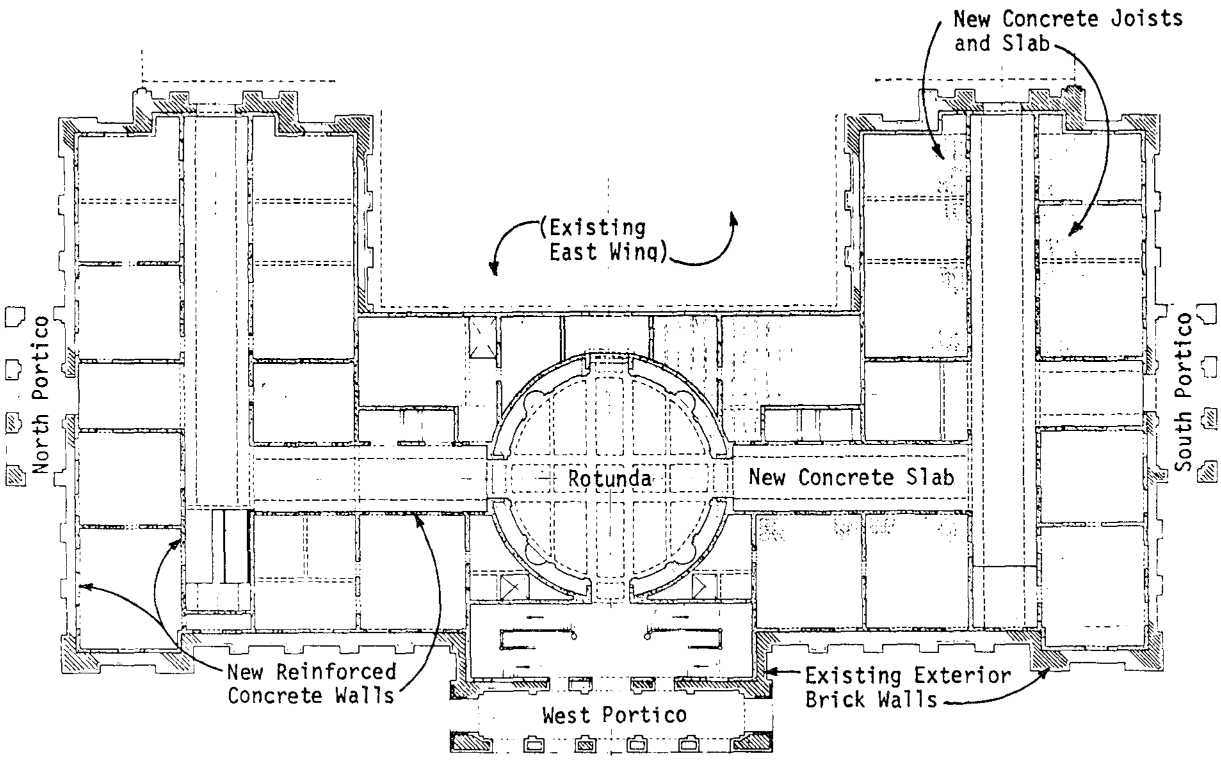


FIGURE 14 TYPICAL FLOOR PLAN

These walls will have two wythes of brick on the inside face removed and replaced with 12 in. of shotcrete. The brick walls will be supported laterally by the new shotcrete walls by means of Celtite anchors. Figure 15 shows typical exterior wall sections.

All the new walls will be supported by a 3-ft-thick concrete foundation mat that will cover the entire area of the building, including the rotunda.

The fourth floor will be retained because the need for office space is great. The level over the chambers will be raised to allow the restoration of the historic ceiling and will be suspended from the new steel roof trusses.

Figure 16 shows a section through the domes. The inner dome is the one seen from inside the building. The top is 115 ft above the first floor. The outer dome is another 62 ft above that, and it is another 26 ft to the ball on top of the cupola. At the upper dome, new structural steel framing will be added between the existing trusses shown in Figure 17 to make it stable against lateral forces. The cast-iron compression ring at the top and the tension ring at the bottom will remain. The wood purlins and sheathing will also remain unless decay is found. The brick wall supporting the outer dome will be strengthened with gunite down to the top of the lower windows.

The lower portion of the drum wall supporting the upper dome (Figure 18) will be completely replaced with concrete. This work will have to be done in sec-

