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CURRENT AND TENTATIVE SEISMIC DESIGN PROVISIONS FOR BUILDINGS: --ETC(U)
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August 1979

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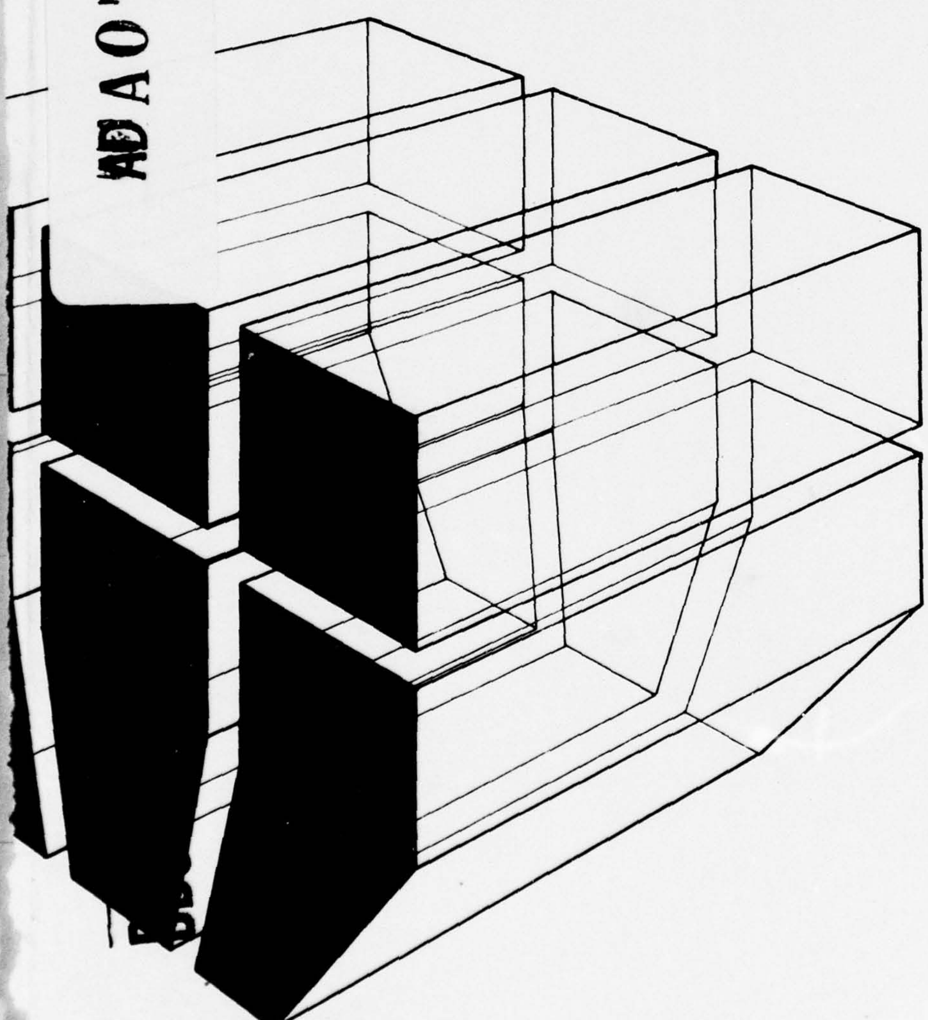
CURRENT AND TENTATIVE SEISMIC DESIGN PROVISIONS FOR BUILDINGS: PRELIMINARY COMPARISONS



by
J. D. Prendergast
W. E. Fisher

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[redacted] E./Fisher
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This report compares current and tentative seismic design provisions for two types of buildings: (1) Letterman Army Hospital, an existing 10-story, reinforced concrete building located in the Presidio of San Francisco, CA, whose design was based upon the 1964 Uniform Building Code (UBC), and (2) a three-story, ductile moment resistant steel frame building located in a region of high seismicity and designed as an essential building. The comparisons for Letterman Hospital include the magnitude and distribution of the seismic story shears and lateral deflections for the 1964 UBC, the 1975 Structural Engineers Association of California (SEAOC) provisions, the 1978 Applied Technology Council's

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tentative design provisions (ATC-3), the TM 5-809-10 Appendix proposed design provisions, Agababian Associates' (AA) two-dimensional time history modal analysis, U.S. Army Construction Engineering Research Laboratory's (CERL) three-dimensional response spectrum modal analysis and PMB Systems Engineering Incorporated's (PMB) three-dimensional response spectrum modal analysis. The comparisons for the three-story, ductile moment resistant steel frame building include the seismic design forces, story shears, lateral deflections, member sizes, and frame weights for the transverse direction of the building when designed in accordance with the 1975 SEAOC provisions, the 1978 ATC-3 equivalent lateral force procedure, the 1978 ATC-3 modal analysis procedure, and the TM 5-809-10 Appendix proposed provisions.

For Letterman Hospital, the 1964 UBC provisions produce equivalent yield stress basis story shears which are approximately 10 to 35 percent of the story shears obtained in AA's time history modal analysis. When the dynamic characteristics of the structure are considered in determining the total lateral forces and their distributions, the 1975 SEAOC provisions produce equivalent yield stress basis story shears which are in reasonable agreement with the story shears obtained in AA's time history analysis except in the base structure and the intermediate stories in the tower in the east-west (E-W) direction. The 1978 ATC-3 modal analysis procedure produces story shears which are generally approximately 100 percent greater than the 1964 UBC equivalent yield stress basis story shears in the tower and approximately 10 percent less than the 1964 UBC equivalent yield stress basis story shears in the base structure. When comparable acceleration levels are specified, the TM 5-809-10 Appendix provisions produce story shears which are in reasonable agreement with the story shears from AA's time history analysis except for the lower stories in the north-south (N-S) direction, where they differ by approximately 20 percent. Furthermore, the comparison of the 1978 ATC-3 and TM 5-809-10 Appendix provisions and the PMB analysis reveals that when comparable acceleration levels are specified, the story shears for the 1978 ATC-3 provisions are approximately 30 percent of the story shears obtained from the TM 5-809-10 Appendix provisions and the PMB analysis. Considering the complexity of the structure and the numerous and different assumptions that were employed, the agreement between the story shears for the TM 5-809-10 Appendix provisions and the PMB analysis is excellent.

The lateral deflections for Letterman Hospital indicate that there is better agreement among the lateral deflections in the N-S direction than in the E-W direction, with the exception that in both directions, the lateral deflections from the 1964 UBC design are quite small and bear little resemblance to the lateral deflections obtained by the other analyses.

For the three-story building, the 1978 ATC-3 story shears are 50 to 60 percent lower than the 1975 SEAOC equivalent yield stress basis story shears, while the TM 5-809-10 Appendix story shears are about 325 percent greater than the 1975 SEAOC story shears. However, there is very little difference between the 1978 ATC-3 and TM 5-809-10 Appendix lateral deflections. Typical lateral force-resistant frames designed in accordance with the 1978 ATC-3 modal analysis procedure and the 1975 SEAOC provisions have nearly identical weights, while frames designed in accordance with the 1978 ATC-3 equivalent lateral force procedure and the TM 5-809-10 Appendix provisions have weights about one and one-half and two times greater, respectively, than the 1975 SEAOC design.

FOREWORD

This investigation was performed for the Directorate of Military Programs, Office of the Chief of Engineers (OCE) under Project 4A762731AT41, "Design, Construction, and Operation and Maintenance Technology for Military Facilities"; Task 04, "Military Construction Technology"; Work Unit 003, "Seismic Upgrading of Existing Critical Facilities." The applicable QCR is 3.07.005. The OCE Technical Monitor was G. M. Matsumura, DAEN-MPE-B.

The investigation was performed by the Engineering Team, Engineering and Materials Division (EM), U.S. Army Construction Engineering Research Laboratory (CERL). Dr. G. R. Williamson is Chief of EM.

COL J. E. Hays is Commander and Director of CERL, and Dr. L. R. Shaffer is Technical Director.

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CURRENT AND TENTATIVE SEISMIC DESIGN PROVISIONS FOR BUILDINGS: PRELIMINARY COMPARISONS

1 INTRODUCTION

Background

Recent earthquake experience in the United States such as that gained from the 1964 Alaskan, the 1969 Santa Rosa, and the 1971 San Fernando earthquakes has shown that the philosophy for earthquake-resistant design at that time was not completely adequate. This philosophy, as expressed in building codes, states in part that: "buildings should be able to resist a major earthquake, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage."[1] The collapse of the Four Seasons apartment house in Anchorage during the 1964 Alaskan earthquake, the damage sustained by the Social Service Building during the 1969 Santa Rosa earthquake, and the near collapse of the multistory Olive View Hospital during the moderate 1971 San Fernando earthquake are obvious examples which indicate that significant engineering and scientific problems still remain to be solved.

However, since 1971, the Structural Engineers Association of California (SEAOC) has revised its "Recommended Lateral Force Requirements" to more realistically reflect the expected dynamic response of real structures, and to provide a means for establishing more stringent design criteria for essential facilities which must be functional for emergency post-earthquake operations. Recently, the National Science Foundation and the National Bureau of Standards completed a jointly sponsored project with the Applied Technology Council (ATC) to develop comprehensive nationally applicable seismic design provisions for the United States. The "Tentative Provisions for the Development of Seismic Regulations for Buildings," developed by this project, embody both current and several new concepts for seismic design.[2] Likewise, the Army is revising and updating TM 5-809-10, *Seismic Design for Buildings*, to improve seismic design provisions for conventional buildings and to provide guidelines for the design of essential facilities that must remain functional after a major earthquake. The latter guidelines, which are based on the technology produced in the ATC project, are tailored to military requirements.[3]

To date, detailed comparisons and evaluations of these various seismic design provisions have not been made, although they are urgently needed in order to reconcile differences and inconsistencies in the design philosophies. This report is a first attempt at comparing and evaluating various seismic design provisions.

Purpose

The purpose of this report is to compare current and tentative seismic design provisions for two types of buildings: (1) Letterman Army Hospital, an existing 10-story reinforced concrete building located in the Presidio of San Francisco, CA, whose design was based on the 1964 Uniform Building Code (UBC) and (2) a three-story, ductile moment resistant steel frame building located in a region of high seismicity and designed as an essential building. The comparisons for Letterman Hospital include the magnitude and distribution of the seismic story shears and lateral deflections for the 1964 UBC, the 1975 SEAOC provisions, the 1978 ATC-3 tentative seismic design provisions, the TM 5-809-10 Appendix proposed seismic design provisions, Agbabian Associates' (AA) two-dimensional time history modal analysis, the U.S. Army Construction Engineering Research Laboratory's (CERL) three-dimensional response spectrum modal analysis, and the PMB Systems Engineering Incorporated's (PMB) three-dimensional response spectrum modal analysis.[4-10] The comparisons for the three-story, ductile moment resistant steel frame building include the seismic design

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[1] *Recommended Lateral Force Requirements and Commentary* (SEAOC, 1968).

[2] *Tentative Provisions for the Development of Seismic Regulations for Buildings*, ATC-3-06 (Applied Technology Council, 1978).

[3] "Seismic Design Guidelines for Critical Buildings," Proposed Appendix to TM 5-809-10 (U.S. Army Construction Engineering Research Laboratory [CERL], September 1978, revised March 1979).

[4] *Uniform Building Code*, 1964 ed. (International Conference of Building Officials, 1964).

[5] *Recommended Lateral Force Requirements and Commentary* (SEAOC, 1975).

[6] ATC-3-06.

[7] Proposed Appendix to TM 5-809-10.

Footnotes continued on page 10.

forces, story shears, lateral deflections, member sizes, and frame weights for the transverse direction of the building when designed in accordance with the 1975 SEAOC provisions, the 1978 ATC-3 equivalent lateral force procedure, the 1978 ATC-3 modal analysis procedure, and the TM 5-809-10 Appendix proposed seismic design provisions.

Scope

Because of the difficulty and cost associated with performing additional static and dynamic analyses of the large and complex Letterman Hospital structure, the comparisons for Letterman Hospital are limited solely to the magnitude and distribution of the story shears and lateral deflections. For the three-story, moment resistant steel frame building, only the lateral force resisting system in the transverse direction was designed. The comparisons include: (1) the magnitude and distribution of the transverse story shears; (2) the transverse lateral deflections of the building under the prescribed lateral loads; (3) the frame member sizes for the transverse lateral force resisting frames; and (4) the weight of a typical steel frame required to resist the prescribed lateral forces in the transverse direction.

Mode of Technology Transfer

The information contained in this report will be used to prepare other material for inclusion in TM 5-809-10, *Seismic Design for Buildings.*

2 LETTERMAN HOSPITAL STORY SHEARS AND LATERAL DEFLECTIONS

Background

Building Description

Letterman Hospital is a 550-bed Army medical treatment center located in the northeast corner of the Presidio of San Francisco. The hospital is approximately 1 1/2 miles (2.4 km) southeast of the Golden Gate Bridge along the San Francisco Bay and approximately 5 to 7 miles (8.0 to 11.3 km) east of the San Andreas Fault (Figure 1). Designed and constructed in the mid-1960s, Letterman Hospital is a 10-story reinforced concrete structure consisting of a nominal 110 x 175 ft (33.5 x 53.3 m) tower and a three-story base structure with plan dimensions of 295 x 350 ft (90.0 x 106.7 m) (Figures 2 through 6). The lower two stories of the tower (the hospital's fourth and fifth floors) extend approximately 72 ft (22.0 m) north of the remainder of the tower (Figure 4). The structure is reasonably symmetrical in the north-south (N-S) direction; however, there is appreciable eccentricity between the tower and base structure in the east-west (E-W) direction.

The exterior walls of the tower and base structure are precast concrete panels, while the other structural elements are cast-in-place concrete. The building has both horizontal and vertical load-resisting systems. The exterior walls, service cores, and floor slabs resist horizontal loads. Lightweight concrete floor slabs transmit wind and earthquake forces in addition to the vertical forces resulting from both earthquake and gravity loads, to the exterior walls and service cores, which carry these forces to the foundation.

Site Ground Accelerations

In 1973, a site-dependent seismic investigation of the Letterman Hospital site was performed by AA as part of a study to assess the seismic resistance of Letterman Hospital and to evaluate the feasibility and cost impact of upgrading the seismic resistance to maintain functional operations following a major earthquake. AA's study determined that an 8.2 Richter magnitude earthquake on the San Andreas Fault having a duration of strong shaking for 40 to 50 sec and a recurrence interval of approximately 100 yr was the maximum earthquake that could reasonably be expected to occur near the Letterman Hospital site.[11] Since ground shaking near the source of a great earthquake (Richter magnitude 8 to 8.5) has never been recorded, Type A-1 and A-2 artificial earthquake records (Figure 7) were selected to represent the site's ground motions. Interrelationships between Richter magnitude, distance from the site to the causative fault, and peak accelerations indicated that the site's maximum horizontal

[8] Agbabian Associates, *Dynamic Seismic Analysis of Primary Structural Systems for Letterman and Hays General Hospitals (Subtasks 2B, 2C, and 2D)*, Draft Report (CERL).

[9] J. D. Prendergast and C. K. Choi, *Three-Dimensional Seismic Structural Analysis of Letterman Hospital*, Technical Report M-175/ADA022085 (CERL, 1975).

[10] Anshen and Allen, *Comprehensive Upgrade (Feasibility Study) Building 1100, Letterman Army Medical Center, Presidio of San Francisco, California* (Sacramento District, Corps of Engineers, 1978).

[11] Agbabian Associates, *Site Dependent Maximum Probable Earthquake Criteria (Task 1)*, Draft Report (CERL).

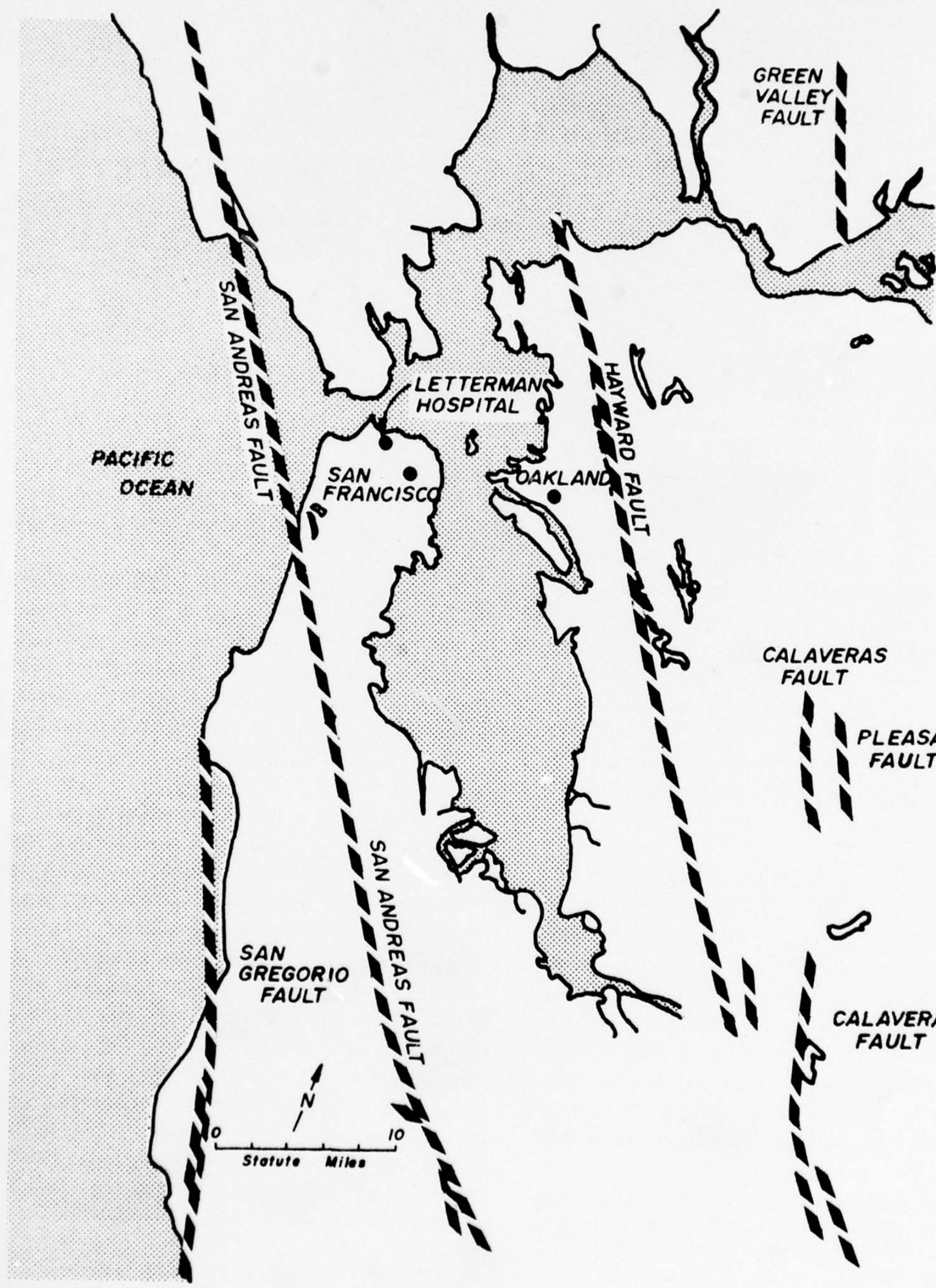

Figure 1. Location of Letterman Hospital.

