Column uplift during seismic response of buildings

COLUMN UPLIFT DURING SEISMIC RESPONSE OF BUILDINGS

by

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SUMMARY

During severe earthquakes, tall buildings are subjected to overturning moments which may tend to lift the column bases off the foundation. Building codes in California require anchorage of the columns against such uplift, although the cost of anchorage may be high. In this paper, shaking table tests of a 9 story building frame model are described, and the relative behavior with and without anchorage is discussed. Excellent correlations are demonstrated between observed performance and computer predictions.

1. INTRODUCTION

1.1 Overturning Effects during Large Earthquakes

The lateral loading applied to a structure during a major earthquake will often greatly exceed that suggested by current building code seismic provisions. This fact has long been recognized by code committees, and provisions requiring adequate detailing to accommodate local excursions into the nonlinear behavior range have resulted.

No rational provisions have yet been incorporated into codes, however, to consider the effect of the large overturning moments resulting from actual intense seismic loadings. Generally, current code provisions require that whatever overturning moment results from the design calculations must be resisted completely by the structural system, even if supplementary anchorage is indicated. It should be noted, however, that design loadings almost always are lower than the maximum credible seismic loading.

1.2 Implications of Overturning Moment Overloads

If an overturning moment is applied to a structure which exceeds the structure's overturning capacity, a transient uplifting of portions of the structure from its foundation will result. This uplifting response, while not implying imminent toppling of any practical building, is highly nonlinear. Rational consideration of this overload condition, therefore, requires investigation into the category of nonlinear response represented by uplifting systems.

2. UPLIFTING STEEL FRAME TEST PROGRAM

2.1 Preliminary Investigation

Any good structural design must be based on a thorough understanding of the behavior of the structural system when subjected to the anticipated loading conditions. Because severe seismic loading conditions will usually result in a nonlinear response, at least for economical structures, this type of response should be understood as completely as possible. To better understand the nonlinear phenomena associated with uplifting response, a preliminary experimental and analytical investigation was undertaken utilizing a relatively simple single-bay, three-story steel frame superstructure system [1]. In this investigaton the uplifting response was compared to the response of the system when supplementary overturning anchorage was provided, for similar intense excitations. The experimental portion of the investigation was conducted on the U.C. Berkeley shaking table.

The results of this preliminary investigation showed a considerable reduction in applied loading and ductility demand for the uplifting response, when compared to the corresponding fixed base response. In addition, analytical predictions of the uplifting response agreed quite well with the observed experimental results. A mathematical model employing bilinear elastic foundation support elements having zero capacity in the upward direction, combined with a tangent stiffness proportional damping matrix, was utilized in this analytical work.

2.2 Nine-Story Frame Model

As a result of these promising preliminary results, an investigation was undertaken into the uplifting response of a more sophisticated superstructure system, one which would be more representative of an actual prototype structure. The 9 story steel frame model, pictured in Fig. 1 and shown schematically in Fig. 2, was designed and fabricated for this purpose [2]. As can be seen from the indicated dimensions of the structure in the diagram, the test system was approximately a 1/3 scale model of a realistic, although hypothetical, prototype steel building frame with moment resisting joints. The column base detail which allowed uplift and included a cushioning pad is shown in Fig. 3.

The test structure and the shaking table, described elsewhere by Rea and Penzien [3], were instrumented extensively for the experimental program. A total of 128 data acquisition channels were utilized, with each channel being sampled at a rate of approximately 50 times per second. The resulting data defined the shaking table accelerations and displacements, the accelerations and displacemdnts at the floor levels of the model, its uplift displacements, and selected local member force and deformation quantities, primarily in the lower two floors.

2.3 Experimental Results

One uplift test and one fixed base test utilizing a time-scaled version of the 1971 Pacoima Dam S74W input signal are discussed in this paper. The table accelerations and displacements in the horizontal direction along with the response spectra of these motions for damping ratios of 0.01, 0.02, 0.03 and 0.05 are shown in Figs. 4 and 5. As can be seen in these figures, the table shaking for the two tests was similar and very intense in nature.

The overturning response for the two base conditions is shown in Figs. 6 and 7. Comparison of these figures demonstrates the dramatic effect of column uplift on the response of the structure. The uplift phenomenon performs as a structural "fuse," limiting the overturning forces generally to those values which initiate uplift. Transient excursions beyond this limiting value of load do occur, primarily at instants when the column bases impact with the foundation. However, these very short-lived impulses, such as that seen at about 3.8 seconds in the uplift time history of Fig. 7, appear to be resisted largely by the inertia of the system; these impulses are not so evident in the local element force records.

2.4 Analytical Results: Uplift Test

The analytical results for the uplift test are shown in Figs. 8 through 11; in these figures the analytical quantities are plotted as solid curves together with the corresponding experimental data presented as dashed curves, in order to facilitate comparison of the two. For this analytical work the uplift response was included in the mathematical model through the use of bilinear elastic foundation elements as described for the preliminary tests. In addition a tangent stiffness proportional viscous damping matrix giving a linear 1st mode damping ratio of 0.007 was employed.

As can be seen from the relative horizontal floor displacements of Fig. 8 and the uplift displacements of Fig. 9, the global structural response was predicted quite accurately by this mathematical model. The local 1st floor column forces of Figs. 10 and 11 also show good correlation. An interesting feature of the column base moments of Fig. 11 is the gradual transition from a fixed-base to a free-base condition as the columns separate from the foundation on one side before the other. In addition, a slight numerical stability problem is evident in the calculated column axial forces in Fig. 10; this impact-associated analytical complication is understandably sensitive to the integration time step employed. A time step of 0.0048 sec. was used for this analysis, and the results seem to be within acceptable engineering resolution.

