Seismic evaluation of lightweight RC flat plate building in region of low to moderate seismicity

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ABSTRACT: An existing lightweight 21-story reinforced concrete building is evaluated for its damage potential under a moderate seismic event. The building is located in eastern United States and consists of non-ductile columns, moderately reinforced shear walls, and a flat slab with spandrel beams in the perimeter. The building was designed and constructed by the end of the 1950s without any provision for seismic loading. The evaluation was carried out in three stages: a preliminary screening using the recommendations of ATC-14 (1987); a static lateral load analysis; and a nonlinear time history analysis using IDARC (Kunnath et. al., 1992). To facilitate a reasonable inelastic dynamic analysis, it was necessary to model the flat slab behavior as accurately as possible. The results of this evaluation will also be useful in regions of high seismicity where older flat-plate buildings still exist.

1 INTRODUCTION

Lightweight reinforced concrete structures with slender frames and flat slab construction are common in regions of low to moderate seismic risk. Numerous such buildings are built regularly in the eastern and central United States, while a number of them, constructed prior to 1971, still exist in the west coast of U.S., a region that experiences severe seismic activity. During the past few years, awareness of the seismic vulnerability of existing structures in eastern United States has grown.

The seismic vulnerability of a twenty one story building (73.6m), located in an area now classified as low to moderate seismicity, was studied. The typical plan of the office building is shown in Figure 1. The structural system, common in the eastern and central United States, consists of a flat slab, with spandrel beams at the perimeter. Four walls can be identified in the North-South direction, located in an eccentric fashion. The building was designed in the late 1950's with no consideration for earthquake loading, although a design for wind forces was probably made. This fact is reflected in a generalized lack of adequate ductile detailing. The main observations detected from the building drawings are as follows: the positive rebar of the slab at the column joints is not continuous; stirrup spacing in the columns is larger than current ACI-318 (1990) code limits for fully seismic detail; shear walls are connected without link beams; and a reduction in strength of some perimeter columns from the second to the first level.

The seismicity of the area has not been well defined yet, mainly because strong earthquakes are infrequent in the area. As noted by Whitman (1987), the ratio of peak ground acceleration for the maximum expected earthquake and the maximum credible earthquake is about 4/5 in California, while this ratio could be about 1/6 for places like Boston, where the code considers a PGA of 0.05g for an earthquake with 50% probability of occurrence in 100 years, but a PGA of 0.3g must be
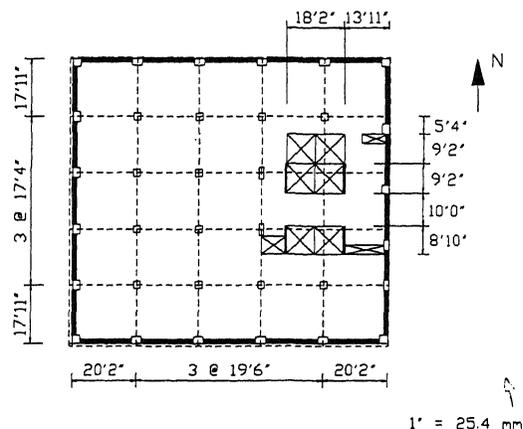

Figure 1. Plan of typical story.

used for nuclear structures. This indicates that the possibility of the maximum credible event being observed during the life of the structure is higher in western U.S., however, the expected level of damage in the constructions of eastern U.S. will be considerably high if the maximum credible earthquake does occur. Figure 2 presents the seismicity of the northeastern United States from 1970 to 1982, taken from Nabelek and Suarez (1987).

2 PRELIMINARY SCREENING

From Figure 1, three types of frames can be identified: the exterior frames, the interior frames, and the interior frames with concrete walls. The building has a flat slab thickness of 178 mm typically, without drop panels. The
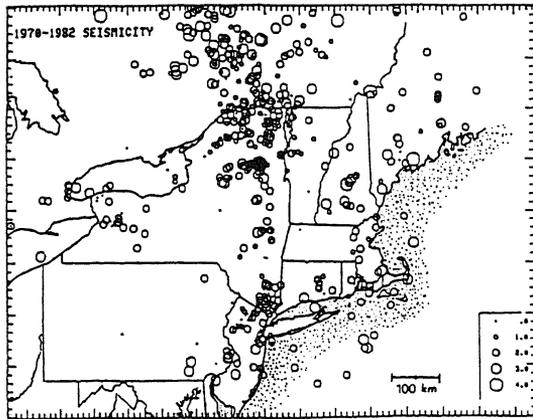
Figure 2. Seismicity of Northeast U.S. from 1970 to 1982 (Nabelek and Suarez, 1987).