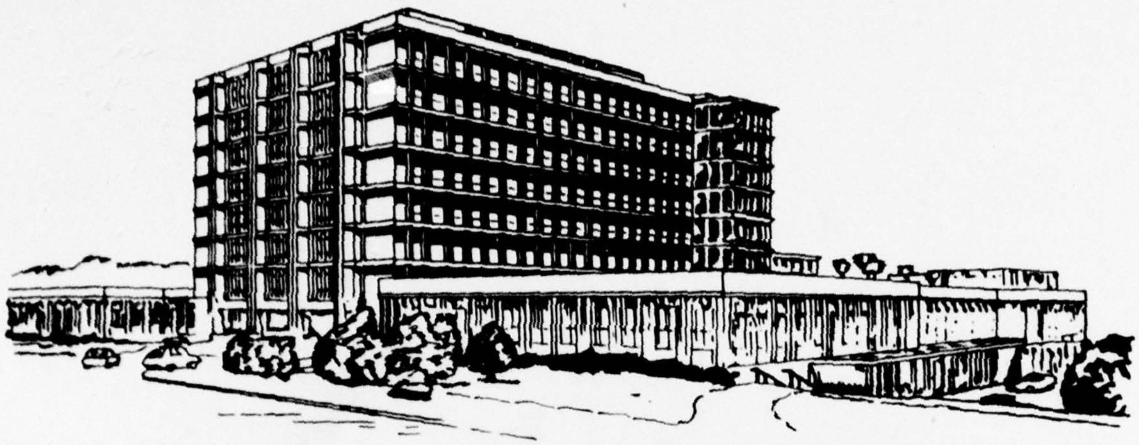

Figure 2. Letterman General Hospital, Presidio of San Francisco, CA.

ground acceleration was approximately 0.5 g. More recently, in conjunction with a supplementary upgrade study of Letterman Hospital, the Office of the Chief of Engineers (OCE) established performance criteria specifying that Letterman Hospital should be strengthened to prevent collapse resulting from a design earthquake which has a 20 percent probability of being exceeded in 50 years. Studies to evaluate the seismic risk and develop the earthquake ground motion criteria were conducted by Dames and Moore, San Francisco, CA. From these studies, the peak ground acceleration of the design earthquake was determined to be 0.35g.[12]

Dynamic Analyses

In AA's seismic upgrading study, the dynamic analyses were confined to modeling the structural system as a series of two-dimensional planar frame and shear wall assemblages, with each assemblage representing part of the structure's total lateral resistance, and connecting the individual assemblages into a concatenated model representing the total lateral resistance system in each principal direction.[13] The mathematical models were then subjected to the Type A-1 artificial earthquake time history and the response of selected structural elements was computed using the time history modal analysis method. Because of the planar aspect of these analyses, the effects of torsion were not investigated.

In support of the seismic upgrading study, CERL conducted a three-dimensional dynamic analysis of the primary structural system for Letterman Hospital to assess the impact of torsion and ascertain whether a simpler method of analysis, namely the response spectrum modal analysis method, would yield comparable results.[14] Since the lateral force resisting system for Letterman Hospital is basically composed of structural elements at a number of column lines in each direction of the structure, the structure was idealized as a series of planar frames with occasional isolated shear walls. Each frame was treated as an independent substructure, and the complete stiffness matrix for the structure was formed under the assumption that all frames were connected at each floor level by a diaphragm which was rigid in its own plane. Then dynamics analyses were conducted, using the response spectrum modal analysis method and the 10 percent damped smoothed average spectrum for the Type A-1 and A-2 artificial earthquake records (Figure 8). The maximum response of the various modes was combined by using the square root of the sum of the squares method.

[12] Anshen and Allen, *Comprehensive Upgrade (Feasibility Study) Building 1100, Letterman Army Medical Center, Presidio of San Francisco, California* (Sacramento District, Corps of Engineers, 1978).

[13] Agbabian Associates, *Dynamic Seismic Analysis of Primary Structural Systems for Letterman and Hays General Hospital (Subtasks 2B, 3C, and 2D)*, Draft Report (CERL).

[14] J.D. Prendergast and C.K. Choi, *Three-Dimensional Seismic Structural Analysis of Letterman Hospital*, Technical Report M-175/ADA022085 (CERL, 1975).

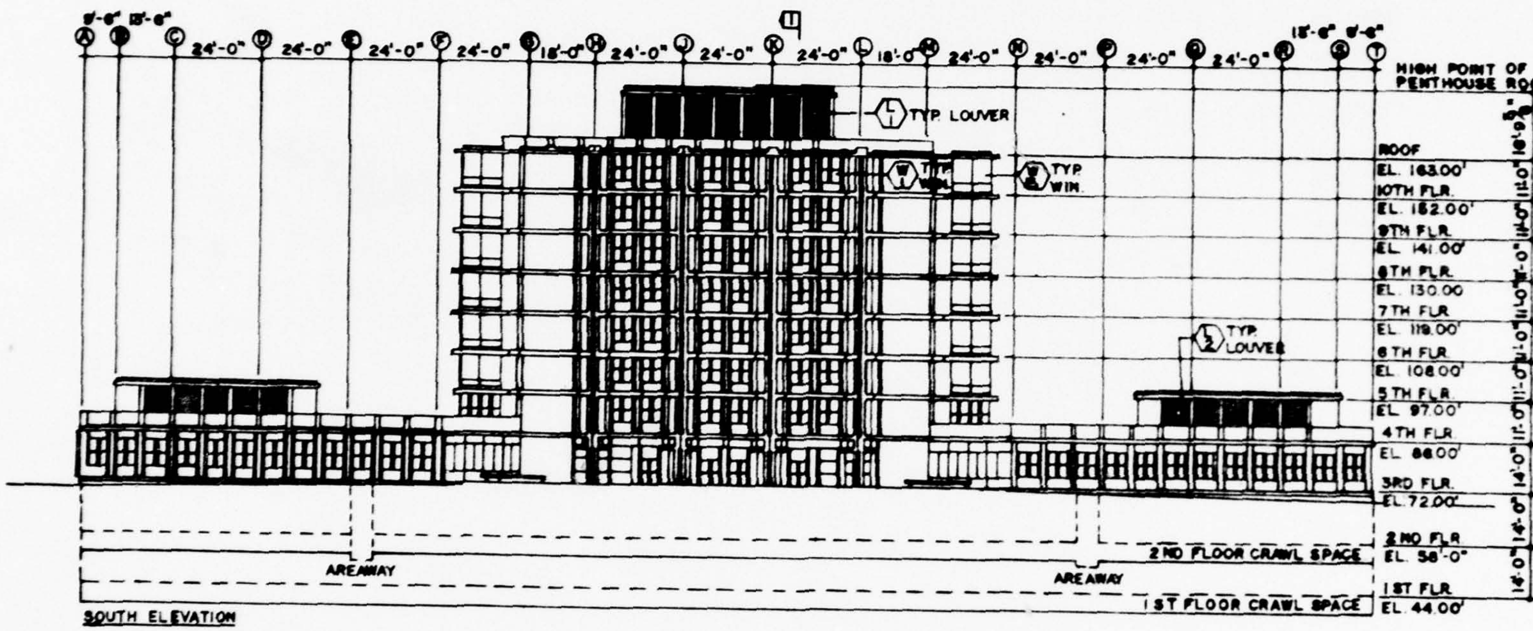

Figure 3. South elevation, Letterman Hospital.
(Metric conversion factor: 1 ft = 0.3 m)

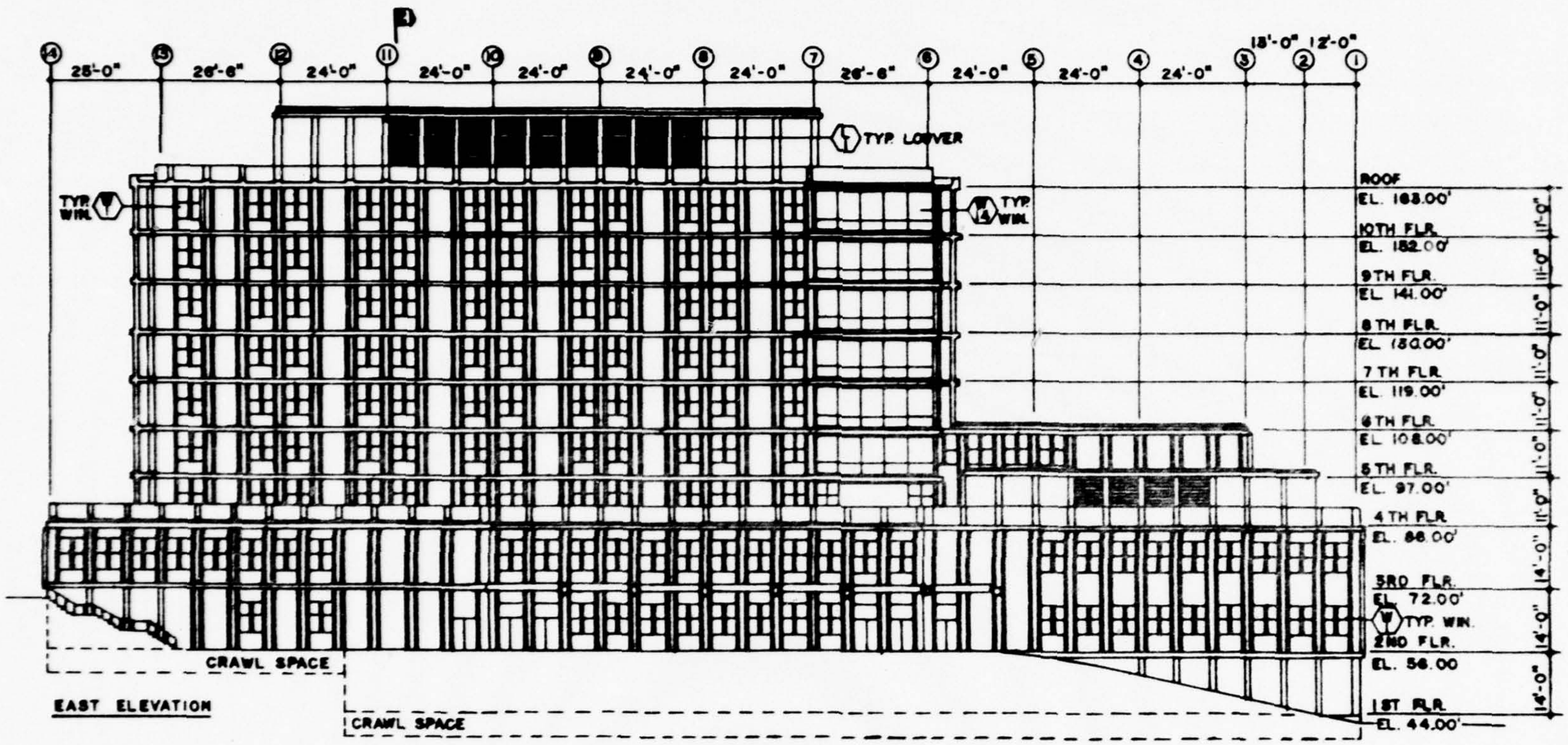

Figure 4. East elevation, Letterman Hospital.
(Metric conversion factor: 1 ft = 0.3 m)

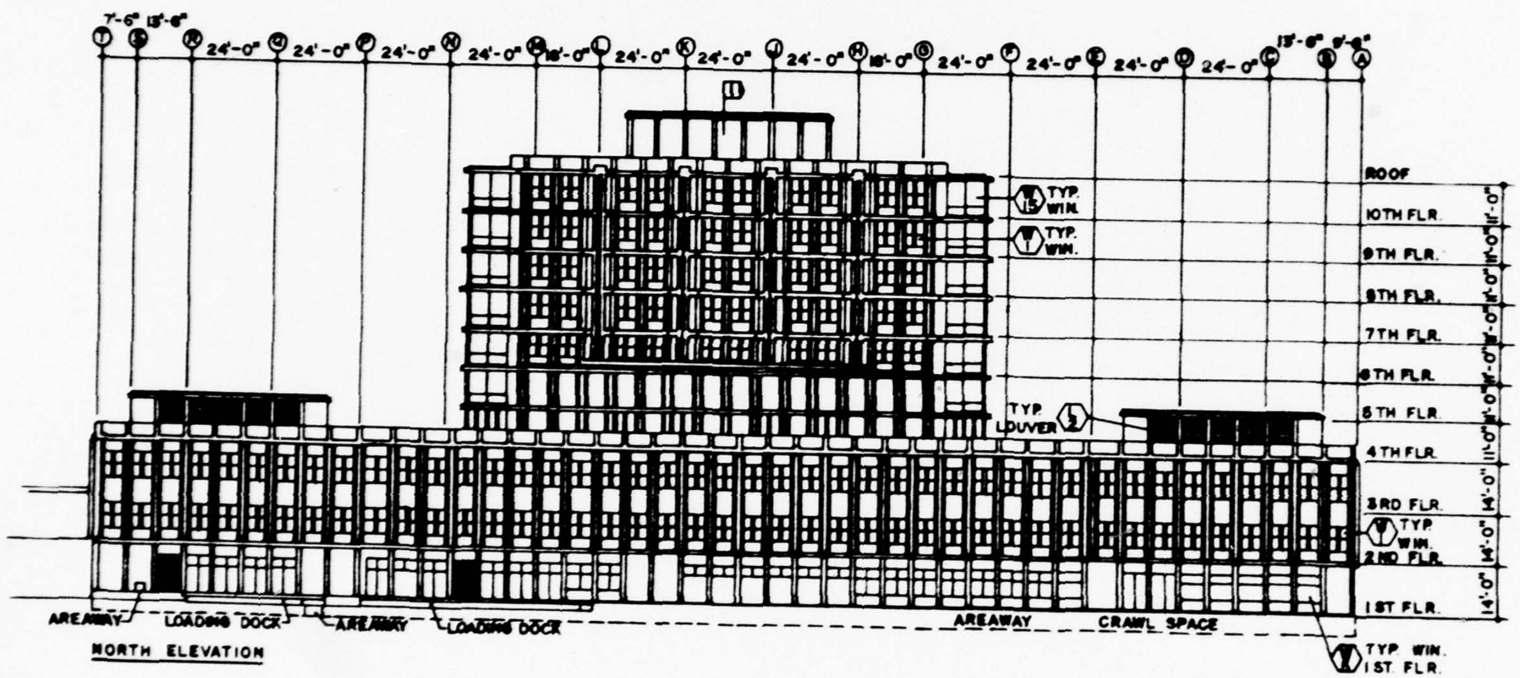

Figure 5. North elevation, Letterman Hospital.
(Metric conversion factor: 1 ft = 0.3 m)

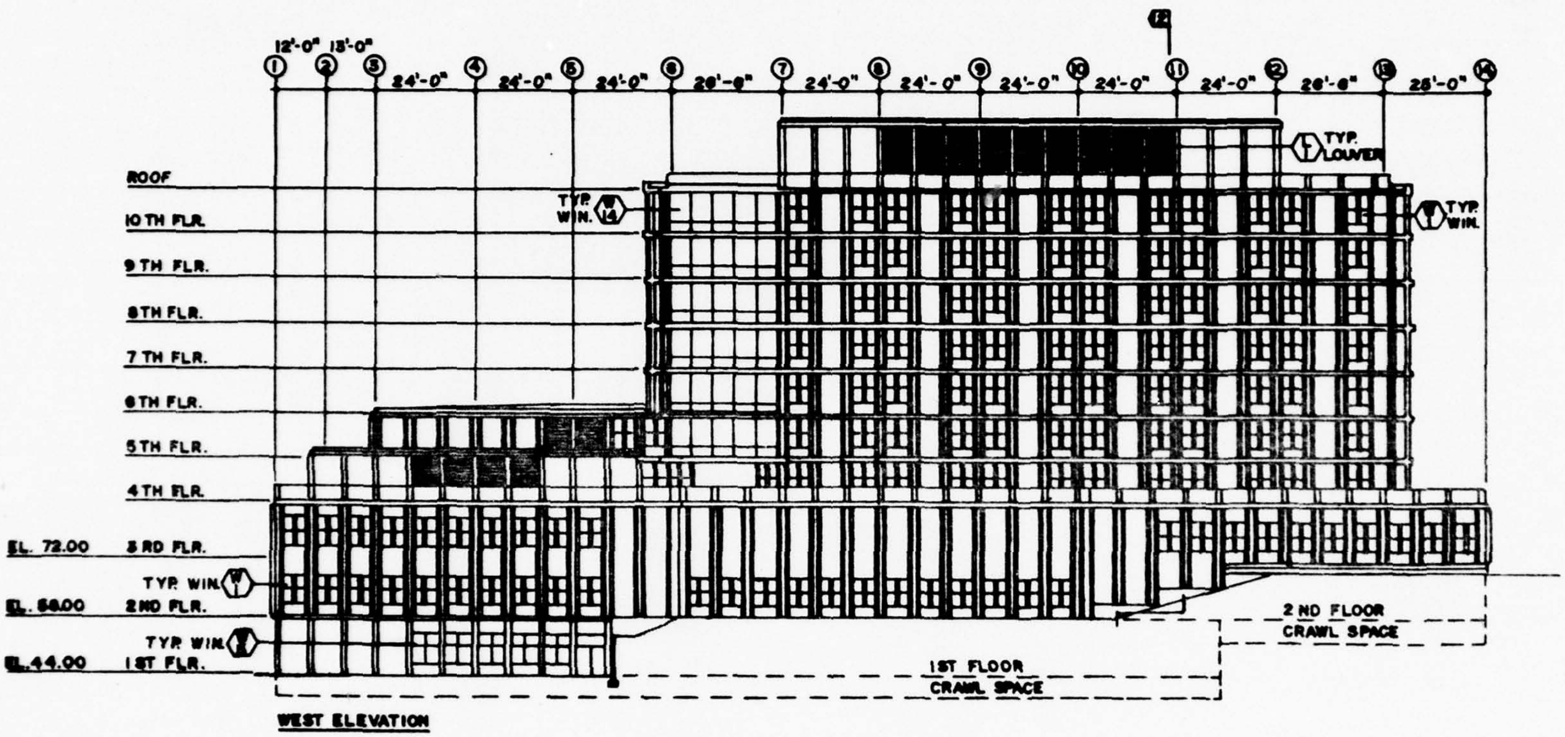

Figure 6. West elevation, Letterman Hospital.
(Metric conversion factor: 1 ft = 0.3 m)

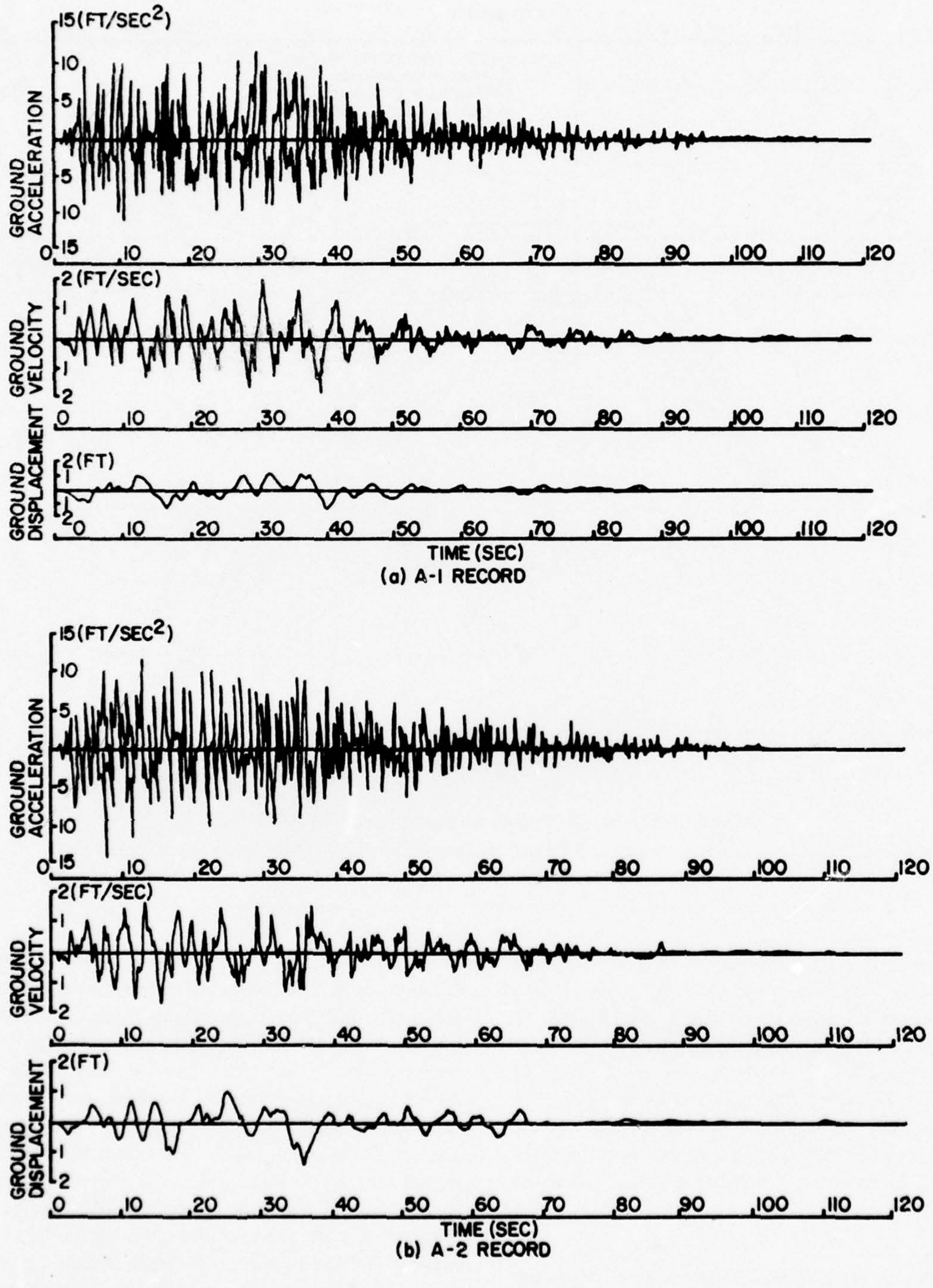

Figure 7. Motion time histories for Type A artificial earthquake records.
(Metric conversion factors: 1 ft = 0.3 m; 1 ft/sec = 0.3 m/sec; 1 ft/sec^2 = 0.3 m/sec^2.)

