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

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SUMMARY

During the Feb. 9, 1971 earthquake, Building 41 of the Veterans Administration Hospital in San Fernando was in the area of very strong shaking. It survived with only minor structural damage, although designed for a lateral force coefficient of 10 percent. The study attempts to reconcile these facts. Analyses of a fixed base 3-dimensional linear model of the structure and a 2-dimensional model incorporating partial uplift and soil yielding show that the key to the successful response of the building is to be found in the large strength built into the structure and the beneficial effects of soil - structure interaction.

INTRODUCTION

Structures that collapse or are heavily damaged in destructive earthquakes are analyzed by engineers to determine why they performed poorly, and how their design could have been improved. It is equally important to analyze buildings that survive exceptionally strong shaking, so as to explain why they were able to do so.

Two major structures in the Veterans Administration Hospital complex in San Fernando, California survived the February 9, 1971 earthquake with only minor damage. This in contrast to several other buildings in the same complex which collapsed, exacting a toll of 46 lives, and to the collapse of a modern reinforced concrete structure at the neighboring Olive View Hospital. The two hospitals were located near the major surface faulting, the V. A. buildings being 1 1/4 miles southwest of Pacoima Dam where peak accelerations over 1 g were recorded. The ground shaking at the V.A. Hospital is believed to have been less severe than at Pacoima Dam, yet peak accelerations of at least 0.5 g were estimated for nearby sites by a number of investigators [1].

The V.A. Hospital Buildings 41 and 43 provide what is probably the best example in the San Fernando earthquake of structures designed to resist nominal loads surviving intense shaking without severe damage. The response of these buildings and the minor damage suffered by North Hall at the University of Santa Barbara during the August 13, 1978 earthquake [2] suggest that excellent performance of low rise shear wall buildings through intense shaking is not uncommon, and that these structures should be studied in detail to document the sources of their reserve strength.

The principal objective of the study is to reconcile the observed behavior of one of the two V.A. structures, Building 41, with the level of

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shaking experienced during the San Fernando earthquake. The need for reconciliation stems from the following constraints: (1) the building was designed for lateral loads of approximately 10 percent of weight at nominal working stresses, (2) the structure received only minor structural damage, indicating no significant inelastic behavior in the main structural components, (3) the ground shaking was very strong, probably inducing effective lateral loads of one half the weight or greater. Because of these controlling conditions it is not easy to explain quantitatively the successful performance of the building by standard engineering analysis, and additional mechanisms beyond those normally considered in seismic analysis are required to reconcile the analytical evaluation of the response with observed behavior.

However, a detailed study of the response of Building 41 is fraught with difficulties, since all the V.A. Hospital buildings were razed some time after the earthauake. Thus no information on material properties is available beyond that which could be gleaned from the structural drawings and brief calculation sheets. Moreover, no record of ground motion at the site is available, and records at nearby sites have serious limitations because of marked differences in terrain or distance. Finally, the type of information on structural damage which is required for a detailed study is not available. In view of these limitations it was realized that a full explanation of the mechanism that enabled the survival of the building could not be given. Yet it is believed that some insight could be gained, and the parameters affecting the response of low rise shear wall concrete buildings to severe ground shaking could be identified.

DESCRIPTION OF BUILDING AND DESIGN CRITERIA

Building 41 was designed in 1937 by the V.A. engineering office and built in 1938. It was four stories high, about 200X50 ft. in plan with a centrally located penthouse. A photograph of the south elevation taken after the earthquake is shown in Fig. 1. The vertical and lateral load carrying system consisted mainly of pierced reinforced concrete shear walls and was supported on spread footing. There were six shear walls in the transverse direction, which was symmetric in plan, and three longitudinal walls. The walls were vertically and horizontally reinforced on both faces and had special edge reinforcement around openings. All bars, including deformed ones, were hooked. The internal walls had reinforced boundary elements facing the central corridor. Apart from the ground floor slab, which was cast on grade, the other floor and roof slabs consisted of R.C. ribbed slabs within the building and a solid slab in the front porch. The general layout of a typical floor is given in Fig. 2 and schematic wall elevations are shown in Fig. 3. A more detailed description of the building is given in Reference 3.

All the buildings on the site were razed after the earthquake, thus the properties of construction materials had to be inferred from the designers' blueprints and calculation sheets as well as from then applicable ASTM standards, the 1937 edition of the Uniform Building Code and from knowledge of practice in the mid 1930's. However, the effect of aging (33 years) on the strength and stiffness of concrete was taken into account. Since only minor cracks were detected after the earthquake in the lightly reinforced concrete walls, it was assumed that the tensile strength of concrete had not been 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. Regarding soil properties, whose variability is high, the basic stiffness parameters were computed on the basis of available data on the shear wave velocity measured near the site, whereas the bearing capacity was estimated from the known properties for deposits of similar nature.

