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Shake table test of a one-eighth scale three-story reinforced concrete frame building designed primarily for gravity loads

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ABSTRACT

Results of a 1/8 scale, 3-story, three-bay by one-bay lightly reinforced concrete (LRC) bare frame building tested on the Cornell University shake table are presented. The reinforcement details of the model were based on typical reinforced concrete frame structures constructed in the Central and Eastern United States since the early 1900's, in which the design was based primarily on gravity loads without regard to significant lateral forces. Special attention was paid to the duplication of the characteristic reinforcement details of this kind of building, especially at critical sections, such as joint regions and splices.

During the seismic tests, the model building showed a high degree of flexibility associated with a significant P-Δ effect and considerable stiffness degradation. The inadequate non-seismic reinforcement details did not form a serious problem to the model as most of the damage occurred in the columns, outside the joint regions. The building finally collapsed in a soft-story mechanism that took place in the first story columns. The seismic response of the model was compared with the predicted response obtained using program IDARC (Park et al 1987). The comparison indicated a very good agreement between predicted and measured global responses (top story displacement and base shear). Predicted specific member responses did not correlate well with the measured response.

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INTRODUCTION

The present investigation is part of a comprehensive research effort currently underway at Cornell University on the damage assessment and performance evaluation of LRC buildings subjected to seismic loads. The experimental work includes both full-scale and small-scale component tests in addition to small-scale complete building tests. In an early stage of this project, better microconcrete and model reinforcement were developed to enhance the simulation of reinforced concrete prototype specimen responses (hysteretic response, cracking patterns, and ultimate strength) (Kim et al 1988). The newly developed materials were used in a 1/6 scale 2-story office building model tested on the Cornell University shake table (El-Attar et al 1991 b). Test results of this model indicated the high flexibility and stiffness degradation associated with the discontinuous positive moment beam reinforcement pullout. The same model materials (with minor modifications) were used for the 3-story model. This model represented a more general case than the 2-story model, in that both exterior and interior joints were included in addition to the greater number of modes to be activated.

The main thrust of the experiment reported here is (a) to introduce some behavioral aspects of LRC structures during earthquakes, with special emphasis on the role of the non-seismic reinforcement details, and (b) to assess the reliability of one of the existing numerical modeling techniques (program IDARC) in predicting the response of this type of building.

DESCRIPTION OF THE TEST STRUCTURE

The test structure was a 1/8 scale true replica model of the prototype 3-story office building shown in Fig. 1. The model story height was 18", with a main frame span of 27". Members sizes were: column section 1.5" × 1.5", beam section 1.125" × 2.25", and a 0.75" thick slab. Reinforcement details of the model building are shown in Fig. 2; note the discontinuous positive beam reinforcement at the columns, the lack of confinement steel in the joints, and the column splice location.

The total weight of the structure was increased by adding lead blocks to meet the dead load similitude conditions. Special attention was paid to mounting the blocks to minimize the stiffening of the floor slab. In addition to the acceleration and displacement measurements at each floor level, the model structure was instrumented with internal force transducers at the mid-height of the first and the second story columns. Details of the similitude requirements, model design and fabrication, model materials, loading technique, and model instrumentation are provided in El-Attar et al (1991 a).

MODEL MATERIALS

The model microconcrete had mix proportions by weight of 0.95 : 1 : 3.6 : 2.4 (water : cement : model sand : model aggregate), where model sand was defined as particles passing ♯8 sieve and retained on ♯200 sieve, and model aggregates were defined as particles passing ♯6 sieve and retained on ♯8 sieve. At the time of model testing, the microconcrete had an axial compressive strength (f'_c) of 3.80 ksi and a splitting tensile strength (f'_t) of 0.34 ksi.

Threaded steel rods were used as longitudinal reinforcement in both beams and columns. The used sizes ranged from 0.099" dia. (3-48) to 0.164" dia. (8-32). All bars were heat treated to achieve the desired yield stress of about 40 ksi and to develop an adequate yield plateau. Shear reinforcement was provided by 0.05" dia. annealed steel wires.