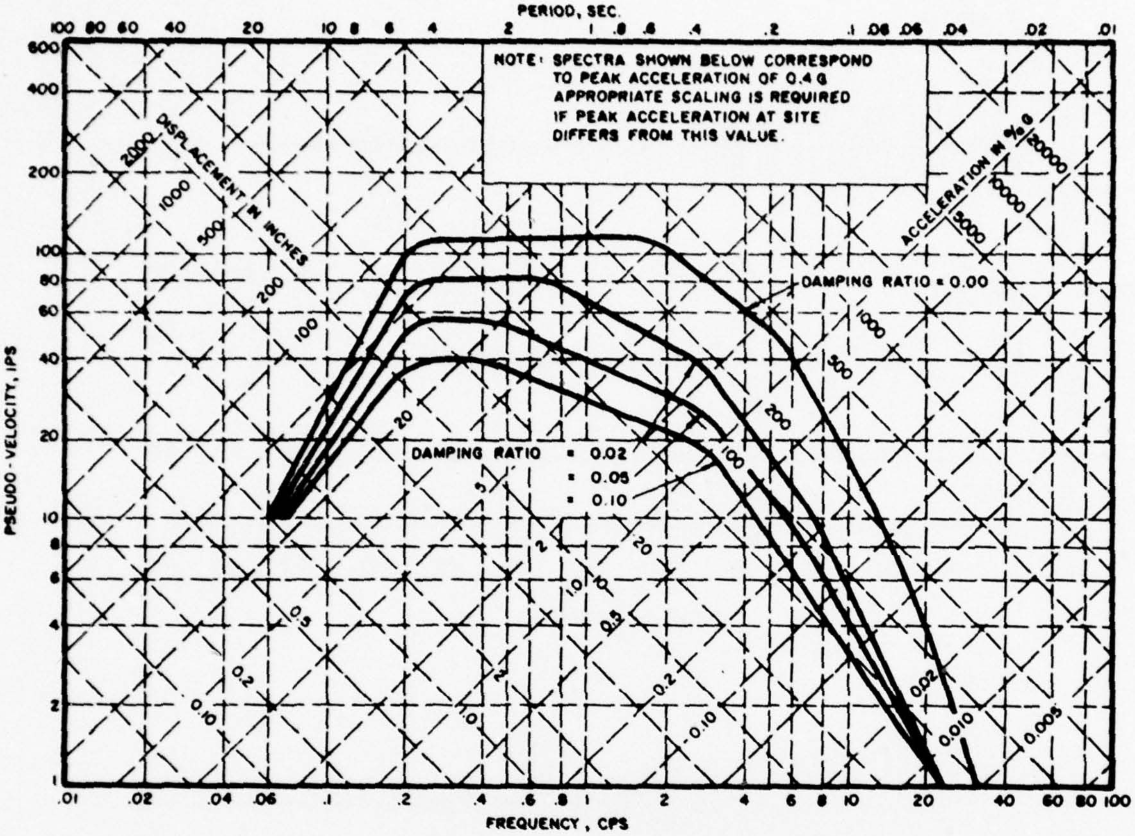

Figure 8. Earthquake response spectra for CERL analysis.
(Metric conversion factors: 1 in. = 2.54 cm; 1 in./sec = 2.54 cm/sec; 1 in./sec^2 = 2.54 cm/sec^2.)

In conjunction with the supplementary upgrade study of Letterman Hospital, PMB also conducted a three-dimensional response spectrum modal analysis of Letterman Hospital, using a computer model designed to improve the mathematical idealization of the existing hospital.[15] PMB used "dummy stories" to approximate the behavior of the caisson foundation system and of the non-rigid diaphragm at the fourth-floor level. In addition, the existing precast elements were modeled as piers and spandrels, with shear panels located where solid walls exist; the concrete columns and waffle slab frame system was modeled as columns joined by beams with properties of the standard design "column strip" of slab. Interior core walls were modeled as shear elements. Basement walls were also modeled as shear elements when they were of significant size. These various elements were assembled into planar frames or shear walls and then combined to form the total structure. Then dynamic analyses were conducted, using the response spectrum modal analysis method and the 10 percent damped spectrum shown in Figure 9. Likewise, the maximum response of the various modes was combined by using the square root of the sum of the squares method.

[15] Anshen and Allen, *Comprehensive Upgrade (Feasibility Study) Building 1100, Letterman Army Medical Center, Presidio of San Francisco, California* (Sacramento District, Corps of Engineers, 1978).

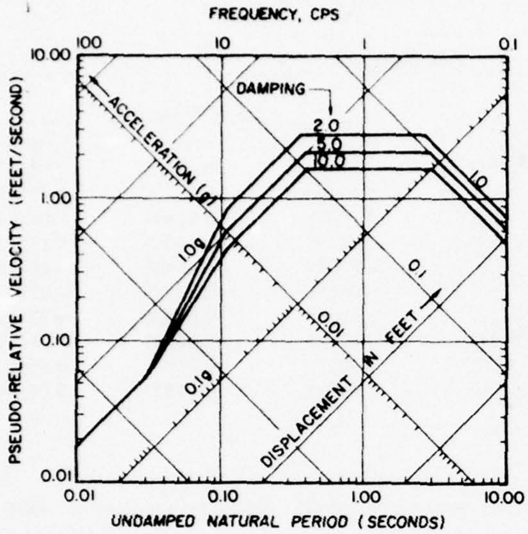

Figure 9. Earthquake response spectra for PMB analysis. (Metric conversion factors: 1 ft = 0.3 m; 1 ft/sec = 0.3 m/sec; 1 ft/sec^2 = 1 m/sec.)

The dynamic characteristics of Letterman Hospital must be known in order to compute the magnitude and distribution of the story shears and lateral deflection specified by the current and tentative design provisions. For consistency in these comparisons, the masses, periods, and mode shapes obtained from the CERL dynamic analyses have been adopted as representing the dynamic characteristics of the Letterman Hospital. Tables 1 and 2 summarize these dynamic characteristics for each of the principal directions.

Table 1

Predominant Periods of Vibration for Letterman Hospital

	Period, sec	
Mode	N-S Direction	E-W Direction
1	0.702	0.752
2	0.244	0.298
3	0.132	0.148

1964 UBC Provisions

The original seismic design of Letterman Hospital was based on the equivalent static lateral load specified in the 1964 UBC, i.e., the Recommended Lateral Force Requirements adopted by SEAOC in 1959.[16] In accordance with these provisions, the hospital was designed to withstand minimum total lateral forces assumed to act nonconcurrently in the direction of each of the building's main axes in accordance with the following formula:

$$V = KCW \qquad \text{[Eq 1]}$$

where V = total lateral load or shear at the base

K = numerical coefficient assigned values of 0.67, 0.80, 1.00, 1.33, depending on the type and arrangement of the resisting elements

C = numerical coefficient determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}} \qquad \text{[Eq 2]}$$

where T is the fundamental period of vibration of the building in seconds in the direction under consideration

W = total dead load

The total lateral force, V, was distributed over the height of the building in accordance with the formula

$$F_x = V \frac{w_x h_x}{\sum_{i=1}^{N} w_i h_i} \qquad \text{[Eq 3]}$$

where F_x = lateral force applied to a level designated as x

h_x = height in feet above the base to the level designated as x

w_x = that portion of W which is located or assigned to the level designated as x

$\Sigma w_i h_i$ = summation of the product $w_x h_x$ for the building

To account for the set-back conditions between the base structure and the tower, the tower was designed as a separate structure and the total shear resulting from

[16] Agbabian Associates, *Aseismic Design Criteria of Rehabilitation at Existing Hospital Facilities, Task 2A–Summary of Existing Static Design Data*, Draft Report (CERL).

Table 2

Masses and Mode Shapes for Letterman Hospital
(Metric conversion factor : $1 \frac{\text{kip-sec}^2}{\text{ft}} = 0.015 \frac{\text{mega kg-sec}^2}{\text{m}}$)

Level	Mass $\frac{\text{kip-sec}^2}{\text{ft}}$	Mode Shape: N-S Direction Mode 1	N-S Direction Mode 2	N-S Direction Mode 3	E-W Direction Mode 1	E-W Direction Mode 2	E-W Direction Mode 3
Roof	145.186	0.16999	0.10728	0.12317	0.17034	0.10212	0.10747
10	119.720	0.14973	0.05912	0.02597	0.14982	0.05911	0.01565
9	119.720	0.12788	-0.00963	-0.06288	0.12761	0.01406	-0.06775
8	119.720	0.10478	-0.03588	-0.11873	0.10393	-0.02866	-0.11518
7	119.720	0.08014	-0.07175	-0.12573	0.07940	-0.06406	-0.10968
6	154.814	0.05791	-0.09314	-0.08512	0.05148	-0.09575	-0.07942
5	157.609	0.03737	-0.09636	-0.01419	0.03647	-0.09820	-0.01802
4	622.671	0.01903	-0.08611	0.05954	0.02238	-0.08705	0.06247
3	513.975	0.00910	-0.05422	0.06102	0.01010	-0.04901	0.05402
2	385.093	0.00243	-0.01684	0.02297	0.00338	-0.01820	0.02548
1	372.671	0	0	0	0	0	0

the tower was applied at the top of the three-story base structure of the building. In the E-W direction, the tower was assumed to extend upward from the fourth floor; however, in the N-S direction, due to the northward extensions of the fifth and sixth floors, the tower was assumed to extend upward from the sixth floor.

The hospital was designed as a box system, i.e., K = 1.33. The values assigned to C for the E-W tower structure, E-W base structure, N-S tower structure, and N-S base structure were 0.070, 0.101, 0.084, and 0.087, respectively.[17] The lateral story forces and story shears used in the working stress design are tabulated in Table 3. Based on these values, Letterman Hospital was designed to resist approximately 0.1 g lateral load. However, to provide a consistent basis for comparison with some of the newer design provisions which are based on permitting material stresses to approach yield stresses, the working stress design story shears for Letterman Hospital must be multiplied by a factor to obtain equivalent yield stress basis design story shears. In reviewing the existing design, the design calculation sheets indicate that the designer used 32.0 ksi (220.6 MPa) as an allowable stress in reinforcing steel for earthquake forces. General notes on the drawings indicate that the yield stress for this reinforcing steel was 40.0 ksi (275.8 MPa).[18] Thus, the working stress design story shears for Letterman Hospital were multiplied by the factor 40/32 = 1.25 to obtain the approximate equivalent yield stress basis design story shears. These story shears are tabulated in Table 4 and presented as Curve A in Figures 10 and 11.

Table 3

Summary of 1964 UBC Working Stress Design Forces
(Metric conversion factor: 1 kip = 4.4 kN.)

Floor Level	N-S Direction Lateral Force kip	N-S Direction Story Shear kip	E-W Direction Lateral Force kip	E-W Direction Story Shear kip
Roof	725		662	
		725		662
10	469		459	
		1194		1121
9	352		382	
		1546		1503
8	234		305	
		1780		1808
7	119		229	
		1899		2037
6	1020		194	
		2919		2231
5	860		99	
		3779		2330
4	2650		3650	
		6429		5980
3	1385		1900	
		7814		7880
2	740		1020	
		8554		8900
1				

[17] *Aseismic Design Criteria.*

[18] *Aseismic Design Criteria.*

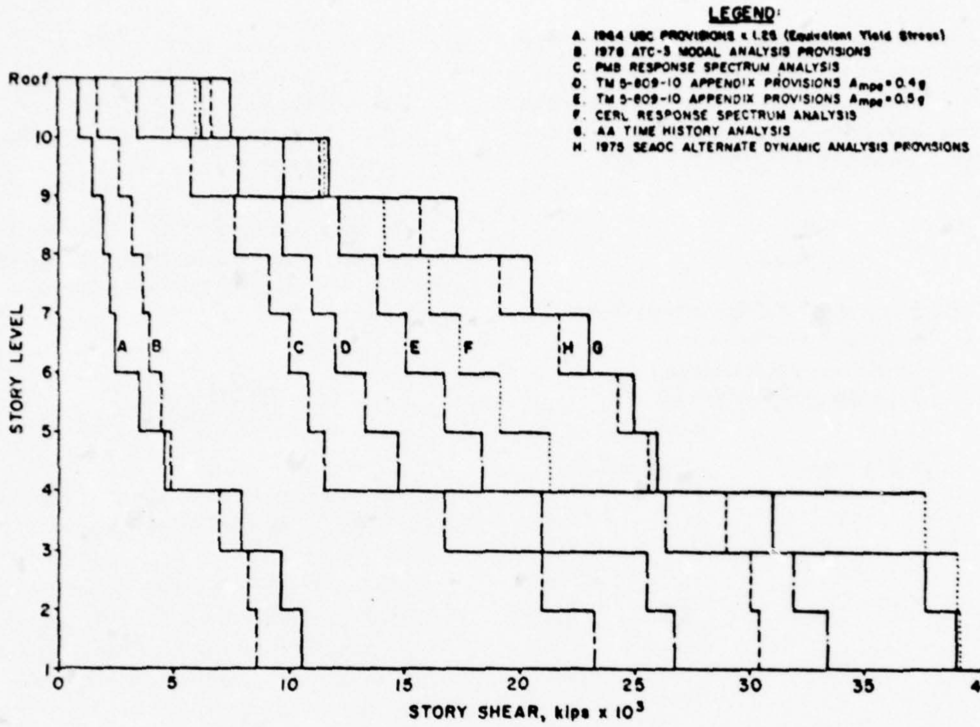

Figure 10. Letterman Hospital story shears (yield stress basis) N-S direction.

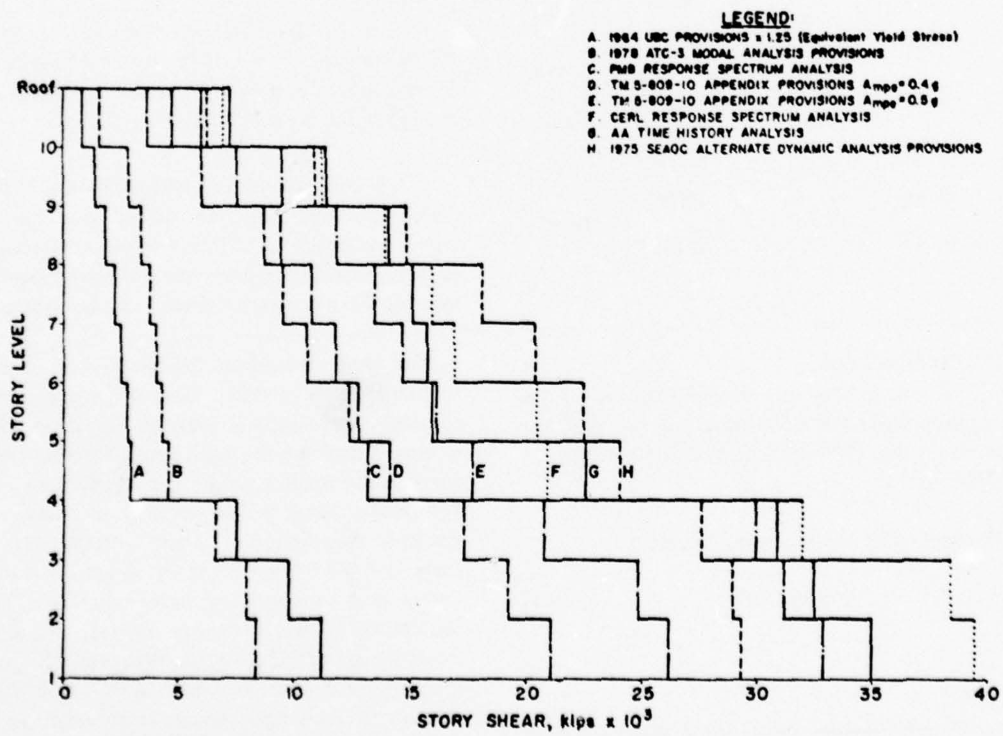

Figure 11. Letterman Hospital story shears (yield stress basis) E-W direction.

The lateral deflections resulting from the working stress design forces were also multiplied by the factor 1.25 to approximate the lateral deflections at yield. These lateral deflections are also tabulated in Table 4 and presented as Curve A in Figure 12.

Table 4

1964 UBC Equivalent Yield Stress Story Shears and Lateral Deflections (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	**Story Shear kip**	**Deflection ft**	**Story Shear kip**	**Deflection ft**
Roof		0.013		0.014
	906		828	
10		0.012		0.014
	1493		1401	
9		0.012		0.014
	1932		1879	
8		0.011		0.013
	2225		2260	
7		0.010		0.012
	2374		2546	
6		0.009		0.011
	3649		2789	
5		0.008		0.010
	4724		2912	
4		0.007		0.009
	8036		7415	
3		0.005		0.005
	9768		9850	
2		0.002		0.002
	10692		11125	
1		0		0

1975 SEAOC Provisions

The 1975 edition of the *Recommended Lateral Force Requirements and Commentary* is substantively different from the 1959 edition, and includes the following changes:[19]

a. The base shear formula was changed to

$$V = ZIKCSW \qquad \text{[Eq 4]}$$

where Z = numerical coefficient related to the region's seismicity
I = occupancy importance coefficient
S = numerical coefficient for site-structure resonance

b. To more realistically reflect the expected dynamic response of real structures in the areas of highest seismicity, the formula for determination of C was changed to

$$C = \frac{1}{15\sqrt{T}} \qquad \text{[Eq 5]}$$

c. The coefficient S was added to the base shear formula to provide a means of evaluating site resonance.

d. The coefficient I was added to the base shear formula. This established a higher level of design criteria for an essential facility (e.g., a hospital) than for a normal facility. The coefficient was assigned values of 1.5 and 1.0 for essential and normal facilities, respectively; however, I values were not established for other classes of facilities.

e. The coefficient Z was added to the base shear formula to permit the use of reduced base shear values for portions of California and elsewhere which are not located in the areas of highest seismicity.

f. 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 features must be determined by considering the structure's dynamic characteristics.

The large set-back at the fourth-story level of Letterman Hospital creates a large difference in the lateral resistance and stiffness between the third and fourth stories. Therefore, the equivalent lateral load method is not directly applicable and the distribution of the lateral forces must be determined by considering the dynamic characteristics of the structure. To approximate the 1975 SEAOC story shears, the total basis shears were computed in accordance with Eq 4 and distributed to the individual stories, considering the dynamic characteristic of the structure. The period and mode shape for the fundamental mode of vibration in each of the principal directions presented in Tables 1 and 2, respectively, were assumed to represent the structure's dynamic characteristics. The base shear in

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[19] *Recommended Lateral Force Requirements and Commentary* (Structural Engineers Association of California, 1975).

