NBS-114A (REV. 9-78)

U.S. DEPT. OF COMM.
BIBLIOGRAPHIC DATA SHEET

1. PUBLICATION OR REPORT NO. NBSIR 80-2111-3

2. Gov't. Accession No.

3. PB81-187569

4. TITLE AND SUBTITLE

Review and Refinement of ATC 3-06 Tentative Seismic Provisions
Report of Technical Committee 3: Foundations

5. Publication Date October 1980

6. Performing Organization Code

7. AUTHOR(S)

Lawrence A. Salomone

8. Performing Organ. Report No.

9. PERFORMING ORGANIZATION NAME AND ADDRESS

NATIONAL BUREAU OF STANDARDS
DEPARTMENT OF COMMERCE
WASHINGTON, DC 20234

10. Project/Task/Work Unit No.

11. Contract/Grant No.

12. SPONSORING ORGANIZATION NAME AND COMPLETE ADDRESS *(Street, City, State, ZIP)*

Federal Emergency Management Agency
Washington, DC 20472

13. Type of Report & Period Covered Final

14. Sponsoring Agency Code

15. SUPPLEMENTARY NOTES

☐ Document describes a computer program; SF-185, FIPS Software Summary, is attached.

16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

The Tentative Provisions for the Development of Seismic Regulations for Buildings were developed by the Applied Technology Council to present, in one comprehensive document, current state-of-knowledge pertaining to seismic engineering of buildings. The Tentative Provisions are in the process of being assessed by the building community. This report is one of a series of reports that documents the deliberations of a group of professionals jointly selected by the Building Seismic Safety Council and the National Bureau of Standards and charged with reviewing the Tentative Provisions prior to conducting trial designs.

This report documents the activities of Technical Committee 3: Foundations. Other committee reports are similarly available. The task of Technical Committee 3 was to review and refine Chapter 6, Soil-Structure Interaction and Chapter 7, Foundation Design Requirements in the ATC report (NBS SP-510) entitled, "Tentative Provisions for the Development of Seismic Regulations for Buildings." Two meetings were held. The opening meeting of the group was on December 11, 1979, and the concluding meeting was on February 5, 1980. The minutes of these meetings and the findings/recommendations of Technical Committee 3 are presented in this report. These recommendations were made to the parent group, the Joint Committee on Review and Refinement, and their action on these recommendations is documented in a companion report.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

Buildings; design; earthquakes; engineering; foundations; professional practice; provisions; soil-structure interaction; standards

18. AVAILABILITY XXX Unlimited

☐ For Official Distribution. Do Not Release to NTIS

☐ Order From Sup. of Doc., U.S. Government Printing Office, Washington, DC 20402, SD Stock No. SN003-003-

☒ Order From National Technical Information Service (NTIS), Springfield, VA. 22161

19. CURITY CLASS (IIS REPORT) CLASSIFIED

20. SECURITY CLASS (THIS PAGE) UNCLASSIFIED

21. NO. OF PRINTED PAGES

22. Price

USCOMM-DC

NBSIR 80-2111-3

REVIEW AND REFINEMENT OF ATC 3-06
TENTATIVE SEISMIC PROVISIONS

Report of Technical Committee 3: Foundations

Richard Simon, Chairman, Association of Soil and Foundation Engineers
Lawrence Salomone, Secretary, National Bureau of Standards
James G. LaBastie, American Society of Civil Engineers
William Travis, Structural Enginners Association of California
Joseph V. Tyrrell, Interagency Committee on Seismic Safety in Construction
Henry Degenkolb, Applied Technology Council
Leroy Crandall, Building Seismic Safety Council

Prepared for use by the:

Building Seismic Safety Council

Sponsored by:

Federal Emergency Management Agency

Center for Building Technology
National Bureau of Standards
Washington, D.C. 20234
October 1980

U.S. DEPARTMENT OF COMMERCE, Phillip M. Klutznick, Secretary
Luther H. Hodges, Jr., Deputy Secretary
Jordan J. Baruch, Assistant Secretary for Productivity, Technology and Innovation
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, Director

ABSTRACT

The Tentative Provisions for the Development of Seismic Regulations for Buildings were developed by the Applied Technology Council to present, in one comprehensive document, current state-of-knowledge pertaining to seismic engineering of buildings. The Tentative Provisions are in the process of being assessed by the building community. This report is one of a series of reports that documents the deliberations of a group of professionals jointly selected by the Building Seismic Safety Council and the National Bureau of Standards and charged with reviewing the Tentative Provisions prior to conducting trial designs.

This report documents the activities of Technical Committee 3: Foundations. Other committee reports are similarly available. The task of Technical Committee 3 was to review and refine Chapter 6, Soil-Structure Interaction and Chapter 7, Foundation Design Requirements in the ATC report (NBS SP-510) entitled, "Tentative Provisions for the Development of Seismic Regulations for Buildings." Two meetings were held. The opening meeting of the group was on December 11, 1979, and the concluding meeting was on February 5, 1980. The minutes of these meetings and the findings/recommendations of Technical Committee 3 are presented in this report. These recommendations were made to the parent group, the Joint Committee on Review and Refinement, and their action on these recommendations is documented in a companion report.

Keywords: Buildings; design; earthquakes; engineering; foundations; professional practice; provisions; soil-structure interaction; standards

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 General

The Tentative Provisions for the Development of Seismic Regulations were developed by the Applied Technology Council (ATC) in an effort that included a wide range of experts in the actual drafting of the provisions. Two external review drafts were circulated to a large portion of the interested and informed community of eventual users. However, because the Tentative Provisions were innovative, doubts about them existed. Consequently, an attempt was made to investigate these doubts and to improve the Tentative Provisions where possible before an expensive assessment of the Tentative Provisions was undertaken by conducting trial designs.

This review and refinement project was planned and conducted by the National Bureau of Standards with the advice and approval of the Building Seismic Safety Council, a private sector organization formed in 1979 for the purpose of enhancing public safety by providing a national forum to foster improved seismic safety provisions for use by the building community.

The assessment of the Tentative Provisions was performed using the committee structure shown in figure 1. Nine Technical Committees were formed with interests that collectively cover the Tentative Provisions. The Joint Committee on Review and Refinement consists of all voting members of the Technical Committees. The chairmen of the Technical Committees form a Coordinating Committee.

Membership of each Technical Committee is made up of representatives of organizations that have particular interest in the Tentative Provisions; the participants are listed in the committee membership section of this report.

In addition to the voting members, each Technical Committee includes a non-voting member from each of the following organizations: The Applied Technology Council (ATC), the Building Seismic Safety Council (BSSC) and the National Bureau of Standards (NBS). The ATC representative served as a technical resource to the committee since he was closely involved with the development of the provisions of interest to the committee. The NBS representative was the technical secretary throughout the effort. The BSSC representative provided a link with the Building Seismic Safety Council, which will be involved in trial designs and evaluations.

1.2 Committee Summary

This report documents the activities of Technical Committee 3: Foundations. Other committee reports are similarly available.

The task of Technical Committee 3 was to review and refine Chapter 6, Soil-Structure Interaction and Chapter 7, Foundation Design Requirement, in the ATC report (NBS SP-510) entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings." Two meetings were held. The opening meeting of the group was on December 11, 1979 at the National Bureau of Standards in Building 226, Room B-113. The topics discussed included:

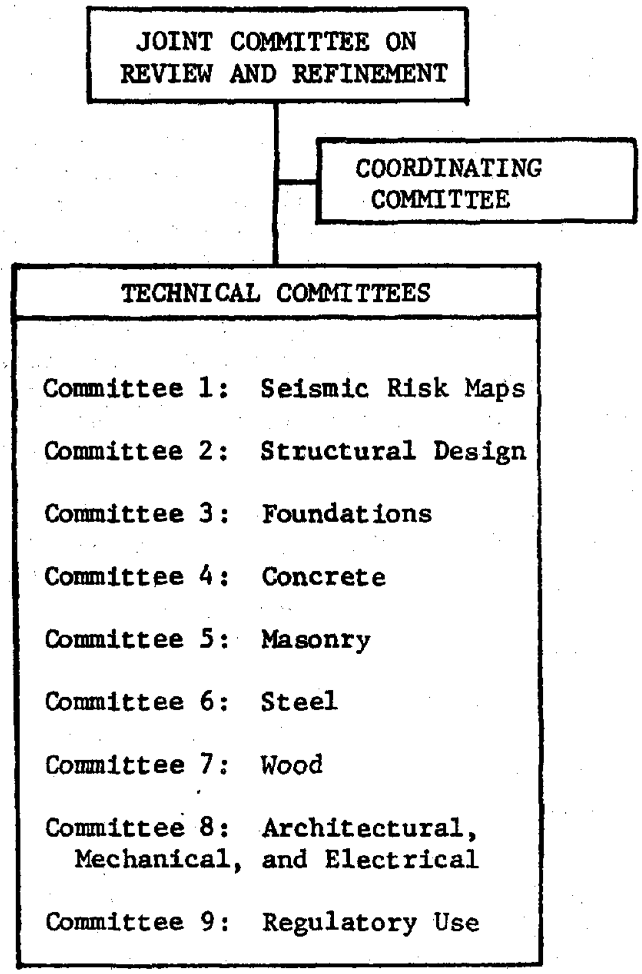

Figure 1: Committee Structure

1) Introduction and Orientation by NBS Secretary
2) Selection of Chairman
3) Selection of Representative to Committee 2
4) Selection of Date and Location for Next Meeting
5) Establishment of Work Plan and Preliminary Discussion of Chapters 6 and 7

The minutes of the opening meeting are provided in Section 3 of this report.

