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GUIDELINES FOR MITIGATION OF SEISMIC HAZARDS IN TILT-UP-WALL STRUCTURES

PHASE I

S.A. Adham
Principal Investigator

This material is based upon work supported by the National Science Foundation under award number PFR-8009736. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the author and do not necessarily reflect the views of the National Science Foundation.

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AGBABIAN ASSOCIATES
El Segundo, California

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16. Abstract (Limit: 200 words)

Efforts to mitigate seismic hazards in tilt-up-wall (TUW) construction are evaluated. Analytical methods that can be used to determine the seismic response of TUW structures are described. Seismic response of a typical warehouse building with plywood roof diaphragms supported on TUW panels was studied using a lumped parameter model. Experimental data on loading and unloading of plywood diaphragms were idealized. A nonlinear stress/strain relationship was included in the model together with viscous damping. A strong motion earthquake record was used as input to the analyses which indicate that the interaction of plywood diaphragms and panels during earthquakes results in amplified forces at the panel connections to the diaphragms, and in amplified moments at the mid-height of the panel. These efforts determine what studies are necessary to achieve guidelines that can be used to provide seismic resistance of these types of components while keeping their cost to a minimum. Specific experimental and analytical studies are recommended for Phase II.

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Robert E. Tobin	Los Angeles, California
Don Musser	Atlanta, Georgia
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Gary Pfuehler	Cincinnati, Ohio
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SECTION 1

INTRODUCTION

A program that evaluates efforts to mitigate seismic hazards in tilt-up-wall (TUW) construction is of great interest to governing agencies and the construction industry, as well as professional engineers. This introductory section provides background information on TUW construction, analysis, and design. It provides a summary of the current research program and its findings, and outlines the organization of this report.

1.1 STATEMENT OF THE PROBLEM

TUW construction is a form of precast concrete construction used primarily for one- or two-story buildings, and in a few cases for multistory buildings. The principal feature of the construction method is the manner in which walls of the building are fabricated and placed. Wall panels are cast in a horizontal position at the site and after curing for as little as two days can be tilted up and moved into place.

The need for such an inexpensive system has increased since World War II, and TUW panel construction has grown very rapidly throughout the United States, including seismically active areas.

The structural integrity of tilt-up buildings during seismic loading has been observed only to a limited degree. Damage to tilt-up buildings was reported in the great Alaskan earthquake of 1964 and in the San Fernando earthquake of 1971. This damage has been attributed mainly to failure of the connections between panels and roof diaphragms. However, TUW structures built with earthquake resistant panel-to-diaphragm connections have not yet been tested by real earthquakes to evaluate their performance.

Both the engineering and construction industries have recognized the need for careful systematic studies concerning the behavior of TUW buildings subjected to seismic loads. Interest in TUW design and construction in seismic zones is evidenced by the SEASC report (1979) where several analysis and design recommendations were made. This report has received wide interest. However, some of the recommendations are already being challenged due to the lack of sufficient supportive experimental data. Therefore, governing agencies, professional engineers, and the construction industry differ considerably in their opinion about the need to enforce specific limits on the earthquake-resistant design of such structures.

There are several basic subjects that must be studied in order to consider mitigating seismic hazards in TUW design and construction: (a) seismic design forces for TUW construction, (b) overall structural behavior, (c) integrity of connections, (d) behavior of roof diaphragms, and (e) design of individual panels. Research has started on some of these subjects.

Major efforts have been launched by several agencies to establish criteria for seismic risk and seismic design input for different types of buildings in various zones of the United States. The integrity of connections of precast elements has attracted much research, because failure of precast construction has occurred mostly in the connections while the structural elements themselves have performed quite well. Evaluations are still needed for seismic design forces for connections and for basic design practices of using statically equivalent lateral forces. Reliable estimates are still needed for design forces that can be absorbed by the hysteretic behavior without component failure, excessive deformation, or loss of stability.

Although some field work has been done, the overall structural behavior of panelized structures has been studied mostly by analytical methods. However, the modeling of typical TUW structures to determine their dynamic response to high-intensity earthquakes has not been reported in the literature. Most of the TUW panel design methods are based on the assumption of a cracked section through the full height of the panel. The assumption of gradual propagation of cracks in the panel has not been incorporated in current analysis methods.

Full-scale tests of TUW panels subjected to out-of-plane low amplitude vibration or high amplitude seismic dynamic forces have not been conducted.

1.2 PURPOSE OF RESEARCH PROGRAM

In September 1980, Agbabian Associates was awarded a Phase I contract by NSF to investigate the following tasks:

a. Categorize TUW construction systems

b. Evaluate seismic hazards for TUW structures

c. Extract failure modes of TUW structures from past earthquake behavior

d. Identify applicable analysis and computer methods

e. Establish material property requirements

f. Perform parametric studies on some pertinent modes of response of TUW components

g. Provide conclusions and recommendations for a Phase II study

Phase I is essentially an exploratory program. The data base resulting from this phase will guide the more specific experimental and analytical work proposed for the Phase II program.

1.3 SUMMARY OF RESULTS

During Phase I, the research effort has been primarily directed towards identifying trends in seismic response of TUW structures and identifying areas that need more experimental and analytical work and therefore warrant further studies. It was concluded that an ensemble of time-history records that correlate with ATC-3* recommendations provide the state-of-the-art tool for describing ground shaking at various sites in the United States. Earthquake ground-motion input at a site is discussed and analysis methods used to determine the response of TUW buildings to earthquake forces are described. It was concluded that the STARS† computer program should be used for this study.

The response of a typical TUW structure with a plywood roof diaphragm supported on TUW panels was studied using a lumped parameter model. Experimental data on loading and unloading of plywood diaphragms were idealized, and a nonlinear stress/strain relationship for the diaphragm was included in the model. Two values of viscous damping for the diaphragm were used in the analyses cases. The 1971 Castaic record was used for earthquake input ground motion. The record was scaled to the 0.40 g effective peak ground acceleration specified for ground shaking in the Los Angeles area. The model is described in detail and the analyses results are discussed.

*The Applied Technology Council's 1978 report *Recommended Comprehensive Seismic Design Provisions for Buildings*.

†STARS is a lumped parameter computer program developed by Agbabian Associates for the dynamic analysis of nonlinear structural systems (*User's Guide for STARS Code*, R-6823-999, Agbabian-Jacobsen Associates, Los Angeles, 1969).

These example analyses show two particularly important trends. First, the interaction of panels and the plywood diaphragm results in amplified accelerations and forces at the connection between the panels and roof diaphragm. Second, the seismic dynamic moment at the midheight of the panels is higher than the equivalent seismic static moment calculated by conventional methods.

Phase I represents only the first step in the development of guidelines for carrying out seismic analyses and evaluation of TUW buildings. Suggested subsequent steps for Phase II are (1) using an ensemble of time-history records for analyses, (2) extending and refining the STARS code analyses of Phase I, (3) conducting an experimental program on panels to include both pseudo-static and dynamic input (these tests would provide data for correlation with results of the analyses of Phases I and II), (4) studying the effects of out-of-plane forces on TUW panels of several height-to-thickness ratios, (5) evaluating the torsional capabilities of TUW buildings, and (6) applying the procedures to a typical TUW structure and evaluating the results.

1.4 REPORT ORGANIZATION

This report is organized in nine sections. Section 2 provides categorization of existing TUW construction in the United States. Section 3 summarizes significant studies of TUW panels. The considerations leading to the choice of design earthquake are given in Section 4. The analytical methods used to determine the response of TUW structures to earthquake forces are given in Section 5. Example analyses of typical TUW structures with plywood roof diaphragms are given in Section 6. Section 7 provides a discussion of failure modes of TUW structures during earthquakes. Conclusions reached from the Phase I study and recommendations for a Phase II study are given in Section 8. References are listed in Section 9.

SECTION 2

CATEGORIZATION OF EXISTING TILT-UP-WALL CONSTRUCTION IN THE UNITED STATES

Two surveys of tilt-up wall (TUW) bearing construction in the United States are represented in this section. The first survey, completed in 1976 by Kripanarayanan and published in the ACI Journal (1980), was limited to seven states. The current survey, conducted by Agbabian Associates as part of an NSF grant to study TUW systems, covers current practices of TUW construction systems nationwide including California, which leads the nation in the total number of TUW buildings constructed.

The Kripanarayanan survey is summarized in Table 2-1. Typical wall panel parameters indicated by this survey are shown in Table 2-2. Several items of this survey are worthy of note:

- Slenderness ratio of wall elements ranges from 35 to 50.
- Vertical reinforcement ratio of wall elements ranges from 0.15% to 0.75%.
- Vertical reinforcement is generally placed in the center of the wall panel.
- Double layers of reinforcement are used in walls having thickness greater than 7-1/2 in.

The Agbabian survey shows that panels as high as 65 ft, with height-to-thickness ratio in excess of 60, were built in California. However, general panel dimensions and reinforcement are similar to those found in the first survey. The second survey addresses some important structural features of components of TUW structures in different parts of the nation and has resulted in the general classifications given in Table 2-3.

Elements of the TUW system (such as roof diaphragms, pilasters, etc.) that the survey found to be prevalent were selected to be the representative type used in the designated region. For example, roof diaphragms used for TUW construction in California are, in general, either wood, steel, or reinforced concrete. However, since more than 90% of these diaphragms are wood, the entry for diaphragms in California categorizes this element as wood in Table 2-3.

The second survey indicates that the majority of TUW construction is used for single-story buildings with the exception of several TUW two- and three-story buildings. Cast-in-place pilasters were commonly used in the early development of TUW construction systems. However, their current use has been limited to systems that carry heavy roof loads where they are cast monolithically with the panel. Also there is a growing tendency in several states to do away with the parapet part of the TUW and load the roof joists directly on the top of the TUW.

Typical details of TUW construction are given in Figures 2-1 through 2-8.

TABLE 2-1. SURVEY OF TILT-UP BEARING-WALL PANEL DIMENSIONS AND REINFORCEMENT IN THE UNITED STATES (Kripanarayanan, 1980)

Region	Panel Dimensions												Reinforcement					Approximate TUW Buildings Constructed	Remarks
	Unsupported Height, L_u, ft			Panel Width, ft			Panel Thickness, h, in.			Slenderness Ratio, L_u/h			As/Ag, %			Location			
	L	A	U	L	A	U	L	A	U	L	A	U	L	A	U	S	D		
Washington and Oregon	20	21	32	20	22	24	5½	5½	8½	43	45	45	.25	.25	.25	✓	*	100 Structures since 1970	Typical Panel Dimensions are usually compatible with a weight of approximately 30 tons
Colorado	16	17	25	12	12	18	5½	5½	8	35	37	37	.25	.25	.25	✓	*	100 Structures since 1970	
Utah	15	21	32	20	20	20	3½	5½	8	41	45	45	.25	.25	.25	✓	*		
Ohio	16	20	30	15	20	25	5½	5½	7½	35	45	45	.25	.25	.50	✓	*	200 Structures since 1965	
Florida and Texas	22	22	25	16	22	30	5½	6	6	48	48	50	.25	.37	.50	✓	*	100 Structures since 1970	

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L = lower limit; A = average limit; U = upper limit.

*Double layer of reinforcement is typically used in 7½ in. or greater panels.

Survey taken in 1976.

TABLE 2-2. WALL PANEL PARAMETERS
(Kripanarayanan, 1980)

Parameter	Value
Unit weight of concrete (w), pcf	150
Compressive strength of concrete (f'_c), psi	≤4,000
Yield strength of reinforcement (f_y), ksi	60
Capacity reduction factor (ϕ)	1.0
Panel thickness (h), in.	5½, 6½, 7½
Reinforcement ratios (ρ), percent	0.15, 0.25, 0.50, 0.75
Transverse loads (q_u), psf	0, 15, 30, 45
End eccentricities (e), in.	1.0, h/2; h/2 + 3½
Slenderness ratios ($k\ell_u/h$)	20 through 50

TABLE 2-3. SURVEY OF TILT-UP CONSTRUCTION SYSTEM IN THE UNITED STATES

Region	Roof Diaphragms	Pilasters	Foundation	Ledgers	Wall Connected to Floor	Code H/t Limit	Remarks
California	Wood	No	Continuous and isolated	60% Wood ‡ 40% Steel sections	Yes	25 using ACI 318-77 25-42 w/slenderness analysis 42-50 in special cases	Diaphragm designed to resist lateral load
Washington and Oregon	Wood	No	Continuous	Wood	Yes	see Calif.	Diaphragm designed to resist lateral loads
Arizona and Utah	Wood	No	Continuous	Wood	Yes	No limit	Diaphragm designed to resist lateral loads
Texas and Southeast Georgia, Florida, Tennessee, Carolinas	Steel	No	Continuous and isolated	Steel sections act as diaphragm chord	Yes	No limit	Diaphragm action depends on proper location of chords
Ohio and North Central States	Steel	No	Continuous and isolated	Steel sections or pockets in wall with bearing plates *	Yes	No limit (Generally $\leq$ 50)	Diaphragm action depends on proper location of chords and bracing
Colorado	Prestressed concrete	Yes	Continuous	Pilasters	Yes	No limit	
New Jersey and Mid-Atlantic Coast States	Steel	Steel cols. or cast in place or no pilasters	Isolated	Steel section	Yes	No limit	
Northeast	TUW is gradually getting popular in some areas						

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Note: Information shown in this table indicates the system that is mostly used in a particular region

Vertical ties between panels are almost eleminated in all states

‡ Approximate Ratios

* Growing tendency to do away with parapet part of TUW and load roof joists directly on the top of TUW.

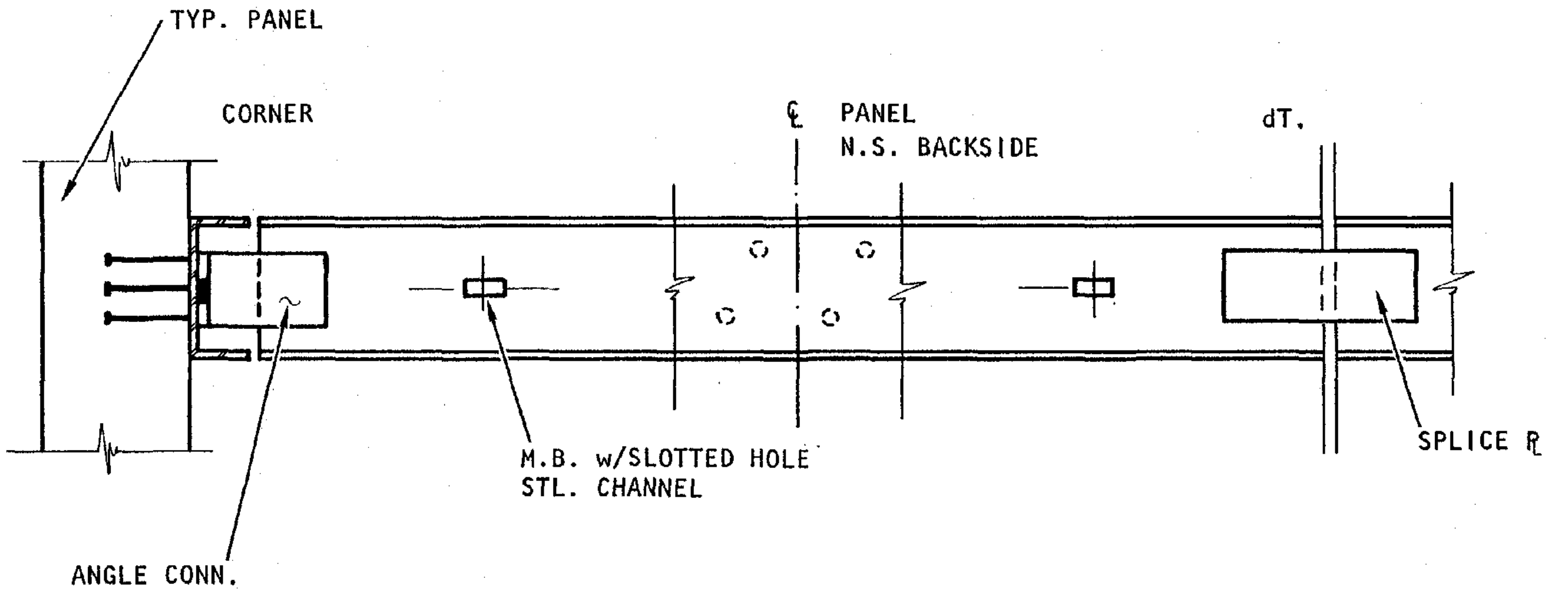


FIGURE 2-1. TYPICAL LEDGER AND CONNECTION (STEEL)