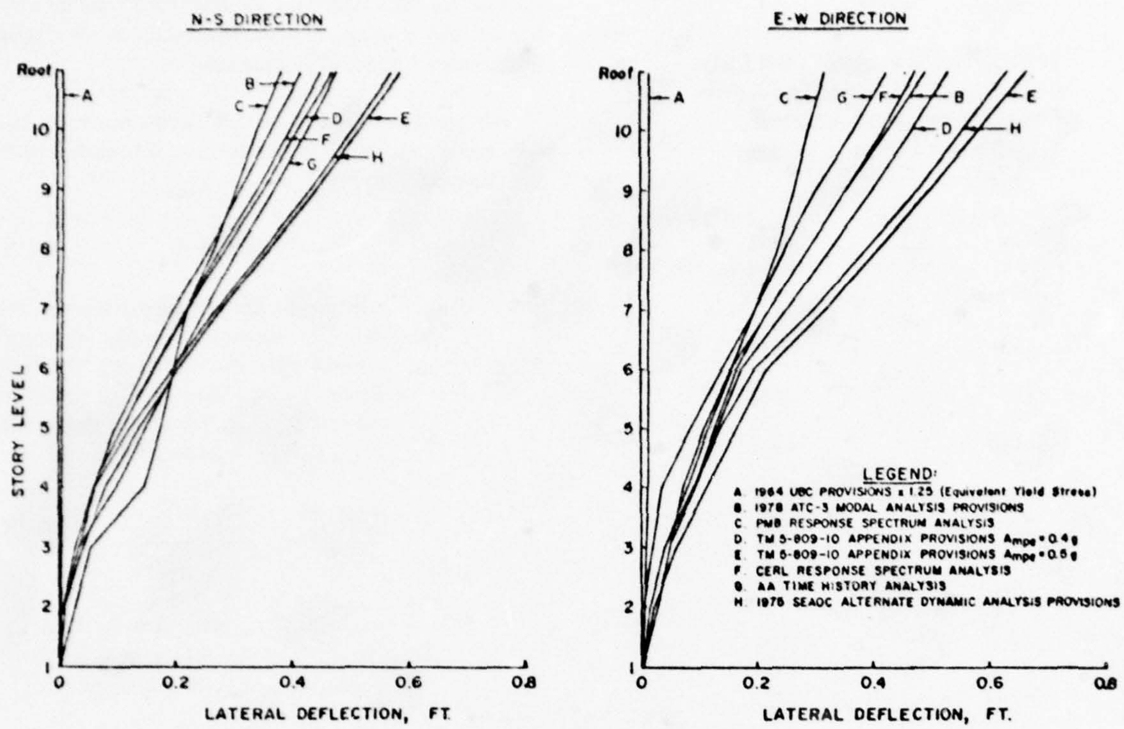


Figure 12. Letterman Hospital lateral deflections.

each of the principal directions was computed in accordance with Eq 4, taking Z = 1.0, I = 1.5, S = 1.5, K = 1.33, and C = 0.0769 and 0.0796 for the E-W and N-S directions, respectively. The coefficient S is dependent on the ratio of the building period to the characteristic site period. When the characteristic site period is not substantiated by proper analysis, as in the case at hand, the value of S shall be 1.5.

The resulting base shears were then distributed to the individual stories in proportion to the mass and deflection at each level to obtain the lateral force of each level. Subsequently, the lateral story forces were summed to obtain the story shear at each level. The story shears obtained by this approximate method of dynamic analysis must be multiplied by the load factor 1.4 to obtain yield stress basis design story shears. These yield stress basis design story shears are tabulated in Table 5 and presented as Curve H in Figures 10 and 11 for the N-S and E-W directions, respectively. Deflections for the 1975 SEAOC approximate method of dynamic analysis were computed using the equivalent yield stress story forces and the dynamic properties of the fundamental mode of vibration in each of the principal directions of the hospital. These deflections are also tabulated in Table 5 and presented as Curve H in Figure 12.

1978 ATC-3 Provisions

The provisions of ATC-3-06, *Tentative Provisions for the Development of Seismic Regulations for Building*, embody several concepts which are significant departures from currently used seismic design provisions.[20] These concepts include:

a. The incorporation of more realistic seismic ground motion intensities.

b. Response modification coefficients (reduction factors) which are based on consideration of the inherent toughness, the amount of damping which occurs when undergoing inelastic response, and the observed past performance of various types of framing systems.

[20] *Tentative Provisions for the Development of Seismic Regulations for Buildings*, ATC-3-06 (Applied Technology Council, 1978).

Table 5

1975 SEAOC Equivalent Yield Stress Story Shears and Lateral Deflections (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.574		0.628
	6674		6369	
10		0.505		0.553
	11521		10988	
9		0.432		0.471
	15661		14922	
8		0.354		0.383
	19053		18126	
7		0.270		0.293
	21648		20574	
6		0.196		0.190
	24072		22626	
5		0.126		0.134
	25665		24107	
4		0.064		0.083
	28869		27695	
3		0.031		0.037
	30134		29032	
2		0.008		0.012
	30387		29367	
1		0		0

c. Classification of building use-group categories into Seismic Hazard Exposure Groups.

d. Seismic performance categories for buildings with design and analysis requirements dependent on the seismicity index and building seismic hazard exposure group.

e. Simplified structural response coefficient formulas related to the fundamental period of the seismic resisting system of the building.

f. Material design and analysis based on stresses approaching yield.

The 1978 ATC-3 provisions require that the magnitude and distribution of the story shears for Letterman Hospital be determined by the modal analysis procedure because of the irregular plan and vertical configuration of the hospital. Moreover, the provisions specify that the lowest three modes of vibrations in each of the two mutually perpendicular axes be considered. For consistency, the periods and mode shapes from Tables 1 and 2 were adopted.

The portion of the base shear contributed by the m^{th} mode, V_m, was determined in accordance with the following formula:

$$V_m = C_{sm} \overline{W}_m \qquad \text{[Eq 6]}$$

where C_{sm} = modal seismic design coefficient for the m^{th} mode determined in accordance with the following formula; however, the value of C_{sm} need not exceed 2.5 $A_{a/R}$ for stiff or deep soils, nor 2.0 $A_{a/R}$ for soft soils

$$C_{sm} = \frac{1.2 A_v S}{R T_m^{2/3}} \qquad \text{[Eq 7]}$$

$\overline{W}_m$ = the effective modal gravity load for the m^{th} mode determined in accordance with the following formula:

$$\overline{W}_m = \frac{\left[\sum_{i=1}^{n} w_i \phi_{im}\right]^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad \text{[Eq 8]}$$

A_a = seismic coefficient representing the effective peak acceleration

A_v = seismic coefficient representing the effective peak velocity-related acceleration

R = seismic response modification coefficient

S = seismic coefficient for the site's soil profile characteristics

T_m = period of vibration of m^{th} mode of the building

ϕ_{im} = displacement amplitude of the i^{th} level of the building when vibrating in its m^{th} mode.

Letterman Hospital was designed as a box system, i.e., a structural system without a complete vertical load-carrying space frame where the required lateral forces are resisted by shears walls, and the reinforced concrete shear walls and precast concrete exterior wall panels are designed to resist the lateral forces. In the 1978 ATC-3 provisions, slightly different terminology is used to describe the various structural systems; the structural system most comparable to the original design would appear to be a bearing wall system with reinforced concrete shear walls. The seismic response modification coefficient, R, and deflection amplification factor, C_d, specified for this particular structural system are 4 1/2 and 4, respectively (i.e., R = 4 1/2 and C_d = 4). Moreover, considering the location of Letterman Hospital, $A_a = A_v = 0.4$, and since the soil properties and profile are not known in detail nor does the known profile fit any specified types, S = 1.2.

The modal force at each level, F_{xm}, was determined in accordance with the formula:

$$F_{xm} = C_{vxm} V_m \qquad \text{[Eq 9]}$$

where

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum\limits_{i=1}^{n} w_i \phi_{im}} \qquad \text{[Eq 10]}$$

The modal deflection, δ_{xm}, was determined in accordance with the formula:

$$\delta_{xm} = C_d \delta_{exm} \qquad \text{[Eq 11]}$$

where

$$\delta_{exm} = \frac{g}{4\pi^2} \, \frac{T_m^2 F_{xm}}{w_x} \qquad \text{[Eq 12]}$$

The story shears and lateral deflection at each level were computed for each mode, and the modal values combined by taking the square root of the sum of the squares of each modal value. The resulting story shears and lateral deflections for each of the principal directions are tabulated in Table 6. The story shears are shown as Curve B in Figures 10 and 11, respectively, while the lateral deflections are shown as Curve B in Figure 12.

1978 TM 5-809-10 Appendix Provisions

The provisions of the proposed 1978 TM 5-809-10 Appendix require that two design earthquakes be considered: the Maximum Credible Earthquake and the Maximum Probable Earthquake.[21]

The Maximum Credible Earthquake (MCE) represents the maximum level of horizontal ground acceleration at the site that appears capable of occurring within the presently known tectonic framework. No consideration is given to the probability of occurrence, but rather it is great enough to be of concern.

The Maximum Probable Earthquake (MPE) represents the maximum level of horizontal ground acceleration at the site that has a specified probability of being exceeded in a given period of time.

Realistically, these design earthquakes can be established only after a detailed site-dependent seismic investigation. The investigation of the Letterman Hospital site, which was conducted in support of the AA analysis, determined that the MCE and MPE were essentially identical: i.e., an 8.2 Richter magnitude earthquake on the San Andreas fault near the Letterman Hospital site with a maximum probable earthquake acceleration of approximately 0.5 g. However, to provide a consistent basis for comparing the story shears and lateral deflections obtained with the TM

Table 6

1978 ATC-3 Story Shears and Lateral Deflections
(Metric conversion factors:
1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	**Story Shear kip**	**Deflection ft**	**Story Shear kip**	**Deflection ft**
Roof		0.419		0.471
	1648		1591	
10		0.368		0.412
	2614		2536	
9		0.314		0.350
	3251		3173	
8		0.258		0.285
	3682		3611	
7		0.199		0.221
	4021		3940	
6		0.147		0.152
	4456		4298	
5		0.099		0.115
	4894		4724	
4		0.057		0.080
	7001		6932	
3		0.031		0.040
	8500		8330	
2		0.009		0.014
	8879		8753	
1		0		0

[21] "Seismic Design Guidelines for Critical Buildings," Proposed Appendix to TM 5-809-10 (CERL, September 1978; revised March 1979).

5-809-10 Appendix provisions with the story shears and lateral deflections obtained with the ATC-3 provisions (acceleration level = 0.4 g) and the AA and CERL dynamic analyses (acceleration level = 0.5 g), the analyses with the 1978 TM 5-809-10 Appendix provisions were performed for each of these acceleration levels. For consistency, the periods and mode shapes shown in Tables 1 and 2 were used. Moreover, the soil profile coefficient, S, and the response modification factor, R, were equal to 1.2 and 1.5, respectively.

The modal participation factor for each mode was computed from the formula

$$\Gamma_m = \frac{\sum_{i=1}^{N} w_i \phi_{im}}{\sum_{i=1}^{N} w_i \phi_{im}^2} \qquad \text{[Eq 13]}$$

where Γ_m = modal participation factor for the mth mode
w_i = weight at or assigned to level i
ϕ_{im} = amplitude of the mth mode at level i
N = number of stories

The seismic coefficient for the MCE and MPE were determined from the following formulas, respectively:

$$\text{MCE, } C_{sm} = \frac{1.2A_{mce}S}{RT_m^{2/3}} \qquad \text{[Eq 14]}$$

$$\text{MPE, } C_{sm} = \frac{1.2A_{mpe}S}{RT_m^{2/3}} \qquad \text{[Eq 15]}$$

where C_{sm} = seismic design coefficient for the mth mode, the value of which need not exceed 2.5 A/R for stiff or deep soils nor 2.0 A/R for soft soils
A = A_{mce} or A_{mpe}, respectively.
A_{mce} = peak horizontal ground acceleration associated with the MCE
A_{mpe} = peak horizontal ground acceleration with the MPE
S = coefficient for soil profile characteristics at the site
R = response modification factor, equal to 1.5 for the MPE
T_m = period of vibration of the building in the mth mode in the direction under consideration

The modal story lateral forces, F_{xm}, at level x for the mth mode were determined from the formula:

$$F_{xm} = C_{sm}\Gamma_m W_x \phi_{xm} \qquad \text{[Eq 16]}$$

and the elastic modal story displacements at each level, δ_{xm}, were determined from the formula

$$\delta_{xm} = \frac{gT_m^2 F_{xm}}{4\pi^2 w_x} \qquad \text{[Eq 17]}$$

The modal story lateral forces were summed to obtain the story shears for each mode. Subsequently, the story shears and deflections in the hospital were obtained by taking the square root of the sum of the squares of the modal values and multiplying the computed total displacements by the allowable ductility factor, C_d, which equals 1.5 for the MPE.

In the case of Letterman Hospital, the MPE governed the analysis. The resulting story shears and lateral deflections for each of the principal directions associated with the acceleration levels of 0.4 g and 0.5 g are tabulated in Tables 7 and 8, respectively. Curves D and E in Figures 10, 11, and 12 represent the respective story shears and lateral deflections.

AA Time History Analysis

To compute the two-dimensional response of Letterman Hospital, AA used the time history modal analysis method in each of the principal directions. The multi-degree-of-freedom system was separated into an array of single-degree-of-freedom systems, each having a unique natural frequency, mode shape, and modal participation factor. Each single-degree-of-freedom system was subjected to the Type A-1 artificial earthquake record, and the response time history of each mode was computed. The responses of each mode at discrete time intervals were summed, with proper attention to mathematical signs to obtain a time history of the structure's response. The maximum story shears and lateral deflections obtained from the AA analysis are tabulated in Table 9 and shown as Curve G in Figures 10, 11, and 12 for the N-S and E-W directions, respectively.

CERL Response Spectrum Analysis

To compute the three-dimensional response of Letterman Hospital, CERL used the dynamic model assembled by the TABS computer program and the smoothed response spectrum for the Type A-1 and A-2 artificial earthquake records scaled to 0.5 g peak horizontal ground acceleration. The dynamic response of

Table 7

TM 5-809-10 Appendix Story Shears and Lateral Deflections, A_{mpe} = 0.4 g (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.472		0.529
	4944		4772	
10		0.414		0.464
	7843		7608	
9		0.353		0.394
	9754		9520	
8		0.290		0.321
	11046		10832	
7		0.224		0.248
	12063		11819	
6		0.165		0.171
	13366		12894	
5		0.111		0.129
	14684		14172	
4		0.065		0.090
	21003		20795	
3		0.035		0.045
	25501		24989	
2		0.010		0.016
	26637		26258	
1		0		0

Table 8

TM 5-809-10 Appendix Story Shears and Lateral Deflections, A_{mpe} = 0.5 g (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.590		0.662
	6180		5965	
10		0.518		0.579
	9804		9510	
9		0.441		0.492
	12192		11900	
8		0.362		0.401
	13808		13540	
7		0.280		0.311
	15079		14774	
6		0.206		0.213
	16708		16118	
5		0.139		0.162
	18354		17714	
4		0.081		0.112
	26254		25995	
3		0.043		0.056
	31876		31237	
2		0.012		0.020
	33296		32823	
1		0		0

Letterman Hospital was obtained by taking the square root of the sum of the squares of the maximum responses of the structures' first 12 vibration modes. These 12 modes included the first four modes in each of the two principal directions and the four torsional modes. The story shears and lateral deflections in the N-S and E-W directions are presented in Table 10 and plotted as Curve F in Figures 10, 11, and 12, respectively.

PMB Response Spectrum Analysis

To compute the three-dimensional response of Letterman Hospital, PMB used a computer model that incorporated a "dummy story" to approximate the behavior of the caisson foundation system and a "dummy story" to approximate the behavior of the flexible fourth floor diaphragm. The design earthquake response spectrum was developed by Dames and Moore on the basis of a peak acceleration of 0.35 g. The dynamic response of Letterman Hospital was obtained by taking the square root of the sum of the squares of the maximum response of the structure's first eight vibration modes. The resulting story shears and lateral deflections are tabulated in Table 11 and presented as Curve C in Figures 10, 11, and 12, respectively.

Story Shear and Lateral Deflection Comparisons

To provide a more consistent comparison of the 1978 ATC-3, TM 5-809-10 Appendix, and PMB story shears and lateral deflections, the story shears and lateral deflections for the 1978 ATC-3 and TM 5-809-10 Appendix design provisions were recomputed using 0.35 g as the design acceleration level. The resulting story shears and lateral deflections are tabulated in Tables 12 and 13 and presented in Figures 13, 14, and 15 for each of the principal directions.

Generally, the TM 5-809-10 Appendix story shears and the PMB story shears are in excellent agreement, while the 1978 ATC-3 story shears are substantially smaller. With regard to lateral deflections, the 1978 ATC-3 and TM 5-809-10 Appendix provisions produce comparable results in the N-S direction; however, the PMB lateral deflections differ by a factor of nearly three at the fourth-floor level. This discrepancy can be explained in part by the modeling assumptions used in the PMB analysis. Both the first and fourth floor were

Table 9

AA Time History Story Shears and Lateral Deflections (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.480		0.419
	7500		7300	
10		0.427		0.362
	11600		11500	
9		0.370		0.297
	17200		14100	
8		0.309		0.244
	20500		15100	
7		0.254		0.191
	23100		15900	
6		0.195		0.138
	24900		15900	
5		0.150		0.081
	25900		22600	
4		0.098		0.033
	31000		30700	
3		0.037		0.016
	37600		32600	
2		0.004		0
	39000		35000	
1		0		0

Table 10

CERL Response Spectrum Story Shears and Lateral Deflections (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.454		0.487
	6000		7000	
10		0.398		0.422
	11500		11300	
9		0.340		0.353
	14100		14000	
8		0.279		0.283
	16000		16000	
7		0.218		0.216
	17300		17000	
6		0.159		0.159
	19100		20500	
5		0.108		0.126
	21200		21000	
4		0.063		0.090
	37500		32000	
3		0.033		0.047
	38900		38500	
2		0.010		0.016
	39000		39500	
1		0		0

modeled using "dummy stories" to more realistically model the structures; thus, the larger deflections are to be expected. However, it also appears that the PMB model of the tower section is somewhat stiffer because the tower deflections are approximately two-thirds of those from the other analyses. In the E-W direction, there are two instances of disparity. Below the third-floor level, the lateral deflections computed with the 1978 ATC-3 provisions are small, yet significantly different from the other two analyses. In the upper-story levels of the tower, the PMB model also appears to be stiffer because the deflections are less.

Some general conclusions are apparent from the story shear plots shown in Figures 10 and 11. The 1964 UBC provisions produce equivalent yield stress basis story shears which are approximately 10 to 35 percent of the story shears obtained in AA's time history modal analysis.