TEST PROCEDURE

The model structure was subjected to four seismic tests using the time scaled Taft 1952 S69E earthquake at amplitudes of 0.05g, 0.18g, 0.35g, and 0.80g. Each seismic test was preceded and followed by a static test and a free-vibration test to determine the change in the structure properties (such as the fundamental period, damping ratio, etc.).

DISCUSSION OF TEST RESULTS

Global response

Story displacements and story shears recorded during run 0.35g are shown in Fig. 3 (a) and (b), respectively. It can be seen from both figures that all three stories were moving in phase, indicating the domination of the first mode of vibration. A brief summary of the seismic tests is provided in Table 1, where it can be seen that during run 0.18g, the model showed a large degree of flexibility associated with a high stiffness degradation (18% reduction of the fundamental frequency $\equiv [\sqrt{\frac{2.2}{1.8}} - 1] \times 100 \equiv 50\%$ stiffness reduction). The maximum base shear during this run (1.252 kips) represented 8.8% of the total load on the structure; the base shear was only 15% less than the model capacity. The damping ratio ζ expressed as a percentage of the critical damping ζ_{cr} (assuming a viscous damping model), increased significantly during this run due to the development of new cracks.

After the 0.35g run, the model fundamental frequency decreased to 1.65 Hz (25% reduction), indicating a 78% stiffness reduction. No significant change was detected in the damping ratio, indicating that few new cracks were developed during this run. The maximum base shear recorded during this run (1.384 kips) represented 97% of the model capacity.

Story shears and mode shapes recorded at the moment of maximum base shear are shown in Table 2. It can be seen that, despite the high non-linearity of the model, the mode shape remained essentially unchanged during all seismic tests. Also, except for the first low amplitude seismic test, the shear force distribution over the three stories remained the same for all subsequent runs. The model collapsed during run 0.80g in a soft story mechanism in the first story columns. Failure was initiated at one of the interior columns, followed by failure of the rest of the first story columns.

Local response

Cracks were detected at the top and bottom sections of the first and second story columns (especially at the structural hinge regions) after the 0.18g run. These cracks were localized at these areas and did not spread over the columns length even after the 0.35g run. No visual damage was detected in the beams, joint regions, or the splice areas, indicating that the non-seismic reinforcement details did not form a potential source of damage to this particular building.

The large degree of flexibility of the model resulted in a pronounced P-Δ effect. During the 0.18g run, the base shear obtained from the column load cell readings was 27% larger than that obtained from the story acceleration; the same effect was measured in all subsequent runs. It was also noticed that each column share of the total story shear was heavily dependent on its axial force. All columns showed a much larger flexural strength than that obtained using conventional flexural capacity calculations. The increase in strength was attributed to several factors such as the strain hardening of the model reinforcement and the strain rate effects.

Comparison with the analytically predicted response

The top story displacements and the base shears computed using program IDARC (Park et al 1987) for run 0.35g are plotted against the experimentally measured responses in Fig. 4 (a) and (b), respectively. It can be seen from both figures that the anaytical results were in good agreement with the experimental results. At the local level, column shears obtained using IDARC were less than the measured shears because the large P-Δ effect was ignored in the analysis. Individual column shares of the total story shears were also inaccurately predicted due to neglecting the effect of the change in the columns axial force on their yield moment.

SUMMARY AND CONCLUSIONS

A 1/8 scale 3-story, three-bay by one-bay bare frame LRC office building was tested on the Cornell University shake table under increasing versions of the Taft 1952 S69E earthquake. The building was designed to resist primarily gravity loads. The following conclusions may be drawn:

1. Lightly reinforced concrete buildings may be subjected to very large deformations associated with a significant reduction in stiffness during a moderate earthquake.
2. Although the non-seismic reinforcement details can form a potential source of damage to LRC buildings, they are probably not sufficient to develop a failure mechanism.
3. Due to their high flexibility, P-Δ effect is significant in LRC structures and should be considered in the analysis.
4. Low and medium rise LRC structures may be subjected to potential collapse in a soft story mechanism due to the higher flexural strength of the beams with respect to the columns.