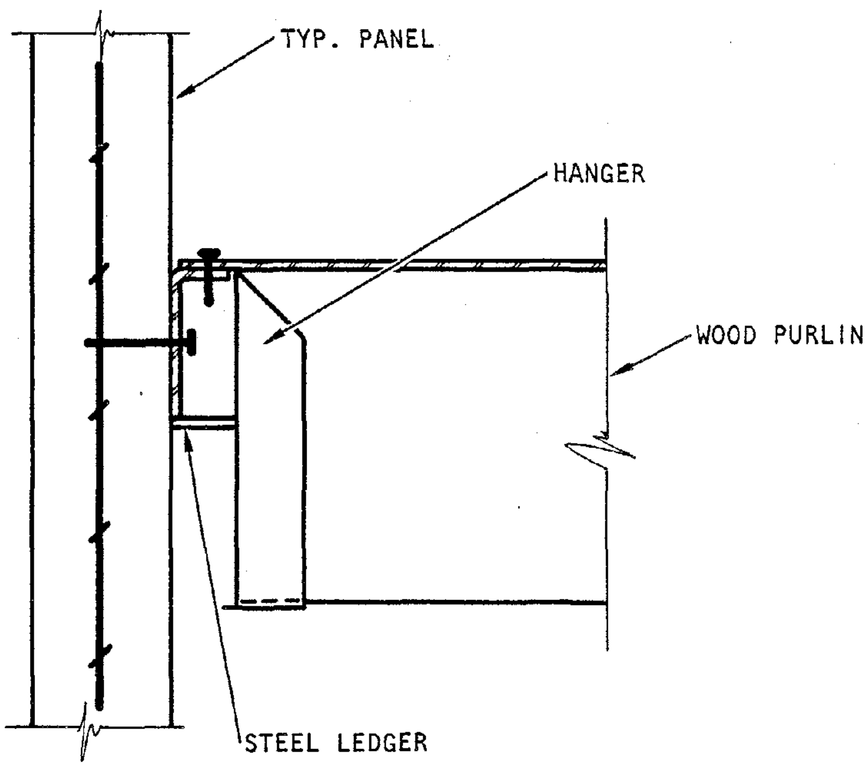


FIGURE 2-2. TYPICAL WOOD PURLIN AND STEEL LEDGER

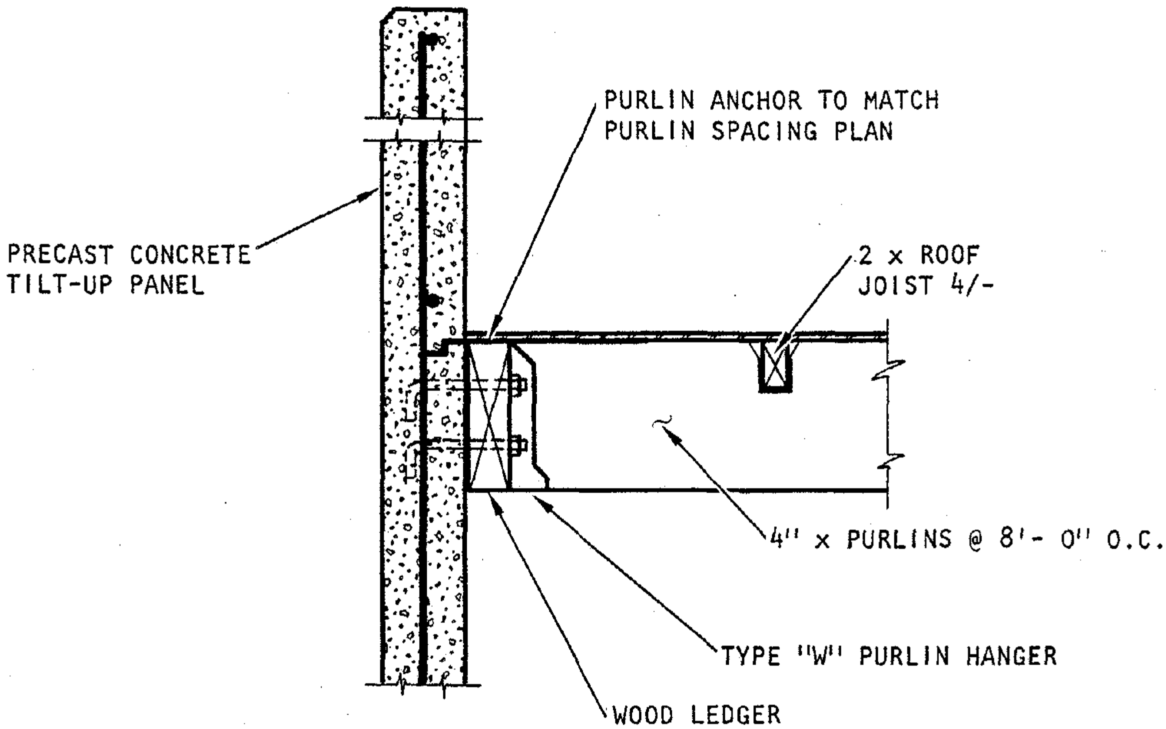


FIGURE 2-3. WOOD LEDGER AND WOOD PURLIN

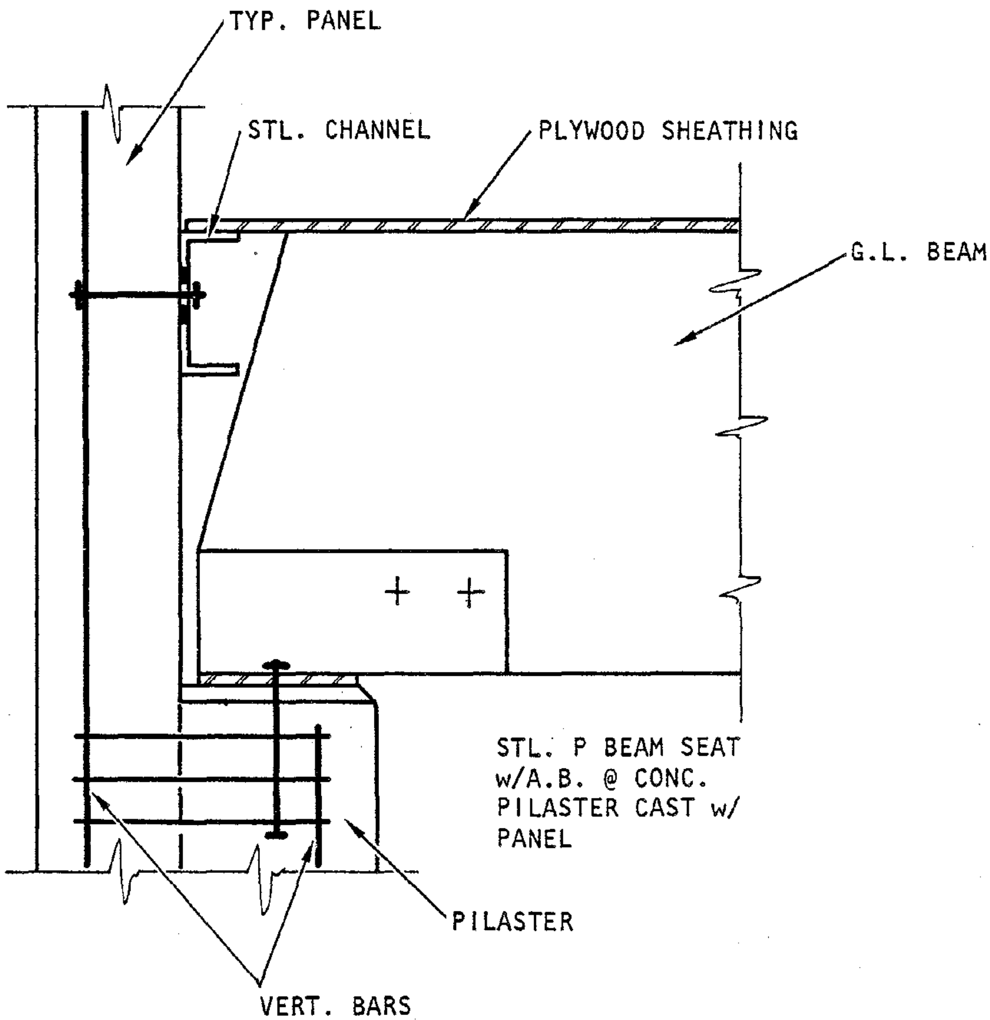


FIGURE 2-4. GLUELAM BEAM AT PILASTER

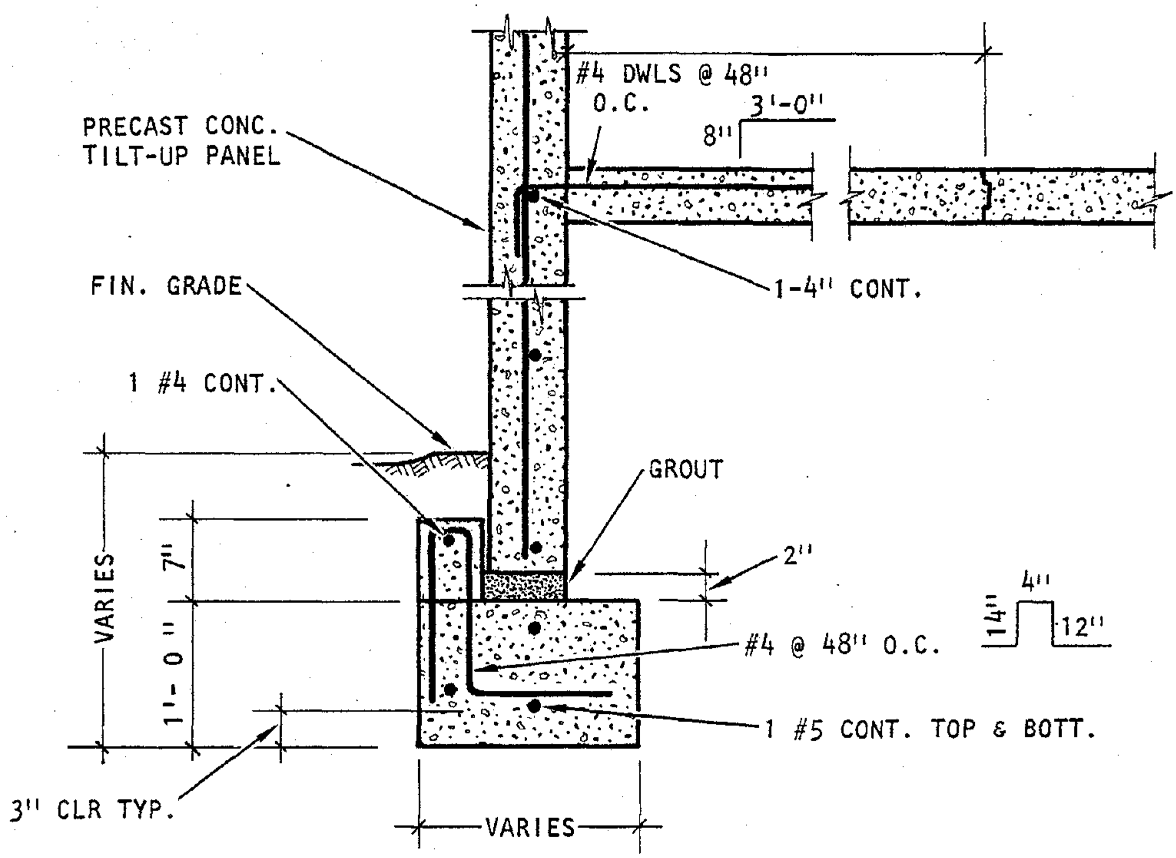


FIGURE 2-5. PANEL FOOTING

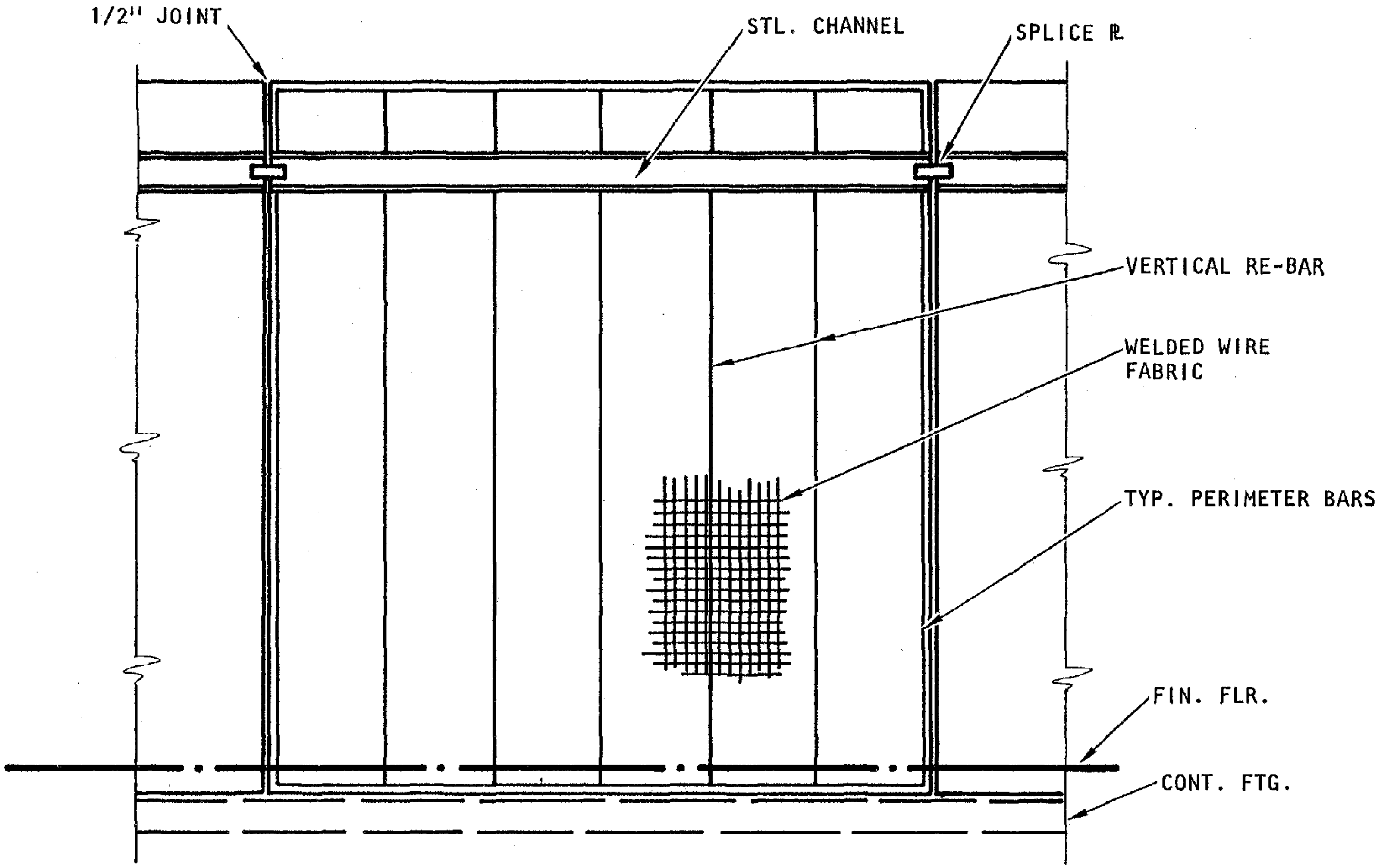


FIGURE 2-6. TYPICAL P.C. CONCRETE PANEL

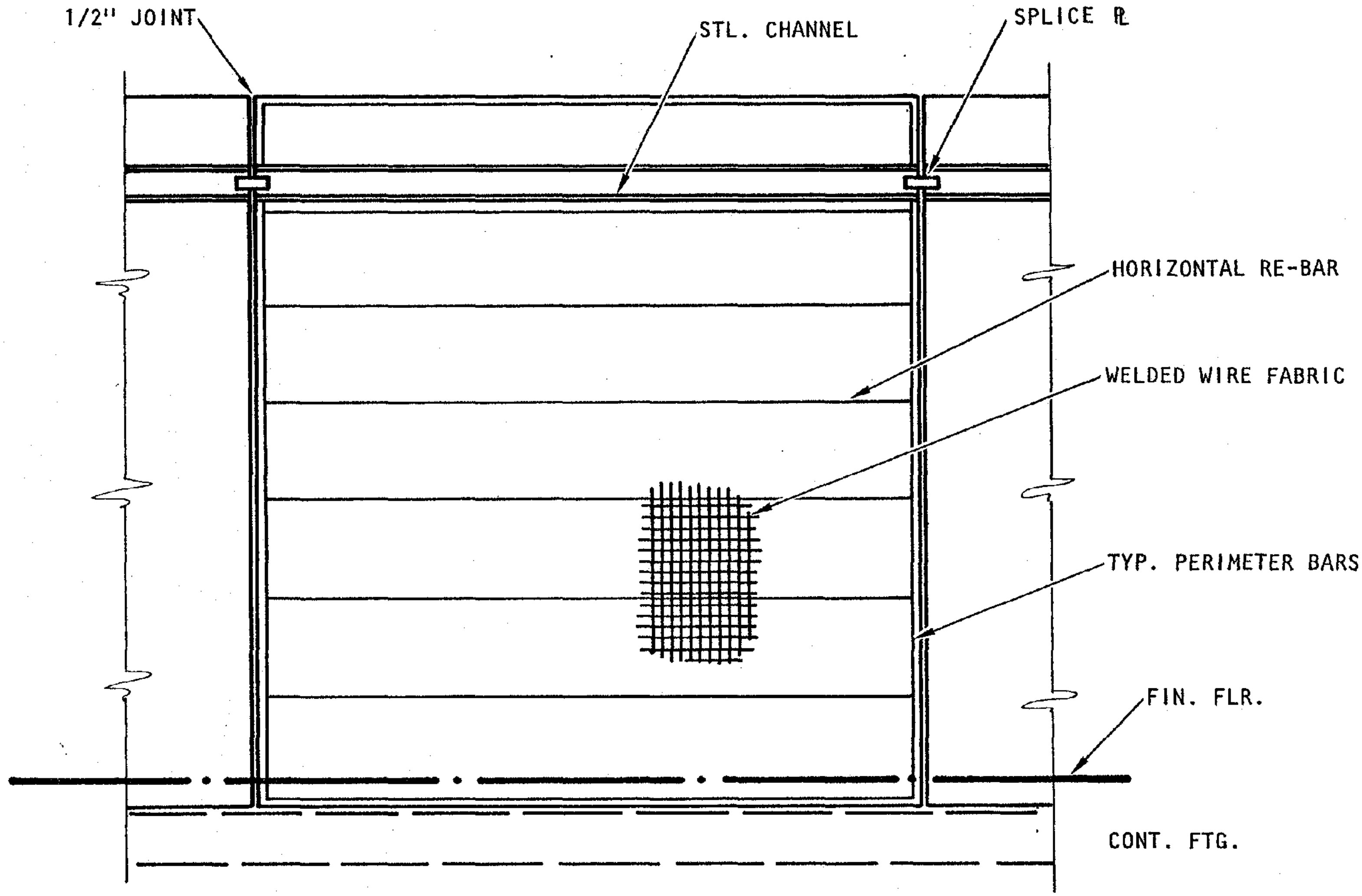


FIGURE 2-7. TYPICAL P.C. CONCRETE PANEL

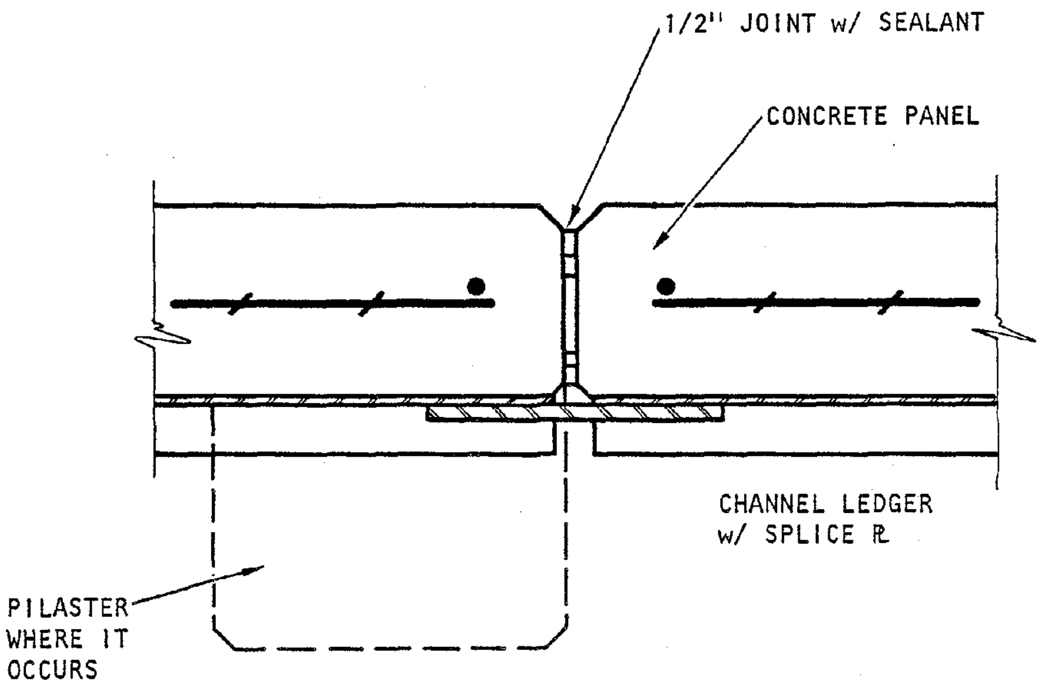


FIGURE 2-8. TYPICAL P.C. PANEL JOINT

SECTION 3

PRIOR STUDIES OF TILT-UP-WALL BEARING STRUCTURES

3.1 INTRODUCTION

A review of prior studies of TUW buildings was conducted in order to evaluate the current state of knowledge in this area and to use this information in the planning and development of the present research program including a Phase II proposal and utilization plan. As outlined by Becker and Llorente (1979), the five principal concerns of these studies are (1) seismic design force levels, (2) precast panels, (3) connections, (4) roof diaphragms, and (5) overall structural integrity.

Procedures and results of these prior studies are summarized in this section, with the exception that the topic of seismic design force levels and seismic motions is covered in the next section, Section 4. This topic was studied by ABK (1980), and Section 4 will cover the recent work on this subject. The remaining areas of concern will be addressed in the following subsections.

3.2 PRECAST PANELS

Current design of TUW panels considers forces encountered during installation and during out-of-plane buckling due to vertical loads and lateral wind and seismic loads. A literature survey on reinforced concrete shear walls subjected to in-plane loads was conducted by Elsesser (1977). He indicated that a variety of load tests on reinforced concrete shear walls have been undertaken over the past 25 years. The first significant tests were monotonic tests of shear wall panels, both with and without openings, carried out at Stanford University in the 1950's (Benjamin and Williams, 1957,1958). The most recent tests have been at the PCA Laboratories in Illinois (Fiorato et al., 1977; Cardenas, 1973), in

New Zealand (Paulay and Santhakumar, 1977; Paulay and Spurr, 1977), and in Yugoslavia (Anicic and Zamolo, 1977), all using cyclic loads. Other tests have been conducted in Japan (Muto et al., 1974; Yamada et al., 1974, 1977), in Canada (Mirza and Jeager, 1977), and at the University of Illinois (Otani, 1977). Single wall tests are summarized in Table 3-1, coupled walls in Table 3-2, and multistory walls in Table 3-3.

Slender wall tests were conducted at the PCA Laboratories (PCA, 1969); however, only concentric loads were used for the tests. Tests on panels using out-of-plane loads have not been reported in the literature.

A joint committee of ACI-SEASC (1980) is currently conducting a test program on slender walls. The test program involves testing of tall and slender wall panels under combined axial and large lateral loads. Panels are 4 ft wide by 24 ft high, varying in wall thickness to represent H/t ratios of 30, 40, 50, and 60 for concrete panels. A total of 12 concrete panels are being tested, each constructed as close to normal field conditions as possible. The bottom of each wall panel rests on a half-steel pipe to minimize the effect of fixity at the base. Wall panels are reinforced with four No. 4 vertical Grade 60 rebars. Axial load is applied on a ledger angle at the top of the panel to simulate the normal tributary roof load. Lateral load is applied on one face of the wall panel using an air bag. Deflections of each panel due to increases in lateral loads are measured at eleven stations along the height of the wall.

The review of prior studies indicates the following: (a) seismic design forces for TUW panels have not been fully addressed; (b) out-of-plane dynamic response has not been investigated; and (c) cyclic behavior and corresponding hysteretic load-deflection relationships need to be developed in order to establish design forces that can be absorbed by TUW without component failure, excessive deformation, or loss of stability.

TABLE 3-1. SUMMARY OF SEVERAL LOAD TESTS OF SINGLE PANEL SHEAR WALLS (From Elsesser, 1977)

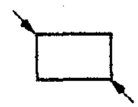
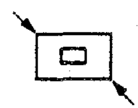
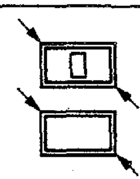
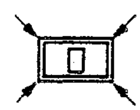
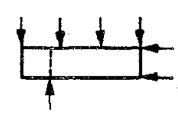
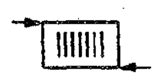
WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
SINGLE PANELS SOLID	Benjamin and Williams, 1957 (Stanford)		MONOTONIC
SINGLE PANELS WITH OPENINGS	Benjamin and Williams, 1958 (Stanford)		MONOTONIC
SINGLE PANELS SOLID AND WITH OPENINGS	Yamada et al., 1974 (Japan)		MONOTONIC
SINGLE PANELS WITH OPENINGS	Yamada et al., 1977 (Japan)		CYCLIC
SINGLE CANTILEVER WALL SOLID	Cardenas and Magura, 1973 (ACI)		MONOTONIC
SINGLE PANEL SLITTED	Muto et al., 1974 (Japan)		CYCLIC

TABLE 3-2. SUMMARY OF SEVERAL LOAD TESTS OF COUPLED SHEAR WALLS (From Elsesser, 1977)

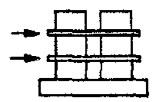
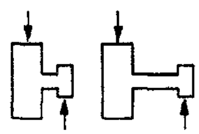
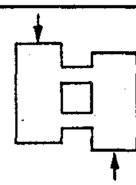
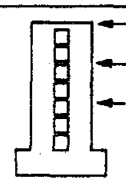
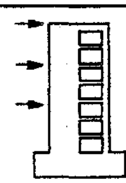
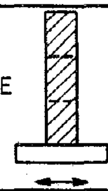
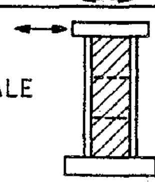
WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
COUPLED WALLS SLAB LINK	Mirza and Jaeger, 1977 (Canada)		MONOTONIC
COUPLED WALLS BEAM LINK	Anicic and Zamolo, 1977 (Yugoslavia)		CYCLIC
COUPLED WALLS BEAM LINK	Various PCA Publications (Illinois)		REVERSING
COUPLED WALLS BEAM LINK	Paulay and Santhakumar, 1977 (New Zealand)		CYCLIC
LINKED WALL-FRAME	Paulay and Spurr, 1977 (New Zealand)		CYCLIC

TABLE 3-3. SUMMARY OF SEVERAL LOAD TESTS OF MULTISTORY SHEAR WALLS (From Elsesser, 1977)

WALL TYPE	REFERENCE	CONFIGURATION	LOADING TYPE
MONOLITHIC 3-STORY WALL	Otani, 1977 (Illinois)	$\frac{1}{6.25}$ SCALE 	SHAKING TABLE
MONOLITHIC 3-STORY WALL	Fiorato et al., 1977 (PCA)	$\frac{1}{3}$ SCALE 	CYCLIC

3.3 CONNECTIONS

Integrity of connections with precast elements has attracted much research because failure of precast construction has occurred mostly in or close to these connections. A great deal of effort has been directed toward testing various configurations and types of connections most commonly used in the construction industry (Aswad, 1979; Spencer and Neille, 1976; Davies, 1967; Birkeland, 1966; PCA Committee, 1969).