When the dynamic characteristics of the structure are considered in determining the total lateral forces and their distributions, the 1975 SEAOC provisions produce equivalent yield stress basis story shears which are in reasonable agreement with the story shears obtained in AA's time history modal analysis except in the base structure and the intermediate stories in the tower in the E-W direction. The 1978 ATC-3 modal analysis procedure produces story shears which are generally approximately 100 percent greater than the 1964 UBC equivalent yield stress basis story shears in the tower and approximately 10 percent less than the 1964 UBC equivalent yield stress basis story shears in the base structure. When comparable acceleration levels are specified, the TM 5-809-10 Appendix provisions produce story shears which are in reasonable agreement with the story shears from AA's time history analysis except for the lower stories in the N-S direction where they differ by approximately 20 percent. Furthermore, the comparison of the 1978 ATC-3 and TM 5-809-10 Appendix provisions and the PMB analysis reveals that when comparable acceleration levels are specified the story shears for the 1978 ATC-3 provisions are approximately 30 percent of the story shears obtained in the TM 5-809-10 Appendix provisions and the PMB analysis. Considering the complexity of the structure and

Table 11

PMB Response Spectrum Story Shears and Lateral Deflections (Metric conversion factors: 1 kip = 4.4 kN; 1 ft = 0.3 m)

	N-S Direction		E-W Direction	
Level	Story Shear kip	Deflection ft	Story Shear kip	Deflection ft
Roof		0.386		0.315
	3700		3500	
10		0.341		0.287
	6100		5700	
9		0.305		0.262
	8800		7700	
8		0.264		0.232
	9600		9200	
7		0.224		0.191
	10600		10100	
6		0.199		0.150
	12400		10900	
5		0.175		0.106
	13300		11600	
4		0.146		0.069
	17300		16700	
3		0.057		0.046
	19400		20900	
2		0.028		0.023
	21000		23400	
1		0		0

Table 12

1978 ATC-3, TM 5-809-10 Appendix and PMB Story Shears (Metric conversion factor: 1 kip = 4.4 kN)

	N-S Direction Story Shear			E-W Direction Story Shear		
Level	ATC kip	TM Appendix kip	PMB kip	ATC kip	TM Appendix kip	PMB kip
Roof						
	1442	4326	3487	1392	4175	3673
10						
	2288	6863	5743	2219	6657	6122
9						
	2845	8535	7692	2777	8330	8775
8						
	3222	9666	9230	3159	9478	9591
7						
	3518	10555	10051	3447	10342	10612
6						
	3897	11696	10871	3761	11283	12449
5						
	4283	12848	11589	4133	12400	13265
4						
	6126	18378	16717	6066	18196	17346
3						
	7438	22313	20923	7288	21866	19387
2						
	7769	23307	23282	7659	22976	21020
1						

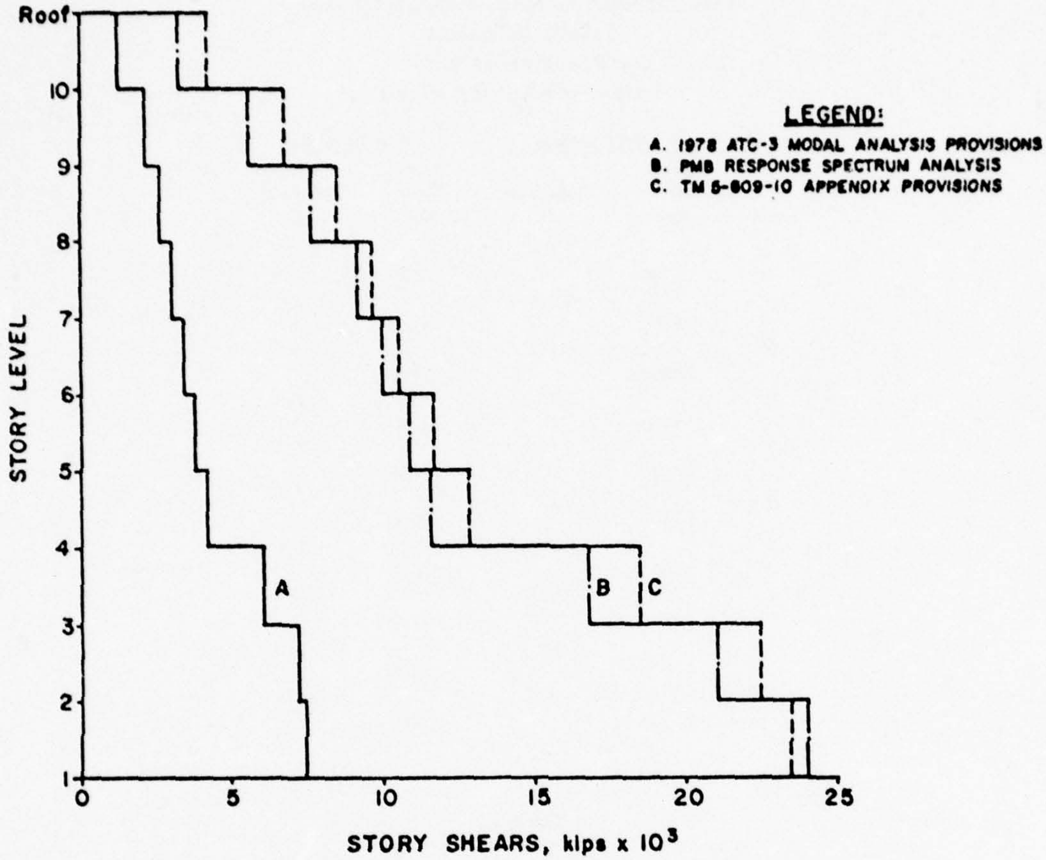


Figure 13. Letterman Hospital upgrade story shears (yield stress basis), N-S direction.

the numerous and different assumptions that were employed, the agreement between the story shears for the TM 5-809-10 Appendix provisions and the PMB analysis is excellent.

Generally, the lateral deflections in the N-S direction are in better agreement than the lateral deflections in the E-W direction, with the exception that in both directions, the lateral deflections from the original design are quite small and bear little resemblance to the lateral deflections obtained by the other analyses. Some of the discrepancy among the curves is due directly to the different force levels that were applied to the structure. For example, Curves D and E provide an indication of the effect that a 25 percent increase in the lateral forces has on the lateral deflection of the structure. However, some of the discrepancy among the curves is also due to the coefficient, C_d. For example, the 1978 ATC-3 provisions yield story shears that are significantly less than the other provisions, yet the ATC-3 *lateral deflections are in good agreement with* many of the curves from the other analyses.

In the N-S direction, the lateral deflections from the PMB analysis clearly reflect the effect of modeling the foundation system and the fourth floor diaphragm using "dummy stories"; however, it is also apparent that the PMB model is somewhat stiffer in the tower sections than in the other models. These effects are more clearly illustrated in Figure 15, where comparable acceleration levels were used among the 1978 ATC-3 and TM 5-809-10 Appendix provisions and the PMB analysis. Likewise, in the E-W directions, when comparable acceleration levels are used among the 1978 ATC-3 and TM 5-809-10 Appendix provisions and the PMB analysis, the lateral deflections are in quite good agreement, except in the upper stories of the tower section.

Table 13

1978 ATC-3, TM 5-809-10 Appendix and PMB Lateral Deflections
(Metric conversion factor: 1 ft = 0.3 m)

	N-S Direction			E-W Direction		
Level	ATC ft	TM Appendix ft	PMB ft	ATC ft	TM Appendix ft	PMB ft
Roof	0.367	0.413	0.386	0.412	0.463	0.315
10	0.322	0.362	0.341	0.360	0.406	0.287
9	0.275	0.309	0.305	0.306	0.344	0.262
8	0.225	0.254	0.264	0.250	0.281	0.232
7	0.174	0.196	0.224	0.193	0.217	0.191
6	0.128	0.144	0.199	0.133	0.149	0.150
5	0.087	0.097	0.175	0.101	0.113	0.106
4	0.050	0.056	0.146	0.070	0.078	0.069
3	0.027	0.030	0.057	0.035	0.039	0.046
2	0.008	0.009	0.028	0.012	0.014	0.023
1	0	0	0	0	0	0

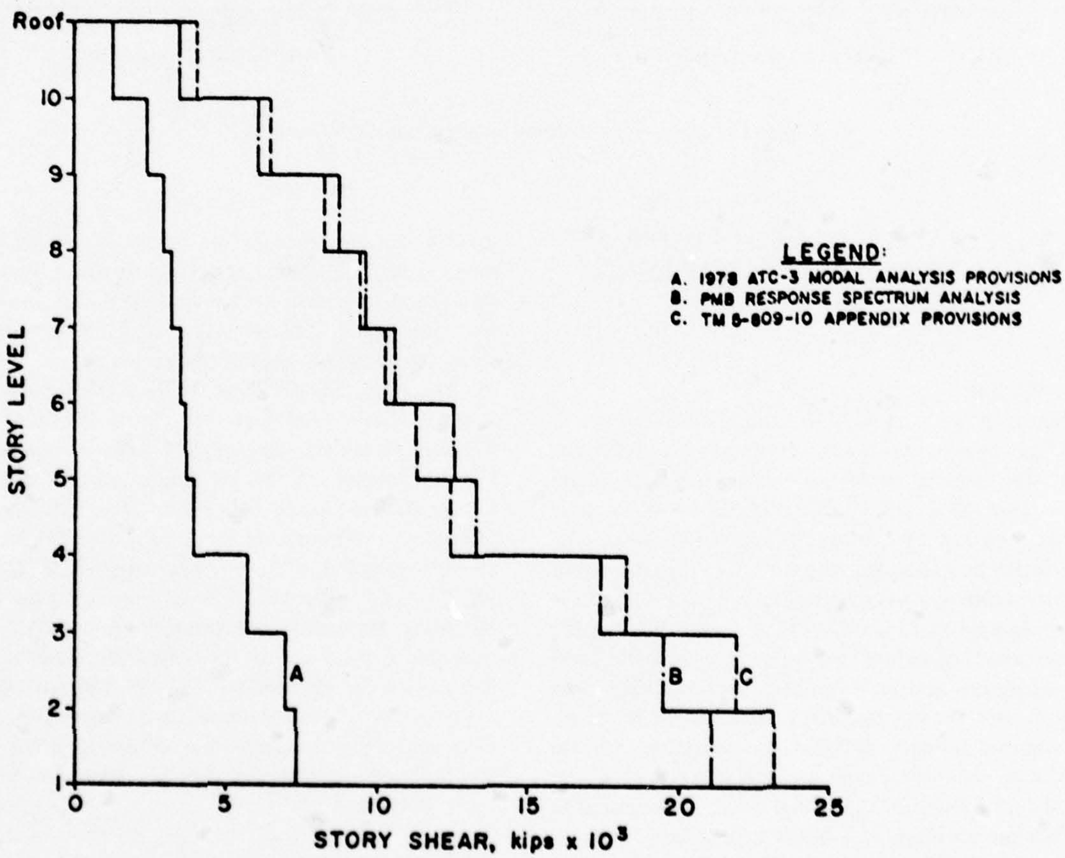


Figure 14. Letterman Hospital upgrade story shears (yield stress basis), E-W direction.

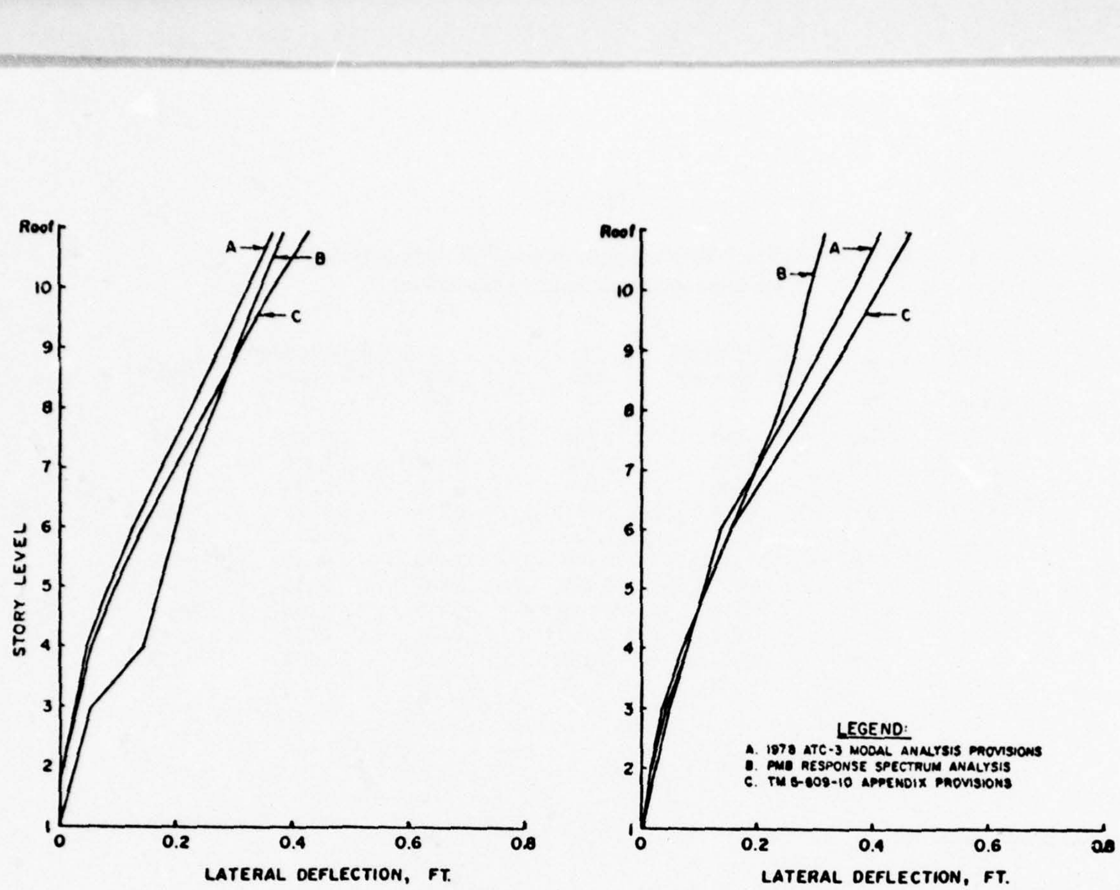


Figure 15. Letterman Hospital upgrade lateral deflections.

3 THREE-STORY, MOMENT RESISTANT STEEL FRAME BUILDING DESIGNS

Introduction

Since a major portion of the military's investment in building construction is associated with low-rise buildings (less than six stories), it was deemed appropriate to compare the magnitude and distribution of the story shears provided by current and tentative design provisions for this class of construction. In the past, many of the techniques for seismic design and analysis have not been applied to this class of structures because the consequences of failure have been considered small and the additional design costs have been thought to be large relative to the building's value. However, nearly all essential military facilities are housed in low-rise buildings; therefore, some consideration must be devoted to the behavior of low-rise buildings subjected to earthquake motions, and to the application of simplified analytical procedures to the design of low-rise structures to insure that they remain functional following a major earthquake.

This section discusses the design of a three-story, moment resistant steel frame building which houses an emergency operation center vital to the installation's post-earthquake operation; the designs (in the transverse direction) are in accordance with the provisions of the 1975 SEAOC, the 1978 ATC-3 Equivalent Lateral Force Procedure, the 1978 ATC-3 Modal Analysis Procedure, and the TM 5-809-10 Appendix. For this comparison, the building is assumed to be on an installation located in a region of sufficiently high seismicity to require that Z = 1.0 for the 1975 SEAOC provisions, and $A_a = A_v = 0.4$ for the 1978 ATC-3 and $A_{mpe} = 0.4$ for the TM 5-809-10 Appendix provisions. Moreover, the soil characteristics at the installation are such that S = 1.5 for the 1975 SEAOC provisions and S = 1.2 for the 1978 ATC-3 and TM 5-809-10 Appendix provisions. The designs are then compared in terms of magnitude and distribution of the resulting story shears, lateral deflections, member sizes, and typical frame weights.

Building Description

The three-story building has the structural configuration, dimensions, and weights shown in Figure 16.

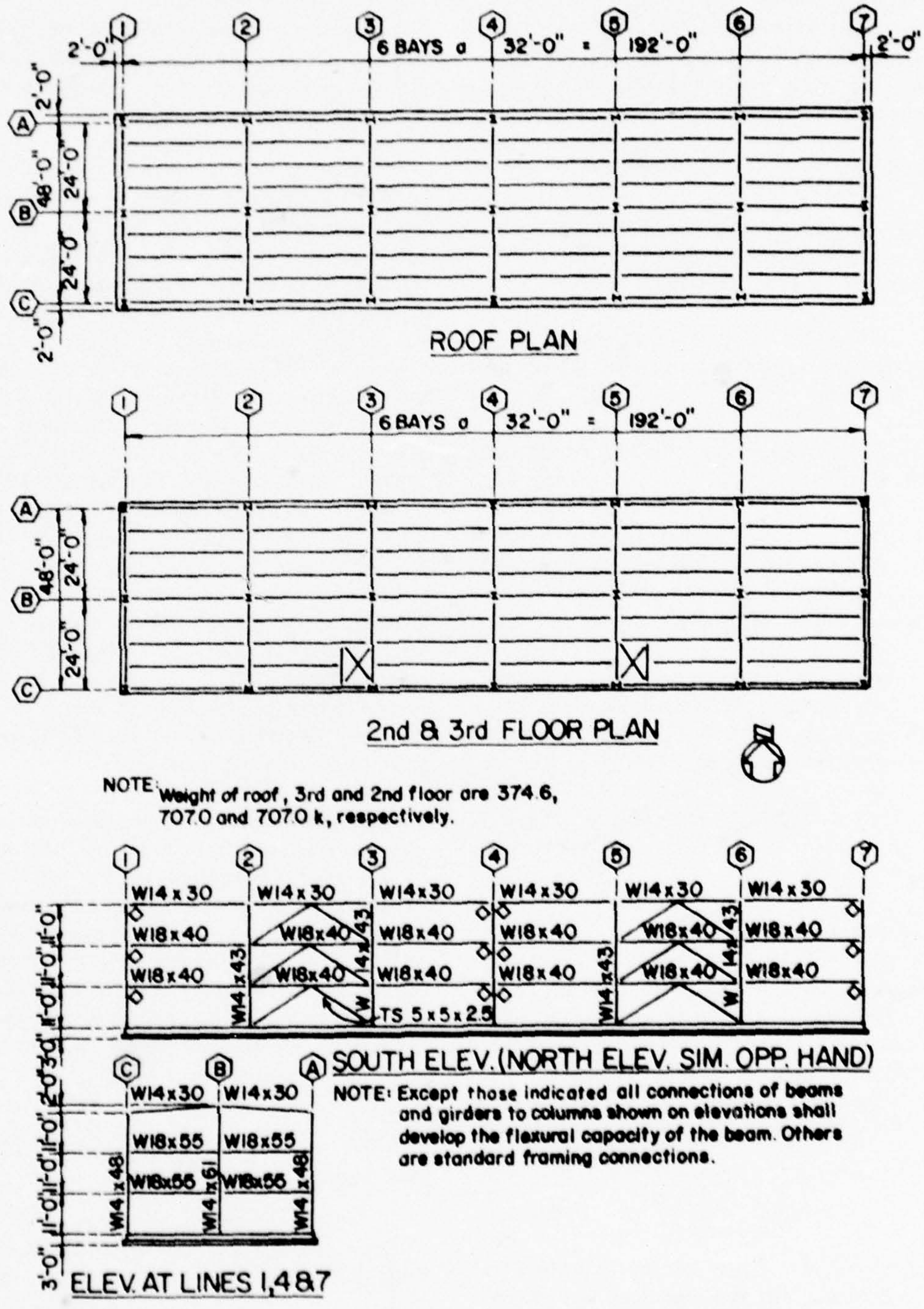


Figure 16. Structural configuration of building.
(Metric conversion factors: 1 ft = 0.3 m and 1 kip = 4.4 kN.)

The building's lateral force resisting system consists of steel ductile moment resisting space frames on column lines 1, 4, and 7 in the transverse direction and steel-braced frames on column lines A and C in the longitudinal direction. All other frames are vertical, load-carrying frames which do not contribute to the building's total lateral resistance. In addition, the following assumptions were made:

1. All columns are fixed at the foundation.

2. Exterior walls are nonbearing, nonshear, flexible, insulated metal panels which do not contribute to the structural system's lateral resistance.

3. Interior walls are nonbearing, nonshear, removable, drywall construction and are isolated from the structure.

4. Stairways are isolated from the structure and do not transfer shear forces.

5. The metal deck roof system forms a flexible diaphragm; therefore, the seismic load is distributed to the lateral force resisting frames in proportion to tributary area rather than stiffness.

6. The metal deck with concrete fill floor system forms a rigid diaphragm; therefore, the seismic loads are distributed to the lateral force resisting frames in proportion to the frame stiffness.

7. The flexural rigidities of the diaphragms are not great enough to contribute significantly to the flexural rigidity of the girders.

1975 SEAOC Provisions

A base shear of 215.7 kip was used to calculate the design forces for the transverse direction of the three-story, moment resistant steel frame building. The base shear was computed from Eq 4, taking $Z = 1.0$, $I = 1.5$, $K = 0.67$, $C = 0.080$, $S = 1.5$, and $W = 1788.6$ kip (7.9 MN). In computing the coefficient C, the building's period in the transverse direction was estimated from the formula

$$T \simeq 0.115 \left(\frac{H}{ZIS}\right)^{2/3} \qquad \text{[Eq 18]}$$

Taking $Z = 1.0$, $I = 1.5$, $S = 1.5$, $H = 34$ ft (10.4 m), yields $T = 0.70$ sec. The basis for using this formula rather than the conventional formula

$$T = 0.1\,N \qquad \text{[Eq 19]}$$

lies in the fact that for very flexible buildings, such as moment resistant frames, the maximum drift limitations may govern the seismic design rather than the minimum force requirements.

The base shear was distributed over the building height in accordance with Eq 3, and the resulting story shears were computed following standard procedures. Table 14 summarizes these calculations.