The second meeting of the group was held on February 15, 1980 at Law Engineering Testing Company, 109 Inverness Drive East, Englewood, Colorado. The meeting served as a workshop to discuss comments received by February 15, 1980 on Chapters 6 and 7 and to develop recommendations for revising these chapters. Comments were provided to the members of the committee in letters from Lawrence Salomone to members of Technical Committee 3 dated January 16 and January 31, 1980. Also, late comments received prior to February 15, 1980 meeting were distributed to committee members at the opening of the second meeting.

The second meeting was conducted in the following manner:

1) Each of the comments received were discussed and a final position of the committee developed. The final position consisted of a recommendation for change for the section in question or a recommendation that the section not be modified.

2) Following development of the final position of the committee a vote was taken.

The results of this meeting and the items voted on are presented in detail in the minutes of the second meeting provided in Section 3. With the completion of Meeting 2 the review of Chapters 6 and 7 by Technical Committee 3 was completed. The recommendations of Technical Committee 3 are presented in Section 2.

1.3 Chairman's Statement

In adopting positions concerning the various sections covered in Chapters 6 and 7 of the tentative provisions, the Committee tried to improve clarity and specificity of the clauses (e.g.: Section 7.2.2, Paragraph #1 - changing "at the elastic limit" to "at acceptable strains" to describe allowable stress on soil). Where analytical developments or concepts permit a better description of the desired structural performance, refined definitions have been recommended (e.g.: require piles to sustain soil-determined displacements rather than specify that the piles resist flexure induced by lateral soil pressures).

With respect to design of piles for earthquake, the Committee adopted a position based primarily on the experiences gained following the San Fernando and Anchorage earthquakes. For this reason, the Committee recommended phrases that stress the importance of providing a ductile connection and a ductile pile section in the vicinity of the pile cap. Since precast concrete piles exhibited unsatisfactory performance in some instances in the past, extra conservatism was judged to be appropriate for their use.

Finally, the Committee considered the Chapter on soil-structure interaction. The discussion was wide ranging, but a strong consensus was reached. The provisions of Chapter 6 are so complex that they are not effective in implementing a new concept. The equations could not be applied by practicing engineers in general without an extraordinary risk of error. The practicing engineer would have little "feel" for what he was calculating. Finally, there are few or no prototype observations that would justify the use of this sophisticated procedure. The Committee recommended that Chapter 6 be deleted from the Provisions at this stage. A compromise position stressing the optional nature of the Chapter was developed by the coordinating committee (Committee 10) as contained on the full committee ballot. Committee 10 accepted that the deletion of the Chapter 6 provisions should be reconsidered following a specific evaluation of their effect during the trail design phase.

2.0 COMMITTEE ACTIONS

2.1 Recommendations for Change

In this section the changes to Chapters 6 and 7 proposed by Technical Committee #3 are presented. These changes have been unanimously adopted by the committee members, i.e. there were four affirmative votes. It should also be pointed out that Joseph V. Tyrrell of the Interagency Committee on Seismic Safety in Construction voted affirmative with reservations for Ballot Item 8e. This item involved Section 7.4.4(A) on page 74 of the ATC report, which the committee decided not to modify. Mr. Tyrrell indicated that he did not want this reservation to be voted on by the committee but merely wanted to state for the record that he recommends that "for caissons greater than 30" diameter, minimum reinforcement (steel ratio) should be .0020." Because the committee decided not to modify this item in Section 7.4.4(A) in their ballot for no change, the item in question does not appear in the following pages.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 1e

ATC-3-06 SECTION REFERENCE: 3.2.1

The last paragraph in Section 3.2.1 should be changed to read "In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 or Soil Profile S_3 shall be used depending on whichever soil profile type results in the higher value of seismic coefficient, C_s, as determined in Section 4.2.1.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Soil Profile Type S_2 is much better than Type S_3. Section 3.2.1 suggests soil profile type S_2 when the soil properties are not known. This did not seem logical. Hence the proposed change was recommended.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 4e

ATC-3-06 SECTION REFERENCE: 7.2.2

The last sentence in Section 7.2.2 should read "For the load combination including earthquake as specified in Section 3.7, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Soils are inherently inelastic materials. To specify stressing below the elastic limit is practically without meaning. Hence, the term "elastic limit" should be replaced with the phrase, "to resist loads at acceptable strains".

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS   COMMITTEE ITEM NUMBER: 7e

ATC-3-06 SECTION REFERENCE: 7.5.2

The first sentence in Section 7.5.2 should be changed to read "Individual spread footings unless founded directly on rock, as defined in Section 3.2.1(1), shall be interconnected by ties".

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The comment was made that it is overly conservative to require structural ties between pile caps equal to 25% of the maximum vertical load for a Category B structure. This conservatism is amplified in the commentary of this paragraph where it states, "Lateral soil pressure on pile caps is not a recommended method; and if the soil is soft enough to require ties, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions." There are many cases in which the use of piles is dictated by deep soil deposits and the near surface materials are relatively stiff and strong (such as compact or dense gravels and sands overlying soft clays or controlled, compacted fill over clays or organic soils. In these cases, it would seem reasonable to permit at least a portion of the lateral tie resistance between the pile caps to be provided by lateral soil resistance with some guidance provided. In light of these considerations and after discussing the terminology that would be appropriate, the committee agreed to recommend the change shown above.

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 9e

ATC-3-06 SECTION REFERENCE: 7.4.4

At the end of paragraph 2 of Section 7.4.4 (before Item A) the following sentence should be added, "Where special reinforcement at the top of the pile is required alternative measures for containing concrete and maintaining ductility will be permitted provided due consideration is given to forcing the hinge to occur in the contained section.

FINAL BALLOT:

4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The committee discussed possible designs for the connection at the top of the pile. It was agreed that the intent was to put the ductile section where the hinge would form. Considering this fact and the comments received, the proposed change was recommended. The minority view as expressed in a comment from Committee 4 to use an exposed strand was rejected by the committee because it was judged that one could not manufacturer a ductile connection between the pile and the pile cap using steel strand. Furthermore, it is at the point where the pile is connected to the pile cap that the greatest damage was observed during the San Fernando and Alaskan earthquakes.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS

COMMITTEE ITEM NUMBER: 10e

ATC-3-06 SECTION REFERENCE: 7.5.3(c)

The last sentence in Section 7.5.3(c) should be revised to read, "Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from a dynamic analysis of the soil profile."

FINAL BALLOT:

4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The basis for the proposed change was that prestressed precast concrete piling can withstand considerable curvature and through proper detailing confinement and ductility can be provided.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS

COMMITTEE ITEM NUMBER: 12e

ATC-3-06 SECTION REFERENCE: Chapter 6

Chapter 6 should be deleted.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

After reviewing Chapter 6 and thoroughly examining the procedures therein, the committee felt strongly that the provisions were not effective in implementing a new concept. Chapter 6 is too complicated for the practicing engineer and it is not justified based on field observations. The sophistication of the analysis is inconsistent with the accuracy of the results and the complexity masks the understanding of the performance of the soil structure system. Further documentation for deletion of Chapter 6 is provided in the minutes for the February 15, 1980 meeting (attachments).

3.0 COMMITTEE RECORDS

3.1 Minutes of Meetings

Committee 3 held two meetings. The opening meeting was on December 11, 1979 at the National Bureau of Standards in Building 226, Room B113. The second and final meeting was on February 15, 1980 at Law Engineering Testing Company, 109 Inverness Drive East, Englewood, Colorado. The minutes for these meetings are provided in this section. Included in the minutes for the February 15, 1980 meeting are the items which were voted on by Committee 3 and the ballot which was used to document the votes of each member.