Connections for tilt-up construction can be categorized into three general groups according to the forces that the joints transfer: compression, tension, and shear. Compression and tensile joints have been tested to a limited degree (Hawkins, 1978). Shear connections have been more thoroughly tested. Dry shear joints (using bolted or welded details) have been tested for cyclic action by several investigators, including Spencer and Neille (1976) and Davies (1967). Wet shear joints (using reinforced or unreinforced cast-in-place concrete) have been tested for various types of edge surfaces of panels and connection reinforcement. An overview of work done with connections is included in the Becker and Llorente (1977) article, which indicated that connections are the critical component in panelized construction with respect to seismic response. Observations of earthquake damage in panelized construction have always indicated cracking in connection areas (Polyakov, 1969, 1974). While connections are normally thought of as the mechanism by which the panels are joined and the load transferred, they also serve as regions of energy dissipation. The ratio of connection strength to gross panel strength was found by Lugez and Zarzycki (1969) to range from a high of 0.88 to a low of 0.19. In a comparable finite element analytical study, Backler et al. (1973) found that the joint strength ranged from a high of 0.51 to a low of 0.17 of the wall strength.

Further studies of connections are underway in the United States by Hegemier of the University of California, San Diego, Scott of the Consulting Engineering Group, and Russel Brown of Clemson University.

3.4 ROOF DIAPHRAGMS

In TUW construction, diaphragm action refers to the transmission of shear forces through the roof of the structure to the lateral load resisting system. In his discussion of diaphragm action, Hawkins (1977) indicated that diaphragms are usually classified as rigid or flexible. Rigid diaphragms transmit loads to resisting elements in proportion to the relative rigidity of those elements and can cause torsional effects when the center of mass is eccentric from the center of rigidity. Flexible diaphragms transmit loads in proportion to the area tributary to each element and do not transmit rotational forces. Between these two limits there is a wide range of flexibilities where the behavior depends on the rigidity of both the diaphragm and the lateral load resisting system (Department of Army et al., 1973).

Adham and Ewing (1978) studied the response of plywood, diagonal-sheathed, and straight-sheathed wood diaphragms using a lumped parameter model. Experimental data on monotonic loading and unloading of wood diaphragms were idealized. This study was part of an NSF Phase I grant to provide a "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings." The data base for diaphragms was extended in an NSF Phase II follow-on study of this program (ABK, 1980), where both steel and wood diaphragms were tested under quasi-static and dynamic earthquake loads (Figs. 3-1 and 3-2). A typical load deflection model for wood diaphragms is idealized in Figure 3-3. The study indicated that plywood diaphragms can amplify earthquake ground motions by a factor of approximately 2, a significant result since the majority of TUW structures built in highly seismic areas have plywood diaphragms (see Table 2-3). However, current simplified analysis methods assume the TUW panel to have an equivalent lateral seismic load reaction at the top equal to half of the total lateral seismic load acting on the panel. This assumption is not conservative since it results in underestimating the reaction at the top of the panel approximately by a factor of 2. A detailed discussion of diaphragm response is provided by Ewing et al. (1980).

3.5 OVERALL STRUCTURAL INTEGRITY

In a fundamental sense, TUW bearing structures are a collection of vertical cantilever beams. For low amplitude motion, Becker and Llorente (1977) indicated that these behave in a manner similar to any bearing wall structure of like geometry. However, as soon as the response enters the nonlinear, inelastic range, TUW structures begin to behave in a distinctive fashion. This unique reaction stems from the effect of connections and roof diaphragms in terms of both stiffness and strength. In addition, the seismic response of TUW structures is also strongly influenced by soil/structure interaction and various types of coupling between wall elements. Refined analyses and testing of full TUW structures have not been reported in the literature.

3.6 STUDIES BY THE STRUCTURES ENGINEERS ASSOCIATION OF SOUTHERN CALIFORNIA (1979)

Recognizing a need for design guidelines for tilt-up wall construction, the Structural Engineers Association of Southern California issued a report on recommended practices (SEASC, 1979). The guidelines include seven areas of consideration: span-to-thickness ratio, joinery between wall panels, panel anchorage, chord element and connections, panel footings, minimum panel reinforcement, and shrinkage. The report recommends certain general design criteria based on the overall performance of existing designs. Discussion of connections is directed toward the kinds of forces that these joints must resist and the performance of connections most commonly used in the industry. Span-to-thickness ratios are discussed more thoroughly. A design method for determining wall capacity is provided, where limiting h/t values are given for certain types of wall panels. This method is discussed in Section 5.

The report, however, indicates a "lack of test data to verify analysis and design techniques used for wall panel design" and warned that openings in panels warrant special considerations.

The tests by the ACI-SEASC Slender Wall Committee currently underway in the Los Angeles area are expected to provide data on the behavior of TUW panels under equivalent static lateral loading and eccentric vertical loading.

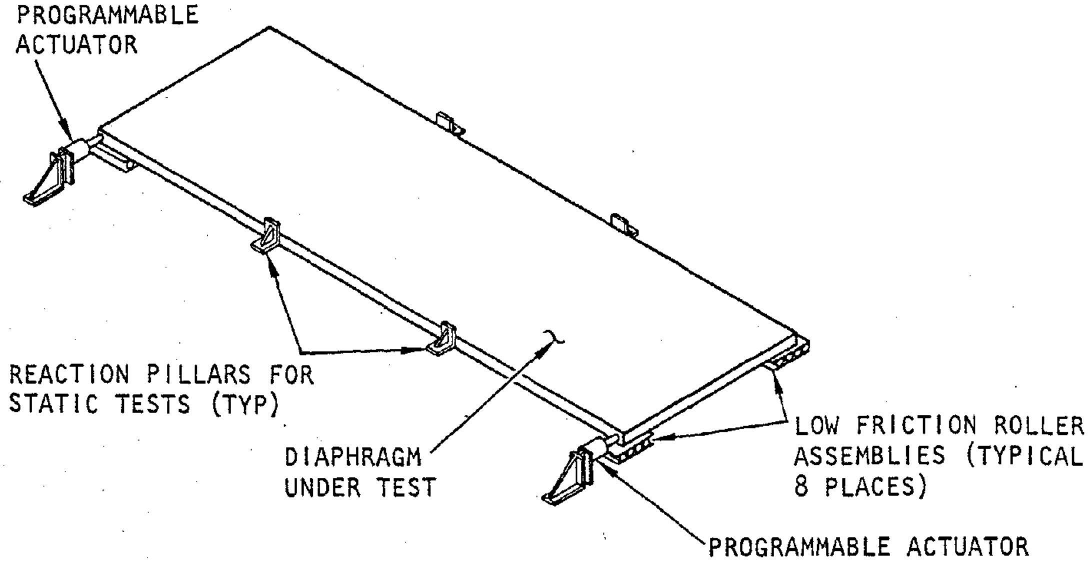


FIGURE 3-1. TEST SETUP FOR QUASI-STATIC TESTING OF DIAPHRAGMS (ABK, 1980)

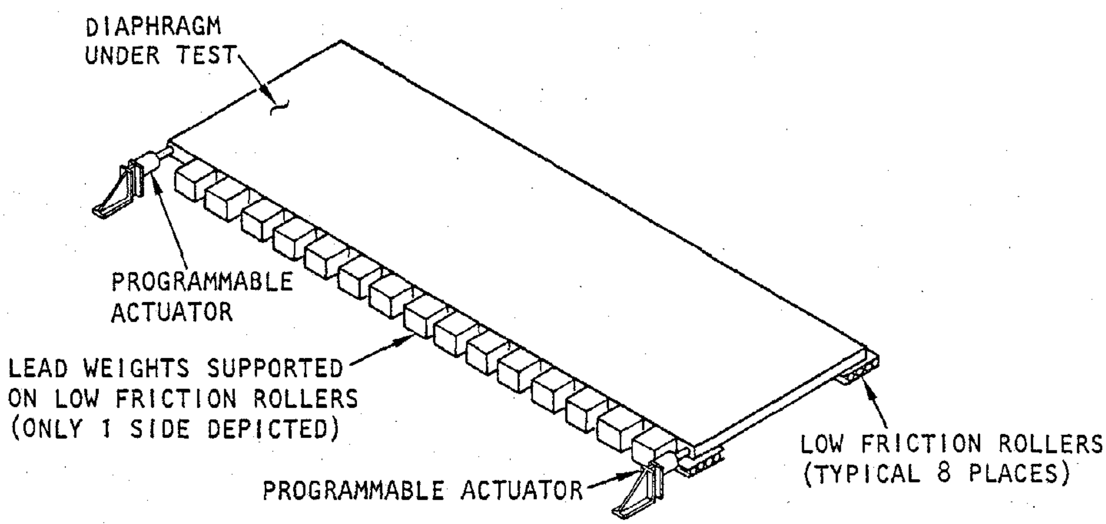
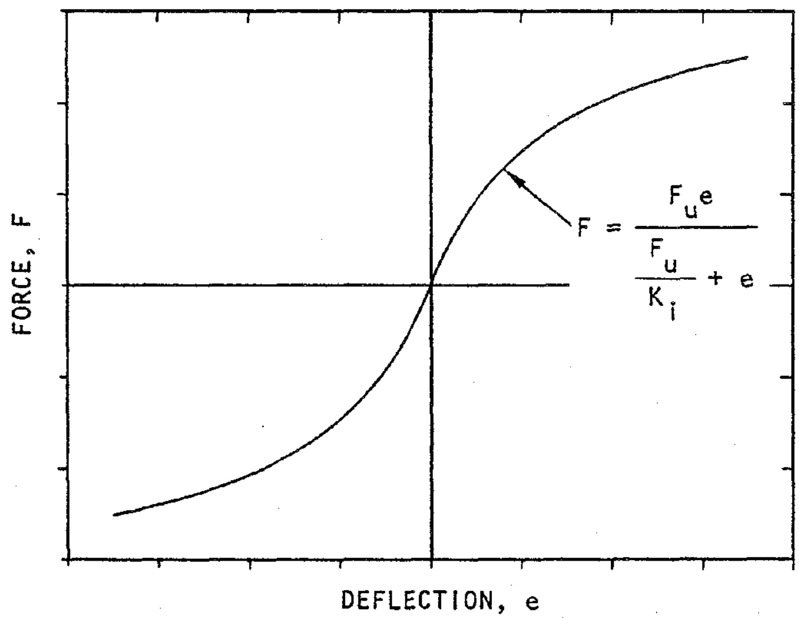
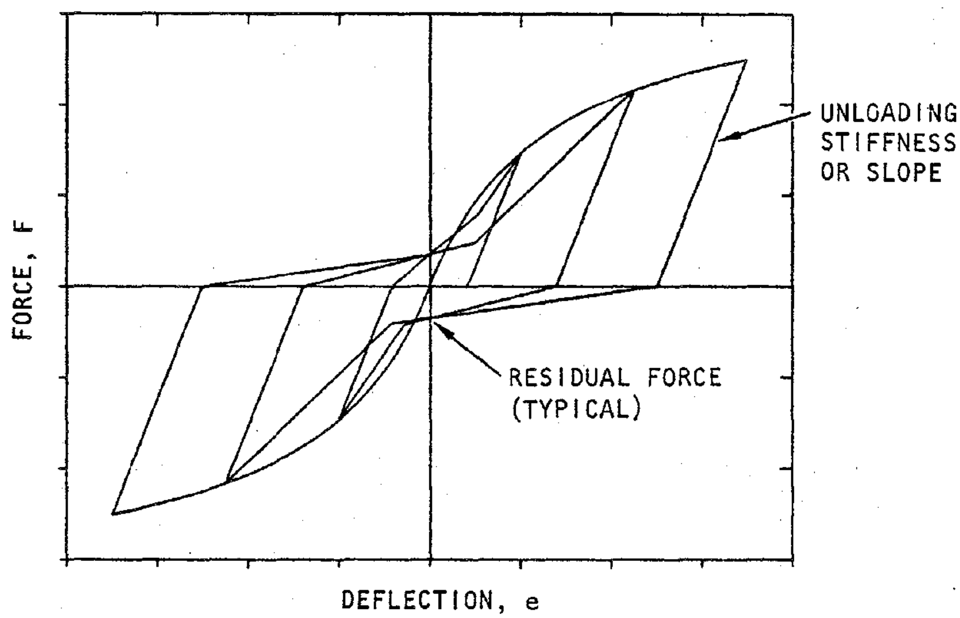


FIGURE 3-2. TEST SETUP FOR DYNAMIC TESTING OF DIAPHRAGMS (ABK, 1980)



(a) Force-deflection envelope of model



(b) Typical cyclic load-deflection diagram for model

FIGURE 3-3. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS (ABK, 1980)

SECTION 4

CONSIDERATION LEADING TO CHOICE OF DESIGN EARTHQUAKES AND RELATED BUILDING CAPABILITY

This section provides background information for selecting design earthquake motions for structures. A detailed discussion of the subject is given by Adham and Ewing (1978) and ABK (1980).

4.1 EARTHQUAKE THREAT TO BUILDINGS

4.1.1 EARTHQUAKE DAMAGE

The 1971 San Fernando earthquake occurred on the fringe of the very large metropolitan area of Los Angeles and provided the first really comprehensive test of modern U.S. building code provisions. It occurred also within the boundary area of a large network of strong-motion accelerographs.

Seismograms from the San Fernando earthquake indicate a background acceleration level of from 0.35 to 0.5 g, with a maximum spike of acceleration at one location of more than 1.0 g. Coupled with this high level of ground acceleration, large ground displacements and surface faulting occurred. The time duration of violent ground motion lasted only from 10 to 12 sec, whereas in a magnitude 8.0+ earthquake, the time duration would be approximately 30 to 40 sec in the epicentral area.

4.1.2 EARTHQUAKE-RESISTANT DESIGN

Building codes intend to provide minimum requirements for lateral force resistance to prevent building collapse under the conditions of the most probable severe earthquake to which the structure would be subjected.

The damage experienced during the 1971 San Fernando earthquake demonstrated that, in general, modern structures designed according to the minimum requirements of the building codes received only architectural damage in areas where the accelerations were 20% g or less. There was minor-to-appreciable structural damage in the 20% to 30% g range, and the damage to buildings of minimum design varied from appreciable damage to collapse in the area of very strong shaking. Had the shaking lasted longer, as it would have in a larger earthquake, the damage would have been more severe and more modern structures would have collapsed.

Building codes that have been revised since the 1971 San Fernando earthquake are now requiring the use of considerably higher coefficient values for computing lateral forces. These new values represent lateral forces that are closer to actual measured earthquake motion loadings determined from measured records. The direction is also toward the requirement for an analysis of building response that considers the time variation of ground motion to validate maximum equivalent static design coefficients. The importance of the critical "use" or "occupancy" of a building is now being recognized in the seismic code requirements. Also, more attention is being paid to nonstructural building components and systems.

4.2 DESIGN EARTHQUAKE EVENT

The earthquake criteria for earthquake-resistant design discussed in this phase of the study is based on the design philosophy that

a. For moderate intensity earthquakes, little structural damage should result, but some damage to nonstructural elements in the building would be allowable.

b. For very high intensity earthquake ground motion, some structural damage could occur, but there should be no possibility of structural collapse. These very high intensity earthquake ground motions would be generated by the design earthquake event.

A design earthquake event specifies the maximum values of certain characteristic parameters that may reasonably be expected to occur over the design life of the structure, or, in the case of a seismic safety plan for existing buildings, over the remaining life of the structure. This design earthquake generally specifies the maximum ground displacements, velocities, and accelerations that are likely to occur. Some measure of the time duration of the ground motions is also included.

Development of criteria for a specific site generally requires consideration of major geological features; tectonics for the site, i.e., the types, locations, and arrangement of faults; seismic history including records of intensity and ground motion, if available; and local soil conditions. Engineering judgment and, in some cases, ground-motion calculations, provide the basis for selecting the required design earthquake event.

Several methods can be used to develop the seismic input at a site. The procedure outlined by the Applied Technology Council (ATC-3, 1978) will be used in this study as a basis for developing seismic input at a site.

4.3 PROCEDURE OUTLINED BY ATC-3 (1978) FOR SPECIFICATION OF EARTHQUAKE GROUND SHAKING AND DEFINITION OF SEISMIC HAZARD INDEX

4.3.1 INTRODUCTION

Two earthquake ground shaking regionalization maps were developed by ATC-3. These maps are based on the following considerations: (1) the design lateral force and the period of a structure should take into account the distance from anticipated earthquakesources; (2) the probability of exceeding the design ground shaking should, as a goal, be roughly the same in all parts of the country; and (3) the regionalization maps should not attempt to delineate microzones. Any such microzonation should be done by experts who are familiar with localized conditions.

4.3.2 DESIGN EARTHQUAKE GROUND MOTIONS

ATC-3 defines the "design ground shaking" for a location as the ground motion that an architect or engineer should have in mind when he designs a building that is to give proper protection to life safety. A smoothed elastic response spectrum for single degree-of-freedom system (Newmark and Hall, 1969) is used.

4.3.3 GROUND MOTION PARAMETERS

The intensity of design ground shaking is represented by two parameters. These parameters are called the effective peak acceleration (EPA) and effective peak velocity (EPV). The EPA is proportional to spectral ordinates for periods in the range of 0.10 to 0.5 sec, while the EPV is proportional to spectral ordinates at a period of about 1 sec. The constant or proportionality (for the 5% damped spectra) is set at a standard value of 2.5 in both cases.

For a specific actual ground motion of normal duration, EPA and EPV can be determined as illustrated in Figure 4-1. The 5% damped spectrum for the actual motion is graphed and fitted by straight lines at the periods mentioned above. The ordinates of the smoothed spectrum are then divided by 2.5 to obtain EPA and EPV.

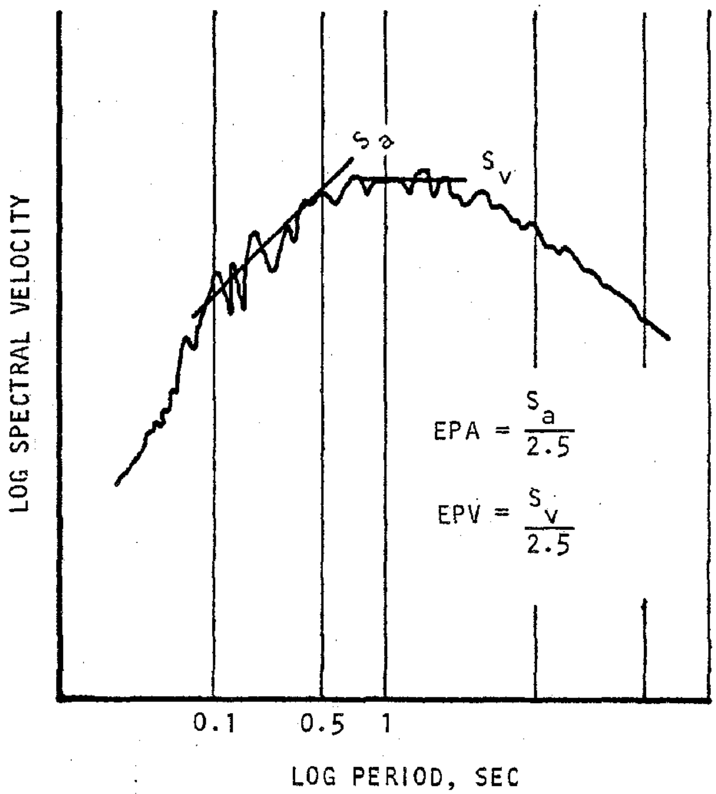


FIGURE 4-1. (ATC, 1978)

4.3.4 DESIGN ELASTIC RESPONSE SPECTRA

The EPA and EPV maps are shown in Figures 4-2 and 4-3 (see ATC, 1978, for a complete description of these maps) and have four contours whose associated values of EPA or EPV are as follows:

	EPA	EPV
Contour	Map 1	Map 2
1	0.05 g	1.5 in./sec
2	0.10 g	3
3	0.20 g	6
4	0.40 g	12

For simplicity in application and to avoid the need for interpretation between contours, the maps for both EPA and EPV have been divided along county boundaries into seven levels of motion (ATC, 1978). A seismic hazard index, which reflects the ability of different types of construction to withstand the effects of earthquake motions, is also included.

Spectral shapes representative of the different soil conditions discussed in ATC-3 (1978) were selected on the basis of statistical studies (Fig. 4-4). These spectra were simplified to a family of three curves by combining the spectra for rock and stiff soil conditions leading to the normalized spectral curves shown in Figure 4-5.

Recommended ground motion spectra for 5% damping for the different map zone levels are thus obtained by multiplying the normalized spectra values shown in Figure 4-5 by the values of effective peak ground acceleration. Soil profile factors were also derived for the above response spectra.

