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CALIFORNIA INSTITUTE OF TECHNOLOGY

EARTHQUAKE ENGINEERING RESEARCH LABORATORY

THE RESPONSE OF VETERANS HOSPITAL BUILDING 41 IN THE SAN FERNANDO EARTHQUAKE

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Report No. EERL 80-03

A Report on Research Conducted under Grants from the National Science Foundation and the Earthquake Research Affiliates Program at the California Institute of Technology

Pasadena, California
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I. INTRODUCTION

1. General

Structures which collapse or are heavily damaged in destructive earthquakes are analyzed by engineers to determine why they performed so poorly and to find out how their design could have been improved. However, it is equally important for buildings that survived exceptionally strong shaking to be analyzed and an explanation given as to why they were able to do so.

During the San Fernando, California earthquake of February 9, 1971 buildings in the strongly shaken region showed both types of performance. For example, the new Olive View Hospital main building was severely damaged, and another major building collapsed whereas two buildings at the Veteran's Administration (VA) Hospital survived with no significant damage. These two hospitals were located just north of the major surface faulting, and the VA buildings were only 1¼ miles southwest of Pacoima Dam. The Dam was effectively over the center of energy release of the magnitude of 6.4 earthquake and the well-known Pacoima Dam accelerogram, with peak accelerations over 1g, was recorded on a steep ridge near the abutment of the Dam. The ground shaking at the VA hospital is thought to have been less severe than that recorded at Pacoima Dam, but more severe than that recorded at the Holiday Inn, which was approximately five miles south of the nearest point on the causative fault.

Two major structures collapsed at the Veteran's Administrative Hospital killing 46 persons, which accounted for most of the casualties in the earthquake. These buildings were constructed in the 1920's and were not designed to resist earthquakes. Within the immediate neighborhood of these collapsed buildings were two other major structures that were built in the 1930's and the 1940's in accordance with building codes requiring earthquake resistance, and these survived the San Fernando earthquake without significant damage. One of these structures is the subject of this report.

The Principal objective of the analysis in this report is to reconcile the observed behavior of the structure with the level of shaking experienced during the earthquake. It would have been extremely valuable to have had available strong-motion records of the ground shaking and the building response, but unfortunately, the site was not instrumented so the ground motion must be estimated from records obtained at nearby sites.

The three basic facts which impose constraints on the analysis are 1) the building was designed for a horizontal load of approximately 10% of its weight at normal working stresses; 2) the structure received only very minor structural damage indicating no significant inelastic behavior in the main structural components; and 3) the ground shaking was very strong, probably inducing effective lateral loads of one half the weight of the building or greater. Because of these controlling conditions, it is not easy to explain quantitatively the successful performance of the building by the usual engineering analyses, and additional mechanisms beyond those normally considered in seismic analyses are required to reconcile the analytical evaluation of the response with the observed behavior.

The Veterans Hospital buildings provide what is probably the best example in the San Fernando earthquake of structures designed to resist nominal lateral loads surviving intense ground shaking without severe damage. Another striking example is provided by North Hall at the University of California, Santa Barbara during the Santa Barbara earthquake of August 13, 1978.[(1)*] This rectangular, three-story reinforced concrete

*References are included at the end of the text.

shear wall structure was built with a deficiency in its lateral resistance, and concrete shear walls were added to make the structure meet the 1976 edition of the Uniform Building Code. The building was instrumented with strong-motion accelerographs under the program of the Office of Strong-Motion Studies of the California Division of Mines and Geology.[2] During the earthquake, the base acceleration in the transverse direction was about 40 percent g, approximately 65 percent g was measured on the third floor, and the roof record reached 1g. These records imply a base shear of 50 to 70 percent of the weight of the structure and yet the damage to the building consisted only of light-to-moderate X-cracking in the concrete shear walls.

From the response of these buildings and others in other earthquakes it is seen that excellent behavior of low-rise shear wall buildings during intense shaking is not uncommon, and the response of these structures should be studied in detail to document the sources of their resistance.

1.2 Setting and Background Information

The VA Hospital complex was situated near the foot of the San Gabriel mountains in the northeastern section of the San Fernando Valley, near the city of San Fernando and about twenty-five miles north of downtown Los Angeles. Figures 1.1 and 1.2 show the general location of the site. In 1971 the hospital complex consisted of fifty-four buildings and ancillary structures, a number of which were constructed in 1925 with major additions made in 1938 and 1949. The plan of the site is shown in Fig. 1.3, and an aerial view is given in Fig. 1.4.

It will be recalled that following the March 10, 1933 Long Beach earthquake, measures were taken by the California State Legislature and

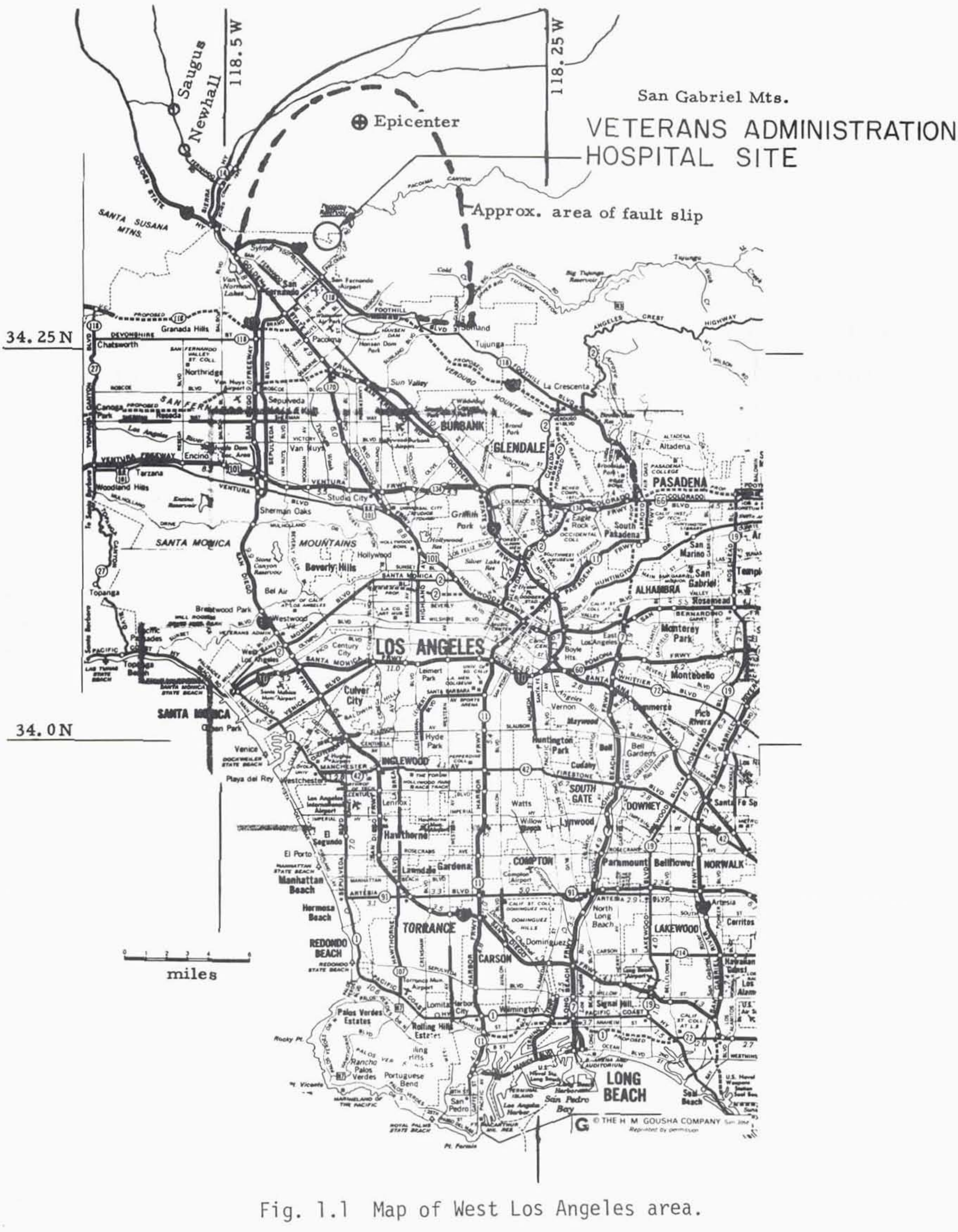


Fig. 1.1 Map of West Los Angeles area.

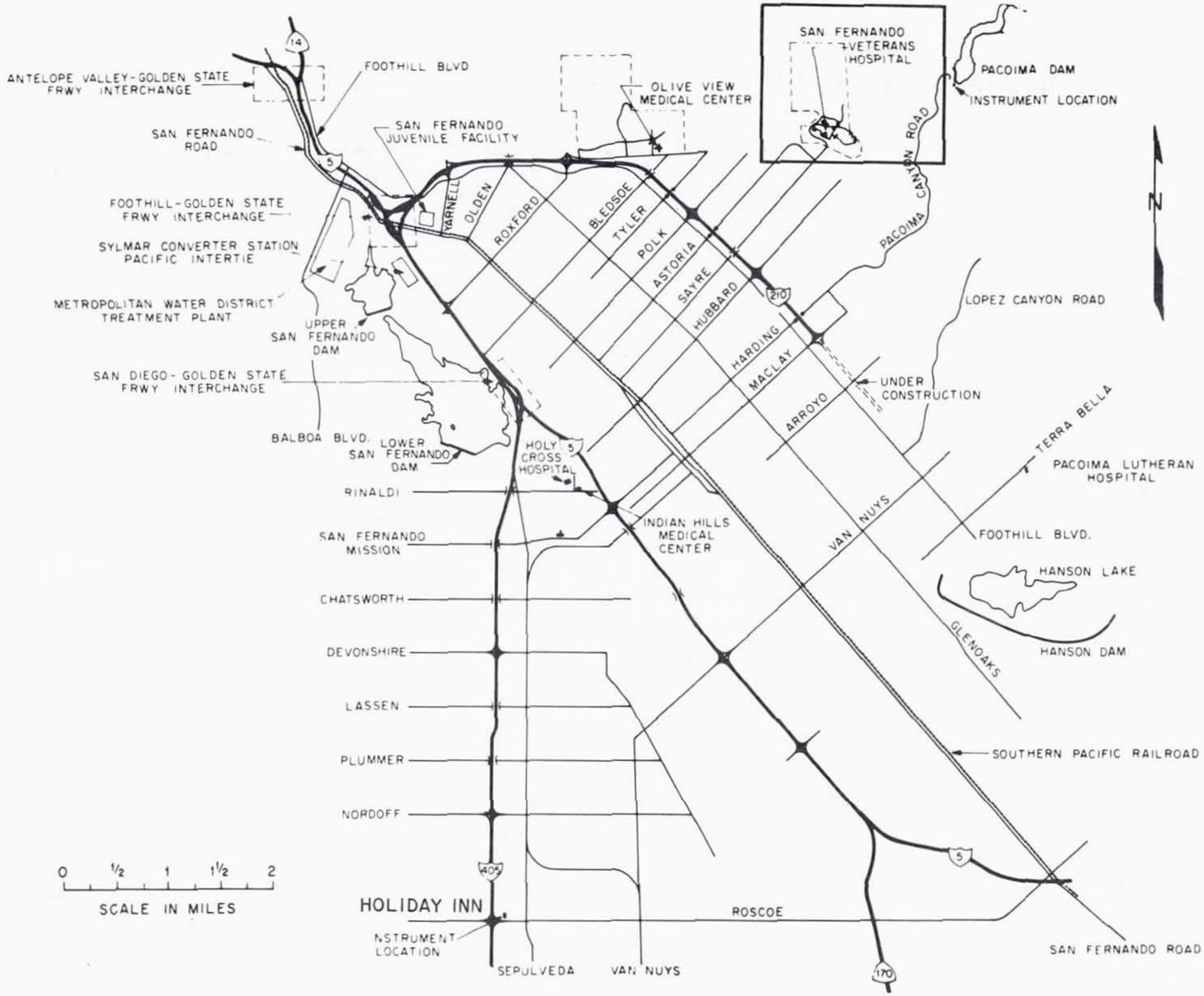


Fig. 1.2 Area subjected to strong shaking.

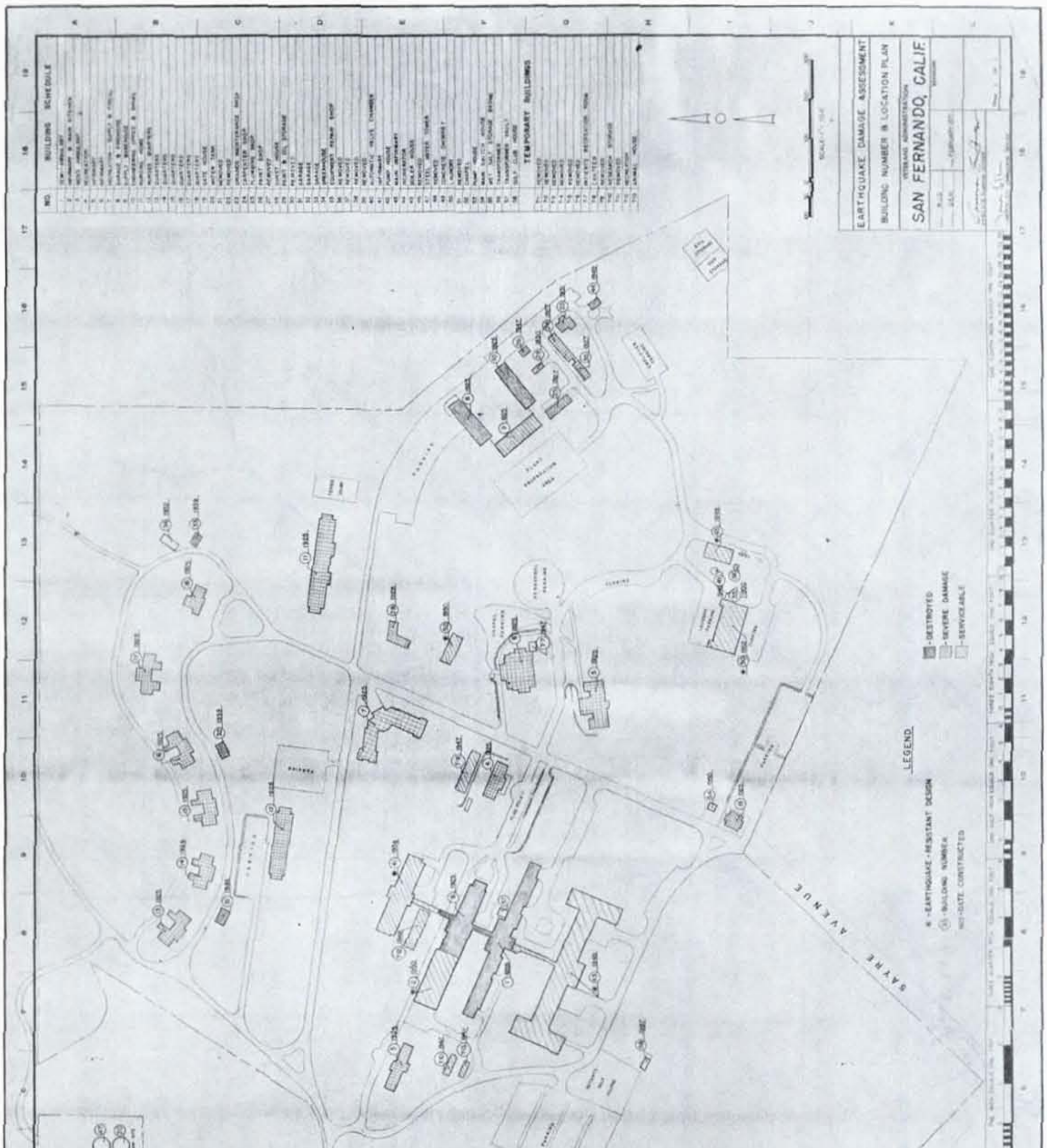


Fig. 1.3 Plan of the Veterans Administration Hospital site and damage distribution.

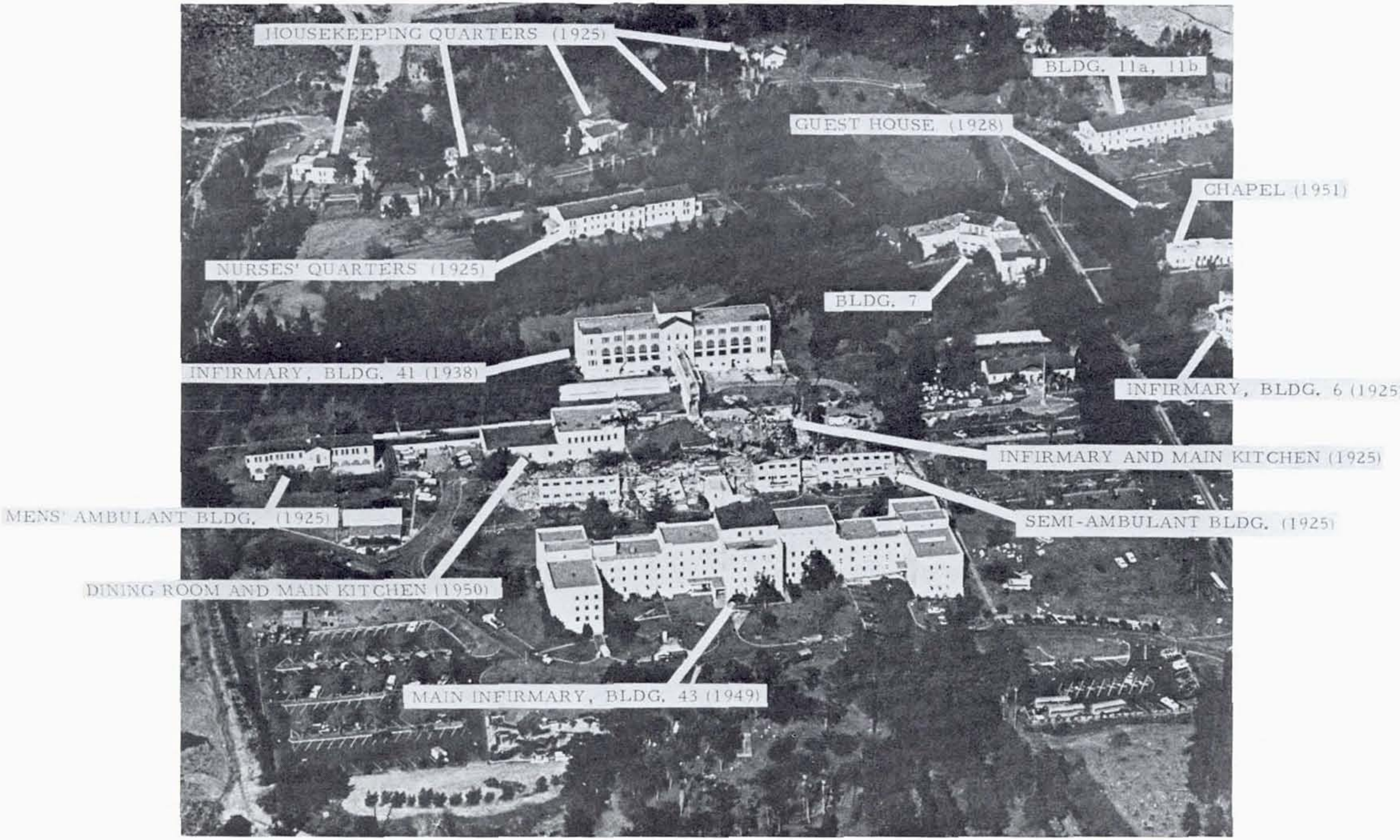


Fig. 1.4 Aerial view of the Veterans Administration Hospital complex taken the day of the earthquake. (Ralph Samuels photo.)

by local authorities to ensure that public buildings be designed to resist earthquakes. Although the Veterans Administration was not bound to follow local regulations, its post-1933 buildings in the San Fernando hospital were in fact designed to resist earthquakes. It was the 26 buildings and additions constructed prior to 1933 in the hospital complex that sustained the greatest structural damage; four of them totally collapsed, and most of the remaining structures in this class experienced various levels of structural damage. The majority of these buildings had floor slabs and a vertical load carrying frame of reinforced concrete, but unreinforced hollow clay tile shear walls. Although the buildings met the existing standards at the time of their construction, they would not be acceptable by modern standards. By contrast, only four of the twenty-two buildings and additions built after 1933 were seriously damaged. The damage distribution in the hospital complex is shown in Fig. 1.3.

Of these newer structures, the major buildings were the main infirmary and another infirmary (No.43 and No.41 in Fig. 1.3), and an added wing to one of the collapsed buildings (No.26).

These structures were built after the earthquake regulations had been instituted, and they all survived the San Fernando earthquake with only minor structural damage. The basic reason for this successful performance was the presence of a large number of interconnected reinforced concrete shear walls in both directions. Although these buildings were designed for a lateral load coefficient of approximately 10 percent of their weight (at working stresses) it was quite clear from their structural layout that they must have been much stronger than indicated by this number.

A brief comparative study of the structural drawings for the two buildings, as well as some preliminary lateral force computations, revealed that Buildings 41 and Buildings 43 were quite similar in their structural layout, design and detailing, and they appeared to have had similar margins of safety under statically applied lateral loads. It was therefore decided to confine the study to only one of the buildings. Building 41 was chosen for detailed investigation because its structural layout appeared to be simpler and therefore easier to model. Building 43 had a symmetric typical floor plan, but had a large asymmetrically located penthouse. Building 41 was more nearly symmetric. Also, the floor slab of Building 43 was much more slender as a horizontal beam than its counterpart in Building 41, and it appeared that one of the basic assumptions which was to be made in the analysis, namely the inplane rigidity of the floor plan, would be less appropriate for Building 43. Because of the greater strength of the building in the longitudinal direction, only the response in the transverse direction is studied in this report.

The main difficulty encountered in this study has been the paucity of information regarding the structural properties of the construction materials and the foundation soils. Since these buildings were razed some time after the earthquake, it was not possible to obtain information on material properties beyond that which could be gleaned from the structural drawings and the brief calculation sheets.

Another difficulty was the selection of ground motion to be used for the dynamic analysis. The nearest record, at the Pacoima Dam site, is believed to have serious limitations for this purpose because of the marked difference in terrain between the hospital site and the location of the accelerograph, whereas the nearest reading obtained on the valley floor is over 8 miles away, and is probably significantly different in character because of the effect of distance on the motion.

Finally, information on the structural damage which is required for a detailed study, e.g., the precise delineation of cracks in concrete walls, the extent of differential foundation settlements, etc., is not available.

In view of these limitations, it was realized that great rigor in the analysis could not serve a useful purpose, and that a full and detailed explanation of the mechanism which enabled the survival of these buildings probably could not be given. Yet, it was believed that some insight into the response would be gained, and the parameters affecting the response of similar buildings to strong ground shaking could be identified.

1.3 Organization of the Report

In Chapter II the building structure and its foundation system are described, its original design and analysis are discussed, and the structural properties of construction materials and foundation soils are examined. The appropriate ground motion for the site during the earthquake is discussed in Chapter III. This is followed, in Chapter IV, by a short review of the damage. The results of an equivalent lateral force analysis along the lines of the building code are presented in Chapter V. These results are then compared in Chapter VI with a response spectrum analysis of a three-dimensional model of the entire building. On the basis of this analysis a representative plane frame is identified and isolated from the structure. The foundation of this frame is modelled to allow uplift and yielding of the supporting soil. The results of several time history analyses carried out for this frame are presented and interpreted in Chapter VII. General conclusions and recommendations are given in Chapter VIII.

II. DESCRIPTION OF THE BUILDING AND ITS DESIGN CRITERIA

2.1 Introduction

Building 41 was located west of Sayre Ave. near the center of the complex (Fig. 1.3). It was designed in 1937 by the engineering office of the Veterans Administration and built in 1938. The building was four stories high (51 ft.), about 200 X 50 feet in plan, with a centrally located penthouse.

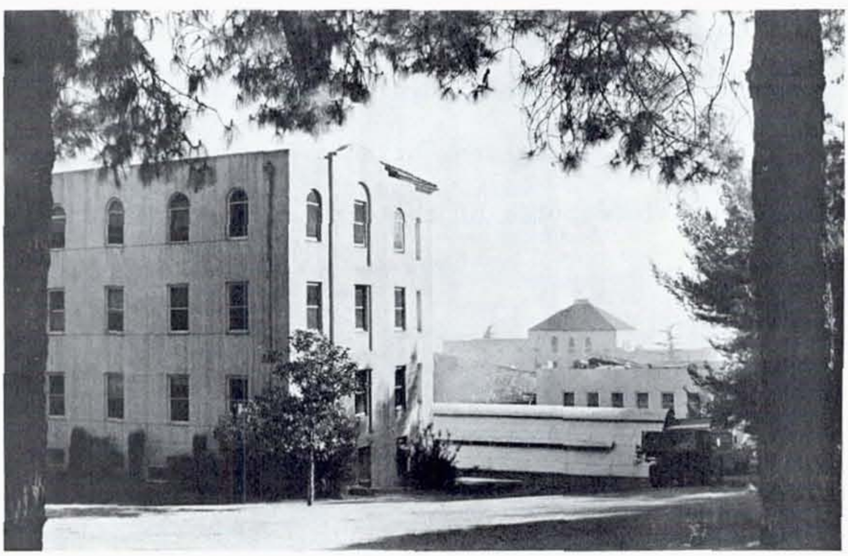
Due to the sloping terrain, the ground story (or basement) was half buried on the north side, whereas it was nearly on grade along the south elevation. The building was connected with Building No.2 by means of a covered walkway at the first level. The canopy served as another walkway from the second floor. An expansion joint separated the two structures.

The vertical and lateral load carrying system of the structure consisted of reinforced concrete shear walls and frames supported on spread footings. Photographs of the south and west elevations taken after the earthquakes are shown in Fig. 2.1. A typical structural floor plan is shown in Fig. 2.2 wherein the one-fold symmetry of the structural system is evident. In the following the essential features of the structural system are described.

2.2 Foundations

The foundation system consisted of strip footings under the bearing walls and square footings under the columns. The base of the footings was stepped, mainly to ensure adequate depths of embedment. A schematic foundation plan is given in Fig. 2.3, and it is seen that the footings under the longitudinal walls form flanges for the strip footings of the transverse walls (and vice-versa).

(a)



(b)

Fig. 2.1 Building 41 (a) South elevation with walkway to Building 2 (b) Northwest corner.

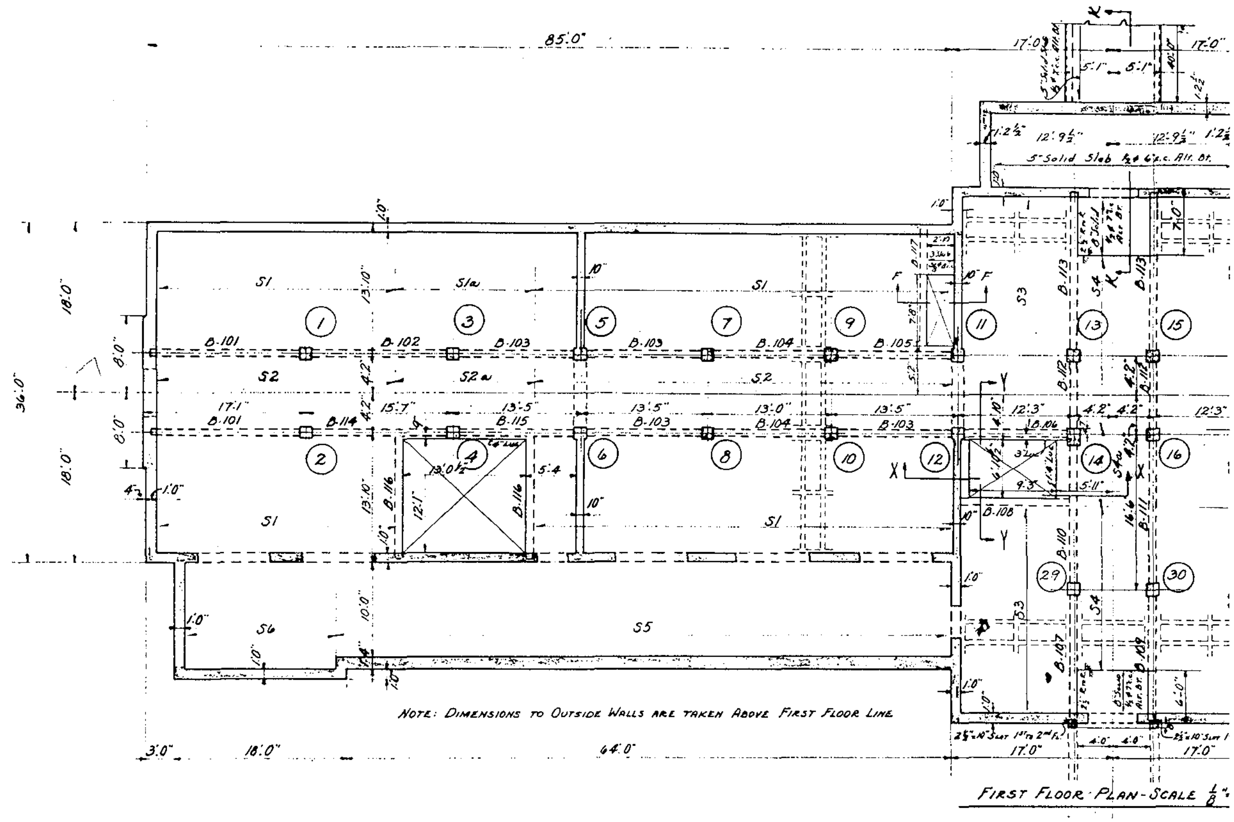


Fig. 2.2 Structural floo

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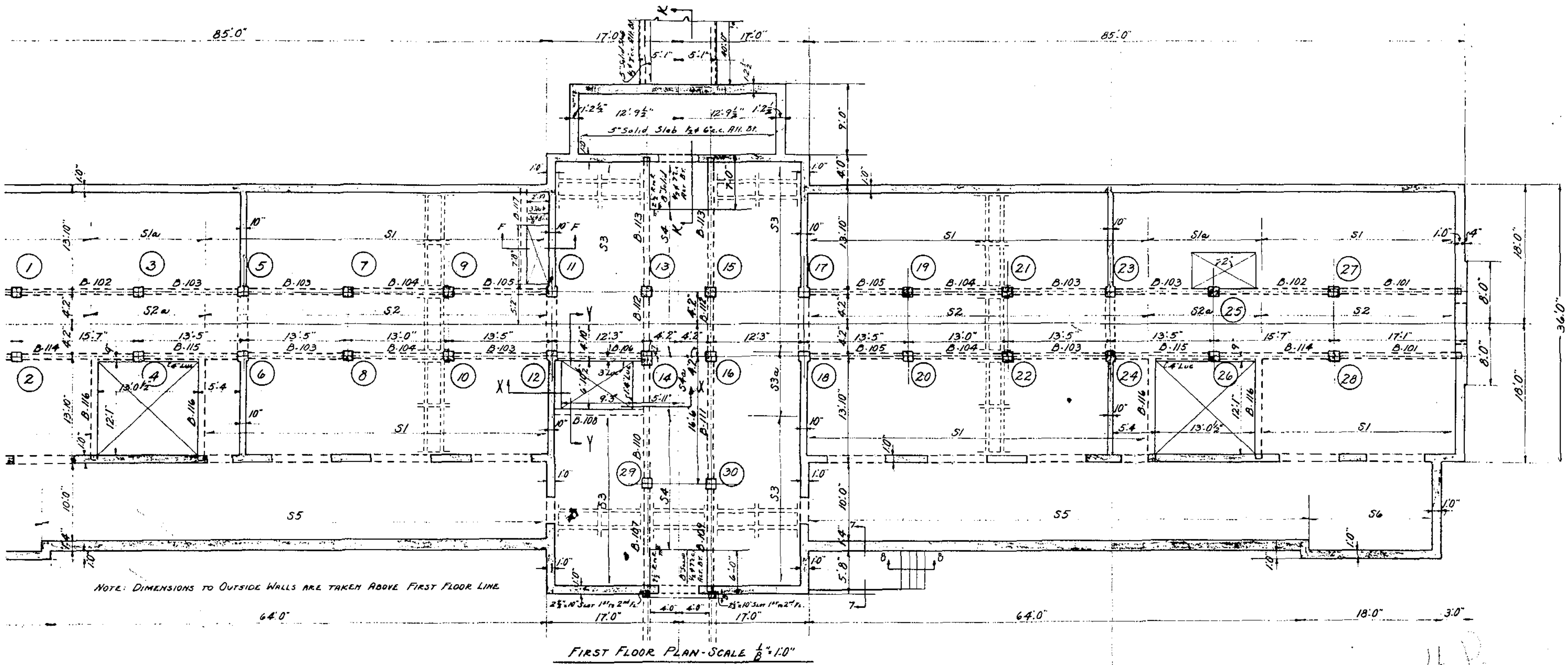