Figure 4. Typical beam-column joint.
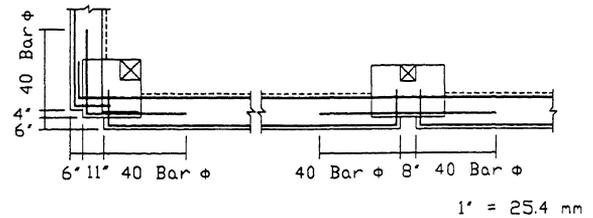
Figure 5. Eccentricity of spandrel beam and column connection in plan.

negative reinforcing bars of the flat plate are alternated near the column strips (see Figure 3), while the rebar for positive moment is discontinuous. The beams at a typical level are located only at the perimeter of the structure, connected eccentrically with the columns: typically 152 mm out of the 456 mm of the width of the beams do not connect to columns. Figure 4 shows the typical beam-column joint detailing in elevation.

The percentage of steel in the columns of the structure varies from 0.24% in the upper stories, to 4% in the lower ones. In general, the columns have less than 1% rebar in their upper two third stories. The longitudinal rebar is not continuous at the foundation. Some of the columns in the structure have a stirrup separation greater than half its minor dimension. One puzzling finding from the examination is that the number of bars for some columns
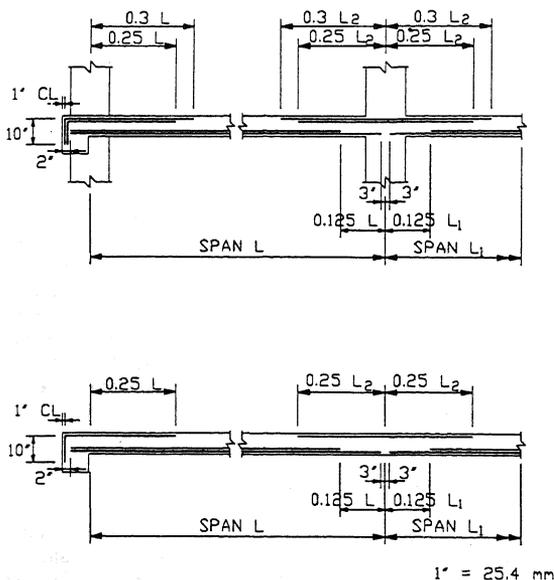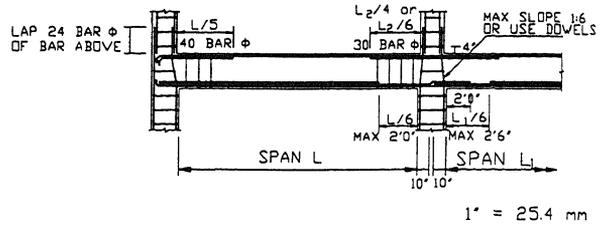
in the perimeter at the ground level decreases, without any change in the bar diameter of column sizes. The reduction is for some columns as much as 30% Beams are provided only along the perimeter of the structure. These spandrel beams, 460 x 660 mm, are eccentric to the column locations, as shown in Figure 5. Additional bars are used to make the reinforcement continuous. Unlike the flat slabs, both the top and bottom beam reinforcements are continuous across the joint region.

2.1 ATC-14 Evaluation

The Applied Technology Council publication ATC-14 (1987), *Evaluating the Seismic Resistance of Existing Buildings* was used to perform the preliminary investigation. A site inspection was carried out to verify building drawings, assess any signs of material deterioration, and complete the ATC-14 checklist. With this preliminary evaluation, the aspects that need additional and more detailed analysis were identified.

An approximate formula to calculate the lateral story drift ratio is given in ATC-14. The formula was derived considering that the beams have a point of contraflexure at midspan, and the columns at midheight. But the formula does not yield accurate results in structures with shear walls or flat plates, because the bending moment diagram in the columns, or walls, often differ significantly from the assumption of zero moment at midheight.

A generalized shortcoming in the approximate ATC methods for seismic evaluation of existing buildings is that no special consideration is given for structures with flat slabs. This defect might be due to the fact that its lateral behavior is not as clearly understood as under gravity loads only. Moehle (1989), who has investigated flat slab behavior under lateral loads, suggests that from one fourth to one sixth of the slab width should be considered as effective when modeling flat slabs with

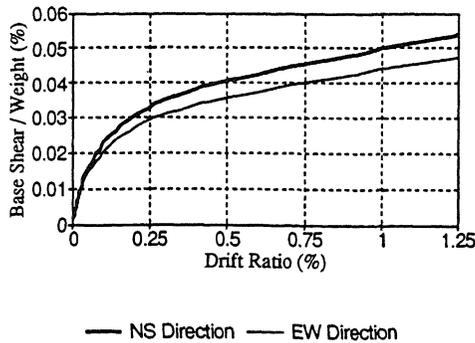Figure 3. Detail of flat slab reinforcement.