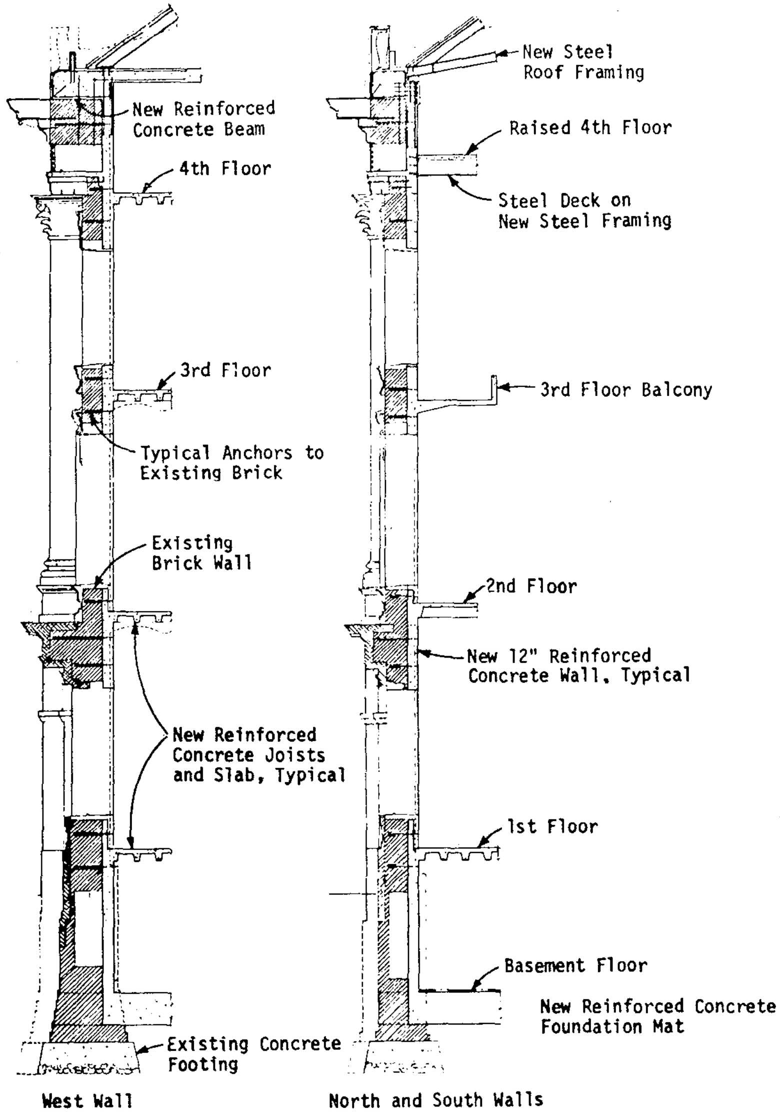


FIGURE 15 TYPICAL EXTERIOR WALL SECTION

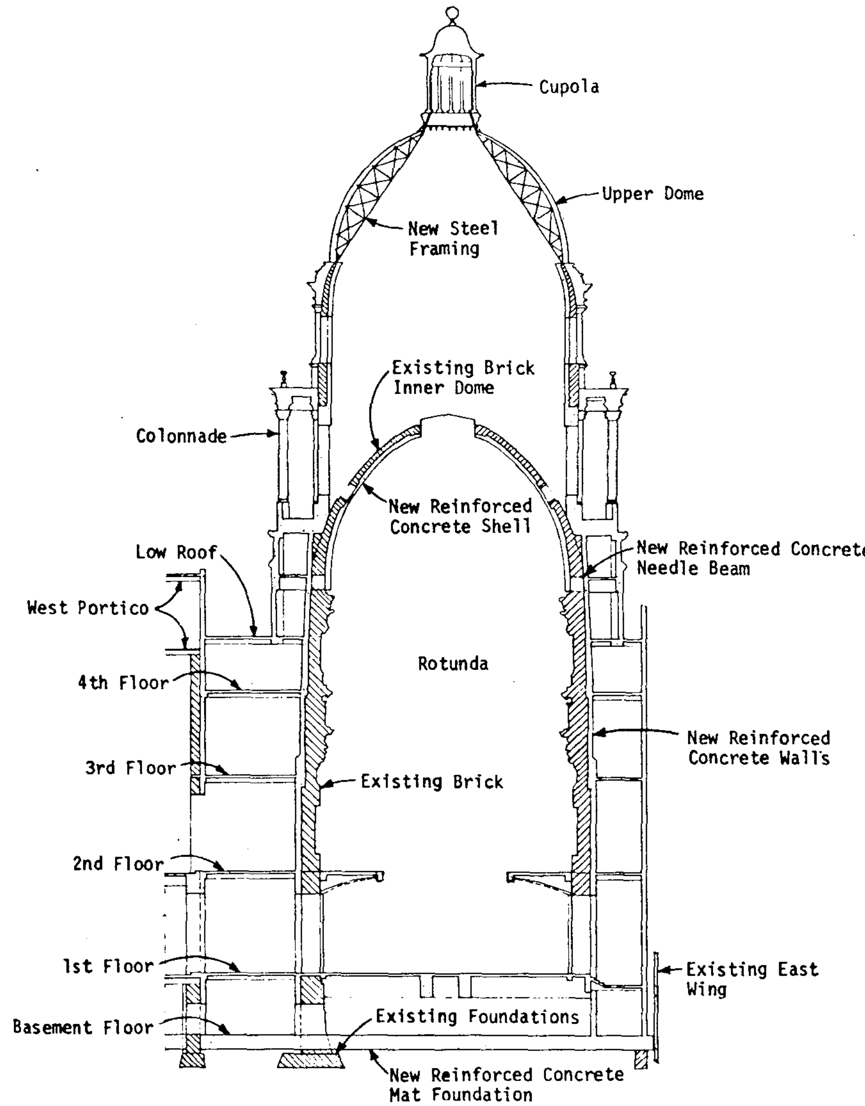


FIGURE 16 TYPICAL SECTION THROUGH DOMES

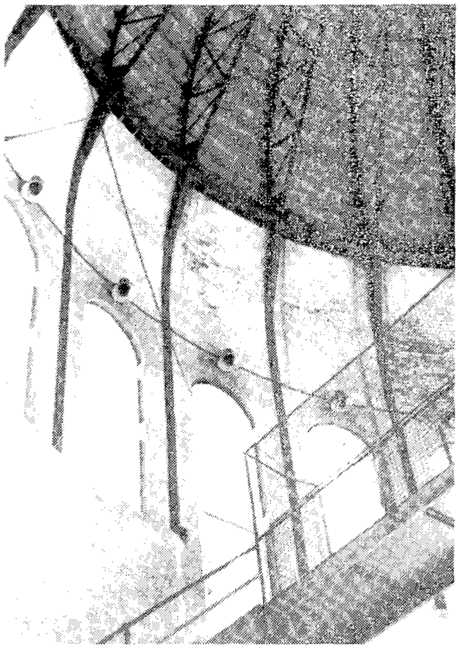


FIGURE 17 EXISTING TRUSSES AT UPPER DOME

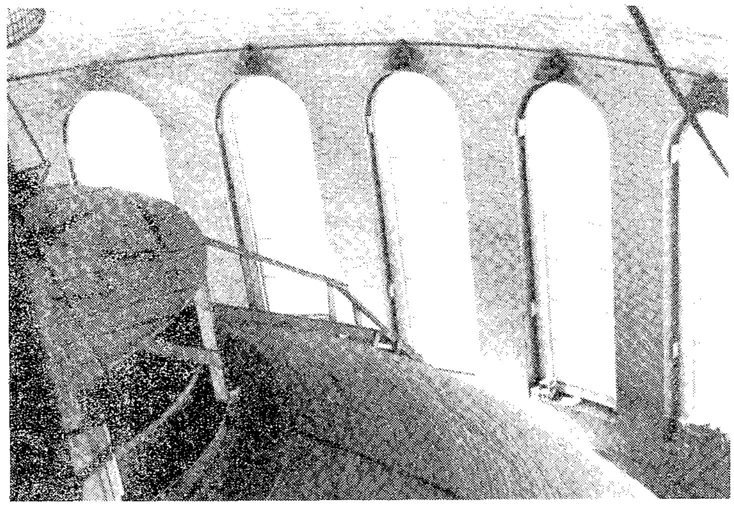


FIGURE 18 DRUM WALL SUPPORTING UPPER DOME, INNER DOME IN FOREGROUND

tions, as in an underpinning operation, because the structure above has to remain in place.

The colonnade roof and columns (Figure 19) are entirely cast iron except for the wood roof deck. The column, capitals, lintels, coffered soffit, and balustrade consist of hundreds of pieces of cast iron bolted together. This part of the structure will be strengthened in place with additional bolts and anchors to the gunite wall.

The brick walls supporting the colonnade will be replaced with concrete, which will be supported on the fifth floor. The old brick colonnade walls extended down to the fourth floor, where they were supported on riveted wrought-iron box beams. The elimination of the massive brick piers on the fourth floor will make available more usable space.

The fifth floor is at about the level of the lower chord of the trusses and is actually used only as an attic space, housing fans and ducts. Structurally, it is the diaphragm that supports the domes and rotunda walls and consequently carries a large shear. It requires a solid slab of 15-in. thickness.

The inner dome, visible in the foreground of Figure 18, is composed of unreinforced brick and is the only one of its kind in the country. For this reason, it will not be replaced with a new concrete dome as originally contemplated. Instead, a gunite shell will be applied directly to the underside of the existing brick and will be supported on a series of 24 needle beams

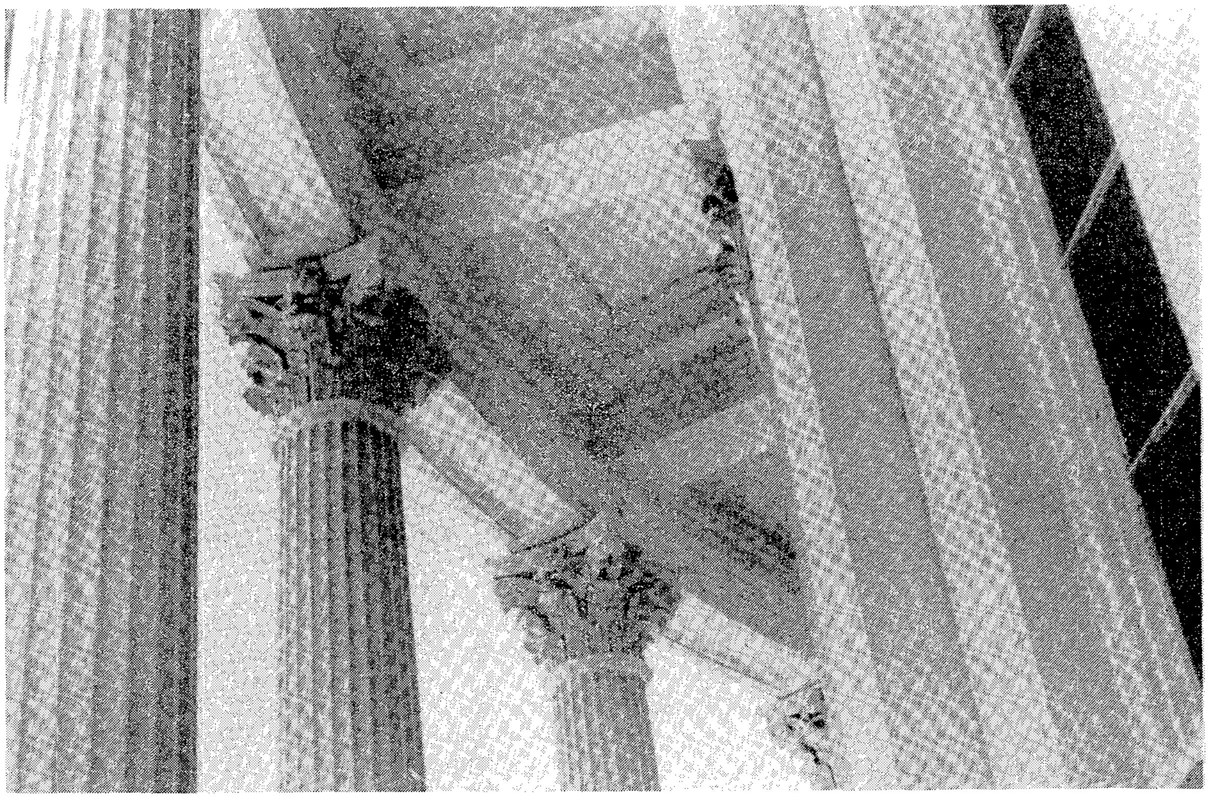


FIGURE 19 COLONNADE ROOF AND COLUMNS

spaced around the circumference of the rotunda wall. The needle beams will be cantilevered from the gunite wall on the outside of the brick rotunda walls, and the back end of the cantilever will be supported by the colonnade wall.

The rotunda gallery at the second floor will be replaced with concrete.

PRESENT STATUS OF WORK

The general contractor who has been on the job since late 1975 has been doing demolition work. Existing ceilings, mezzanines, and partitions have been removed. Window and door frames and other architectural items that will be reinstalled have been removed, cataloged, and stored. The exterior down to about the basement floor level has been excavated. A tower crane and man lift have been set up. Bracing frames (Figure 20), which will provide lateral support to the existing exterior walls and porticos during construction, are being installed. This work and the rest of the structural work will be done on a cost-plus basis with a negotiated guaranteed maximum price. This price, which includes contingencies, has been submitted to the state and is just under $15,000,000. Our estimate for the structural cost was $15,000,000.

The general contractor will take bids for all other subcontract work. This cost will not be known until the architectural drawings are finished sometime early next year. Special legislation was required to permit this arrangement of payment for construction.

FIGURE 20 BRACING FRAMES

PROJECT CREDITS

Owner - Joint Rules Committee of the State Legislature, John Worsley, Consulting Architect.

Architect - Welton Becket and Associates, Robert B. Mathews, A.I.A., Project Architect.

Consulting Structural Engineers - URS/John A. Blume & Associates, Engineers, Lloyd A. Lee, Project Manager.

Historical Consultant - Raymond Girvigian, F.A.I.A.

General Contractor - Continental-Heller/Swinerton & Walberg, A Joint Venture.

REFERENCES

1. *Seismic Study: West Wing, California State Capitol*, State of California, Office of Architecture and Construction, Sacramento, California, June 1972.

2. *Restoration and Development of the Capitol*, Welton Becket & Associates, Los Angeles, California, February 1975.

3. *Uniform Building Code*, International Conference of Building Officials, Pasadena, California, 1973, 1976.

4. *Recommended Lateral Force Requirements and Commentary* (fourth edition), Seismology Committee, Structural Engineers Association of California, San Francisco, California, 1975.

5. *Title 24, California Administrative Code*, Office of Administrative Hearings, Department of General Services, Sacramento, California, undated.

6. *Recommendations for Shape of Earthquake Response Spectra*, URS/John A. Blume & Associates, Engineers, prepared for the U.S. Atomic Energy Commission under Contract No. AT(49-5)-3011, WASH-1254, February 1973.