GROUND MOTION

In the absence of an actual record in the vicinity of the site 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. As a simple indicator one may use the peak acceleration. However, the range of peak acceleration values measured or inferred at nearby sites is rather wide: 0.5 g to 1.25 g [1]. Moreover, the peak acceleration is not as important as the frequency content of the record in the range of structural frequencies. In this respect, the choice of available relevant ground motion records is very limited. In fact, none of the three records, which due to their proximity are likely candidates: Pacoima Dam, Holiday Inn and the lower Van Norman Dam seismoscope trace, is directly applicable to the V.A. Hospital site [3]. It follows that the spectral amplitudes can only be established within a broad range. On the basis of earlier studies it appears that 0.7 g to 1.5 g is a reasonable estimate for the spectral amplitudes in the period range of interest.

LINEAR ANALYSES

In view of the apparent great strength of the longitudinal walls the investigation focused on the transverse response only. The lateral load capacity of the building was first estimated on the basis of conventional code oriented procedures. The model consisted of somewhat simplified versions of the six transverse coupled shear walls. In view of the uncertainties regarding the effective width of the longitudinal walls acting as flanges to the cross-walls, two alternative flange widths were considered in the analysis. The walls were loaded as required by 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 displacement at roof level only (Fig. 3). The results indicated that with a fixed base, the structure could resist lateral loads implied by a lateral load coefficient of about 0.4 g; the limiting factor being the tensile strength of concrete in the walls.

A complete three dimensional model of the entire structure was then dynamically analyzed for a flat acceleration spectrum using the program ETABS [4]. With such an analysis there is less need to resort to arbitrary assumptions regarding lateral load distribution among the walls, and the effective width of flanges. It was also possible to consider, albeit approximately, the ability of the connecting beams to yield before the tensile capacity of the walls was exhausted. The general scheme of framing assumed for the analysis is shown in Fig. 2 . There were, however, some difficulties in applying ETABS to Building 41: for example, in ETABS, floor slabs are assumed rigid in their own plane, an assumption known not to be particularly suitable for narrow low-rise buildings with full width shear walls. Also, the shear walls, continuous stress transfer around corners to perpendicular walls, as well as shear lag effects cannot be efficiently modelled. Nevertheless, the results of several computational checks indicated that the resulting inaccuracies were not significant. From the computer analyses it was found that the dynamic properties of the structure, including the lateral force capacity, as determined by the code oriented procedure, were in very good agreement with those determined by the modal analysis.

However, the important result was that the inferred level of ground acceleration as indicated by a spectral acceleration of about 0.5 g, is still quite low compared with credible lower bound estimates. Thus, it seems likely that the structure had a capacity significantly in excess of that revealed by the analysis made so far. The nonlinear soil-structure interaction analysis described in the following section helps to resolve the discrepancy between observed and calculated capacities.

NON LINEAR SOIL-STRUCTURE INTERACTION

The need for a nonlinear analysis stems mainly from the fact that already at relatively low levels of excitation, partial uplift of the structure from the supporting soil is to be expected. The reduced contact area between the base and the soil leads to lowering the rocking and lateral rigidities of the foundation, and with increasing separation may lead to partial yielding of the soil. However, in order to study 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 was isolated from the structure, and its response to two accelerograms recorded near the site was investigated using the program DRAIN-2D [5]. A schematic representation of the model is shown in Fig. 4. A bilinear force displacement relationship was assumed for the soil. The possibility of uplift was modelled by taking advantage of the buckling capability of the program. However, for this purpose it was necessary to "hang" the model of the superstructure from the foundation soil as shown in the detail in Fig. 4. The corridor coupling beams were modelled as elasto-plastic elements. The limitations of using axial springs to model soil-structure interaction are well known. However, these were overcome to a large extent by considering a relatively wide range of important interaction parameters. The level of damping appropriate for a given structural system is always problematic, particularly when soil-structure interaction effects are believed to be important. Since only Rayleigh type damping can be modelled with DRAIN-2D, the damping coefficients were somewhat arbitrarily chosen so as to produce 5% critical damping in the first two modes of the rigidly founded structure. The amplitude decay was then computed for a base impulse on the interactive system. The resulting 7%-8% effective damping at lower levels of excitation seems reasonable. The two records chosen for the analysis were those obtained during the Feb. 9, 1971 San Fernando earthquake at Pacoima Dam (S16E) factored by 0.4, and at the Holiday Inn at 8244 Orion Blvd (1st floor, N00W), factored by 2.0. This scaling gave approximately the same fixed base shear as a constant acceleration spectrum with a level of 0.9 g. This level is compatible with the lower estimates of strength of ground motion at the site during the earthquake.