The story shears computed in Table 14 were distributed to the vertical elements of the moment frames on column lines 1, 4, and 7 in proportion to their contribution to the story's lateral stiffness and considering the stiffness of the diaphragm. Since the roof diaphragm is very flexible in its own plane relative to the vertical components, the story shears were distributed to the vertical elements in proportion to their tributary area. Consequently, the shears distributed to the vertical elements of the frames on column lines 1, 4, and 7 at the roof level were 19.0, 38.1, and 19.0 kip (84.5, 169.5, and 84.5 kN), respectively. The floor diaphragms were assumed to be infinitely rigid; thus, the story shears are distributed among the moment frames in proportion to their contribution to the story's lateral stiffness. Since the frames are identical in stiffness, the shear transferred to each frame is identical and equal to one-third of the story shear. Therefore, the lateral shear forces distributed to the vertical elements of the frames at the third-floor and second-floor levels are 56.4 and 71.9 kip (250.9 and 319.8 kN), respectively.

Since the building is symmetrical, the centers of mass and resistance coincide and the torsional moment to be considered in the design of the elements is caused by accidental torsion, i.e., the story shear times 5 percent of the dimension of the building in the story under consideration perpendicular to the direction of the applied earthquake force. Using the story shears computed in Table 14 and 5 percent of the length of the building, the accidental torsional moments at each level of the building were:

Roof: $M_{ta} = (76.1)\,(0.05)\,(192) = 730.6$ ft-kip (990.5 km-N)

Third Floor: $M_{ta} = (169.1)(0.05)(192) = 1623.4$ ft-kip (2201.0 km-N)

Second Floor: $M_{ta} = (215.7)(0.05)(192) = 2070.7$ ft-kip (2807.5 km-N)

Table 14

Vertical Distribution of Lateral Forces and Story Shears for 1975 SEAOC Provisions
(Metric conversion factors: 1 ft = 0.3 m and 1 kip = 4.4 kN)

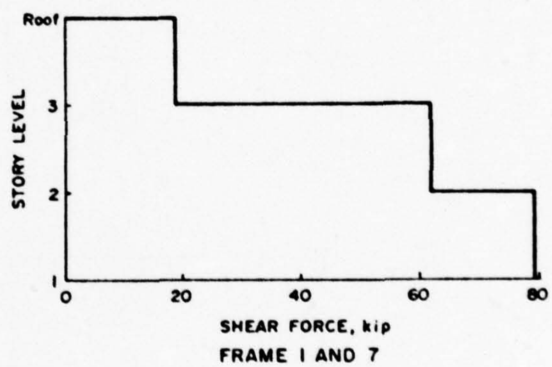
Level	w_x kips	h_x ft	$w_x h_x$ ft-kips	$\frac{w_x h_x}{\Sigma W_x h_x}$	F_x kips	V_x kips
Roof	374.6	34	12736.4	0.353	76.1	76.1
Third Floor	707.0	22	15554.0	0.431	93.0	169.1
Second Floor	707.0	11	7777.0	0.216	46.6	215.7
First Floor		0	0			
			Σ36067.4	1.000	215.7	

The accidental torsional moments produce additional shears in the vertical elements of the frames on column lines 1, 4, 7, A, and C. The effects of the accidental torsional moments are distributed to the vertical elements in proportion to their contribution to the story's torsional stiffness and considering the diaphragm's stiffness. Since the roof diaphragm is very flexible, each frame acts almost independently and the effects of accidental torsion can be ignored. At the third-floor level, the additional shear in the vertical elements of the frames on column lines 1 and 7 was computed to be ±5.3 kip (23.6 kN). Likewise, at the second-floor level, the additional shear produced in these frames was ±7.6 kip (33.8 kN). The additional shear force due to torsion must be added to the previously computed lateral shear forces for the vertical elements on column lines 1 and 7 to obtain the total shear forces for design. These total shear forces for the vertical elements on column lines 1, 4, and 7 are summarized in Figure 17.

The lateral forces applied to each frame were determined from the total shear forces. Figure 18 shows these lateral forces, together with the dead and reduced live loads. Each frame was analyzed for the loading condition:

$$U = 1.0D + 1.0L + 1.0E \qquad \text{(Eq 20)}$$

These analyses were performed using the TABS 77 computer program.[22] Figure 19 summarizes the results. Moments, axial forces, and shears are tabulated for each column, while only the moments are tabulated for the girders.



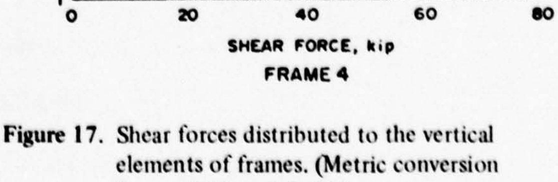


Figure 17. Shear forces distributed to the vertical elements of frames. (Metric conversion factor: 1 kip = 4.4 kN.)

[22] A. Habibullah, E. Wilson, and H. Dewey, *TABS 77–Three-Dimensional Analysis of Building Systems*, Computer Program, EERC-72-8 (Computer Structures International, December 1972, revised April 1979).

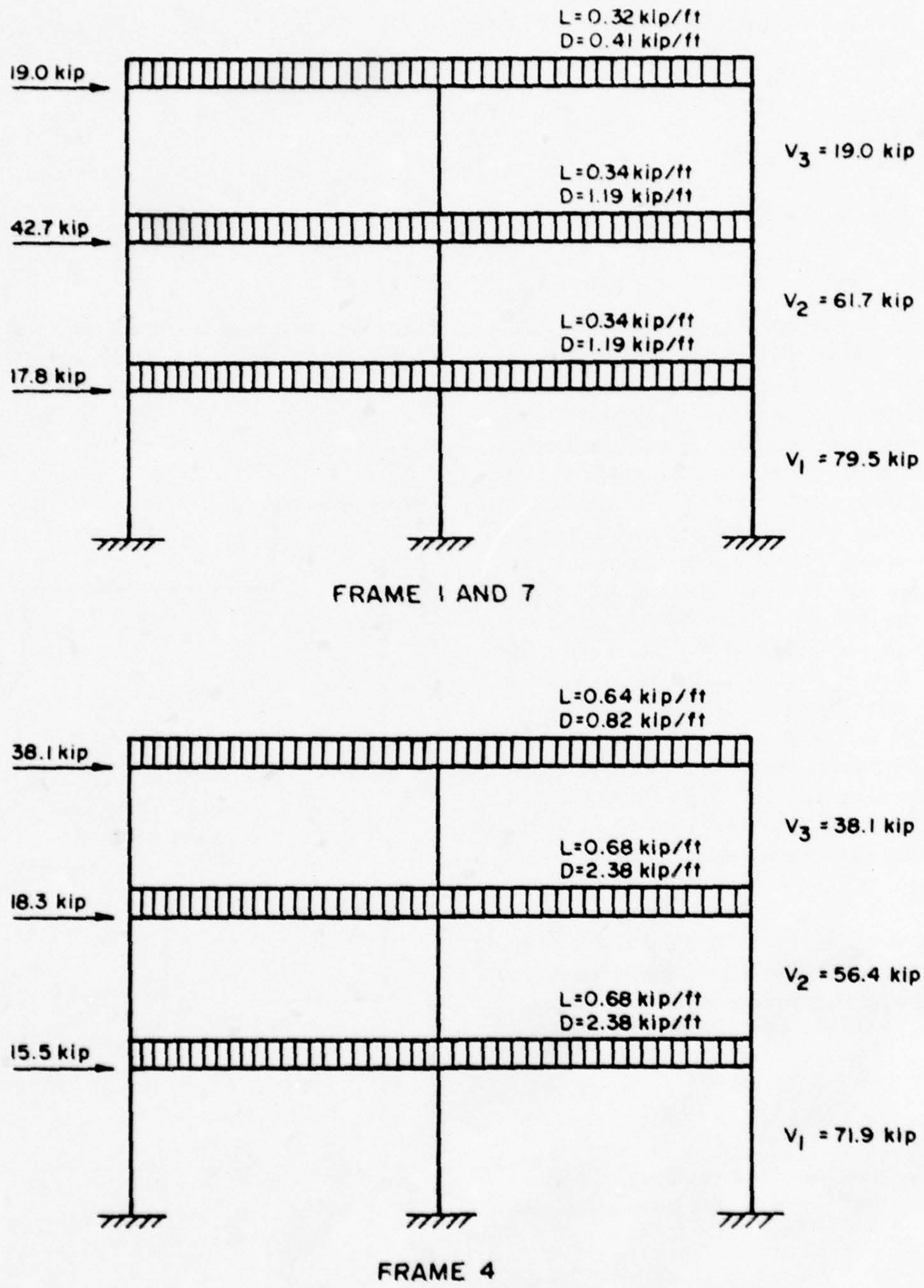
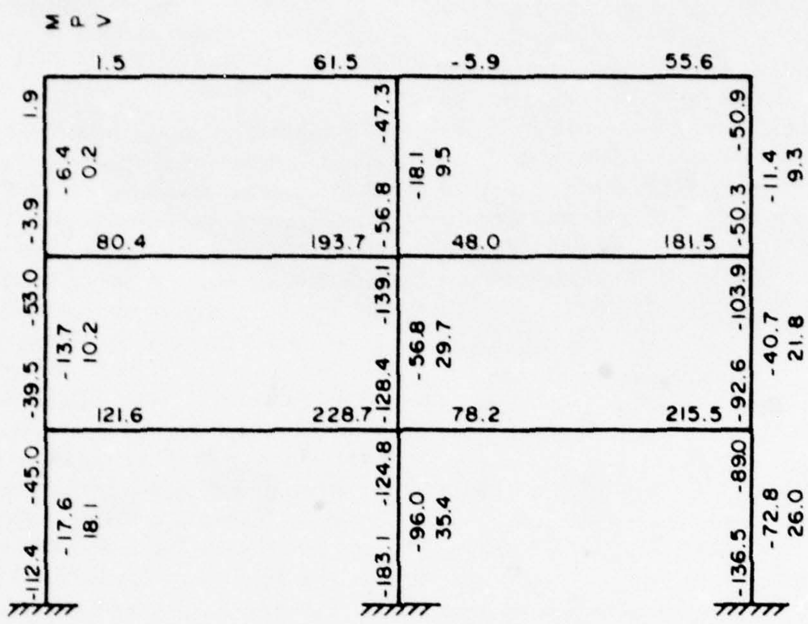
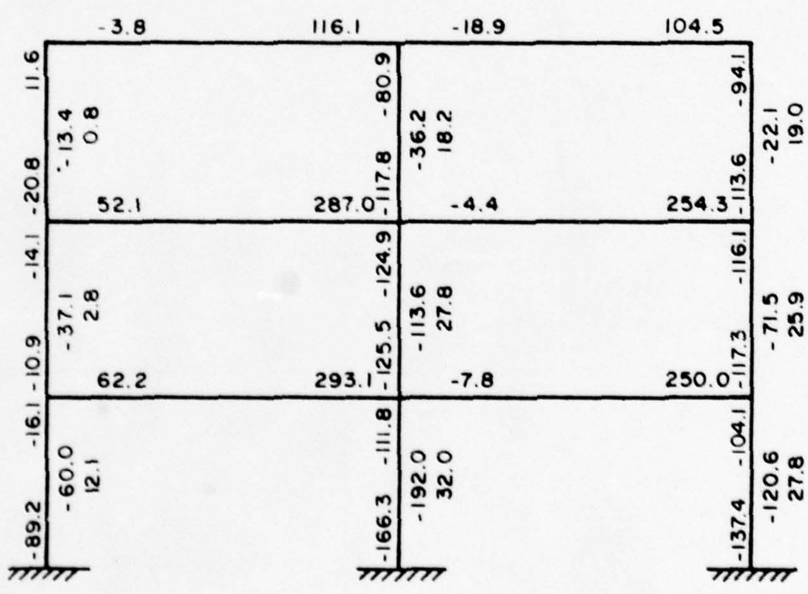


Figure 18. Frame loadings for 1975 SEAOC provisions.
(Metric conversion factors: 1 ft = 0.3 m and 1 kip = 4.4 kN.)



FRAME 1 + 7: 1.0D + 1.0L + 1.0E



FRAME 4: 1.0D + 1.0L + 1.0E

Figure 19. Frame member moments and forces for 1975 SEAOC provisions. Moments are in ft-kip and forces are in kips. (Metric conversion factors: 1 kip = 4.4 kN and 1 ft-kip = 1.4 km-N.)

Member sizes were proportioned according to AISC specifications, using type A36 steel with a working stress of 24 ksi (165.5 MPa) and a modulus of elasticity of 29,000 ksi (199.9 GPa). Prior to proportioning the members, the moments and forces were divided by the factor 1.33 to account for the allowable one-third increase in working stresses for earthquake loads. The moments, axial forces, and shears induced in the frame on column line 4 governed the design. Figure 20 summarizes the final member sizes. In addition, the story drifts for the building were computed and found to comply with the allowable drift criterion of 0.005 of the story height.

Since story shears are a fundamental measure of a structure's lateral resistance, the story shears computed in Table 14 are plotted as Curve A in Figure 21. However, the story shears in Table 14 were multiplied by the factor 1.7/1.33 = 1.28 to transfer them to a yield stress basis that would provide a consistent basis for comparison with the ATC-3 provisions. In addition, the lateral deflections for the building are plotted as Curve A in Figure 22. These lateral deflections represent the displacement calculated from the application of the required lateral forces multiplied by the factor 1/K = 1/0.67 = 1.33.

1978 ATC-3 Provisions:
Equivalent Lateral Force Procedure

Since the building being designed is located in a Seismic Index 4 region ($A_a = A_v = 0.4$) and is considered to be an essential facility, the 1978 ATC-3 provisions specify that the lateral-force-resisting frames on column lines 1, 4, and 7 must be designed as special moment frames using R = 8 and C_d = 5.5. A base shear of 164.6 kip (732.2 kN) was used to calculate the design forces for the frame in the transverse direction. The magnitude of the base shear was computed from the formula

$$V = CW \qquad \text{[Eq 21]}$$

taking C = 0.092 and W = 1788.6 kip (7.9 MN). The period of the structure was assumed to be 0.70 sec, the same as for the 1975 SEAOC provisions.

The base shear was distributed over the height of the building according to the formula

$$F_x = C_{vx} V \qquad \text{[Eq 22]}$$

where

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{[Eq 23]}$$

In the above expression, k is an exponent related to the period of the building and was taken to be 1.10 for the building under consideration. Table 15 summarizes the computations of the lateral shear force, F_x, applied at each level and the resulting story shears. Since the results of the previous SEAOC design indicated that the member moments and forces for the frame on column line 4 governed the design, only the loadings imposed on this frame were determined. The story shears in Table 15 were distributed to the vertical elements of the moment frame on column line 4, in accordance with the procedure previously discussed (p 34). Subsequently, the lateral forces applied to the frame on column line 4 were determined from the total shear forces. These lateral forces, together with the dead and reduced live loads, are shown in Figure 23. The ATC-3 provisions require that the frame be analyzed for two loading conditions.

$$\begin{aligned} U &= 1.2D + 1.0L + 1.0E \\ U &= 0.8D + 1.0E \end{aligned} \qquad \text{[Eq 24]}$$

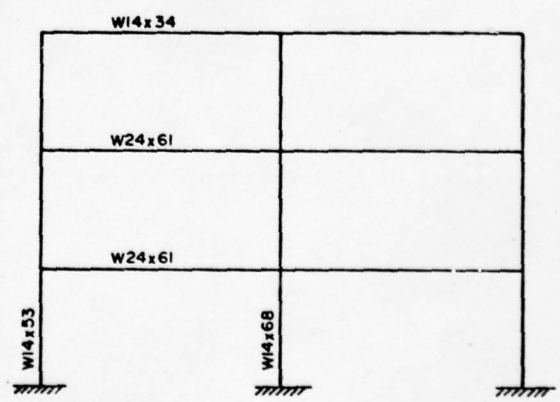


Figure 20. Frame member sizes for 1975 SEAOC provisions.

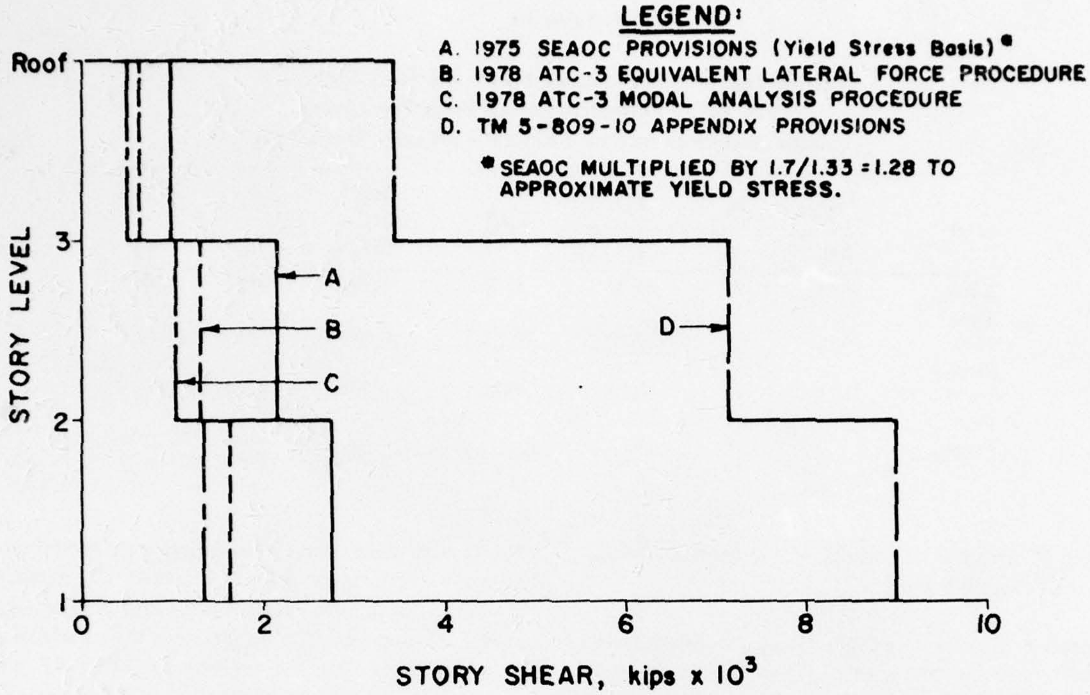


Figure 21. Story shears for ductile steel frame building, transverse direction. (Metric conversion factor: 1 kip = 4.4 kN.)

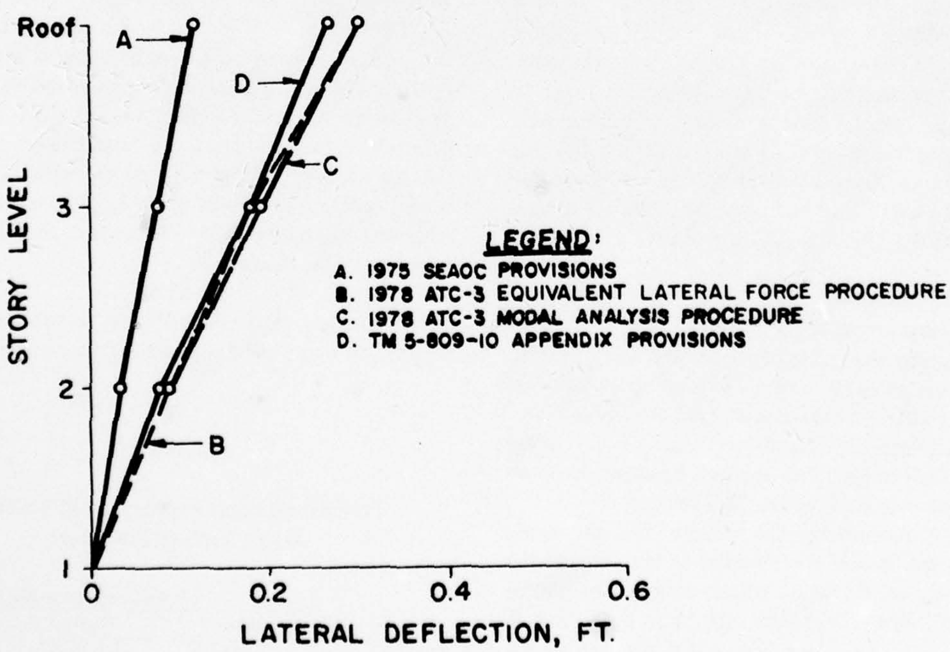


Figure 22. Lateral deflections of ductile steel frame building, transverse direction. (Metric conversion factor: 1 kip = 4.4 kN.)