Figure 6. Overall drift versus base shear from monotonic analysis.

the equivalent width approach. Moehle notes that flat slab structures have relatively large drifts, which implies that if the design is governed by a drift limit, limited ductility (inelastic behavior) can be attained, and therefore only a small reduction factor can be used.

Given this drawback of the ATC procedure, it was not possible to make adequate conclusions about the seismic vulnerability of the building. Hence it decided to carry out more detailed elastic and inelastic dynamic analyses of the superstructure to ascertain more clearly the effects of the detailing and structural configuration.

3 STATIC MONOTONIC ANALYSIS

To model the flat slab the effective width concept was used, considering 20% of the parallel span as the effective portion of the slab. For comparison purposes, the wind forces in the structure were calculated using the ANSI (1982) provisions. The wind base shears are 2.2% and 1.9% of the total structure weight in the North-South and East-West directions, respectively.

The program IDARC (Kunnath et al., 1992) was used to model the structure. The fundamental periods of the structure were determined from analysis to be 2.03 seconds in the North-South direction, and 2.18 seconds in the East-West direction.

From a static monotonic increment of lateral load applied in inverted triangular fashion, the overall drift ratio is plotted as a function of the base shear of the structure (see Figure 6). Note that the response due to wind is reasonably within the elastic portion of the curve. According to the ATC-14 suggested spectra, for a period of approximately two seconds, the elastic base shear is 5% of the weight of the structure. Using a reduction factor (R_w) of four, the base shear acting on the structure would be 1.25% of the building weight, that is below the wind response.

The lateral deformation of the exterior frames is of the shear type, due to the presence of the perimeter spandrel beams, on the other hand, the deformation of the interior frame with or without walls is analogous to a bending type deformation, due to the limited restriction the flat slab poses to the rotation of the columns. Due to this difference in deformation characteristic, the exterior frames resist most of the lateral load in the upper levels,
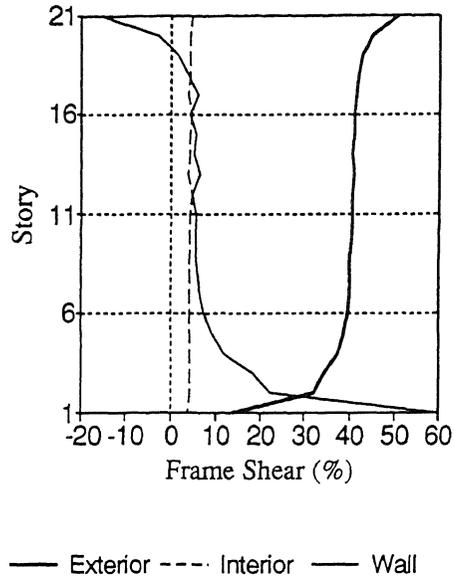

Figure 7. Shear distribution per frame in the North-South direction.

and less in the lower stories. Figure 7 presents the shear distribution in the frames for the North-South direction. *Therefore, a reduction of the story shear carried by the exterior frames was observed from the second to the first floor*, which explains the reason for the reduction in column strength from the 2nd to 1st floor levels.

4 NONLINEAR DYNAMIC ANALYSIS

The program IDARC (Kunnath et al., 1992) was also used in this stage of evaluation. To determine the hysteretic parameters: strength degradation, stiffness degradation, slip load, and slip strain, the experimental results obtained by Robertson and Durrani (1990) in testing of flat slab subassemblies were numerically simulated. According to Pan and Moehle (1988), who tested a number of flat-slab assemblies, a maximum displacement ductility of 2 may be achieved based on the ratio of gravity shear to punching shear capacity.

4.1 Choice of ground motion

Due to the lack of strong motion records at the vicinity of the construction, the following records were used for the analysis:
- the 1985 Nahanni earthquake;
- a convolution of the Nahanni earthquake for a soft soil profile in eastern United States;
- the Taft earthquake; and
- a modified version of the Taft earthquake to fit the recommended ATC-14 spectra.