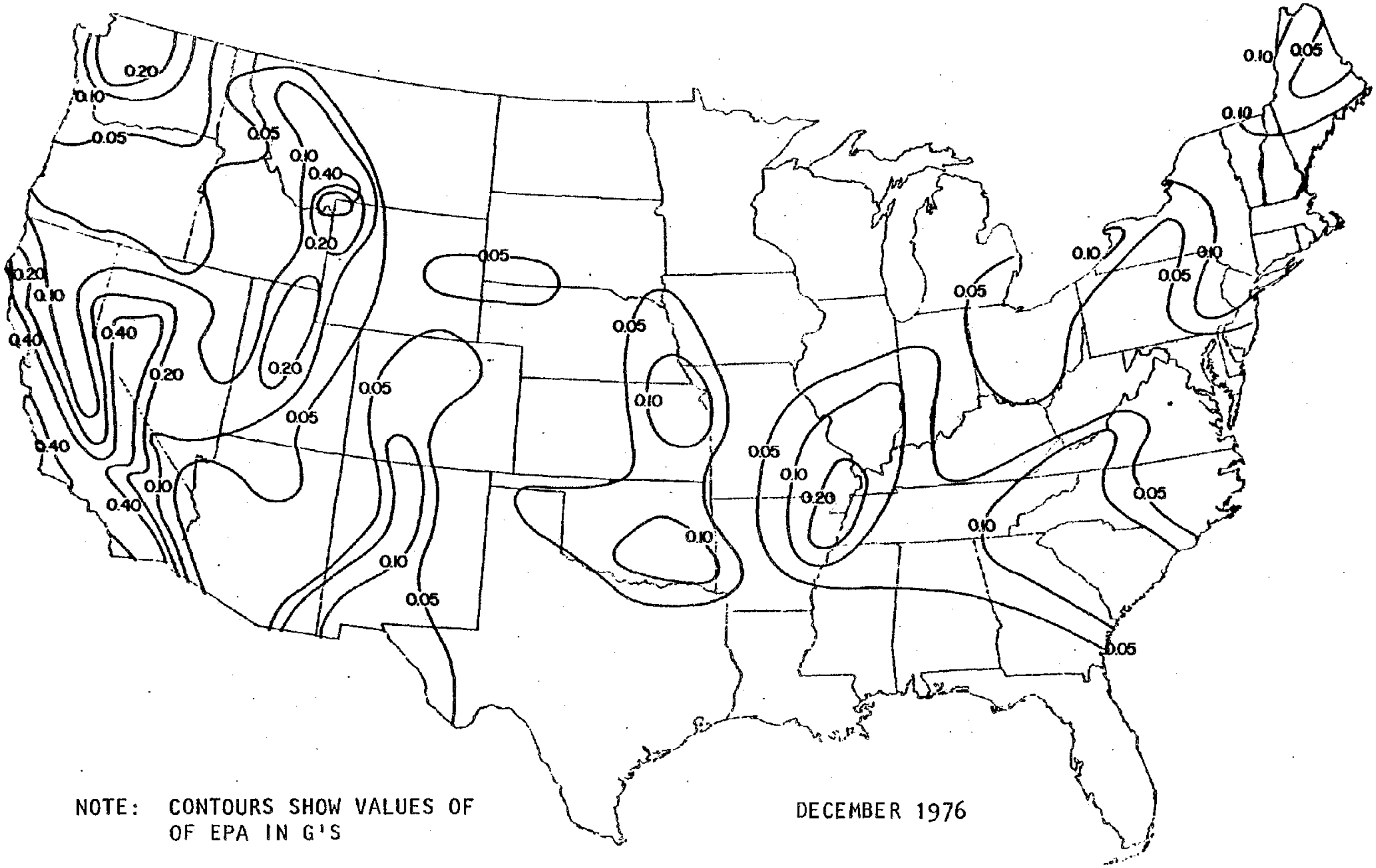


FIGURE 4-2. EFFECTIVE PEAK ACCELERATION MAP (ATC, 1978)

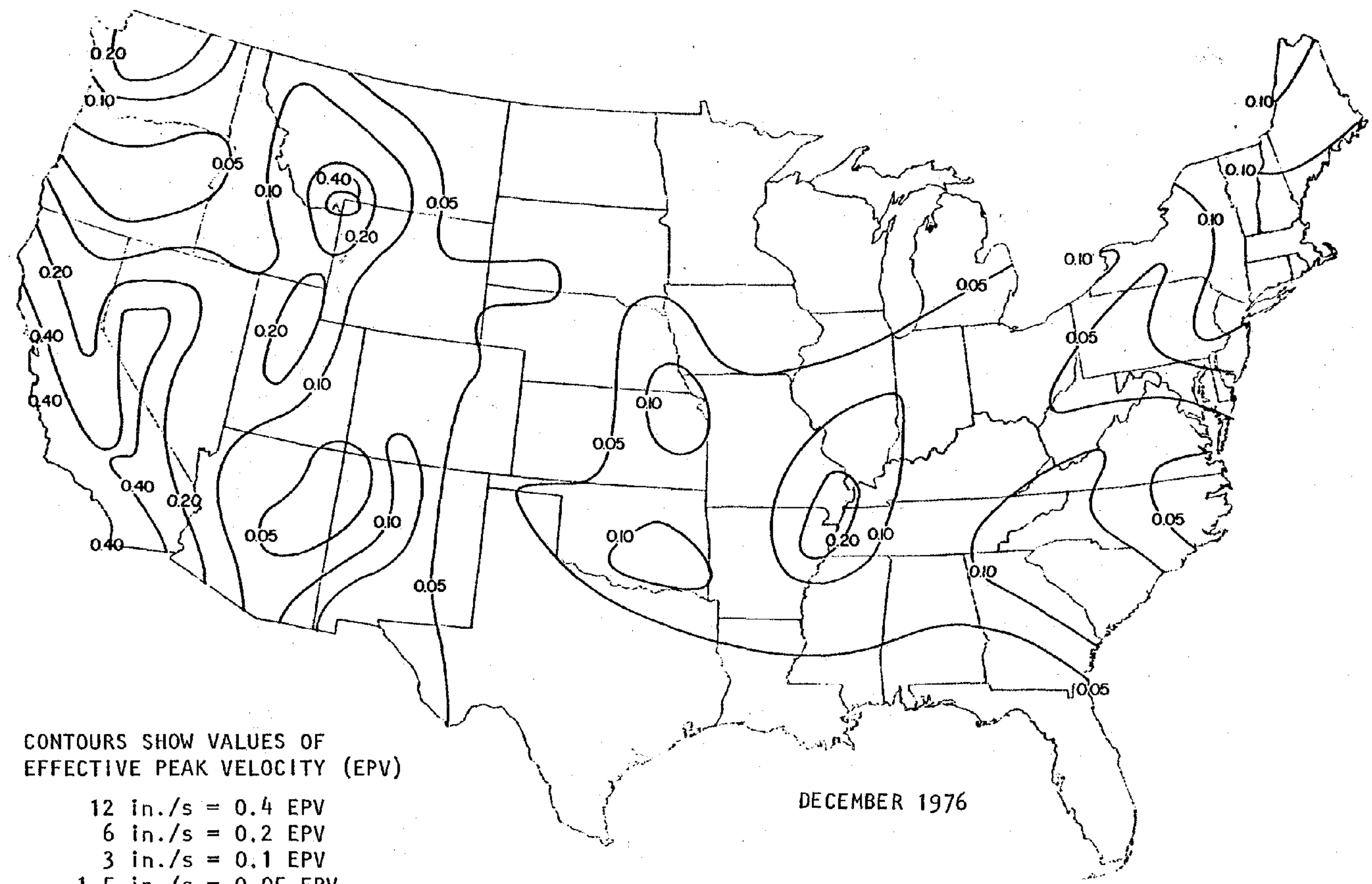


FIGURE 4-3. EFFECTIVE PEAK VELOCITY MAP (ATC, 1978)

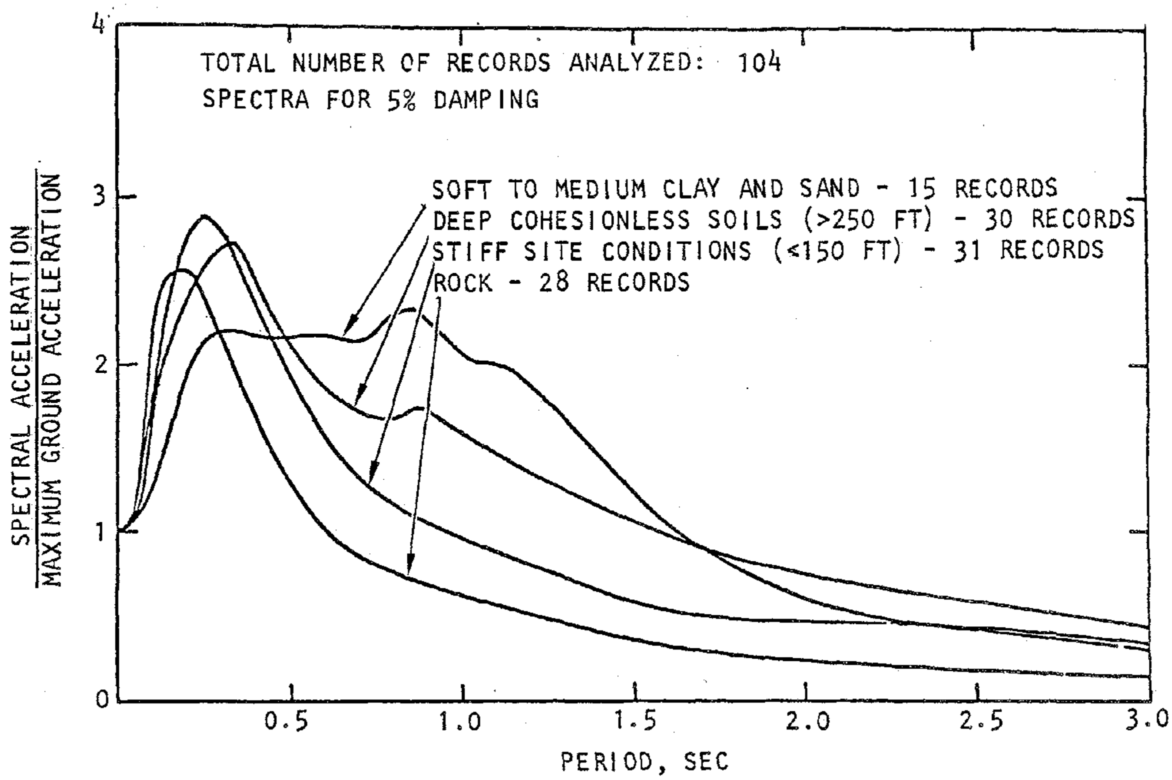


FIGURE 4-4. AVERAGE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS (ATC, 1978)

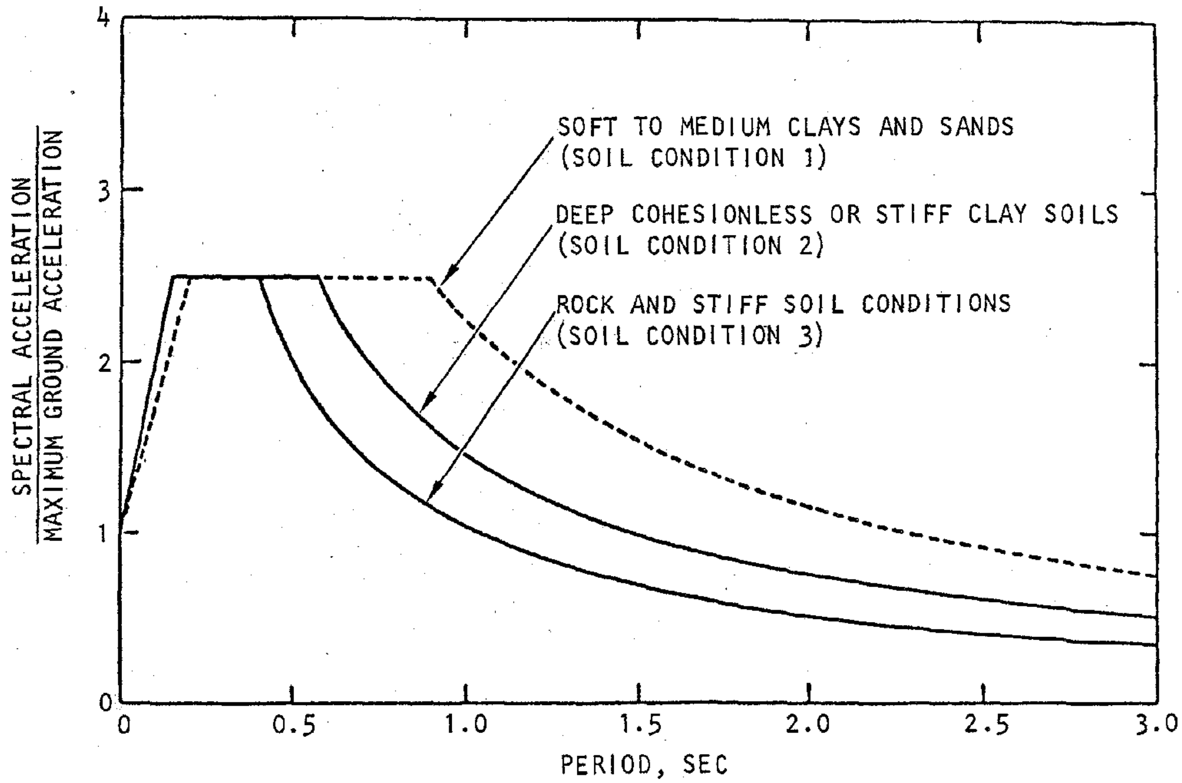


FIGURE 4-5. NORMALIZED SPECTRAL CURVES RECOMMENDED FOR USE IN BUILDING CODE (ATC, 1978)

ATC-3 represents a state-of-the-art workable tool for describing the design ground shaking as a smoothed elastic response spectrum. A set of four or more acceleration time histories, whose average elastic response spectrum is similar to the design spectrum, are recommended for this study. This approach is desirable for research studies of the seismic response of TUW buildings.

SECTION 5

ANALYTICAL METHODS

5.1 INTRODUCTION

This section provides a discussion of the general methods used for analyses of TUW structures and their range of application. Some of the methods that are currently being used as a basis for design are evaluated.

The analytical approach for analyzing TUW structures can be divided into two basic categories:

1. Component Analysis Methods
2. Assemblage Analysis Methods

The first category is currently used as the design basis for TUW structures (Table 5-1). The second category is used mostly for very special cases or for research studies to evaluate modes of response and failure of TUW structures. The two categories are discussed in the following subsections.

5.2 COMPONENT ANALYSIS METHODS

Due to the popularity of TUW construction in the United States for the past three decades, various organizations and agencies have developed analysis methods and computation guidelines for treating TUW structural components. These methods are typically static or equivalent static. The following assumptions are usually made:

- Diaphragms are rigid and the roof creates a stiff structure in which lateral sway is negligible.

- Panel behavior is idealized as a column, hinged along its loaded edges and free along vertical edges.

TABLE 5-1. DESIGN BASIS FOR TILT-UP LOAD-BEARING WALLS (Used in the United States)

Level of Accuracy of Analysis	Empirical	Approximate (Hand Calculation)			Refined (Computer)
Method	ACI 318-77	SEASC (1979) (Yellow Book) James Johnson	Wyatt (1980)	Musser (1980)	Kripanarayanan* (1979)
H/t	$\leq$ 25	$\leq$ 36	Limited by yield of steel	40 to 50	35 to 50
Slenderness Analysis	No	Yes	Yes	Yes	Yes
f_c at ε_o	0.85 f'_c	0.85 f'_c	f'_c	0.85 f'_c	0.85 f'_c
Basic Assumptions	Concentric loading Short wall	Cracked section full height Lateral load is high compared to other loads Rectangular stress block	Cracked section full height Lateral load is high compared to other loads Triangular stress block	Cracked section full height Lateral load is high compared to other loads Parabolic deflected shape	Variable EI Tensile strength of concrete = 0 Parabolic stress block
Comments	Satisfactory only for short columns	Compares well with Kripanarayanan Higher than Kripanarayanan Underestimates P-Δ effects	Compares well with Kripanarayanan Higher than Kripanarayanan Lower than SEASC Underestimates P-Δ effects	Slightly conservative Compares well with Kripanarayanan	Interpolation and extrapolation is not convenient Lateral load is high compared to other loads f_c is conservative
Compression Buckling		Not likely	Not likely	Not likely	Not likely

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*An alternative refined method has been developed by Weiler and Nathan (1980) in Canada.

- Design for lifting or tilting is to be considered by lifting companies.
- The reinforcement ratio is small.
- Lateral load and eccentricity should to be accounted for.

Assumptions are also made regarding the constitutive relations of concrete and steel, the distribution of internal stresses, cracked section properties, and the support conditions. Results of the analyses are obtained through the use of empirical or simplified formulas, families of curves or tables, and in some cases by simple computer programs. The results include wall deflections, critical forces, and moments. Some analysis methods give the "pick-up stresses," which correspond to the stresses in the wall during lifting of the panels. A summary of the currently available analytical methods is given in Table 5-1.

Because of the relative simplicity and convenience of these methods, this group of working formulas and tables is generally accepted as the basic design tools for TUW systems. However, the conveniences of these methods are not achieved without some shortcomings. The first major shortcoming is that in isolating the TUW panel alone, the interaction between various components of the structure during seismic loading cannot be considered accurately. The assumption of a rigid roof diaphragm with negligible lateral sway would be in error for certain types of roof diaphragms. Studies of plywood roof diaphragms by ABK (1980) indicate that roof diaphragms amplify certain strong earthquake ground motions by factors as high as 2. Therefore, wall-to-roof connections may be underdesigned when the design is based on current simplified methods. It is also known that the dynamic response of a structure may vary considerably depending on the dynamic characteristics of the earthquake signal and the structure itself. The equivalent static forces applied by the structural assemblage to the individual panels, as calculated by the simplified methods, may be grossly in error. The second shortcoming is that the resulting design may be overly conservative or overly stiff due to assumptions that have led to successive application of conservative factors, such as a fully cracked section over the total height of the panel. Such stiffer elements in a structure may not be desirable in achieving a balanced seismic design (Fintel and Ghosh, 1981). The third major shortcoming is that this group of methods basically solves linear problems. Nonlinear material properties such as hysteretic stress/strain relations in tension and compression are not incorporated. Initiation of cracking, propagation of cracking, and gradual deterioration of panel stiffness in relation to changes in applied loads are not modeled. Furthermore, geometrical irregularities in the building shape or wall openings are only approximately accounted for. In spite of these shortcomings, the methods listed in Table 5-1 and other similar methods not included in this table constitute a group of useful tools in the design practice of TUW structures.

5.3 ASSEMBLAGE ANALYSIS METHODS

A number of general purpose computer codes for assemblage analysis have been developed by the engineering community that can be used to analyze tilt-up wall structures. In contrast to the hand calculation methods, these codes occupy the other end of the spectrum in terms of sophistication and generality. Most of these codes can perform either static or dynamic analyses, accept a large variety of linear and nonlinear materials, and use different element types to represent different components in the structure. A list of some of the representative codes is given in Table 5-2.

TABLE 5-2. NONLINEAR FINITE ELEMENT ASSEMBLAGE CODES

Items for Comparison	ANSYS (DeSalvo and Swanson, 1979)	TRANAL (Baylor, Bieniek, and Wright, 1974)	MARC (Marc, 1979)	NASTRAN (McCormick, 1979)	ADINA (Bathe, 1978)
Static Analyses	Linear, thermal, plastic, buckling, creep Nonlinear material properties and geometry	None	Linear, thermal Nonlinear material properties and geometry	Linear, buckling, thermal Nonlinear material properties and geometry	Linear, thermal Nonlinear material properties and geometry
Eigenvalues and Eigenvectors	Yes	No	Yes	Yes (3 methods, restartable, complex roots)	Yes
Dynamic Analysis	Modal (linear) Nonlinear Transient Implicit Harmonic response	Nonlinear Step-by-step Explicit	Linear and nonlinear Step-by-step Implicit Explicit Modal	Linear and nonlinear Step-by-step Implicit	Linear and Nonlinear Step-by-step Implicit Explicit
Element Types					
Truss	Yes	No	Yes	Yes	Yes
Beam	Yes	No	Yes	Yes	Yes
Plate and Flat Shell	Yes	No	Yes	Yes	Yes
Curved Shell	Yes	No	Yes	Yes	Yes
Solid	Yes	Yes	Yes	Yes	Yes
Two-Dimensional	Yes	No	Yes	Yes	Yes
Special	Pipe and fluid elements	No	Concrete pipe and fluid elements	Substructures Pipe and fluid elements	Fluid elements
Loading					
Nodal Point	Yes	Yes	Yes	Yes	Yes
Member	Yes	Yes	Yes	Yes	Yes
Gravity	Yes	Yes	Yes	Yes	Yes
Initial Stress/Strain	Yes	No	Yes	Yes	Yes
Kinematic Boundary Conditions for Dynamic Analysis	Displacement, velocity, and acceleration	Displacement	Displacement	Displacement, velocity, and acceleration Energy-absorbing boundary conditions	Displacement
Maximum Number of Node Points	Dynamically allocated (D.A.)	D.A.	D.A.	D.A.	D.A.
Maximum Number of Elements	D.A.	D.A.	D.A.	D.A.	D.A.
Maximum Half-Bandwidth	Wavefront technique	D.A.	D.A.	D.A. (active column technique)	D.A.
Maximum Number of Load Cases	D.A.	D.A.	D.A.	D.A.	D.A.
Maximum Number of Materials	D.A.	D.A.	D.A.	D.A.	D.A.
Maximum Number of Cross Sections	D.A.	D.A.	D.A.	D.A.	D.A.
Graphic Output					
Grid or Mesh Plot	Yes	Yes	Yes	Yes	No
Mode Shapes	Yes	No	Yes	Yes	No
Time History	Yes	Yes	Yes	Yes	No
Response Spectra	Yes	No	Yes	Yes	No
Contour Plot	Yes	No	Yes	Yes	No
Automatic Mesh Generation	Yes	Yes	Yes	Yes	Yes
Bandwidth Minimization	Wavefront technique	Not applicable	Yes	Yes	No
Constrained DOF (slaving)	Yes	No	Yes	Yes	Yes
Special Features	Early conversion of cartesian to cylindrical or polar coordinates Large deflections Interactive mode of computation Extensive output graphics	Bonding and debonding capability Subcycle integration capability	Extensive output graphics	Sparse matrix methods Extensive output graphics Executive control	

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Generally, three different analysis methods are used in these codes:

- Response spectrum method
- Modal superposition method
- Time step integration method

Each of these methods can be utilized with one-, two-, or three-dimensional models. All three methods can be used for linear elastic models; however, nonlinearities can be treated only in an approximate way with the first two methods. Therefore, the third method must be employed when nonlinear effects are significant.