Minutes of 1st Meeting

of

Technical Committee No. 3 on Foundations

for

Review and Refinement of

Tentative Seismic Provisions (ATC-3-06)

at

National Bureau of Standards

December 11, 1979

Introduction

On December 11, 1979 a meeting was held at the National Bureau of Standards in Building 226, Room B113 with the members of ATC Review and Refinement Committee No. 3, Foundations. This was the opening meeting of the group assembled for reviewing and refining Chapter 6, Soil-Structure Interaction, and Chapter 7, Foundation Design Requirements, in the ATC report (NBS SP-510) entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings". The topics discussed included:

1) Introduction and Orientation by NBS Secretary

2) Selection of Chairman

3) Selection of Representative to Committee 2

4) Selection of Date and Location for Next Meeting

5) Establishment of Work Plan and Preliminary Discussion of Chapters 6 and 7

The participants in the meeting are listed in Table 1, List of Participants. With the exception of the ATC representative, Henry Degenkolb, and the Building Seismic Safety Council representative who had not been selected as of December 11, 1979, all members of Committee 3 attended the opening meeting. To aid in the review of Chapters 6 and 7, the following handouts were provided:

1) List of Members of ATC Review and Refinement Committee No. 3 (Table 2)

2) ATC report (NBS SP-510) entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings"

3) NBS Technical Note 1100 entitled "Analysis of Tentative Seismic Design Provisions for Buildings"

4) Work Plan for Review and Refinement of Tentative Seismic Provisions dated 11/27/79

5) Edward O. Pfrang's letter to the Participants on the Review and Refinement of Tentative Seismic Provisions dated November 30, 1979

A summary of the meeting highlights follows.

Proceedings

General

The proceedings of the meeting will be summarized using the following categories:

Item A - Background Information

Item B - Work Plan

Item C - Preliminary Comments on Chapters 6 and 7

Item A: Background Information

1) Lawrence Salomone called the meeting to order, presented the agenda for the meeting, summarized some of the milestones in the schedule, informed the members of the source documents which are available (Handouts 2 and 3), circulated the List of Committee 3 Members for review and comment and answered any questions.

2) Richard Simon was elected Chairman.

3) Joseph V. Tyrrell was selected to represent Committee 3 in Committee 2.

4) The next meeting will be held in the offices of Law Engineering Testing Company, 109 Inverness Drive East, Suite West B, Englewood, Colorado 80110 on February 15, 1980 at 9:00 A.M. Arrangements for the

meeting will be made by James LaBastie.

5) In response to the question raised at the meeting why no BSSC representative has been selected, the writer has learned that an individual will be selected. However, similar to other committees, circumstances did not permit selection of a BSSC representative before the December 11, 1979 meeting.

6) Lawrence Salomone informed the members that Chapter 3 of TN1100 (pages 20-29) summarizes the findings of the study of the ATC - 3 provisions and that Committee 3 may find Section 3.3.5, and Sections 3.1.4, 3.1.5 and 3.5 helpful.

Item B: Work Plan

1) It was agreed that Chapters 6 and 7 were difficult to read and that not enough background information was provided. It was hoped that the ATC representative will be able to attend the next meeting in order to provide the necessary background information. Richard Simon will call Henry Degenkolb, the ATC representative, to inform him of the time and location of the next meeting.

2) The schedule will require that the equations presented in Chapter 6 be accepted. However, the committee will use the review time available to see if the equations can be used.

3) Richard Simon suggested that the members read some of the other sections in the ATC report (e.g. Chapters 1 and 2) before the next meeting to aid in their understanding of the contents of Chapters 6 and 7.

4) Richard Simon encouraged the committee to solicit comments from their colleagues.

Item C: Preliminary Comments on Chapters 6 and 7

1) The committee discussed the need for spread footings to be interconnected by ties in Section 7.5.2. William Travis questioned the need and thought that the economic impact of this requirement would be great. J. LaBastie indicated that some conservatism may be necessary considering the different levels of background and experience of the users of a standard. William Travis said the problem is that this section does not allow the designer to demonstrate that ties between spread footings are not necessary. William Travis will call Henry Degenkolb to ask him why this requirement is in the ATC provisions.

2) Richard Simon pointed out the relative absence of the need to evaluate liquefaction potential of foundation soils in the report. Liquefaction

does not seem to be extensively discussed. The committee agreed that some provision may be necessary. R. Simon will draft a recommendation regarding the evaluation of the liquefaction potential of foundation soils.

3) Joseph Tyrrell said that it may be necessary to add something about site investigations.

4) The committee agreed that reference to the elastic limit in Section 7.2.2 should be changed.

5) William Travis discussed the need for requiring ties in pole-type structures. It was concluded that it may not be necessary because of the relative importance of the structures covered by Section 7.4.2.

Respectfully submitted,

Larry Salomone

Larry Salomone, Secretary

Table 1

List of Participants

Participant	Organization
James G. LaBastie	American Society of Civil Engineers
Richard Simon	Association of Soil and Foundation Engineers
Joseph V. Tyrrell	Interagency Committee on Seismic Safety in Construction
William Travis	Structural Engineers Association of California
Lawrence A. Salomone (Secretariat)	National Bureau of Standards
Riley Chung	National Bureau of Standards

Visitor	
Felix Y. Yokel Chairman, ASCE Committee on Foundation and Excavation Standards	National Bureau of Standards

TABLE 2

COMMITTEE 3: Foundations

American Society of Civil Engineers

Mr. James G. LaBastie
6252 Powell Road
Parker, Colorado 80134

Phone: 303-771-8641

Association of Soil and Foundation Engineers

Mr. Richard Simon (Chairman)
Goldberg, Zoino, Dunnicliff & Assoc., Inc.
30 Tower Road
Newton Upper Falls, Massachusetts 02164

Phone: 617-969-0050

Interagency Committee on Seismic Safety in Construction

Mr. Joseph V. Tyrrell
Director, Civil/Struc. Div.
Naval Facilities Engineering Commd.
200 Stovall Street
Alexandria, VA 22332

Phone: 202-325-0064

Structural Engineers Association of California

Mr. William Travis
7851 Mission Center Court
Suite 250
San Diego, California 92108

Phone: 714-291-2800

Applied Technology Council

Mr. Henry Degenkolb
H. J. Degenkolb & Associates
350 Sansome Street
San Francisco, CA 94104

Building Seismic Safety Council

Committee 3 (continued)

National Bureau of Standards

Mr. Larry Salomone
Secretariat
Committee 3, Foundations
National Bureau of Standards
Rm. B168, Bldg. 226
Washington, D. C. 20234

Phone: 301-921-3128

(Dr. Riley Chung)

Phone: 301-921-2137

(Dr. Felix Yokel)

Phone: 301-921-2648
301-921-2647

Minutes of 2nd Meeting

of

Technical Committee No. 3
on Foundations
for
Review and Refinement
of Tentative Seismic
Provisions (ATC-3-06)

at

Law Engineering Testing
Company

February 15, 1980

Introduction

On February 15, 1980, a meeting was held at Law Engineering Testing Company, 109 Inverness Drive East, Englewood, Colorado. This was the second meeting of the group assembled for reviewing and refining Chapter 6, Soil-Structure Interaction, and Chapter 7, Foundation Design Requirements, in the ATC Report (NBS SP-510) entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings." The meeting served as a workshop to discuss comments received by February 15, 1980 on Chapter 6 and 7 and to develop recommendations for revising these chapters. The participants in the meeting are listed in Table 1, List of Participants. A numerical listing of the sections in which comments were received was prepared by Richard Simon, Chairman of Technical Committee No. 3 and provided to the committee members (Table 2). At the beginning of the workshop the secretary of Technical Committee 3, Lawrence Salomone, distributed comments from the Concrete Reinforcing Steel Institute and Professor V. V. Bertero of the University of California, Berkeley. These comments were sent to Committee 4 and were not received by Committee 3 until just prior to the workshop. Joseph V. Tyrrell, representative of Committee 3 in Committee 2, distributed the comments sent to Committee 2.

Proceedings

The workshop was conducted in the following manner:

1) Each of the comments received (Table 2) were discussed and a final position of the committee developed. The final position consisted of a recommendation for change for the Section in question or a recommendation that the Section not be modified.

2) Following development of the final position of the committee a vote was taken.

The meeting highlights were:

1a Section 3.2.1 Paragraph 4

Author of Comment - Joseph V. Tyrrell, Interagency Committee on Seismic Safety in Construction.

1b Excerpt from ATC 3-06

"In location where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 should be used."

1c Comment:

Soil Profile Type S_2 is much better than Type S_3. This section suggests soil profile Type S_2 when the soil properties are not known. This does not seem logical.

1d Discussion

The discussion centered around Figure C1-13, Normalized Lateral Design Force Coefficients (A_a=A_v=1.0) and the effect of soil type on the normalized design coefficient.