2.5 Analytical Results: Fixed Base Test

The analytical results for the fixed-base test are shown in Figs. 12 through 14, plotted in a similar manner to the previous results. For this analysis the 1st mode damping ratio was specified to be 0.032, and the integration time step was 0.0096 sec. The longer time step was permissible because of the lack of any impact problem in this test. The response during this test extended slightly into the nonlinear strain range for the beams and columns in the lower two floors; their nonlinear moment-curvature behavior was included in the mathematical model through the use of concentrated bilinear plastic hinges at the ends of the members.

As is shown in Fig. 12, the displacements for this test were again predicted accurately. The displacements for the fixed base tests were only slightly lower than those observed during the uplift test, indicating that the internal structural deformations were considerably higher for the fixed base test because there was no "rigid body" contribution to these displacements. The local column forces of Figs. 13 and 14 again show generally good agreement between experimental and analytical values. It can be seen however, that the 2nd mode response was not predicted nearly as accurately as the 1st mode response. Mathematical models, due to assumptions made during their construction, generally tend to overestimate higher mode frequencies. It is interesting to note that the dynamic column compression forces of the fixed base test in Fig. 13 are higher than those of the uplift test, shown in Fig. 10, demonstrating that the impact effect resulting from uplift is not very severe.

3. PRACTICAL DESIGN IMPLICATIONS

The results of this test program indicate that a structural system including a rationally designed uplifting capability would have an enhanced probability of surviving a severe earthquake in a functional condition. Both the structural frame and the nonstructural exterior and interior walls are subjected to reduced strains and deformations. In order to fully exploit this improvement, it is suggested that the two-level design philosophy be employed in the design of building frames in which uplift is allowed. Under expected moderate earthquake conditions, corresponding to normal building code design requirements, it is probable that the dead weight overturning constraint will not be exceeded, and the design may follow standard procedures, because no uplift will occur.

However, under severe seismic conditions, corresponding to the maximum expectable earthquake at the building site, a dynamic response analysis should be performed to determine whether the structure develops overturning tendencies. If so, a nonlinear dynamic uplift analysis should be made to evaluate the maximum frame stresses which can be expected. As is evident from the results of this investigation, these stresses will be significantly reduced by allowing uplift to take place; in effect, the uplift mechanism absorbs the seismic displacements and greatly reduces the ductility demand on the structure frame.

In designing for uplifting performance, the following factors should be considered.

There should be relatively little restraint to vertical separation of the column bases from the foundation, although a rather flexible energy dissipation mechanism could be incorporated if deemed desirable. Sufficient resistance to steady-state wind loading must, of course, be provided.

A reliable "shear key" is required to prevent the columns from walking off the foundation during uplift response. The flexure plate concept used in the test program seems worthy of serious consideration, but would have to be extended to accommodate a biaxial type of response.

An impact pad must be provided at each column, which will tolerate impact and protect potentially more brittle components.

Flexibility is required in service connections to the structure. A centrally located service core, where little or no separation should occur, would seem a logical design concept.

A reliable nonlinear dynamic analysis which takes account of the uplift response must be made to ensure a tolerable amplitude of uplift motion, even during the most severe credible earthquake.

Looked at in this light, a planned uplift capability can be thought of as a supple ment to the current detailing requirements which ensure a safe behavior during overload conditions. Due to the fact that little experience has been acquired with the uplift phenomenon, however, it should be considered explicitly in the design process, through an actual dynamic analysis.

Although this investigation was concerned with a steel moment frame, there is no reason to limit uplift behavior to only that category of structural system. Ductility may be achieved readily in steel moment frames, and systems without thi inherent ductility would have their seismic performance enhanced to an even great degree by the inclusion of an uplift capability. Achieving a ductile behavior in reinforced concrete frames, for example, requires adequate confinement steel, wit associated increased material and placement costs. Shear wall and braced frame systems also present special problems in achieving high ductility. Eliminating or at least substantially reducing the ductility demand for those systems could easily lead to considerable savings in the superstructure system, and simultaneously provide enhanced safety for the occupants.

As an added economy to these mentioned above, potential foundation savings could result in structural systems for which current code overturning provisions requir supplementary anchorage. Such overturning anchorage can be exceedingly expensive requiring deep piles or caissons to provide the required tensile capacity.

4. CONCLUSIONS

This research presents strong evidence for including a rationally planned uplift capability as part of a dual seismic performance criterion. By this approach increased safety can potentially be coupled with increased economy. Analytical tool are presently available to accurately predict this type of nonlinear response; indeed this catagory of nonlinearity lends itself very well to analysis, due to its inherently simple nature.

ACKNOWLEDGEMENTS

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3. REA, D., and PENZIEN, J., "Dynamic Response of a 20' x 20' Shaking Table," Proceedings of the 5th World Conference on Earthquake Engineering, Rome, 1973.