In the response spectrum method, the seismic input is defined in the form of a response spectrum. This input spectrum specifies the peak responses to a specific seismic excitation of single-degree-of-freedom (SDOF) linear oscillators with various values of fundamental frequencies and equivalent viscous damping. The values represent the peak relative displacements, pseudorelative velocities, and absolute pseudoaccelerations of the SDOF system. The time corresponding to the occurrence of these peak responses is not included in the response spectrum analysis. The frequency-dependent peak structural responses can be determined corresponding to each of the selected number of normal modes of the structure. The overall structural response is obtained by combining the contributions from all the modes considered. Since the peak modal responses do not necessarily occur at the same instant of time and the response spectrum does not provide information on phase relationships, the overall structural response is estimated by combining the peak model responses in a probabilistic manner. Several procedures currently used are:

- Square root of the sum of the squares (SRSS)
- Peak (or peaks) plus the SRSS of the rest
- Absolute sum

The procedure selected will depend on the modes obtained. Equivalent viscous damping can be included to simulate energy dissipation and account for nonlinearities in an approximate way.

The modal time-history analysis method uses a time-history input rather than a response spectrum. Based on a limited number of normal modes of the structure, the structural response time history in each mode is obtained by direct integration. The response time history of the complete structure is determined directly by combining the contributions from each mode. As in the response spectrum method, equivalent viscous damping can be included to simulate energy dissipation and account for nonlinearities in an approximate way.

The direct integration time-history method is the most general method for the seismic analysis of structures. It provides the time-dependent response to a time-history input. In this scheme, the numerical integrations are carried out directly on the coupled set of simultaneous differential equations of motion in the structural system's physical coordinates. This method allows for the inclusion of nonlinearities that can be very important for tilt-up wall structures.

The main advantage for using these assemblage codes is that the structure can be represented as a whole. In some cases the ground can also be represented in the model. Therefore, the dynamic response obtained can be much more realistic. The most severe shortcoming is their cost. Depending on the method of solution, to model and perform a dynamic analysis of a typical industrial-type building can be a sizable undertaking. For this reason this analysis approach is not commonly used except in the class of structures such as hospitals, nuclear power plants, and the like.

In addition to the codes listed in Table 5-2 there exist other computer codes that are developed specifically to handle panelized structures (Becker and Llorente, 1977). They are more research oriented codes and not generally available to the public.

The STARS code developed by Agbabian Associates is a nonlinear code used for the dynamic analysis of structural systems. This code has been used extensively in the ABK program for the dynamic analyses of three-dimensional structures (Adham and Ewing, 1978; Ewing et al., 1980). In contrast to the nonlinear finite element codes, STARS is relatively simple and economic for the analysis of one- and two-story TUW buildings, and was selected for the preliminary analysis of TUW structures conducted in this study. A brief summary of this code is given in the following subsection.

5.4 GENERAL DESCRIPTION OF STARS COMPUTER CODE

The STARS/III computer code selected for use in the present series of calculations computes the dynamic, nonlinear response of discrete mass multidegree-of-freedom systems. These systems are idealized in the code by an assemblage of different types of one-dimensional elements; they include elastic beam elements and various types of linear and nonlinear spring elements with specified loading, unloading, reloading, hysteretic, and damping characteristics (a typical element is shown in Figure 5-1).

The code uses a time marching scheme to compute the dynamic response of a discrete mass system. First the system of equations of motion is formed and the coupled response is obtained at a specific instant of time in the following manner. At the

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*STARS is a lumped parameter computer program developed by Agbabian Associates for the dynamic analysis of nonlinear structural systems (*User's Guide for STARS Code*, R-6823-999, Agbabian-Jacobsen Associates, Los Angeles, 1969).

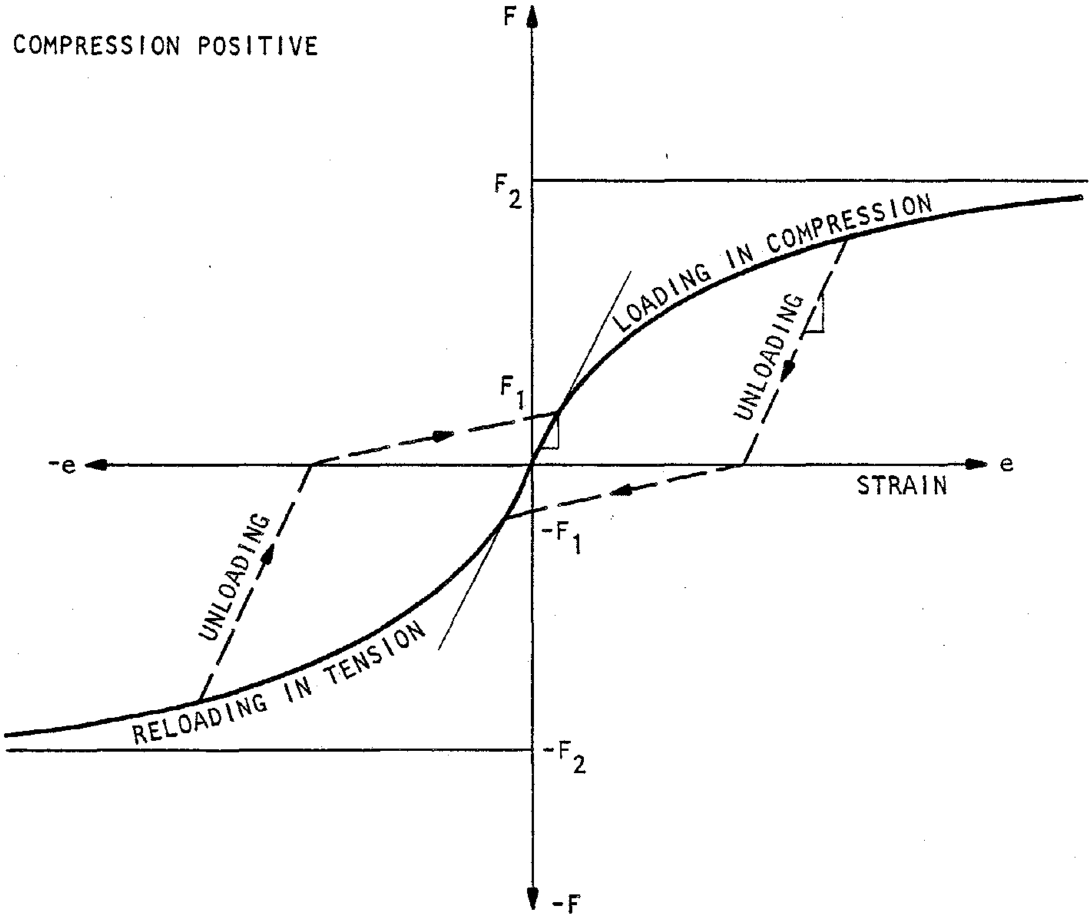


FIGURE 5-1. EXAMPLE OF NONLINEAR INELASTIC, HYSTERETIC, TENSION AND COMPRESSION SPRING USED IN *STARS* CODE

beginning of a time step the motions are held fixed and the internal forces are computed. Based on the response motion and internal forces at earlier times, the responses at the end of the time step are predicted. With these predicted values of the motion, the internal forces are recalculated. Using these forces, a "corrected" response motion at the end of the time step can then be determined. Therefore, at each time step, the relationship between the response motion and the internal forces is satisfied. This process is continued step-by-step throughout the time interval of interest. The numerical method used for this step-by-step solution involves two types of integration formulas. The fourth-order Adams-Bashforth "2/3" predictor-corrector method, which is a stable solution method, is used for the majority of the solution. However, this method is not self-starting and the fourth-order Runge-Kutta method is used to start the solution process. A detailed description of the STARS code together with example problems are given in the user's guide.

SECTION 6

EXAMPLE ANALYSIS OF A TILT-UP-WALL STRUCTURE SUBJECTED TO EARTHQUAKE EXCITATION

This section presents results obtained by using the STARS computer program for the analysis of a typical TUW structure in Seismic Zone 4 (UBC, 1979) or Area 7 (ATC, 1978).

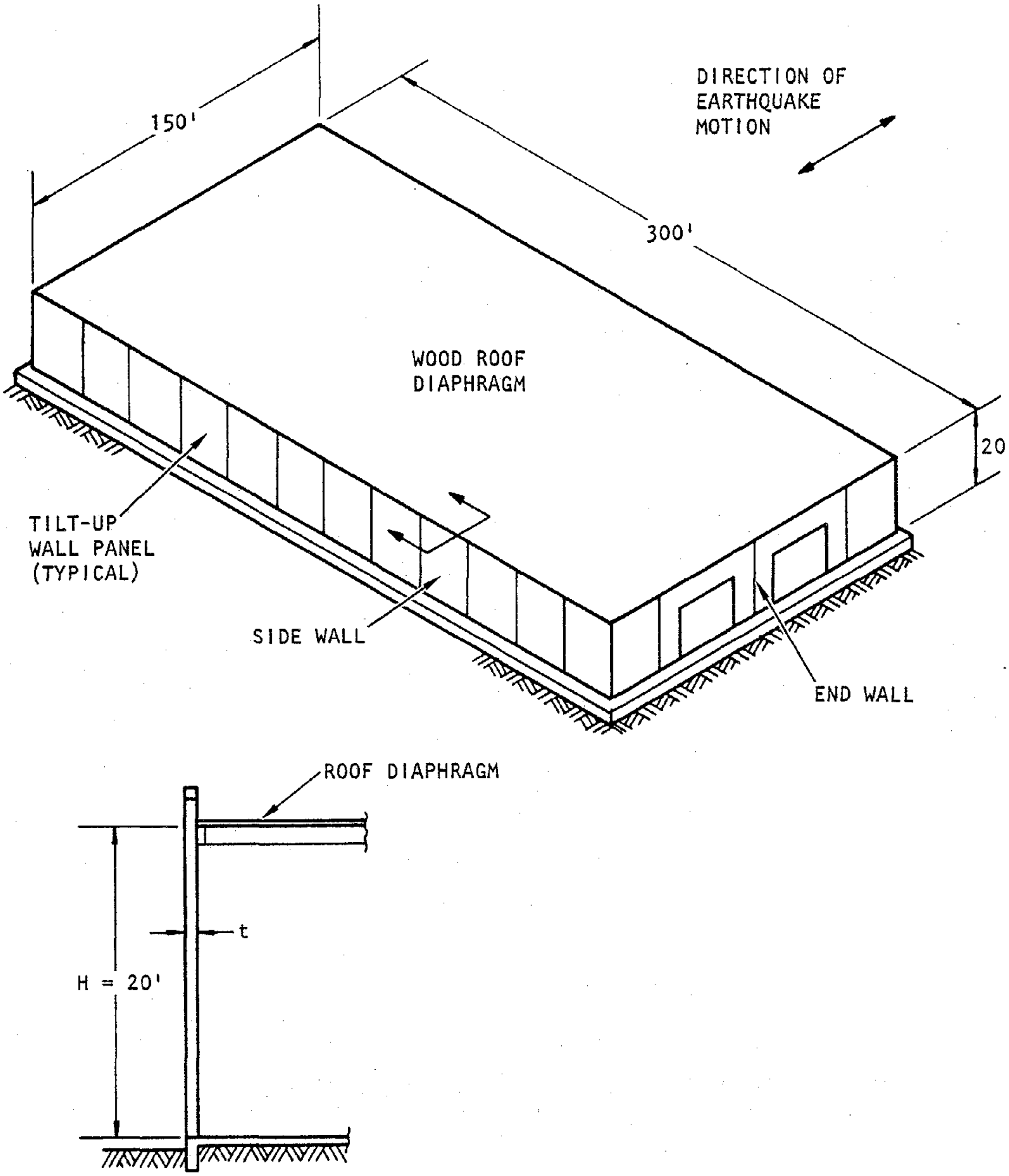
6.1 DESCRIPTION OF MODEL

A typical one-story warehouse type building was selected for analyses. The building consists of a wood diaphragm roof 300 ft by 150 ft supported on four sides by concrete tilt-up walls 20 ft high. Earthquake motion is assumed to be in a plane perpendicular to the long dimension of the structure (Fig. 6-1).

The roof diaphragm is modeled as a deep shear beam. This beam is divided into eight equal segments. Because of the assumed symmetry configuration in both geometry and loading, only half of the building about the centerline is considered in the model (Fig. 6-2).

For the current phase of the analysis, the end wall is assumed rigid. Since the earthquake motion for the aspect ratio of the end wall considered in this example will be transmitted from the foundation level (Level D) to the top of the end shear wall (Level B) with very little modification (Adham and Ewing, 1978), the earthquake motion is therefore prescribed directly at the end of the roof diaphragm and at the bottom of the TUWs.

For the critical orientation of the earthquake motion, the side TUWs will undergo primarily bending deformation while the diaphragm will undergo primarily shear deformation. Thus the analytic model uses linear beam elements to represent the side TUWs and nonlinear shear spring and damper elements to represent the diaphragm. A schematic of such a model is shown in Figure 6-3.



CROSS SECTION

FIGURE 6-1. ONE-STORY TILT-UP-WALL BUILDING WITH A WOOD DIAPHRAGM ROOF

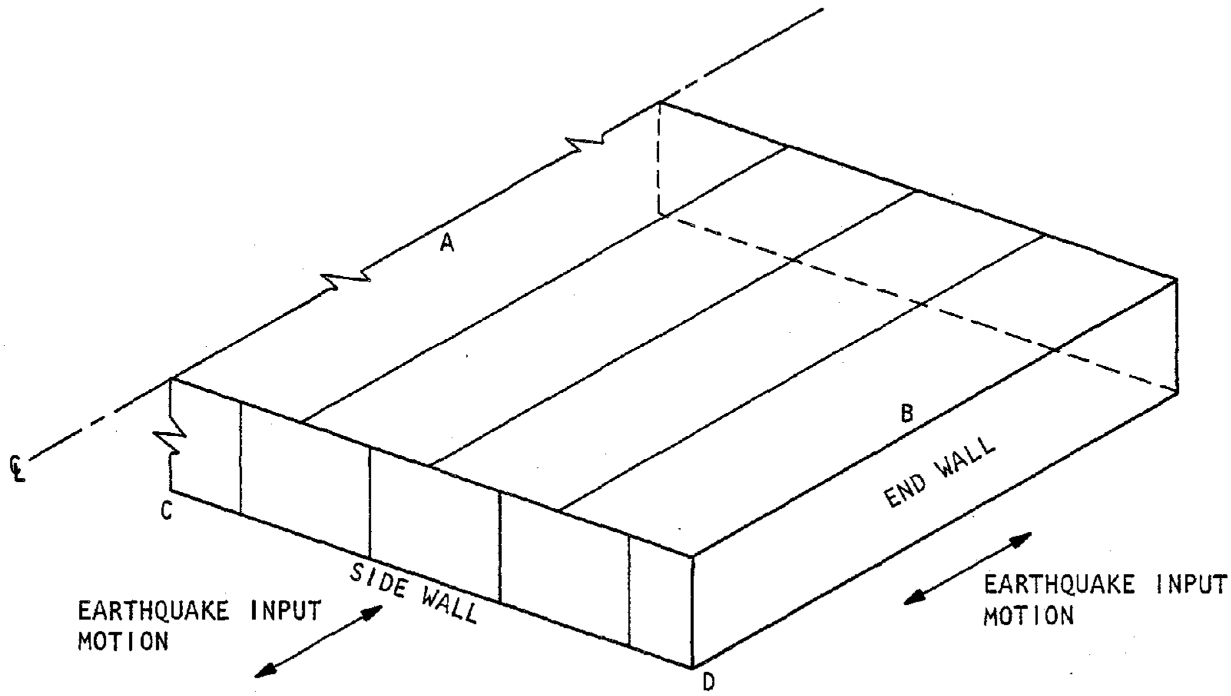


FIGURE 6-2. FOUR SEGMENT HALF MODEL

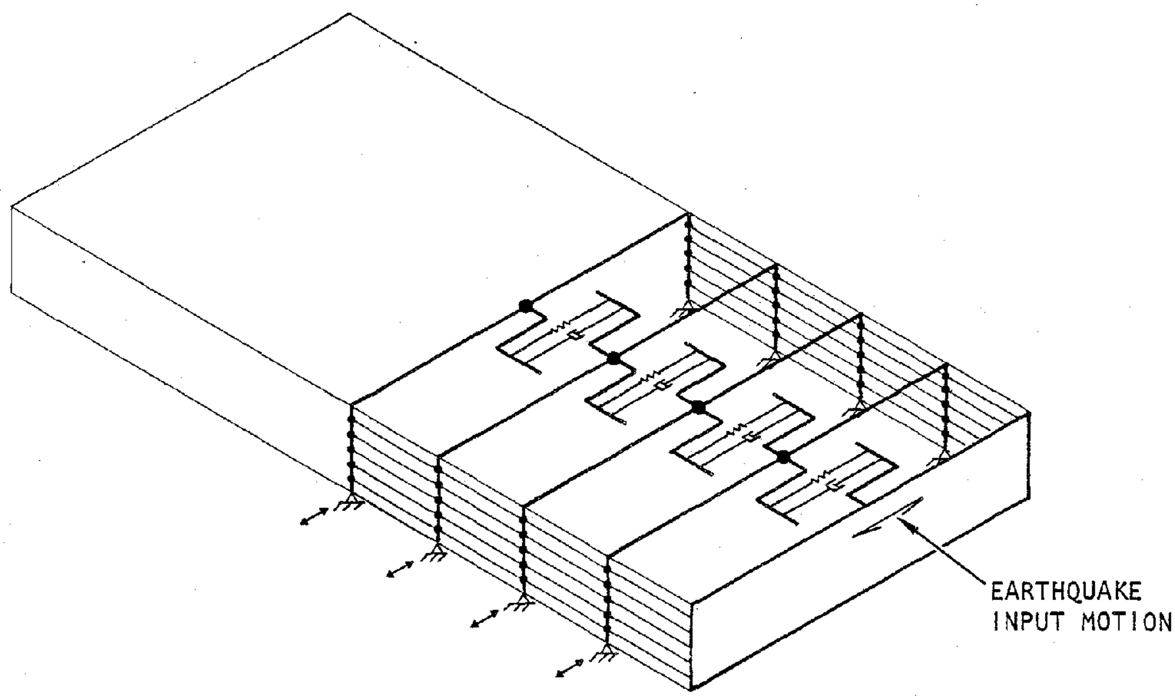


FIGURE 6-3. BEAMS SPRINGS AND DASHPOTS FOR HALF MODEL

A further simplification is made by assuming the response of the two side walls to be identical. The corresponding upstream and the downstream tilt-up side wall sections can be lumped into one single beam. Since the most critical TUW section is probably in the midsection of the long span, the beam located at the mid-section is represented by six short elements. This division will give better definition of the moments and displacements developed along the section. To simplify this calculation, the remaining three beams representing the rest of the TUW side uses only one element each to represent them. The roof diaphragm sections are represented by nonlinear spring and damper elements. A schematic of this model is depicted in Figure 6-4.

6.2 MATERIAL MODELS

Two types of elements are used in the modeling of the TUW structure. The linear elastic uniform beam element is used to represent the tilt-up side walls (Fig. 6-4). The geometrical properties of these beam elements are prescribed by the length of the beam, shear area, and principal moment of inertia associated with bending. The material properties are prescribed by Young's modulus, shear modulus, and unit weight of reinforced concrete. These properties are specified in the STARS code through the values of bending stiffness and shear stiffness. The values of the various parameters used to compute the two stiffnesses are given in Table 6-1. A 5% damping ratio is used in this analysis.