7. *Foundation and Seismic Investigation for the Existing and Proposed State Capitol Building Sites*, State of California, Department of Transportation, Sacramento, California, October 1974.

8. *Supplemental Foundation Investigation for the State Capitol West Wing Rehabilitation*, State of California, Department of Transportation, Sacramento, California, March 1976.

9. *Seismic Analysis Studies: California State Capitol, Restoration of West Wing*, URS/John A. Blume & Associates, Engineers, San Francisco, California, August 1976.

THE VETERANS ADMINISTRATION HOSPITAL STORY

by
Robert S. Henderson

In the earthquake of February 9, 1971, the Veterans Administration Hospital at San Fernando, California was so heavily damaged that it was subsequently demolished. Major loss of life took place in Buildings #1 and #2, which were designed and built in the early 1920's with no deliberate provision for lateral forces. These two buildings were flanked on either side by Buildings #41 and #43, which were built in the middle 1930's with resistance to lateral forces as prescribed in an early version of the Uniform Building Code. Despite the fact that Buildings #41 and #43 were similar architecturally to Buildings #1 and #2, they suffered only minimal damage.

The San Fernando earthquake, as you all know, was the most completely instrumented major earthquake up to that time. Furthermore, it came at a time when sophisticated digital computer methods and facilities were newly available. The seismographic records from many buildings pointed out serious deficiencies in the "rule of thumb" seismic design methods contained in the Uniform Building Code. Lateral forces greater than those contemplated by the Code could be expected, (as seismologists had long contended). Reductions of overturning forces by the UBC "J" factor were not justified, and the attenuation of seismic forces permitted in the UBC for tall buildings could not be defended.

The San Fernando earthquake also emphasized many of the lessons of the past. These included the vulnerability of nonstructural systems.

At the time of the earthquake the Veterans Administration was operating 168 hospitals of which 68 were located in UBC Zone 3 (areas having experienced major damage) or UBC Zone 2 (areas having experienced moderate earthquake damage).

The map contained in the Uniform Building Code was originally developed by Ted Algermissen of the U. S. Geological Service. It doesn't consider the frequency of recurrence of earthquakes.

Design by the Atomic Energy Commission is to meet the forces of a "maximum credible earthquake". The Veterans Administration has selected a 100 year frequency earthquake as its standard. This lesser standard requires some judgment in its estimation.

Under the direction of James Lefter, a comprehensive analysis of the earthquake risks at these 68 hospitals was undertaken, as well as the preparation and implementation of design standards for new hospitals, and for reinforcement of existing hospitals. The preparation of design standards and the analysis of existing buildings were both conducted primarily by private consultants. Many SEAOC members were involved.

The design standards resulting from these studies, and now in use by the Veterans Administration are:

1. Handbook H-08-8: "Earthquake Resistant Design Requirements for VA Hospital Facilities"
2. "A Study to Establish Seismic Protection Provisions for Furniture, Equipment and Supplies for VA Hospitals"
3. Construction Standard CD-54: "Post-Earthquake Emergency Utility Services and Access Facilities"
4. Construction Standard CD-55: "Earthquake-Resistive Design of Non-structural Elements of Buildings".

The Veterans Hospitals located in Zone 2 or Zone 3 in the Uniform Building Code have been or are being studied for possible need of seismic reinforcement. These studies have been divided into the following phases: (The completion of each phase is a decision point as to whether further study is necessary.)

1. Site Evaluation (Recommended design forces)

2. Phase I (Do the buildings meet VA standards?)

3. Phase II (Preliminary design and cost estimate)

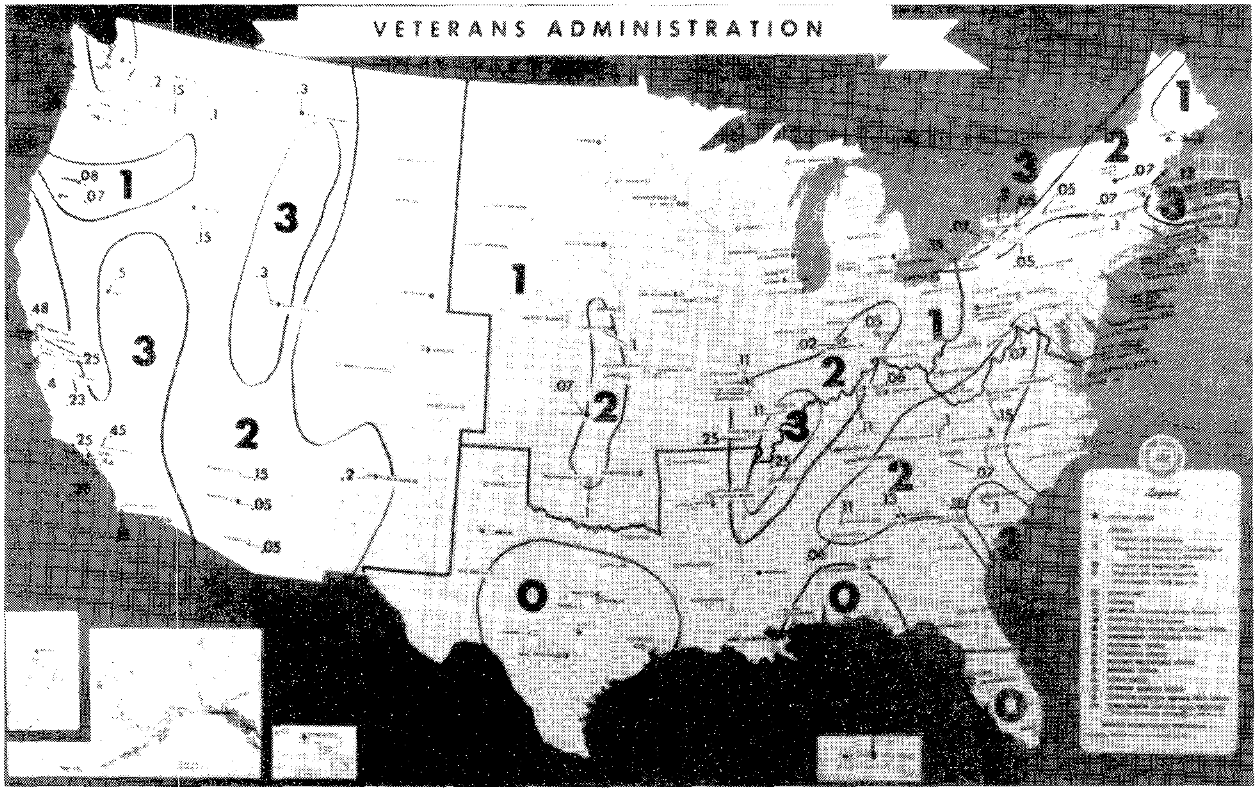
4. Contract documents and construction

The site evaluation studies have been performed by consultants who had previously performed similar studies for the Atomic Energy Commission or power companies seeking licenses from the AEC for projects close to the specific VA Hospital under consideration, or for the State of California Department of Water Resources. These studies take into account the amplification or attenuation of motion between bed rock and the foundation of the building, therefore no soil factor is included in the design procedure.

The site evaluation study resulted in a lateral acceleration, which when used with Handbook H-08-8 supplies a static equivalent design force.

The results of these studies were checked by the National Oceanic and Atmospheric Administration and by the United States Geological Service. Their logic was also investigated by the Veterans Administration staff. In two cases, the study predicted amplification of the bed rock motion in soft soil to levels of lateral acceleration exceeding 1.5 times gravity at foundation level. The basis for these studies was the Modified Mercalli assessment of earthquakes at a given site. It was assumed that this motion was generated by a specific geological fault; further assumed that such an earthquake could occur anywhere on that fault, and finally, that this would result in a given bed rock motion at the hospital site. This bed rock motion was then applied to a computer analysis of the dynamic response of a column of soil which resulted in the very high lateral acceleration at the surface.

The question the VA raised was: "If a lateral acceleration 20 times as great as that of the recorded earthquake could be expected at the VA Hospital site, why was a greater earthquake not observed at the Hospital site? Subsequent evaluation revealed errors in the computer analysis.



SLIDE #1

The bold numerals at each site indicate the basic lateral accelerations recommended by the site evaluation surveys.

Amax

Boston .10g

Charleston .25g

Memphis .25g

SLIDE #2

Some eastern and midwestern sites reflect significant basic lateral accelerations.

The Phase I studies involve analysis of the buildings at each hospital by methods contained in H-08-8 for forces defined by the site evaluation study. The reports contain a detailed description of the buildings, since many of them have been extensively modified; structural analysis, identification of deficiencies and hazards; and classification. The buildings were classified as:

1. Requiring immediate correction
2. Requiring correction or
3. In essential conformance with VA standards

One of the problems with many of the buildings is the extensive use of unreinforced masonry. It was found however, that this material usually could be considered capable of resisting out of plane forces (by wedging action), and contributing to the strength of the building for in-plane forces. The structural value of the masonry was determined by tests using methods defined in a report to the Veterans Administration entitled: "Evaluation of Strength of Existing Masonry Walls" by George Fattal and Lou Cattaneo of the National Bureau of Standards. Considerable study was made of the actions of infilled masonry in concrete frames. Even in cases where masonry would fail in shear, it will contribute to the stiffness of the frame, and reduce the bending moments in the columns. Based on research at the University of Illinois, a method of analysis was developed and programmed by James Lefter and Robert Holiday of the Veterans Administration.

Tests of a great many buildings indicated that the fundamental period of typical Veterans Administration buildings was virtually independent of the dimensions of the building. In H-08-8 the period is taken as .05 times the number of stories for shear wall and existing masonry buildings, .08N for concrete frame buildings, and .12N for steel frame buildings.