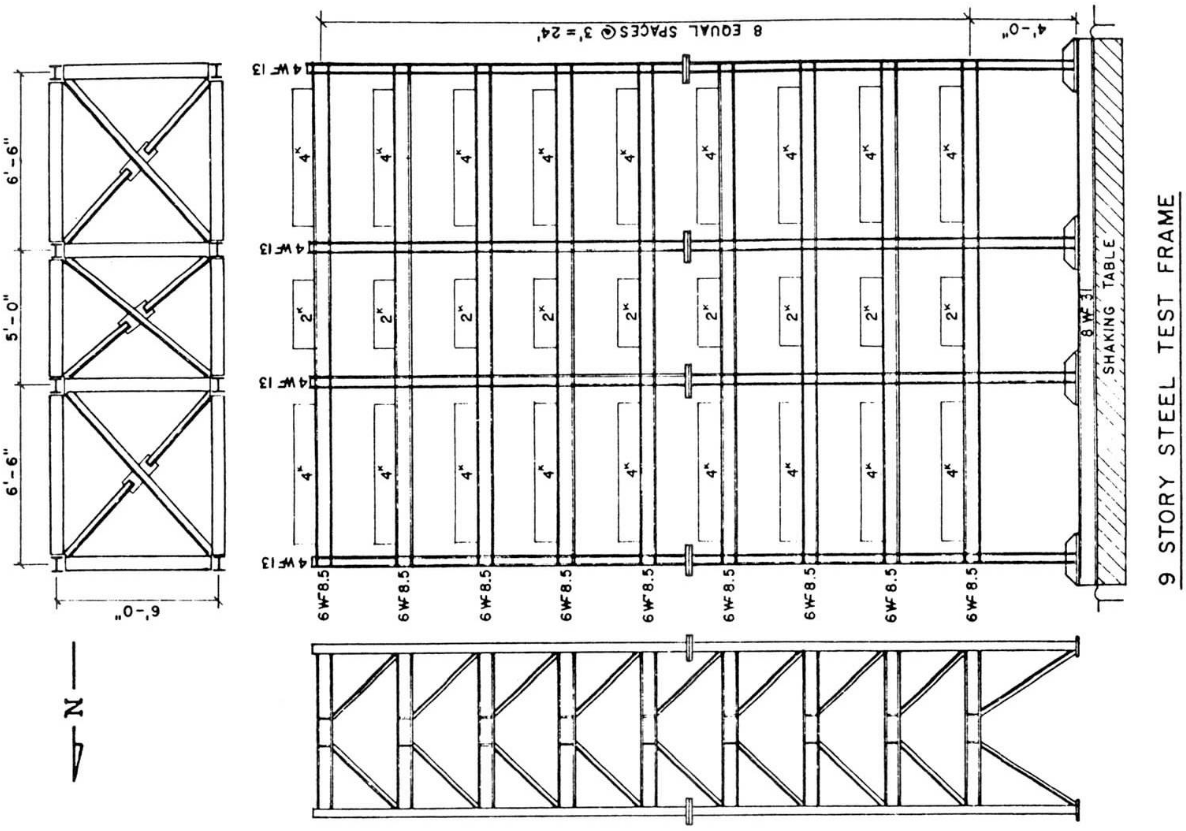


Fig. 2 Test Model Schematic

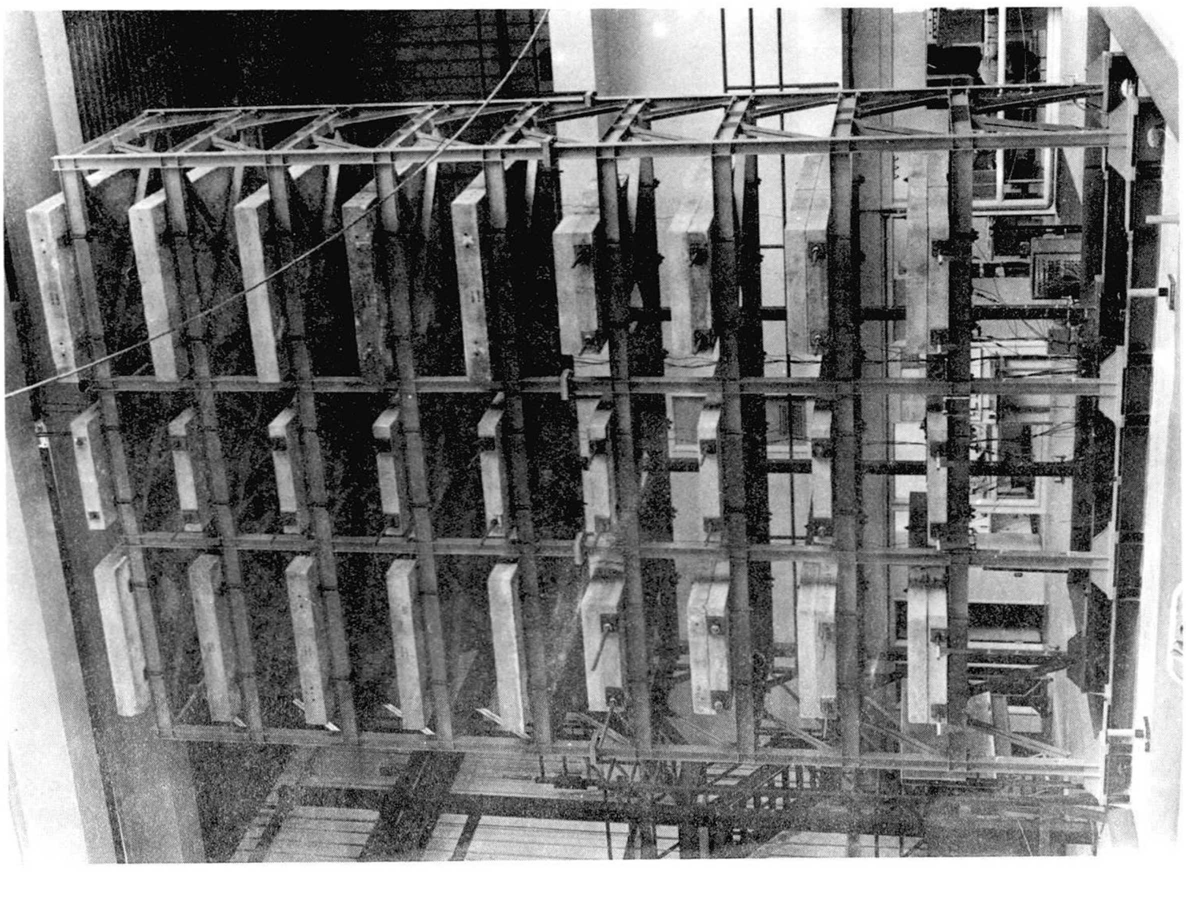