Fig. 2.2 Structural floor-typical layout

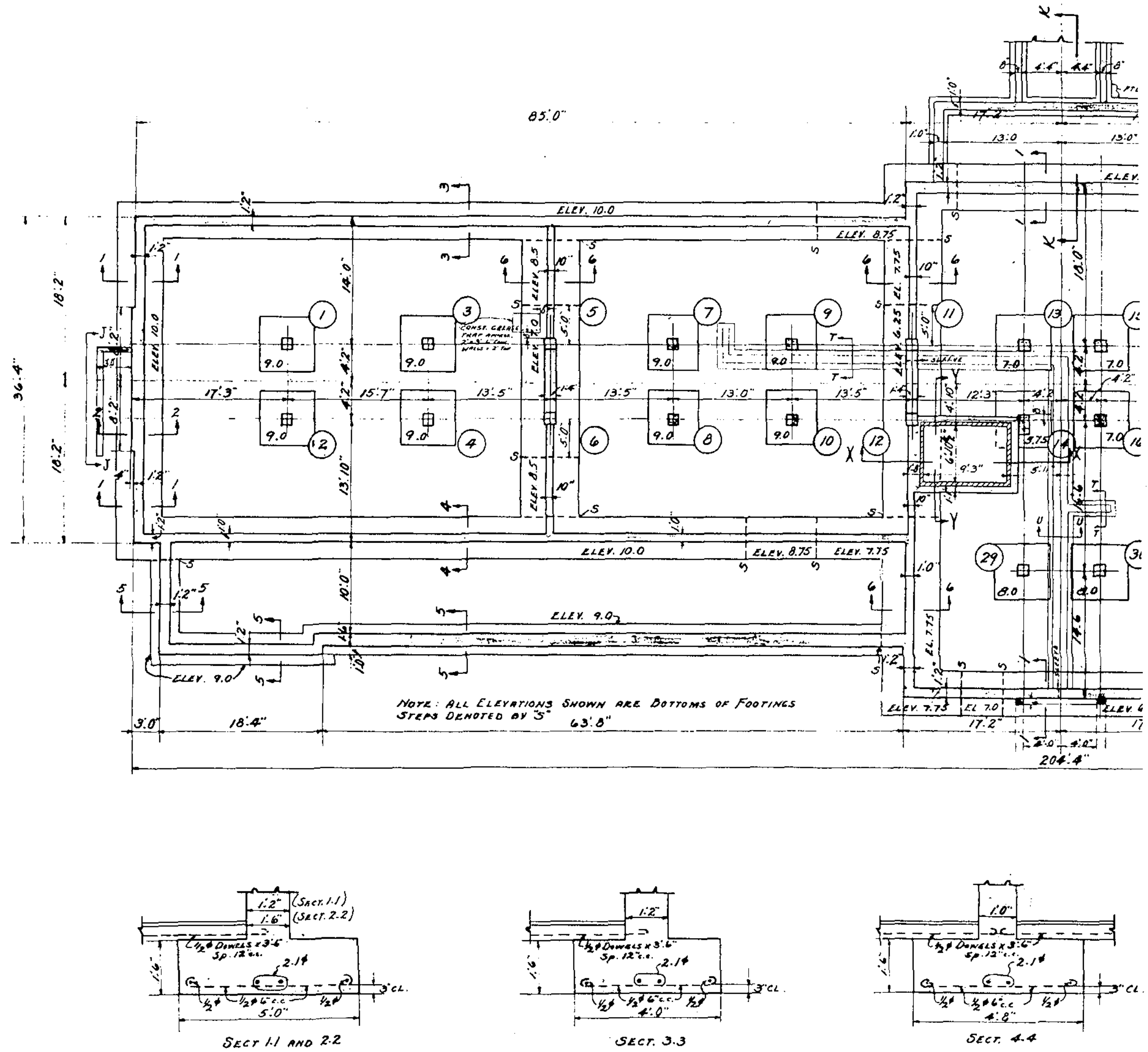

Fig. 2.3 Foundation pla

14a

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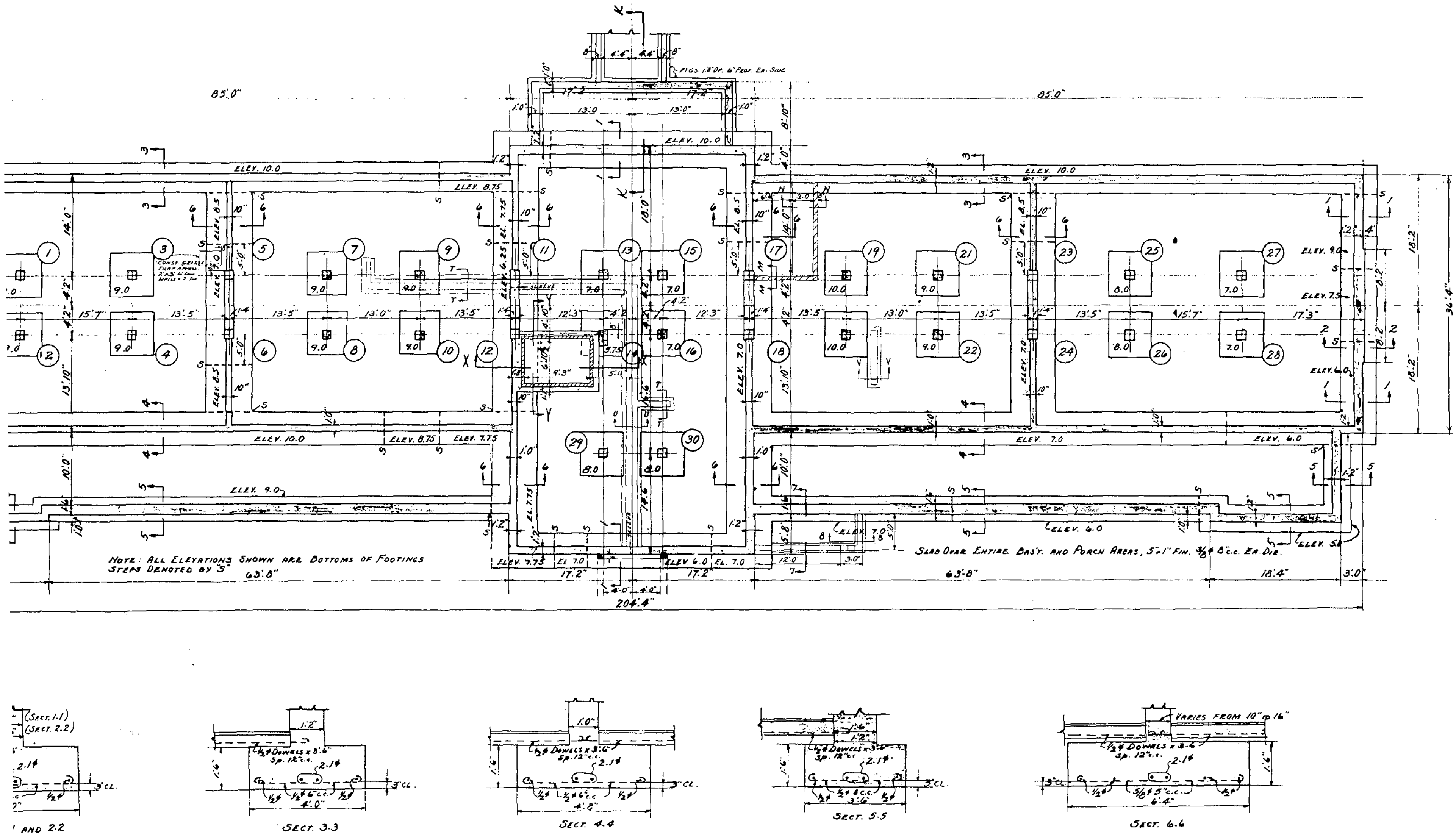


Fig. 2.3 Foundation plan

14a

14b

2.3 Superstructure

2.3.1 RC Walls and Columns

As can be seen from Fig. 2.2, most of the vertical loads, and practically all the lateral loads, were carried by the bearing walls. There were six walls symmetrically located in the transverse direction (N-S), and three in the longitudinal direction (E-W). The total length of the latter walls was about 580 ft. which is more than twice the total length of the transverse walls. All walls were perforated for door and window openings, so that structurally, they can be classified as coupled shear walls. Structural elevations of the walls are shown in Fig. 2.4. Above the first floor level wall thicknesses varied between 10" and 16". Below this all peripheral walls were thickened by 2 inches.

As can be seen from the drawings, the walls were vertically and horizontally reinforced on both faces, the area of distributed steel in each direction exceeded 0.003 of the cross-sectional area of the walls. Special edge reinforcement around the openings was also provided. All bars, including the deformed ones, were hooked. Note that all the internal cross walls had boundary elements facing the central corridor. These elements were detailed as tied columns, although they were not designed to carry the axial loads resulting from the horizontal forces as would now be the practice for heavily loaded walls (Ref.3). Also, the two exterior cross walls were wider at these boundaries, although no special boundary reinforcement was detailed. Coupling beams were reinforced top and bottom with closely spaced stirrups along their entire length.

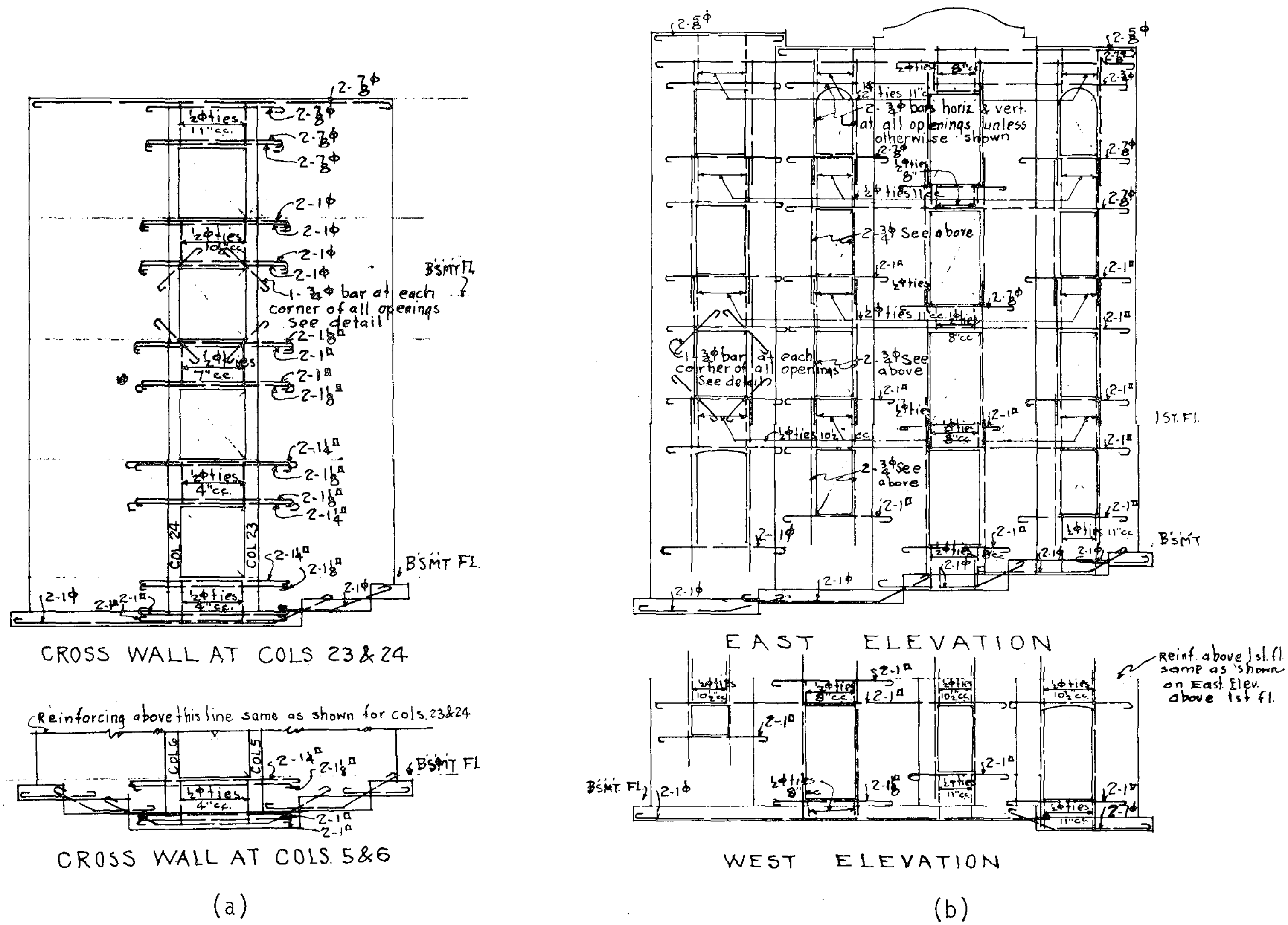
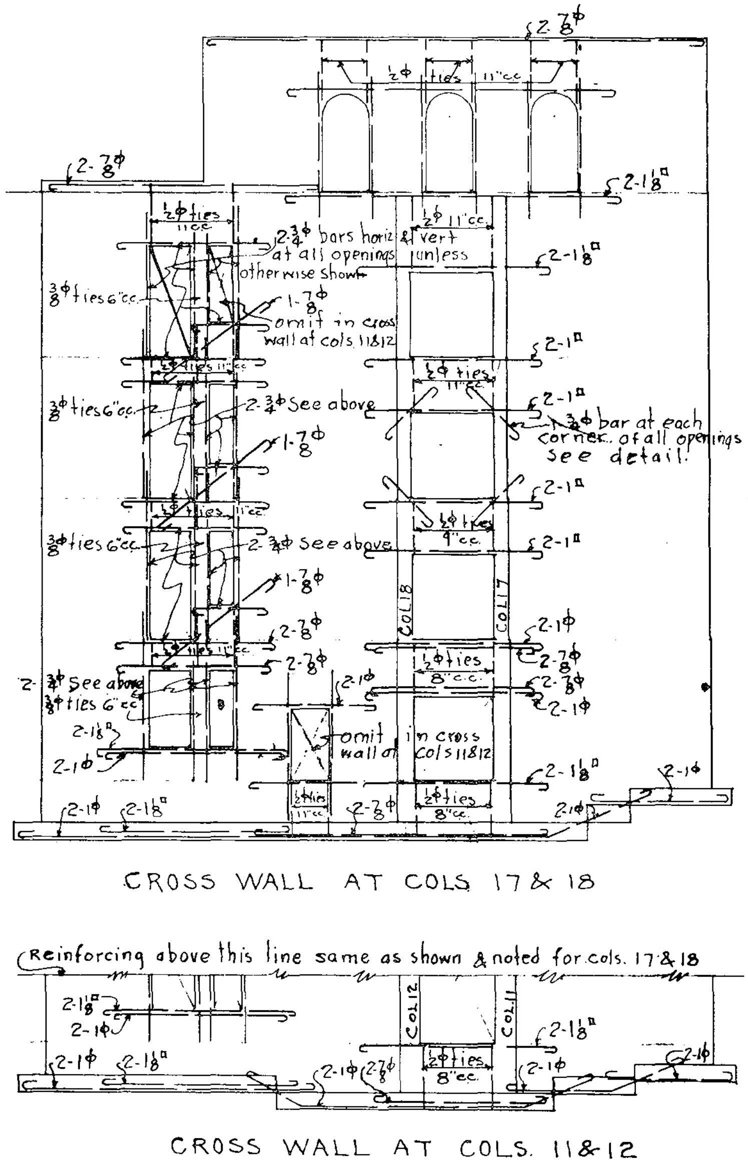
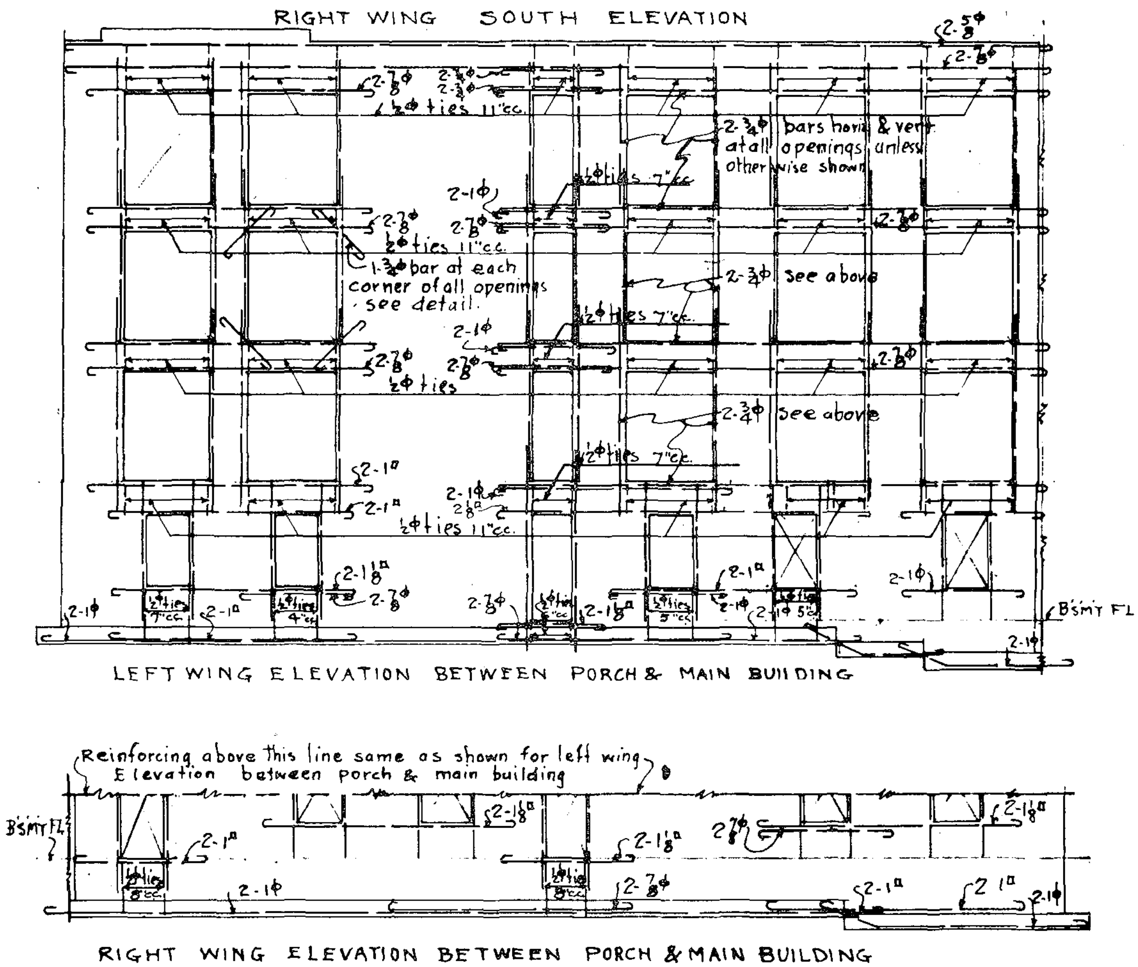


Fig. 2.4 Structural elevation of shear walls (a) Wall A (b) Wall B



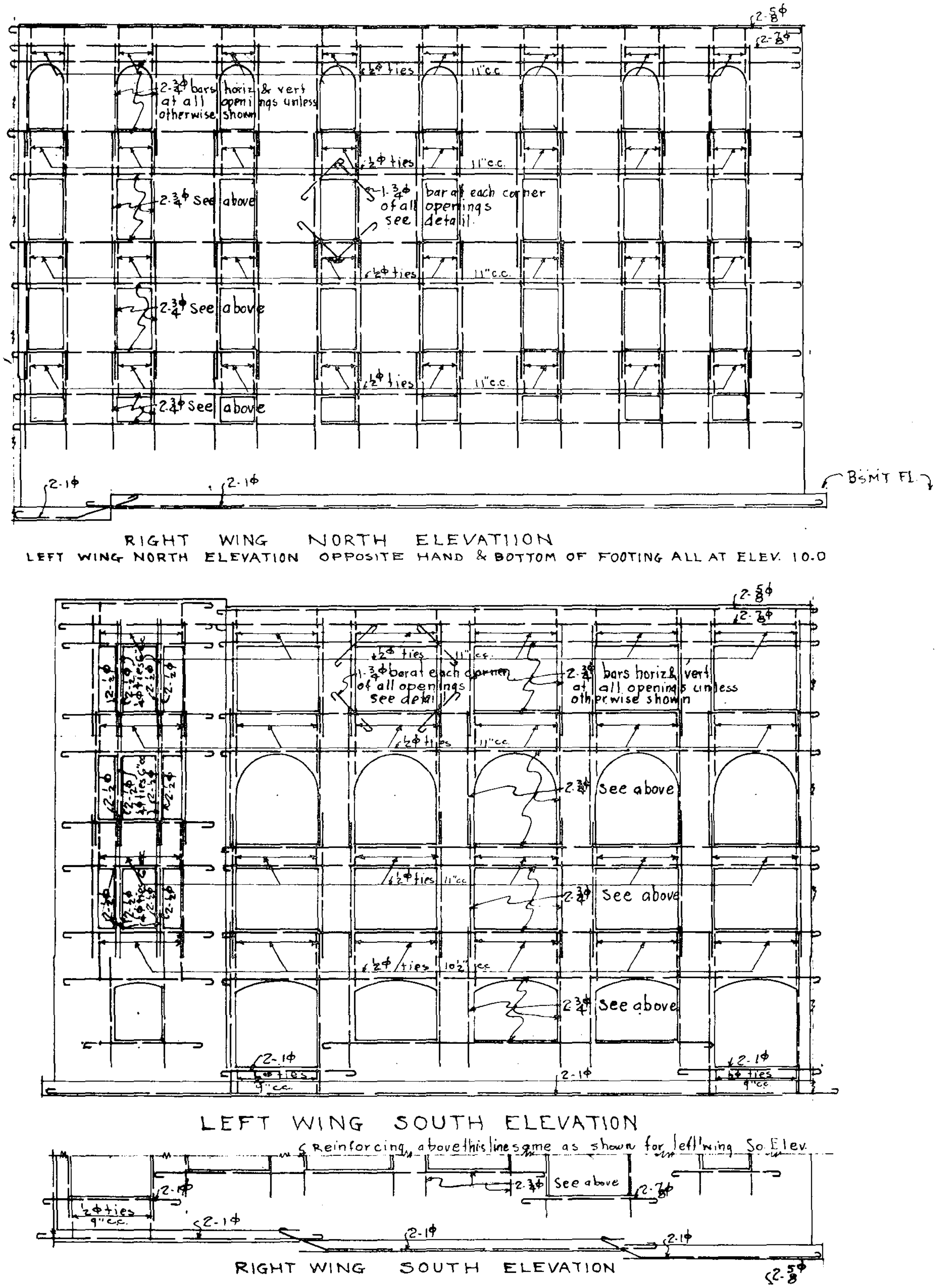
(c)

Fig. 2.4 Structural elevation of shear walls
(c) Wall C



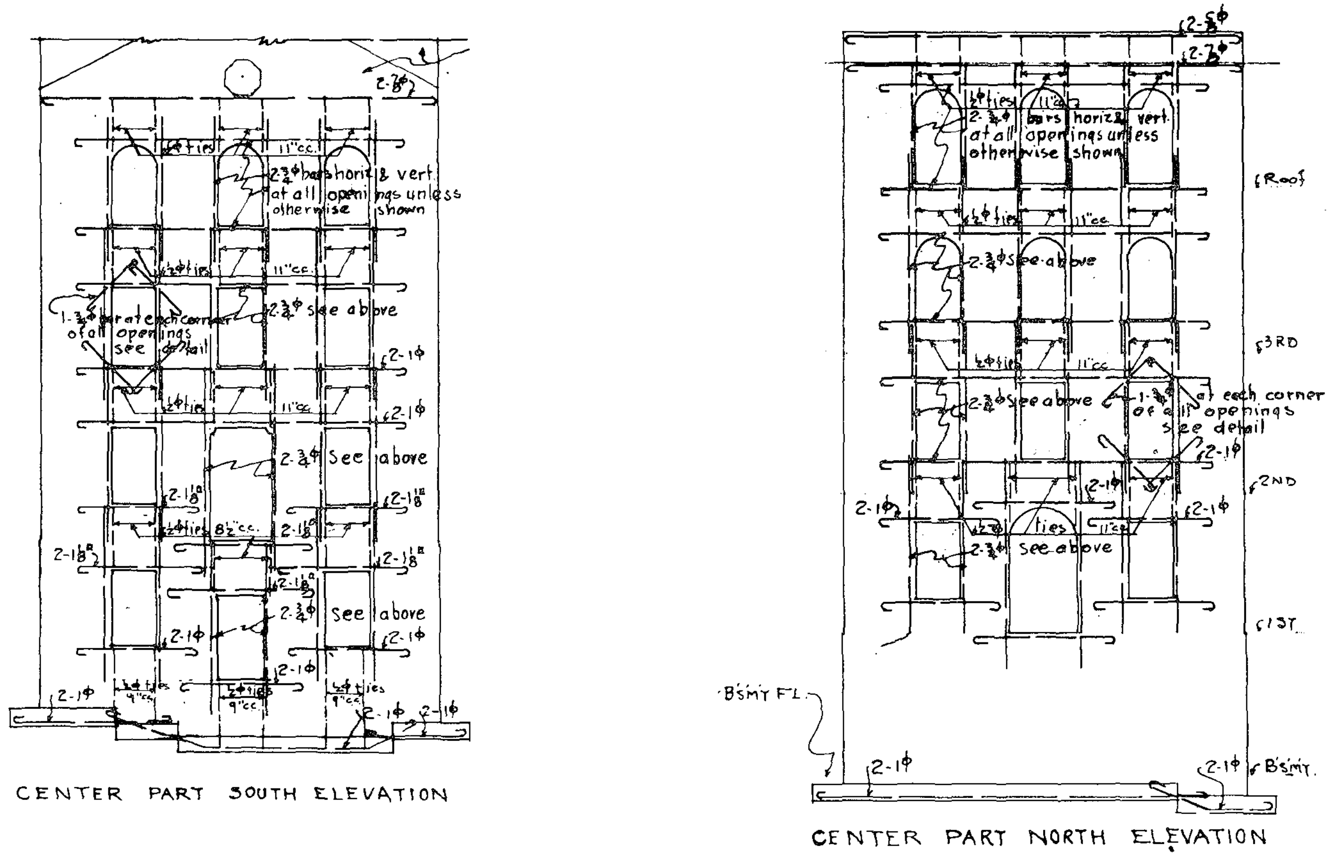
(d)

Fig. 2.4 Structural elevation of shear walls
(d) Wall between porch and building



(e)

Fig. 2.4 Structural elevation of shear walls
(e) North and South elevation



(f)

Fig. 2.4 Structural elevation of shear walls
(f) Center part north and south elevation

2.3.2 Floors

The basement floor slab was cast on grade (with bottom reinforcement 3/8" at 8" in each direction), and as such was not load carrying. The other three floor slabs consisted of ribbed slabs (h = 8" + 2½") within the building proper, and a solid slab in the front porch (h = 5¼", average). These slabs were supported on beams spanning between columns and bearing walls. Fig. 2.2 shows the structural layout of a typical floor. The roof slab was of similar construction but shallower, because of lower gravity loads. This design was also followed for the penthouse ceiling.

All columns were square or rectangular with minimum cross-sectional dimensions of 12" x 12", with ¼" ties at 8".

2.3.3 Note on Structural Symmetry

It can be seen from Figs. 2.2-2.4 that there are several types of structural asymmetry in the transverse direction of the building:

(1) Different openings in otherwise identical walls.

(2) Off-center plan location of the elevator shaft.

(3) Different foundation details and embedment depths for otherwise identical walls.

(4) Asymmetric embedment of the building due to sloping terrain.

These differences do not appear to be of sufficient importance to warrant foregoing the useful assumption of symmetry. This is particularly so in view of the very high stiffness of the longitudinal walls which effectively precluded any torsion of the structure due to excitation in the transverse direction.

2.4 Design Criteria

One of the major difficulties in performing a meaningful analysis of the earthquake response of Building 41 is the paucity of information on the criteria which governed the design and construction of the building. This is probably simply a reflection of the times; the architectural and structural drawings, and the computation sheets were obtained from the V.A., but they do not indicate what building codes were followed by the structural designers, nor the structural properties of the materials used.

Therefore, these points had to be inferred from the working stresses adopted, from the numerical coefficients used in key formulae, and from what is believed to have been the design practice during the second half of the 1930's (4). In the present case the task has been rendered even more difficult due to the fact that the V.A. did not have to follow, and apparently did not follow, the then applicable Uniform Building Code (UBC).

From the foregoing it was concluded that the provisions of the 1937 edition of the UBC (5) (or perhaps a somewhat earlier one, since they appear to be practically identical) most likely served to establish the minimum requirements for loading and materials.

As was the practice in those days, the structural analysis followed working stress procedures, and the structural members were designed for the combined effects of gravity and earthquake loadings.

2.4.1 Gravity Loading

It is evident from the physical description of the structure that most of the gravity loads consisted of structural concrete in the walls, slabs and footings. The unit weight of concrete was taken as 150 pcf. In table 2.1

the floor loads computed by the design engineer are compared with those evaluated for the purpose of the present study. Although there are some discrepancies, they are of no real significance since compensating inaccuracies appear to have been made by the designers in computing the weight of the concrete walls, leading to essentially identical gravity loads on the complete structure. There is one significant difference: the 1937 analysis included 20 pcf live load whereas the present analysis has not. This resulted from a change of approach in the UBC.