Analysis of earthquake motions that have occurred at the 68 Veterans Hospitals constitutes an analysis of most of the country. The study of these forces was an influence for the Veterans Administration Earthquake and Wind Forces Committee to include a provision in H-08-8 that all new buildings and major additions be designed to resist at least 5% gravity horizontally, regardless of their location.

Three methods for design are defined in H-08-8. Method #1 involves the use of the peak ground acceleration from the site evaluation study, a dynamic amplification (based on the period of the building), and a ductility factor which includes softening and energy dissipation characteristics.

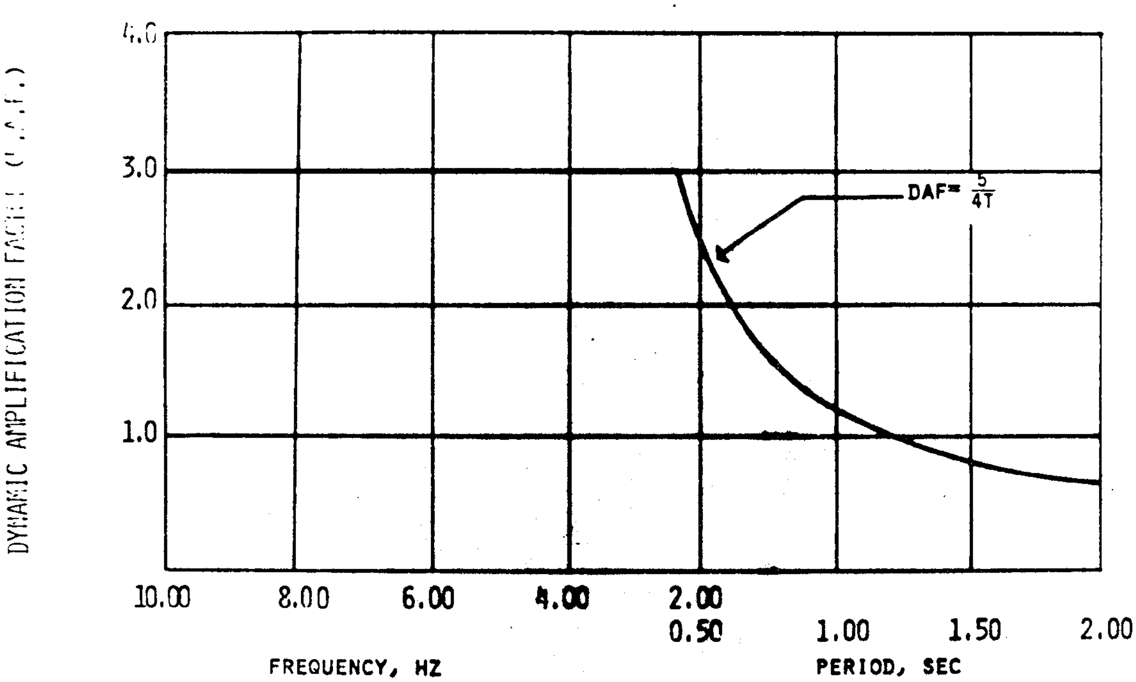


FIGURE 1 VARIATION OF D.A.F. WITH FREQUENCY AND PERIOD

SLIDE #3

TYPICAL α VALUES	
Shear walls	$\frac{1}{3}$
Ductile frame	$\frac{1}{4}$
Infill frame	$\frac{1}{2}$

SLIDE #4

UBC:

$$V = ZKCW$$

VA:

$$V = A_{max} \propto DAFW$$

SLIDE #5

Method #2 is a modal dynamic analysis and Method #3 is a time history method of analysis.

"Time History" analyses of old masonry buildings are not productive, since the period of the building changes with each cycle of the earthquake, as the building suffers progressive deterioration. The Veterans Administration has refrained from this kind of analysis.

In the Phase II studies, the consultant was asked to consider the various methods whereby a building could be strengthened to resist earthquake forces, to choose between these methods on the basis of cost and minimization of disruption of the hospital, and to estimate the construction cost of the method chosen.

As you well know, the construction cost of imposing seismic forces on the design of a new building may run from 2% to 5%. The cost of reinforcing an old building for the same forces may be ten times as much, or may even exceed the replacement cost of the building. In some cases corrective measures have been proposed that in effect: "destroy the building in order to save it".

Medical treatment procedures change so rapidly that extreme measures are being taken to ensure that new hospitals can be easily altered. The structural implications of these measures are dramatically demonstrated by the fact that column spacing in VA Hospitals a generation or two ago were commonly 12 ft X 20 ft. In the new VA Hospitals, column spacing is often 60 or 80 ft. In order to permit partitions to be moved almost at random, the hospital floors have in effect become bridges. Furthermore, the cost of operating a hospital will equal its replacement cost in a very few years. Consequently, just as with an old car, the time comes when the prudent thing to do is to "jack up the radiator cap and drive a new car under it". The Veterans Administration is building some new hospitals, and is discontinuing the use of some. The demand however is greater than the supply and many VA Hospital buildings are in use after more than fifty years. One building, still in use at the Van Couver, Washington Veterans Hospital was used as a quarters by President Grant when he was a Second Lieutenant.

In some cases it may develop that adverse seismic reports may be just one of many factors that contribute to a decision to abandon an old hospital. In other instances hospitals will be reinforced. More than 20 million dollars has been ear-marked for correction of seismic deficiencies in the next fiscal year. The choice of where to spend this amount is difficult. 20 million dollars falls far short of enough to correct all deficiencies at all hospitals, but of equal significance may be the essential obsolence of some hospitals for other reasons. It remains to be seen how much will be appropriated in the future, but it is significant that the VA is taking steps to reinforce buildings east of the Rocky Mountains. Few, if any, other organizations are doing as much.

Construction is under way on seismic reinforcement of the VA Hospital at Boise, Idaho. The exterior walls of these buildings are of unreinforced brick. The reinforcement includes the addition of a wythe of brick to the exterior of the building, with grout and reinforcing steel in the space between the new and existing brick, and mechanical ties between the reinforced walls and the existing diaphragms. This method of reinforcement is under consideration for use at other hospitals.

Corrective measures proposed for other hospitals include gunite concrete applied to exterior walls, interior vertical shear elements, and all of the other schemes familiar to you. The significant difference between the reinforcement of VA Hospitals and some seismic reinforcement projects is that few, if any, of the hospitals can be closed down during construction. Solutions that can be constructed primarily on the outside of the buildings, therefore, have a strong appeal.

In summary, the Veterans Administration has developed good seismic design standards for buildings. These include the design of both structural and non-structural elements, as well as provisions to keep hospitals operational in the wake of earthquakes. These standards are being applied to new buildings. The old buildings in UBC Zones 2 and 3 have been analyzed by these standards. As funds are available, buildings shown to be deficient are being strengthened.

THE CENTER FOR EDUCATIONAL DEVELOPMENT

A Case Study in Medium Scale Seismic Upgrading

by

Harold A. Davis

Rutherford & Chekene, San Francisco

Many engineers are now familiar with the problems and technical details of rehabilitation work and seismic upgrading, both for public agencies and private owners. The project I wish to review includes some unique features deserving our examination for ideas on design investigation, analysis and implementation.

Figure 1

The Center for Educational Development is located in the Mission District of San Francisco, at Fifteenth and Folsom Streets. The site borders a residential, small commercial retail area and light-to-medium industry. Main traffic thoroughfares such as freeways, bridges and transit facilities like BART (under construction when our involvement with this building started) are nearby. When planning began, the San Francisco Chamber of Commerce actively campaigned to save older existing structures by urging real estate investors to consider rehabilitation as an alternative to the demolition of older, neglected buildings, or to relocation outside the city itself. The Cannery and Ghirardelli Square projects had recently been completed and stood as guideposts for future efforts along the same lines. (See Figure 1)

The problem, as it faced us in 1969, was this: the building is six stories, containing some 300,000 square feet of floor space, measuring approximately 190 feet by 255 feet. Constructed in 1926-27, it was the second structure to occupy the same site, built and used as a warehouse for merchandising and retail concerns such as Woolworth's. In 1969 it stood empty and neglected, a visual deterrent to change.

At this point, Far West Laboratory, an energetic team of educational research development professionals from Berkeley, were looking for space to house 75,000 square feet of offices and an experimental school in the same building! Thanks to a happy connection with the Office of Education of HEW, this building was purchased with part of a $4.75 million federal grant. The grant was to pay for all construction work on the condition that there be no major disruption to the building, so that unused space retain a certain flexibility and ease of future occupance. The architect and consultants were selected and discussions began on the basic philosophy or design approach to this problem.

The first questions are familiar to many of you who have experience in rehabilitation work: Does it meet the code? How much do we have to do? What's the minimum? How much will it cost? Why are your fees so high? These are certainly important and basic considerations, but the answers to such questions alone do not provide a sufficient basis for proceeding on a project of this scale. The design professions (and I include structural engineers in this category) need to be made more aware of issues related to planning, esthetics, community impact, and flexibility for future uses. What I propose to discuss is how our solutions to these questions were developed, integrated with other design requirements, and implemented to produce the final happy result.