Fig. 1 Test Structure on the Shaking Table

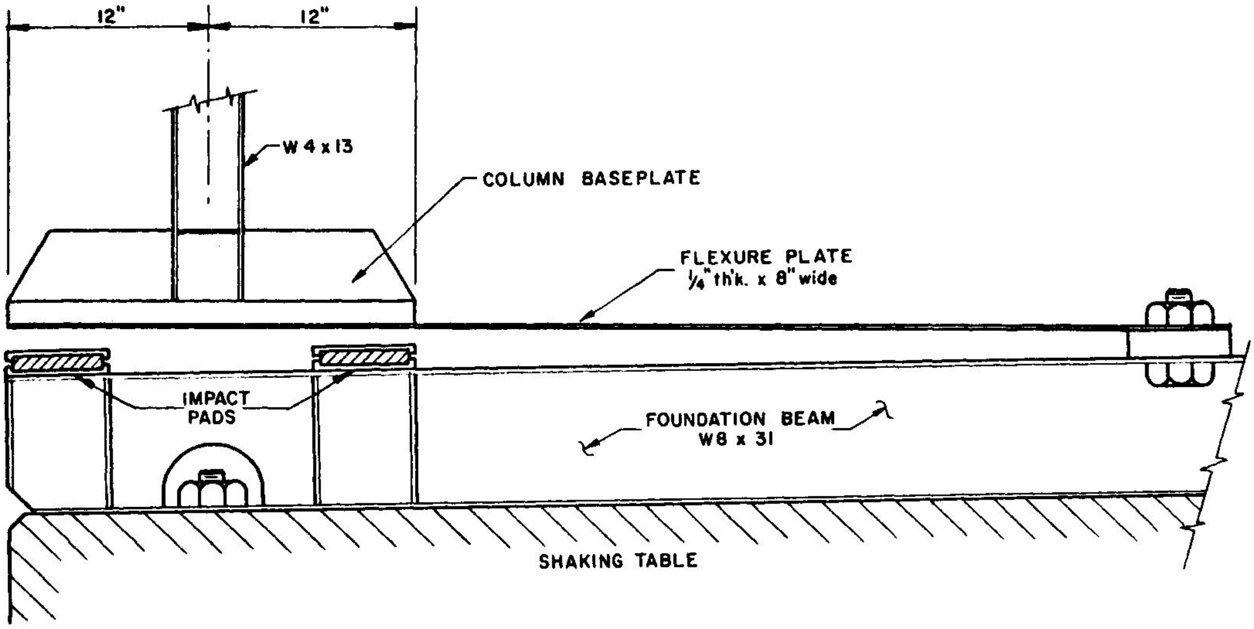