2.4.2 Lateral Forces

In accordance with then accepted practice, the lateral shear force acting on the building at a given elevation was determined from the following UBC formula (5).

$$F = CW \tag{2.1}$$

where F = the lateral shear, W = the total dead plus one-half the vertical design load at and above the elevation under consideration, C = lateral load coefficient specified in the code, its numerical value being dependent on the seismic zone and soil conditions. Pertinent excerpts of the 1937 edition of the UBC (5) are given in Appendix A.

According to that Code, C could have been taken as 0.08 for the site of building 41. However, for unstated reasons, a 10 percent coefficient was in fact adopted by the designers (a 25 percent increase).

TABLE 2.1

Comparison of Gravity Loads for Lateral Force Analysis

	1937*		1979*	
Level	Unit Load (PSF)	Total (KIP)	Unit Load (PSF)	Total (KIP)
PH	193	280	205	300
Roof	172	1720	180	1800
3,2,1†	234	2340	230	2300
Total		9020		9000
Ground		-		2700

* 1937 - Includes live load; 1979 - does not.

† 1st Floor load assumed identical.

It will be observed that there are several major differences between the seismic provisions of earlier versions of the UBC such as Reference 5 and those in force today,[6] three of which are listed below:

(1) The numerical value of C is now made dependent on the fundamental vibrational period of the structure, or an estimate thereof,[6] the type of the structural system (being indicative of its ductility) and the occupancy importance. For Building 41 these lead to a base shear coefficient of 0.28 (This point will be discussed in Chapter V.)

(2) Early versions of the UBC permitted a one-third increase in the stresses due to lateral loading. For comparative purposes, however, the net effect of this is somewhat lower. This is because the early versions included in W fifty percent of the design load, as already mentioned. Significantly, the $\frac{1}{3}$ increase was not taken advantage of by the structural designers of Building 41.

(3) For low rise buildings, the distribution of the lateral forces along the height now essentially follows an inverted triangular shape, compared with the rectangular distribution required by the earlier versions of the UBC. This leads, for an identical base shear, to a smaller shear envelope, a difference which may affect the forces in the connecting beams of coupled shear walls. The rectangular distribution also leads to a lower base moment, by approximately 25 percent.

2.5 Structural Materials

In trying to understand the performance of Building 41 in this destructive earthquake, it would have been very useful to have detailed information on the quality of structural materials in the building. However, the building was razed so that no relevant material studies are available and the drawings and available computation sheets are not particularly informative on material properties.

2.5.1 Reinforcing Steel

Two types of reinforcing bars are referred to in the structural drawings: (1) round deformed bars and (2) square bars. Their yield strengths are not given, but the working stress used was $f_s = 18,000$ psi. From the 1937 edition of the UBC it appears that the steel usually used for reinforcing bars was either intermediate grade billet steel or rail steel. Their working stress is given as: $f_s = 20,000$ psi. From ASTM Standard Specification for Billet Steel Reinforcing Bars (7) it is found that the yield stress $f_y = 40,000$ psi, whereas the yield stress for rail steel reinforcing bars, given in the relevant ASTM Standard (8) is 50,000 psi. These yield values apply to plain and deformed bars.

In the computations that follow, it is assumed that $f_y = 40,000$ psi, although the actual yield strength was probably somewhat higher, since f_y is more nearly a minimum value than an average.

2.5.2 Concrete

The structural drawings did not specify the nominal strength of concrete, f_c' ,used in the structure. The computation sheets are somewhat more informative. Although f_c' is not given explicitly, it may be concluded that $f_c' = 2,000$ psi. This conclusion results from the following considerations: (1) Young's modulus was given as $E_c = 2.0 \times 10^6$ psi, and in those days it was assumed that (5):

$$E_c = 1,000 f_c' \tag{2.2}$$

(2) The shear stress for beams with no web reinforcement, but with special anchorage of longitudinal steel, was taken as $v_c = 60$ psi. The 1937 UBC specifies:

$$v_c = 0.03\ f'_c \tag{2.3}$$

(3) The lever arm, j ,used for computing shear stresses and longitudinal reinforcement areas in beams was taken as 0.872 - 0.875. This value leads to somewhat lower extreme fiber stresses in compression than specified in the UBC. It is thus compatible with the foregoing assumptions.

(4) It is also understood that $f'_c = 2{,}000$ psi was the standard nominal concrete strength in the mid-thirties, and this was confirmed by a structural engineer working during that time in the Los Angeles area (4).

Whereas steel properties do not vary appreciably with time, this is not true of concrete. As is well known, both the strength and the stiffness of concrete increase with time. Therefore, the 33 years that elapsed from the time of construction until the San Fernando earthquake must have affected these properties appreciably. From the results quoted by Neville (9) it appears that the instantaneous compressive strength of concrete specimens may increase, after a very long period of time, by 2,000 - 3,000 psi over their 28 days value. Also, an increase of 35 percent in the compressive strength is permitted by the CEB-FIP recommendations after only one year (10). Moreover, the probable compressive strength is higher than the specified nominal value of f'_c which is, in practice, a minimum. It is therefore believed that $f'_c = 4{,}000$ psi, which is assumed in this study, is a realistic estimate.

2.6 Structural Modelling, Analysis and Detailing (1937)

This section is concerned with the design assumptions, analytical procedures and detailing adopted by the structural designers of Building 41 in so far as these are related to its earthquake resistance.

In their analysis the designers assumed that the lateral forces were resisted by the shear walls only, without any participation of the beams and columns acting as structural frames. This assumption is quite appropriate considering the relative stiffnesses and strengths involved, and the same assumption was made by the authors in the analysis reported below.

Thus, the forces in the N-S direction (see Fig. 2.2 for orientation) were assumed to be resisted by the six coupled walls, and those in the E-W direction by the three longitudinal walls (Fig. 2.4)

The distribution of the lateral forces among the walls in each direction was made on the basis of tributary areas related to the vertical load. The floor area tributary to a particular cross wall was computed on the assumption that the floor slab consisted of several beams each simply supported in the horizontal direction by the cross walls at its two ends. Today this approach is sometimes referred to as the flexible (but not continuous) diaphragm assumption.

Table 2.2 shows the lateral loads and story shear distribution among the three cross walls. The small disagreement with Table 2.1 is due to consistent rounding down of horizontal forces by the designer. Due to what appears to be an oversight in the original computations, the loading on wall C (in the central area) is underestimated by approximately 30 percent. It will be observed that if the appropriate correction is

TABLE 2.2

Lateral Shear Force Distribution Assumed by Designers

	Wall A		Wall B		Wall C	
Level	Force (KIP)	Percent	Force (KIP)	Percent	Force (KIP)	Percent
PH	-	-	-	-	16	100.0
3rd	23	23.3	33	33.3	43	43.4
2nd	56	26.8	80	38.3	73	34.9
1st	89	27.9	127	39.8	103	32.3
Base	123	28.5	175	40.5	134	31.0

made, the load distribution becomes more like that obtained when a rigid floor diaphragm is assumed.

In view of the relative slenderness of the floor slab taken as a horizontal beam, the large stiffness of the cross walls and the small number of stories in the building, the flexible diaphragm assumption may in fact be quite realistic.

The rigid foundation assumption made by the designers appears to be quite realistic in view of the high strength of the soil and the large plan area of the spread footings. Also, in view of the low level of lateral loading assumed, yielding of the soil as well as uplift could be ruled out. From the analytical point of view, foundation flexibility may affect the lateral load distribution among the walls, and make the assumption that the floors are rigid in their own plane somewhat more plausible. These effects will be discussed more fully in the following chapters.

It was quite difficult to follow the detailed lateral load computations carried out by the designer. Yet, it is interesting to note that moment distribution was used to evaluate the internal forces in the walls and in the coupling beams which were taken as equivalent frames, although - as is only to be expected - axial force effects and shear deformation, now known to be of importance, were ignored. Also, the fact that the longitudinal walls form wide flanges to the cross wall, i.e., the box effect, was overlooked. This effect, of course, tends to increase the flexural stiffness thereby leading to an even larger contribution of shear to the deflected curve of the structure.

It is thus seen that a substantial effort was made by the engineers to evaluate the internal forces in the structure, and that they used a relatively sophisticated structural model.

It is also of some interest to note that in the original calculations the fundamental period of vibration, T_1, in the transverse direction was estimated. This was done based on the roof displacement due to lateral forces proportional to the masses at every level. Significantly, this estimate was done separately for each of the two walls that were computed first (A and B). The intended purpose of that calculation is not clear. It can only be surmised that these estimates were made to check on the accuracy of the lateral force distribution among the cross walls, and that the substantial differences between the two frequencies obtained (T_1 = 0.51 sec for wall A and T_1 = 0.31 sec for wall B) may have indicated to the designers that the lateral force distribution used was only a crude approximation. They did not see the need to repeat the calculations for wall C, nor for the three longitudinal walls.

There is no doubt that the designers were cognizant of the importance of proper reinforcing details in earthquake resistant construction. As mentioned earlier, special reinforcement bars were detailed around openings, and ample anchorage was specified for the longitudinal and transverse reinforcement in the walls. Walls had boundary elements which were usually adequately reinforced both transversely and longitudinally. Coupling beams were reinforced top and bottom with equally spaced stirrups. The beam reinforcement areas followed roughly the values computed in the lateral load analysis, and as suggested earlier, were not optimally distributed along the height of the building. The structural drawings are clear, with sufficient details.

On the whole it appears that the designers were highly qualified and very conscientious.

2.7 Foundations

In contrast to the nearby and more recently constructed Olive View Hospital, there is very little geotechnical information relating to the Veterans Hospital site in San Fernando. It is known that old alluvial deposits occur in the hospital area: well graded and poorly consolidated deposits (fanglomerates) (11). Some indications as to the possible nature of the sub-soils may be obtained from the logs of the numerous bore holes drilled in 1976 about one and a half miles to the west near the Olive View Hospital (12).

From the bearing stresses allowed by the designers (up to 8,000 psf), and the seismic refraction tests carried out about 500 ft N-E of Building 41 (10), it is evident that the building was founded on good foundation materials, with practically no potential for liquefaction.

The extensive geological survey carried out in the San Fernando earthquake area following the 1971 event did not reveal any active faulting at the building site. The nearest approach of the Veteran's fault, a secondary fault of the San Fernando earthquake, is approximately 1 km from Building 41.

The soil properties assumed for the present study, presented in Fig. 2.5, were based on the results of the seismic refraction test carried out by Duke, et al. (11) at the hospital site. It is known that the shear wave velocity, v_s, in the soil is strain dependent. It is also known that for soil compliance computations the effective depth of the soil beneath the footings depends on the stress distribution in the soil, so that the effective depth

SITE: U.S. VETERANS HOSPITAL, SYLMAR

No. 1

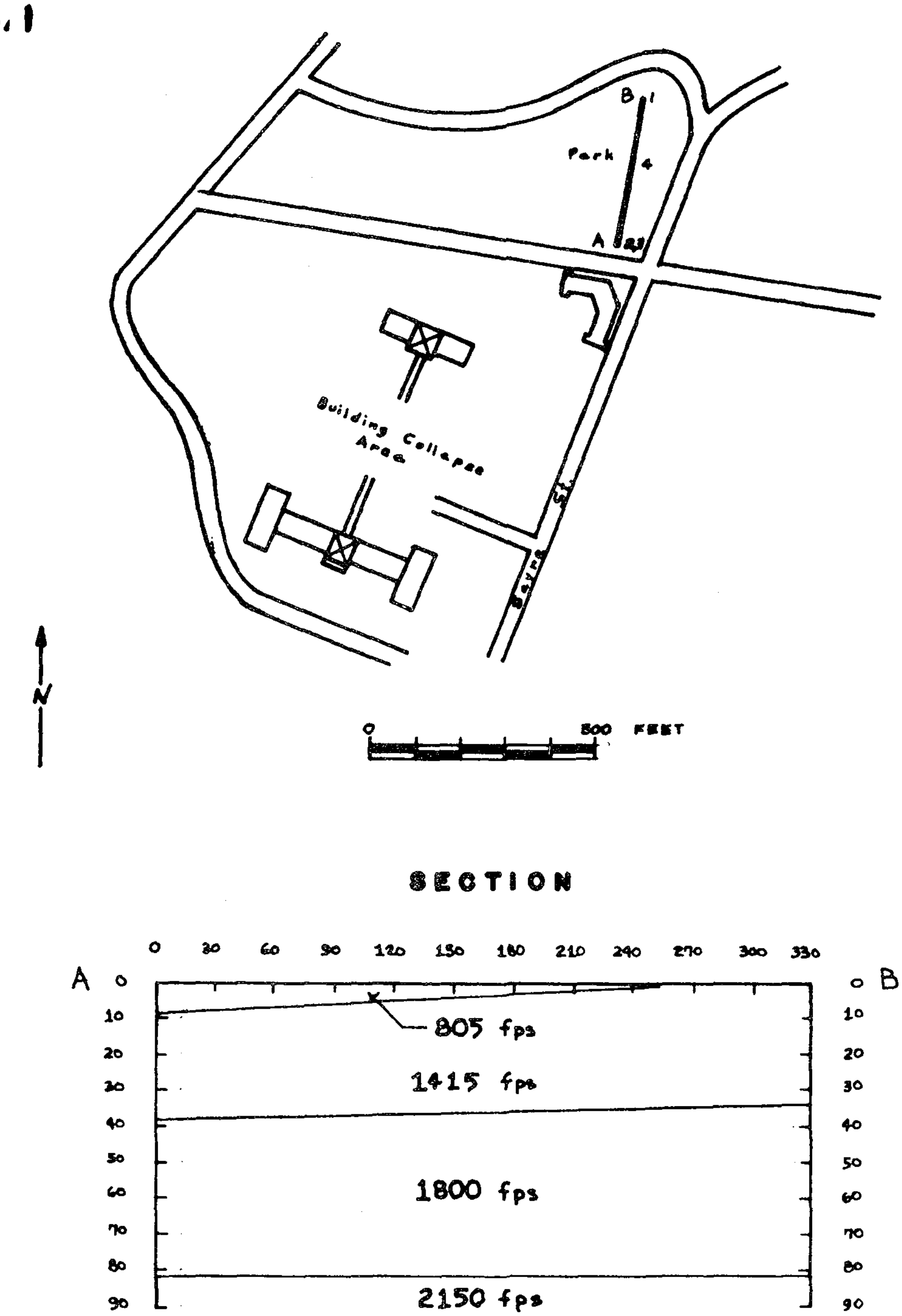


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Fig. 2.5 Shear wave velocity profile at the Veterans Hospital site (Ref. 11) for rocking is smaller than that for horizontal motion. However, these differences are not major, and to simplify matters it was assumed that $v_s \cong 1000$ ft/sec at all strain levels both for rocking and for lateral motions. It was also assumed that the unit weight of soil $\gamma \cong 110$ pcf. Using the expression

$$G = \gamma v_s^2/g$$

where G = shear modulus of the soil and g = acceleration of gravity we have $G \cong 3.6 \times 10^6$ psf. A Poisson's ratio of $\nu = 0.33$ was also assumed; this is consistent with the v_s/v_p ratio (v_p = compression wave velocity) given in Ref. 11 for near surface soils of similar properties.

III. STRUCTURAL DAMAGE

Following the Feb. 9, 1979 San Fernando earthquake, the Veterans Administration authorized a firm of consulting engineers (13) to assess the earthquake damage to buildings and other structures in the hospital complex. Other interested parties (14,15) also inspected the damage. The following description of the damage to Building 41 is taken directly from Ref. 13 which is the most detailed study.

The frame was found to be essentially intact. Some walls had diagonal hair-line cracks. The basement slab, which was cast on grade over the spread footings, had a 1/16" wide continuous crack running parallel to, and 6 ft south of the north exterior wall. One to three inches of downward displacement of the adjacent grade was observed on the north and on the west sides of the building.

A later visual survey to assess additional damage sustained during the numerous aftershocks that followed the Feb. 9, 1979 event was also carried out by the same consulting engineers (13). They found that the longitudinal crack in the basement floor slab had widened slightly. Also, an additional hairline crack was observed in the north exterior reinforced concrete wall on the first floor towards the east end.

From the foregoing description it appears that the tensile stresses in the concrete walls were not very high, and that major excursions of the coupling beam reinforcement into the post elastic range did not take place. It is therefore believed that using a linear mathematical model to describe the behavior of the superstructure during the earthquake may be quite realistic.

It will be observed, however, that the information regarding the location and pattern of the diagonal hairline cracks is not sufficiently detailed to identify critical regions in the concrete walls. Yet, it does suggest that the diagonal tensile strength of concrete in some areas of the walls may have been exceeded.

The continuous crack in the basement slab which widened following the aftershocks, as well as the appreciable downward displacement of the grade suggest that the foundations along the north end of the building may have moved. In this regard it should be noted that damage in the V.A. Hospital complex was precipitated by intense ground motion rather than by localized surface faulting [(see e.g., (15)] so that permanent ground displacements do not appear to have played a significant role in the damage.

IV. GROUND MOTION

In the absence of an actual record on the site or in the vicinity, it is impossible to reconstruct the high frequency components of the ground motion at the V.A. Hospital site which are important for this study. However, several studies of the possible character of the ground motion in this general area have been performed, for example, in connection with studies of the collapse of the neighboring Olive View Hospital (see Fig. 1.2). These studies afford some insight into the general features that may have characterized the ground motion at the V.A. Hospital site.

One of the simplest and most common indicators of strong ground motion is the peak acceleration. A summary of measured or inferred peak accelerations during the San Fernando earthquake at several sites in the vicinity of the V.A. Hospital, including Olive View Hospital (12), reveals a range of values from .50g to 1.25g depending on the site and associated conditions. A later study by Mahin, et al. (16) suggested a value of 0.65g for the peak surface acceleration for the Feb. 9, 1971 event at the Olive View Hospital site.

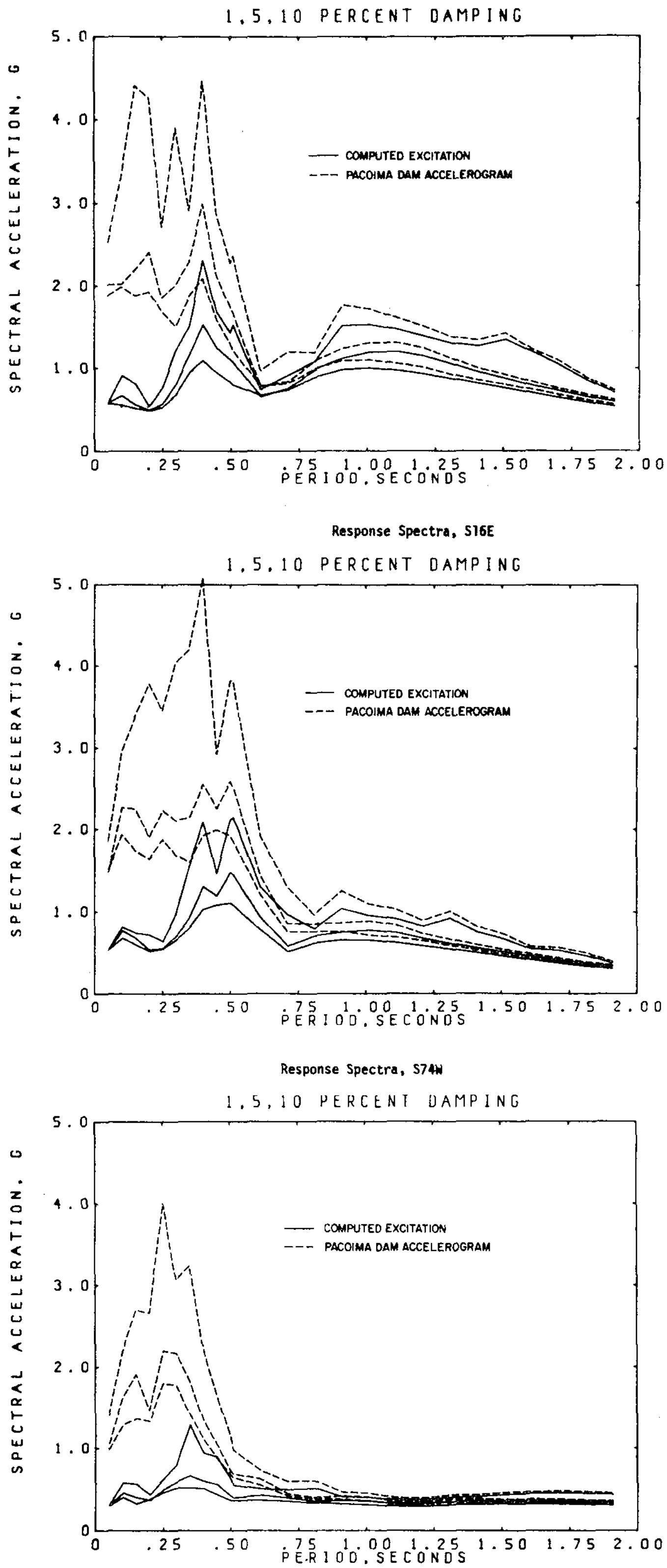
Although useful as a rule-of-thumb, the peak acceleration is not as important as other features of the ground motion such as the frequency content in the range of the structural frequencies and the possible presence of large acceleration pulses of long duration such as seen in the Pacoima Dam record (16).

The records of ground motion which are likely to shed some light on the shaking that took place at the V.A. Hospital site are, due to their proximity, the Pacoima Dam accelerogram, the Lower Van Norman Dam seismoscope trace (4 miles S.W., see Fig. 1.1) and the Holiday Inn accelerogram (8 miles S.S.W.). However, as will presently be suggested, none of these records is directly applicable to the V.A. Hospital site.

Studies by Reimer (17) show that the Pacoima Dam record was strongly affected by the particular features of the site of the accelerograph. The base rock excitations required to produce the Pacoima Dam accelerogram computed by Reimer (Fig. 4.1) differ in many respects from the accelerograms which were recorded on a ridge above the crest of the dam. In particular, the acceleration ordinates in the high frequency range of the spectrum are dramatically lowered. The fundamental vibration period of Building 41 was quite low (T_1 = 0.14 sec, rigid base), and is in this range. The applicability of the derived spectra in Reference 17 may be questioned in view of their dependence on simplified modelling assumptions. However, Reimer's results suggest that the high accelerations in the low period range of the Pacoima Dam spectra should not be used to represent motion in the valley floor. This suggestion is supported by the work of Wong and Jennings(18) who studied the effect of canyon topography upon the Pacoima Dam accelerogram.

The difficulties in obtaining reliable accelerograms from seismoscope traces are well known [(eg., (19)], and this is particularly so in the case of the lower Van Norman Dam trace, where strong high-frequency signals were present. Acceleration spectrum estimates of 1.5 - 2.0g in the period range of 0.15 to 0.25 seconds were indicated by Scott (19) at Van Norman Dam on the basis of deconvolution of the seismoscope response. The peak ground acceleration was estimated to be in the range of 0.7 - 0.8g.

The Holiday Inn strong motion accelerogram was recorded about 8 miles to the south of the Veterans Hospital. The most important problems associated with using this record are that the high frequency range of the spectrum is very sensitive to distance, and that the Hospital site is located on the



Response Spectra, Down

Fig. 4.1 Pacoima Dam acceleration response spectra: Actual vs. derived base rock (Ref. 17) other side of the fault. In view of these limitations, the applicability of this record to the V.A. Hospital site is also questionable.

Regarding the large acceleration pulses of long duration in the Pacoima Dam record, it was pointed out by Mahin, et al. (16) that these pulses were also present in the Lower Van Norman Dam record derived from the seismoscope by Scott (19). Mahin, et al., concluded that similar features must have been present in the ground motion at the Olive View Hospital. This is also consistent with recent seismological studies of the ground motion done by Heaton (20). It is thus reasonable to assume that the long acceleration pulses also characterized the motions at the V.A. Hospital site. As was the case in the analysis of Olive View Hospital, it was expected that the large pulses might well be the controlling feature in any nonlinear effects in the response of Building 41.

In summary, it is suggested that the spectral amplitudes in the high frequency range can only be established within a broad range. The particular records that might be chosen to simulate the Feb. 9, 1971 event at the V.A. Hospital site for the purpose of nonlinear soil-structure interaction analysis should preferably include acceleration pulses of long duration as well as significant energy at high frequencies. The levels of amplitude of acceleration spectrum ordinates in the period range of interest (0.1-0.3 sec) are estimated to be in the range from about 0.8g to over 1.5g. Depending on the damping and the mode shapes of the structure, this indicates a base shear on the order of 50 to 100 percent of the weight of the structures.

V. EQUIVALENT LATERAL FORCE ANALYSIS

5.1 Modelling Assumptions

In this chapter a code-oriented, static, lateral force analysis is presented and discussed. The main aim of the analysis is to estimate the lateral capacity of the structure on the basis of conventional, simplified procedures.

The analysis is based on the requirements of the 1976 edition of the UBC (6) with the exception of the provisions relating to soil-structure interaction where a somewhat different approach was followed.

It has already been stated that, in view of the greater strength of the building in the longitudinal direction, only the response in the transverse direction has been considered.

Although the structural layout of Building 41 is relatively simple, a linear analysis - either static or dynamic - is not a trivial task due to several complicating factors:

(1) A basic assumption usually made in the lateral load analysis of building structures, namely, the in-plane rigidity of the floor slab, is not strictly valid, in view of the small number of floors, the large aspect ratio (length/width) of the floor slabs and the high stiffness of the cross-walls.

(2) The cross-walls are basically low-rise coupled shear walls of irregular configuration, with some deep connecting beams. These are not easy to model due to large effects of shear and the strong effect of the assumed stiffness of the connecting beams on the overall response. In particular, there is an abrupt change in the geometry of the cross-walls enclosing the central area of the building (Walls C, Fig. 2.4) above roof level.

(3) Some interaction between adjacent cross-walls through the longitudinal walls acting as wide flanges (box effect) is to be expected.

(4) Foundation flexibility and interaction between walls through interconnected foundations may be a significant factor.

(5) The minor structural asymmetries, already noted, are difficult to model and even with crude simplifying assumptions, estimates of their effects require a substantial increase in the computational effort.

It is thus evident that even for a structure of this type, a conventional lateral load analysis carried out by means of hand computations must be based on a somewhat simplistic mathematical model. However, this is the nature of virtually all such calculations, and is the price paid for the advantages of a brief, straightforward analysis.

The simplifying assumptions made for the purpose of this analysis are nevertheless believed to lead to results that are consistent with the more detailed analyses which are described in Chapter VI. The assumptions are:

(1) The structure is symmetric about the centerline of the floor plan.

(2) All the lateral loads are resisted by the cross-walls.

(3) The relative rigidity of the walls is computed assuming equal lateral displacements at the roof level only.

(4) The lateral load distribution among the walls is assumed to follow approximately the distribution given in the 1976 edition of the UBC, irrespective of their different stiffness variations along the height of the building.

(5) Any interaction between adjacent cross-walls through the longitudinal walls and interconnected footings is neglected.

(6) The local effects of the penthouse are ignored, i.e., the building is assumed to be four stories in height and shears and moments from the penthouse are added at the roof level.

(7) The assumptions of a continuous medium, as applied to coupled shear walls, are used, i.e., it is assumed that irregularities can be averaged, and that the continuum approach is applicable to the four-story structure.

(8) Shear deflections were assumed to be proportional to the net area (i.e., excluding openings) of the webs in each wall.

(9) Strength of materials formulae for shear and bending stresses are assumed valid even for members with small aspect ratios.

(10) The stiffness properties of the soil are approximated by using ATC 3-06 (21) formulae.

(11) The material properties are as assumed in Chapter II.

Some comments on these assumptions now follow. As already suggested, the effect of structural asymmetry on the load distribution among the walls is not believed to be large, although the internal forces are likely to be affected to a greater extent. Probably the most controversial assumptions are those regarding the load distribution among the walls and the use of the continuous medium procedure for the analysis of the coupled shear walls. The load distribution among the walls depends on their relative stiffness and the variation thereof along the height of the building, the in-plane rigidity of the floor slabs relative to the walls, and the mass distribution in the building. If all the shear walls had similar deflected shapes for similarly distributed lateral loadings, i.e., proportional stiffness matrices, and if the floor slabs were much stiffer in their own plane than the walls, then assumptions 3 and 4 would hold exactly. The deflected shape of low rise coupled shear walls with nearly constant cross-sections throughout the height and with substantial flanges, as is the case in Building 41, are likely to be similar under similarly distributed lateral loadings, so this portion of the assumption is not believed to be a problem. Neglecting the in-plane flexibility of the floor slabs - the so-called rigid diaphragm assumption - is quite realistic for high rise buildings, and was also shown to hold for some frame-supported low-rise buildings with floor plans having large aspect ratios (22). However, when such structures also have numerous and rather stiff shear walls, this assumption is no longer self-evident. To clarify this point, a simplified analysis was carried out, and it was found that even in this extreme case the errors resulting from this assumption are not excessive. Appendix C summarizes the main findings of that analysis.

The interaction of walls through common flanges and footings is not likely to be strong due to the large distances between neighboring cross-walls and the large number of openings in the longitudinal walls. Nevertheless, it is quite difficult to assess the importance of an incorrect assumption regarding effective flange widths. The computer analysis described in the following chapter sheds some light on this problem, and a more detailed discussion is made there. Differences in foundation flexibilities (rocking as well as lateral) are again likely to affect the load distribution between the walls, particularly when foundation soils are soft and the stiffnesses of the footings are not proportional to those of the walls they support.

The use of continuous medium procedures for the analysis of low-rise coupled shear walls, which are in addition not particularly uniform, is perhaps a major source of error in the computation of internal forces, although this is unlikely to be the case for the purpose of load distribution among the cross-walls. It was felt, however, that the method was the only analysis which could be performed by hand with the minimum computational effort. This simplified approach, which also reflects the first author's personal predilection, was found to be justified when compared with results of more detailed analyses presented in the following chapter.

5.2 Lateral forces on Shear Walls

In this section the distribution of the lateral forces over the height of the building in accordance with the 1976 edition of the UBC is first determined. Then the distribution of these forces among the three cross-walls is discussed. For ease of reference the relevant seismic provisions of the Code are reproduced in Appendix B.

For regular buildings the Code requires that the total lateral force V (base shear) be distributed over the height of the structure as follows:

$$F_x = \frac{(V-F_t)W_x h_x}{\sum_i W_i h_i} \tag{5.1}$$

where the terms are defined as in Appendix B. Since the structure is very stiff (fundamental period of vibration $T_1 < 0.7$ sec.) $F_t = 0$.