We began with a set of drawings for the existing construction which provided enough information to allow preparation of comprehensive calculations of the existing capacity for vertical loads. The building was designed for an unreduced live load of 200 psf on all floors, a factor which we were able to use later. We performed the usual ritual cores, measurements, chases in walls, and water leveling the floors. Our first surprise came from the results of the water leveling -- as much as nine inches of differential settlement, and no major, fundamental cracking to show for it. (See Figure 2)

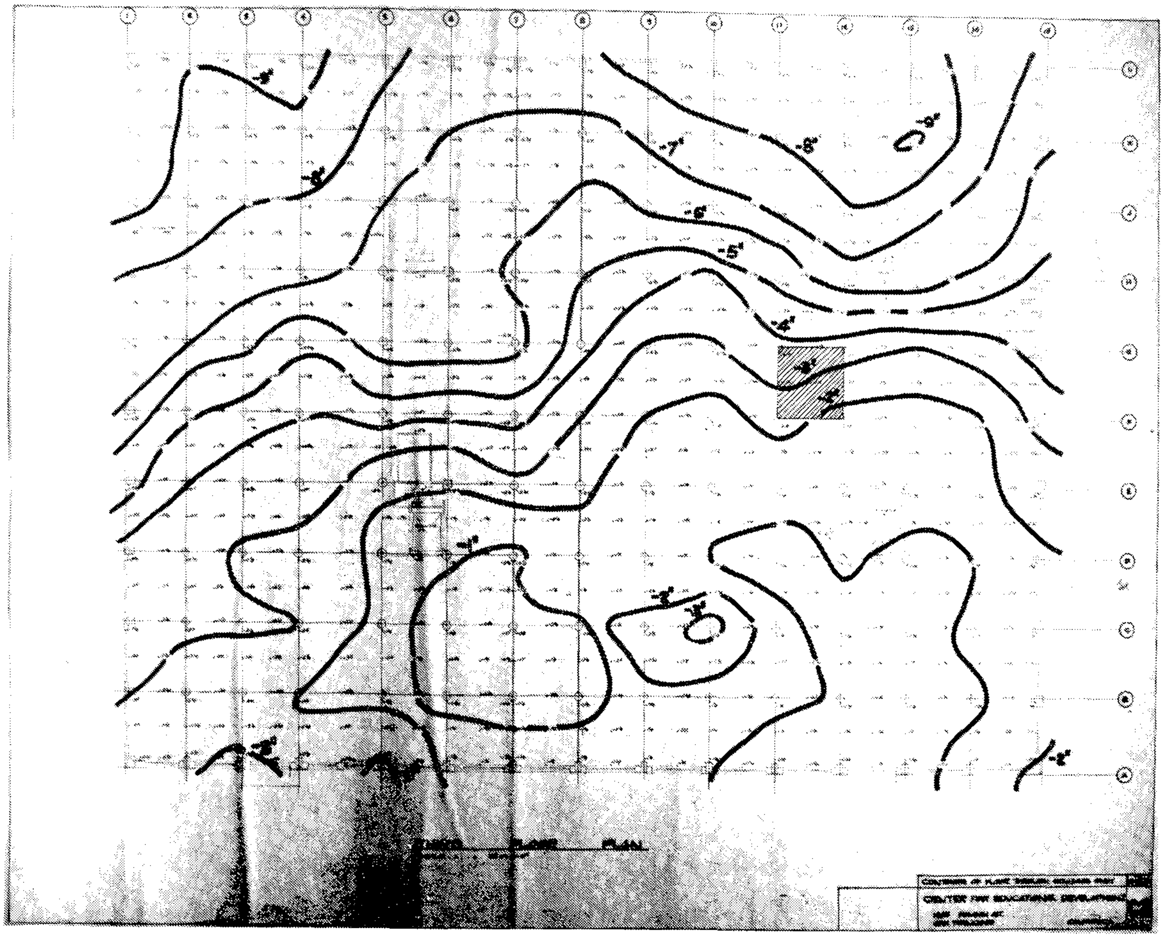


Figure 2

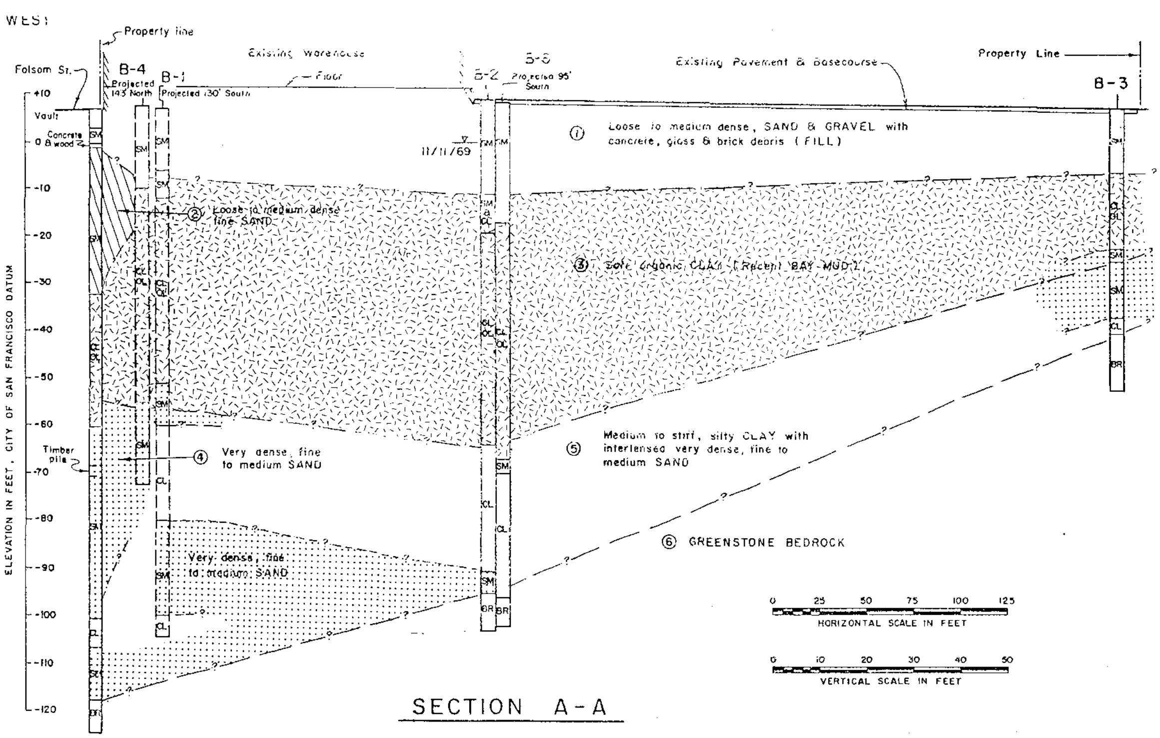


Figure 3

The structure is founded on wood piles, of unknown depth and tip size. We verified the number, butt sizes, and conditions by visual observation in several exploration pits. An initial soils investigation was performed by Shannon & Wilson, to see if correlation could be established between the settlement picture and the use of piling, and to determine if we were dealing with a building doomed to permanent or short-term foundation failure. (See Figure 3)

Results showed that there could indeed be a logical explanation for the differential, but without information on driving, depth or size, we could reach no definite conclusions, especially if new dead loads were to be required. As the preliminary thinking of structural concerns proceeded, it became apparent that dead load would be increased, and therefore two pile load tests were performed. (See Figure 4)

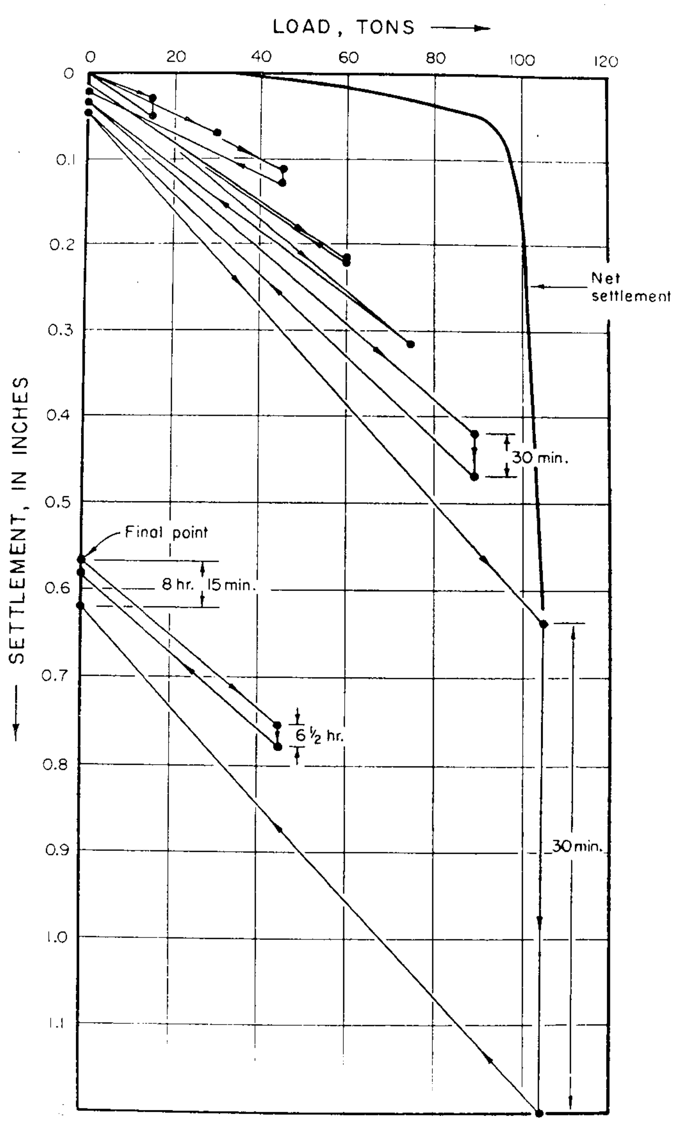


Figure 4

These tests indicated that substantial increase in dead load and real live load were possible. Had this not been the case, this project could have ended at that point. We concluded then that the settlement was gradual, due to differing conditions for pile bearing (and possibly driving) and that the rate of settlement had decreased considerably because of the age of the fill over the slough deposits. We cored the ground floor slab and learned that the ground had settled away from the slab in areas of the building which were 'high', but was right up against the slab-on-grade where the building was most depressed. Correlation at last!