1e Recommendation

The last paragraph in Section 3.2.1 should be changed to read "In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 or Soil Profile S_3 shall be used depending on whichever soil profile type results in the higher value of seismic coefficient C_s, as determined in Section 4.2.1.

1f Ballot - The recommendation for change was voted on and unanimously accepted by the members of Committee 3.

2a Section 3.2.1 Paragraph 1 "...stable deposits of sands, gravels or stiff clays."

Author of Comment - Richard M. Simon, Association of Soil and Foundation Engineers

2b Excerpt from ATC 3-06

"Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays."

2c Comment

The use of the word "stable" was questioned.

2d Discussion - The group discussed the appropriateness of the use of the word "stable". J. LaBastie, American Society of Civil Engineers, stated that he had no problem with the term stable. The committee agreed and no change was recommended.

3a Section 7.1, Paragraph 1

Author of Comment - Joseph V. Tyrrell, Interagency Committee on Seismic Safety in Construction.

3b Excerpt from ATC 3-06

"These include, but are not limited to provisions for the extent of investigation, fills, slope stability, bearing and lateral soil pressures, reports, drainage, settlement control, and pile requirements and capacities."

3c Comment:

The word "reports" here does not read right. Suggest the last sentence be changed to: "... These include ... fills, slope stability, bearing capacity, lateral soil pressure and support, drainage and settlement control, and pile requirement and capacities."

3d Discussion

The committee referred to the commentary on page 403 of the ATC 3-06 document in which Section 7.1 is discussed. The committee felt that this section is discussed in the commentary and the proposed change would not add anything.

3e Recommendation

J. Tyrrell suggested dropping the comment and the committee agreed to J. Tyrrell's recommendation.

4a Section 7.22, Paragraph 1

Authors of Comments - Joseph V. Tyrrell and Richard M. Simon

4b Excerpt from ATC 3-06

"... the soil capacities must be sufficient to provide resistance at the elastic limit or less ...."

4c Comment (Joseph V. Tyrrell)

This section is not clear as the elastic limit of the soil is generally difficult to define. Furthermore, the soil bearing capacity is developed from the plastic equilibrium concept. Thus, to use the elastic limit to restrain the capacities of the soil resistance is not reasonable. This needs clarification.

Comment (Richard M. Simon)

The last sentence of this paragraph states ...Soil capacities must be sufficient to provide resistance at the elastic limit or less considering both the short time of loading and the dynamic properties of the soil." Soils are inherently inelastic materials and to specify stressing below the elastic limit is practically without meaning. The term "elastic limit" should be replaced with a term such as peak shear strength, yield limit, or other which has a more significant meaning for soils. Because elastic has so little meaning for soils, it is not possible to know what the author of this section had in mind when writing this sentence.

4d Discussion

The committee discussed the fact that "elastic limit" was not the correct term to use and discussed alternate terminology for elastic limit.

4e Recommendation

The last sentence in Section 7.2.2 should read "For the load combination including earthquake as specified in Sec. 3.7, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil.

4f Ballot

The recommendation for change was voted on and unanimously accepted by the members of Committee 3.

5a Section 7.4.2

Author of Comment - Wm. Travis, Structural Engineers Association of California.

5b Excerpt from ATC 3-06

Construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth may be used to resist both axial and lateral loads. The depth of embedment required for posts or poles to resist seismic loads shall be determined by means of the design criteria established in the foundation investigation report.

5c Comment

No formal comment was proposed.

5d Discussion

William Travis discussed the need for requiring ties in pole-type structures. Henry Degenkolb mentioned that in San Francisco ties are required in Class C and D structures. However, it was also stated that pole-type structures were used only for agriculture purposes.

5e Recommendation

William Travis suggested dropping the comment and the committee agreed to his recommendation.

6a Section 7.4.3, Paragraph 1

Author of Comment Richard M. Simon

6b Excerpt from ATC 3-06

Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger pile cap or column load unless it can be demonstrated equivalent restraint can be provided by other approved means.

6c Comment

In my opinion, it is overly conservative to require structural ties between pile caps equal to 25% of the maximum vertical load for a Category B structure. This conservatism is amplified in the commentary of this paragraph where it states, "Lateral soil pressure on pile caps is not a recommended method; and if the soil is soft enough to require ties, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions." There are many cases in which the use of piles is dictated by deep soil deposits and the near surface materials are relatively stiff and strong (such as compact or dense gravels and sands overlying soft clays or controlled, compacted fill over clays or organic soils. In these cases, it would seem reasonable to permit at least a portion of the lateral tie resistance between the pile caps to be provided by lateral soil resistance with some guidance provided. In light of these considerations, the following changes in the tentative provisions and commentary are suggested:

a) Section 7.4.3 -- Sentence 2

$A_v/4$ should be changed to $A_v/6$.

b) Section 7.5.2

"Individual pile caps, drilled piers, caissons or spread footings shall be interconnected by ties. All ties shall be capable of carrying by tension or compression $A_v/4$ of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by other approved means."

c) Commentary -- Section 7.4.3 Paragraph 3

In my opinion it is overly conservative to preclude the use of lateral soil pressure on pile caps and spread footings from consideration in tying of foundation elements in all cases. The alternative paragraph below is a suggested replacement for the paragraph provided in the Commentary:

1. Alternative methods of tying foundation together are permitted, such as use of properly reinforced floor slab that can take both tension and compression. Lateral soil pressure on pile caps is not a recommended method if the pile caps are embedded in soft soil because the motion is transmitted from soil to structure (not inversely, as is commonly assumed); if the soil is soft enough to require ties, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions. However, if pile caps or caissons are embedded in stiff soil such as medium dense to dense sand or sand and gravel or stiff to hard clay, and backfill surrounding the pile caps or caissons is systematically compacted to a minimum of 95% of the modified Procter density, lateral soil pressure may be used to provide 100% of the required lateral force for category B structures. No contribution to the lateral force on the pile cap may be provided by shear stresses on the base of the pile cap because underlying soils may settle the soil from beneath this pile cap leaving a gap. For category C or D structures, lateral soil pressure may be used to provide up to 50% of the lateral force required for foundation ties. For category C or D structures on spread footings, the lateral forces may be used to provide up to 50% of the total required tie force and both lateral pressure on sides of footings as well as shear stresses on the base of footings may be considered to provide the necessary force.

6d <u>Discussion</u>

Richard Simon said that one should be able to take advantage of the compacted backfill. William Travis said that piles should be tied together. The committee discussed using the term "rock" in Section 7.4.3 (material with a shear wave velocity of 2500 fps).

6e <u>Recommendation</u>

The committee decided not to modify Section 7.4.3.

7a Section 7.5.2, Paragraph 1

Author of Comment Richard M. Simon

7b Excerpt from ATC 3-06

Individual spread footings shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by other means.

7c Comment

See Item 6c.

7d Discussion

The committee discussed terminology that would be appropriate.

7e Recommendation

The first sentence in Section 7.5.2 should be changed to read "Individual spread footings unless founded directly on rock, as defined in Section 3.2.1 (1), shall be interconnected by ties."

7f Ballot

The recommendation for change was voted on and unanimously accepted by the members of Committee 3.

8a Section 7.4.4, Item A, Uncased Concrete Piles

Author of Comment Joseph V. Tyrrell

8b Excerpt from ATC 3-06

(A) UNCASED CONCRETE PILES

Reinforcing steel shall be provided in the top portion of uncased cast-in-place concrete piles or caissons for a distance of ten pile diameters, with a minimum steel ratio of 0.0025 with a minimum of four No. 5 bars. Ties (or equivalent spirals) shall be provided at 16-bar diameter spacing (maximum spacing) with a maximum spacing of 4 inches in the top 2 feet.

8c Comment

The pile diameters generally range from 12 inch to 16 inch, but caisson shaft diameters are generally 24 inch to 30 inch minimum. The ten pile diameter reinforcement requirement for caisson would be excessive. I believe the word "caisson" here means drilled piers or drilled caissons.

8d Discussion

J. Tyrrell thought a minimum steel ratio of .0025 was too high. The committee then discussed the need for steel and the amount of steel required in caissons. J. LaBastie said he never saw caissons without steel. William Travis pointed out that steel may be needed since stresses due to concrete shrinkage and tests have shown the steel is necessary. J. Tyrrell concluded the discussion by saying that he did not have strong feelings about his comment.

8e Recommendation

The committee decided not to modify Section 7.4.4, Item A.

9a Section 7.4.4, Item E, Paragraph 1

Author of Comment - David A. Sheppard, Prestressed Concrete Institute

9b Excerpt from ATC 3-06

(E) PRECAST-PRESTRESSED PILES.