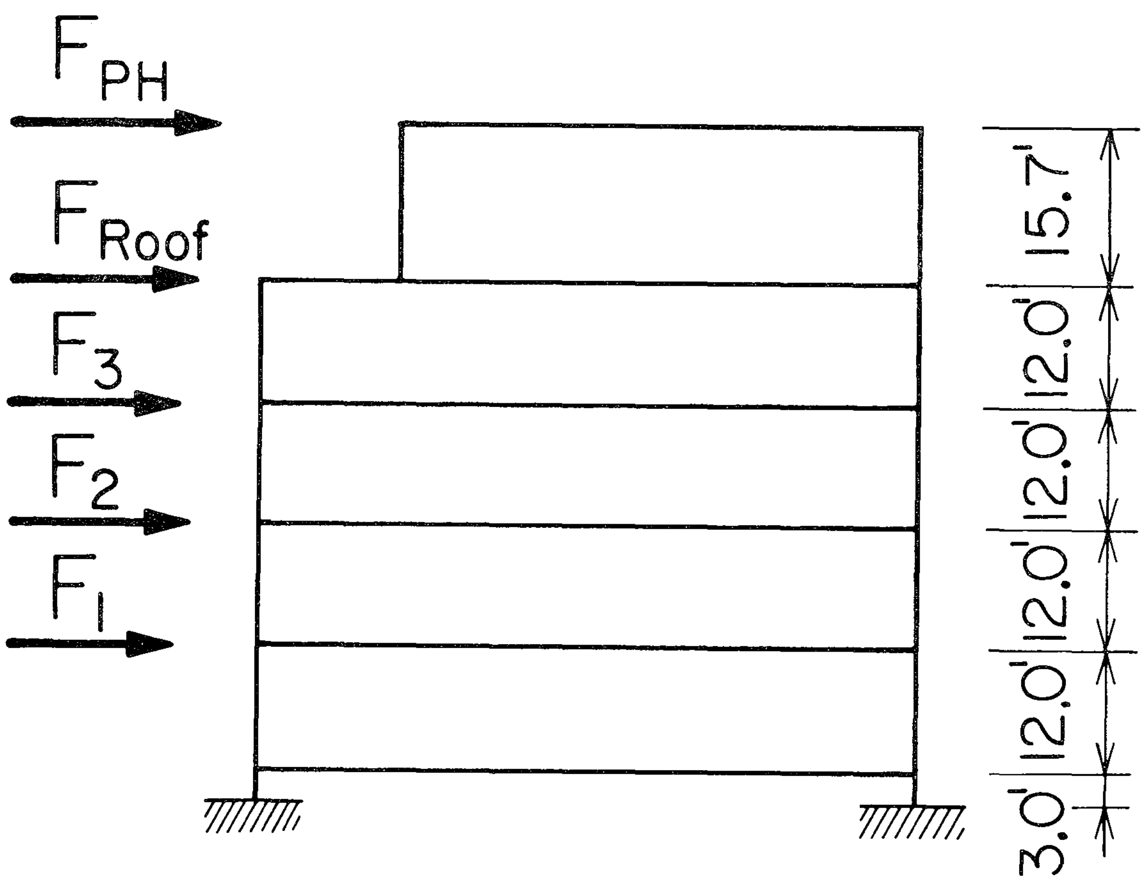
The distribution of the lateral loads per Eq.5.1 is given in Table 5.1. Note that for convenience at this stage in the calculation V is taken equal to the total gravity load on the building W. In order to compute the actual minimum lateral loads required by the 1976 Code it is necessary to apply the following formula:

TABLE 5.1

Lateral Force Distribution Over Building Height: Approximate Analysis

Level	F_x*(Kip)	Shear (Kip)	Moment Kip/ft
PH	605	605	0.0
Roof	2770	3375	9500
3	2710	6085	50000
2	1875	7960	123000
1	1040	9000	219000
0			354000

*These forces are arbitrarily scaled so the base shear equals the weight of the building.



$$V = ZIKCSW \tag{5.2}$$

where again all the coefficients are as defined in Appendix B. Since the spectral coefficient C is dependent on the fundamental period of vibration, T, it is first necessary to estimate the period

$$T = \frac{0.05h_n}{\sqrt{D}} = \frac{0.05 \times 51}{\sqrt{49.0}} = 0.36 \text{ sec} \tag{5.3}$$

Note that h_n = h (roof), and 49 ft is an average value for the planar dimenstion of the building in the transverse direction. Next C is found

$$C = \frac{1}{15\sqrt{T}} \cong 0.11 \tag{5.4}$$

Letting z = 1.0 (Zone No.4), k = 1.33 (box system), I = 1.5 (essential facility), CS = 0.4 (maximum value assumed in the absence of accurate foundation related data), and substituting in Eq. 5.2 we have:

$$V/W = 1.0 \times 1.5 \times 1.33 \times 0.14 = 0.28 \tag{5.5}$$

and it follows that the forces in Table 5.1 are to be multiplied by a factor of 0.28. It will be observed that this value is three and one-half times the shear coefficient stipulated by the 1937 edition of the UBC. It is, of course, recognized that part of this large numerical discrepancy is due to differences in structural philosophy. However, a common basis only reduces the ratio of the two lateral load coefficients to about 3.0, as can be seen from the following calculation. Assuming a material factor of 1.4, a capacity reduction factor of 0.9, allowing

33 percent increase in stress, and noting that 50 percent of the design load is roughly equivalent to 10 percent of W, this ratio becomes (for steel reinforcing bars):

$$R = \frac{0.28}{0.08} \frac{18,000 \times 1.33 \times 1.4}{40,000 \times 0.9 \times 1.1} \cong 3.0$$

It is thus seen that the present code does represent a substantial increase in the lateral force coefficient for publicly important box-like structures with difficult foundation conditions. Note that the importance factor and the soil factor together contribute a factor of 2.25, which accounts for most of the difference. These factors were introduced following the San Fernando earthquake.

In the following section the distribution of these forces among the three transverse walls is discussed.

5.3 Analysis of Walls

From Fig. 2.4, it is seen that in every wall the pattern of openings is relatively regular, with the exception of wall C which has an abrupt change in geometry at roof level. This regularity suggests that, as a first approximation, the structural system could be modelled by means of the continuous medium representation for coupled shear walls, as discussed previously (e.g., Ref.23). In view of the uncertainties regarding the correct effective width of the longitudinal walls acting as flanges to the cross-walls (shear-lag effects) the relative stiffness of the walls was calculated with two alternative flange widths. Fig. 5.1 shows the cross-sectional dimensions of the three walls for each of the two assumptions.

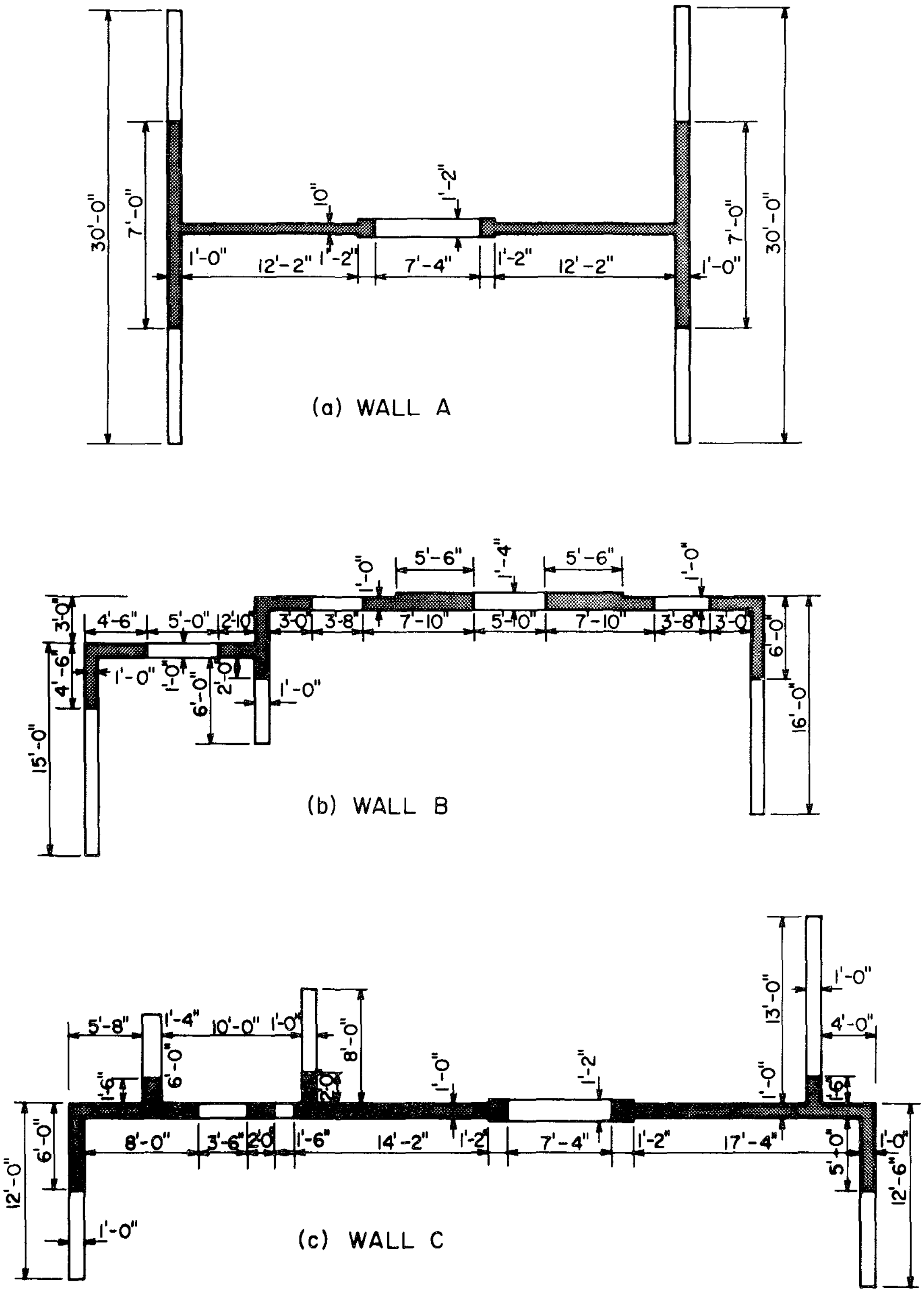


Fig. 5.1 Cross-section of walls as assumed in analyses
(a) Wall A (b) Wall B (c) Wall C

Graphical presentations of the solution to the differential equation governing the static behavior of coupled shear walls are readily available (24,25,26). These, however, were computed for walls with a single row of openings (or for walls with two symmetrically located rows), and thus are strictly applicable to wall A only. In order to avoid the solution of simultaneous differential equations as would be required for the "exact" analysis of walls B & C, average values were computed for the stiffness parameters, and these were used as inputs for the design chart. Using the design charts of Coull and Choudhury (24, 25) the relative stiffness of the walls was evaluated for the two alternative flange widths based, as mentioned above, on the deflection at roof level.

The results are presented in Table 5.2. The good agreement between the results of the several alternatives shown does not prove, of course, that they are "correct." It does show, however, that the load distribution among the walls is not very sensitive to the relative effects of shear and flexure.

The fundamental period of vibration of the building, also given in Table 5.2, was computed based on a straight line deflected shape. However, in view of the large proportion of shear-induced displacements, the numerical values T_1 = 0.140 sec for the narrow flange assumption and T_1 = 0.123 sec for the wide flange, probably underestimate the true period.

Note that these periods are appreciably lower than the period computed earlier using the code formula, Eq. 5.3. This is to be expected considering the large number of wide shear walls in this building. The effects of foundation compliance, also shown in Table 5.2, are discussed in the following section.

TABLE 5.2

Lateral Shear Force Distribution Among Walls (Percent); and Natural Period (Sec.)

Wall	Narrow Flanges			Wide Flanges			Original Analysis (Table 2.2)
	B	B+S	B+S+F	B	B+S	B+S+F	B
A	25	25	21	31	27	27	29
B	27	29	29	25	29	29	40
C	48	46	50	44	44	44	31
Period T_1		0.140	0.233 0.243(1)		0.123	0.204 0.213(1)	

B = Bending deformation: S = Shear deformation.
F = Foundation compliance.

(1) Including mass and rotary inertia of foundation.

5.4 Soil-Structure Interaction

The effects of soil-structure interaction on the load distribution among the walls and on the natural period of vibration are reported in this section and follow, as far as the stiffness coefficients are concerned, the procedure recommended by the ATC study (21). Briefly, the lateral (swaying) stiffness of the foundation K_y and its rocking stiffness K_θ are computed on the basis of the well known static formulae for the displacements of a rigid circular disc on an elastic half-space. Special expressions are given in Reference 21 for evaluating the effective radii for non-circular footings. The formulae read:

$$K_y = \frac{8Gr_a}{2-\nu} \quad ; \quad r_a = \sqrt{\frac{A_o}{\pi}}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)} \quad ; \quad r_m = \sqrt[4]{\frac{4I_o}{\pi}} \tag{5.6}$$

Where A_o = plan area of the base, I_o = moment of inertia of the area of the base about a horizontal centroidal axis normal to the direction in which the structure is analyzed, G = shear modulus of soil beneath the foundation (36×10^6 pcf) and ν = Poisson's ratio (0.33). Using these formulae, the additional deflection of each wall at roof level was computed, and its relative stiffness evaluated. The alternative effective widths shown in Figure 5.1 were considered with results given in Table 5.2. Again it is seen that the load distribution is not affected to an appreciable degree by the assumed effective widths of the longitudinal walls and footings beneath them.

The effect of soil compliance on the fundamental period of vibration is very noticeable. This is a direct result of the more than 50 percent reduction in lateral stiffness of the building compared with the fixed base case.

In the Table, two values of T_1 are given for each case. The larger value is obtained when the mass and rotary inertia of the foundations are included. The smaller value results when these inertia effects are ignored. The two values were computed by means of Dunkerley's formula, namely:

$$T^2 = T_0^2 + T_y^2 + T_\theta^2 \tag{5.7}$$

where T_0 = fixed-base vibration period, T_y = swaying period of a rigid body on elastic foundation, and T_θ = rocking period of that body on elastic foundations.

5.5 Lateral Load Capacity of the Structure

In the present analysis we are interested in the lateral load that the structure could resist without appreciable cracking and with only limited excursions of the reinforcing steel into the plastic range. The desire to find this level of capacity follows from the very minor damage observed in the structure after the San Fernando earthquake.

The critical states considered were 1) tensile capacity of the reinforced concrete walls, 2) shear capacity and yield strength of the connecting beams, and 3) foundation failure, including effects of incipient overturning.

The reinforced concrete strength criteria used followed the ACI recommendations (1) in particular: the tensile strength of the concrete was taken as $f_t = 6\sqrt{f_c'}$ (in psi). The capacity of the soil in sliding was not considered critical.

Although it is satisfactory for calculating external force resultants, the computation of the internal forces in low-rise buildings by means of the continuum approach unavoidably leads to further approximations due to the sensitivity of the shear forces in the connecting beams to the assumptions inherent in the method. To this, one should add the effects of uncertainties regarding the support conditions of the entire structure, the effective stiffness and the degree of end fixity of the beams themselves, as well as the effects of restrained shear deformations in the walls. These and other limitations discussed earlier reinforce the characterization of the analysis as approximate.

Table 5.3 summarizes the results of the analysis based on the wide flange alternative. First yield is indicated when the base shear reached a value of 15-20 percent of gravity. The coupling beams of wall B were found to be the weakest link in the structure and they were evidently under-reinforced. If vertical shear redistribution among the floors as well as load redistribution among the walls were permitted, a lateral load capacity of 35-45 percent of gravity could be reached. Obviously, an appreciable load redistribution must entail major changes in the relative stiffnesses of the walls, and probably requires ductile response of the under-reinforced coupling beams. As noted earlier, evidence of ductile response was not observed in the structure after the San Fernando earthquake. Without further analysis it is difficult to assess the extent of cracking to be expected with the redistribution assumed in these approximate calculations. However, it is believed that only limited excursions into the non-linear range might permit appreciable redistribution of forces among the cross-walls.

As expected, the capacity of the building against incipient overturning gives the upper bound on the resistance to lateral forces. It is of the order of 45-50 percent of gravity, and if the walls are assumed to act independently (redistribution) they can carry somewhat more. The overturning

TABLE 5.3

Lateral Force Capacity of Building 41

(Approximate Analysis)

	First Yield	Cumulative*	Overturning (Minimum)	Overturning & Redistribution
Percent of Weight	15-20	35-45	45-50	50-55

*Yield & redistribution.

capacity was computed on the basis of the vertical load tributary to each wall. Walls A and C were assumed to rotate about a horizontal axis through the center line of the footings forming the flanges of the foundation system. Wall B was assumed to rotate about an axis through the center line of the contact area between the soil and the footing as computed by means of Terzaghi's bearing capacity factors (27), a procedure recently applied by Meek (28). For this purpose, a conservative value of 10,000 psf was assumed. Figure 5.2 explains the approach.

In summary, the approximate analysis indicates that the lateral load capacity of Building 41 at the observed level of damage was in the range of 35 to 50 percent of gravity. A higher capacity with comparable damage could have been easily achieved with only token additional reinforcement in critical regions.

The comparison of this result with the much higher probable strength of ground motion at the site during the San Fernando earthquake is made in the following chapter, where the results of a computer-aided analysis of the structure are also discussed. With such an analysis there is less need to resort to arbitrary assumptions regarding the distribution of external forces among the resisting walls, and of internal forces following local yield.

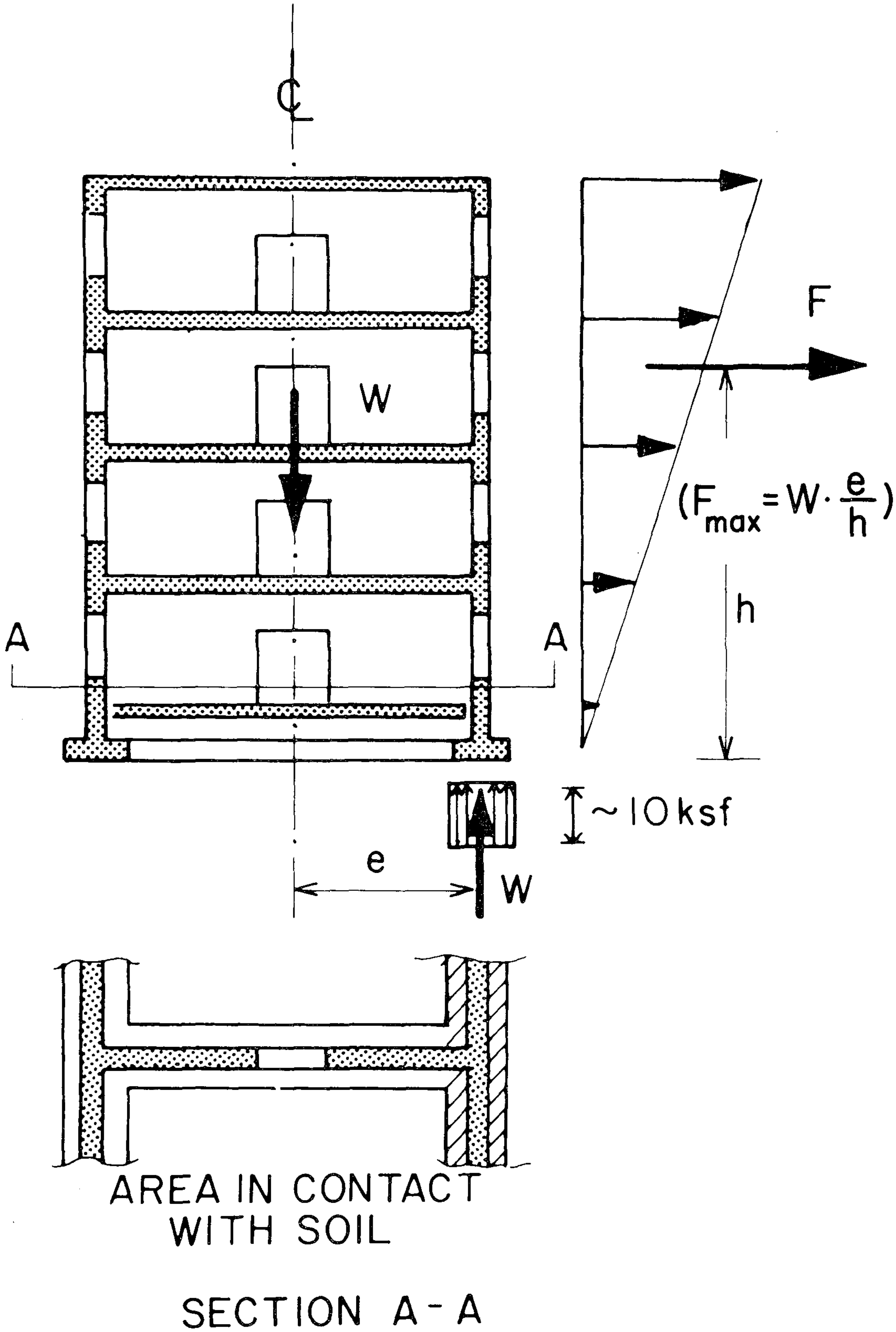


Fig. 5.2 Equilibrium at incipient overturning.

VI. LINEAR DYNAMIC ANALYSIS

6.1 Introduction

In this chapter a model of the building suitable for a computer-aided analysis is described. Its dynamic characteristics and dynamic response were evaluated by means of a standard computer program and these are compared with the results discussed in Chapter V. As a check on the computations, a simplified mathematical model of wall A was isolated from the rest of the structure, and its dynamic properties were compared with the results of the analysis of the entire structure. This simplified single-bay frame is used as the structural model in the nonlinear analysis presented in Chapter VII.

6.2 Computer Program

The program ETABS, Three-Dimensional Analysis of Building Systems, (Extended Version) (29) was used in this analysis. It performs linear elastic analyses of frame and shear-wall buildings under static loading and under lateral earthquake loadings. The following description of the program is a partial quotation from the Abstract to Reference 29: "The building is idealized by a system of independent frame and shear wall elements interconnected by floor diaphragms which are rigid in their own plane. Within each column bending, axial and shearing deformations are included. Beams and girders may be nonprismatic and bending and shearing deformations are included. Also, shear panels can be considered. Finite column and beam widths are included in the formulation. Nonsymmetric, nonrectangular buildings which have frames and shear walls located arbitrarily in plan can be considered. Three independent vertical and two lateral static loading conditions are possible. The static loads may be combined with a lateral earthquake input which is specified as a time-dependent ground acceleration or as an acceleration spectrum response. Three dimensional mode shapes and frequencies are evaluated."

In this program two basic assumptions are made: (1) floors are rigid in their own plane and (2) lateral forces act at floor levels. The problems associated with the first limitation have already been discussed. The second assumption is an acceptably realistic approximation for the inertia forces, even in the present case where the mass of the walls is relatively high.

A more fundamental difficulty with the program is the need to model low rise shear walls as beams*. Also, the continuous transfer of stresses along a common vertical edge of two walls meeting at an angle is poorly modelled. Apart from the problems resulting from the finite depth of structural members, i.e., linear stress distribution irrespective of aspect ratio, increased stiffness on the one hand and joint panel zone distortations on the other, there are the known difficulties associated with assigning stiffnesses to reinforced concrete members under stress gradients. Whereas the latter effect was not considered to be a problem for the walls, which apparently did not crack, the connecting beams - which were much shallower

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*The one story high panel representation of shear walls which is available in ETABS does not appear to be an appreciable improvement on the beam element for the type of problem encountered in modelling Building 41, although shear deformations are somewhat better modelled. It also leads to some increase in the computational effort.

and more heavily reinforced - may have cracked without this effect becoming noticeable on visual inspection.

In the following section the modelling of the structure is described.

6.3 Structural Model

The model was designed for an analysis only in the transverse (N-S) direction, since the combined effects of inputs in two perpendicular directions were not believed to be critical for this structure in view of the very high structural strength in the E-W direction.

As in the code-oriented analysis, the slight structural asymmetry was overlooked, and thus only one half of the structure was considered. The structure was assumed to be rigidly founded, although it was evident that soil-structure interaction may have been of some importance. Thus, it was tacitly assumed that 1) the acceleration spectrum ordinates were the same for the fixed-base and the flexible foundation cases, and 2) the distribution of the inertia forces along the height of the structure was not affected by the soil-structure interaction.

From the relative dimensions of the structural elements it was quite evident that the effect of slabs and beams acting with walls and columns as structural frames must have been quite small. To reduce the computational effort, accurate modelling of their contribution was not attempted.

A schematic plan of the idealized framing system at a typical floor level is shown in Fig. 6.1. It is seen that the center-to-center distances between columns are much larger than the net spans. The effect of these finite widths was considered in the analysis. However, the effect of the finite depths of the beams on the stiffnesses of the columns was generally

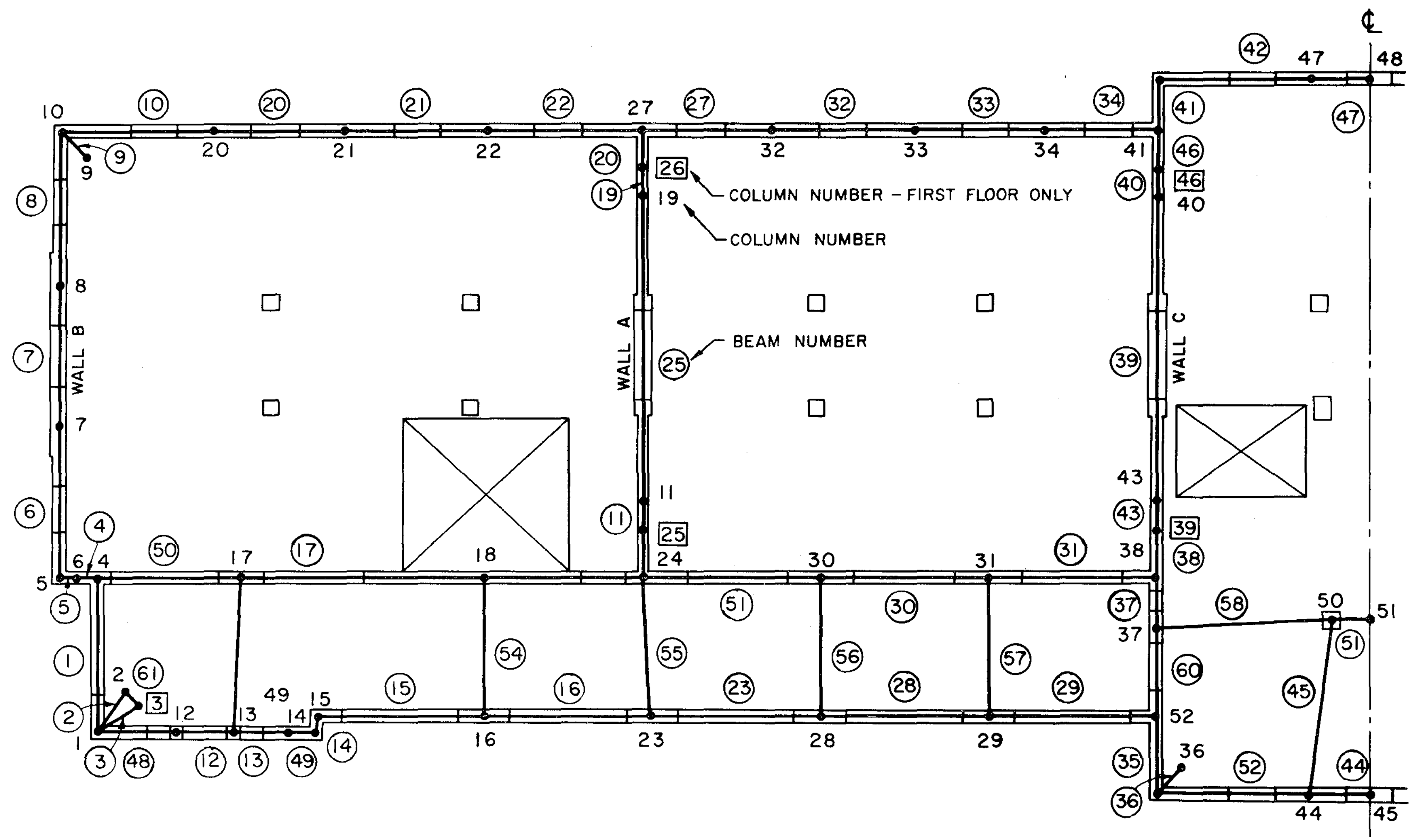

Fig. 6.1 Schematic plan of framing system at typical floor level.

ignored. This was done in order to offset, at least partially, the deformation taking place within the joint panel zones. However, no analysis was carried out to verify this assumption. The shear areas of the walls and the beams were taken as 80 percent of the gross concrete section of the web*. Gross moments of inertia were used, but the effect of slabs acting as beam flanges was ignored. This was believed reasonable since the floors were mainly of ribbed slab construction with slab thicknesses of only 2 in, the net spans were usually very small, and in the transverse direction the beams were bent in double curvature, i.e., with tension of the upper fibers along a substantial portion of their lengths.

Modelling the joint between two walls meeting at a right angle is certainly a problem in low rise buildings if only one-dimensional elements are to be used. This is due to the fact that if each wall is modelled as a separate beam with stiffness in one direction only (Fig. 6.2) the two walls can be joined, for the purpose of the analysis, only at floor levels, unless the number of "stories" is arbitrarily increased, a most expensive solution. Such modelling thus leads to stress incompatibility between contiguous elements, with an increase in their maximum stresses, a lowering of the rigidity of the assemblage, and to some extent, a redistribution of the internal forces among the frames.

In order to overcome these difficulties a somewhat different modelling technique usually was adopted. Whenever applicable, the cross-walls were taken as either L or T-shaped as the case may have been, with stiffnesses in two perpendicular directions. They were assumed to be located (in plan)

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*This value, with Poisson's ratio $\nu = 0.2$, is equivalent to a value of 5/6 with $\nu = 0.25$.

at their cross-sectional center of gravity (Fig. 6.3). This point, rather than the shear center, was considered to be more appropriate, since an incorrect location for the shear center of the walls cannot affect the load distribution among them in the case of a symmetric structure undergoing translational displacements only. On the other hand, an incorrectly located center of gravity may appreciably affect the axial stresses in flexural members. The main difficulty with such modelling is the fact that when a concentrated bending moment acts on the flange of a wall from an edge beam in the plane of the flange, the whole moment of inertia of the wall is not necessarily mobilized, since within a given story the element may warp (Fig. 6.3).

In modelling the penthouse, a simple representation was made. This was done by means of an equivalent frame with a single bay and a rigid lintel, a standard device in the preliminary analysis of multi-story building structures (30). Another reason to do this was the ETABS requirement that all column lines be continuous along the total height of the building. It was felt that modelling the four additional wide columns in the penthouse wall (Fig.2.4) would significantly increase the computational effort without a commensurate improvement in the quality of representation.

The idealized frames representing walls A, B and C are shown in Fig. 6.4.

6.4 Dynamic Analysis of Building 41

6.4.1 Simplified Model

In order to gain some insight into the structural behavior and to provide a check on the analysis of the entire building which was to be performed at a later stage, it was decided to analyze first a simplified model of the structure.