TABLE 6-1. LINEAR BEAM ELEMENT PROPERTIES

Item	Density, lb/ft^3	Young's Modulus E, psi	Shear Modulus G, psi	Thickness, in.	Width, ft	Shape Factor, K
Value	145	3.122×10^6	1.3×10^6	5.5	37.5	0.85

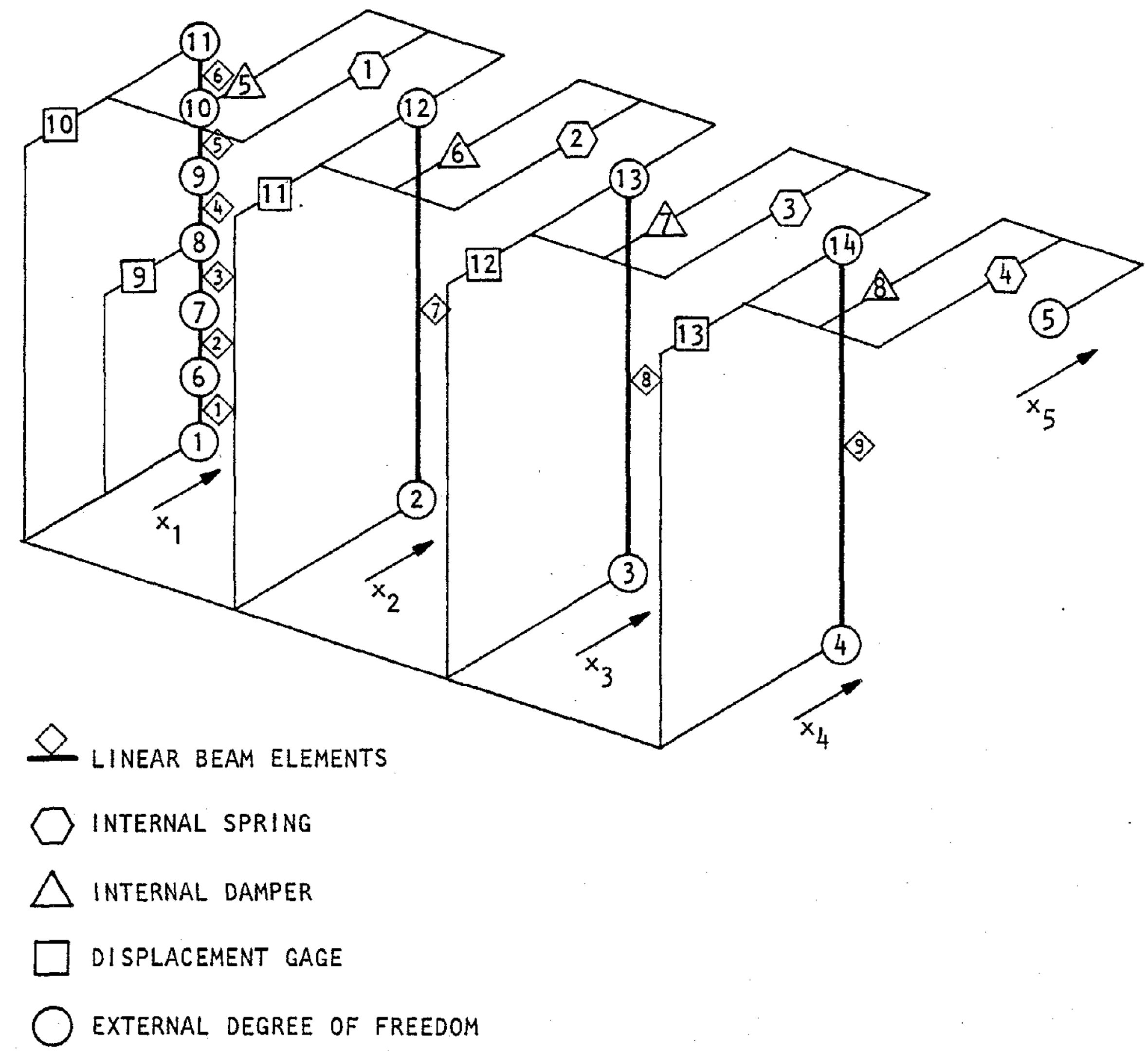


FIGURE 6-4. LUMPED PARAMETER MODEL FOR HALF MODEL

The roof diaphragm segments are represented by nonlinear, inelastic, hysteretic shear springs and dampers. The force-deformation relationship of these spring elements are modeled based on cyclic in-plane static loading tests on 20-ft x 60-ft plywood diaphragms as shown in Figure 6-5a and b (Ewing et al., 1980). In this model the total deformation mechanism of the diaphragm between the two points of loading is smeared into a nonlinear, hysteretic shear spring element (Fig. 6-5c). A typical cyclic load path is shown in Figure 6-6a, and the overall load-deflection envelope in Figure 6-6b. A mathematical model corresponding to a second-order curve is selected to represent this envelope. This model takes the form

$$F(e) = \begin{cases} \dfrac{F_u e}{\dfrac{F_u}{K_i} + e} & \text{for } e > 0 \text{ (compression)} \\[2ex] \dfrac{F_u e}{\dfrac{F_u}{K_i} - e} & \text{for } e < 0 \text{ (tension)} \end{cases}$$

where

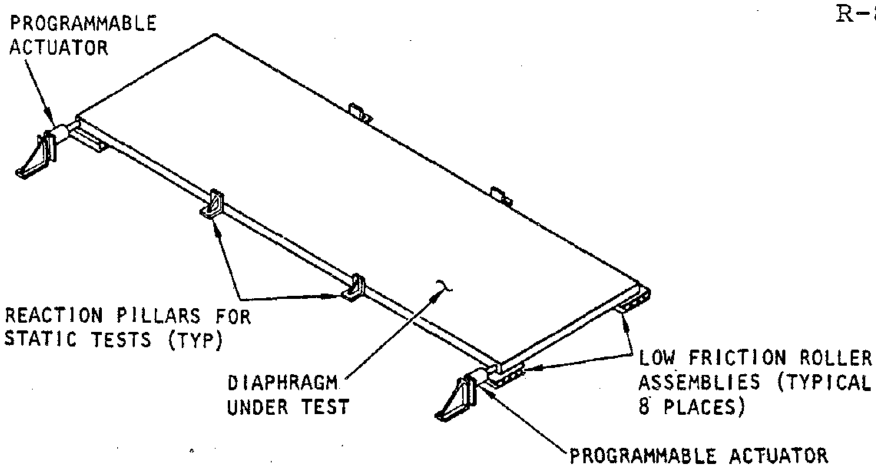
$F(e)$ = Spring force

e = Spring deformation in terms of relative displacement

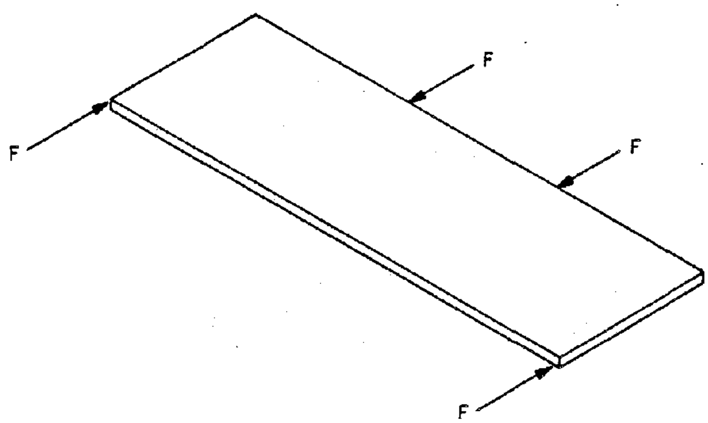
F_u = Ultimate spring force capacity of spring for large e

K_1 = Initial spring modulus

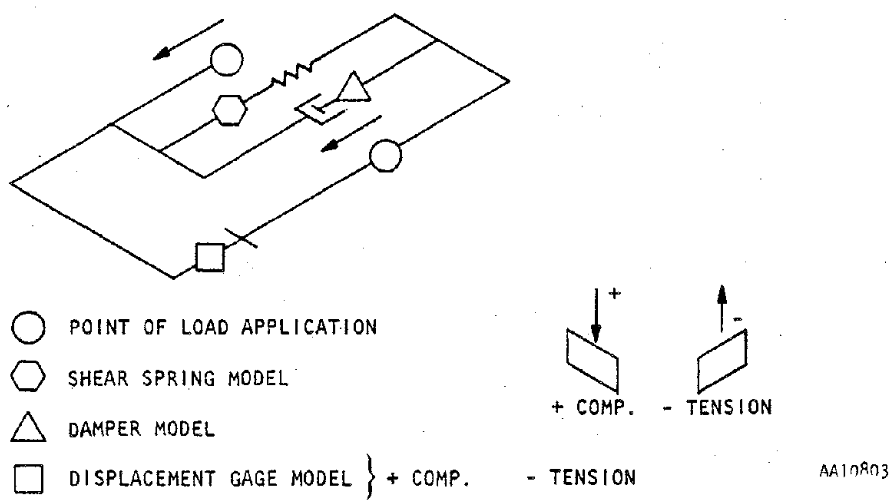
The unloading and reloading portions of the force-deformation curve are idealized by piecewise linear segments (Fig. 6-6c). The values of the constants defined above are based on test data shown in Figure 6-7. Because of the difference in dimensions between



(a) Test setup for quasi-static testing of diaphragms

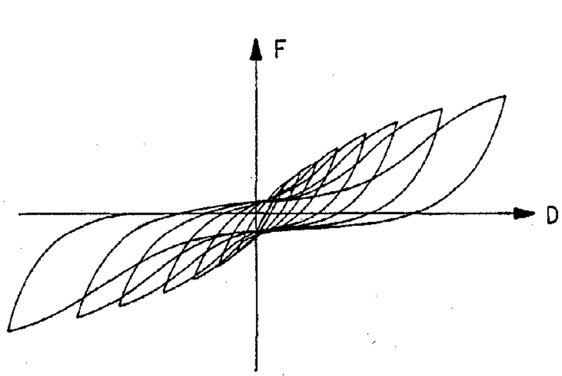


(b) Idealized loading configuration

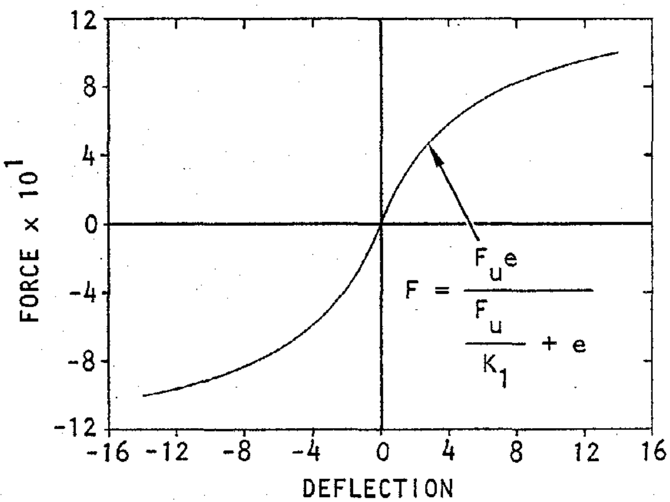


(c) Nonlinear spring and dashpot model representing diaphragm

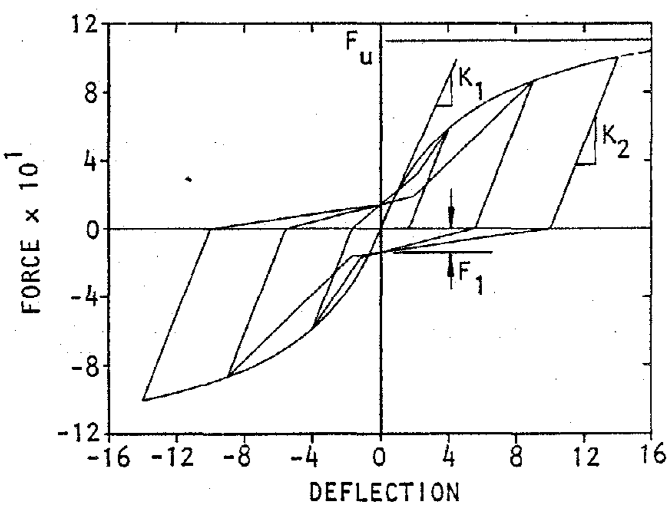
FIGURE 6-5. TEST CONFIGURATION FOR OBTAINING PROPERTIES OF SHEAR DIAPHRAGMS



(a) Typical cyclic load deflection diagram for plywood diaphragms



(b) Force-deflection envelope of model



(c) Typical cyclic load-deflection diagram for model

FIGURE 6-6. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS (Ewing et al., 1980)

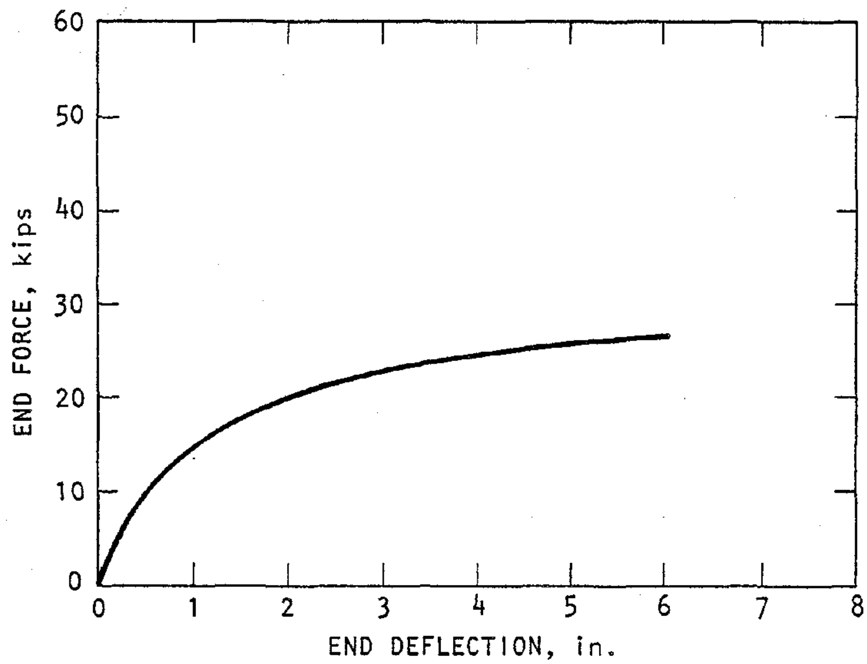


FIGURE 6-7. FORCE DEFLECTION ENVELOPE FOR A 20' x 60' WOOD DIAPHRAGMS (Ewing et al., 1980)

those of the model and of the test specimen, a scaling law (Fig. 6-8) has been used to calculate the values of diaphragm properties for the model. These values are shown in Table 6-2.

TABLE 6-2. PARAMETERS FOR THE ROOF DIAPHRAGM MODEL

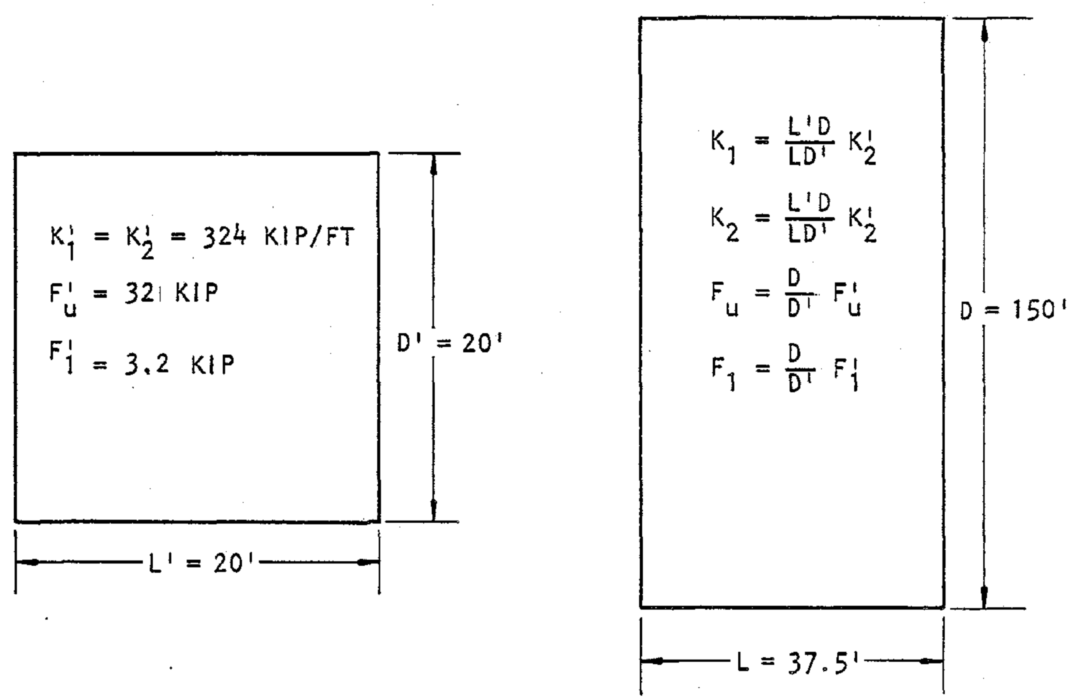
Item	Unit Weight, lb/ft^2	K_1, kip/ft	K_2, kip/ft	F_1, kip	F_u, kip
Values	20	1296	1296	24	240

6.3 INPUT GROUND MOTIONS

The N69W component of the 1971 Castaic acceleration record was selected as the basis for the input ground motion to the TUW building analysis. The reason for selecting this record is because of its high-frequency content and the relatively early arrival of the peak acceleration (-104.5 in./sec^2 at 1.9 sec); moreover, it represents a typical, strong nearby event for the California Pacific Coast region. Significant structural response can be obtained using the early portions of this record and result in relatively inexpensive calculations.

For the TUW analysis, the critical orientation of earthquake input motion is shown in Figure 6-1. In this configuration, the end TUWs are oriented parallel to the plane of particle motion, while the side TUWs are perpendicular to the plane. The most critical response is expected to occur in the midsection of the structure.

The intensity of ground shaking used in this study represents the level of shaking expected in a highly seismic area such as Los Angeles. Effective peak acceleration of such an area is 0.40 g (ATC, 1978). The acceleration input was, therefore, scaled to



(a) Test diaphragm dimensions

(b) Dimension of model of segment of diaphragm

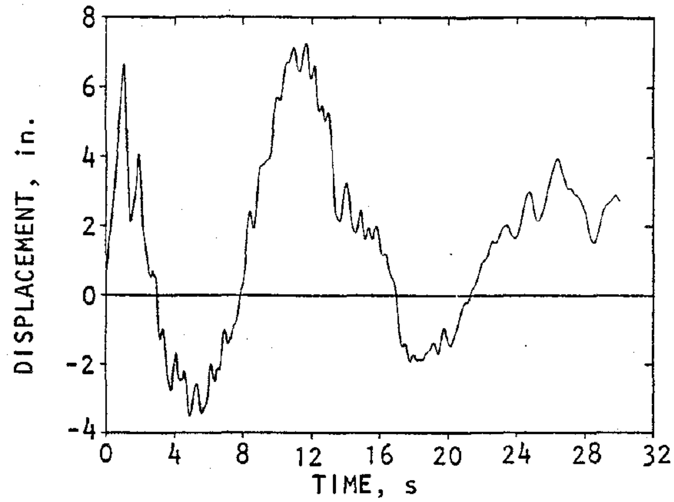
FIGURE 6-8. SCALING LAW FOR DIAPHRAGM PROPERTIES (Dimensions not to scale)

0.40 g level by multiplying by a uniform scaling factor of 1.8. The resulting scaled displacement, velocity, and acceleration records are shown in Figure 6-9. The unscaled response spectra are shown in Figure 6-10.

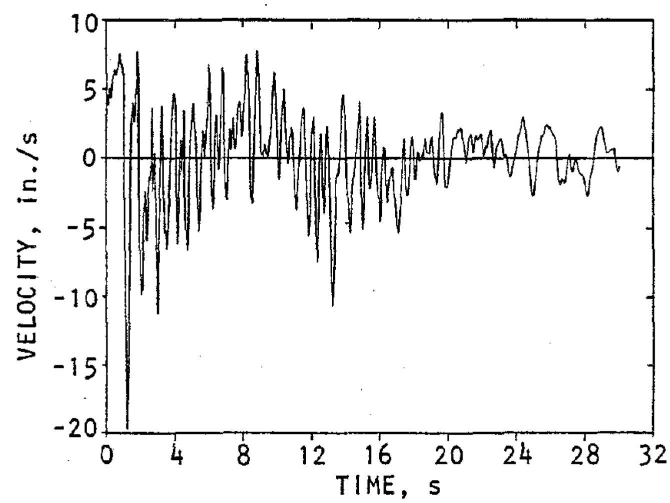
6.4 RESULTS OF ANALYSIS

Two sets of calculations were performed on the model TUW building using the STARS code. The two calculations used the same values for all input parameters except for the viscous damping constant for the nonlinear spring model representing the plywood diaphragm. In the first calculation (Case 1), a damping constant was selected which corresponded to a critical damping value of 0.07%; in the second calculation (Case 2), the critical damping value was 10%. Since in computing these damping values only the initial slope of the spring constant of the diaphragm spring model was used and the value of this constant decreases very rapidly with increasing deformation, these damping values are conservative. The two cases can therefore be considered as a lightly damped and a moderately to heavily damped TUW structure, respectively. Input to the two calculations is based on the first six seconds of the scaled 1971 Castaic record N69W component. Figure 6-9 indicates that major peaks in this acceleration record occur within the first two seconds of the time history. Therefore, this input motion should excite significant response of the structure.