The upper 2 feet of the pile shall have No. 3 ties minimum at not over 4-inch spacing, or equivalent spirals. The pile cap connection may be by means of dowels as required in Sec. 7.4.4(C).

9c Comment

Revise Section 7.4.4(E) to read as follows: The upper 2 feet . . . . or equivalent spirals. The pile cap connection may be made by developing exposed strand or by the use of field placed anchor dowels grouted into sleeves cast in the pile top as outlined in Section 11.9.

BASIS: Present accepted practice in UBC-79 and CAL-TRANS specifications

9d Discussion

H. Degenkolb showed pictures of how piles shattered at the top during the 1971 San Fernando Earthquake. Precast prestressed piles performed poorly.

The committee discussed possible designs for the connection at the top of the pile. It was agreed that the intent was to put the ductile section where the hinge would form. Considering this fact and the comment from David Sheppard, PCI, a recommendation for changes was prepared. When addressing Sheppard's comment the committee did not accept the use of an exposed strand.

9e Recommendation

At the end of paragraph 2 of Section 7.4.4 (before Item A) the following sentence should be added, "Where special reinforcement at the top of the pile is required alternative measures for containing concrete and maintaining ductility will be permitted provided due consideration is given to forcing the hinge to occur in the contained section."

9f Ballot

The committee unanimously rejected the comment made by the Prestressed Concrete Institute. The recommendation for change was voted on and unanimously accepted by the members of Committee 3.

10a Section 7.5.3 (C) Paragraph 1

Authors of Comments - David A. Sheppard, PCI and Joseph G. Manning, Concrete Reinforcing Steel Institute

10b Excerpt from ATC 3-06

(C) PRECAST CONCRETE PILES.

> Ties in precast concrete piles shall conform to the requirements of Sec. 11.6.2 for the top half of the pile. Precast concrete piles shall not be used to resist flexure caused by earthquake motions unless it can be shown that they will be stressed to below the elastic limit under the maximum soil deformations that would occur during an earthquake.

10c Comment (D. Sheppard, PCI)

Revise the last sentence of Section 7.5.3(c) to read as follows: "Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from a dynamic analysis of the soil profile present, with detailing as specified in Section 11.9."

BASIS: See my letter and accompanying documentation from Gerwick, et al presented at the BSSC meeting on November 8, 1979.

Comment (Joseph G. Manning, CRSI)

Revise second sentence to read as follows:

> Precast concrete and prestressed precast concrete piling shall be designed to withstand maximum imposed curvatures resulting from the maximum soil deformations that would occur during an earthquake.

Reason: Prestressed precast concrete piling can withstand considerable curvature and through proper detailing confinement and ductility can be provided.

10d Discussion

The committee discussed the validity of D. Sheppard's comment and agreed that it should be accepted. The committee also considered the comment from Joseph G. Manning, Concrete Reinforcing Steel Institute, and decided the comment was similar to D. Sheppard's comment.

10e Recommendation

The last sentence in Section 7.5.3(c) should be revised to read, "Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from a dynamic analysis of the soil profile.

10f Ballot

The recommendation for change was voted on and unanimously accepted by the members of Committee 3.

11a Section 7.6.1, Paragraph 1

Authors of Comments - David A. Sheppard, PCI, and Joseph G. Manning, CRSI

11b Excerpt from ATC 3-06

Sec. 7.6.1 SPECIAL PILE LIMITATIONS

Precast-prestressed piles shall not be used to resist flexure caused by earthquake motions.

11c Comment (David A. Sheppard)

Sec. 7.6.1

Revise this section to read as follows: "All piling types in Category D shall be designed to withstand maximum imposed curvatures resulting from a dynamic response analysis of the soil profile present."

BASIS: Same as 7.5.3 above. Foundation requirements should be performance oriented, and not arbitrarily penalize certain materials (prestressed concrete) because of local bias, in spite of recent tests and successful design applications developing large curvatures resulting from layered soil movements in maximum credible seismic conditions.

Comment (Joseph G. Manning)

Section 7.6.1 Special Pile Limitations

Delete this section.

Reason: See comments on Section 7.5.3.

11d Discussion

The committee considered the comments that were received, but decided that additional conservatism was required for Category D structures. Therefore, Section 7.6.1 should not be modified.

11e Recommendation

The committee recommended not to change or delete Section 7.6.1

12a Chapter 6

Author of Comment - Richard M. Simon and William M. Travis

12b Excerpt from ATC 3-06

Refer to pages 65-71 in ATC 3-06 report.

12c Comment (Richard M. Simon)

1. Chapter 6 --SOIL-STRUCTURE INTERACTION

Chapter 6 contains some relatively complex equations for adjustment of the equivalent lateral force and modal analysis procedures for evaluation of earthquake induced forces. It may be inappropriate to include these soil structure interaction (SSI) equations in the tentative provisions document at this time for the following reasons:

a) Both the equivalent lateral force and modal analysis procedures are approximate in themselves. There have been few prototype observations to explore the accuracy of either procedure. The equations contained in Chapter 6 are based solely on analytical studies with almost no prototype or even laboratory verification. Because of the approximate nature of the entire analysis procedures it is, in my opinion, inappropriate to add an additional level of sophistication to the equations. The SSI equations are so complex as to make me question whether they could be properly applied by the average structural/civil engineer. There are no guidelines provided in the provisions or commentary as to the approximate magnitude of the effect produced by these equations other than the reduced base shear shall in no case be taken less than 0.7 times the rigid support base shear. Little guidance is provided in the readily available published literature. Although failure to incorporate the effects of SSI is in almost all cases a conservative assumption, it is not clear that applying the equations without complete understanding and with potential errors will lead to conservative design forces and displacements.

b) The Tentative Provisions have emphasized the inelastic behavior of structures in response to earthquake loading. The analytical procedures used as the basis of the Chapter 6 provisions assume a linear response for the superstructure which is inconsistent with the general philosophy of the Tentative Provisions.

c) It may be inconsistent with the charges of this committee to expurgate the entire chapter of the provisions at this review stage, however, as a minimum, it is recommended that the trial designs be carried through both with and without use of the SSI equations so that an evaluation of the effect of these equations on the final design may be determined. If the effect of the SSI equations is shown by trial designs to be of little significance, it may be proper to omit the entire section at that time.

2. Section 6.1 -- General

Inasmuch as the SSI equations require the expenditures significant additional design effort with questionable return in terms of design savings, Section 6.1 does not contain a clear enough statement that they may be conservatively ignored. To clarify this idea, it is recommended that the following sentence be impended to the end of paragraph 1 of the section: It is acceptable to evaluate the design earthquake forces and corresponding displacement of the building using only the equations contained in chapters 4 and 5.

Comment (William L. Travis)

See letter from William L. Travis to Lawrence A. Salomone dated 2/20/80 (attached).

12d Discussion

R. Simon opened the discussion by outlining three approaches with respect to Chapter 6:

1) Do not change Chapter 6
2) Omit Chapter 6
3) Do trial designs both ways

In addition, H. Degenkolb presented the background on Chapter 6. He said the section is theoretical.

Simon then said that this section lowers coefficient and hardware to resist earthquakes. William Travis followed by presenting the need for having a practical feel for what the structure is doing. He showed the complexity of the procedure in Section 6.0 (see Travis's letter dated February 20, 1980). The theoretical approach results in losing sight of what actually is happening. William Travis said he thought Chapter 6 should be cleaned up and it should be modeled after UBC. He thought that an error could be made using Chapter 6 and the designer would not know it. Chapter 6 is too complicated for the practicing engineer and it is not justified.

J. Tyrrell said his people ran through the Chapter 6 procedure and he supported William Travis's comments. It was pointed out that John Christian of Stone & Webster said that the procedure was sound. However, William Travis said he did not agree. J. La Bastie explained that John Christian's experience is mainly in soils and nuclear power plants.

After these comments were expressed the committee discussed what course of action could be taken. It was pointed out that in the beginning of the ATC 3-06 report it was said that Chapter 6 could be ignored (it stands alone). Therefore, the impact of eliminating or modifying Chapter 6 would be minimal. Furthermore, page 4 of the guidelines given to committee members states that the effectiveness of a section can be questioned. The committee felt that soil conditions were considered by S factors. Consequently, they formulated and agreed to the position stated below.

12e Recommendation

After reviewing Chapter 6 and thoroughly examining the procedures therein, the committee feels strongly that the provisions are not effective in implementing a new concept. Chapter 6 is too complicated for the practicing engineer and it is not justified based on field observations. The sophistication of the analysis is inconsistent with the accuracy of the results and the complexity masks the understanding of the performance of the soil structure system. Consequently, the committee recommends deleting Chapter 6 from the tentative provisions.

