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Seismic analysis of a 16 storey building in Armenia

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ABSTRACT: The seismic behavior of a sixteen story lift-slab apartment building which was severely damaged during the 1988 Armenia Earthquake is evaluated. Three dimensional finite element models of the structure are developed and used to predict the response to local code design forces, UBC lateral forces and time history analyses using aftershock recordings. Results indicate that failure was primarily due to shear in the central core and that local lateral force design requirements for this type of structure are too low.

INTRODUCTION

During the past twenty years, a large number of nine to sixteen story lift slab apartment buildings have been built in Soviet Armenia [1]. One of these buildings was located in the region that experienced heavy damage during the Armenia Earthquake of December 7, 1988 [2]. The building, located in the city of Leninakan, is a sixteen story lift slab which is identified as a "threefoil". This structure has a single core and a floor plan that is symmetrical about only one of the major axes as shown in Fig. 1. During the earthquake, this building sustained severe structural and nonstructural damage but was still standing after the event.

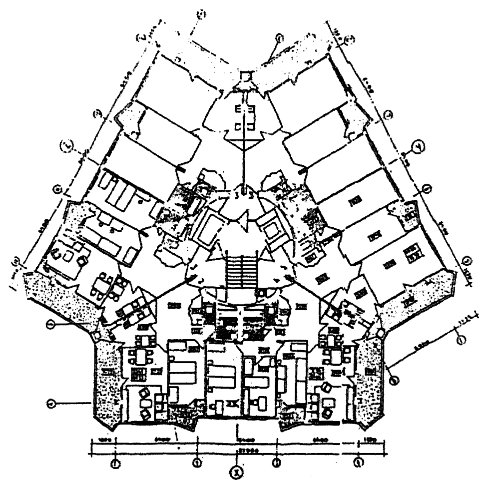

Fig. 1 Threefoil Floor Plan

The purpose of this study is to investigate the dynamic response of this structure and to correlate high stress regions indicated by the mathematical models with observed damage. Critical comparisons will also be made between the lateral force design requirements used in the design of this building and those that would be used in the United States for a similar type of structure.

STRUCTURAL SYSTEM

Lift slab structures of this type are generally referred to as core structures. In these systems, the gravity load is carried by the core and perimeter columns. Resistance to lateral loads is provided entirely by the central core of the building which contains the stairwells and elevators used for vertical transport. In addition to the economical benefits of lift slab construction, this type of system provides a relatively column free floor space which can easily be partitioned into apartments. The disadvantage of this type of system

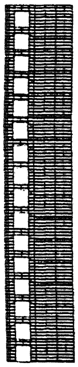

Fig. 2 SAP90 Core Model

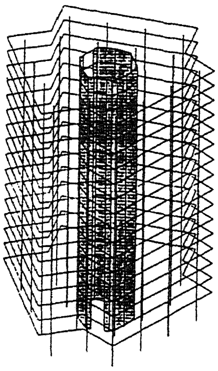

Fig. 3 ETABS Building Model

is the lack of any redundancy for resisting lateral loading.

The core structure of this building has an octagonal shape with an outside diameter of approximately 9 meters (29.5 feet). The wall thickness of the core varies between 350 mm (13.8 inches) and 600 mm (23.6 inches). At each story level, the core has three openings which allow movement into and out of the building. The typical story height is 2.96 m (9.71 feet) although at the base, the core extends downward 3.0 m (9.8 feet) to the top of the foundation. Lateral support for the floor slabs is provided by the core and vertical support is provided by the core and 21 columns which are peripheral to the core. The floor slab has a uniform thickness of 18 cm (7 inches). Precast panels are hung on the slabs to enclose the living space. Due to the low temperatures experienced in this region, these panels are relative thick at 23 cm (9 inches) in order to provide the necessary insulation. Story weights for a typical story were estimated by the authors to be 623 tons (1370 kips).

ANALYTICAL MODEL

Two linear elastic, finite element models were used in the initial investigations. In order to evaluate the dynamic characteristics of the core structure, a detailed finite element model was developed using the SAP90 [3] computer program as shown in Fig. 2. This model permitted an accurate modeling of the openings in the core wall and the core floor slab. It also permitted a detailed evaluation of the distribution of stresses in the core. The results from this model will also be used to calibrate less detailed models.

In order to study the design considerations and the effect of the perimeter columns, a linear-elastic model based on the ETABS Program [4] was also developed. In this model, the core structure is represented by a panel element which is based on an isoparametric finite element membrane formulation with incompatible modes. The variable thickness wall of the core is discretized using thicknesses of 600 mm (23.6 inches) and 475 mm (18.7 inches). Peripheral vertical load carrying columns are included and are either 450 mm (17.7 inches) square or 400 mm (15.7 inches) square. Concrete strength is assumed to be 26.0 MPa (3770 psi.). The ETABS program makes the assumption that the floor diaphragm is rigid in its own plane. In this manner the core and the peripheral columns are connected by the floor diaphragm. Because of this assumption, openings in the floor slab of the core are neglected and so are the connections of the slab to the core. An isometric view showing the complete structural system is given in Fig. 3. In this view some of the openings in the core can be seen. The segments of the core are connected by a stiff link beam across the top of the story level.