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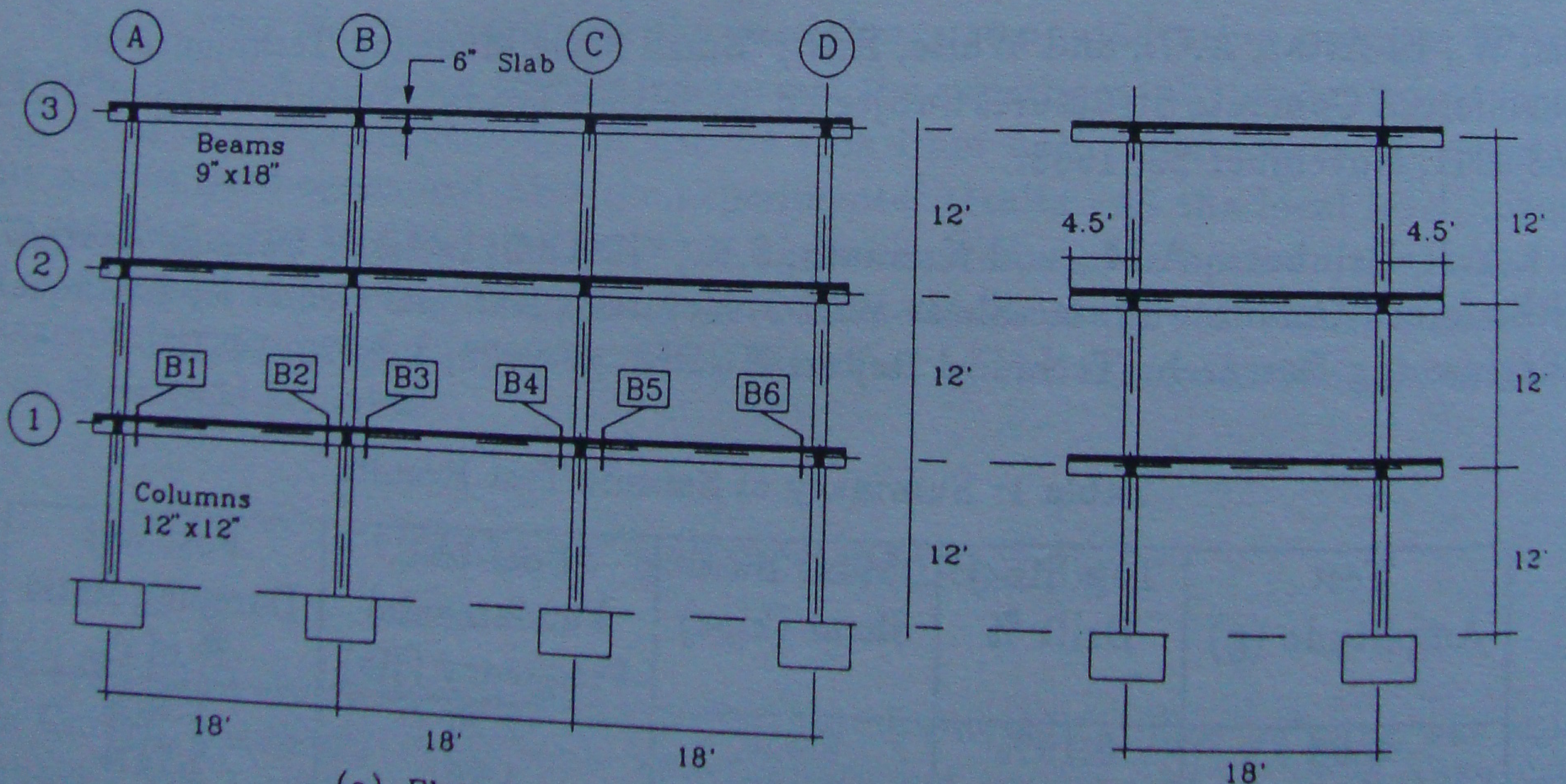
Table 1: Summary of Seismic Test Results

Test Amplitude (g)	Top Story Drift %	Max. Base Shear (kips)	Post-test Fundamental Frequency (Hz)	Post-test Damping Ratio % of ζ_{cr}
0.05	0.19%	0.338	2.20	1.30%
0.18	2.02%	1.252	1.80	2.74%
0.35	2.84%	1.384	1.65	2.76%
0.80	—	1.430	—	—