With the completion of the discussion of Chapter 6 the meeting was adjourned. It was agreed that:

1) A draft of the minutes will be distributed for review and comment by the committee members.

2) William Travis will send the required backup for elimination of Chapter 6 within 3 weeks of the date of the meeting.

3) Committee members will send letters documenting their vote regarding the items discussed. (Please use the enclosed form for this documentation).

4) There is no need to send out a ballot for the items discussed except for Chapter 6.

5) Any future discussions that were required would be performed by a telephone conference call.

TABLE 1

Meeting 2/15/80

List of Attendees

Participant	Affiliation
Lawrence Salomone	NBS
William L. Travis	SEAOC
Henry J. Degenkolb	ATC 3-06
J. G. LaBastie	ASCE
J. V. Tyrrell (IACSSC)	Naval Fac., Eng. Com.
Richard M. Simon	ASFE

TABLE 2

Numerical List of Sections

Prepared by Richard Simon

Prior to February 15, 1980 Meeting

1	§ 3.2.1	a. Default S_2	Tyrrell
		b. "Stable" deposits of S & G	Simon
2	Cpt 6	a. As is	Simon
		b. Omit	
		c. Trial designs both ways	
3	§ 7.1	"reports"	Tyrrell
4	§ 7.2.2	"elastic limit"	Tyrrell, Simon
5	§ 7.4.2	Pole structures	Travis
6	§ 7.4.3	Foundation ties	Simon
7	§ 7.44	Exposed strand	PCI
8	§ 7.53	"withstand curvature" elastic limit	PCI
9	§ 7.6.1	PCC piles	

REVIEW AND REFINEMENT
OF TENTATIVE SEISMIC PROVISIONS (ATC 3-06)

COMMITTEE 3 BALLOT

Committee 3 on Foundations

Issue Date 3/10/80

Deadline for Receipt 4/1/80

Information and Instructions

1. The return of this ballot is required from voting members of Committee 3.

2. When voting affirmative with reservations (Column 4) or negative (Column 3) the member can provide an explanation in the remarks column (Column 5).

3. The member should refer to the minutes for the February 15, 1980 meeting for a detailed description of the item listed in Column 1.

Signature ______________________ Date ______________________

Return ballot to Lawrence A. Salomone
Technical Committee No. 3
Tentative Seismic Provisions Project
B168, Bldg. 226
National Bureau of Standards
Washington, D. C. 20234

Section No. - Item No. (see minutes from 2/15/80 mtg.)	Affirmative	Negative	Affirmative with Reservations	Remarks (Check box and use Pg 2)
Section 3.2.1 - Item 1e				
Section 3.2.1 - Item 2d				
Section 7.1 - Item 3e				
Section 7.2.2 - Item 4e				
Section 7.4.2 - Item 5e				
Section 7.4.3 - Item 6e				
Section 7.5.2 - Item 7e				
Section 7.4.4(A) - Item 8e				
Section 7.4.4(E) - Item 9e				
Section 7.5.3(C) - Item 10e				
Section 7.6.1 - Item 11e				
Chapter 6 - Item 12e				

Review and Refinement
of Tentative Seismic Provisions (ATC 3-06)

Committee 3 Ballot

Section No. - Item No.	Remarks

TRAVIS, VERDUGO & ASSOCIATES

February 20, 1980

Mr. Lawrence A. Salomone
Secretary Committee 3
National Bureau of Standards
Room B168, Bldg. 226
Washington, D.C. 20234

Subject: Comments on Chapter 6, ATC-3

Dear Larry:

As agreed, I have put together some background material for our position as formulated in Denver, February 15, 1980.

Hopefully this will reach you soon enough to distribute with the minutes. Let me know if I can do anything else at this time.

Very truly yours,

TRAVIS, VERDUGO & ASSOCIATES

William L. Travis
President

WLT:cdp

Enclosures

STRUCTURAL CONSULTANTS
7851 MISSION CENTER COURT, SUITE 250
SAN DIEGO, CALIFORNIA 92108
PHONE (714) 291-2800

COMMENTS ON CHAPTER 6, ATC-3

A. While trying to find out how a young practicing engineer might apply Chapter 6, I tried to analyse the steps he would go through. By the 32nd step I was so confused that I forgot what I was trying to find out! Also gone was any feel that I might have had for the effect of the various formulae. (Refer to handwritten notes.)

B. A comparision seemed in order to the relatively simple UBC Chapter 23. The same material is adequately covered on what amounts to less than 4 pages (copies of pages 132-136 from 1976 UBC).

C. A recent publication by Mr. Roy Becker refers to work by Mr. Edward Teal who, while noting that the formula for building period, T=0.10N is inadequate for buildings less than say 40 stories in height and that the Rayleigh Formula should be used suggests that the Rayleigh Formula is impractical to use. Instead, he proposes a simplified version and notes that the SEAOC recommends only two-thirds of the completely arbitrary 0.5% drift limitation for initial building period calculations. A few of these pages are attached. Note that after these simplifications and approximations with assumed S-values and so on, the SEAOC recommends a further limitation of T.

D. As stated by Mr. Richard Simon, Chairman of the technical committee, the "additional level of sophistication" added to the equations is inappropriate when considering the approximate nature of the entire analysis procedure, at least at this time. Those portions of his comments are attached, with an "AMEN" from Mr. Henry Degenkolb.

1. Chapter 6, Sec. 6.2.1 Base Shear, must refer to Ch. 4 p. 55 $V = C_s W$
2. Ch. 4, p. 55, $C_s = \dfrac{1.2 A_v S}{R T^{2/3}}$ 3. See 1.4.1 for A_v
4. sec 1.4.1 A_v 5. refer to Fig 1-2 (Map) for A_v map area
6. with Map Area, refer to table 1-B for A_v & Seismicity Index
7. S - refer to Table 3-A
8. for 3A must have Soil Profile Type - refer to p. 45
9. p. 45 determine profile type, S_1, S_2 or S_3 - back to 3-A Select S
10. R - refer to Table 3-B
11. 3-B select R (special notes refer to Ch. 10, 11, 12)
12. T - refer to 4.2.2
13. $T_{max} = 1.2\, T_a$ or $= T_a$
14. $T_a = C_T h_n^{3/4}$ where C_T & h_n defined same page
    or $T_a = 0.05 h_n / \sqrt{L}$
15. Check C_s max per formula 4-3 or 4-3a
16. Formula 4-3: $C_s = 2.5 A_a / R$
17. A_a from Fig 1-1 Map Area
18. with A_a map area, refer to Table 1-B for A_a
19. Formula 4-3a: $C_s = 2 A_a / R$
20. Compare to C_s of step 2
21. Solve $V = C_s W$
22. Ch. 6 p. 65 $\tilde{V} = V - \Delta V$
23. $\Delta V = \left[C_s - \tilde{C}_s \left(\dfrac{0.05}{\tilde{\beta}} \right)^{0.4} \right] \overline{W}$
24. $\tilde{C}_s$ - for flexibly supported structure depends on $\tilde{T}$ per sec 6.2.1 (A)
25. 6.2.1 (A) $\tilde{T} = T \sqrt{1 + \dfrac{\overline{k}}{K_y} \left(1 + \dfrac{K_y \bar{h}^2}{K_\theta} \right)}$

A

26 $\bar{k} = 4\pi^2 \frac{\overline{W}}{gT^2}$ p. 66

27 $\overline{W}$ defined p. 65 as 0.7W except . . .

28 with g = accel. of gravity & T from above solve for $\bar{k}$

29 K_y p. 66 = static horizontal force at the level of the foundation required to produce a unit deflection at that level . . . ?:

30 K_θ = the rocking stiffness of the foundation, . . . the static moment necessary to produce a unit average rotation of the foundation . . . ??

31 Commentary p. 386 → 395 ?

32

(i) **Moment of Stability.** The overturning moment calculated from the wind pressure shall in no case exceed two-thirds of the dead load resisting moment.

The weight of earth superimposed over footings may be used to calculate the dead load resisting moment.

(j) **Combined Wind and Live Loads.** For the purpose of determining stresses all vertical design loads except the roof live load and crane loads shall be considered as acting simultaneously with the wind pressure.

> EXCEPTION: Where snow loading is required in the design of roofs, at least 50 percent of such snow load shall be considered acting in combination with the wind load. The Building Official may require that a greater percentage of snow load be considered due to local conditions.

Earthquake Regulations

Sec. 2312. (a) **General.** Every building or structure and every portion thereof shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the base. The force shall be assumed to come from any horizontal direction.

Structural concepts other than set forth in this Section may be approved by the Building Official when evidence is submitted showing that equivalent ductility and energy absorption are provided.