Table 15

Vertical Distribution of Lateral Forces and Story Shears for 1978 ATC-3 Equivalent Lateral Force Procedure (Metric conversion factors: 1 kip = 4.4 kN and 1 ft = 0.3 m)

Level	w_x kip	h_x ft	h_x^k ft	$w_x h_x^k$	$\frac{w_x h_x^k}{\sum w_x h_x^k}$	F_x kip	V_x kip
Roof	374.6	34	48.38	18121.5	0.368	60.6	60.6
Third Floor	707.0	22	29.97	21187.3	0.431	70.9	131.5
Second Floor	707.0	11	13.98	9884.4	0.201	33.1	164.6
First Floor		0	0				
				Σ49193.6	1.000	164.6	

Figure 24 presents the results of the analyses for the frame on column line 4.

Member sizes were proportioned according to the ATC-3 provisions and the AISC specifications, using type A36 steel with a yield stress of 36 ksi (248.2 MPa) and a modulus of elasticity of 29,000 ksi (199.9 GPa). To enable the direct use of the existing AISC specifications, the controlling moments and forces in Figure 24 were divided by the factor (0.9) (1.7) = 1.53, i.e., the capacity reduction factor times the 1.7 allowable stress increase factor, prior to proportioning the member. Figure 25 summarizes the final member sizes. In addition, the lateral deflections for the building were checked against the required story drift criteria. The story shears and lateral deflections are also presented in Figure 21 and 22, respectively, as Curve B for comparison with the 1975 SEAOC provisions.

1978 ATC-3 Provisions: Modal Analysis Procedure

The member sizes obtained from the 1975 SEAOC design of the building were used as the starting point for the 1978 ATC-3 modal analysis procedures although additional iterations were necessary to achieve a final design. This section, however, summarizes only the properties of the final design.

Table 16 summarizes the periods and the mode shapes for the lowest three modes of vibration in the transverse direction for the final design. The modal base shears were computed using Eqs 6, 7, and 8. Taking $A_a = A_v = 0.4$, $S = 1.2$, and $R = 8.0$, and using the periods and mode shapes in Table 16, the modal base shears were computed to be 129.7, 29.5, and 11.2 kip (576.9, 131.2, and 49.8 kN), respectively. These modal base shears were used to compute modal story forces in accordance with Eqs 9 and 10. Table 17 provides the modal story forces and the resulting modal story shears. The design story shears which are summarized in the last column of Table 17 were obtained by taking the square root of the sum of the squares of the modal story shears.

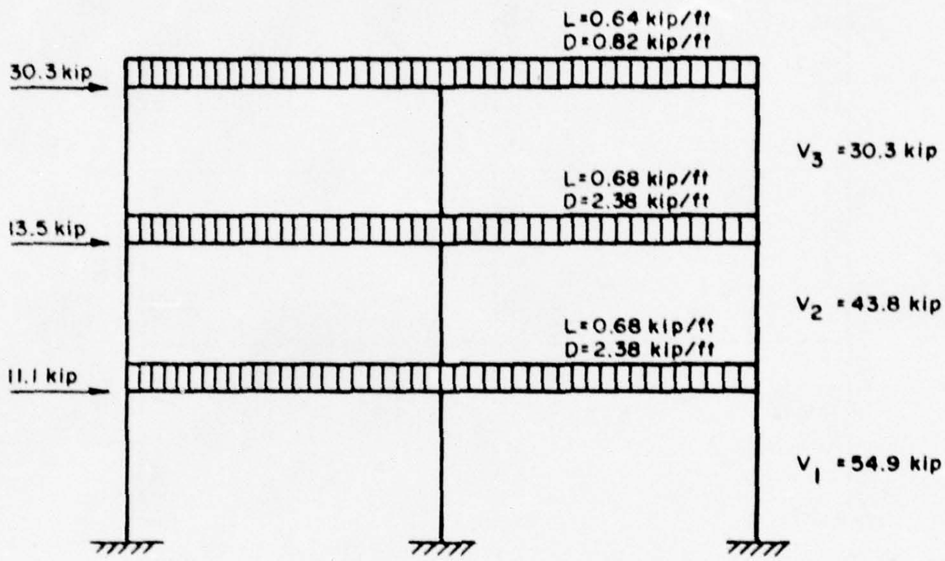
The modal deflections were computed in accordance with Eq 11 and 12 and are summarized in Table 18. The modal story drifts, which were computed as the difference between the deflections at the top and bottom of the story, are also summarized in Table 18. The design values for deflection and story drift were determined by combining the modal values by taking the square root of the sum of the squares for each modal value. The design story drifts were compared with the allowable story drifts, and all values were found to be satisfactory.

The design story shears were distributed to the vertical elements of the moment frame on column line

Table 16

Periods and Mode Shapes for 1978 ATC-3 Modal Analysis Procedure

Level	Period and Mode Shape: Mode 1 T = 0.732	Mode 2 T = 0.287	Mode 3 T = 0.153
Roof	0.2113	0.1940	0.0606
Third Floor	0.1354	-0.1087	-0.1240
Second Floor	0.0596	-0.1174	0.1680



FRAME 4

Figure 23. Frame loadings for 1978 ATC-3 equivalent lateral force procedure. (Metric conversion factors: 1 ft = 0.3 m and 1 kip = 4.4 kN.)

4 in accordance with the procedures previously discussed (p 34). Subsequently, the lateral forces applied to the frame on column line 4 were determined from the total shear forces. These lateral forces, together with the dead and reduced live loads, are shown in Figure 26. The frame was analyzed for two loading conditions as shown in Eq 24. Figure 27 summarizes the results of these analyses.

Member sizes were proportioned following the same procedures used for the ATC-3 equivalent lateral force procedure. Figure 28 summarizes the final member sizes.

The story shears and lateral deflections are plotted as Curve C in Figure 21 and 22, respectively, for comparison with the values from the SEAOC and ATC-3 equivalent lateral force procedure.

TM 5-809-10 Appendix Provisions

The member sizes obtained from the 1975 SEAOC design of the building were also used as the starting point for the TM 5-809-10 Appendix design. Additional iterations were necessary to achieve a final design. This section summarizes only the properties of the final design.

Table 19 summarizes the periods and mode shapes for the lowest three modes of vibration in the transverse direction. Eq 13 was used to compute modal participation factors which were determined to be 0.411, 0.321, and 0.268, respectively. The values of the seismic design coefficient were determined from Eqs 14 and 15. Taking A_{mpe} = 0.4 g, S = 1.2, and R = 1.5, and the appropriate modal period, the seismic design coefficients were computed to be 0.600, 0.667, and 0.667. The latter two values were limited by the requirement that the seismic design coefficient need not exceed 2.5 A/R = 0.667 for the case at hand.

The modal story forces associated with the MPE were determined from Eq 16 and are summarized in Table 20, together with the resulting modal story shears and the design story shears obtained by taking the square root of the sum of the squares of the modal values.

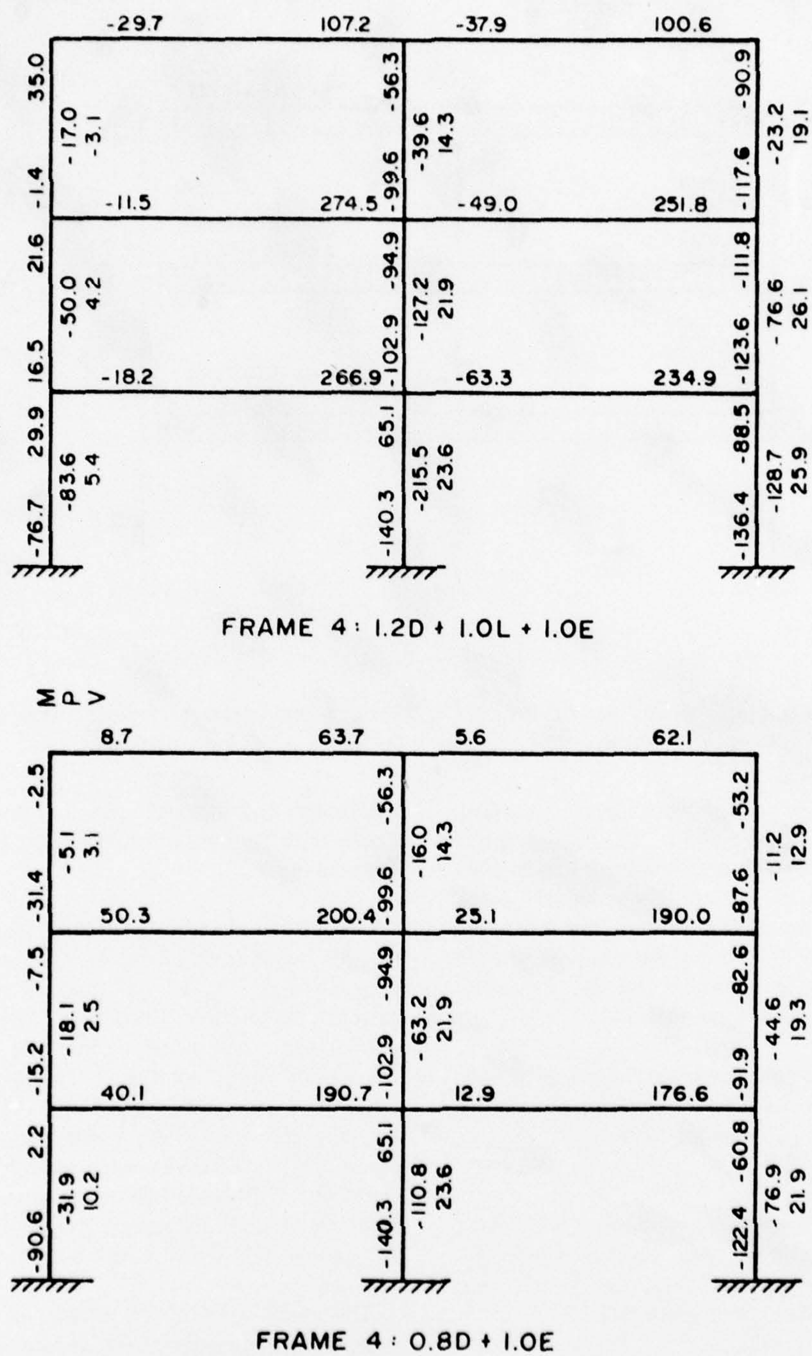


Figure 24. Frame 4 member moments and forces for 1978 ATC-3 equivalent lateral force procedure. Moments are in ft-kips and forces are in kips. (Metric conversion factors: 1 kip = 4.4 kN and 1 ft-kip = 1.4 km-N.)

Table 17

Vertical Distribution of Lateral Forces and Story Shears for 1978 ATC-3 Modal Analysis Procedure (Metric conversion factor: 1 kip = 4.4 kN.)

	Modal Forces and Story Shears						Design
	Mode 1		Mode 2		Mode 3		Story
Level	F_{xm} kip	V_{xm} kip	F_{xm} kip	V_{xm} kip	F_{xm} kip	V_{xm} kip	Shears kip
Roof	47.3		-24.6		4.7		
		47.3		-24.6		4.7	53.5
Third Floor	57.2		26.0		-18.3		
		104.5		1.4		-13.6	105.4
Second Floor	25.2		28.1		24.8		
		129.7		29.5		11.2	133.5
First Floor							

Table 18

Deflections and Story Drifts for 1978 ATC-3 Modal Analysis Procedure (Metric conversion factor: 1 ft = 0.3 m)

	Deflections				Story Drift, ft			
Level	Mode 1	Mode 2	Mode 3	Design Value	Mode 1	Mode 2	Mode 3	Design Value
Roof	0.3035	- 0.0243	0.0013	0.3044				
					0.1089	- 0.0379	0.0040	0.1154
Third Floor	0.1945	0.0136	- 0.0027	0.1950				
					0.1089	- 0.0011	- 0.0064	0.1091
Second Floor	0.0856	0.0147	0.0037	0.0869				
					0.0856	0.0147	0.0037	0.0869
First Floor	0	0	0	0				

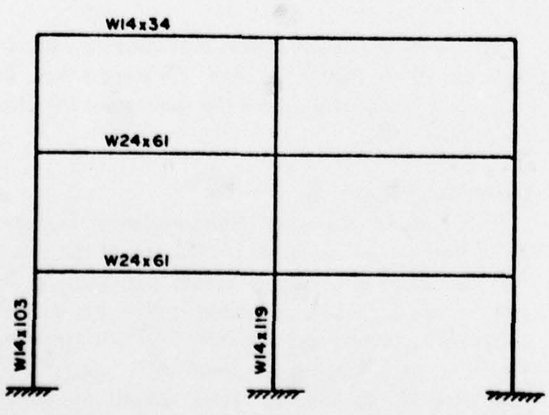


Figure 25. Frame member sizes for 1978 ATC-3 equivalent lateral force procedure.

The modal displacements were computed in accordance with Eq 17 and Table 21 summarizes the modal displacements and modal story drifts which were obtained by computing the difference between the displacement of the top and bottom of the story. The modal displacements and modal story drifts were combined by taking the square root of the sum of the squares of the modal values to obtain the total displacements and story drifts. The maximum lateral deflections and story drifts for the building were obtained by multiplying the total displacement by C_d = 1.5. These results are also summarized in Table 21. Subsequently, the computed story drifts were compared with the allowable story drifts and found to be satisfactory.

The story shears and the additional shears resulting from the accidental torsional moment were distributed to the vertical elements of the moment frames on

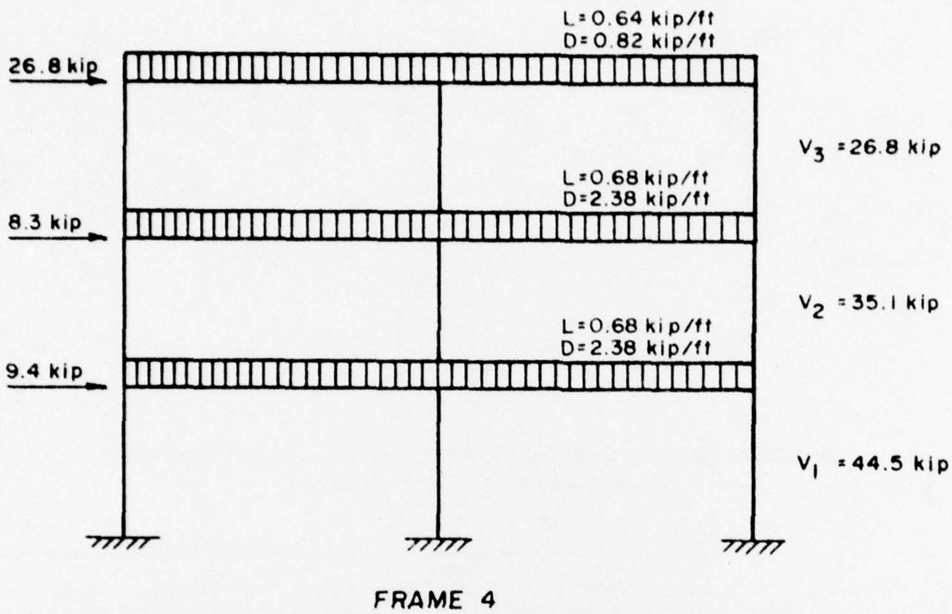


Figure 26. Frame loadings for 1978 ATC-3 modal analysis procedure. (Metric conversion factors: 1 ft = 0.3 m and 1 kip = 4.4 kN.)

Table 19

Periods and Mode Shapes for 1978 TM 5-809-10 Appendix Provisions

	Period and Mode Shape		
Level	**Mode 1 T = 0.512**	**Mode 2 T = 0.194**	**Mode 3 T = 0.106**
Roof	0.2081	0.1954	0.0669
Third Floor	0.1375	-0.1032	-0.1264
Second Floor	0.0606	-0.1211	0.1649

column lines 1, 4, and 7 in accordance with the procedures previously discussed (p 34). Figure 29 summarizes the resultant frame loadings. Each frame was analyzed for the two loading conditions shown in Eq 24. Figures 30 and 31 present the results of these analyses.

Member sizes were proportioned following the same procedures used for the 1978 ATC-3 provisions, except that the moments and forces were divided by the factor 1.7 rather than 1.53, because the TM 5-809-10 Appendix provisions do not consider capacity reduction factors. Figure 32 summarizes the final member sizes.

The story shears and lateral deflections are plotted as Curve D in Figures 21 and 22, respectively, for purposes of comparison with the value from the other provisions.

Design Comparison

The plots of the story shears presented in Figure 21 provide a good graphical comparison of the results obtained using the various design provisions. The ATC-3 modal analysis procedure yields the smallest design story shears and the TM 5-809-10 Appendix provisions yield the largest design story shears. When referenced to the SEAOC story shears, the ATC-3 modal analysis procedure yields story shears that are approximately 50 percent of the SEAOC story shears, the ATC-3 equivalent lateral force procedure yields

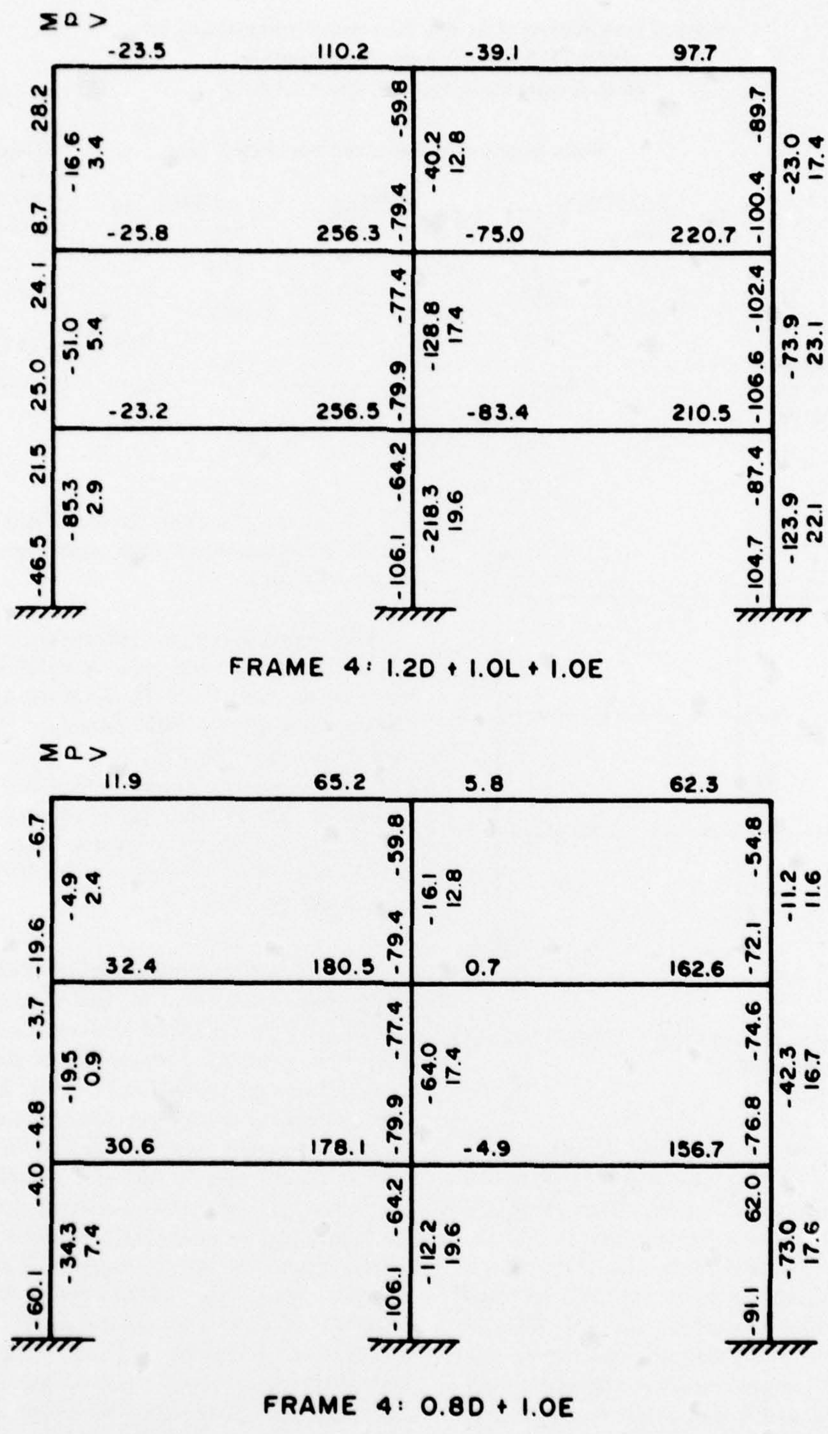


Figure 27. Frame 4 member moments and forces for 1978 ATC-3 modal analysis procedure. (Metric conversion factors: 1 kip = 4.4 kN and 1 ft-kip = 1.4 km-N.)