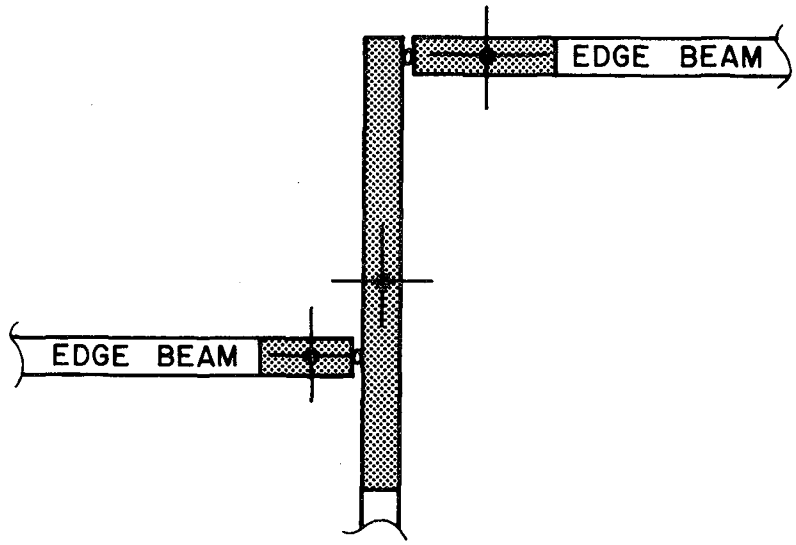


Fig. 6.2 Walls meeting at right angles modelled as separate beams

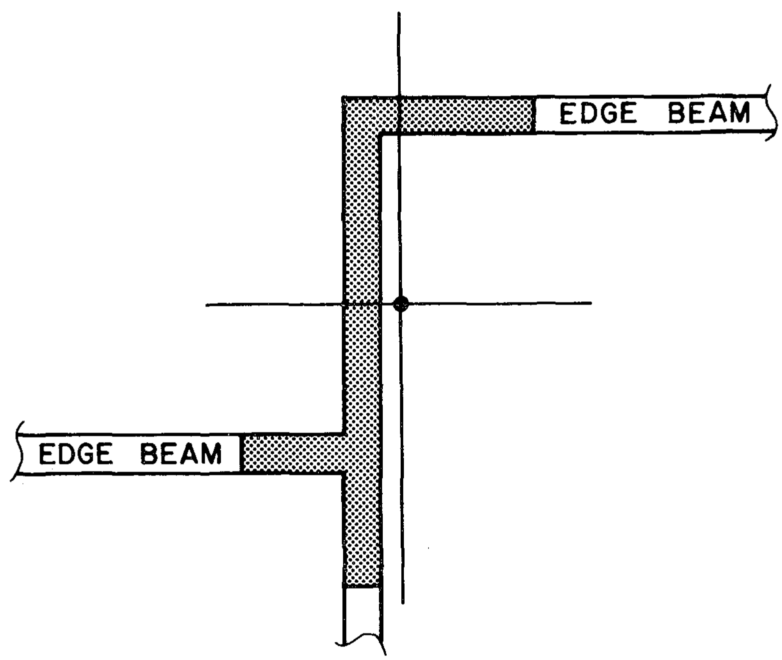


Fig. 6.3 Walls meeting at right angles modelled as a single beam

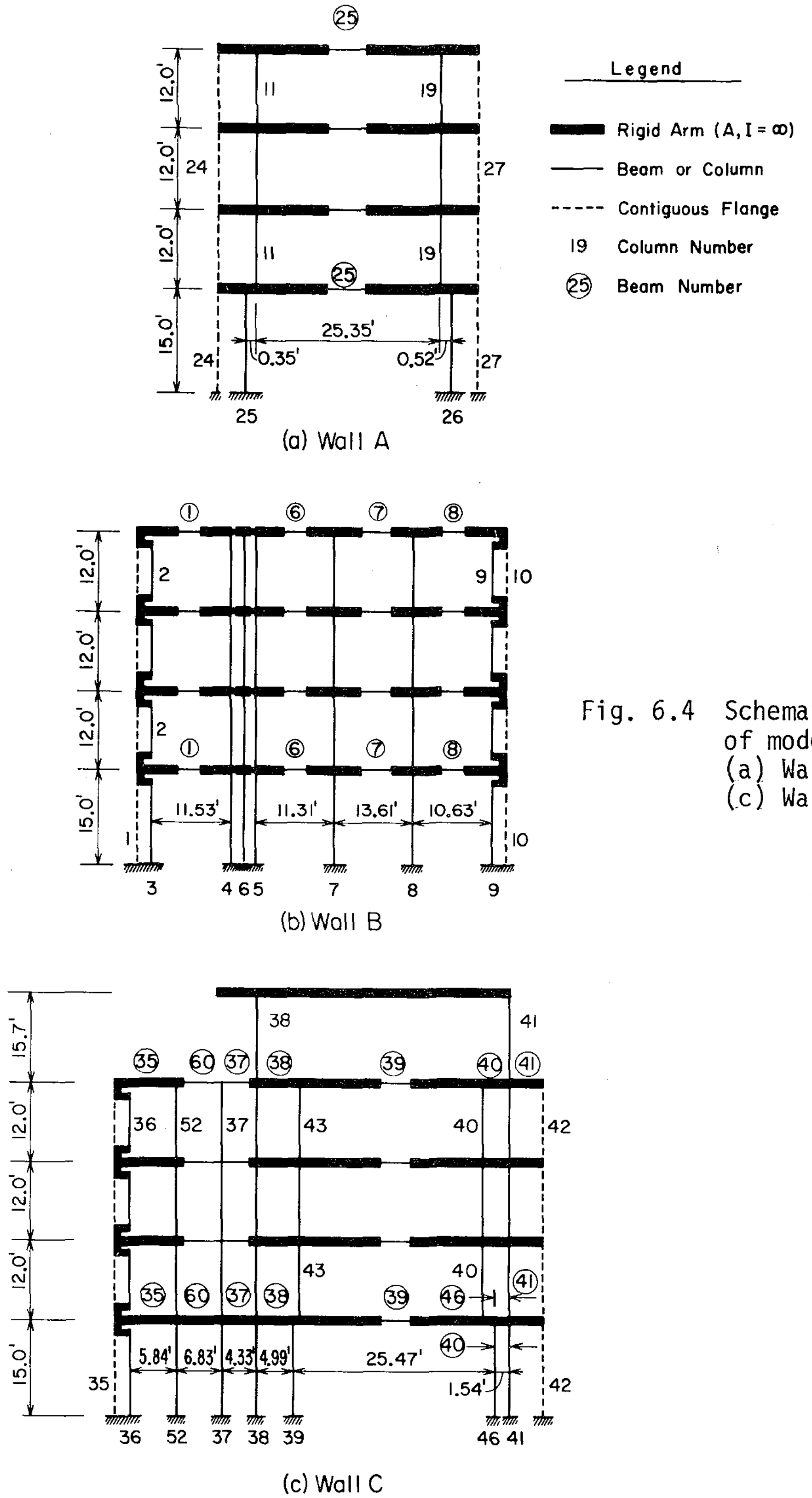


Fig. 6.4 Schematic representation of models for walls (a) Wall A (b) Wall B (c) Wall C

For this purpose wall A was isolated from the structure, since it was the simplest lateral load carrying member in the building. The model was further simplified by assuming identical flanges at the two external edges of the wall. The model analyzed is shown in Fig. 6.5. The flange assumed to be participating with wall A was very wide. In fact, two, rather than three, bays would have represented a more realistic participating width. A frame with a large number of bays was chosen, however, in order to obtain some information on the extent of the shear lag to be expected through the lintels. Note, however, that only shear lag through the lintels can be modelled simply with ETABS, although shear and flexural deformations do take place within the "rigid end zone" of the lintels.

A response spectrum analysis using ETABS was performed assuming constant spectral acceleration for all modes, i.e., a flat spectrum. As a first step in this analysis, numerical values were obtained for the first three periods and mode shapes. These results are listed in Table 6.1, including periods for two values of tributary mass namely: 27 percent and 31 percent of total, corresponding respectively to the proportion of the total base shear taken by the wide flange alternative of wall A in Chapter V, and in the full analysis that is discussed later. The value $T_1 = 0.122$ is, rather surprisingly, practically equal to $T_1 = 0.123$ obtained for the same alternative in Chapter V. Since the present model is in itself a simplified approximation to the real structure, this agreement, though encouraging, should not be taken as a strong confirmation of the continuous medium approach.

A plot of the fundamental mode shape is given in Fig. 6.6 together with the corresponding shape for the full model and the straight line approximation used by building codes. It is seen that the agreement among the three, when

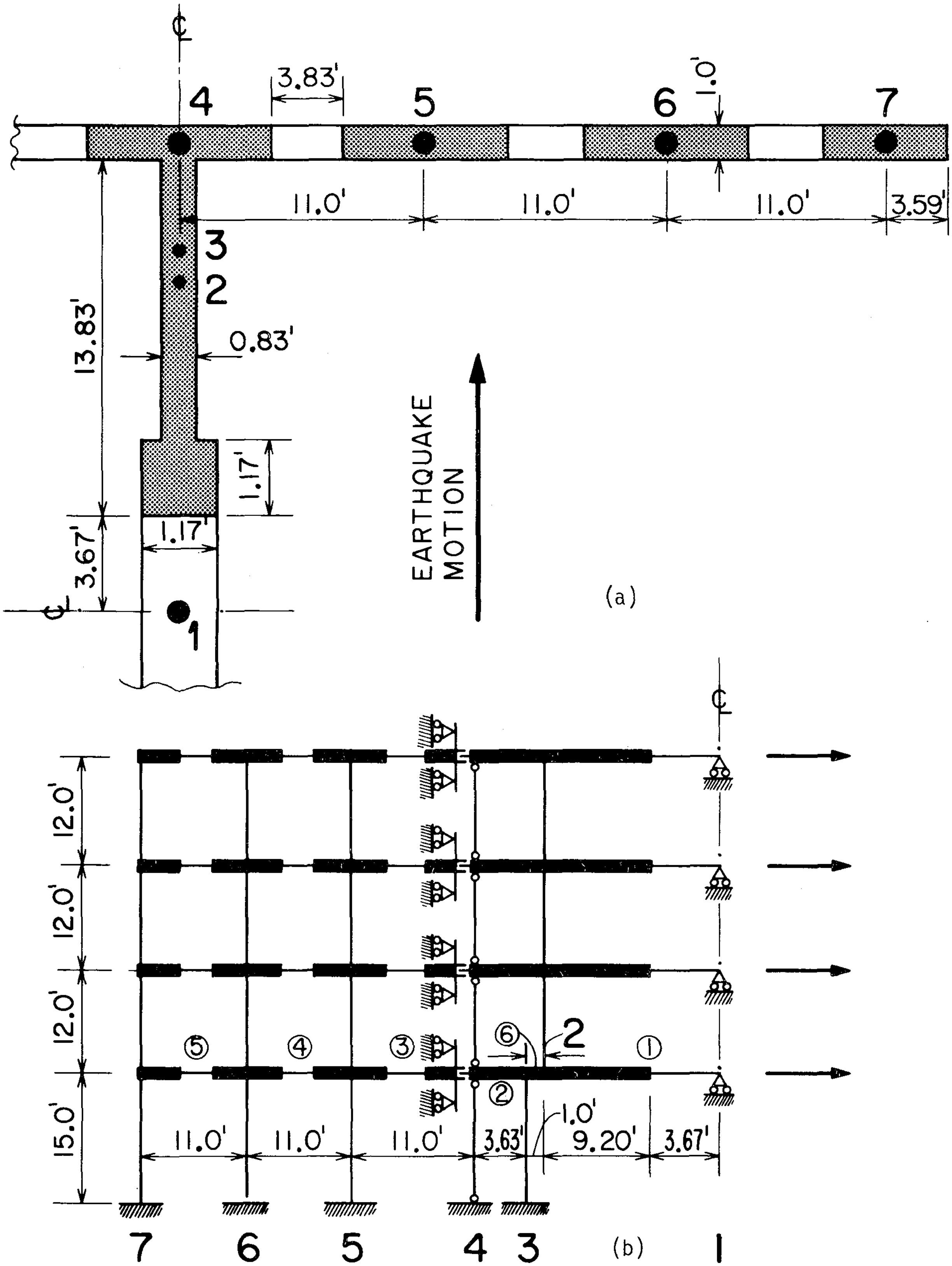


Fig. 6.5 Simplified model (a) Plan of simplified frame A
(b) Schematic representation of model

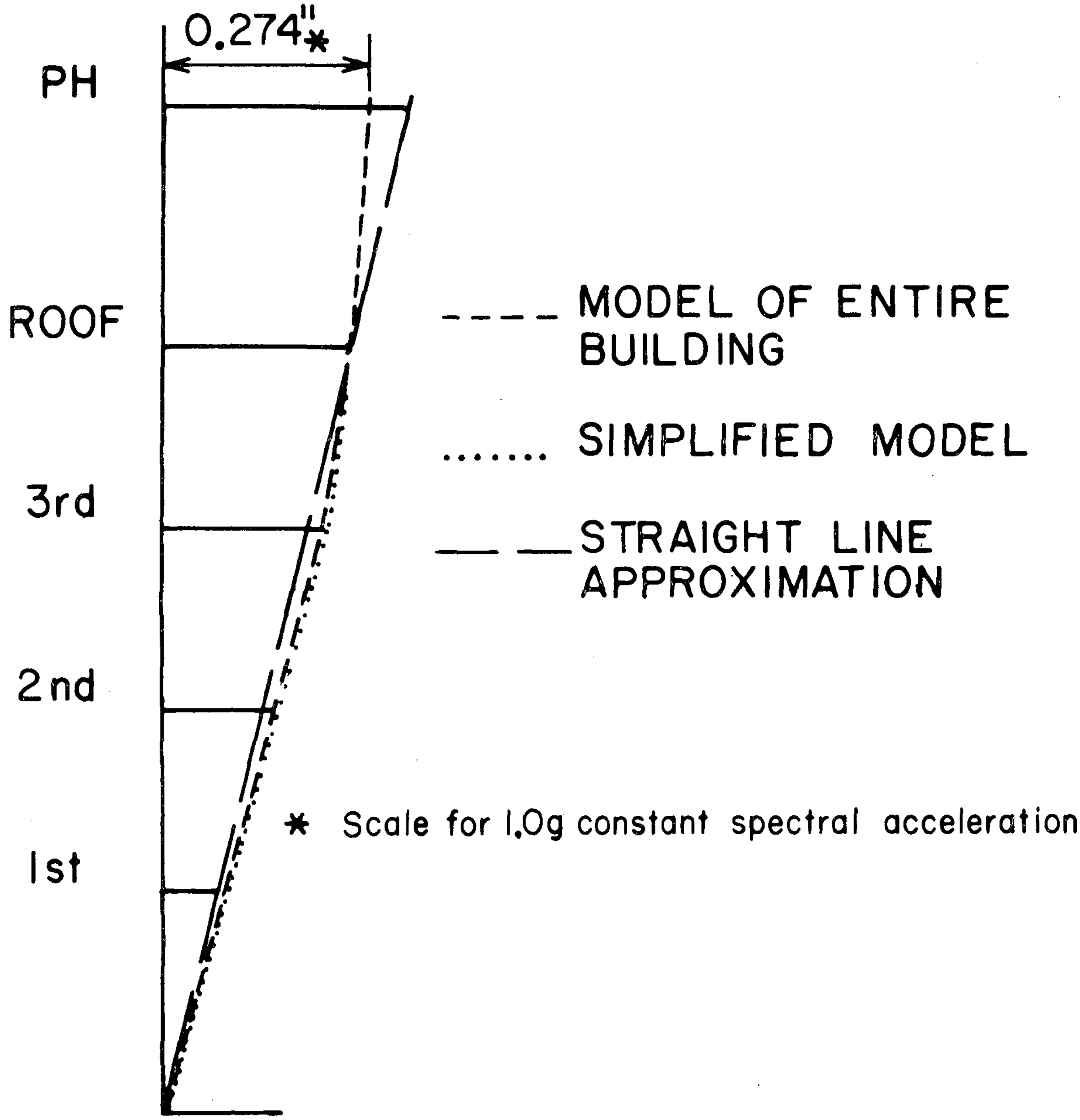


Fig. 6.6 Fundamental mode shape for three different modelling assumptions

normalized to equal displacement at roof level, is very good. Other important structural characteristics of Building 41 became apparent as a result of this analysis: 1) The shear lag in the longitudinal walls was considerable, yet the transverse walls, which were approximately forty feet apart, were likely to be coupled vertically through the axial stiffness of the longitudinal walls. This suggests that modelling the structure as three parallel frames, namely walls A, B and C, constrained to deflect equally in the horizontal direction at floor levels - a standard device in the structural analysis of tall buildings - may lead to erronous results, unless more is known on the effective widths of the flanges. 2) The effect of higher modes of vibration on the response was very small when a flat acceleration spectrum was assumed - a realistic assumption for stiff structures having very low natural periods. It was concluded, as expected, that the model of the entire building should incorporate the longitudinal walls if coupling between the cross-walls were to be accounted for. It was also concluded that a time-history analysis of the entire structure was not warranted, since sufficiently accurate results could be obtained from modal analysis, and the choice of input for the time history analysis raised further uncertainties.

6.4.2 Analysis of the Entire Structure

The natural periods and modal displacements were computed for the first three modes of the entire structure. Mode shape ordinates and periods are given in Table 6.1. A plot of the square root of the sum of squares (RSS) combination of the lateral displacement shape, indistinguishable from that of the fundamental mode, is shown in Fig. 6.6. The satisfactory agreement

Table 6.1

Mode Shapes and Natural Periods:
Simplified Model vs. Model of Entire Building

		Mode 1		Mode 2		Mode 3	
		Simplified	Full	Simplified	Full	Simplified	Full
PH		-	1.111	-	2.048	-	-3.602
Roof		1.000	1.000	1.000	1.000	-1.069	0.008
3rd		0.854	0.841	0.089	-0.131	1.000$^{(1)}$	1.000$^{(1)}$
2nd		0.618	0.595	-0.821	-1.099	0.551	-0.004
1st		0.329	0.305	-0.897	-1.049	-1.271	-1.041
Period T	31%	0.131	0.141	0.0435	0.0464	0.0257	0.0286
	27%	0.122		0.0406		0.0240	

(1) Normalized at 3rd Floor.

of the fundamental mode shape from the study of the entire building with the code approach and the simplified model has already been observed. The poor agreement of the third mode shape values in Table 6.1 is not surprising: the full model had five stories whereas the simplified model had only four. Because of the small contribution of the higher modes, the differences are not significant for purposes of this study.

There are several other differences among the three approaches which deserve comment. Starting with the fundamental period of vibration we note that there is a seven percent difference between the average value for the two alternative flange widths computed in Chapter V and the value obtained from the full model. This difference is not considered serious and is probably due to the accumulated effect of several approximations.

The results of the simplified model are of similar accuracy (7 percent underestimation of the fundamental period) in comparison to the full model when the mass tributary to wall A is made proportional to the wall's share in carrying the base shear. An even better approximation could have been obtained if a more realistic flange width were assumed (two bays instead of three). Also, the symmetric representation of wall A tended to overestimate the participation of the longitudinal wall along the porch, which at one side of wall A was connected to the wall by relatively shallow and long span beams (Fig. 2.4).

The response analysis performed by ETABS yielded numerical values for the RSS forces acting on the structural members. From these values the distribution of the lateral shear forces among the three walls in every story was evaluated, and results are presented in Table 6.2. Their most

Table 6.2

Lateral Shear Force Distribution Among Walls (S_a = 1.0g): Floor Diaphragms

Level \ Wall	A	B	C	Total	Percent of Code	Simplified Model	Model for Non-Linear Analysis
PH			252.3 / 100.0	2523	83.4		
Roof	379.8 (1) / 26.7 (2)	392.3 / 27.6	651.2 / 45.7	1423	84.3	372 / 26.1	381 / 26.8
3rd	791.0 / 30.5	649.0 / 25.0	1153.2 / 44.5	2593	85.2	778 / 30.0	791 / 30.5
2nd	1080.1 / 31.3	829.5 / 24.0	1541.7 / 44.7	3451	86.7	1068 / 31.0	1081 / 31.4
1st	1191.4 / 31.0	888.4 / 23.1	1765.2 / 45.9	3845 (3)	85.5	1235 / 32.1	1248 / 32.5

(1) Kips.

(2) Percent.

(3) 84 Kips or approx. 2 percent resisted by other elements.

striking feature is the relative uniformity of the shear forces tributary to each wall with height. This suggests a similarity between the individual deflection curves of the three walls, lending credence to the assumption made in the approximate analysis whereby the relative wall stiffnesses were obtained on the basis of their roof deflections.

From the numerical results it was also possible to assess the effects of shear lag. As expected, the axial forces in the south wall columns away from wall B and C were very low. On the other hand, all columns on the north and intermediate walls were highly stressed. It was suggested earlier that the model used underestimated the shear lag, nevertheless, the results demonstrate that the box effect can be quite important even in low rise buildings with pierced flange walls.

If the results given in Table C.1 (Appendix C) are compared with those of Table 6.2, it may be seen that the in-plane flexibility of the floor slabs was very unlikely to affect appreciably the fundamental period of vibration.

From the computed member forces it was also possible to evaluate the upper bound on the spectral acceleration using the same strength criteria as in Chapter V. Table 6.3 summarizes the findings.

Before discussing these results, which form one of the main objectives of the present study, it may be of some interest to describe how they were evaluated.

The "first yield" value was obtained from the level of spectral acceleration S_a at which the reinforcing steel in the weakest beam reached yield (assumed here to be 40,000 psi). This occurred in wall C. The capacity of the building after redistribution was evaluated as follows: From the analysis

TABLE 6.3

Spectral Acceleration Levels at Different Stages of Resistance

	First Yield	Yield and Redistribution	Cumulative Capacity
Percent g	25-30	35-45	45-50

it was found that the tensile capacity of the weakest wall segment was reached at about 0.5g, whereas several beams started yielding at lower spectral acceleration levels. The stiffnesses of these beams were then assumed to be factored by a value somewhat lower than the ratio of the spectral acceleration level at the onset of their yield and 0.5g. The program was then rerun with the adjusted stiffness values, and the resulting member forces checked against their capacities. Since only a small number of elements and marginal changes in total stiffness were involved, the forces in the yielding members were found to be reduced by practically the same ratio. On the basis of this calculation it was found that the upper bound on capacity was of the order of 35 to 45 percent of gravity.

The last range of values in Table 6.3, 0.45g - 0.50g, was obtained by adding together the capacities of the three walls, i.e. some load redistribution among walls was assumed to have taken place over and above the limited redistribution following yielding in some members. Since the onset of failure in walls A and B was non-ductile and indicative of observable damage, such an approach does not appear to be justified as the basis of the earthquake response. However, one could accept it on the condition that the actual load distribution among the walls may have been different from the one computed in the present analysis.

It will be observed that the lateral force capacity of the building was governed by the tensile strength of concrete — a brittle type of failure — rather than by yielding of the reinforcing steel, and thus must have a wide confidence band. It is also important to note that no two-dimensional elastic analysis was performed so that possible tensile stress concentrations were not revealed. On the other hand, the vertical edges of the walls as well as all re-entrant corners were reinforced and carefully detailed, so that this reinforcement may have arrested the propagation of cracks into the members. Also, cracks were not likely to propagate far from the corners in view of the steep gradient in the strain energy levels with increasing distances from the edges.

Although no particular effort was made to model the out-of-plane frame action of the floor slabs and the longitudinal walls, it was found that approximately two percent of the base shear was carried by these members. It is believed that with a more realistic modelling, this secondary effect may have reached as high as four to five percent. Another neglected factor was the contribution of the longitudinal reinforcement to the strength of concrete members strictly in terms of increasing their "transformed" cross-section. This may have amounted to another two percent.

On the other hand, a factor which may have contributed to lowering the capacity of the structure was the concrete foundation system. In all the analyses that were carried out it was tacitly assumed that the walls were fixed against relative rotation, and in this chapter it was assumed that the walls were completely fixed at their base. In structural terms, the former assumption implies that the foundation beams connecting all the wall segments in each cross-wall or longitudinal wall were very stiff and sufficiently strong to transfer the shear forces and bending moments required by the assumption (See Fig. 6.7). However, from Fig. 2.4 it can be seen that, for example, in wall B on the west elevation, the depth of the foundation beam does not appear to be large enough to do this. The capacity reduction, if any, which may have resulted from this source was not investigated; however, it is believed to have been small.

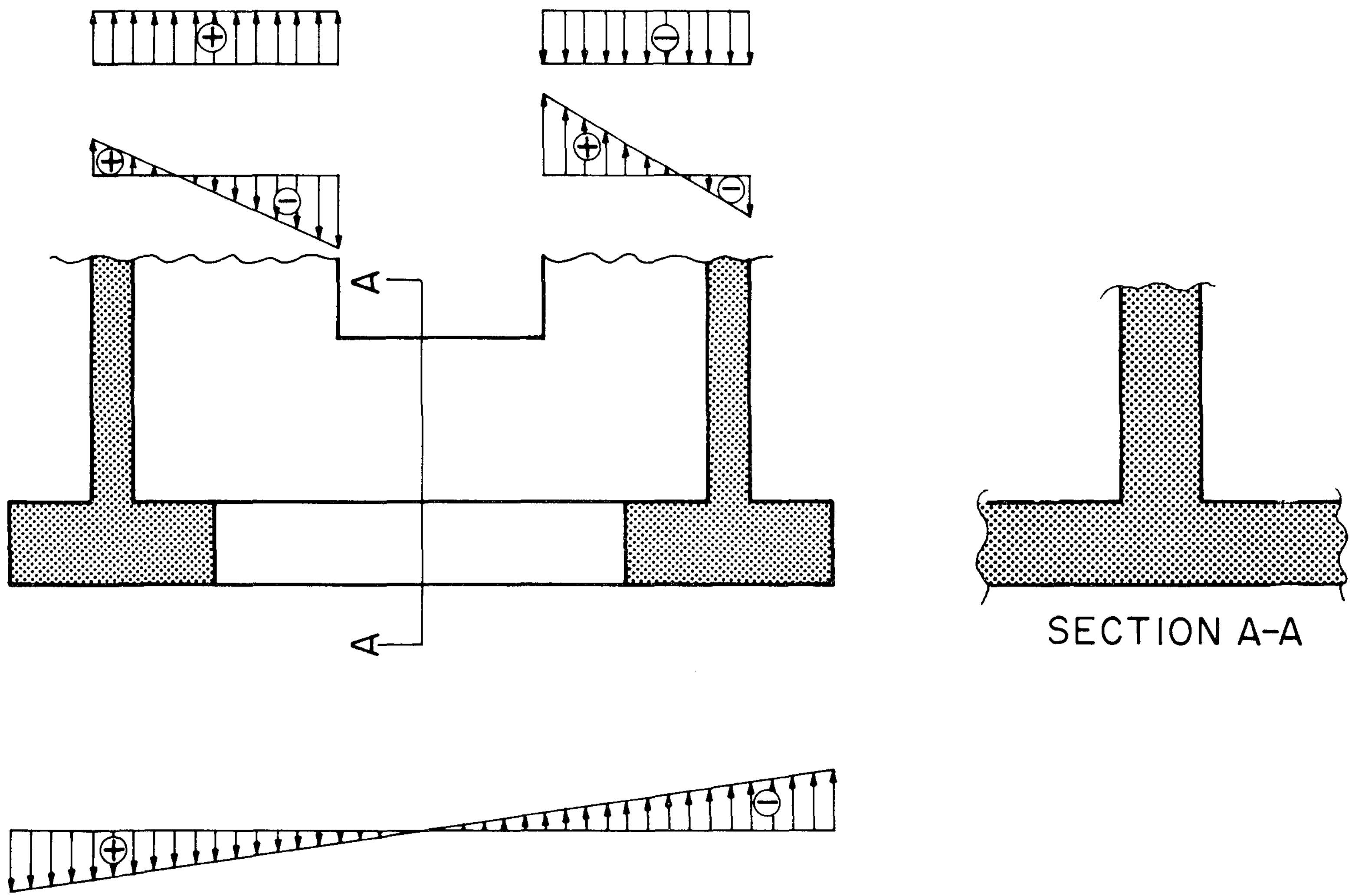


Fig. 6.7 Loading state of rigid foundation beam (gravity effects not shown).

When comparing the results in Table 6.3 with Table 5.3 it is important to bear in mind that the base shear obtained from modal analysis for a constant acceleration spectrum is lower than the base shear that would result if the same spectral acceleration level were assumed to act as a horizontal load on a rigid structure, which is the approach implicit in the way the code type analysis is done. Table 6.2 shows that for the full model of the building this ratio is approximately 85 percent. Indeed, if the values in columns one and three of Table 6.3 are multiplied by 0.85 the results in columns one and two of Table 5.3 are closer to agreement. Although encouraging, the consistency between the two results does not necessarily mean that greater confidence should be placed on the upper bound on the spectral acceleration thus derived.

In summary, the linear elastic analysis described in this chapter predicts that the fixed-base lateral load capacity of Building 41 was on the order of 0.5g spectral acceleration or base shear of about 0.40-0.45g. This value should now be interpreted in terms of the probable ground motion at the site during the San Fernando Earthquake. This, however, is quite difficult. One approach is to scale some representative ground motion records so that the spectral acceleration at the fundamental period of the structure would match the value computed above. The results matched by scaling peak accelerations are shown in Table 6.4 for several records, two of which are "artificial." The effect of small variations in the natural period are evident in the first two records. This is due to the irregular nature of the spectra at low damping ratios. Similar variations are most probably also present in the last two records, but could not be given

TABLE 6.4

Peak Ground Acceleration Scaled to Give
0.5g Spectral Acceleration at T = 0.14(0.15) Seconds; η = 5%

	Factor	Peak Acceleration Percent
Pacoima Dam (S14W)	0.22 (0.26)	26 (30)
Holiday Inn (1st Floor, N00W)	1.18 (0.96)	30 (24)
Computed Pacoima Rock (S14W) [4]	~ 1.0 (~ 1.0)	40 (40)
Computed Van Norman Dam [4]	~ 0.55 (~ 0.55)	33 (33)

numerical values since only the graphical presentation of the spectra were available (16).

The important result, however, is that the inferred level of ground motion indicated by a spectral acceleration of about 0.4g is still lower than the lowest credible estimate of the strong ground shaking at the site. Thus, it seems likely that the structure had a capacity significantly in excess of that revealed by the analyses done so far.

In the next chapter the results of a time history analysis are described in which foundation compliance, nonlinearities and uplift are incorporated. This analysis sheds more light on the structural response, and helps to resolve the discrepancy between observed response and calculated capacity.

VII. NONLINEAR SOIL-STRUCTURE INTERACTION

7.1 Introduction

Although a significant discrepancy remained between observed and calculated responses, it was shown in Chapter VI that on the assumption of full base fixity Building 41, designed for a lateral load coefficient of ten percent at working stress level, could sustain lateral forces approximately four times larger than this with practically no damage.

One of the problems with extending this simple description of response to resolve the discrepancy entirely is that the assumption of full base fixity, or even that of full contact between the base mat and the soil, becomes increasingly questionable with increasing base overturning moment. Even at spectral acceleration levels of about 0.5g, less than that thought to have occurred at the fixed-base period of the structure, it was found from static analysis that full contact between the base and the soil could not be maintained.

It has already been suggested that a linear elastic analysis that takes into account the foundation compliance is not likely to predict a higher response capacity in this case. This is because the resulting elongation in the fundamental vibration period is relatively small, and the level of strains in the soil is still quite low. Yet, a higher capacity of the system is required if observed behavior is to be reconciled with the apparent level of ground motion.