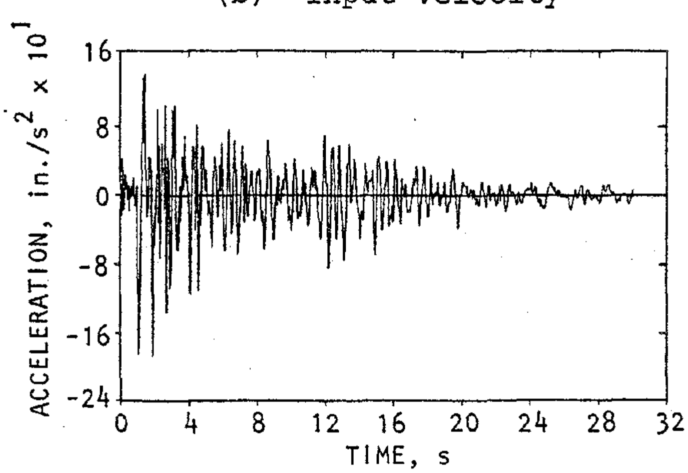
In the analyses conducted in this phase, the nonlinear model described in Section 6.2 was used as the basis for the material model for the diaphragm. The nonlinear elastic loading, unloading, and reloading segments were activated. The nonlinear hysteretic segment of the model is planned to be activated in a more detailed analyses of a proposed Phase II study. The nonlinear elastic model coupled with low and high viscous damping values provides the first



(a) Input displacement



(b) Input velocity



(c) Input acceleration

FIGURE 6-9. EARTHQUAKE INPUT MOTION, CASTAIC SCALED BY 1.80 (1 in. = 2.54 cm)

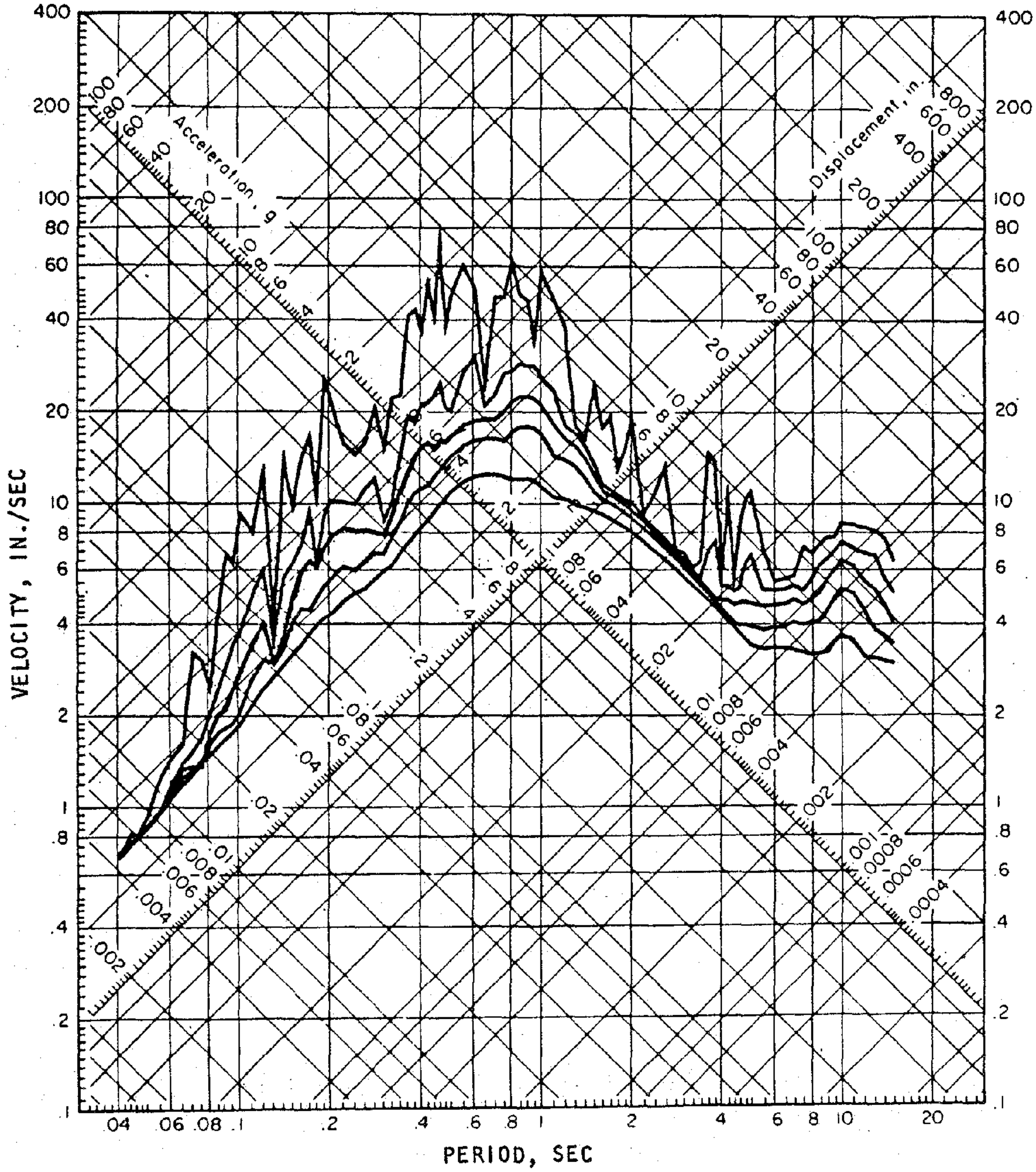


FIGURE 6-10. RESPONSE SPECTRUM OF SAN FERNANDO EARTHQUAKE, 9 FEBRUARY 1971: CASTAIC OLD RIDGE ROUTE, COMP N69W (Damping values are 0, 2, 5, 10 and 20 percent of critical--unscaled)

attempt for a better understanding of the response of a TUW building to seismic loads. Therefore, the analyses using the above nonlinear model should be regarded as preliminary and the results should be considered to provide trends. More refined versions of this model should be activated in a second phase of study in order to confirm the trends found in this preliminary phase.

The results of the two analyses cases are presented in three groups. Since the most critical response is expected to occur at the midspan, the results presented correspond to various points along this wall (Beam Nos. 1 to 6). The first group of plots depicts the acceleration and bending moment time histories at the midheight and the top of the wall (Figs. 6-11 to 6-16). The second group depicts the variations of the absolute values of the maximum bending moment and acceleration along the height of the wall (Fig. 6-17). The third group tabulates and compares the current results with those obtained using conventional static equivalent calculation methods (Table 6-3).

Figures 6-11 and 6-12 show the acceleration time histories at the midheight and the top of the wall for Case 1 (lightly damped). Figures 6-13 and 6-14 show the corresponding plots for Case 2 (10% damped). The following observations can be made from these plots:

- For the lightly damped Case 1, the amplitude of motion appears to be larger at the midheight than at the top.

- For the moderate-to-heavily damped Case 2 (damping = 10%), the corresponding average amplitude is reduced to about half of that in Case 1.

TABLE 6-3. COMPARISON OF RESULTS WITH THOSE OBTAINED BY STATICALLY EQUIVALENT METHOD* AT MIDHEIGHT OF MIDSPAN PANEL

Item / Cases	Dynamic Seismic Moment, kip-ft/ft	Ratio of Dynamic Seismic Moment / Static Seismic Moment	Total Dynamic Moment, kip-ft/ft	Ratio of Total Dynamic Moment / Total Static Moment	Ratio of Total Dynamic Moment / Design Moment Strength
Case 1 Lightly Damped	3.88	2.08	4.19	1.82	1.67
Case 2 10% Damped	1.69	0.91	2.13	0.93	0.85

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*See Wyatt (Table 5-1)

Definition of Terms:

Dynamic Seismic Moment: Maximum bending moment (absolute value) computed by STARS code

Static Seismic Moment: Portion of moment due to statically equivalent inertia load as computed by statically equivalent method

Total Static Moment: Static seismic moment plus static moments due to eccentric loading and P-δ effect

Total Dynamic Moment: Dynamic seismic moment plus static moments due to eccentric loading and P-δ effect

Design Moment Strength: Ultimate moment the TUW section can carry

- The average peak amplitudes for the midheight and the top are closer (more comparable) to each other for Case 2, than they are for Case 1. This observation implies that increased damping in the diaphragm reduces both the general absolute motion (acceleration response) and the relative motion (bending response).

- The apparent dominant frequencies at the midheight for both cases are approximately 6 cps. Since this is considerably higher than the most significant frequency (approximately 2 cps) of the input response spectrum (see Fig. 6-10), the former probably corresponds to the fundamental frequency of the TUW panel. At the top, the apparent dominant frequencies vary. In Case 2 its value is approximately 4.5 cps. In Case 1, a superposition of two frequencies is apparent. The dominant component is approximately 1 cps. A higher frequency component with much smaller amplitude is also present at approximately 6 cps. The significant change in frequency content demonstrates the damping effects of the diaphragm on the response characteristics of the TUW structures.

Figures 6-15 and 6-16 show the bending moment time histories at the midheight of the TUW. Trends shown in these plots reinforce the observations made from the study of acceleration time histories.

Figure 6-17 shows the maximum moments and accelerations developed along the height of the midspan panel. These curves represent the envelopes of the maximum acceleration and bending moment (absolute values) distributions. The range of damping values of the diaphragm has an effect of reducing (or increasing) the maximum responses by a factor of approximately 2.

Table 6-3 lists the ratio between results obtained by using the STARS code and those by using the "statically equivalent method." The P-Δ and eccentric load effects were calculated using conventional methods and added to the dynamic seismic moments obtained by the STARS code to account for the total dynamic moment on the panel. These total dynamic moments are then compared with the total static moments. The results show that Case 2 (10% damped) moments compare very closely with those of the equivalent static calculations. However, the Case 1 moments are about twice the static counterparts. Compared with the design moment strength, the Case 2 total dynamic moment is within the limit, while Case 1 exceeds the limit by about 67%.

Based on these results, the following conclusions can be drawn:

- Depending on the value of damping in the diaphragm, the acceleration at the top of the TUW exceeds that at the base by a factor of 1.4 to 2.6. Therefore, these results imply that the top connection carries a larger portion of the seismic load than the bottom support. This is contrary to the current design practice that the seismic load is carried evenly by the top connection and the bottom support of the TUW.

- The bending moment developed at the midheight may exceed the design moment strength of the TUW panel, so nonlinear beam modeling is necessary.