Table 20

Vertical Distribution of Lateral Forces and Story Shears for 1978 TM 5-809-10 Appendix Provisions (Metric conversion factor: 1 kip = 4.4 kN)

	Modal Story Lateral Forces and Story Shears, kip						Design Story Shears kip
	Mode 1		Mode 2		Mode 3		
Level	F_{xm}	V_{xm}	F_{xm}	V_{xm}	F_{xm}	V_{xm}	
Roof	316.9		-129.4		27.2		
		316.9		-129.4		27.2	343.4
Third Floor	395.2		129.0		-96.8		
		712.1		-0.4		-69.6	715.5
Second Floor	174.3		151.4		126.3		
		886.4		151.0		56.7	901.0
First Floor							

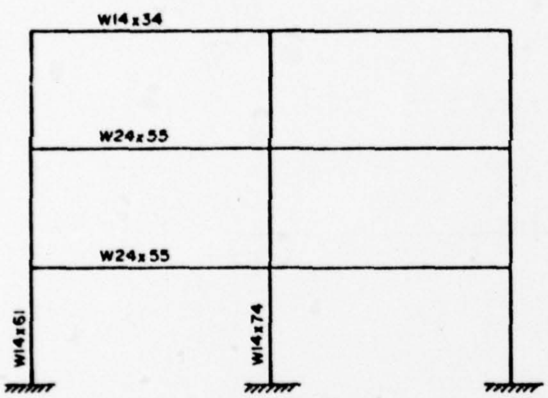
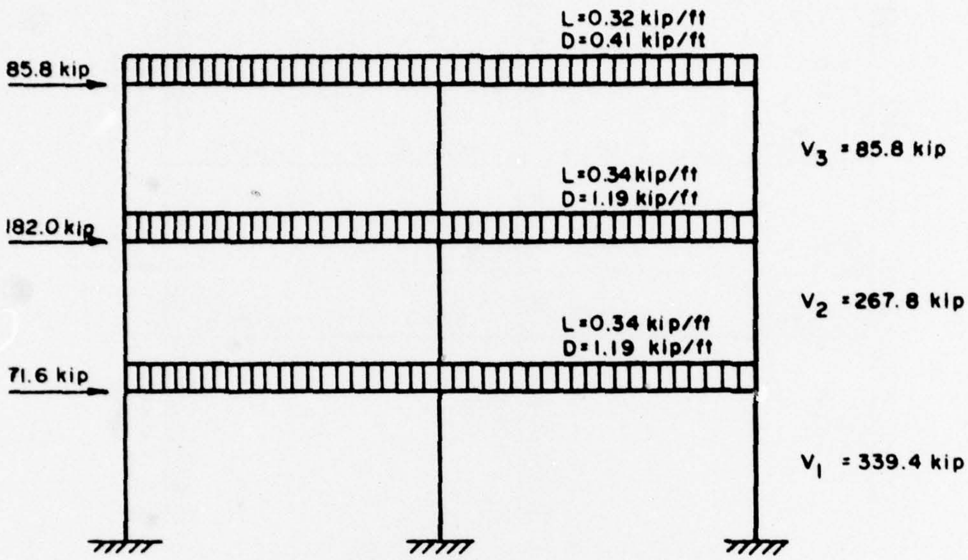


Figure 28. Frame member sizes for 1978 ATC-3 modal analysis procedure.

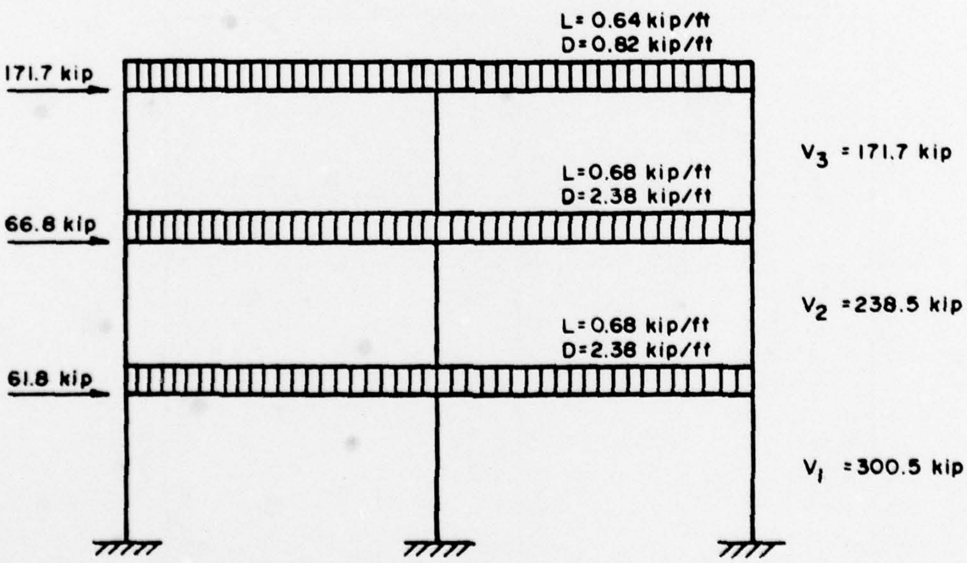
story shears that are approximately 60 percent of the SEAOC story shears, and the TM 5-809-10 Appendix provisions yield story shears that are approximately 325 percent greater than the SEAOC story shears. One explanation for this difference is the underlying design philosophy associated with the various seismic design provisions. The TM 5-809-10 Appendix provisions are intended to provide reasonable assurance that essential facilities will remain functional following an earthquake, while the other provisions are intended to minimize the hazard to life. Essential facilities should experience only limited damage so that critical functions can be maintained, i.e., they cannot undergo large inelastic deformations which are synonomous with large response modification coefficients.

The comparison of the lateral deflections in Figure 22 indicates that while there is significant difference between the ATC-3 and TM 5-809-10 Appendix story shears, there is very little difference in their lateral deflections. The SEAOC lateral deflections are not directly comparable to the ATC-3 and TM 5-809-10 Appendix lateral deflections because the SEAOC deflections correspond to yield stress in the members, while the other deflections incorporate varying amounts of ductility.

Perhaps a better method for comparing the various design provisions would be the quantity of steel required for a typical lateral force-resistant frame in the transverse direction. The various weights of a single typical frame are summarized in Table 22. In addition, the frame weights have been normalized by the 1975 SEAOC provisions' frame weight, and the added weight per unit floor area of the entire building has been computed. The lateral force-resistant frames designed in accordance with the ATC-3 modal analysis procedure and the SEAOC provision have nearly identical weights, while the lateral force-resistant frames designed in accordance with the ATC-3 equivalent lateral force procedure and TM 5-809-10 Appendix provisions require about one and one-half to two times as much steel as the SEAOC design. However, this increased quantity of steel should have only a nominal (1 to 2 percent) impact on the building's total cost because:

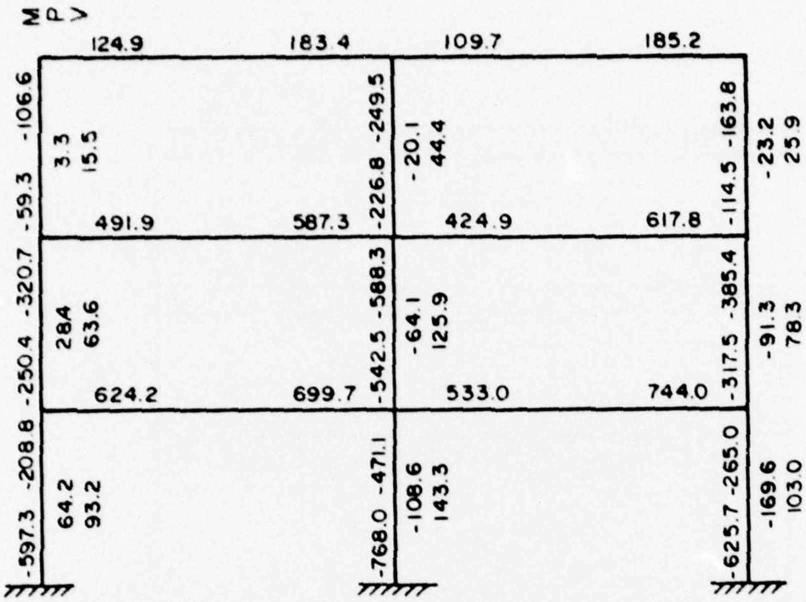


FRAME 1 AND 7



FRAME 4

Figure 29. Frame loadings for TM 5-809-10 appendix provisions.
(Metric conversion factor: 1 kip = 4.4 kN and 1 ft = 0.3 m)



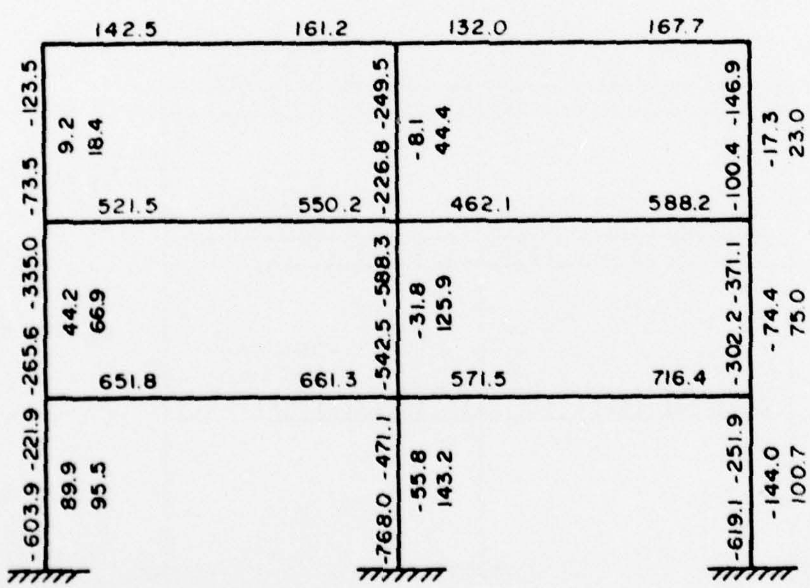
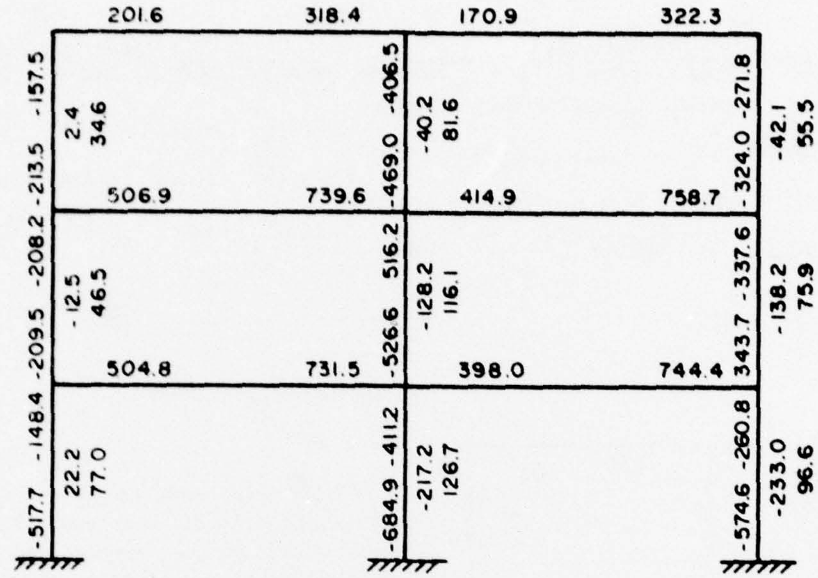
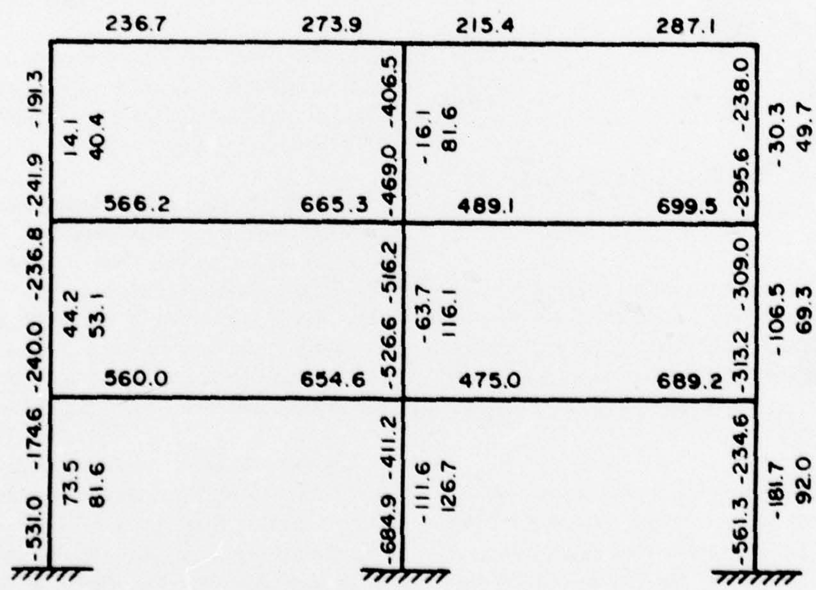


Figure 30. Frame 1 and 7 member moments and forces for TM 5-809-10 appendix provisions. (Metric conversion factors: 1 kip = 4.4 kN and 1 ft = 1.4 km-N.)



FRAME 4: 1.2D + 1.0L + 1.0E



FRAME 4: 0.8D + 1.0E

Figure 31. Frame 4 member moments and forces for TM 5-809-10 appendix provisions. (Metric conversion factors: 1 kip = 4.4 kN and 1 ft-kip = 4.4 km-N.)

Table 21

Displacements and Story Drifts for 1978 TM 5-809-10 Appendix Provisions
(Metric conversion factor: 1 ft = 0.3 m)

Level	Displacements, ft					Story Drift, ft				
	Mode 1	Mode 2	Mode 3	SRSS	Maximum	Mode 1	Mode 2	Mode 3	SRSS	Maximum
Roof	0.1809	-0.0106	0.0007	0.1812	0.2718					
						0.0614	-0.0162	0.0020	0.0635	0.0953
Third Floor	0.1195	0.0056	-0.0013	0.1196	0.1795					
						0.0668	-0.0010	0.0029	0.0669	0.1003
Second Floor	0.0527	0.0066	0.0016	0.0531	0.0797					
						0.0527	0.0066	0.0016	0.0531	0.0797
First Floor	0	0	0	0	0					

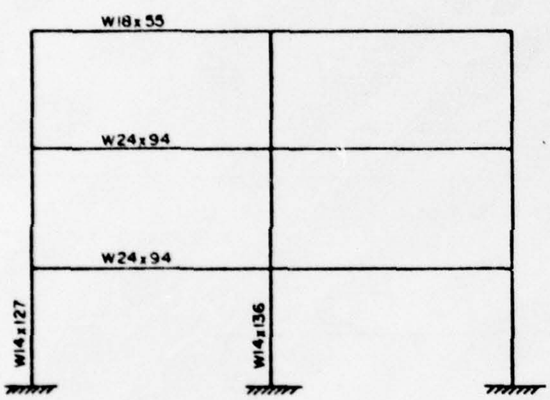


Figure 32. Frame member sizes for TM 5-809-10 appendix provisions.

1. The building's structural system consists of seven load-carrying frames in the transverse direction, secondary framing systems, and floor systems. With the exception that three of the seven frames are lateral force-resistant frames, the remainder of the structural system is not affected.

2. For this type of building, the structural system represents 15 to 20 percent of the total cost of the building. Thus, if the structural system's cost increased by approximately 10 percent, the building's total cost would increase by only about 2 percent. However, this does not imply that the disparity between the various provisions should not be evaluated in greater detail to assure the validity of designing with the larger story shears.

4 CONCLUSIONS

Based on the comparison of the story shears and lateral deflections for Letterman Hospital and the designs of a three-story, moment resistant steel frame building carried out in accordance with various design provisions, several conclusions are apparent.

The major conclusions with respect to the Letterman Hospital story shears and lateral deflections are:

1. The 1964 UBC provisions produce equivalent yield stress basis story shears which are approximately 10 to 35 percent of the story shears obtained from AA's time history modal analysis.

2. When the dynamic characteristics of the structures are considered with regard to determining the total lateral forces and their distributions, the 1975 SEAOC provisions produce equivalent yield stress basis story shears which are in reasonable agreement with the story shears obtained from AA's time history modal analysis except in the base structure and the intermediate stories in the tower in the E-W direction.

3. The 1978 ATC-3 modal analysis procedure yields story shears which are generally approximately 100 percent greater than the 1964 UBC equivalent yield stress basis story shears in the tower and approximately 10 percent less than the 1964 UBC equivalent yield stress basis story shears in the basic structure.

4. When comparable acceleration levels are specified, the TM 5-809-10 Appendix provisions yield story shears which are in reasonable agreement with

Table 22

Comparison of Frame Weights and Base Shears for Three-Story, Moment Resistant Steel Frame Building (Metric conversion factors: 1 lb = 4.4 N, 1 kip = 4.4 kN, and 1 lb/ft^2 = 47.9 Pa.)

	Design Provisions			
Quantity	75 SEAOC	78 ATC-3 Static	78 ATC-3 Dynamic	TM 5-809-10 Appendix
Frame Weight (lbs of steel/frame)	13,041	18,164	13,194	24,329
Ratio: Frame Weight/75 SEAOC Frame Weight	1.00	1.39	1.01	1.87
Added Weight/Unit Floor Area (lbs/ft^2)	0	0.56	0.02	1.22
Center Frame Base Shear (kip)	92.0*	54.9	44.5	300.5
Ratio: Base Shear/75 SEAOC Base Shear	1.00	0.60	0.48	3.27

*SEAOC computed base shear multiplied by 1.28 to provide consistent yield stress basis for comparison.

the story shears obtained in AA's time history modal analysis, except for the lower stories in the N-S direction where they differ by approximately 20 percent.

5. Generally, the lateral deflections in the N-S direction are in better agreement than the lateral deflections in the E-W directions, with the exception that in both directions, the lateral deflections from the original design are quite small and bear little resemblance to the lateral deflections obtained by the other analysis.

The major conclusions with respect to the application of the various provisions to the design of a three-story, moment resistant steel frame building are:

1. The 1978 ATC-3 provisions yield story shears that are approximately 50 to 60 percent of the 1975 SEAOC story shears, while the TM 5-809-10 Appendix provisions yield story shears that are approximately 325 percent greater than the 1975 SEAOC story shears.

2. The TM 5-809-10 Appendix story shears are significantly greater than either the 1975 SEAOC or the 1978 ATC-3 story shears. This difference is primarily attributable to the difference in philosophies which underlie the various design provisions. The TM 5-809-10 Appendix provisions are intended to provide reasonable assurance that essential facilities will remain functional following an earthquake, while the SEAOC and ATC-3 provisions are intended to minimize the hazard to life and provide reasonable assurance that facilities will not collapse. Essential facilities simply cannot undergo the large inelastic deformation which occurs near collapse and still remain functional.

3. While there is significant difference between the 1978 ATC-3 and TM 5-809-10 Appendix story shears, there is very little difference in their lateral deflections.

4. The lateral force-resistant frames designed in accordance with the 1978 ATC-3 modal analysis procedure and the SEAOC provisions have nearly identical weights, while frames designed in accordance with the 1978 ATC-3 equivalent lateral force procedure and the TM 5-809-10 Appendix provisions have weights that are approximately one and one-half and two times as much, respectively, as the SEAOC design. However, this increased quantity of steel should have only a nominal (1 to 2 percent) impact on the building's total cost because:

1. The building's structural system consists of seven load-carrying frames in the transverse direction, secondary framing systems, and floor systems. With the exception that three of the seven frames are lateral force-resistant frames, the remainder of the structural system is not affected.

2. For this type of building, the structural system represents approximately 15 to 20 percent of the total cost of the building. Thus, if the structural system's cost increased by approximately 10 percent, the building's total cost would only increase by about 2 percent.

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