MODAL ANALYSES

In order to obtain an estimate of the dynamic properties of the structure, modal analyses were

TABLE 1. MODAL ANALYSES

Configuration /Program	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
Core without openings SAP90	.91	.91	.22	.22	.21	.19
Core without openings ETABS	.99	.99	.20	.20	.09	.09
Core with openings SAP90	1.25	1.25	.40	.28	.28	.16
Core with openings ETABS	1.25	1.19	.66	.28	.26	.20
Complete building ETABS	1.21	1.16	.65	.28	.26	.21

performed considering four configurations. The first considered the core as a uniform, hollow tube with no openings and an average wall thickness of 533 mm (21 in). The second considered the core with openings and the third considered the complete system including the perimeter columns. Results of these analyses for the first six modes of vibration are given in Table 1. Here it can be seen that there is little difference in the dynamic properties of the core and the complete building indicating that the perimeter columns do not contribute to the lateral resistance. Computed mode shapes are shown graphically in Figs. 4 thru 7 in plan view. The first mode, shown in Fig. 4 indicates a translational mode in the Y-Y direction. Since this axis is an axis of symmetry, there is no torsional component in this mode shape. Modes two and three, shown in Fig. 5 and 6 respectively, both have a strong torsional component. Mode 4, shown in Fig. 7, is almost a pure torsional mode with the center of rotation near the core.

SEISMIC GROUND MOTIONS

No recordings of strong ground were obtained at Leninakan during the main earthquake. Estimates of peak ground acceleration in Leninakan were on the order of 40% of gravity. The reconnaissance team from the United States brought 24 portable seismographs and associated computer equipment with them and deployed these instruments to record aftershock data. The largest aftershock recorded in Leninakan was from a magnitude 4.7 event that occurred on December 31. The peak acceleration recorded at Leninakan during this event was approximately 3.1% of gravity.

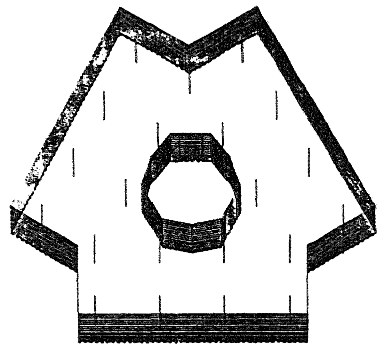

Fig. 4 First Mode Shape

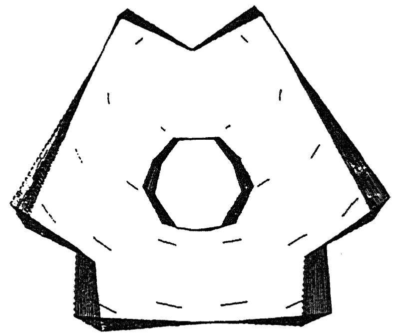

Fig. 5 Second Mode Shape

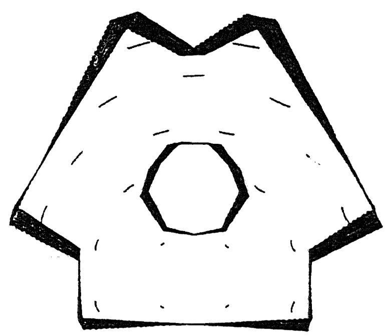

Fig. 6 Third Mode Shape

Although the recorded accelerations from the aftershock are relatively

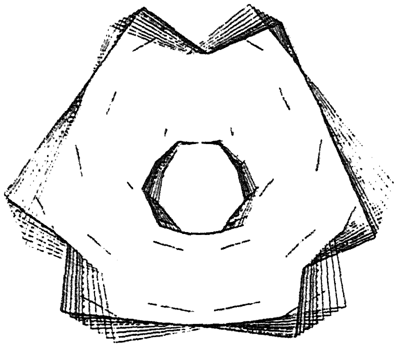

Fig. 7 Fourth Mode Shape

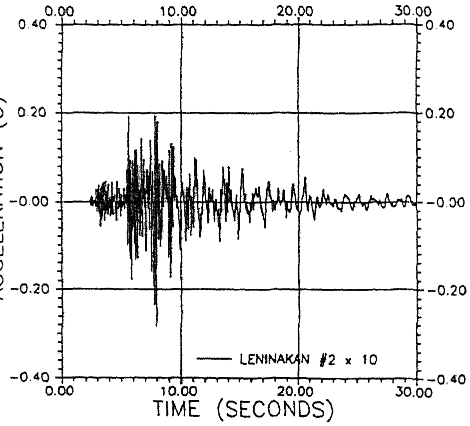

Fig. 8 Aftershock Acceleration Recorded at Leninakan (x10)

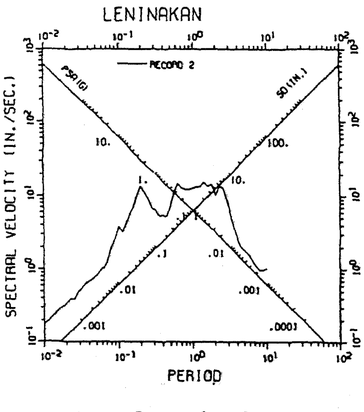

Fig. 9 Aftershock Spectrum

small compared to the main shock, this data is the best available having the characteristics of the ground motion experienced by the lift slab buildings. Therefore, the aftershock data was scaled by a factor of 10 for use in the dynamic response analyses. The time history for one of the horizontal components, identified as Leninakan2, is shown in Fig. 8. To obtain this record, the aftershock data has been multiplied by ten, resulting in a maximum acceleration of 0.3g. The corresponding linear elastic response spectrum for 5% of critical damping is shown in Fig. 9.