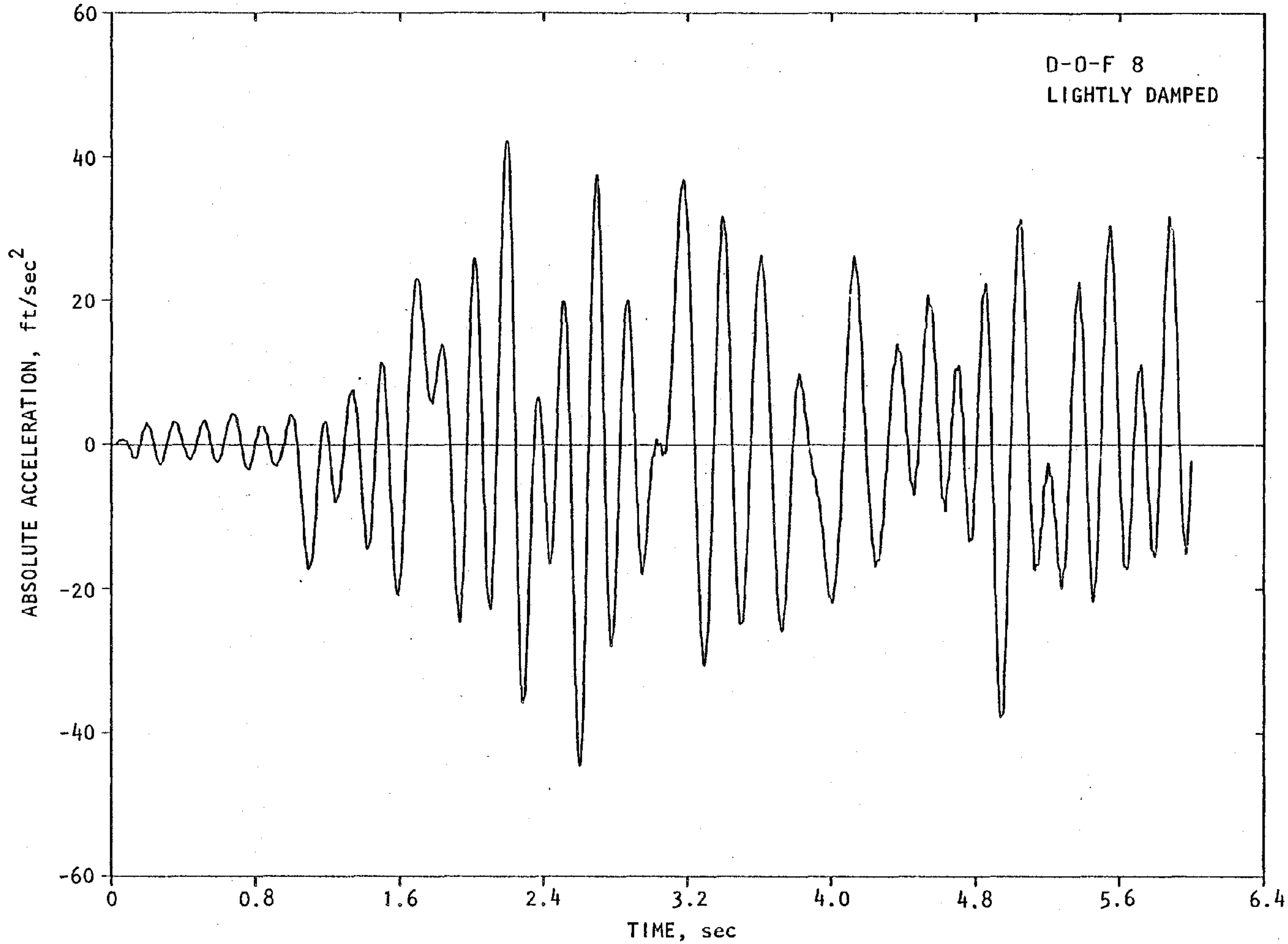


FIGURE 6-11. ACCELERATION AT MIDHEIGHT OF THE MIDSPAN PANEL CASE 1
LIGHTLY DAMPED

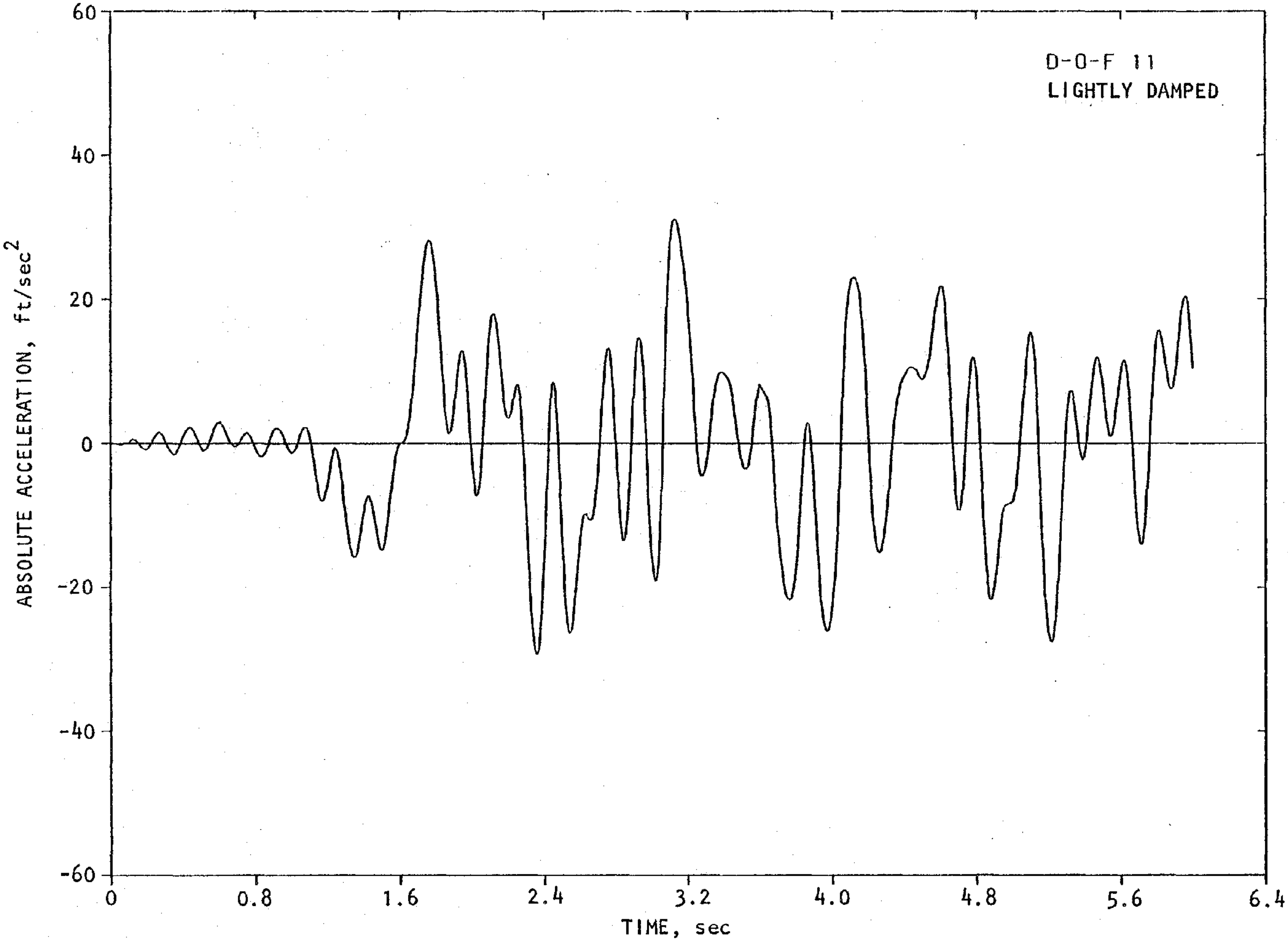


FIGURE 6-12. ACCELERATION AT TOP OF THE MIDSPAN PANEL CASE 1 — LIGHTLY DAMPED

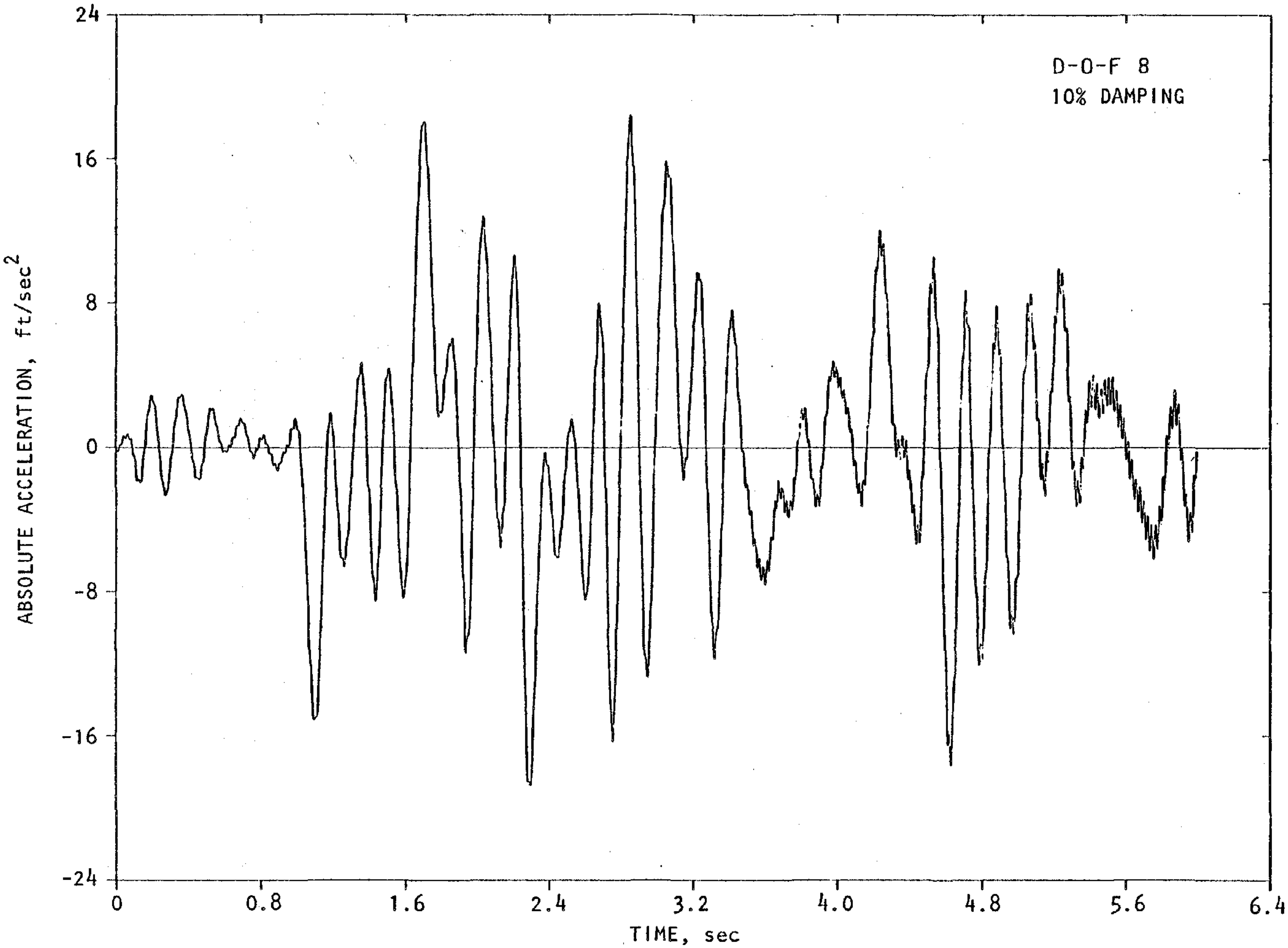


FIGURE 6-13. ACCELERATION AT MIDHEIGHT OF THE MIDSPAN PANEL CASE 2 — 10 PERCENT DAMPED

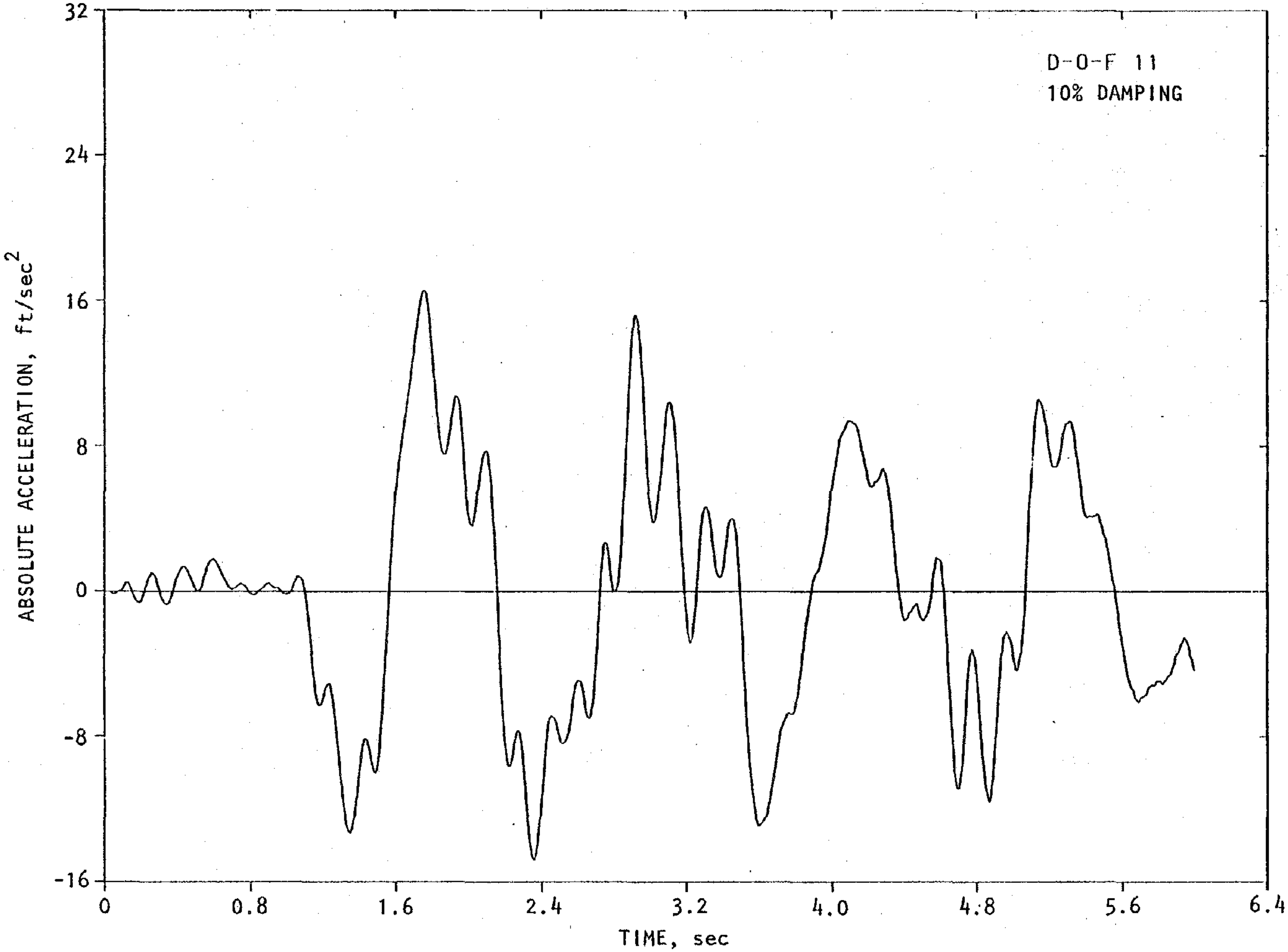


FIGURE 6-14. ACCELERATION AT TOP OF THE MIDSPAN PANEL CASE 2 — 10 PERCENT DAMPED

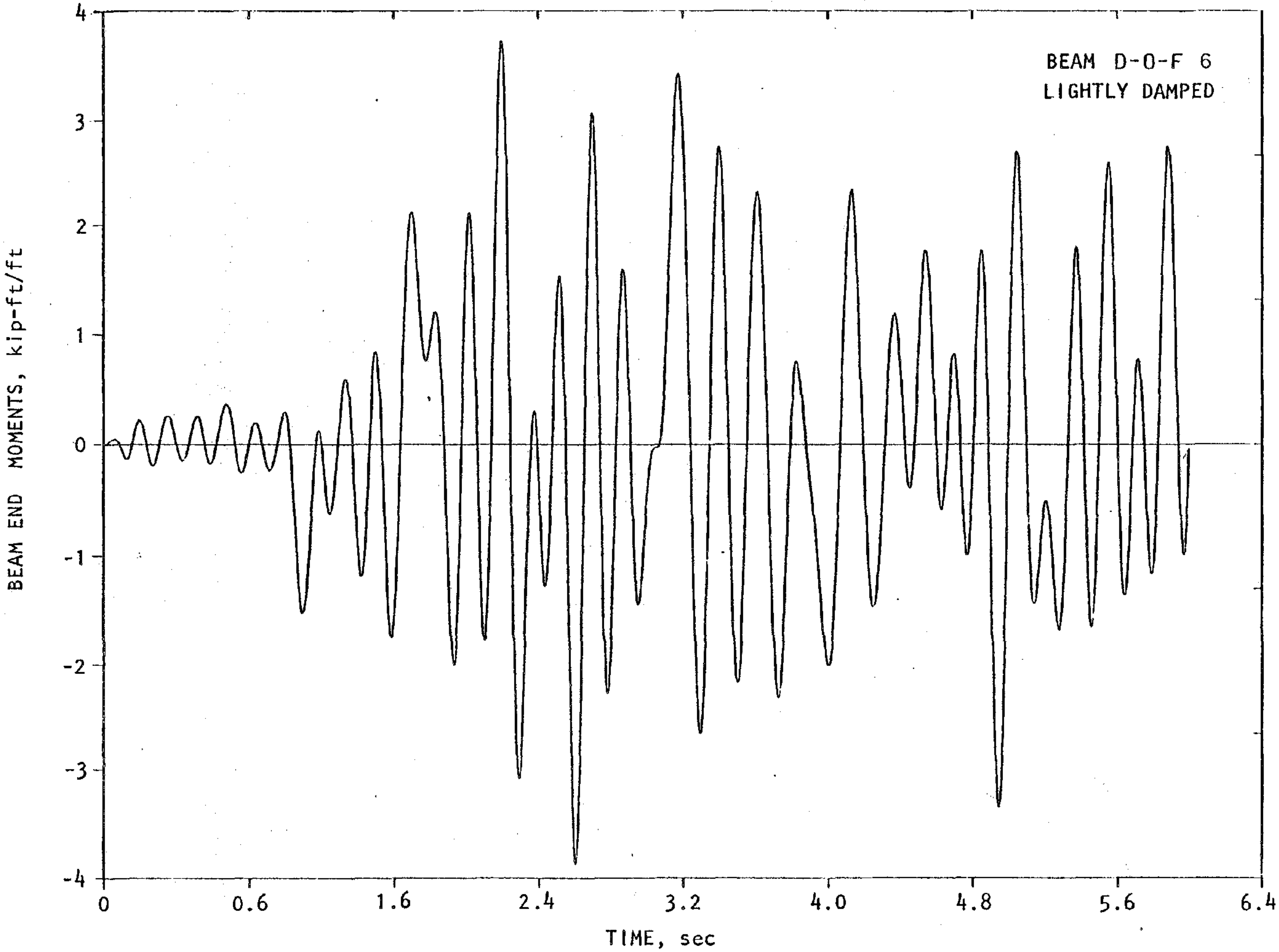


FIGURE 6-15. BENDING MOMENT AT MIDHEIGHT OF THE MIDSPAN PANEL CASE 1 — LIGHTLY DAMPED

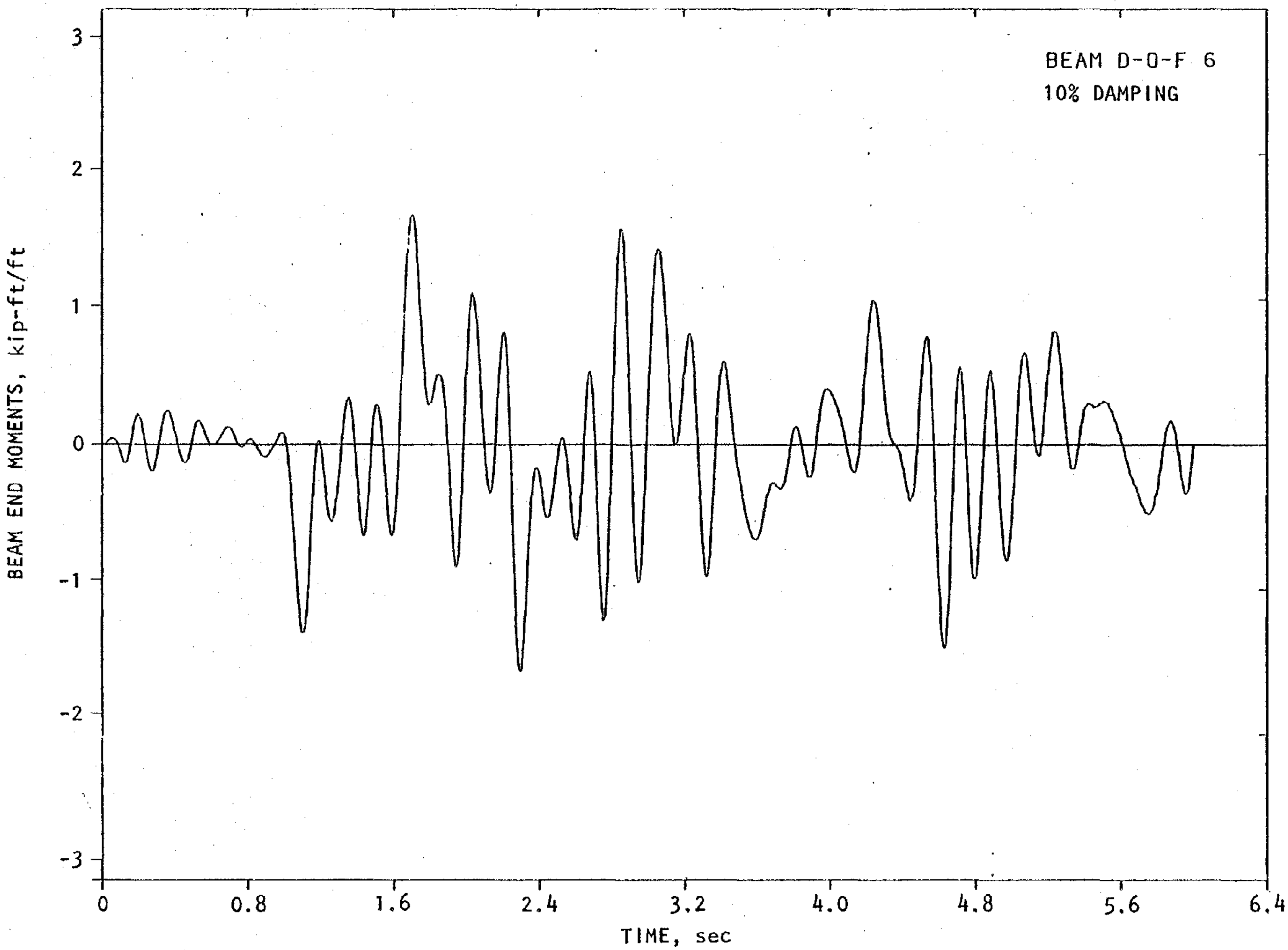
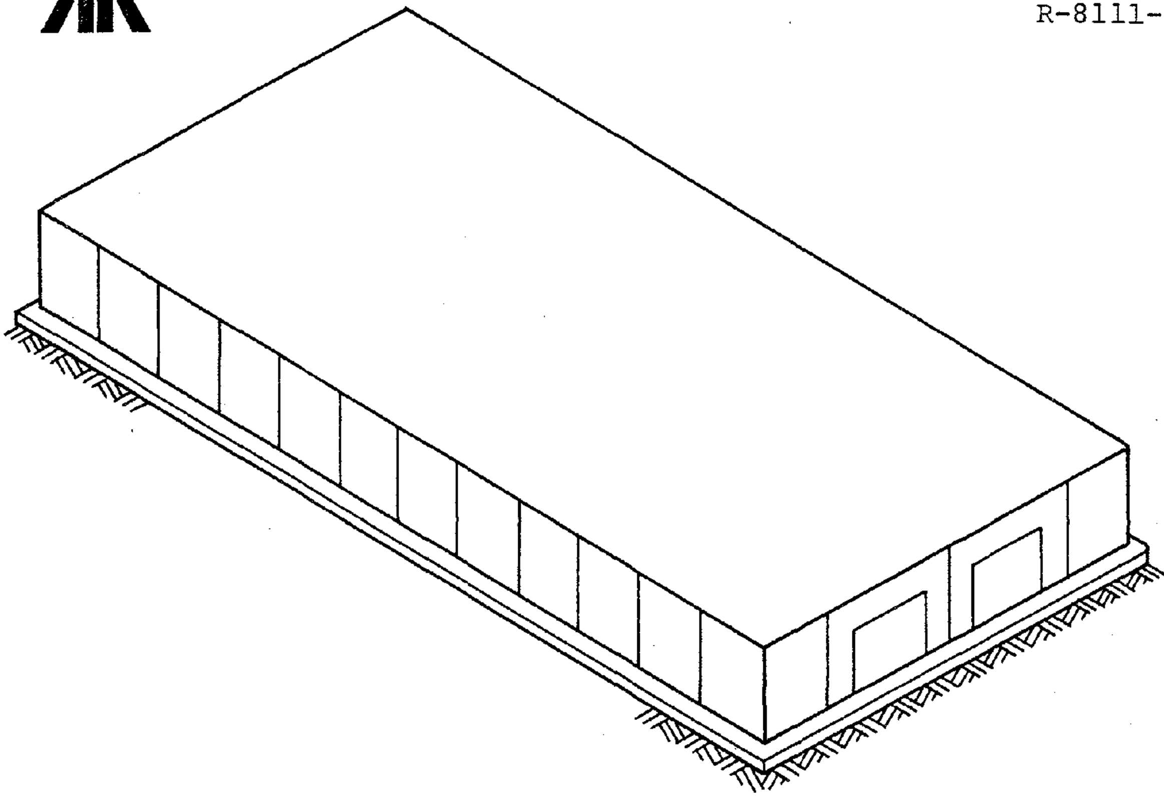
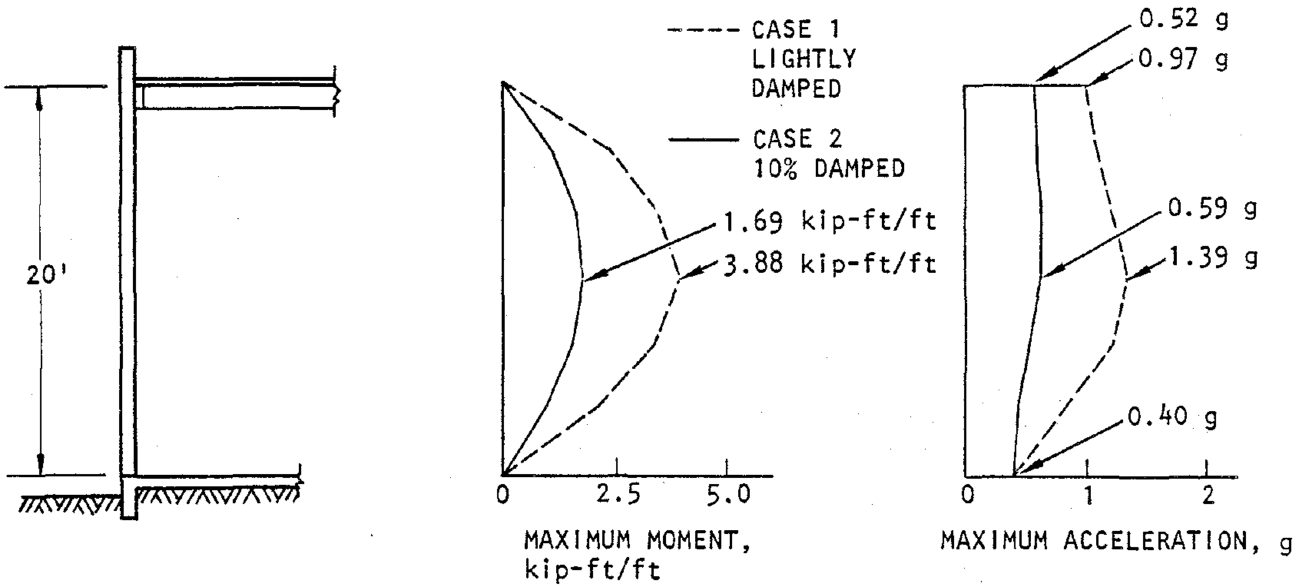


FIGURE 6-16. BENDING MOMENT AT MIDHEIGHT OF THE MIDSPAN PANEL CASE 2 — 10 PERCENT DAMPING



(a) Idealized model of TUW building



(b) TUW section

(c) Absolute maximum moment at midspan TUW

(d) Absolute maximum acceleration at midspan TUW

FIGURE 6-17. MAXIMUM MOMENTS AND ACCELERATIONS IN THE MIDSPAN TUW

SECTION 7

FAILURE MODES OF TILT-UP-WALL STRUCTURES

7.1 FAILURE MODES FROM PAST EARTHQUAKES

A review of the damage reports for the 1971 San Fernando and 1964 Alaska earthquakes (Murphy, 1973; Kunze et al., 1965) indicates that the most typical mode of failure for a warehouse type of TUW structure is caused by the failure of the roof-to-wall connections. Most of the TUW structures damaged in these earthquakes have roofs that consist of plywood supported by wood rafters and purlins, which in turn are supported by steel beams and tapered steel carrying girders. These girders bear on columns of three types: steel pipes, steel-rolled sections, and reinforced concrete. The most common failure mode occurs when the large horizontal forces induced by the seismic accelerations exceed the capacity of the roof-to-wall connection. The supporting precast concrete walls and beams separate from the roof when the nailing that attaches the plywood to the ledger pulls through the edges of the plywood, and, in some instances, the wood ledger bolted to the wall splits at the bolt line. Consequently, wood purlins supporting the roof lose their bearing, carrying the roof down with them. In some cases, TUW panels fall flat to the ground after losing lateral support provided by the roof. In other cases, TUW panels remain in the upright position, although somewhat out of plumb and bowed.

When the TUW panels are connected by poured-in-place concrete pilasters, the most heavily damaged area is usually at the interface between the roof beams or girders and the pilasters. Considerable movement between girders and wall pilasters undoubtedly causes severe cracking and spalling of concrete at these connections. Walls are usually damaged extensively by cracking of the concrete. However, roofs usually do not collapse, and walls remain upright and connected to the pilasters.

Another failure mode of a TUW structure involves a more complicated dynamic response of the structure due to the unsymmetric arrangement of TUWs. An example of this type of failure mode is the partial collapse of the J.C. Penney store during the 1964 Alaska earthquakes. After the earthquake, the building assumed a wracked position, indicating a permanent counter-clockwise deformation in plan. It should be recognized that the unsymmetrical layout of structural and nonstructural elements, including cutouts, partitions and doorways, may induce torsional response in the structure that is beyond the design limits of the walls and joints.

Admittedly, the current efforts in extracting failure modes of TUW structures under seismic environment are by no means exhausting. However, it appears generally true that failure of the TUW itself seldom is the primary cause of structural failure. Connections are the critical components in panelized construction with respect to acceptable seismic response.

7.2 ALTERNATIVE MODES OF FAILURE

Although observations of past earthquakes suggest the need for designers to harden connections and to induce failure in the panels, the role that the panels can play as energy dissipation mechanisms and as elements essential to the integrity of the TUW structure is not immediately clear. It is generally accepted conceptually that panelized structures are nonductile. Both current evidence and practical consideration indicate that panels will not be the main source of inelastic response. This is because ductility, which is the most widely accepted energy dissipating mechanism, is associated with flexural inelasticity. For practical reasons, the development of such flexural ductility in panelized structures may not be feasible and the normal flexural ductility of the wall cannot be counted upon as an energy dissipating device in all future development of the TUW design. This does not, however, exclude the possibility that the panels be purposefully designed to dissipate energy through controlled shear deformation; nor does it eliminate the exploitation of other forms of energy dissipation mechanism, though potentially less efficient than flexural ductility in the panels. An examination of the past failure modes thus indicate the need for improvement to achieve a more well balanced design. Such a new design approach should be examined carefully using current sophisticated assemblage analyses techniques and verified by experimental data.

The following areas should be emphasized in the investigation of TUW structures:

- Connections must be designed not only as the mechanism by which the panels are joined and the load transferred, but also as regions of energy dissipation under seismic loading. Proper connection designs must be analyzed and tested before connection failure can be eliminated as the primary cause of structural failure.

- With the elimination of premature failure of the connections, the general observation that TUW panels seldom fail or crack during an earthquake may no longer be true. The moments in TUW panels can now be allowed to develop and the most critical moments in the TUW panels during an earthquake must be compared against their design values.

- Since the seismic loading is somewhat proportional to the mass of the TUW, reducing the wall thickness also reduces the dynamic portion of the loading. However, the static portion, which is composed mainly of roof loading, is not reduced with decreasing wall thickness.

It appears that the simplest way of inducing inelastic deformation in the TUW panels can be achieved by reducing the wall thickness. The question of what type of failure mechanism can be induced in the TUW panels and what role the failure mechanism plays in maintaining the integrity of the TUW structure must be investigated.

SECTION 8

CONCLUSIONS OF PHASE I STUDY AND RECOMMENDATIONS FOR PHASE II STUDY

8.1 CONCLUSIONS

1. Preliminary seismic dynamic analysis conducted for a typical TUW building in a highly seismic area resulted in moments in TUW panels and forces at roof connections that are higher than those otained by current equivalent static analysis methods. This implies that for the building analyzed the interaction of TUW panels and roof diaphragm during the selected earthquake results in higher forces at the connections and amplified moments in panels.

2. Damage of panel-to-roof connections has been the main mode of failure of TUW structures during past earthquakes.

3. Strengthening of panel-to-roof connections may switch failure mode from connections to panels during future earthquakes.

4. Test data on response of TUW panels to out-of-plane seismic dynamic forces are not available in the literature.

5. Tests on hysteretic cyclic behavior of TUW panels have not been conducted.

6. Test data on static deflection of panels subjected to lateral loads have not been generated.

7. Assumptions used in current TUW panel analyses methods have not been correlated to test data.

8. For TUW structures in highly seismic areas, plywood diaphragms are most commonly used for roof diaphragms. Test data indicate that these diaphragms are stiff and amplify earthquake ground motions.

9. Preliminary trends found in this research are in general agreement with those concluded from ABK studies. Namely, anchorages of walls to roof diaphragms during strong earthquake shaking are subjected to forces that are considerably higher than those obtained by current design methods.

10. Only one typical TUW building, modeled with one nonlinear material, was analyzed using one typical strong earthquake ground motion and two damping values. Therefore, conclusions based on these numerical results should be regarded only as preliminary trends, and additional analyses are needed.

8.2 RECOMMENDATIONS

In order to further evaluate and extend the findings of Phase I, the following recommendations are given for a Phase II study.

a. Analyses of the one-story TUW building, studied in Section 6, should be extended to include

   1. a larger number of segments in the model
   2. an ensemble of earthquake input motions
   3. modeling of side shear walls
   4. a hysteretic model for panels
   5. a more refined diaphragm nonlinear model
   6. several damping models
   7. different TUW building configuratons
   8. inclusion of the P-Δ effect
   9. varying vertical overburden loads

b. An experimental program for panels that include a pseudo-static and dynamic series of tests is needed. Pseudo-static tests will account for loading and unloading under load reversal. Dynamic tests will be performed for both low amplitude and large amplitude earthquake forces. These tests will provide data for correlation with the results of the analyses conducted in the Phase II study and for confirmation of the trends described in the Phase I study.

c. More refined reinforced concrete models that account for deterioration of stiffness with the propagation of cracks, should be used in analyses to assess current simplified design basis methods.

d. Data from current static tests on slender TUW panels should be used to evaluate assumptions used in current simplified analyses methods.

e. Torsional capabilities of the assemblage of panels and roof diaphragms need to be studied.

SECTION 9

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