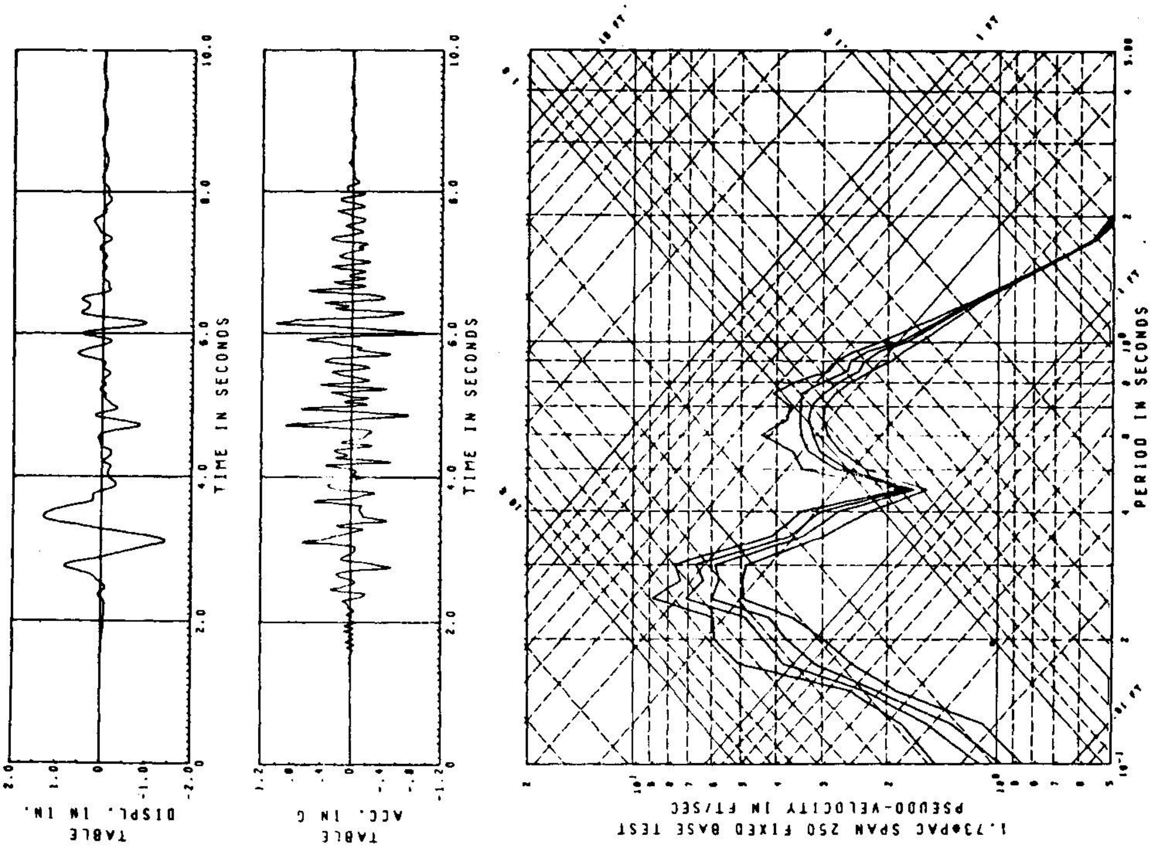
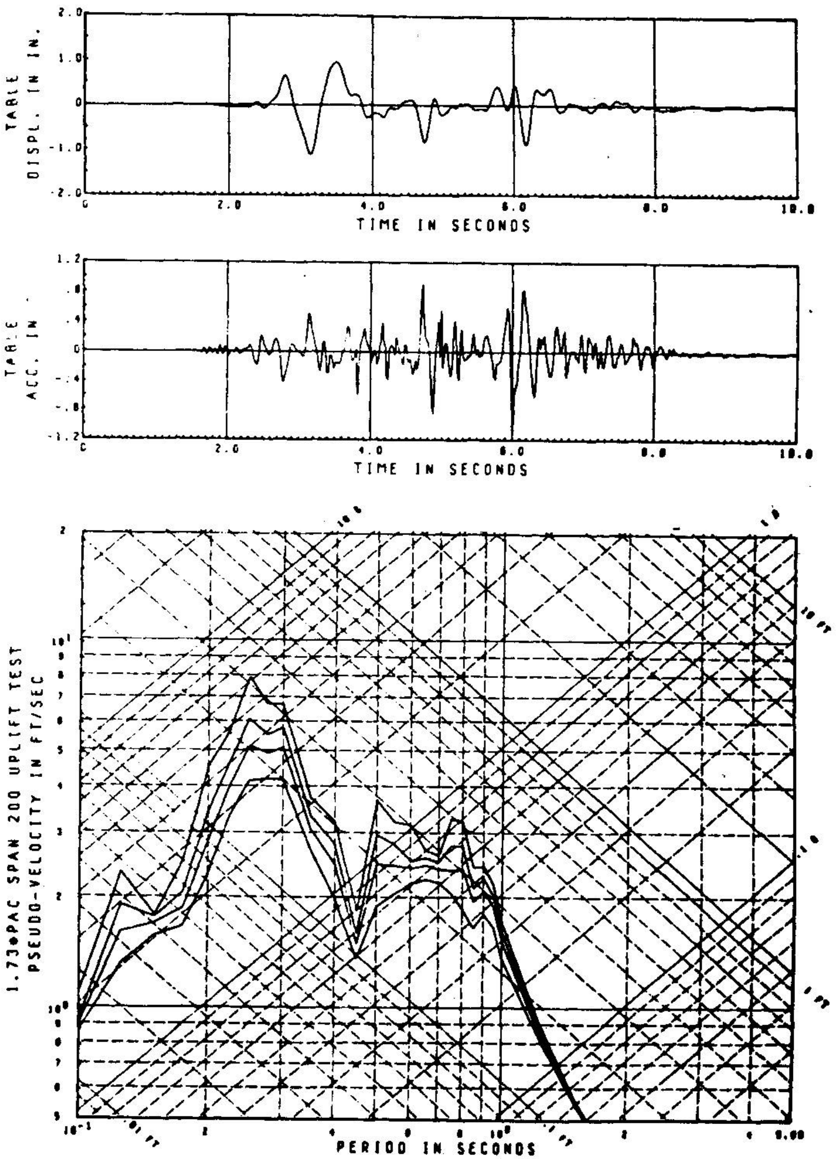