It may well be asked why a nonlinear response analysis is likely to be more promising in view of the uncertainties in the simpler, linear analyses.

That is so because at higher levels of excitation some existing effects as well as new phenomena become increasingly more prominent and thereby may give additional insight into the problem.

Starting with foundation compliance, it is quite clear that the reduced contact area between the base and the soil following separation leads to lowering of the rocking and lateral stiffnesses of the foundation. The loss of rocking stiffness with increasing base separation is very large, indicating a strongly nonlinear increase in period with amplitude. As an example, for a rectangular footing on a linear elastic half space, the stiffness is approximately proportional to the third power of the ratio of the contact width to the total width [(e.g., (31)]. With an appreciable decrease in rocking stiffness, the effective fundamental period of the structure may be increased enough to reduce the spectral acceleration. The loss in lateral in lateral stiffness is much more moderate.

In addition, with the reduction in contact area, soil stresses become larger, and the resulting yielding in the soil is accompanied by some hysteretic energy losses; also, the effective stiffness of the soil decreases when significant yielding occurs.

It is thus seen that an analysis incorporating these effects is likely to imply a longer period and larger damping. This means a reduction in the forces induced in the building by a given ground motion, that is, a higher earthquake resistance for the structure. However, these favorable effects may be offset partially by other phenomena that arise from base separation. The most important of these may be the excitation of vertical motion due to impact at the closing of the gap separating the base from the soil and, since the impact is not symmetric about the vertical centerline of the structure, coupling between vertical and lateral motion may take place. It was believed that this vertical motion, although mitigated by energy losses upon impact, might result in a less favorable state of stresses within the structure. Other possibly unfavorable effects, believed to be minor, include that of vertical ground acceleration and the P-delta effect. In view of the complexity of the problem, it is not easy to account for all these effects in an adequate manner. However, an attempt has been made to model the structure in such a way that the phenomena described above would manifest themselves, at least qualitatively, thus affording a more realistic interpretation of the response. Because of the complexity of the modelling, the analyses were done for several sets of the most important parameters.

7.2 Computer Program

The program Drain-2D: Dynamic Analysis of Inelastic Plane Structures (32) was used. This program performs a nonlinear analysis of frames and shear walls under lateral and vertical time dependent loadings. The program consists of a series of "base" subroutines which carry out a step-by-step dynamic analysis. Subroutines for truss elements, beam-column elements, shear panels, semi-rigid connections, and reinforced concrete beam elements are available. It is thus seen that fairly broad capability in modelling the superstructure is available. The program was not designed to analyse soil-structure interaction problems, although it was recently used for uplift analysis of a steel frame (33). However, it was the only suitable program readily available to the authors, and it was decided to adopt it to the problem at hand. The following section describes the way the available elements were used to model the soil behavior.

7.3 Mathematical Model

7.3.1 Superstructure

The single frame model used in Chapter VI was further simplified by replacing the longitudinal wall segments which restrained the vertical displacements of the external edges of wall A by simple truss elements. The truss elements are shown in the schematic representation of the model given in Fig. 7.1. Note that the possibility of uplift precludes taking advantage of the structural symmetry assumed for the simplified model. The cross-sectional areas of columns C1 to C4 which represent the extended flanges, were so chosen as to match closely the fundamental mode shape and frequency of the fixed base superstructure. The cross-sectional properties of columns and beams are given in Table 7.1. It will be observed that the yielding beam element available in Drain-2D permitted a more realistic modeling of the connecting beams than had previously been done. Note, however, that such yielding could take place only so long as the tensile capacity of the columns, modeled as linear elastic elements, was not reached.

7.3.2 Foundations

The soil was assumed to consist of a number of independent axial springs, i.e. a discretized Winkler foundation. Truss elements were used for this purpose. The possibility of uplift — the no-tension condition — was modeled by taking advantage of the buckling capability available in Drain-2D for these elements. Since stress directions had to be reversed for bucking to occur it was necessary to "hang" the model of the superstructure from the foundation soil as shown in Fig. 7.1. In this configuration buckling simulates loss of contact. This type of modeling was also adopted by

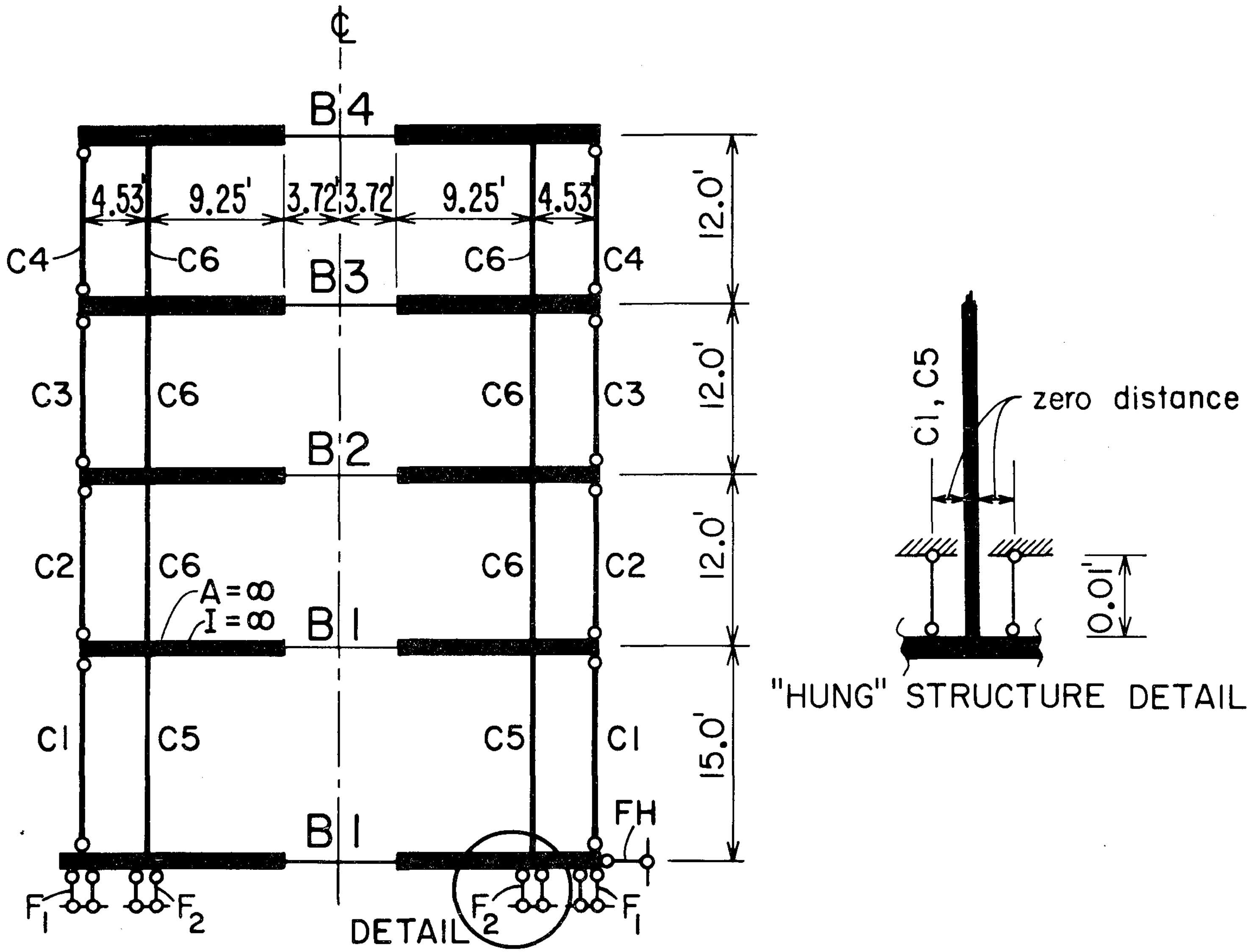


Fig. 7.1 Schematic representation of model used in nonlinear foundation analyses

TABLE 7.1

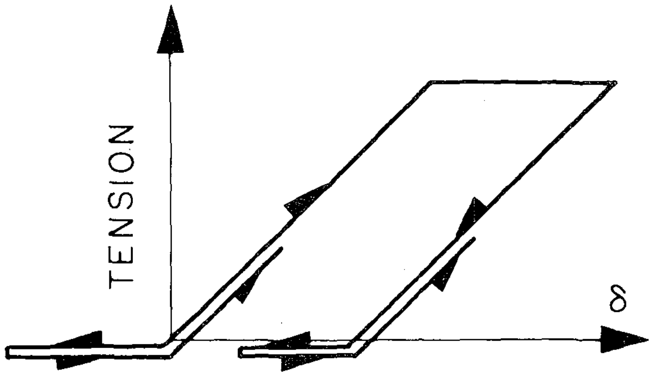
Superstructure Cross-Sectional Properties

Element	C1	C2	C3	C4	C5	C6	B1	B2	B3	B4
A(ft^2)	12.5	10.0	20.0	25.0	20.7	17.7	∞	∞	∞	∞
I(ft^4)	0.0	0.0	0.0	0.0	479	395	13.88	13.88	13.88	13.88
AS(ft^2)	0.0				955	955	5.33	5.33	5.33	5.33
Yield Level (kip-ft)	← Elastic →						975	830	575	440

Evision (34) in his study of Rocking foundations. The hysteretic energy absorption characteristics of the soil were approximated by trilinear force-displacement curves. Since Drain-2D can only model bilinear truss elements, two parallel elements of equal length and area but with different yield levels were used. Fig. 7.2 shows the postulated behavior of the soil, which, although quite approximate, incorporates the most essential features of the situation.

The problem of assigning realistic numerical values to the trilinear spring modelling the soil-structure inteaction is by no means trivial. Starting with the elastic values, it will be observed that any frequency dependence of the stiffness coefficients is ignored, and that the static values are assumed to hold. Note also, that the conventional procedure of evaluating these parameters from elastic half-space theory by means of "equivalent" circular footings (21) used in Chapter V is not well-suited to the footing of wall A which was H-shaped in plan, with narrow flanges and web. It is believed that a more appropriate approach is to treat the base as composed of two independent, narrow, rectangular strip footings representing the flanges, with allowance made for the web, comprising as it did approximately 40 percent of the base's plan area. This approach differently increases the numerical values of the vertical, horizontal and rocking stiffnesses of the foundations compared with their conventionally computed counterparts.

Regarding the non-linear soil response, it is believed that locating the axial springs according to the conventional approach would misrepresent the most important feature of the foundation system, namely, that during



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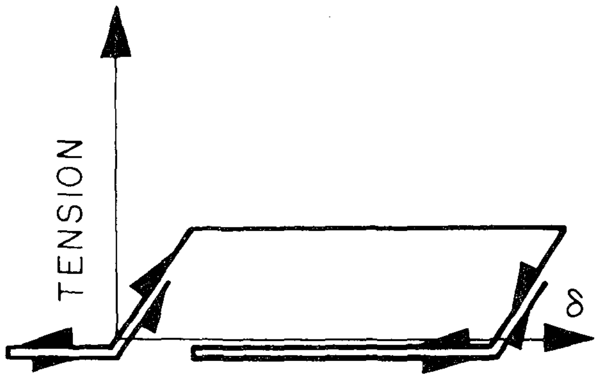
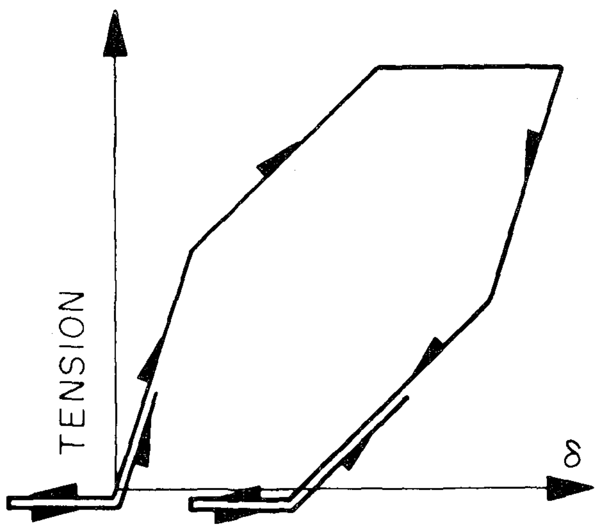


SPRING 2



COMBINED ACTION

Fig. 7.2 Tri-linear soil behavior modelled with two parallel bi-linear springs.

partial uplift all the axial forces are concentrated in a narrow strip at the edge of the footing. Another difficulty is assigning a numerical value to the lateral stiffness coefficient K_x. After partial uplift, the area of the soil in contact with the footing is reduced, and this has to be reflected in the value of K_x. On the other hand, the lateral resistance due to embedment is probably not affected materially. It is possible with Drain-2D to model the lateral load resisting springs in such a way that they would be disconnected on the onset of uplift. However, since the soil-structure interactive model is made of discrete springs, it was believed that such stepwise increase or reduction in horizontal stiffness might result in some spurious high frequency motions. Therefore, such modelling was not used, and a single horizontal spring as shown in Fig. 7.1 was employed. The stiffness coefficients, K_x, K_y and K_θ, and the associated foundation moduli C are given in Table 7.2.

The nonlinear force displacement characteristics of the soil were chosen to model the reduction of stiffness with increasing strain levels, and to introduce some hysteretic damping into the system. This is in addition to the viscous damping assumed, which is discussed subsequently. The combined compressional behavior of the two axial springs under each column line is shown in Fig. 7.3, and it can be seen that for a yield ratio $\alpha = 0.33$ the secant stiffness at a ductility ratio of ten (for example) is 40 percent of its initial value. The absolute yield level of 20 ksf is somewhat higher than would be obtained with Terzaghi's bearing capacity factors (27), since they are known to be conservative (35). However, it turned out that due to the large flange area of the base, the yield plateau was never reached in any of the analyses that were carried out, so that, in fact, the response

TABLE 7.2

Soil Properties Assumed in Analysis
(variant in parentheses)

Coordinate	Stiffness Coefficient		A or I	Winkler Modulus $C(Kip/ft^3)$
	$K_0^{(1)}$	$K^{(2)}$		
x	$K_x = 1.6 \times 10^5 - 2.2 \times 10^5$ (kip-ft)	2.0×10^5 (0.83×10^5)	$A = 500$ ft^2	400 (167)
y	$K_y = 2.1 \times 10^5 - 2.8 \times 10^5$ (kip-ft)	6.0×10^5 (2.5×10^5)	$A = 500$ ft^2 $F_1 = 160;\ F_2 = 90^{(3)}$	1200 (500)
θ	$K_\theta = 0.55 \times 10^8 - 1.1 \times 10^8$ (kip-ft)	1.54×10^8 (0.64×10^8)	$I = 128{,}000$ ft^4	1200 (500)

(1) As per Reference 21; $G = 3.6 \times 10^3$ ksf, $\nu = 0.33$

(2) Values used in present study.

(3) $A = 2F_1 + 2F_2$.

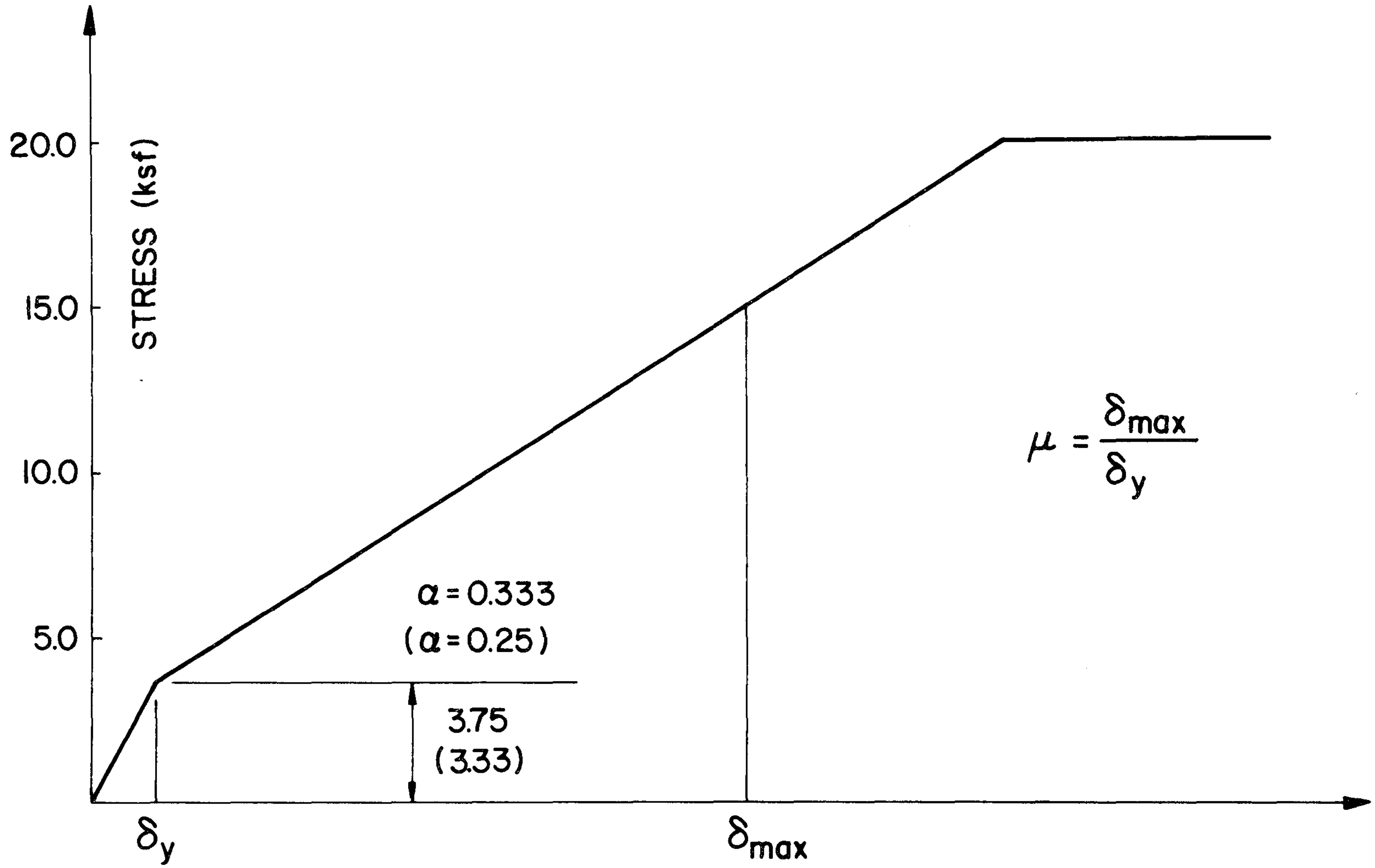


Fig. 7.3 Combined behavior of vertical soil springs under compression (variants in parentheses).

was not sensitive to the bearing capacity level of the foundation soil. This also lead to a completely elastic recovery of the soil, a somewhat unrealistic assumption.

The two alternatives postulated for the horizontal spring are shown in Fig. 7.4. It is seen that the possibility of sliding was not modelled.

It will be observed that the yield ratios 0.25 and 0.33 chosen for the springs are higher than would be obtained with the values of Table 6-A of Reference 21 at moderate ductility ratios.

The level of damping appropriate for a given structural system and response is always problematic, particularly when soil effects are thought to be important. Since Drain-2D can only model viscous damping of the Rayleigh type ($\underline{C} = \alpha\underline{M} + \beta\underline{K}$), the problem was reduced to assigning numerical values to α and β, the coefficients factoring, respectively, the mass matrix and stiffness matrix components of the damping matrix.

Partial uplift is associated to a large extent with rigid body motion and an associated lengthening of the period. Thus a damping matrix proportional to the mass matrix which gives a damping factor that increases with the natural period is not very appropriate. Similarly, the choice of β is complicated because it appears to be more reasonable to associate β with the current tangent stiffness matrix, rather than with the original one (β_0). As a rather arbitrary compromise it was decided to use the damping coefficients that produce five percent critical damping in the first two modes of the rigidly founded structure. This led to $\alpha = 3.380$ and $\beta_o = 0.00055$. As a check, two analyses were performed in which the effect of stiffness loss on damping was examined.

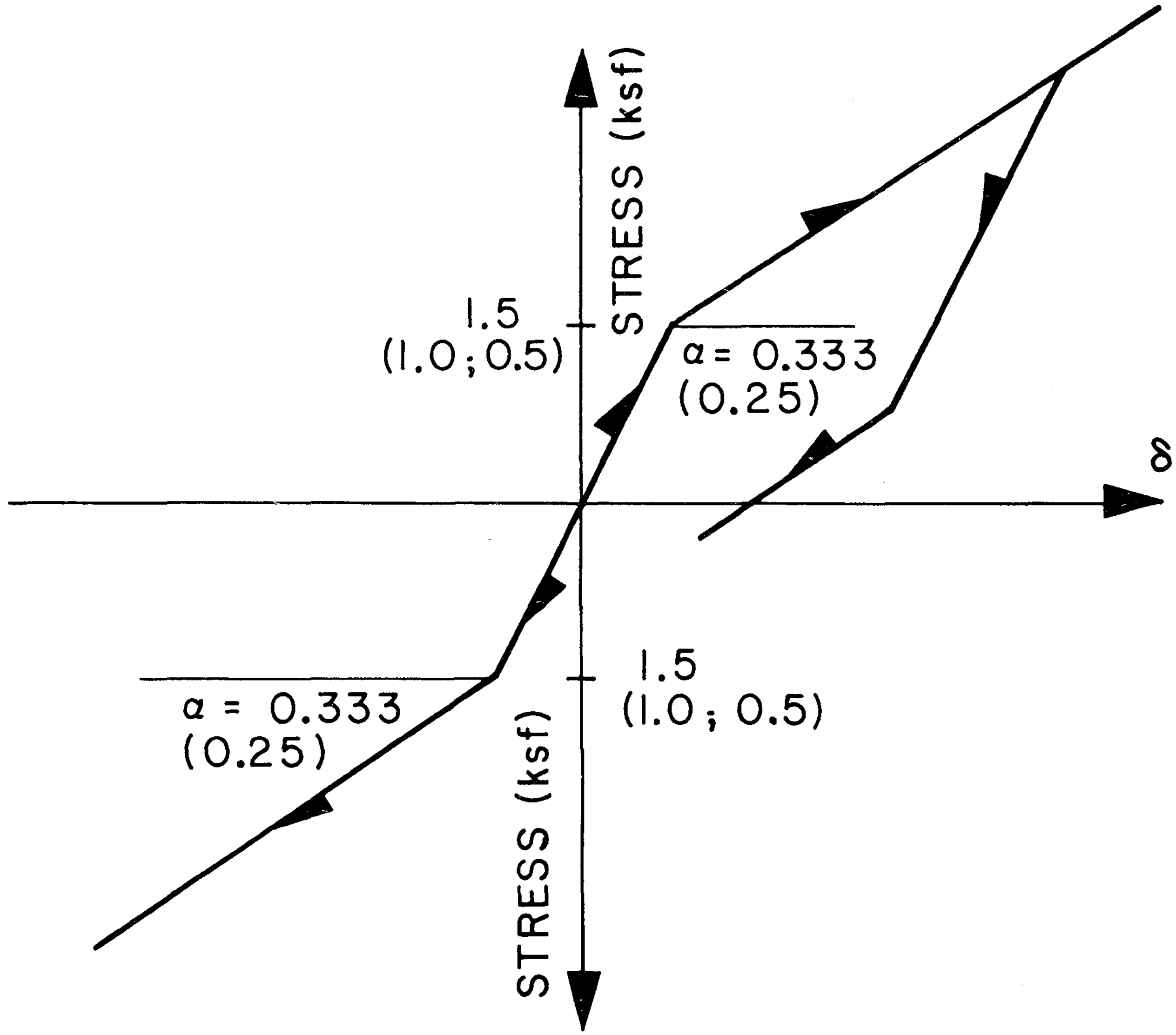


Fig. 7.4 Bilinear behavior of horizontal soil spring.

To provide a measure of the additional damping present in the system, it is useful to express the hysteretic energy dissipation in the soil springs in terms of a viscous damping ratio. However, it is most convenient to do so for steady-state conditions. One way of doing this is to compute the ratio of the hysteretic energy dissipation per cycle ΔW to the strain energy W stored in a viscously damped linear oscillator vibrating at its resonant frequency, and having the same amplitude. It is evident that this ratio depends on the ductility ratio μ (the ratio of the maximum displacement to the displacement at yield). However $\Delta W/W$ is a slowly varying function of μ, so that it is possible to compute a sufficiently representative single value. Simple calculations show that for the horizontal spring, a strain hardening or yield ratio $\alpha = \frac{1}{3}$ and $\mu = 4$ lead to $\Delta W/W = 1.0$, so that the equivalent damping ratio $\eta = 15.9$. This value is in agreement with recently published results on hysteretic damping energy dissipation in the Santa Felicia Dam (36). Regarding the vertical springs, the yielding-buckling model is not suitable for this type of analysis. Yet it is evident that the energy dissipated will be smaller than that computed on the basis of a double bilinear hysteresis model with similar properties. For this model $\eta = 15.9/4 \cong 4$ percent, which is already quite low.

It should not be overlooked, however, that these values refer to steady state conditions. During an earthquake, the response amplitudes vary irregularly, and only a few excursions well into yield occur. It is estimated that the equivalent contribution to damping from soil hysteresis is on the order of 2 or 3 percent with a maximum near 4 percent.

An estimate of the <u>total</u> damping in the system may be obtained by adding the effective viscous damping in the elastically founded linear system to the equivalent damping computed above. Since the former is approximately 7 to 8 percent (Fig. 7.5) we obtain an upper bound of about 12 percent. This value is in agreement with that found in the earthquake response of buildings with similar levels of minor damage (37) and with Eq. 6-9 of Reference 21 ($\tilde{T}/T \geq 15$; $\bar{h}/r \cong 2$).

In practice, it is quite difficult to assess the overall effect of the various contributions for damping. Therefore, it was decided to subject the structure to a base impulse, and then to compute the amplitude decay. The analysis was carried out for the soil parameters in Figs. 7.3 and 7.4 (unparenthesized values). The time history of the lateral displacement of the roof is shown in Fig. 7.5. The maximum amplitude obtained is on the order of that thought to have been experienced during the San Fernando earthquake. It can be seen that the displacement is dominated by the first mode. From the ratios of successive peaks, it was possible to evaluate the effective equivalent damping in the system. The first ratio led to $\eta \approx 0.20$. This includes, however, a contribution from the yielding of some connecting beams between the walls.

The damping ratio at lower levels of excitation $\eta = 0.07$-0.08 as against $\eta = 0.05$ for the fixed-base structure seems reasonable. The effective period at the early stages of motion is near 0.35 sec, approximately 150 percent of the linear interactive system, $\tilde{T}_1 = 0.215$ seconds, and a factor of about 2.3 higher than the fixed-base period of 0.14 seconds. It is thus seen that the period shift, although considerable, is not large enough to support the expectation of lower spectral acceleration levels.

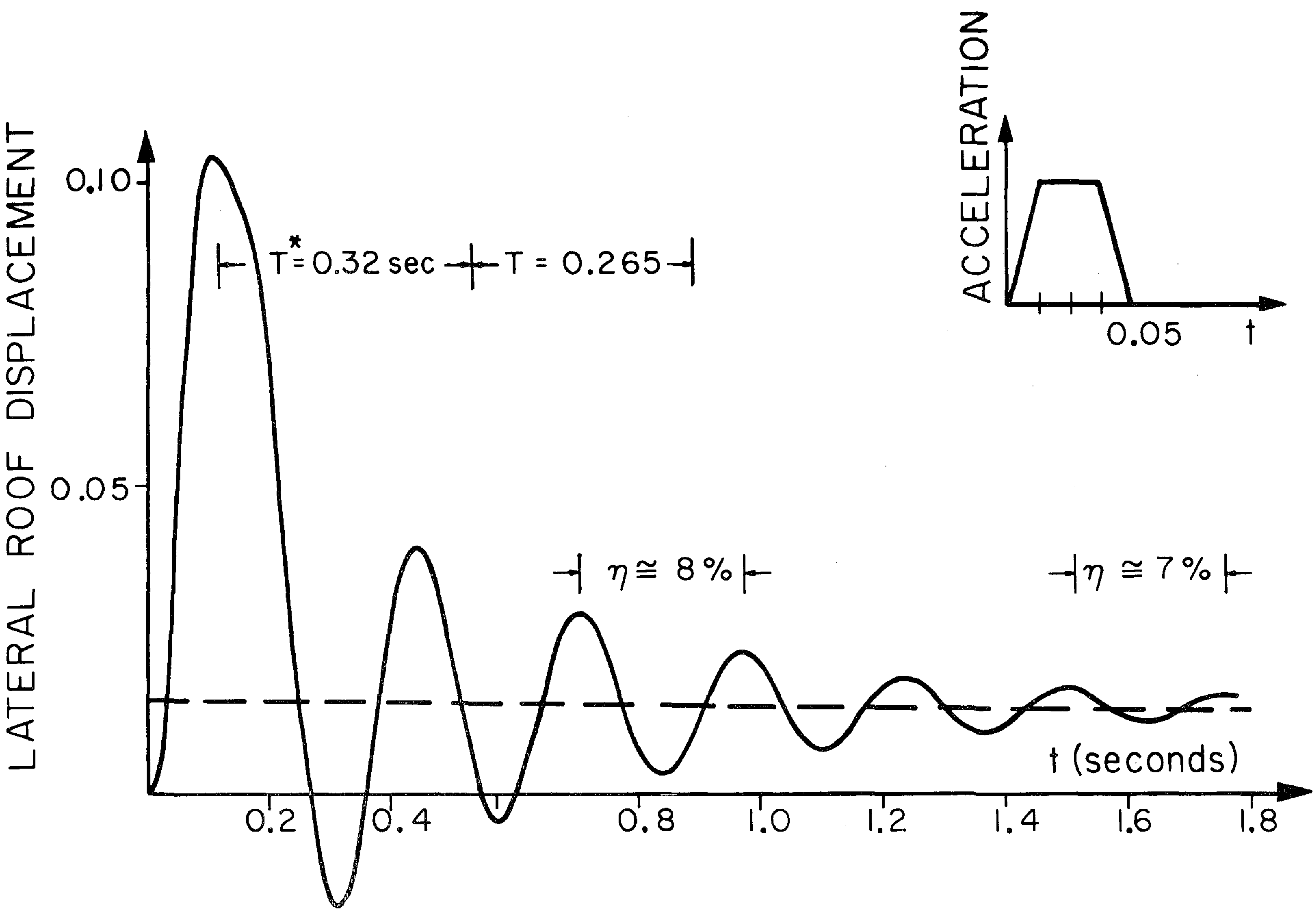


Fig. 7.5 Impulse response time history: Lateral roof displacement.

7.4 Ground Motion

The uncertainties regarding the ground motion at the site during the San Fernando earthquake suggested that more than one record be used for the nonlinear analysis. The two records chosen for this purpose were those obtained during the February 9, 1971 San Fernando earthquake at Pacoima Dam (comp. S16E) and at the Holiday Inn at 8244 Orion Blvd. (1st floor, comp. NOOW).