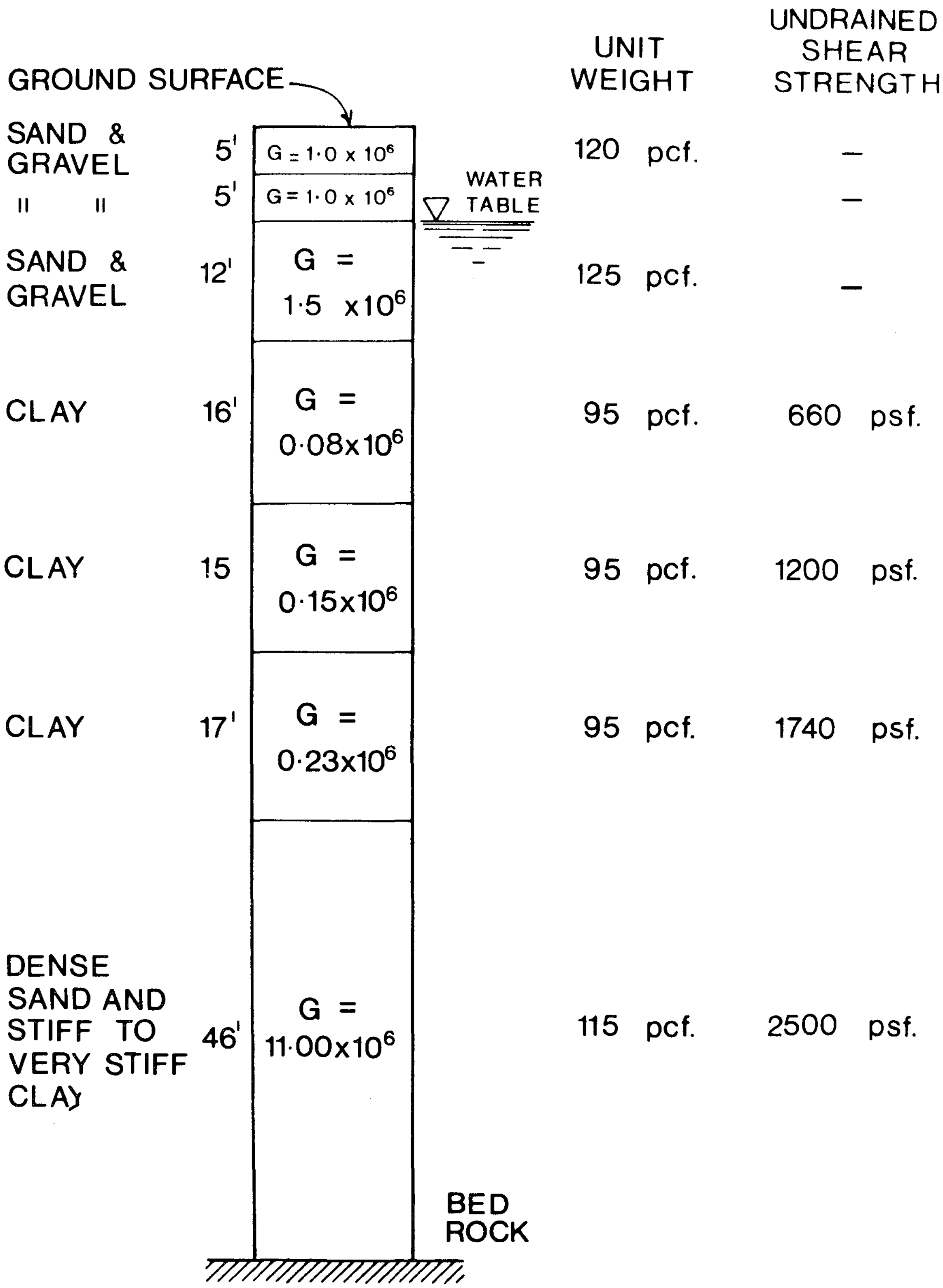
At this time, the reviewers in Washington raised the question of liquefaction of the softer sands due to earthquake motions. Because borings were taken external to the building, some softer materials did show up, but during the excavation of the pile pits it was noted that the sands were densely packed for a considerable distance beyond the zone of the pile groups. Since the pile clusters are close together in relation to the size of the groups, we concluded that there was little or no liquefaction potential. Professor H. Bolton Seed, of the University of California at Berkeley, was our consultant on this and other questions of soils engineering. Federal review authorities were interested also in the question of the relationship between our proposed solution to upgrading and the fact that the building was located in a site area containing a significant depth of muds, and sands over bedrock. A dynamic response analysis was undertaken for the site, using characteristics of bedrock motion based on fault breaks of ten miles and seventy miles distant, of Richter magnitude 8.25. Based on techniques recently developed at that time by Seed and other colleagues, characteristics were determined for the earthquakes as shown in Figure 5.

	Fault Break @ 10 mi.	Fault Break @ 70 mi.
Maximum Acceleration	0.42 g.	0.095 g
Maximum Period (for 5 sec. interval)	0.58 sec.	0.78 sec.
Minimum Period	0.41 sec.	0.62 sec.
Total Time	60 seconds	30 seconds

Figure 5

The idealized soil layers, for the purposes of this analysis, were determined as shown in Figure 6, next page.

Using the computer technique developed by Seed, Idriss and Dezfulian, the characteristics of the response of this idealized layer were calculated due to the input motions described previously. For the purposes of this qualitative analysis, we concluded that a 'stiff' short-period structure such as a shear-wall type solution would be most suitable for the expected characteristics of ground motion. Code predictions of period indicated a T of approximately 1/4 second, the peaks in the response spectra were grouped at a value near one second.



IDEALIZED SOIL LAYER

Figure 6

At this point, we were convinced that certain engineering factors should be considered major parameters to any proposed architectural solutions:

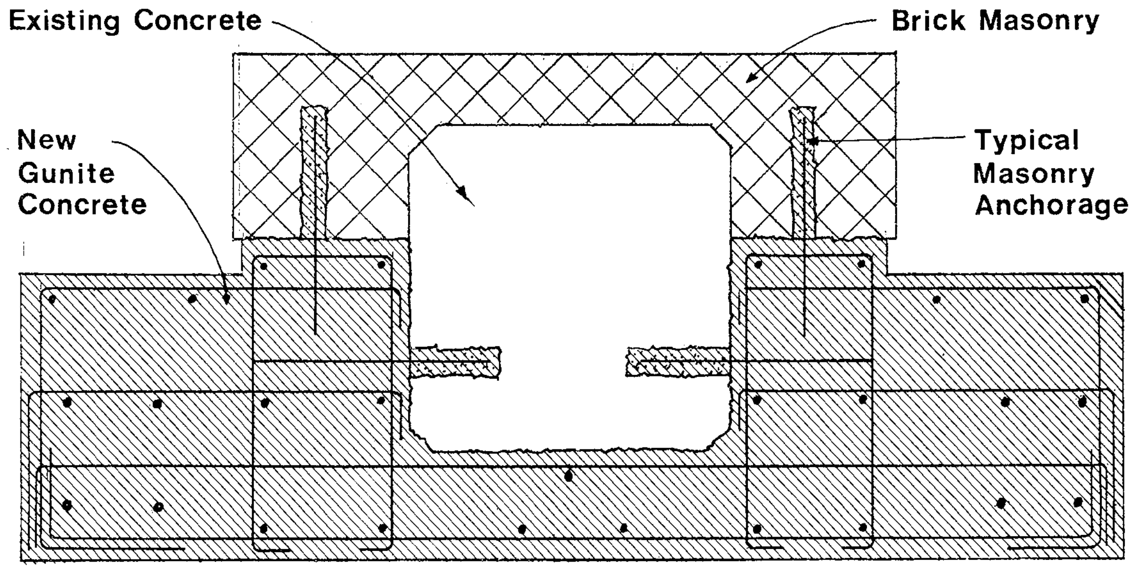
A. Dead load could be added, but a limit was set which would require some trade-off in weight with the existing loads.
B. Shear-wall type solutions were desirable for lateral forces.
C. Floor slabs were adequate to support the 50 psf and 100 psf loads, based on the slab load test.
D. Application of new dead and real-live loads were required to be distributed evenly over the building to avoid costly foundation revisions.

About six months after starting the project, we began to firm up our ideas on the system of lateral bracing to be used. It was clear to everyone involved that the solutions to the engineering problems would have a major impact on architectural planning and design. We operate on the conviction that in a rehabilitation project like this, the structural engineer occupies the position of leadership, and if he is sensitive to the overall needs of the project his work is in the nature of a creative contribution rather than merely a supply of services. In response to the original appearance of the building and the fact that projected occupancies were offices and schools, we proposed that the lateral scheme use the form of the building as an outline, and that the entire exterior wall become a new concrete shear wall. From the sixth floor down, pier and spandrel elements become larger and larger; window and door openings diminish. At first, no judgement was made whether the concrete elements would be poured-in-place or gunite, but as the design progressed, gunite clearly seemed the more appropriate material.

The design detail sections are worth examining -- not to count reinforcing bars, but to demonstrate the relationship between our design and the original construction. Figures 7 and 8 show a Pier Section and a Spandrel Section, both on the first floor.