Fig. 3 Uplifting Column Base Detail



Damping = .01, .02, .03, .05 Critical

Fig. 4 Fixed Base Test Horizontal Table Motion



Damping = .01, .02, .03, .05 Critical

Fig. 5 Uplift Test Horizontal Table Motion

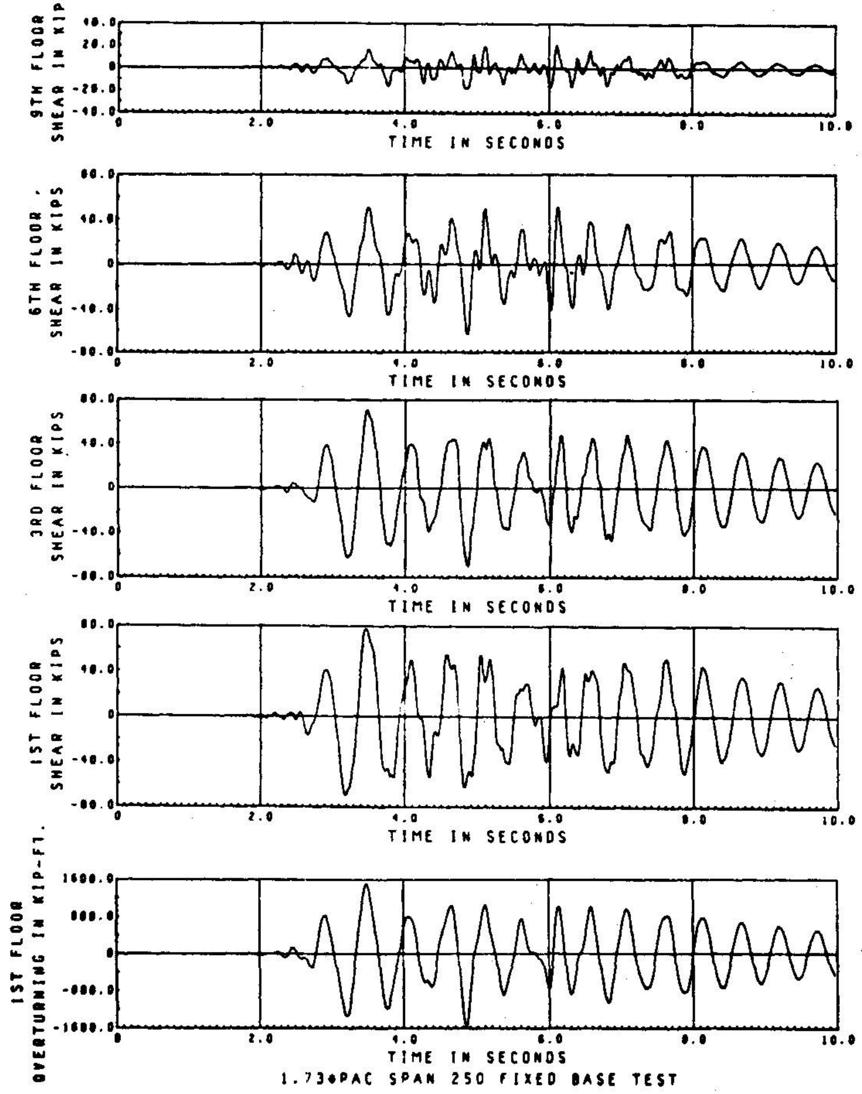


Fig. 6 Fixed Base Story Shears and Base Overturning

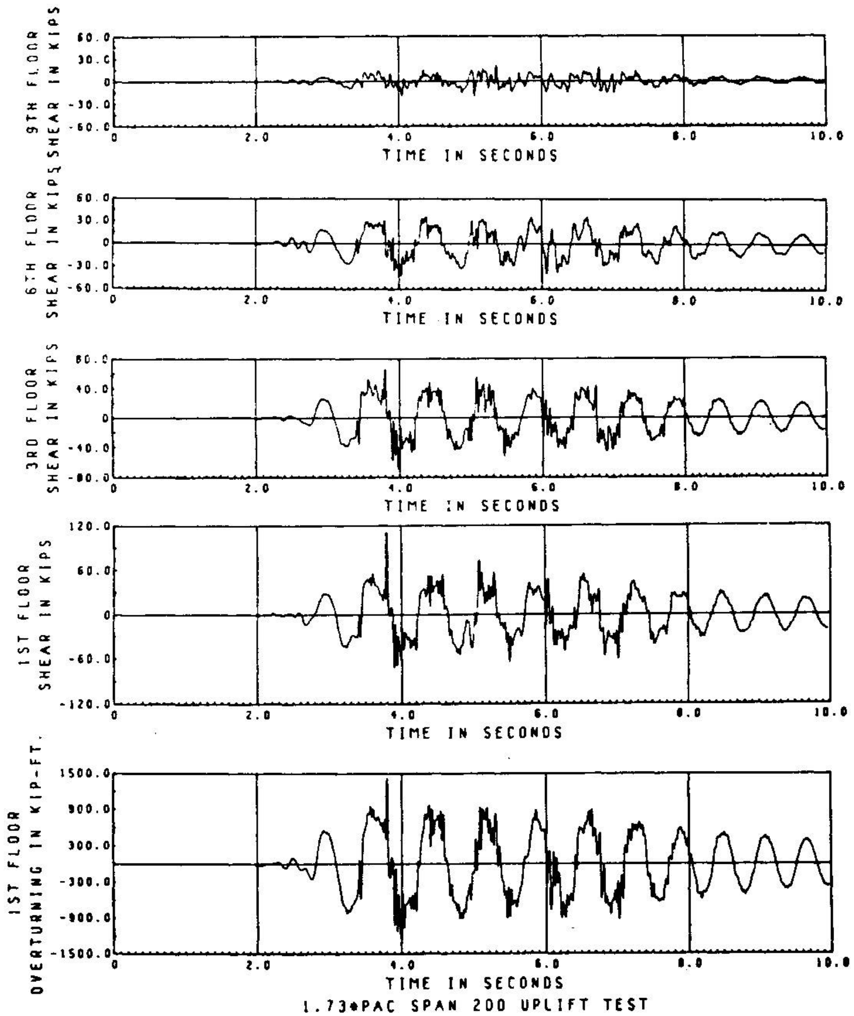


Fig. 7 Uplift Story Shears and Overturning

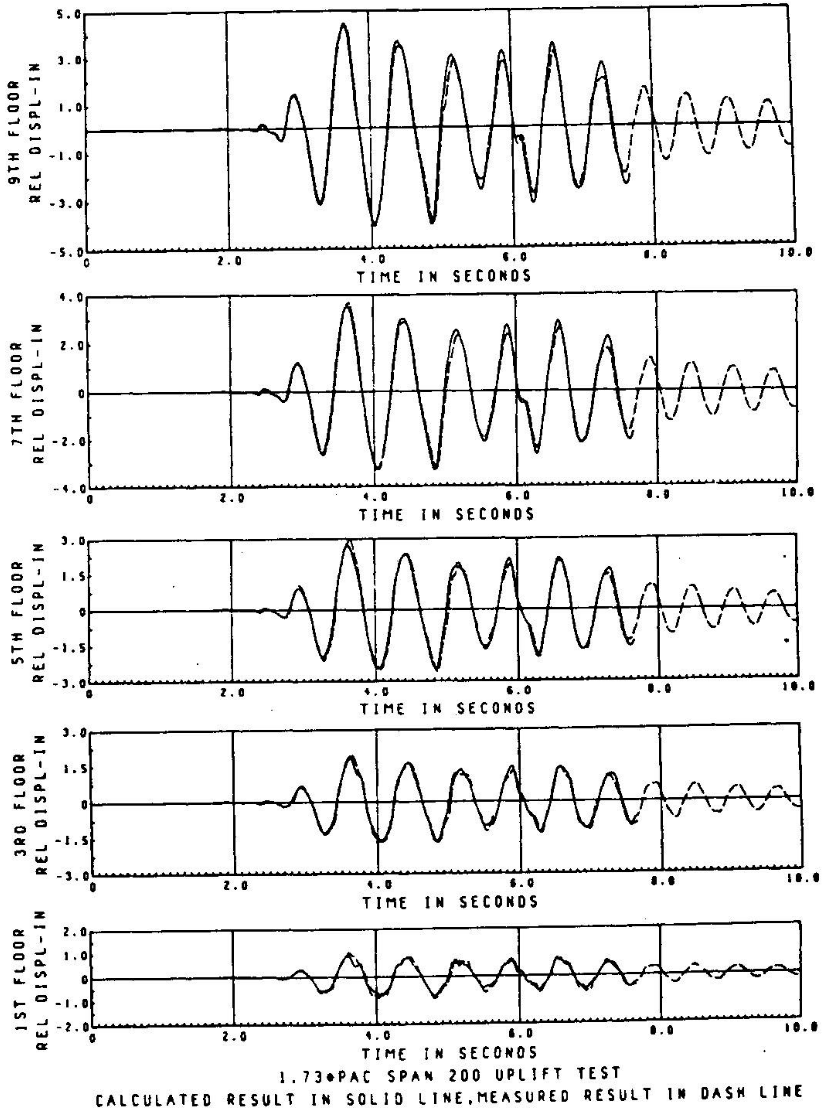


Fig. 8 Uplift Test Results; $\beta = .0011$, $\Delta t = .0048$ sec.