Table I summarizes the main findings. The complete results for the 23 cases analyzed are given in the full report [3]. It can clearly be seen that the soil-structure interaction effects lead to lower shear forces 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 shaking.

DISCUSSION AND CONCLUSIONS

Considering the approximate nature of the modelling and the uncertainties in the ground motion, it cannot be claimed that any of the analyses produced 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 nonlinear soil-structure interaction. This conclusion is reinforced by the unlikelihood of an alternative reconciliation based on a combination of higher assumed concrete strength and changes in structural modelling procedures. For example, assuming a 40% higher tensile concrete strength is equivalent to assuming a 100% higher compressive strength, i.e. 8000 psi, which is unrealistic. Also, it appears that a more realistic modelling of frame action of the slabs, internal columns and longitudinal walls (weak direction) could have contributed at most 5% to the total capacity. Larger participation implies substantial stiffness degradation in the coupled cross-wall system. This was not observed on inspection after the earthquake.

Some additional conclusions from the study are summarized below:
1. A low level of tensile stresses is necessary to ensure survival of nominally reinforced low-rise coupled shear wall structures through a severe earthquake. This can only be achieved through strong and stiff coupling beams.
2. Considering linear soil-structure interaction in the analysis does not necessarily lower the level of internal forces in the structure compared with the fixed base situation; i.e. one should not overlook the possibility that in some earthquakes the spectral accelerations may rise steeply with increasing period even though the damping is increased.
3. Partial uplift and soil yielding tend to reduce the seismic forces in the structure and should not necessarily be avoided by designers.
4. Regarding structural modelling, it has been shown that very simple models can lead to results which are in good agreement with more sophisticated analytical techniques capable of modelling the 3-dimensional nature of the system. It appears that the continuum approach to the analysis of coupled shear walls can be useful even in situations which are not ideally suited for its application. Also, in view of the difficulties in correctly modelling force transfer through corners and shear lag effects in perforated flanges by means of standard space frame programs, it is believed that a finite element program specifically designed to analyze 3-dimensional shear wall and frame structures should be developed for general use.

In summary, the excellent performance of Building 41 of the Veterans Administration Hospital through the 1971 San Fernando earthquake is one of many cases which show that well designed structures are able to resist intense ground shaking. This study indicates that successful performance depends on their possessing great strength, and on the beneficial effects of nonlinear soil-structure interaction.

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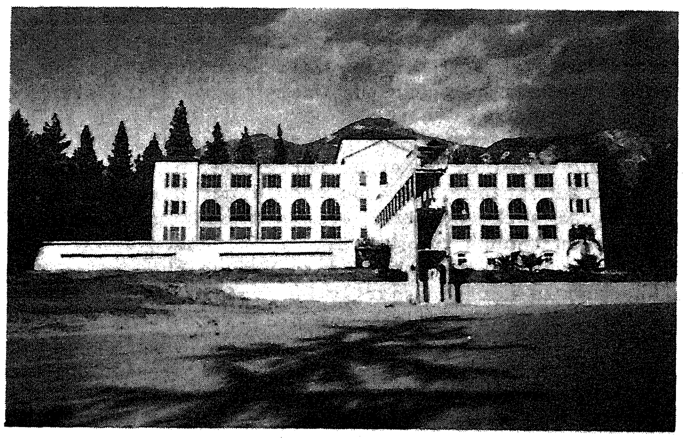

Figure 1: Veterans Administration Hospital Building 41, South Elevation with walkway to adjacent buildings.