The Nahanni earthquakes consisted of two strong motions recorded in 1985: the first of magnitude 6.6 in October, and the second of magnitude 6.9 in December.
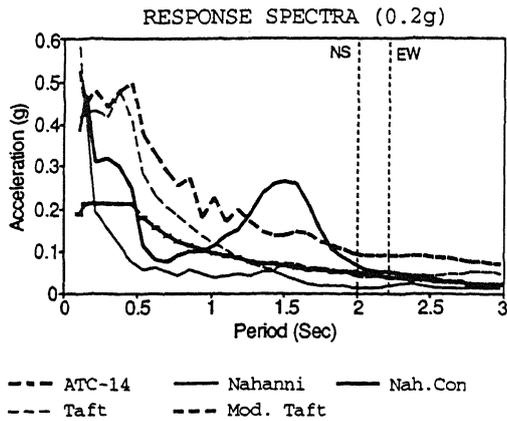
RESPONSE SPECTRA (0.2g)

Figure 8. Response spectra of input ground motions.

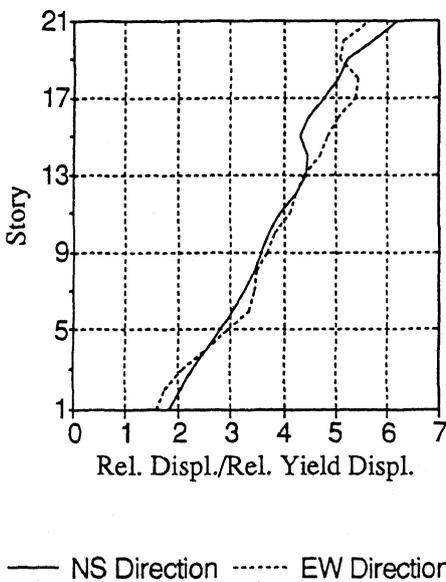
—— NS Direction ······· EW Direction

Figure 9. Maximum story ductility demand during the modified Taft earthquake.

The motions were recorded within 8 to 12 Km of the energy release center. The rupture was detected mainly in the upper 9 Km, but penetrated up to 12 Km. This shallow focal depth is congruent with the observed median of focal depths in northeastern U.S. (11 Km). In addition, the Nahanni earthquakes were observed in an area where a magnitude 5 was the maximum historical record before 1985. Due to the similarities in the seismotectonics of the Nahanni region with the area where the structure is located, and considering that a magnitude 5.1 record has already been observed in the area, the Nahanni earthquake seems a good representative earthquake to be used for the analysis.

Figure 8 shows the response spectra of the input motions. Because of the uncertainties in the actual seismicity of the zone, a peak ground acceleration of 0.2

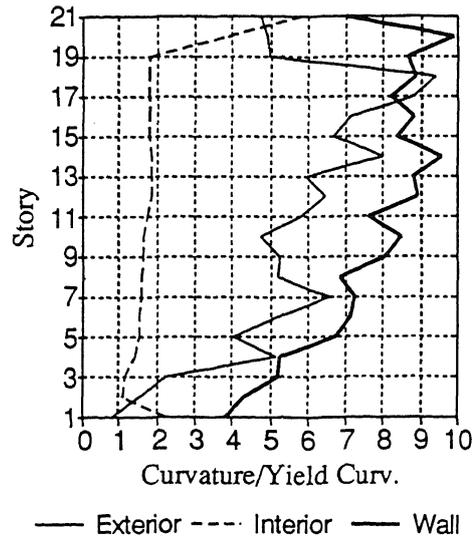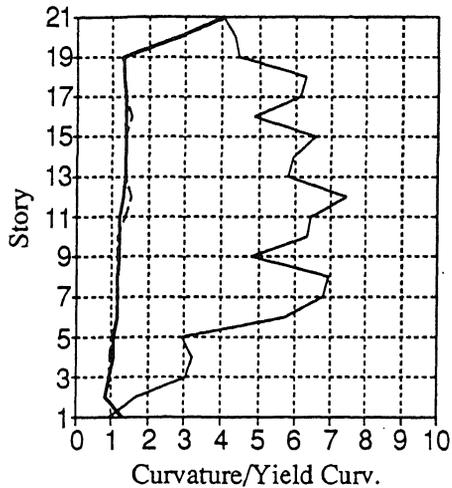
—— Exterior - - - Interior —— Wall

Figure 10. Beam/slab curvature ductility demand in the North-South direction, modified Taft earthquake.

g was used instead of the 0.1 g that the ATC-14 suggests. This choice of PGA, and assuming 0.3 g as the maximum credible earthquake as used by the nuclear industry, leads to a ratio of PGA for maximum expected to maximum credible of 2/3. For comparison purposes, additional runs were made with the modified Taft earthquake with a PGA of 0.1 g. Only a set of representative results are presented to highlight the performance of typical flat-plate structures.