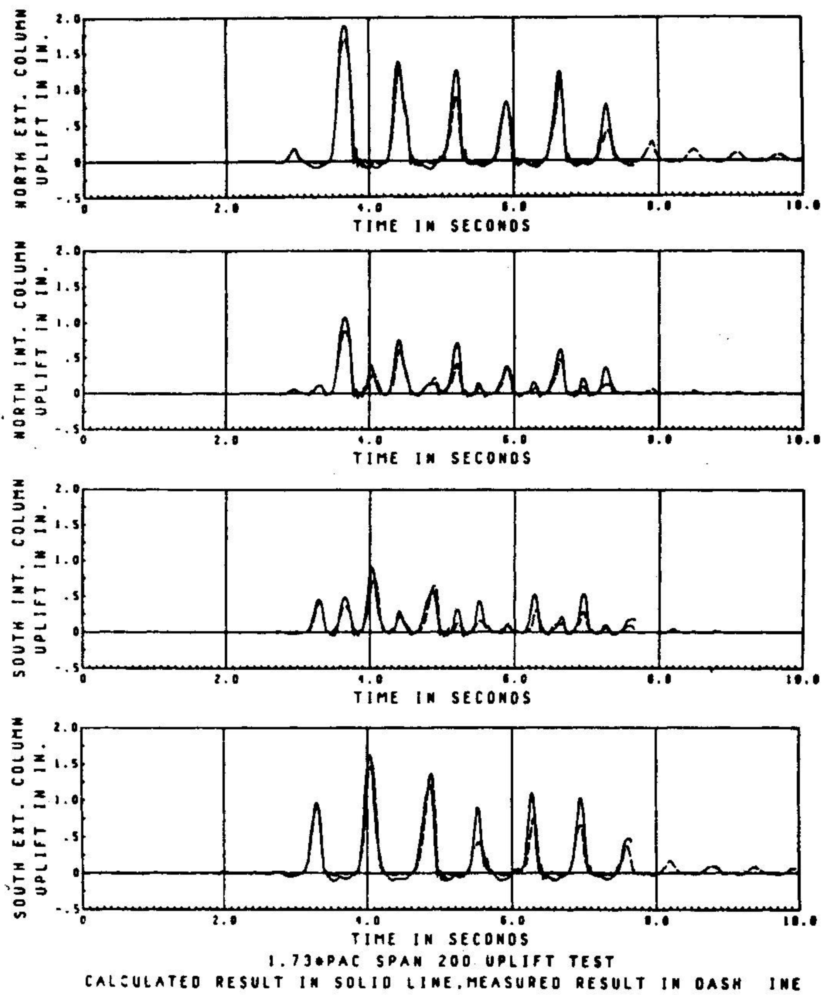


Fig. 9 Uplift Test Results; β = .0011, Δt = .0048 sec.

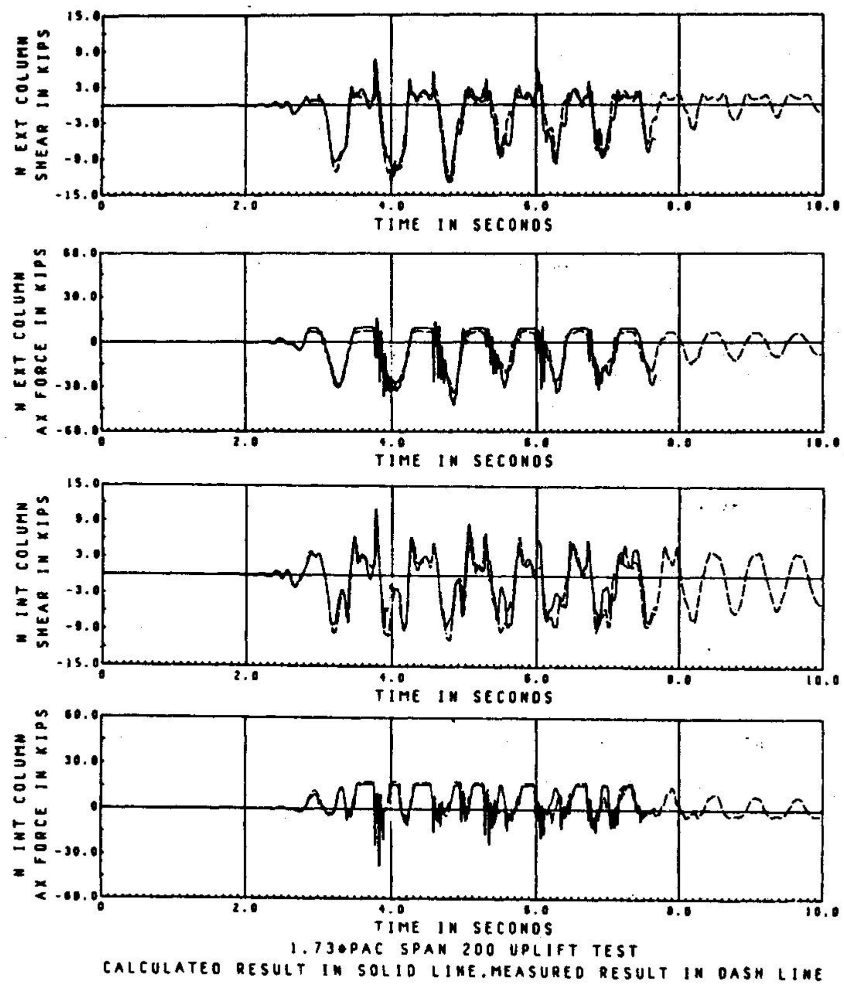


Fig. 10 Uplift Test 1st Floor Column Forces; β = .0011; Δt = .0048 sec.