Table 2: Mode Shapes and Shear Distribution at The Maximum Base Shear

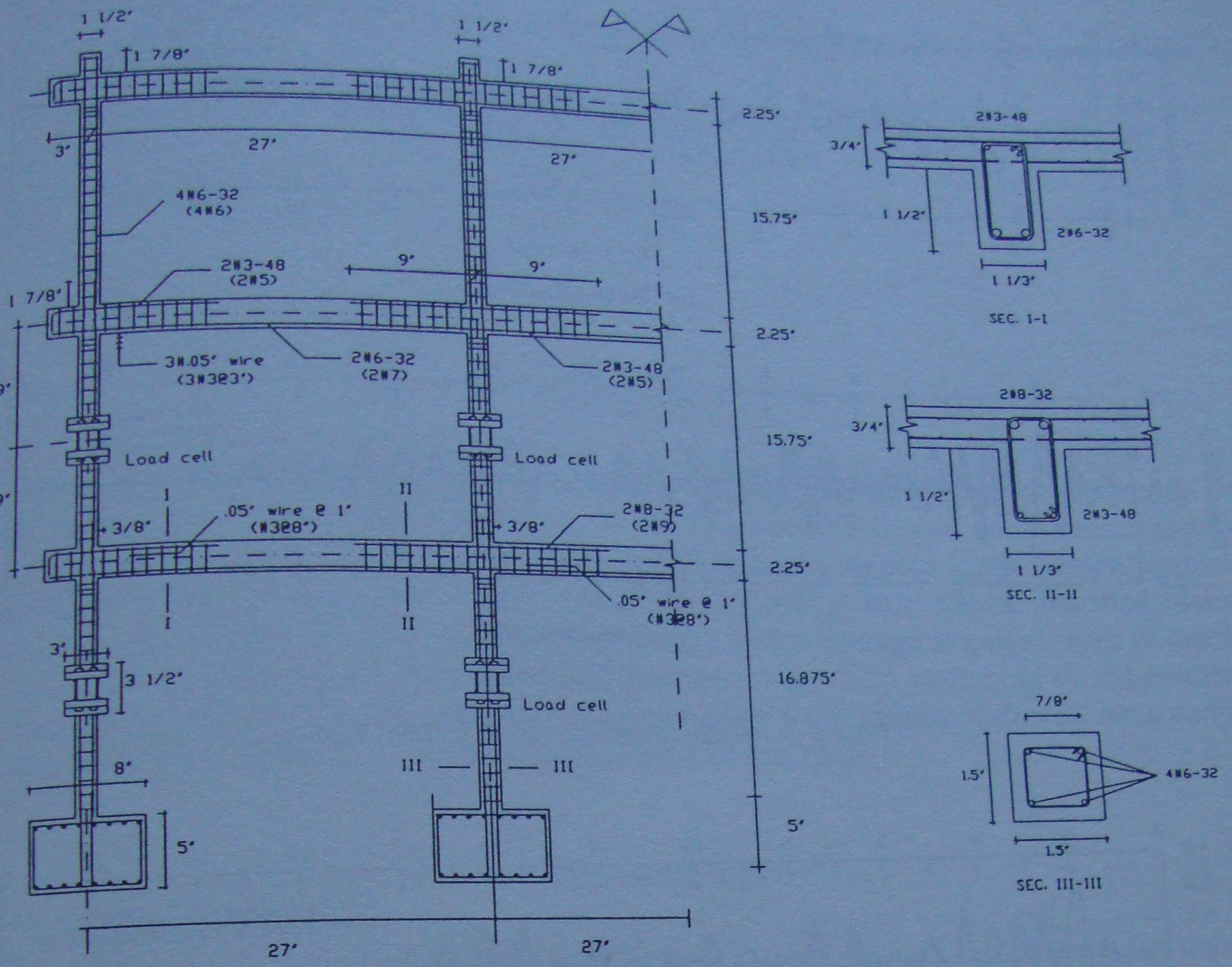
Run	Mode Shape	Shear Distribution
Taft 0.05-G	0.071" (100%) 0.055" (76%) 0.032" (45%)	0.110 kips (32%) 0.213 kips (63%) 0.338 kips (100%)
Taft 0.18-G	0.725" (100%) 0.571" (79%) 0.332" (46%)	0.580 kips (46%) 1.077 kips (86%) 1.252 kips (100%)
Taft 0.35-G	1.042" (100%) 0.824" (79%) 0.474" (46%)	0.646 kips (47%) 1.232 kips (89%) 1.384 kips (100%)