Where prescribed wind loads produce higher stresses, such loads shall be used in lieu of the loads resulting from earthquake forces.

(b) **Definitions.** The following definitions apply only to the provisions of this Section:

BASE is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

BOX SYSTEM is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls or braced frames as hereinafter defined.

BRACED FRAME is a truss system or its equivalent which is provided to resist lateral forces in the frame system and in which the members are subjected primarily to axial stresses.

DUCTILE MOMENT RESISTING SPACE FRAME is a moment resisting space frame complying with the requirements for a ductile moment resisting space frame as given in Section 2312 (j).

ESSENTIAL FACILITIES—See Section 2312 (k).

LATERAL FORCE RESISTING SYSTEM is that part of the structural system assigned to resist the lateral forces prescribed in Section 2312 (d) 1.

MOMENT RESISTING SPACE FRAME is a vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

SHEAR WALL is a wall designed to resist lateral forces parallel to the wall.

41

SPACE FRAME is a three-dimensional structural system without bearing walls, composed of interconnected members laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

VERTICAL LOAD-CARRYING SPACE FRAME is a space frame designed to carry all vertical loads.

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this Section:

C = Numerical coefficient as specified in Section 2312 (d) 1.

C_p = Numerical coefficient as specified in Section 2312 (g) and as set forth in Table No. 23-J.

D = The dimension of the structure, in feet, in a direction parallel to the applied forces.

$\delta_i\ \delta_n$ = Deflections at levels i and n respectively, relative to the base, due to applied lateral forces or as determined in Section 2312 (h).

$F_i F_n F_x$ = Lateral force applied to level i, n, or x, respectively.

F_p = Lateral forces on a part of the structure and in the direction under consideration.

F_t = That portion of V considered concentrated at the top of the structure in addition to F_n.

g = Acceleration due to gravity.

$h_i h_n h_x$ = Height in feet above the base to level i, n, or x respectively.

I = Occupancy Importance Factor as specified in Table No. 23-K.

K = Numerical coefficient as set forth in Table No. 23-I.

Level i

I = Level of the structure referred to by the subscript i.

i = 1 designates the first level above the base.

Level n

= That level which is uppermost in the main portion of the structure.

Level x

= That level which is under design consideration.

x = 1 designates the first level above the base.

N = The total number of stories above the base to level n.

S = Numerical coefficient for site-structure resonance.

T = Fundamental elastic period of vibration of the building or structure in seconds in the direction under consideration.

T_s = Characteristic site period.

V = The total lateral force or shear at the base.

W = The total dead load as defined in Section 2302 including the partition loading specified in Section 2304 (d) where applicable.

> EXCEPTION: "W" shall be equal to the total dead load plus 25 percent of the floor live load in storage and warehouse occupancies. Where the design

snow load is 30 psf or less, no part need be included in the value of "W." Where the snow load is greater than 30 psf, the snow load shall be included; however, where the snow load duration warrants, the Building Official may allow the snow load to be reduced up to 75 percent.

$w_i w_x$ = That portion of W which is located at or is assigned to level i or x respectively.

W_p = The weight of a portion of a structure.

Z = Numerical coefficient dependent upon the zone as determined by Figures No. 1, No. 2 and No. 3 in this Chapter. For locations in Zone No. 1, $Z = 3/16$. For locations in Zone No. 2, $Z = 3/8$. For locations in Zone No. 3, $Z = 3/4$. For locations in Zone No. 4, $Z = 1$.

(d) Minimum Earthquake Forces for Structures. Except as provided in Section 2312 (g) and (i), every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = ZIKCSW \qquad \text{(12-1)}$$

The value of K shall be not less than that set forth in Table No. 23-I. The value of C and S are as indicated hereafter except that the product of CS need not exceed 0.14.

The value of C shall be determined in accordance with the following formula:

$$C = \frac{1}{15\sqrt{T}} \qquad \text{(12-2)}$$

42

The value of C need not exceed 0.12.

The period T shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis such as the following formula:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div g\left[\sum_{i=1}^{n-1} F_i\delta_i + (F_t + F_n)\delta_n\right]} \qquad \text{(12-3)}$$

where the values of F_i, F_t, δ_i and δ_n shall be determined from the base shear V, distributed approximately in accordance with the principles of Formulas (12-5), (12-6) and (12-7) or any arbitrary base shear with a rational distribution.

In the absence of a determination as indicated above, the value of T for buildings may be determined by the following formula:

$$T = \frac{0.05h_n}{\sqrt{D}} \qquad \text{(12-3A)}$$

Or in buildings in which the lateral force resisting system consists of ductile moment-resisting space frames capable of resisting 100 percent of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces:

$$T = 0.10N \qquad \text{(12-3B)}$$

The value of S shall be determined by the following formulas, but shall be not less than 1.0:

For T/T_s = 1.0 or less

$$S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2 \qquad \text{(12-4)}$$

For T/T_s greater than 1.0

$$S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2 \qquad \text{(12-4A)}$$

WHERE:

T in Formulas (12-4) and (12-4A) shall be established by a properly substantiated analysis but T shall be not less than 0.3 second.

The range of values of T_s may be established from properly substantiated geotechnical data, in accordance with U.B.C. Standard No. 23-1, except that T_s shall not be taken as less than 0.5 second nor more than 2.5 seconds. T_s shall be that value within the range of site periods, as determined above, that is nearest to T.

When T_s is not properly established, the value of S shall be 1.5.

EXCEPTION: Where T has been established by a properly substantiated analysis and exceeds 2.5 seconds, the value of S may be determined by assuming a value of 2.5 seconds for T_s.

(e) Distribution of Lateral Forces. 1. Structures having regular shapes or framing systems. The total lateral force V shall be distributed over the height of the structure in accordance with Formulas (12-5), (12-6) and (12-7).

$$V = F_t + \sum_{i=1}^{n} F_i \qquad \text{(12-5)}$$

The concentrated force at the top shall be determined according to the following formula:

$$F_t = 0.07TV \qquad \text{(12-6)}$$

F_t need not exceed 0.25V and may be considered as 0 where T is 0.7 second or less. The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to the following formula:

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum\limits_{i=1}^{n} w_i h_i} \quad \ldots\ldots\ldots\ldots\ldots \quad (12\text{-}7)$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. **Setbacks.** Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75 percent of the corresponding plan dimension of the lower part may be considered as uniform buildings without setbacks providing other irregularities as defined in this Section do not exist.

3. **Structures having irregular shapes or framing systems.** The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.

4. **Distribution of horizontal shear.** Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

Rigid elements that are assumed not to be part of the lateral force resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

5. **Horizontal torsional moments.** Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear-resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than 5 percent of the maximum building dimension at that level.

(f) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements specified in Section 2311, or the earthquake forces specified in this Section, whichever governs.

At any level the incremental changes of the design overturning moment, in the story under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning

SECTION II: DUCTILE MOMENT FRAME DESIGN*

A.) *EAST-WEST SEISMIC FORCES:*

$V = ZIKCSW$ 1976 UBC Formula (12-1)
$Z = 1.00$ for Zone No. 4
$I = 1.00$ per UBC Table No. 23-K
$K = 0.67$ per UBC Table No. 23-I
$V = (1.00)(1.00)(0.67)(CSW) = 0.67CSW$

In order to determine C, the period T may be taken as:

$$\boxed{T = 0.10N = 0.10(7) = 0.70 \text{ sec}} \qquad (12\text{-}3B)$$

However, use of this formula often results in a poor estimate of the period for a moment-resisting frame building. As stated by Mr. Edward Teal in the AISC Engineering Journal, Fourth Quarter, 1975, "this formula yields reasonable period estimates for buildings in the 40-story range, but very poor estimates for short buildings." Therefore, Formula (12-3), sometimes known as Rayleigh's Formula, should be used rather than Formula (12-3B) for determining the period.

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div g\left[\sum_{i=1}^{n-1} F_i\delta_i + (F_i + F_n)\delta_n\right]} \qquad (12\text{-}3)$$

For an initial approximation of the building period, Formula (12-3) is impractical to use. However, there is a convenient formula which can be utilized for a good initial approximation without having to do any trial and error design. It is known as Teal's Method or Formula which is given by Edward Teal in the AISC Engineering Journals, Second and Fourth Quarters 1975. This formula very closely approximates the actual building period, and is really a simplified version of Formula (12-3):

√ $T = 0.25\sqrt{\Delta/C_1}$

where

T = Period of building, sec.
Δ = Lateral deflection at top of building, in.
C_1 = Lateral force coefficient by which the total weight of the building is multiplied in order to obtain the seismic lateral force due to the building's response to a given base motion.