EARTHQUAKE RESPONSE ANALYSIS

Static and dynamic response analyses were conducted using the ETABS model of the structure described previously. Static loads were those specified in the 1988 UBC for a structure located in zone 3 with the response modification factor, R_w, for this structure taken as 6. The design, investigation and implementation of lift slab buildings is done by the All-Union Experimental Design and Technology Institute (PEKTI) located in Yerevan. A static lateral force analysis was also done for the lateral loads used by this group and identified as PEKTI. Time history analyses were done using 30 seconds of the aftershock data identified as Leninakan2. The envelopes of maximum lateral displacement for these three load conditions are shown in Fig. 10. Here it can be seen that the PEKTI loading results in displacements which are about 55% of those obtained for UBC Zone 3. It can also be seen that the displacements from the time history analysis are more that three times larger than those of PEKTI. The envelopes of base shear are shown in Fig. 11. In this case, the maximum shears obtained from the PEKTI loading are about 950 kips compared to 2750 kips for the time history analysis. Preliminary analysis of the shear capacity at the base of the threefoil indicates a value of approximately 1100 kips which compares well with the lateral force requirements of the PEKTI loading but is much less than the seismic demand.

DISCUSSION AND CONCLUSIONS

The shear reinforcement at the base of the three foil consists of 8mm (0.32 inch) diameter bars spaced at 300mm (11.8 in.). This amount of

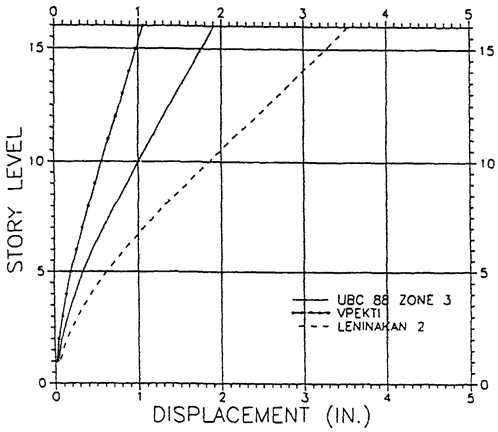

Fig. 10 Max. Story Displacement

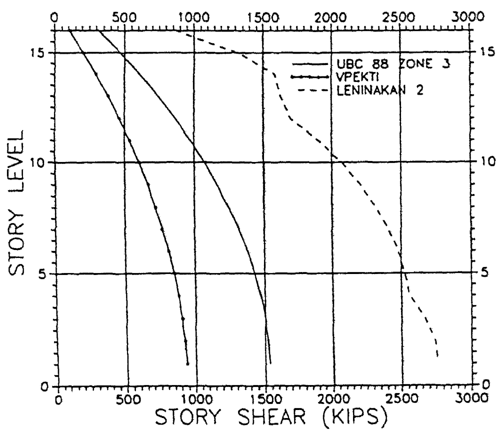

Fig. 11 Max. Story Shear

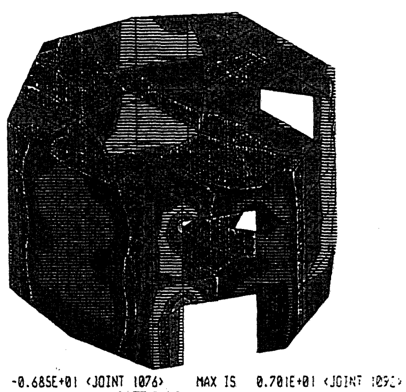

Fig. 12 Shear Stress Contours

horizontal reinforcement contributes little shear resistance although it was adequate based on the PEKTI design criteria. The lateral force criteria used by PEKTI incorporates a coefficient to account for damage to the building. In the PEKTI calculations, this value was taken to be 0.25 which implies that residual deformations, fractures, and individual component damage are permitted. In view of the basic characteristics of a core structure (no redundancy) and the apparent mode of failure (shear), this reduction is too large. Members failing in shear exhibit very limited ductility.

Reports of the reconnaissance team which visited the building following the earthquake indicate there was a crushing of the entire core at the first floor level. Furthermore, the crushing was noted to be most extensive at the front entrance to the building. The distribution of the shear stresses in the areas of the openings in the core are shown in Fig. 12 for the first two story levels. Here it can be seen that there is a definite concentration of shear stress adjacent to an opening. It was also reported that diagonal cracks had formed in the core wall along the entire height of the building and that there was a strong torsional movement of the core. This reported behavior agrees well with that obtained from the dynamic response analysis of the three dimensional models.

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