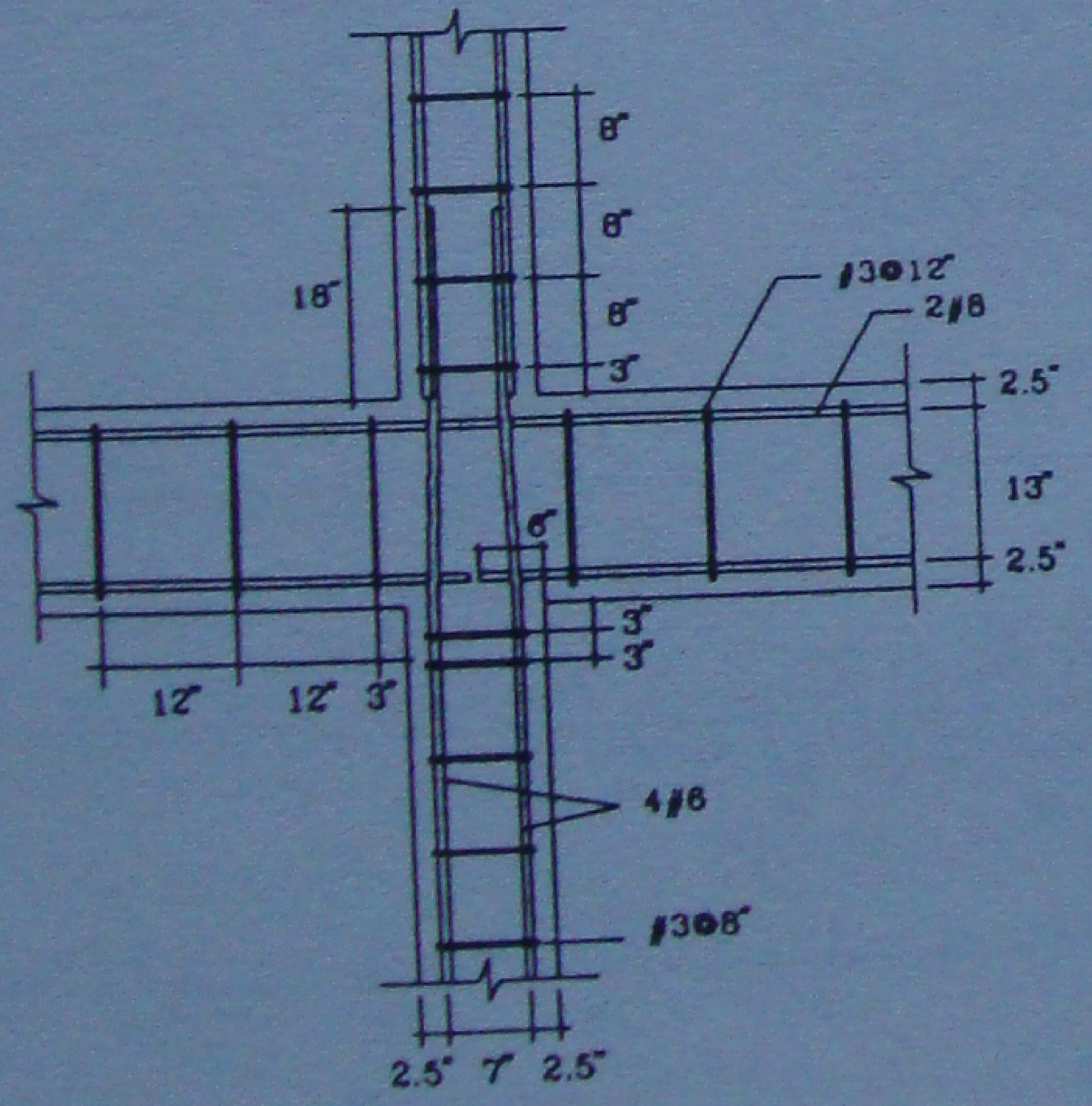
(a) Elevation.