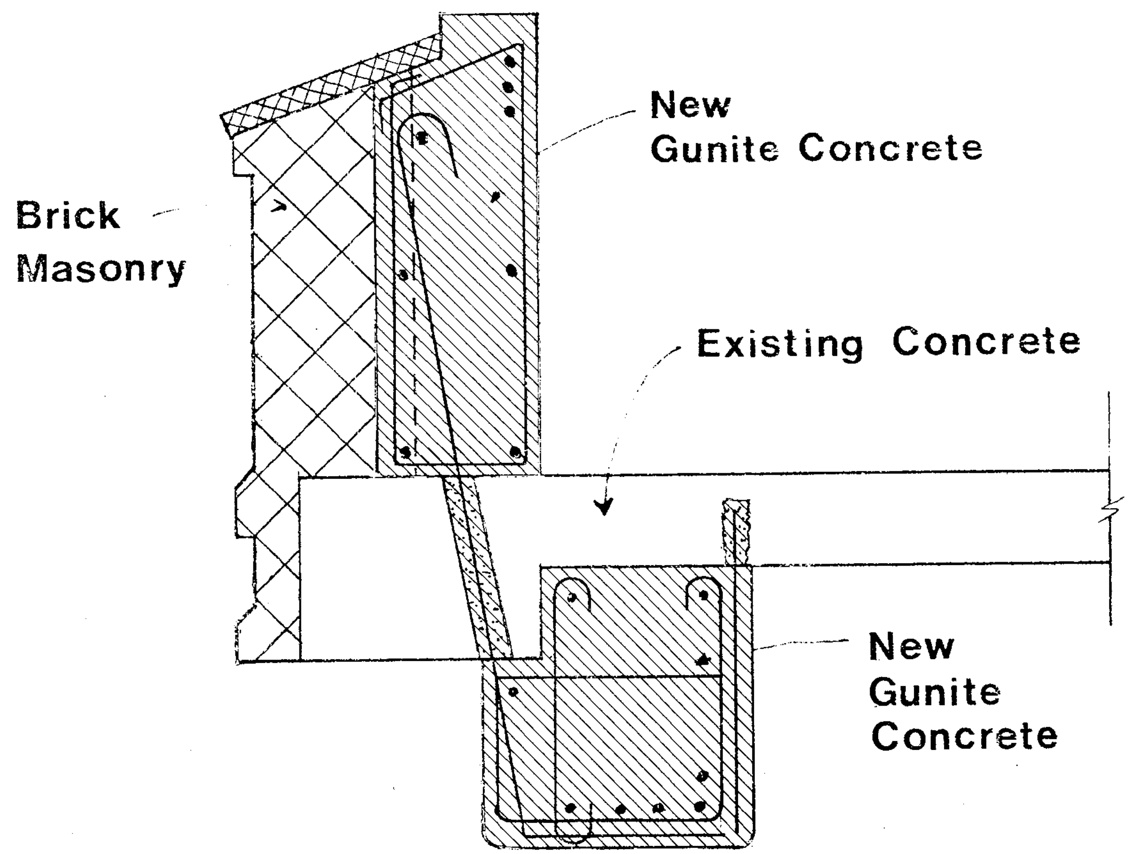
Foundation work was limited to the construction of a major perimeter grade beam to receive the nearly vertical wall elements and distribute the load to the existing pile caps. The ground floor slab had failed in many bays due to the ground settlement, so a new flat slab was designed, with the old slab cut away from the columns to remove any possible load transfer and to provide space for a drop panel.

The decision to use gunite rather than concrete was based partly on considerations of economy in formwork, but also was influenced by the necessity of achieving positive bond in a variety of surfaces. Some of these surfaces would require temporary ports, chases and intricate placing schemes to receive concrete properly and develop bond.



PIER SECTION- FIRST FLOOR

Figure 7



FIRST FLOOR SPANDREL SECTION

Figure 8

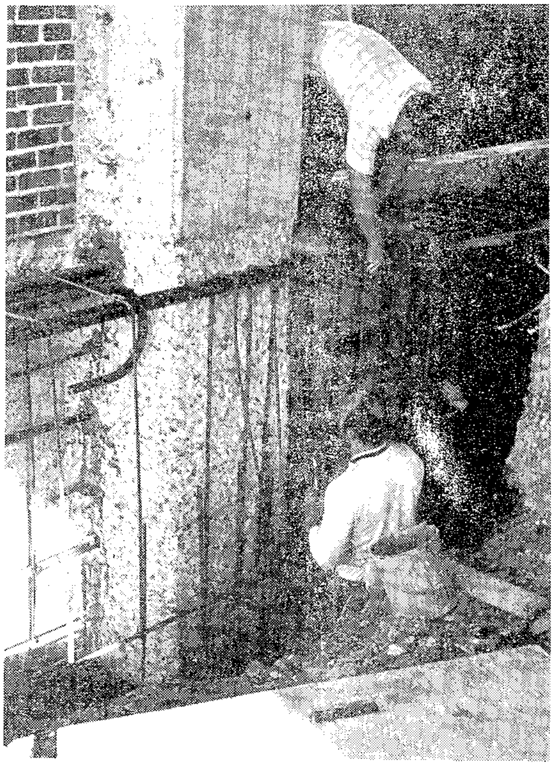


Figure 9

Figure 9 shows the construction work itself. Foundation work was the first phase following demolition. While foundation construction proceeded, holes and chases were drilled in slabs, columns, and brick walls throughout the building (See Figure 10)