Drift limitations usually control the design of a moment frame and UBC Sect. 2312(h) limits the drift to 0.5%.

---

* In lieu of a <u>ductile</u> moment frame, this building could utilize a moment frame not meeting the special ductility requirements. See Section III for design of this type of framing system, especially Section III A, "Comparison of Moment Frame with Ductile Moment Frame."

The SEAOC Commentary recommends that for an initial approximation for this building period, that two-thirds of the allowable drift be used. This factor of two-thirds is used to account for the fact that the maximum drift limits are seldom met over the building's height. (In general, this recommended factor of two-thirds is too small for a steel moment frame unless wind loading criteria governs the design.)

Therefore,

$$\Delta = (0.67)(.005)H = (0.67)(.005)(83.0 \times 12) = 3.34 \text{ in.}$$

Use $\Delta = 3.4$ in.

As related to the UBC:

$$C_1 = ZICS = (1.0)(1.0)(CS) = CS$$

Note that the factor K is omitted from the above equation, since it is a factor assigned to a type of construction in recognition of its inherent resistance to earthquakes and, therefore, is not directly related to stiffness and drift.

Substituting, $T = 0.25 \sqrt{3.4/CS}$

Since both C and S are rather complex functions of T, the solution to this equation might be by trial and error. However, a more direct solution can be achieved by assuming a value for S, which has a rather narrow range of values: $1.0 \leq S \leq 1.5$. Assume $S = 1.30$.

$$C = \frac{1}{15\sqrt{T}} \qquad (12\text{-}2)$$

$$C_1 = CS = \left(\frac{1}{15\sqrt{T}}\right)\left(1.30\right)$$

Substituting into Teal's formula,

$$T = 0.25\sqrt{\frac{3.4}{\left(\frac{1}{15\sqrt{T}}\right)\left(1.30\right)}} = 0.25\sqrt{(3.4)\left(\frac{15}{1.3}\right)T^{1/2}} = 1.57T^{1/4}$$

$$\underline{T^{3/4}} = 1.57; \boxed{T = (1.57)^{1.33} = 1.82 \text{ sec.}}$$

Checking the assumed value of S:

$$\frac{T}{T_S} = \frac{1.82}{1.00} = 1.82 > 1.0$$

$$S = 1.2 + 0.6\left(\frac{T}{T_S}\right) - 0.3\left(\frac{T}{T_S}\right)^2 \qquad (12\text{-}4A)$$

$$= 1.2 + 0.6\ (1.82) - 0.3\ (1.82)^2 = 1.30 = \text{assumed } S$$

Therefore, T = 1.82 sec. is valid.

However, as stated in the SEAOC Commentary, "the Committee feels that the period determination of frame structures should be thoroughly examined if a final period greater than $T = 0.5\ N^{2/3}$ is calculated." "Thoroughly examined" probably implies that the following be taken into account: P-Δ effects, participation of non-moment frames, composite action between girders and floor slab, etc. In order to avoid taking these difficult to determine items into account, the period of the building will be limited by this recommendation.

$$T = 0.5\ N^{2/3}$$

$$= (0.5)(7.0)^{2/3} = 1.83 \text{ sec.}$$

This is approximately the same period obtained by using two-thirds of the allowable drift.

Hence, for design purposes,

$$T_{design} = 1.8 \text{ sec.}$$

( ) . 0.05 × 1.3 = 0.65 < 0.14

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{1.8}} = 0.050 \qquad (12\text{-}2)$$

$$S = 1.2 + 0.6\left(\frac{T}{T_S}\right) - 0.3\left(\frac{T}{T_S}\right)^2 \qquad (12\text{-}4A)$$

$$= 1.2 + 0.6\left(\frac{1.8}{1.0}\right) - 0.3\left(\frac{1.8}{1.0}\right)^2 = 1.31$$

Substituting into Formula (12-1),

$$V = ZIKCSW$$

$$= (1.0)(1.0)(0.67)(.050)(1.31)W$$

$$V = .044\ W$$

$W_{fl.} = (122.5 \times 77.5)(.085) + (400 \times 11.5)(.015) = 874$ kips

$W_{rf.} = (122.5 \times 77.5)(.067) + (400 \times 8.75)(.015) = 687$ kips

$W = 6(874) + 687 = 5930$ kips (total dead load)

$V = 0.044W = (.044)(5930) = 260$ kips (total lateral force)

The total lateral force is distributed over the height of the building in accordance with UBC Formulas (12-5), (12-6) and (12-7). See Fig. 4.2.

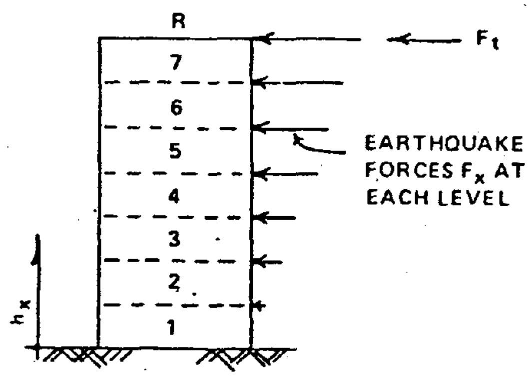

Figure 4.2 Distribution of earthquake forces over height of building.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (12\text{-}5)$$

$$F_t = 0.07TV = .07(1.8)(260) = 33 \text{ kips} \qquad (12\text{-}6)$$

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} = \frac{227\ w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (12\text{-}7)$$

The distribution of lateral forces over the height of the building is shown in Table 4-1.

FROM: Richard M. Simon, Goldberg-Zoino & Associates, Inc.

a) Both the equivalent lateral force and modal analysis procedures are approximate in themselves. There have been few prototype observations to explore the accuracy of either procedure. The equations contained in Chapter 6 are based solely on analytical studies with almost no prototype or even laboratory verification. Because of the approximate nature of the entire analysis procedures it is, in my opinion, inappropriate to add an additional level of sophistication to the equations. The SSI equations are so complex as to make me question whether they could be properly applied by the average structural/civil engineer. There are no guidelines provided in the provisions or commentary as to the approximate magnitude of the effect produced by these equations other than the reduced base shear shall in no case be taken less than 0.7 times the rigid support base shear. Little guidance is provided in the readily available published literature. Although failure to incorporate the effects of SSI is in almost all cases a conservative assumption, it is not clear that applying the equations without complete understanding and with potential errors will lead to conservative design forces and displacements.

b) The Tentative Provisions have emphasized the inelastic behavior of structures in response to earthquake loading. The analytical procedures used as the basis of the Chapter 6 provisions assume a linear response for the superstructure which is inconsistent with the general philosophy of the Tentative Provisions.

From: Henry J. Degenkolb

1. Chapter 6 - Soil-Structure Interaction

First, it must be understood that I was in the minority on this when it was decided to include the soil-structure interaction provisions in ATC 3-06. However, I'm also in the spot of defending them. I do believe that in some (many?) structures the soil-structure relationship is very important. It is so complex, however, that little is known about it and almost everything we do know about it is from theory and not from measurements or observations - especially at the level of strains associated with earthquake motions. Even the theory has only been advanced for certain unique cases such as mat foundations, etc. Therefore, I cannot honestly argue with the reasons that Simon presents under (a) or (b).

3.2 ROSTER

COMMITTEE 3: Foundations

<u>American Society of Civil Engineers</u>

Mr. James G. LaBastie
6252 Powell Road
Parker, Colorado 80134

Phone: 303-771-8641

<u>Association of Soil and Foundation Engineers</u>

Mr. Richard Simon (Chairman)
Goldberg, Zoino, Dunnicliff & Assoc., Inc.
30 Tower Road
Newton Upper Falls, Massachusetts 02164

Phone: 617-969-0050

<u>Interagency Committee on Seismic Safety in Construction</u>

Mr. Joseph V. Tyrrell
Director, Civil/Struc. Div.
Naval Facilities Engineering Comnd.
200 Stovall Street
Alexandria, VA 22332

Phone: 202-325-0064

<u>Structural Engineers Association of California</u>

Mr. William Travis
7851 Mission Center Court
Suite 250
San Diego, California 92108

Phone: 714-291-2800

<u>Applied Technology Council</u>

Mr. Henry Degenkolb
H. J. Degenkolb & Associates
350 Sansome Street
San Francisco, CA 94104

Committee 3 (continued)

Building Seismic Safety Council

Mr. Leroy Crandall, President
L. Leroy Crandall and Associates
711 North Alvarado Street
Los Angeles, California 90026

Phone: 213-413-3550

National Bureau of Standards

Mr. Lawrence Salomone
Secretariat
Committee 3, Foundations
National Bureau of Standards
Rm. B162, Bldg. 226
Washington, D.C. 20234

Phone: 301-921-3128