(b) Sideview.

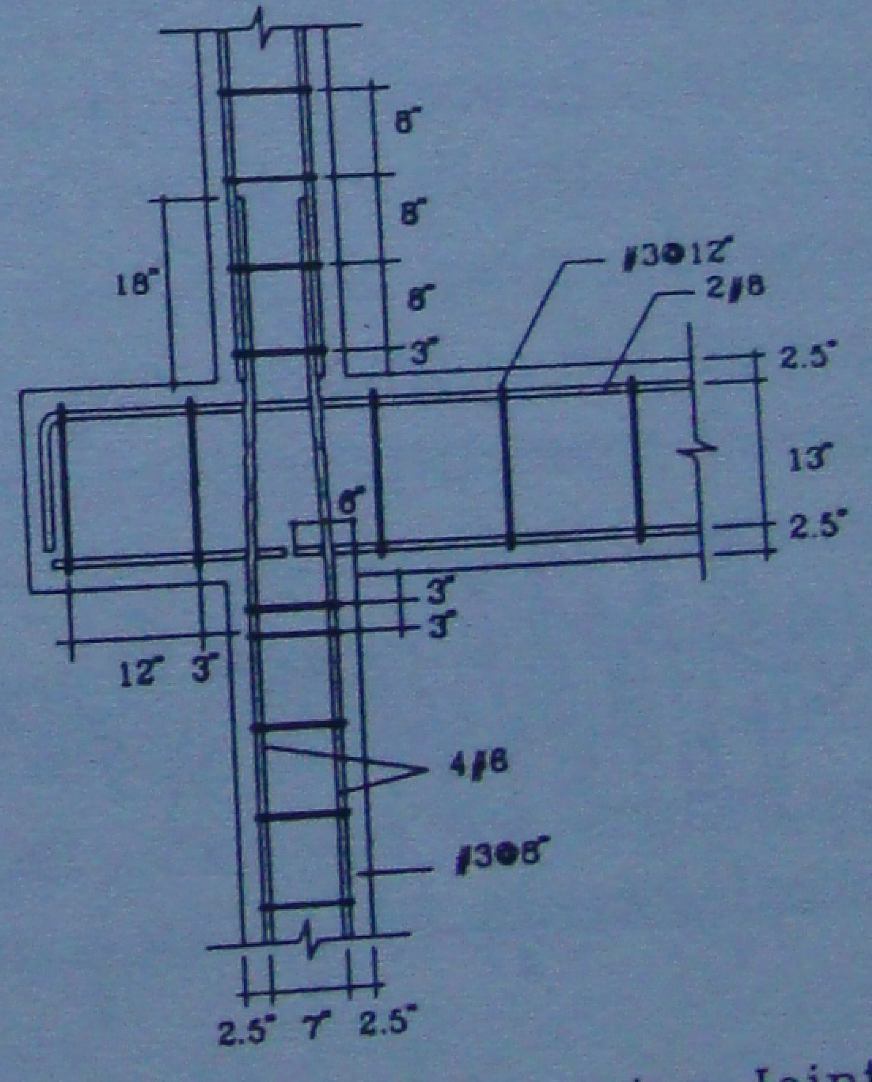
Figure 1: General Layout of The 3-Story Prototype Building.



(a) Reinforcement Detail of The Model (Prototype) Main Frame.