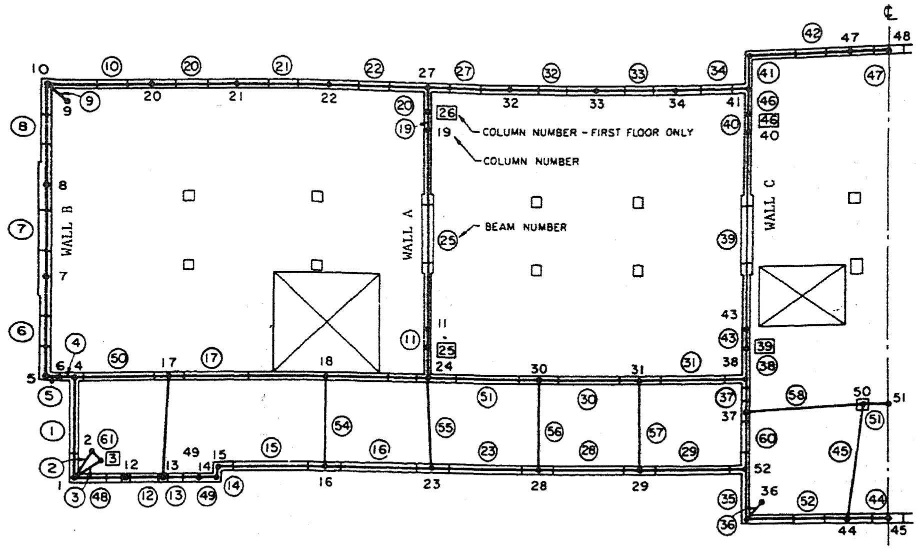

Figure 2: Plan of Assumed Framing System at Typical Floor Level

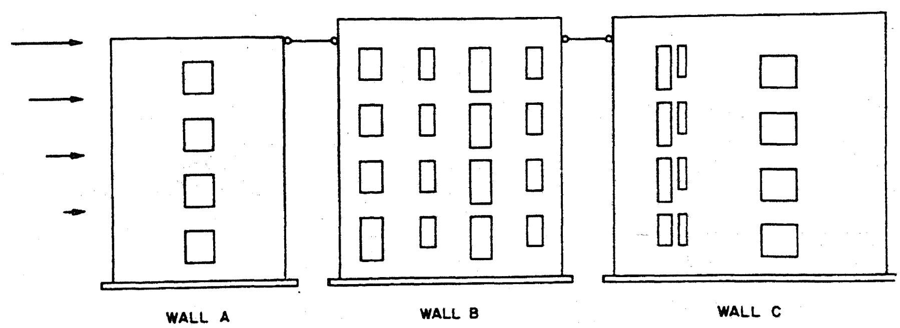

Figure 3: Simplified Coupled Shear Wall Model.

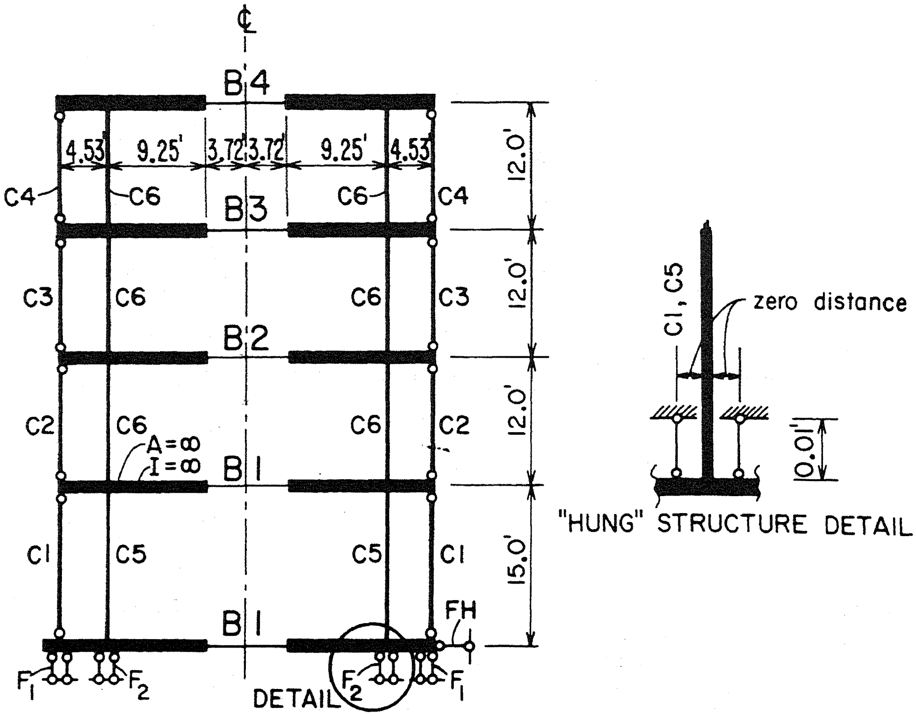

Figure 4: Schematic Representation of Nonlinear Model

Table I: Comparative Results: Fixed-base vs. Nonlinear Interactive Model.

		WALL SHEAR (kip)	WALL MOMENT (kip.ft)	WALL AXIAL COMPRESSN. (kip)	BASE SHEAR (kip)	LATERAL ROOF DISP. (inch)	UPLIFT (inch)
FIXED BASE	PD	571	6991	698	1142	0.23	----
	HI	559	6730	661	1118	0.20	----
INTER-ACTIVE	PD	367	3499	800	839	0.46	0.10
	HI	481	4754	901	1223	0.95	0.40

PD - Pacoima Dam; HI - Holiday Inn