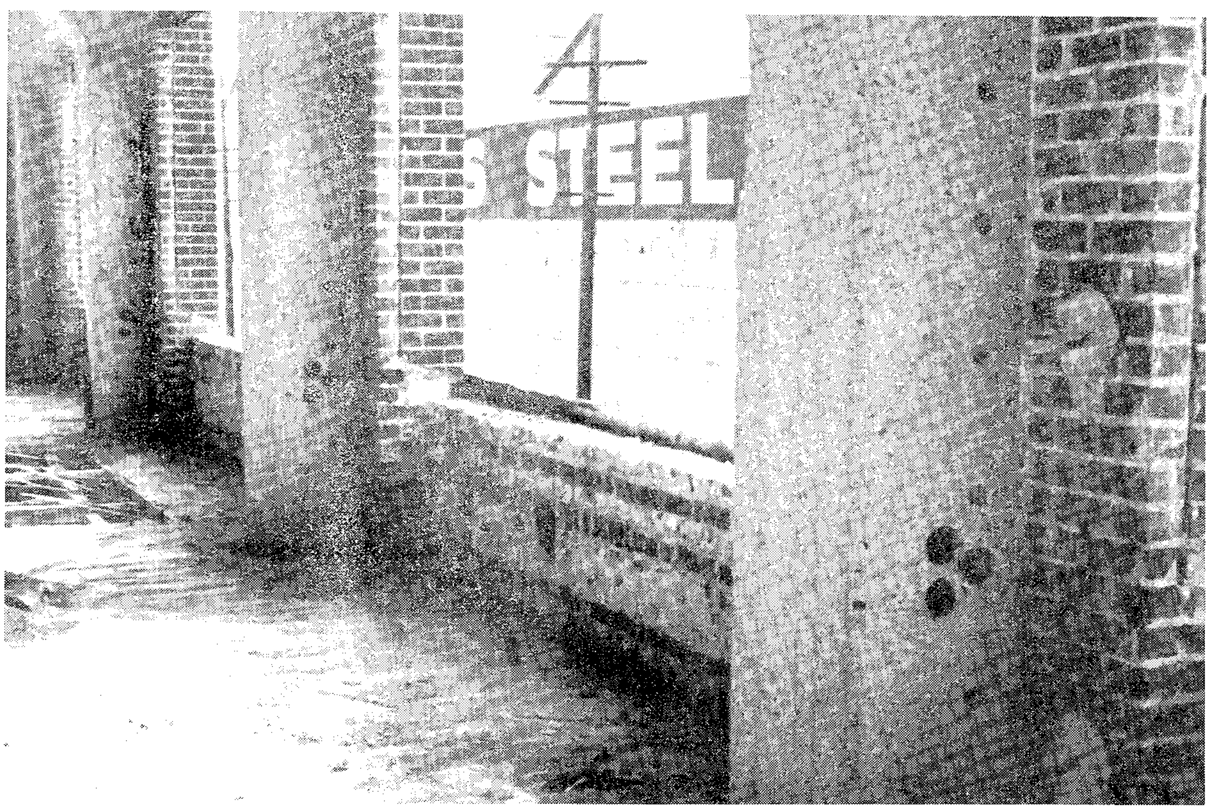


Figure 10

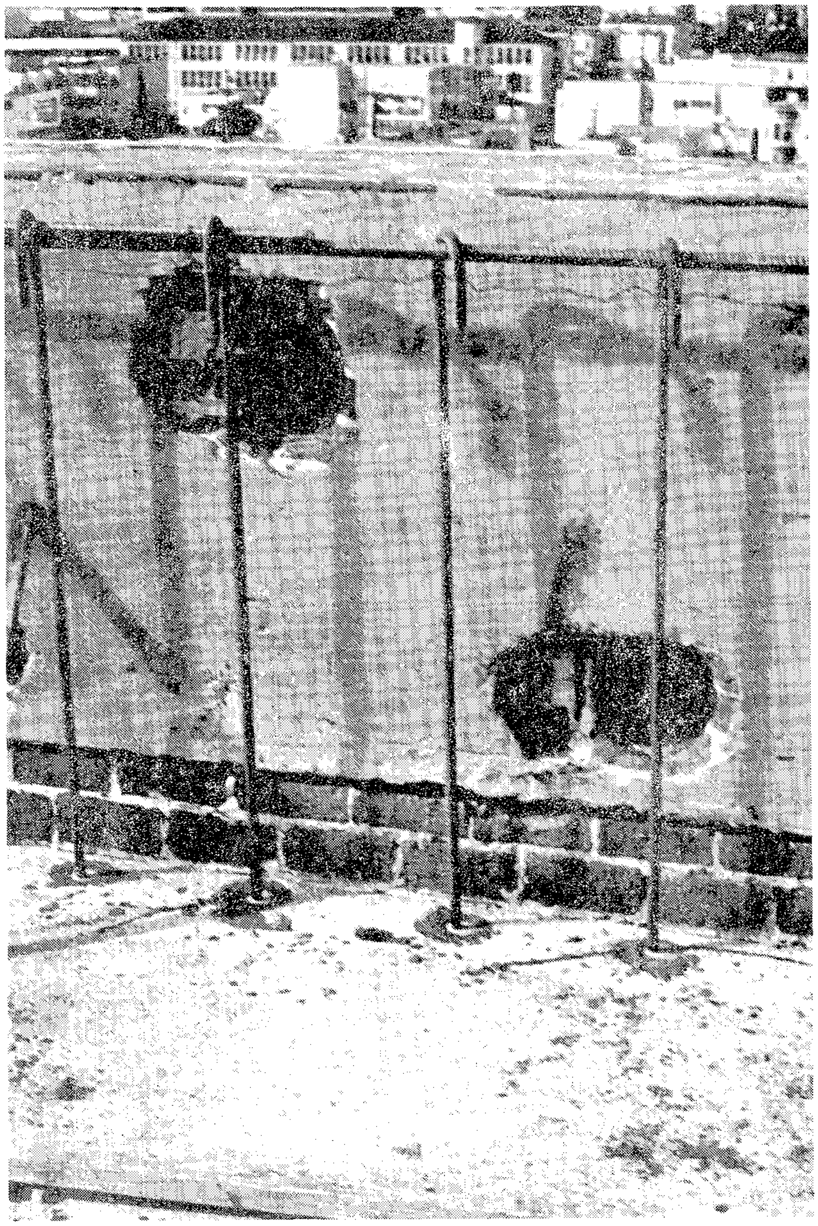


Figure 11

The installation of reinforcing was a major, time-consuming portion of the work. Because of the number and distribution of the bars, and clearances necessary for gunite work, very close coordination between the men drilling the holes and the reinforcing installation was necessary. Our construction observation required one man to be assigned on a continuing basis to assist the owner's project representative and review conflicts as they developed. (See Figure 11)

As soon as possible after the completion of reinforcing, screed sires were set, back forms installed and gunite applied. Although difficulties in mixing and application did occur at first, they were soon overcome. Only the workmanship of the nozzlemen remained problematic. The key to gunite, of course, is the proper compaction of the material in place, and the elimination of rebound material from the section. OSA requirements determined the minimum core testing, but on several occasions, additional cores were taken because of low breaks or cores which indicated the inclusion of rebound.

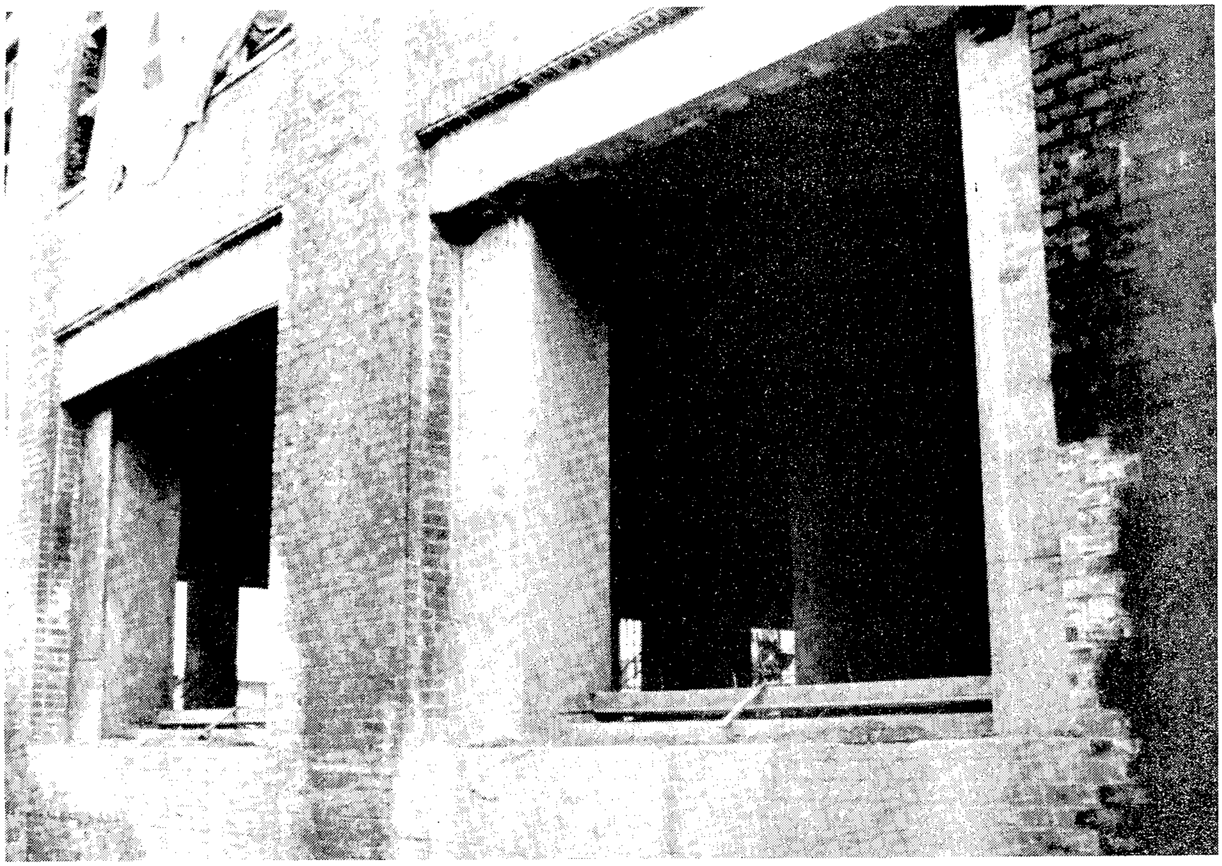


Figure 12

The finish coat, applied by plasterers, was between 1/4" to 1/2" in thickness. This dimension was originally intended to be an integral application with a wood float finish, but nozzlemen simply were not able to control the tolerances to that degree, so plasterers were called in to apply the finish. Figure 12 shows the general shape and appearance of spandrel and pier section after the forms are stripped.

The finished building houses offices and mechanical service areas on two floors (5th and 6th), a television studio and combined joint-use space on the first floor for community groups and the Laboratory. There is also a pre-school children's center on the first floor.

Our solution to the seismic upgrading has produced a scheme which did not significantly alter the exterior appearance, provided total interior flexibility for space planning, and met the criteria for OSA-approved school occupancy without further structural modifications. School occupancy of the second, third and fourth floors has not yet occurred, but numerous proposals have been discussed. If the community board and the Laboratory were to agree, I have no doubt that other, non-educational tenants would readily locate here. A Mexican museum has recently been established in the building, exhibiting Mexican and Mexican-American paintings, traditional crafts, and other works of art.

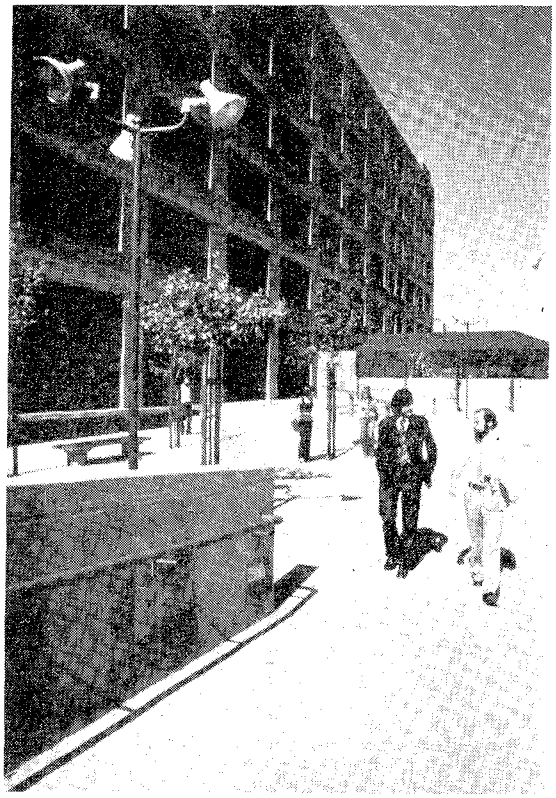


Figure 13

Figures 13 and 14 show some aspects of the completed work, and demonstrate how the structural work was integrated into the architectural solution.

Total cost of the construction was $3,500,000 or roughly $11.50 per square foot, bid in 1971. The contractor was DeNarde Construction Company of San Francisco.

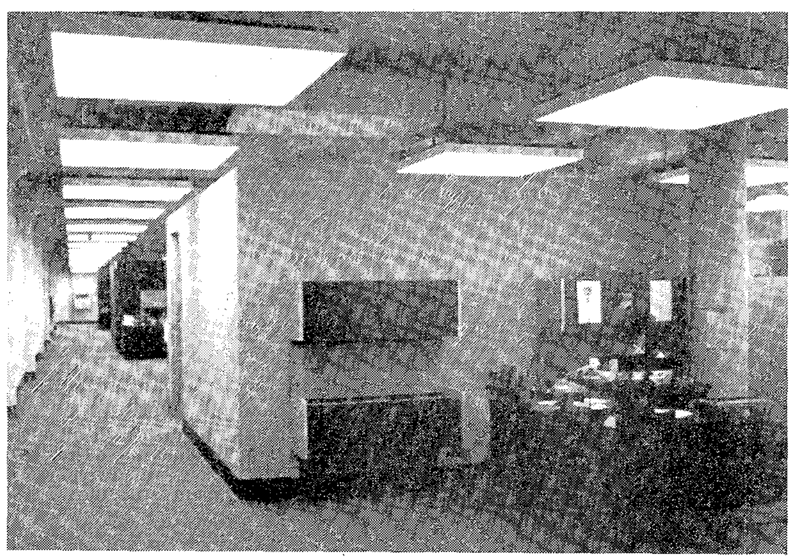


Figure 14