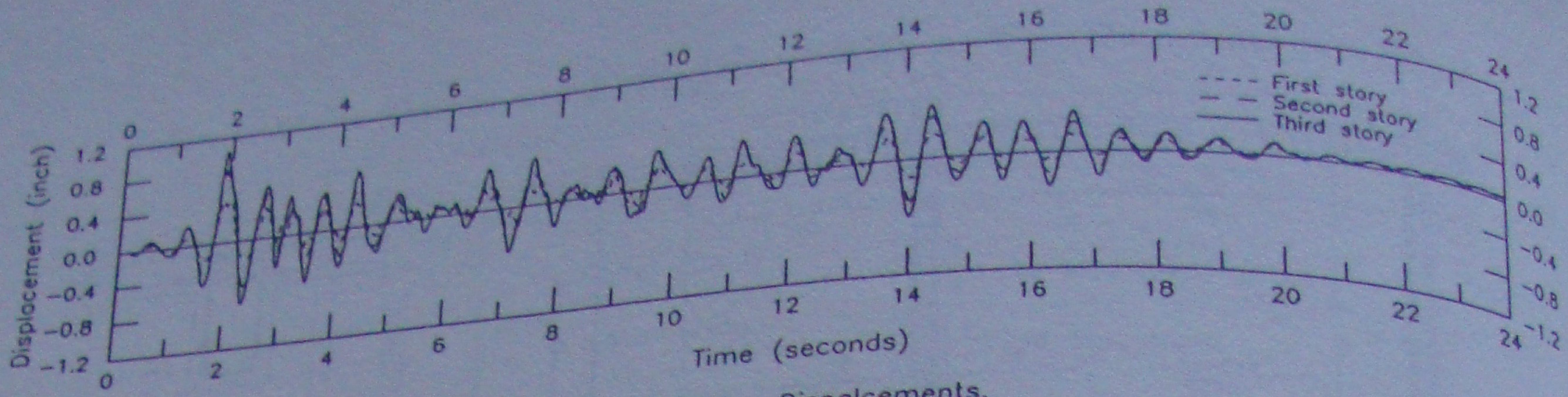


(b) Prototype Interior Joint

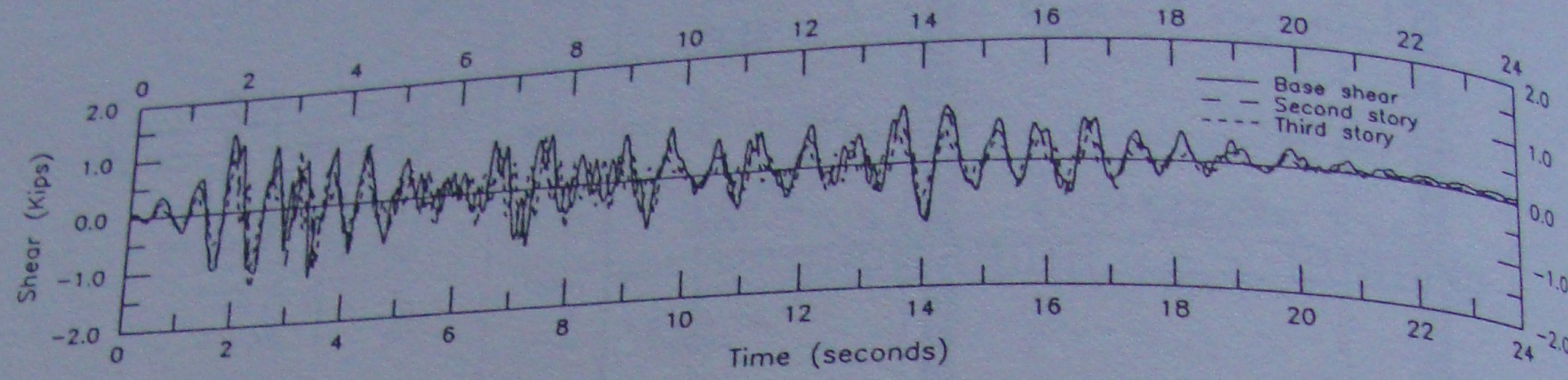


(c) Prototype Exterior Joint

Figure 2: Reinforcement Details of The 3-Story Office Building.