4.2 Summary of Analytical Observations

As observed during the static lateral analysis, there is a reduction of the shear force carried by the exterior frames from the second to the first story. The reduction was typically in the order of 30%, that is the same percentage of reduction observed in the column rebar. All columns and walls remained elastic during the time history analysis, while some beams and flat slab portions yielded considerably. As expected, the preferable beam sway mechanism is observed. Figure 9 presents the maximum displacement ductility demand of the structure at each floor. Note that a value greater than two is observed almost throughout the structure, indicating the possibility of punching shear failures in the flat slab.

Figures 10 and 11 show the beam or slab curvature ductility demands for each of the principal directions of the building. The figures show the negative effect the presence of the walls produce in the local responses. The high curvature ductility demands in the wall frame of the North-South direction may initiate punching shear failures in the areas close to the concrete walls. It should be mentioned that smaller demands are observed for the level of excitation suggested in the ATC-14, although the wall frame in the North-South direction has a curvature ductility demand of four, indicating that punching shear might still be observed to a minor degree.

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Figure 11. Beam/slab curvature ductility demand in the East-West direction, modified Taft earthquake.
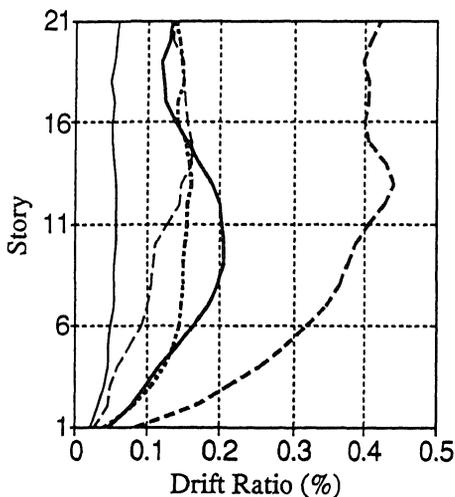
Figure 12. Drift ratios for building in NS direction for all earthquakes considered. (Notation: NahC - Nahanni convolution)

The expected drift ratios for the building in the NS direction is shown in Figure 12 for all earthquakes considered. Finally, the displacement history of the top story is presented in Figure 13 for three cases. The important observation here is that even for an ATC design event, the peak displacements exceed the existing gap between adjacent buildings.
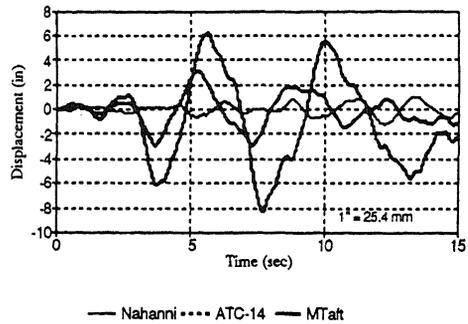

Figure 13. Top story displacement history in NS direction.

5 SUMMARY OF EVALUATION

The first part of the evaluation was performed using ATC guidelines. The methodology tries to cover all possible types of structures commonly encountered in practice. However, no special provisions are given for structures with flat slabs. Moreover, the simplified procedures suggested are not applicable for high rise structures and for structures with shear-walls. Therefore, this approach only helped to identify the areas of concern that needed further study.

Research to define more accurately the seismicity of the area is still underway. Uncertainties are present on the maximum magnitude of earthquake that might be observed within the lifetime of the structure. Since the recommended spectra in the ATC-14 is based on old seismicity observations, and is dependent on the frequency of events, an earthquake twice the suggested PGA was considered as basis for the analysis.

The results show an overall response of the structure within the code limits, although some local failures will be observed in the slabs for the strong earthquakes considered. Punching shear failures may be observed particularly in the upper floors, and in the area close to the elevator walls. The local punching failure will cause an increment in the shear forces at adjacent slab column joints that could lead to progressive failure of a story level.

Based on past observations, the probability of a strong earthquake, with a peak ground acceleration of 0.2 g, in the vicinity of the structure is small. If, however, an earthquake of such magnitude did occur, because of the expected spectral frequency content, it might cause major damage to mid-height structures in the area.

This building, as are typical medium to high-rise structures in eastern U.S., is separated from adjacent buildings by approximately 12-20 mm on two adjoining sides. So while the problem of punching shear can be addressed in reasonable ways, the larger issue of pounding will still remain to be resolved.

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