The two records were scaled to give approximately the same fixed-base shear as a constant acceleration spectrum with a level of 0.9g. Because the first mode dominates the response, this is about the same as normalizing to 0.9g spectral acceleration at T = 0.14 sec and 5 percent damping. This level of response is compatible with the lower estimates of the strength of ground motion at the site during the earthquake. Thus, a factor of 0.4 was used to multiply the acceleration ordinates of the Pacoima Dam record, and a factor of 2.0 for the Holiday Inn record. This scaling produced acceleration peaks in the two records of 0.47g and 0.51g respectively. It may be suggested that a more appropriate basis for scaling would be the response of the linear soil-structure interactive system, rather than that of the fixed-base one. The choice of scaling method is somewhat arbitrary, however, and the fixed-base normalization used permitted direct comparisons with the models analyzed in Chapters V and VI, and helped to highlight the difference between the records.

It was decided to consider also the effect of vertical motion on the response in some of the calculations. For this purpose, the horizontal and vertical components for each of the two records were assumed to act simultaneously.

In order to reduce the computational effort, only the first ten seconds of each record were considered. Fifteen seconds were at first taken for the Holiday Inn record in recognition of the late arrival of some of the peaks in the ground motion. However, a trial run showed that response peaks of interest were reached in less than ten seconds.

7.5 Choice of Integration Time Step

In Drain-2D the equations of motion are integrated numerically assuming a constant acceleration within each integration time step. Since changes in the state of the structure may occur within a time step, the resulting out-of-balance force must be corrected. The corrective forces are applied, however, during the following time step. To reduce the resulting overshoot, the integration time step should be quite small. In the present analyses this time step was taken as 0.005 seconds. This very small time step was also considered necessary since it was important to identify any feature of the response associated with high frequencies such as axial forces in the walls. To verify whether this step was small enough, one problem was also analysed with a time step of 0.003 seconds. The only significant differences between the two sets of results were found, as expected, in the vertical axial forces in the walls and the foundation springs: these differences were on the order of 5 percent.

7.6 Dynamic Response

In this section the results of the several sets of linear and nonlinear analyses are examined, and the sensitivity of the response to the modelling assumptions is discussed.

Table 7.3 lists the parameters considered in the response analyses that were carried out for the Holiday Inn and Pacoima Dam records. These are

TABLE 7.3

Soil and Superstructure Parameters for Cases Analyzed

Case	E.Q. Scale	Damping	Soil Properties						Variations in Superstructure Properties
			Lateral			Vertical			
		α, β	c_x (kip/ft^3)	Y_x (ksf)	α_x	c_y (kip/ft^3)	Y_y (ksf)	α_y	
CSA	0.9	$\alpha=3.38$; $\beta_0=0.00055$	∞	∞	1.000	∞	∞	1.000	None
HI.1	2.0		∞	∞	1.000	∞	∞	1.000	
HI.2	2.0		400	∞	1.000	1200	∞	1.000	
HI.3	2.0		400	1.5	0.333	1200	3.75	0.333	
HI.4	2.0		400	1.5	0.333	1200	3.75	0.333	$E=0.8E_0$
HI.5	2.0		400	1.0	0.333	1200	3.75	0.333	None
HI.6	2.0		400	1.0	0.250	1200	3.33	0.250	
HI.7	2.0		400	0.5	0.333	1200	3.00	0.333	
HI.8	2.0		400	1.5	0.333	500	3.75	0.333	
HI.9	2.0		167	1.5	0.333	500	3.75	0.333	
HI.10	2.0	$\alpha=0$ $\beta_0=0.00228$	400	1.5	0.333	500	3.75	0.333	

TABLE 7.3

Soil and Superstructure Parameters for Cases Analyzed

(Continued)

Case	E.Q. Scale	Damping	Soil Properties						Variations in Superstructure Properties
			Lateral			Vertical			
		α,β	c_x(kip/ft³)	γ_x(ksf)	α_x	c_y(kip)ft³	γ_y(ksf)	α_y	
HI.11	2.0	$\alpha=0$ $\beta=0.0028$	400	1.5	0.333	500	3.75	0.333	None
HI.12	1.5	$\alpha=3.38$; $\beta=0.00055$	400	1.5	0.333	500	3.75	0.333	
HI.13	2.0		400	1.5	0.333	500	3.75	0.333	
HI.14	2.0		400	1.5	0.333	500	3.75	0.333	
PD.1	0.4		∞	∞	1.000	∞	∞	1.000	
PD2	0.4		400	∞	1.000	1200	∞	1.000	
PD3	0.4		400	1.5	0.333	1200	3.75	0.333	
PD4b	0.4		400	1.5	0.333	1200	3.75	0.333	$I_g=0.5I_p$ 4 top beams
PD5	0.4		400	1.0	0.333	1200	3.75	0.333	None
PD6	0.4		400	1.0	0.250	1200	3.33	0.250	
PD8	0.4		400	1.5	0.333	500	3.75	0.333	
PD12	0.5		400	1.5	0.333	1200	3.75	0.333	

designated in the table as HI and PD respectively. Following the fixed-base cases, HI.1 and PD.1, is that of constant spectral acceleration (CSA), which is also analyzed in this case by spectral techniques. The linear soil-structure interactive system is considered in cases HI.2 and PD.2. In the remaining cases uplift was permitted, and the soil properties were assumed to be nonlinear. It can be seen that large variations were allowed in the stiffness specified by the Winkler stiffness Cx, and yield levels (Y_x) of the lateral soil spring, and in the vertical spring stiffness (Cy). Smaller variations were made in the yield levels of the vertical springs (Y_y) and in the yield ratios α_x and α_y (see Fig. 7.3). The lower bound on the parameters was rather arbitrarily chosen, but with a view to ensuring conservative values for the hysteretic damping. Some variations in superstructure parameters were also considered (HI.4 and PD.4), and changes in the viscous damping parameters were introduced in cases HI.10 and HI.11. The effects of record scaling are explored by means of cases HI.12 and PD.12, while the effects of simultaneous excitation by the horizontal and vertical components of ground motion are considered in cases HI.13 and PD.13.

The main findings are summarized in Tables 7.4, 7.5 and 7.6. In Table 7.4 the response values are presented for the linear fixed-base and interactive systems. Tables 7.5 and 7.6 present the results for the non-linear interactive system. It will be recalled that apart from cases HI.12 and PD.12 the acceleration levels of the two records were scaled to obtain similar fixed-base first-story responses in the linear analysis.

In these tables the maxima of the most important response values are listed. For the superstructure these are the wall shear forces, bending moments and axial forces at ground level, as well as the lateral roof

TABLE 7.4

Linear Response: Comparative Results

	Fixed Base			Interactive	
	CSA(1)	HI.1	PD.1	HI.2	PD.2
Wall Shear (kips)	562	559	571	802	580
Wall Moment (kip-ft)	6858	6730	6991	8043	5728
Wall Axial Compression (kips)	724	661	698	1035	859
Base Shear (kips)	1124	1118	1142	1787	1331
Foundation Axial Force (kips)	1561	1367	1449	2507	2048
Lateral Roof Displacement (in)	0.21	0.20	0.23	0.66	0.48
Downward Displacement (in) (2)				0.11	0.09
Upward Displacement (in) (2)				0.06	0.07

(1) Constant spectral acceleration at 0.9g

(2) At external column line; includes static settlement of about 0.03 in.

displacements. The maximum horizontal and vertical forces in the soil springs are also given together with the maximum downward displacement and uplift of the outermost foundation spring. Note that the vertical axial force in the foundation springs given in the table is an upper bound, since it consists of the sum of the maxima in two springs, and these do not always occur simultaneously.

Cases HI.3 and PD.3 are emphasized in Tables 7.5 and 7.6 as these are considered the most representative of the nonlinear calculations. The results of other cases, which arise from changes in the properties as indicated in Table 7.3, can be compared with HI.3 and PD.3 to see the effects of changes in specific parameters. The general effect of the non-linearities is perhaps best illustrated by comparing the results for HI.3 and PD.3 both to the linear interactive cases, HI.2 and PD.2 in Table 7.4, and finally, back to the fixed-base cases, HI.1, PD.1 and CSA.

The most important feature of these comparisons is the lower level of shear forces and moments and the higher levels of displacements, and in most cases compressive axial forces, which result when the nonlinear effects of soil-structure interaction are considered. For example, the lowest value of the first-story wall shear is found in case PD.6, i.e., 337 kip, which is only 59 percent of 571, the corresponding fixed-base case, for PD.1. Similarly, the lateral roof deflection in case HI.7 (1.07"), is five times higher than that of case HI.1 (0.20").

A closer look at the tables shows that the reduction in the wall shears is less pronounced than that in the bending moments. It is also seen that a weaker foundation system, which usually leads to a more flexible

TABLE 7.5

Nonlinear Response: Comparative Results

Response	HI.3	PD.3	HI.4a	PD4b	HI.5	PD.5	HI.6	PD.6	HI.8	PD.8	HI.7	HI.9
Wall Shear (kips)	481	367	469	354	507	370	481	337	455	358	518	407
Wall Moment (kip-ft)	4754	3499	4795	3692	4997	3583	4830	3264	4574	3833	5156	4042
Wall Axial Compression (kips)	901	800	890	748	898	712	940	644	944	611	944	1004
Base Shear (kips)	1223	839	1245	828	1195	842	1157	750	1068	875	1313	1139
Foundation Axial Force (1) (kips)	1911	1591	1948	1573	2018	1525	1985	1428	2047	1462	2039	2112
Lateral Roof Deflection (in)	0.95	0.46	1.01	0.47	0.79	0.46	1.00	0.46	1.33	0.71	1.07	1.24
Downward (2) Displacement (in)	0.16	0.12	0.16	0.12	0.18	0.11	0.23	0.13	0.42	0.25	0.19	0.44
Uplift (2) (in)	0.40	0.10	0.42	0.11	0.27	0.09	0.34	0.05	0.50	0.15	0.45	0.46

(1) Upper bound.
(2) At external column line; includes static settlement of about 0.03 in.

TABLE 7.6

Nonlinear Response: Comparative Results (Continued)

	HI.3	PD.3	HI.10	HI.11	HI.12	PD.12	HI.13	PD.13
Wall Shear (kips)	481	367	525	539	371	410	458	359
Wall Moment (kip-ft)	4754	3499	5298	6364	3605	4012	4617	3525
Wall Axial Compression (kips)	901	800	864	1611	685	785	925	806
Base Shear (kips)	1223	839	1297	1536	887	976	1134	821
Foundation Axial Force (1) (kips)	1911	1591	1907	3158	1570	1761	1927	1618
Lateral Roof Deflection (in)	0.95	0.46	0.94	2.29	0.48	0.65	0.84	0.44
Downward Displacement (2) (in)	0.16	0.12	0.16	0.31	0.12	0.15	0.15	0.12
Uplift (2) (in)	0.40	0.10	0.35	1.21	0.11	0.23	0.31	0.10

(1) Upperbound.
(2) At external column line; includes static settlement of about 0.03 in.

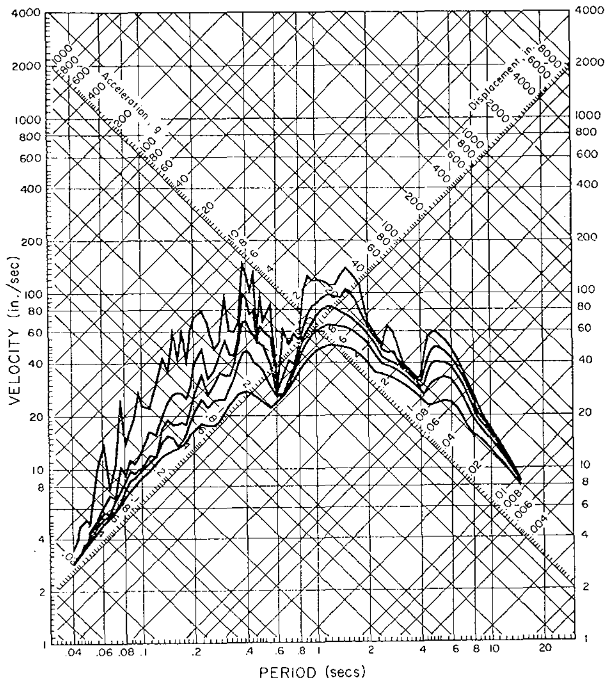
system, does not necessarily lead to lower stresses (e.g., case HI.3 vs. case HI.5).

Another feature, which is immediately apparent, is the difference in the structural response levels for the two records, although their fixed-base responses to the same excitations were similar.

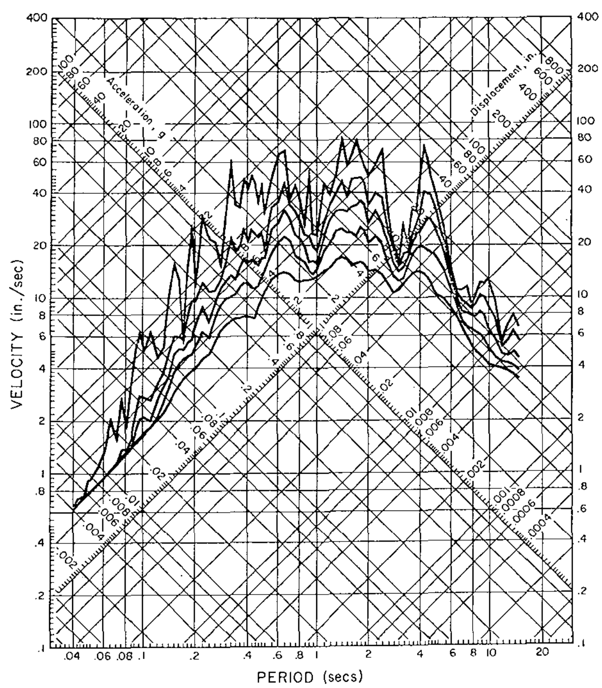
As can be seen from Fig. 7.6, the differences between the HI and PD results are mainly due to differences in the response spectra in the relevant range of natural periods. At the fundamental period of the interactive system $\tilde{T}_1 \cong 0.22$ seconds, the HI acceleration spectrum is higher than that at $T_1 = 0.14$, the fixed-base period, whereas the PD spectrum at these two points is approximately equal. The differences can be virtually removed by a change of scaling factor for the HI record, as can be seen from the results of case HI.12 in Table 7.6.

As can be seen from Tables 7.5 and 7.6, the scatter of the results is quite large, reflecting the combined effects of the irregular nature of the response spectra and the variation of parameters. Nevertheless, it is quite important to realize that all the main effects of the nonlinear soil-structure interaction were found to be favorable, i.e., they tend to lower the tensile stress level in the concrete walls -- the stress which sets the upper bound on the capacity of the structure. The very low level of axial tensile forces in the walls was not anticipated, since some unfavorable effects of the lateral-vertical coupling were expected.

It is quite easy to show that Wall A, which the present model approximates, could have carried the load combination resulting from the least favorable of the nonlinear responses to 0.4 x Pacoima Dam record. It could also survive the level of forces in case PD.12 representing peak



(a)



(b)

Fig. 7.6 Tri-partite response spectra (a) Pacoima Dam (b) Holiday Inn.

ground acceleration 25 percent higher (0.58g). This, of course, is on the basis of the same behavioral assumptions made for this wall in Chapter VI. The wall probably could have survived some of the more favorable of the 2x Holiday Inn nonlinear cases. However, the capacity of the wall would probably not have been sufficient to survive the least favorable of these cases. It is not possible to be more definite than that, since the model is only an approximation of Wall A, and the application of a load system pertaining to one structural configuration to a different one is not legitimate. Also, on the basis of the available data, it is impossible to predict how Wall A would have interacted with other walls when nonlinear soil-structure interaction behavior were assumed.

Some important features of the nonlinear effects can be gleaned from the results given in Tables 7.5 and 7.6. Apart from the lowered level of shear and moments, and the increase in lateral displacements and axial forces already noted, the most interesting one is, perhaps, the fact that response modification by the nonlinearity of the system depends to some extent on the interplay between the spectral effects of period shift due to uplift and yield in the soil on the one hand, and the increasing viscous and hysteretic damping on the other. If, with increasing effective period spectral accelerations tend to fall, the beneficial effects of interaction would be more pronounced than if the reverse were to be the case. This basically appears to be the main difference between the Holiday Inn and the Pacoima Dam records in the period range under consideration.

It had been suggested earlier that period lengthening due to partial uplift and yield might be large enough to shift the system into that part of the spectrum where the acceleration response decreases with period. This, as already noted, has not been observed.

One interesting feature of all the various cases was the low level of hysteretic energy that was actually dissipated during the motion in the coupling beams and in the vertical soil springs. Since the energy dissipated by the vertical springs is quite small due to uplift, most of the dissipation is done by the horizontal springs. However, the proportion of the time the horizontal and the vertical springs spend in the post-yield range is relatively small.

The extent to which types of viscous damping affect the response can be seen by comparing cases HI.10, HI.11 and HI.3. It will be recalled (Table 7.6) that the only difference between cases HI.10 and HI.11, is the fact that in Case HI.10 β_0 factors the original stiffness matrix of fixed-base structure, whereas in Case HI.11 β factors the current tangent stiffness matrix. The appreciably larger axial forces, roof displacement and uplift in Case HI.11 reflect the lower damping which results from the lower effective lateral stiffness of the structure. They also indicate an increase in the contribution of the higher modes. Note, however, that the larger moment in this case is to some extent offset by the higher axial compression.

The results for cases HI.13 and PD.13 given in Table 7.6 show that the vertical component of the ground motion did not appreciably affect the structural response. It thus appears that, as expected, the effect of vertical acceleration was probably of minor significance for Building 41. This is in agreement with earlier findings for other structures (38).

It is interesting to note that the long-period acceleration pulse present in the Pacoima Dam record did not appear to have had a controlling effect on the response. It appears that the period lengthening of the structure was not sufficient to make it sensitive to the pulse, which has a period of about two seconds. For periods of the range of building 41, 0.1 to 0.35 secs., other portions of the accelerogram are apparently more significant.

One may well ask whether the foundation displacements indicated by the nonlinear analysis were in any way observed on inspection following the San Fernando earthquake. It has already been stated that there was 1 to 3 in downward displacement of the grade adjacent to the north wall, and a longitudinal crack was found on the basement slab-on-grade. These effects may have possibly been caused, at least in part, by the uplift of the structure during the earthquake. In all the analyses, it was found that the yield suffered by the reinforcing steel in the corridor coupling beams was very minor. In fact, on inspection, no cracks were detected in them.

The more favorable state of stress found in the nonlinear analyses suggests that the additional capacity required to complete the reconciliation of the observed response of Building 41 with the high level of excitation at the site during the San Fernando earthquake probably came from the effects of nonlinear soil-structure interaction.

In summary, the high earthquake resistance built into the structure by the design engineers, and the additional beneficial effects of nonlinear soil-structure interaction are believed to be the two major factors responsible for the highly successful performance of Building 41 during the San Fernando earthquake.

VIII. SUMMARY AND CONCLUSIONS

8.1 Summary

Buildings 41 and 43 of the Veterans Administration Hospital complex at San Fernando survived the February 9, 1971 earthquake with only minor damage. This was in contrast to several other buildings in the same complex which collapsed, exacting a toll of 46 lives, and to the collapse of modern reinforced concrete structures at the neighboring Olive View Hospital.

The survival through very strong shaking of one of these buildings, Building 41, has been the subject matter of this investigation. This building was designed after the adoption of the earthquake design provisions in the building codes, but at a time when understanding of the earthquake resistance of structures was in its early stages. It was believed that such an investigation would lead to the identification of the most important parameters affecting the capacity of low-rise reinforced concrete shear wall buildings to resist very strong shaking.

It was also felt that an attempt should be made to see whether these parameters could be identified by means of simple, building-code oriented analyses. This was of particular importance since the structure was designed, albeit at working stress, to resist a seismic lateral force coefficient of only ten percent. Without some investigation, such a low level of seismic design criteria cannot be reconciled with the successful performance of the building through an earthquake with ground acceleration levels in the range of 0.50g to 0.75g.

Building 41 was four stories high, approximately 200 x 50 ft in plan, with a centrally located penthouse. The lateral load resisting system consisted of pierced reinforced concrete walls supported on strip footings. There were six such walls — symmetric in plan — in the transverse direction, and three longitudinal walls. In view of the apparent great strength of the longitudinal walls, the investigation focused on the transverse response only.

All buildings on the site were razed sometime after the earthquake, and thus, the properties of construction materials had to be inferred from the blueprints and calculation sheets of the structural designers, as well as from ASTM Standards, the 1937 edition of the Uniform Building Code and from knowledge of structural engineering practice in the mid-30's. Since only minor cracks were detected after the earthquake in the lightly reinforced concrete walls, it was assumed that the tensile strength of concrete was not exceeded. This strength is approximately proportional to the square root of the compressive strength. Thus, the resisting capacity of the structure was not very sensitive to errors in the assumed compressive strength of the concrete.

Regarding soil properties, whose variability is high, the basic stiffness parameters were computed on the basis of available data on the shear wave velocity near the site, while the bearing capacity was estimated from the known properties of deposits of similar nature.

First, an approximate lateral load analysis of a simplified fixed-base model of the structure was carried out. The model consisted of the six transverse coupled shear walls, with flanges to simulate the longitudinal walls. These walls were loaded as required by the seismic provisions of the current edition of the Uniform Building Code, and analyzed by hand using the continuous medium approach. The lateral loads were distributed among the walls assuming equal lateral displacements at roof level. The results indicated that with a fixed-base the structure could resist loads implied by a lateral force coefficient of about 0.4g. This result is based on uncracked concrete sections in those walls which were insufficiently reinforced.

A complete three-dimensional model of the structure was then dynamically analyzed for a flat acceleration spectrum and the lateral force capacity of the structure was again evaluated. From this analysis it was also possible to consider, in an approximate fashion, the ability of the connecting beams to yield before the tensile capacity of the walls was exhausted. It was found that the dynamic properties, including the lateral capacity of the structure, determined by the static lateral load procedure were in good agreement with the more exact dynamic analysis.

In order to investigate the nonlinear effects of soil-structure interaction, it was necessary to reduce the complexity of the structural model. For this purpose, a simplified two-dimensional version of the model, in fact an approximation of Wall A, was isolated from the structure and its response to two accelerograms recorded near the site was investigated. The sensitivity of the response to modelling assumptions for the structure and soil was also considered. The results indicated that non-linear soil-structure interaction effects lead to lower shears and moments and to higher compressive axial forces in the concrete walls. All these effects tend to increase the ability of the structure to survive strong ground shaking.

8.2 Conclusions

Considering the approximate nature of the modelling and the uncertainties in the ground motion, it cannot be claimed that any of the analyses produces a completely satisfactory reconciliation of the three controlling factors of the problem: the observed successful behavior of Building 41 during the San Fernando earthquake; its dynamic resisting capacity as indicated by the material properties, design and construction; and the level of strong shaking that occurred at the site. It is believed, however, that the analyses do show fairly convincingly that the key to the successful response of the building is to be found in the combined effects of the two factors: 1) the large strength built into the structure, which was sustained through proper detailing; and 2) the beneficial effects of non-linear soil-structure interaction.

The material and soil properties of Building 41 are not known accurately, thus it is not possible to know with precision the extent to which they contributed to the successful performance of the building. The question then arises as to whether the analysis and response of the building can be reconciled simply by a combination of higher, but still reasonable material strengths, and acceptable changes in the modelling procedures. For example, a case can perhaps be made for a reconciliation of response and analysis based on the hypothesis that the tensile strength of concrete may have been appreciably higher. However, it should not be overlooked that assuming a tensile strength which is 40 percent higher is equivalent to assuming a 100 percent increase in the compressive strength, i.e., 8000 psi. Such strength does not appear likely.

Also, it might be claimed that the contribution of the frame action of the ribbed slabs acting as beams together with the internal columns and the longitudinal walls (acting as wide columns in their weak direction) estimated in the study to be at most 5 percent may, in fact, have been higher. Since the ribbed slabs were not modelled in any of the analyses, it is difficult to assess their contribution in carrying the base shear. Yet, in the analysis of the entire building the longitudinal walls were taken into account as cantilever columns (in the weak direction) and some frame action was considered through the 4-1/2" concrete slab in the porch. Nevertheless, their total contribution to base shear amounted to only two percent. Since the effective stiffness of the cross walls is substantially lowered when yielding of the connecting beams occurs, the relative contribution of the frame system, which is affected by a lesser extent, is thereby increased. However, in view of the low level of cracking observed in the structure, and the small lateral displacements estimated, it is hard to believe that frame action could have contributed a substantial amount to the structural response.

From analyses not reported herein it was found that, as expected, when soil properties are assumed to be linearly elastic, the base shear becomes higher (for both records). Thus it appears that assuming a stronger foundation soil does not help the reconciliation.

There is the further possibility that either radiation damping was substantially underestimated or that due to some special features of the local geology the ground acceleration at the site may have been much lower than elsewhere in the vicinity. Both possibilities seem unlikely. For high frequencies, when the wavelength of the ground motion is small compared with the width of the building, the earthquake input is less than that of the free-field ground motion. This is because the inplane rigidity of the structural foundation system tends to average the phase differences in the ground motion under the building (see e.g. Ref. 39). For wide buildings this may appreciably lower the acceleration response spectra at lower natural periods. In this case the width of the building, 200 ft., does not appear to be large enough to suggest that the response spectra could have been significantly lower at the fundamental period of the building.

It was the consideration of the unlikely nature of these possibilities for reconciling response and analysis that led the authors to the conclusion that the most likely reason for the successful performance of the structure, in addition to its high basic strength, was the effect of non-linear soil behavior.

In the following paragraphs other conclusions are summarized, and some additional observations are made.

8.2.1 Superstructure

A low level of tensile stresses is necessary to ensure the survival of nominally reinforced low-rise coupled shear wall structures through a severe earthquake. This can only be achieved by means of strong and stiff coupling beams, including the highly-stressed coupling beam at foundation level, a fact which was recognized by the designers of Building 41. Such coupling beams ensure a large lever arm for the overturning moment on the structure, and reduce the flexural moment in the individual wall sections. The axial tensile forces are further lowered, not only by the weight of the structure but also by the impact that occurs with recontact after partial uplift. In fact, it is quite inexpensive to raise the moment capacity of the walls to a level which would preclude brittle failure.

Note that in such structures energy can be dissipated by hysteresis only in the coupling beams. However, in view of their high strength, their contribution to the overall damping in the system is marginal.

8.2.2 Soil-Structure Interaction

Although it nearly always helps on the average, taking into account linear soil-structure interaction in an individual earthquake does not necessarily lower the level of internal forces in the structure compared with the fixed-base situation. Although the results of the analyses reported in Chapter VII make this quite clear, the statement should be qualified in view of the simplifying assumptions made in modelling the soil, and the relatively low level of effective viscous damping stipulated (7-8 percent in the majority of the cases). Nevertheless, one should not overlook the possibility that in some earthquakes the spectral acceleration may rise steeply with increasing period, thereby leading to a higher response than for the fixed-base case, even though soil-structure interaction may also increase the damping.

Partial uplift and yielding in the soil tend to reduce the seismic forces in the structure, and therefore, should not necessarily be avoided by structural designers. These effects may not be as beneficial as might be expected, however, for the reasons outlined above. Note that special attention should be paid to possible stress reversals due to uplift in overhanging elements, and to the much higher level of stresses in those parts of the base remaining in contact with the soil.

In most cases studied, uplift was very small (<0.5 in). However, together with the downward displacement at the opposite edge of the building, it accounted for at least one half of the lateral roof displacement.

An H-shape gives the base very large strength, since during uplift the area in contact with the soil is not substantially reduced, and this accounts for the low level of stresses and displacements in the soil found in the analyses.

Considering the vertical component of ground motion does not appear to increase the level of response for this structure. Yet, it would be useful to know the circumstances under which unfavorable states of stress could arise.

8.2.3 Structural Modelling

It has been shown that very simple analytical procedures can lead to results which are in good agreement with more sophisticated analytical techniques capable of modelling the three-dimensional nature of the structural system. It appears that the continuum approach used for the static analysis of coupled shear walls can be useful, for example, even in situations that are not ideally suitable for its application.

Correct modelling of force transfer through corners and shear lag effects through flanges still remain difficult problems with computer programs oriented towards analyzing beams and frames, even if rigid zones at the joints are included. With such programs a correct accounting for these effects proves to be a rather difficult problem in engineering judgment. It is, therefore, believed that finite element programs specifically designed to analyze three-dimensional shear wall and frame structures should be developed for use in engineering practice.

The assumption of in-plane rigidity of the floor slabs becomes less tenable with increasing length-to-width ratios, particularly for shear wall structures with low height to width ratios. However, it was found to be a working hypothesis, for first mode dominated response, even for Building 41 in which the floow plan aspect ratio was about four, and the height to width ratio was less than 1.5. It is interesting to add, moreover, that allocating the wall forces by means of tributary areas, which is often inappropriate, may well lead to acceptable results under such circumstances.

Modelling soil-structure interaction stiffness parameters by means of axial springs is a relatively crude approximation, but is quite straightforward to implement. Apart from the geometrical assumptions, it is not easy to evaluate the damping factors for the equivalent viscous damping in the soil-structure interactive system, particularly when uplift is considered. In this respect the approach presented in the Applied Technology Council Tentative Provisions ATC3-06 (21) is believed to be a step in the right direction.

The survival with minor damage of Buildings 41 and 43 of the Veterans Administration Hospital through the San Fernando earthquake of February 9, 1971 is one of many cases which show that well designed structures are able to resist the effects of intense ground motion. The analyses performed in this study indicate that for many of these structures, their successful performance depends on possessing great strength and on the beneficial effects of nonlinear response of the foundation soils.

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APPENDIX A

THE 1937 EDITION OF THE UNIFORM BUILDING CODE

Lateral Bracing

Sec. 2312. (a) **General.** Every building or structure and every portion thereof, except Type V buildings of Group I occupancy which are less than twenty-five feet (25') in height, and minor accessory buildings, shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation. Such force shall be proportional to the total dead plus one-half the vertical design live load, except for warehouses, in which case such force shall be proportional to the total dead plus the total vertical live load. The force shall be assumed to come from any horizontal direction.

All bracing systems both horizontal and vertical shall transmit all forces to the resisting members and shall be of sufficient extent and detail to resist the horizontal forces provided for in this section and shall be located symmetrically about the center

TABLE No. II-A—HORIZONTAL FORCE FACTORS

Part or Portion	Value of "C"*	Direction of Force
The building as a whole**	.02 on soil, over 2000 lbs. .04 on soil, up to 2000 lbs.	Any direction horizontally
Bearing walls, curtain walls, enclosure walls, panel walls	.05	Normal to surface of wall
Cantilever parapet and other cantilever walls, except retaining walls	.25	Normal to surface of wall
Exterior and interior ornamentations and appendages	.25	Any direction horizontally
Towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses	.05	Any direction horizontally

*See map on page 282 for zones. The values given "C" are minimum and should be adopted in locations not subject to frequent seismic disturbances as shown in Zone 1. For locations in Zone 2, "C" should be doubled. For locations in Zone 3, "C" should be multiplied by four.

**Where wind load of 20 pounds per square foot would produce higher stresses, this load should be used in lieu of the factor shown.

of mass of the building or the building shall be designed for the resulting rotational forces about the vertical axis.