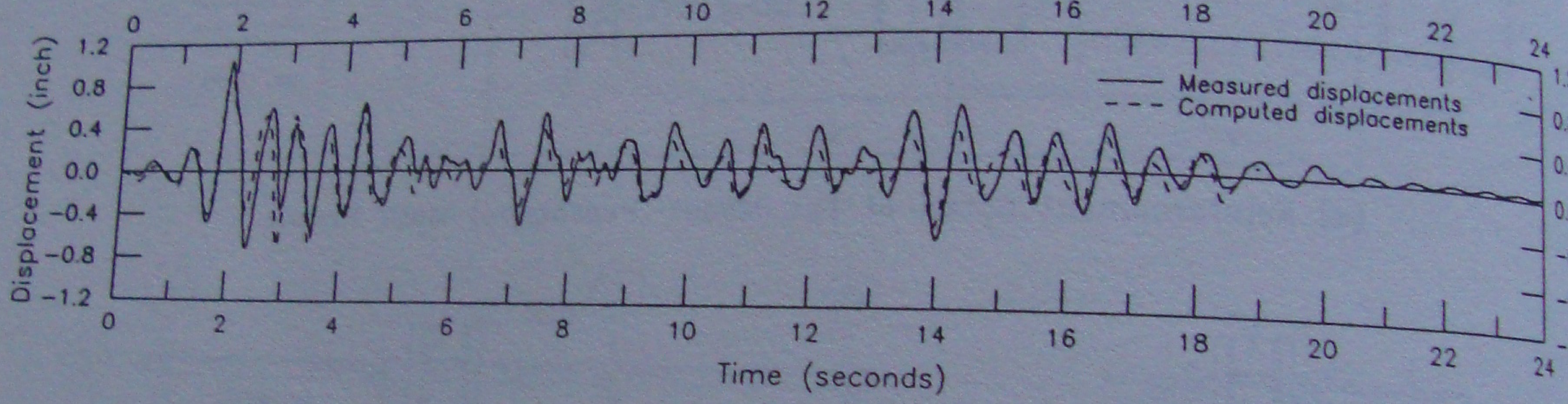


(a) Story Dispalcements.

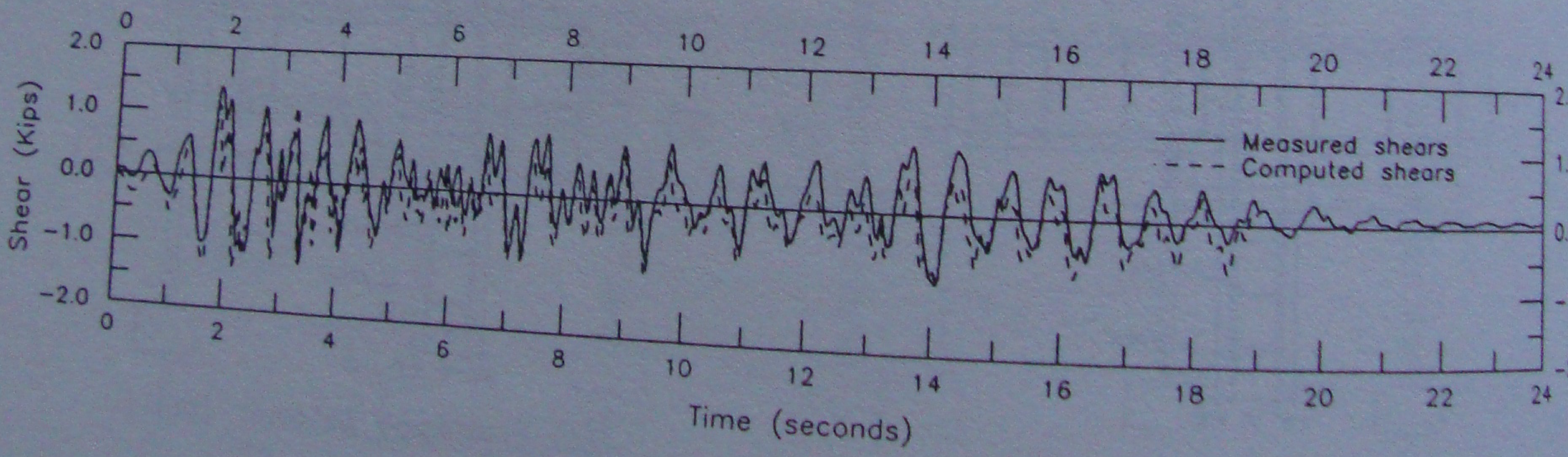


(b) Story Shears.

Figure 3: Model Response at Run Taft 0.35 g.



(a) Top Story Displacement.



(a) Base Shear.

Figure 4: Computed Versus Measured Structure Response (Run Taft 0.35 g).