Junctures between distinct parts of buildings, such as wings which extend more than twenty feet (20') from the main portion of the building, shall be designed at the juncture with other parts of the building for rotational forces, or the juncture may be made by means of sliding fragile joints having a minimum width of not less than eight inches (8"). The details of such joints shall be made satisfactory to the Building Inspector.

(b) **Horizontal Force Formula.** In determining the horizontal force to be resisted, the following formula shall be used:

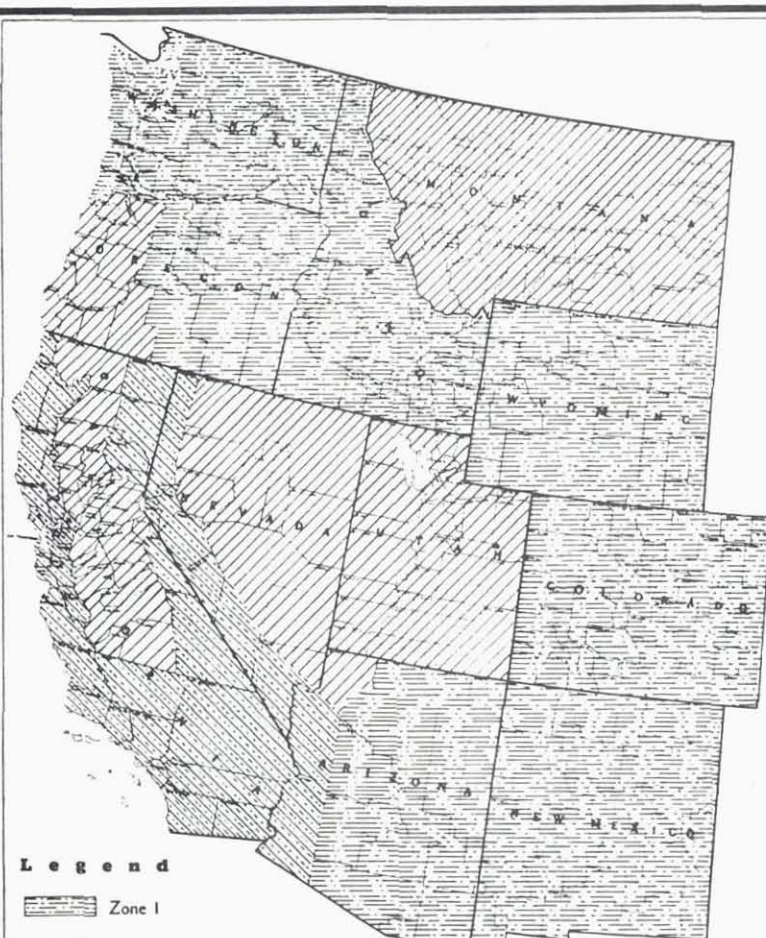
$$F = CW$$

where "F" equals the horizontal force in pounds.

"W" equals the total dead load plus one-half the total vertical designed live load,

at and above the point of elevation under consideration, except for warehouses, in which case "W" shall equal the total dead load plus the total vertical designed live load at and above the point or elevation under consideration. Machinery or other fixed concentrated loads shall be considered as part of the dead load.

"C" equals a numerical constant as shown in Table No. II-A.



Map of the 11 Western States

showing

Zones of Approximately Equal Seismic Probability

Lateral Bracing (Con't.)

(c) **Foundation Ties.** In the design of buildings of Types I, II and III, where the foundations rest on piles or on soil having a safe bearing value of less than 2,000 pounds per square foot, the foundations shall be completely inter-connected in two directions approximately at right angles to each other. Each such inter-connecting member shall be capable of transmitting by both tension and compression at least 10 per cent of the total vertical load carried by the heavier only of the footings or foundations connected. The minimum gross size of each such member if of reinforced concrete shall be twelve inches by twelve inches (12"x12") and shall be reinforced with not less than the minimum reinforcement specified in Section 2620. If the inter-connecting members are of structural steel, they shall be designed as provided in Section 2702, and encased in concrete. A reinforced concrete slab may be used in lieu of inter-connecting tie members, providing the slab thickness is not less than one-forty-eighth of the clear distance between the connected foundations; also providing the thickness is not less than six inches (6").

The inter-connecting slabs shall be reinforced with not less than eleven-hundredths square inch (.11 sq. in.) of steel per foot of slab in a longitudinal direction and the same amount of steel in a transverse direction. The bottom of such slab shall not be more than twelve inches (12") above the tops of at least 80 per cent of the piers or foundations. The footings and foundations shall be tied to the slab in such a manner as to be restrained in all horizontal directions.

(d) **Plans and Design Data.** With each set of plans filed, a brief statement of the following items shall be included:

1. A summation of the dead and live load of the building, floor by floor, which was used in figuring the shears for which the building is designed.

2. A brief description of the bracing system used, the manner in which the designer expects such system to act, and a clear statement of any assumptions used. Assumption as to location of all points of counter-flexure in members must be stated.

3. Sample calculation of a typical bent or equivalent.

(e) **Stresses.** Stresses in materials shall not exceed by more than 33⅓ per cent the allowable working stresses permitted in this Code, except that rivets may be stressed the same in tension as is allowed in shear. The allowable shear in reinforced concrete walls, six inches (6") or more in thickness, shall not exceed five one-hundreds of the ultimate compressive strength of the concrete.

(f) **Detailed Requirements.** 1. **Bonding and Tying.** Cornices and ornamental details shall be bonded in the structure so as to form an integral part of it. This applies to the interior as well as to the exterior of the building.

2. **Overturning Moment.** In no case shall the overturning moment of any building and/or structure due to the forces provided for in this Section exceed 50 per cent of the moment of stability of such building and/or structure.

3. **Additions.** Every addition to an existing building and/or structure shall be designed and constructed to resist and withstand the forces provided for in this Section, and in any case where an existing building and/or structure is increased in height all portions thereof affected by such increased height shall be reconstructed to resist and withstand the forces provided for in this Section.

4. **Alterations.** No existing building and/or structure shall be altered and/or reconstructed in such a manner that the resistance to the forces provided for in this Section will be less than that before such alteration and/or reconstruction was made; provided, however, that this provision shall not apply to non-bearing partitions, and shall not apply to other minor alterations which are made in a manner satisfactory to the Building Department.

(e) **Lime Mortars.** Lime mortars shall not be used in any unit masonry construction forming a part of a building.

(f) **Veneer Ties.** Veneer ties provided in Section 2936 shall be of sufficient strength to support the full weight of the veneer in tension.

INTENTION OR INTERPRETATION OF LATERAL FORCE PROVISIONS

These lateral force requirements are intended to make buildings earthquake-resistive. The provisions of this Section apply to the buildings as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

The provisions incorporated in this Section are general and, in specific cases, may be interpreted and/or added to as to detail by rulings of the Building Inspector in order that the intent shall be fulfilled.

APPENDIX B

EARTHQUAKE REGULATIONS OF THE 1976 EDITION OF THE UNIFORM BUILDING CODE

Earthquake Regulations

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

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

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

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

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

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

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

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

ESSENTIAL FACILITIES—See Section 2312 (k).

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

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

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

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

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

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

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

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

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

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

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

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

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

g = Acceleration due to gravity.

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

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

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

Level i

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

i = 1 designates the first level above the base.

Level n

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

Level x

= That level which is under design consideration.

x = 1 designates the first level above the base.

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

S = Numerical coefficient for site-structure resonance.

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

T_s = Characteristic site period.

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

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

EXCEPTION: "W" shall be equal to the total dead load plus 25 percent of the floor live load in storage and warehouse occupancies. Where the design snow load is 30 psf or less, no part need be included in the value of "W." Where the snow load is greater than 30 psf, the snow load shall be included; however, where the snow load duration warrants, the Building Official may allow the snow load to be reduced up to 75 percent.

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

W_p = The weight of a portion of a structure.

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

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

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

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

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

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

The value of C need not exceed 0.12.

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

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

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

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

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

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

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

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

$$\text{For } T/T_s = 1.0 \text{ or less} \quad S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2 \ldots\ldots\ldots\ldots\ldots\ldots (12\text{-}4)$$

$$\text{For } T/T_s \text{ greater than } 1.0 \quad S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2 \ldots\ldots\ldots\ldots\ldots\ldots (12\text{-}4A)$$

WHERE:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(g) **Lateral Force on Elements of Structures.** Parts or portions of structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = ZIC_p S W_p \qquad \text{(12-8)}$$

EXCEPTION: Where C_p in Table No. 23-J is 1.0 or more the value of I and S need not exceed 1.0.

The distribution of these forces shall be according to the gravity loads pertaining thereto.

(h) **Drift and Building Separations.** Lateral deflections or drift of a story relative to its adjacent stories shall not exceed 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated.

The displacement calculated from the application of the required lateral forces shall be multiplied by $(1.0/K)$ to obtain the drift. The ratio $(1.0/K)$ shall be not less than 1.0.

All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

(i) **Alternate Determination and Distribution of Seismic Forces.** Nothing in Section 2312 shall be deemed to prohibit the submission of properly substantiated technical data for establishing the lateral forces and distribution by dynamic analyses, in such analyses the dynamic characteristics of the structure must be considered.

(j) **Structural Systems.** 1. **Ductility requirements.** A. All buildings designed with a horizontal force factor $K = 0.67$ or 0.80 shall have ductile moment resisting space frames.

B. Buildings more than 160 feet in height shall have ductile moment resisting space frames capable of resisting not less than 25 percent of the required seismic forces for the structure as a whole.

> **EXCEPTION:** Buildings more than 160 feet in height in Seismic Zone No. 1 may have concrete shear walls designed in conformance with Section 2627 of this Code in lieu of a ductile moment resisting space frame, provided a K value of 1.00 or 1.33 is utilized in the design.

C. In Seismic Zones No. 2, No. 3 and No. 4 all concrete space frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be ductile moment resisting space frames.

> **EXCEPTION:** Frames in the perimeter line of the vertical support of buildings designed with shear walls taking 100 percent of the design lateral forces need only conform with Section 2312 (j) 1D.

D. In Seismic Zones No. 2, No. 3 and No. 4 all framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity and induced moment due to $3/K$ times the distortions resulting from the Code required lateral forces. The rigidity of other elements shall be considered in accordance with Section 2312 (e) 4.

E. Moment resisting space frames and ductile moment resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

F. The necessary ductility for a ductile moment resisting space frame shall be provided by a frame of structural steel with moment resisting connections (complying with Section 2722 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2723 for buildings in Seismic Zone No. 1) or by a reinforced concrete frame (complying with Section 2626 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2625 for buildings in Seismic Zone No. 1).

G. In Seismic Zones No. 2, No. 3 and No. 4 all members in braced frames shall be designed for 1.25 times the force determined in accordance with Section 2312 (d). Connections shall be designed to develop the full capacity of the members or shall be based on the above forces without the one-third increase usually permitted for stresses resulting from earthquake forces.

Braced frames in buildings shall be composed of axially loaded bracing members of A36, A440, A441, A501, A572 (except Grades 60 and 65) or A588 structural steel; or reinforced concrete members conforming to the requirements of Section 2627.

H. Reinforced concrete shear walls for all buildings shall conform to the requirements of Section 2627.

I. In structures where $K = 0.67$ and $K = 0.80$, the special ductility requirements of structural steel (complying with Section 2722 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2723 for buildings in Seismic Zone No. 1) or by reinforced concrete (complying with Section 2626 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or with Section 2625 for buildings in Seismic Zone No. 1), as appropriate, shall apply to all structural elements below the base which are required to transmit to the foundation the forces resulting from lateral loads.

2. Design requirements. A. **Minor alterations.** Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this Section.

B. **Reinforced masonry or concrete.** All elements within structures located in Seismic Zones No. 2, No. 3 and No. 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 24 and 26. Principal reinforcement in masonry shall be spaced 2 feet maximum on center in buildings using a moment resisting space frame.

C. **Combined vertical and horizontal forces.** In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered. Consideration should also be given to minimum gravity loads acting in combination with lateral forces.

D. **Diaphragms.** Floor and roof diaphragms shall be designed to resist the forces set forth in Table No. 23-J. Diaphragms supporting concrete or masonry walls shall have continuous ties between diaphragm chords to distribute, into the diaphragm, the anchorage forces specified in this Chapter. Added chords may be used to form sub-diaphragms to transmit the anchorage forces to the main cross ties. Diaphragm deformations shall be considered in the design of the supported walls. See Section 2312 (j) 3 A for special anchorage requirements of wood diaphragms.

3. **Special requirements.** A. **Wood diaphragms providing lateral support for concrete or masonry walls.** Where wood diaphragms are used to laterally support concrete or masonry walls the anchorage shall conform to Section 2310. In Zones No. 2, No. 3 and No. 4 anchorage shall not be accomplished by use of toe nails, or nails subjected to withdrawal; nor shall wood framing be used in cross grain bending or cross grain tension.

B. **Pile caps and caissons.** Individual pile caps and caissons of every building or structure shall be interconnected by ties, each of which can carry by tension and compression a minimum horizontal force equal to 10 percent of the larger pile cap or caisson loading, unless it can be demonstrated that equivalent restraint can be provided by other approved methods.

C. **Exterior elements.** Precast, nonbearing, nonshear wall panels or similar elements which are attached to or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of cast-in-place concrete or by mechanical fasteners in accordance with the following provisions.

Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or $(3.0/K)$ times story drift caused by required seismic forces; or ¼ inch, whichever is greater.

Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

Connections to permit movement in the plane of the panel for story drift shall be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel or other connections providing equivalent sliding and ductility capacity.

(k) Essential Facilities. Essential facilities are those structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public. Such facilities shall include but not be limited to:

1. Hospitals and other medical facilities having surgery or emergency treatment areas.
2. Fire and police stations.
3. Municipal government disaster operation and communication centers deemed to be vital in emergencies.

The design and detailing of equipment which must remain in place and be functional following a major earthquake shall be based upon the requirements of Section 2312 (g) and Table No. 23-J. In addition, their design and detailing shall consider effects induced by structure drifts of not less than $(2.0/K)$ times the story drift caused by required seismic forces nor less than the story drift caused by wind. Special consideration shall also be given to relative movements at separation joints.

(1) Earthquake Recording Instrumentations. For earthquake recording instrumentations see Appendix, Section 2312 (1).

TABLE NO. 23-I—HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE[2] OF K
1. All building framing systems except as hereinafter classified	1.00
2. Buildings with a box system as specified in Section 2312 (b)	1.33
3. Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames using the following design criteria: a. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames b. The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces c. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force	0.80
4. Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force	0.67
5. Elevated tanks plus fill contents, on four or more cross-braced legs and not supported by a building	2.5[3]
6. Structures other than buildings and other than those set forth in Table No. 23-J	2.00

[1]Where wind load as specified in Section 2311 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

[2]See Figure Nos. 1, 2 and 3 this chapter and definition of "Z" as specified in Section 2312 (c).

[3]The minimum value of "KC" shall be 0.12 and the maximum value of "KC" need not exceed 0.25.

The tower shall be designed for an accidental torsion of five percent as specified in Section 2312 (e) 5. Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described above shall be designed in accordance with Section 2312 (g) using "C_p" = .2.

TABLE NO. 23-J—HORIZONTAL FORCE FACTOR "C_p" FOR ELEMENTS OF STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
1. Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions. Masonry or concrete fences	Normal to flat surface	0.20[1]
2. Cantilever parapet	Normal to flat surface	1.00
3. Exterior and interior ornamentations and appendages.	Any direction	1.00
4. When connected to, part of, or housed within a building: a. Towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouse	Any direction	0.20[2]
b. Storage racks with the upper storage level at more than 8 feet in height plus contents		0.20[2,3]
c. Equipment or machinery not required for life safety systems or for continued operations of essential facilities		0.20[2,4]
d. Equipment or machinery required for life safety systems or for continued operation of essential facilities		0.50[4,5]
5. When resting on the ground, tank plus effective mass of its contents.	Any direction	0.12
6. Suspended ceiling framing systems (Applies to Seismic Zones Nos. 2, 3 and 4 only)	Any direction	0.20[6]
7. Floors and roofs acting as diaphragms	Any direction	0.12[7]
8. Connections for exterior panels or for elements complying with Section 2312 (j) 3C.	Any direction	2.00
9. Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	0.30[8]

[1]See also Section 2309 (b) for minimum load on deflection criteria for interior partitions.

[2]When located in the upper portion of any building where the h_n/D ratio is five-to-one or greater the value shall be increased by 50 percent.

[3]W_p for storage racks shall be the weight of the racks plus contents. The value of C_p for racks over two storage support levels in height shall be 0.16 for the levels below the top two levels. In lieu of the tabulated values steel storage racks may be designed in accordance with U.B.C. Standard No. 27-11.

Where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the design coefficients may be as for a building with K values from Table No. 23-I, $CS = 0.20$ for use in the formula $V = ZIKCSW$ and W equal to the total dead load plus 50 percent of the rack rated capacity. Where the design and rack configurations are in accordance with this paragraph the design provisions in U.B.C. Standard No. 27-11 do not apply.

[4]For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

[5]For Essential Facilities and life safety systems, the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider drifts in accordance with Section 2312 (k). The product of IS need not exceed 1.5.

[6]Ceiling weight shall include all light fixtures and other equipment which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.

[7]Floors and roofs acting as diaphragms shall be designed for a minimum force resulting from a C_p of 0.12 applied to w_x unless a greater force results from the distribution of lateral forces in accordance with Section 2312 (e).

[8]The W_p shall include 25 percent of the floor live load in storage and warehouse occupancies.

TABLE NO. 23-K
VALUES FOR OCCUPANCY IMPORTANCE FACTOR I

TYPE OF OCCUPANCY	I
Essential Facilities[1]	1.5
Any building where the primary occupancy is for assembly use for more than 300 persons (in one room)	1.25
All others	1.0

[1]See Section 2312 (k) for definition and additional requirements for essential facilities.

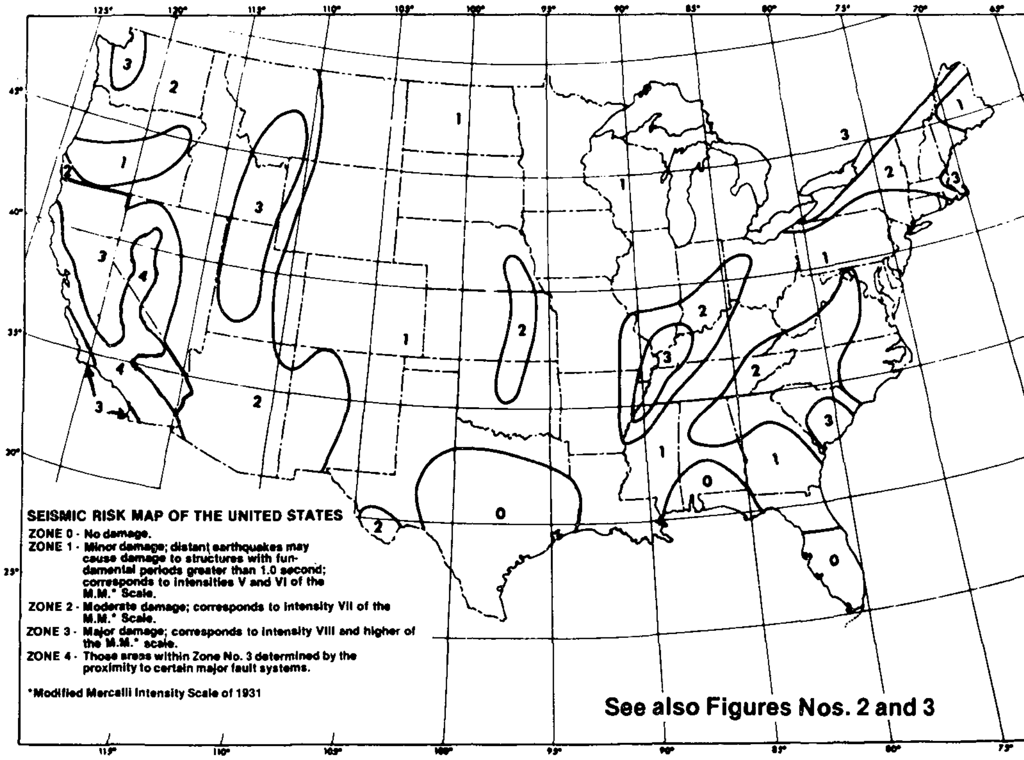


FIGURE NO. 1—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States see Appendix Chapter 23

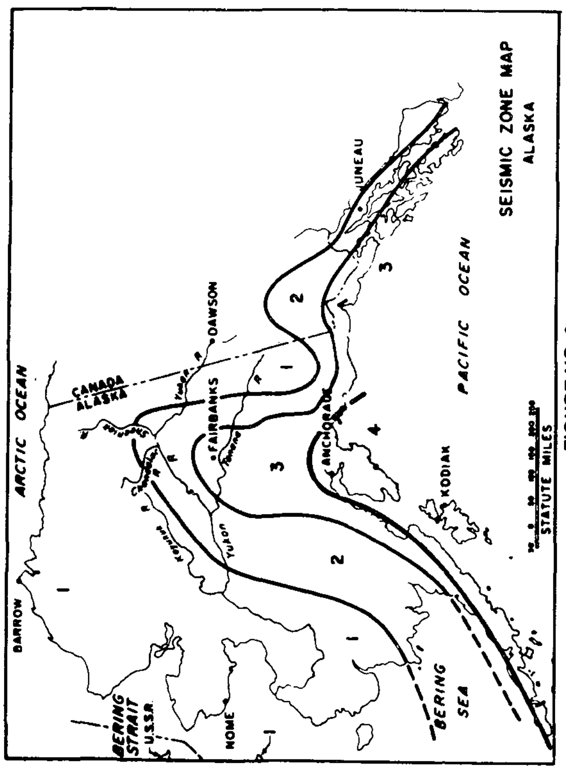


FIGURE NO. 2

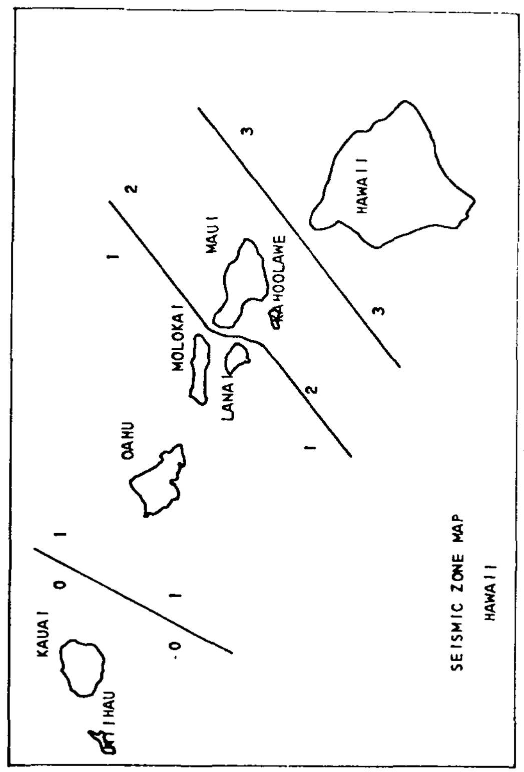


FIGURE NO. 3

APPENDIX C

EFFECT OF IN-PLANE FLOOR FLEXIBILITY ON LATERAL FORCE DISTRIBUTION AMONG SHEAR WALLS

In the analysis of multi-story structures for earthquake loading — either static or dynamic — it is usually assumed that the in-plane flexibility of the floor slabs does not significantly affect the load distribution among the structural assemblages, nor does it change the dynamic properties of the structure which are associated with the lower modes of vibration. Some numerical evidence was presented to show that this is also the case in low-rise elongated building structure laterally supported by frames (22). However, this assumption becomes less realistic for such buildings when they are supported by wide shear walls. Thus the assumption of in-plane floor rigidity is in fact the major limitation on the applicability of the program ETABS to structures having the characteristics of Building 41. Since the program was used for the analysis reported in Chapter VI, it was important to obtain some estimate on the errors likely to be associated with this assumption. However, a check on the accuracy of the assumption requires a computer program in which floor flexibility can be considered which, evidently was not available. Therefore, an alternative structural scheme had to be adopted. This scheme was based on the similarity between the stiffness matrix associated with the shear beam and that associated with axially loaded rods, i.e. both responses are near coupled, thus permitting the solution of the 3-dimensional problem by a plane frame program.

From the approximate analysis reported in Chapter V it was found that the shear related displacements were of the order of 40 to 60 percent of the total roof deflections, depending on the assumption made regarding the effective width of the flanges (longitudinal and participation). Therefore, it appeared that assuming the total deflection to be shear dependent was quite reasonable for the purpose of this check. The equivalent shear rigidity of each wall was computed based on the roof displacements as derived from the approximate analysis for the wide flange assumption.

The shear dependence of the deflected shape of the structure permitted replacing the beams by columns having a cross-sectional area equal to the beams shear areas and with Young's modulus being equal to the beams' shear modulus. Similarly, the lateral forces acting on the structure were replaced by numerically equal vertical loads.

The floors were modelled as flanged beams; their effective wed thickness was taken as the slab thickness plus the rib area smeared over the rib spacing. The effective width of the flanges was taken as the net height of the longitudinal walls between window (or door) openings. The stairwell was assumed to reduce the effective shear area of the web (floor slab) in proportion to the ratio of its total plan area to the total floor area in the respective span. The model solved as a two-dimensional problem is shown in Fig. C.1.

The results of the computation by means of the ETABS program are summarized in Table C.1. In the table the distribution of the lateral shear forces among the three walls at every floor level is given for three alternative ways of distributing the lateral loads. The column headed by $I=\infty$ shows the percentage of the load carried by the walls when the rigid floor assumption is made. Similarly, under the heading $I=0$ the lateral loads are assumed to be distributed among the walls in proportion

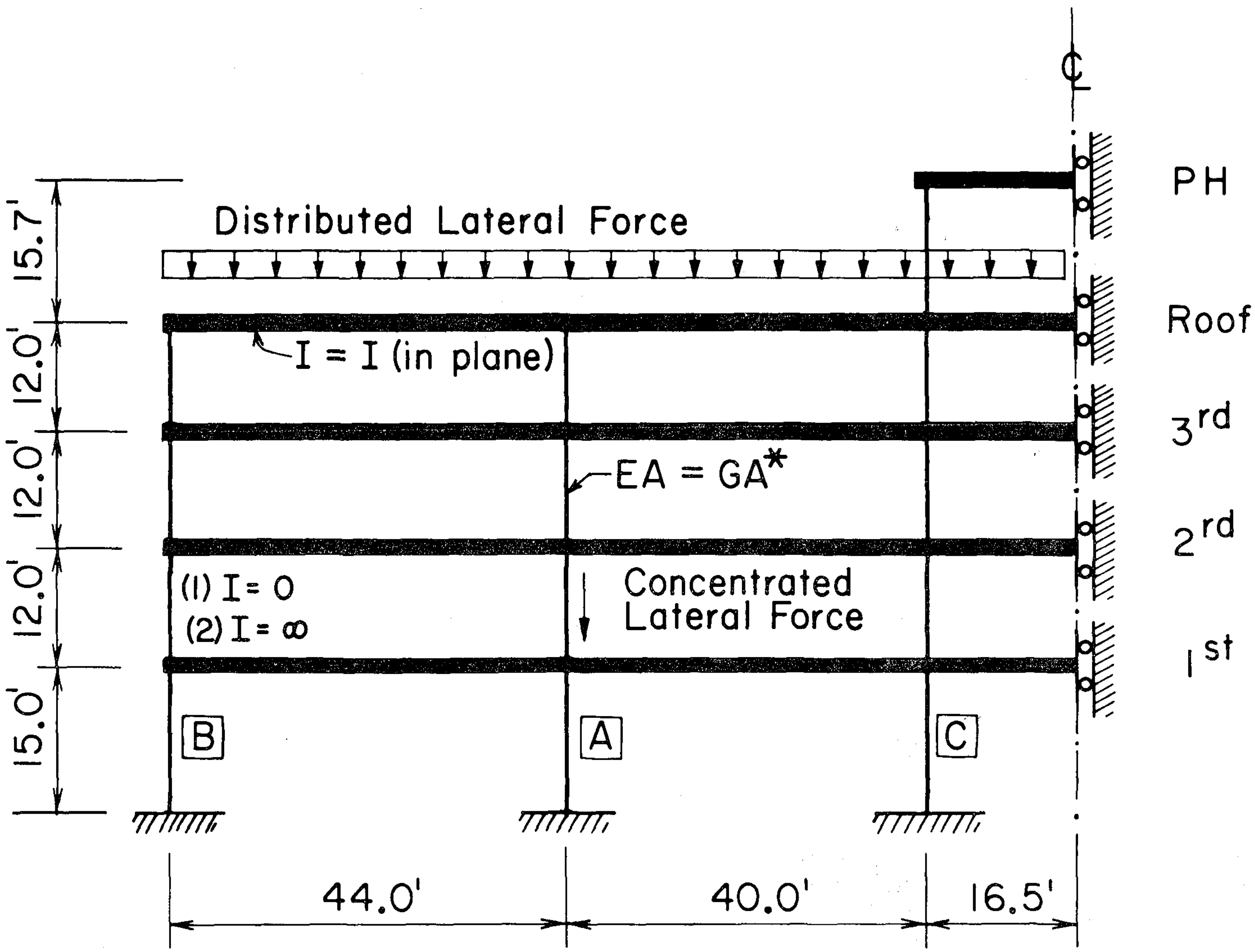


Fig. C-1. Axial force analogy for structure with finite floor slab rigidity.

TABLE C.1

Effect of In-Plane Floor Flexibility on Lateral
Shear Force Distribution Among Walls (Percent)

	WALL A				WALL B				WALL C			
	$I=\infty$	$I=I$		$I=0$	$I=\infty$	$I=I$		$I=0$	$I=\infty$	$I=I$		$I=0$
		(1)	(2)			(1)	(2)			(1)	(2)	
Roof	27.3	25.5	25.1	28.3	28.9	21.8	23.9	19.2	43.8	51.7	51.0	52.5
3rd	27.3	28.0	27.6	32.2	28.9	25.1	27.0	22.0	43.8	46.9	45.4	45.8
2nd	27.3	28.3	28.0	33.3	28.9	26.2	27.9	22.9	43.8	45.5	44.1	43.8
1st	27.3	28.4	28.1	33.7	28.9	26.6	28.2	23.2	43.8	45.0	43.7	43.1

(1) Rotation about vertical axes of walls permitted.

(2) No rotation about vertical axes.

to their tributary areas (no continuity assumed). The load distribution given under I=I was obtained from the analysis of the model in Fig. C.1, i.e. the finite stiffness of the floor slabs was considered. This analysis was carried out for two assumptions: 1. the floor slab was free to rotate about a vertical at the centerline of each wall. 2. the floor was assumed to be restrained against rotation at the walls due to the very large lateral stiffness of the longitudinal walls.

It is seen that the assumption of rigid floor diaphragms is not a significant improvement on the tributary area method which was used by the designers of the building.

In summary it is seen that the rigid floor slab assumption adopted in Chapter V is unlikely to lead to appreciable errors in the distribution of lateral loads among the three walls.

In view of the low sensitivity of the lateral load distribution among the walls to changes in relative stiffness, it is believed that if foundation compliance were considered in this analysis, it would not have affected the results to any significant extent.