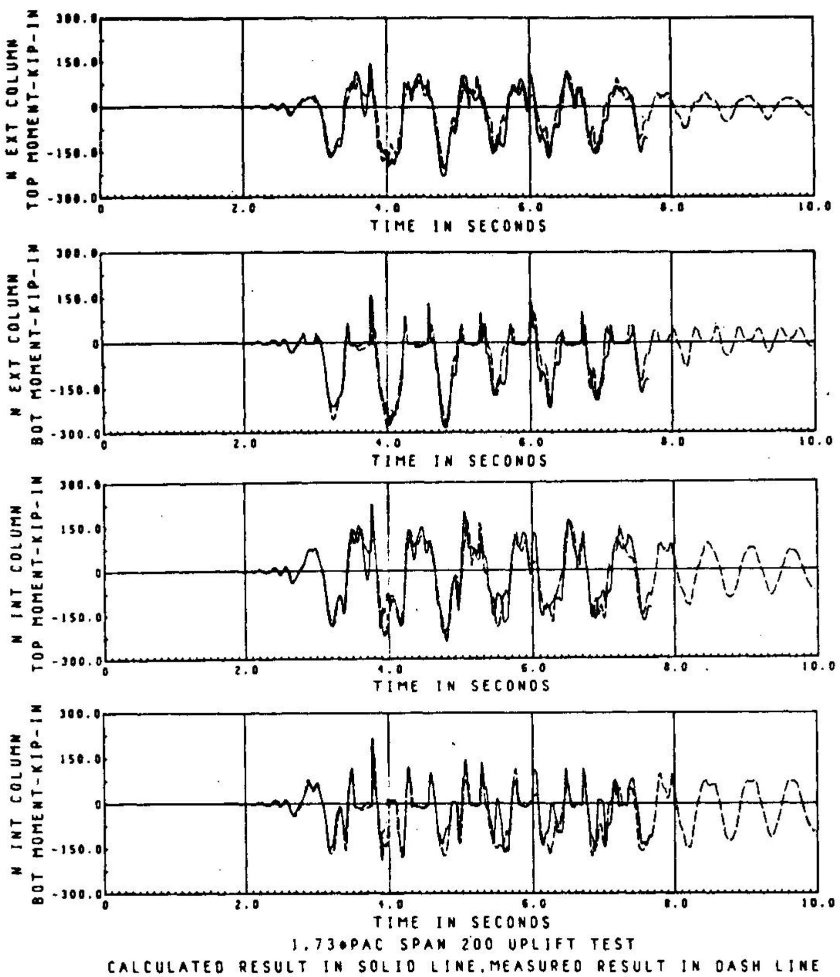


Fig. 11 Uplift Test 1st Floor Column Moments; β = .0011, Δt = .0048 sec.

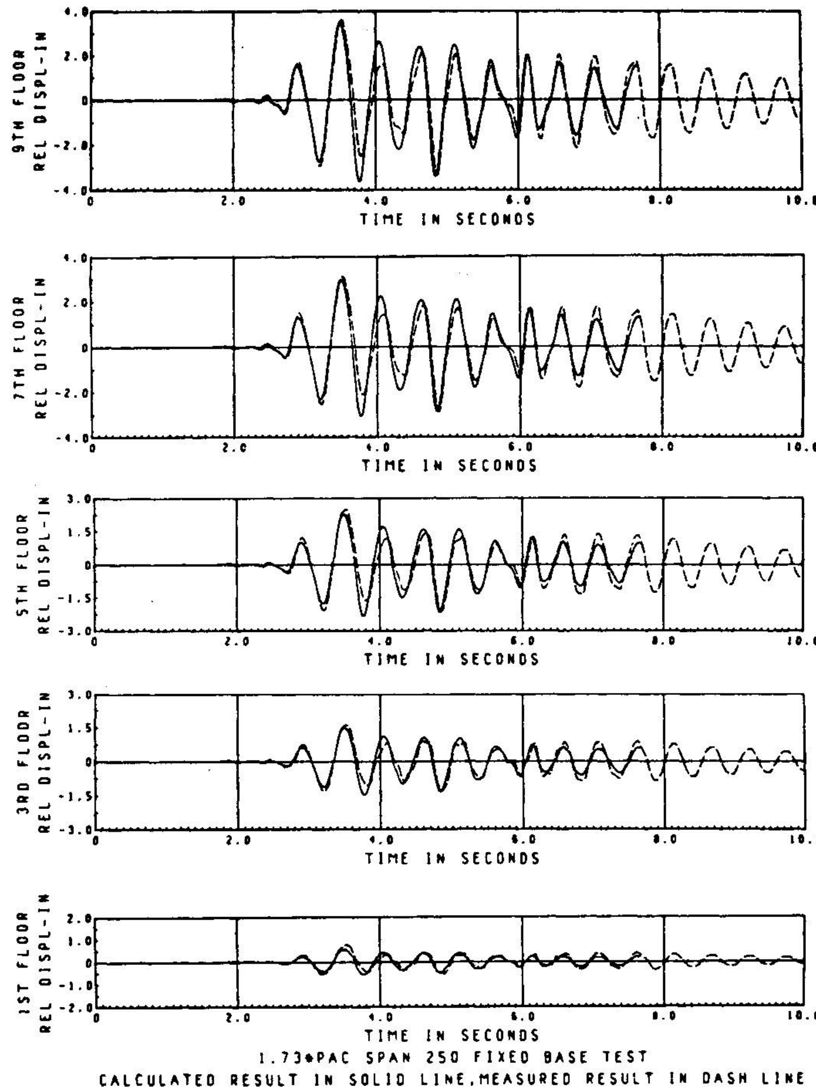


Fig. 12 Fixed Base Test Results; β = .005127; Δt = .0096 sec.

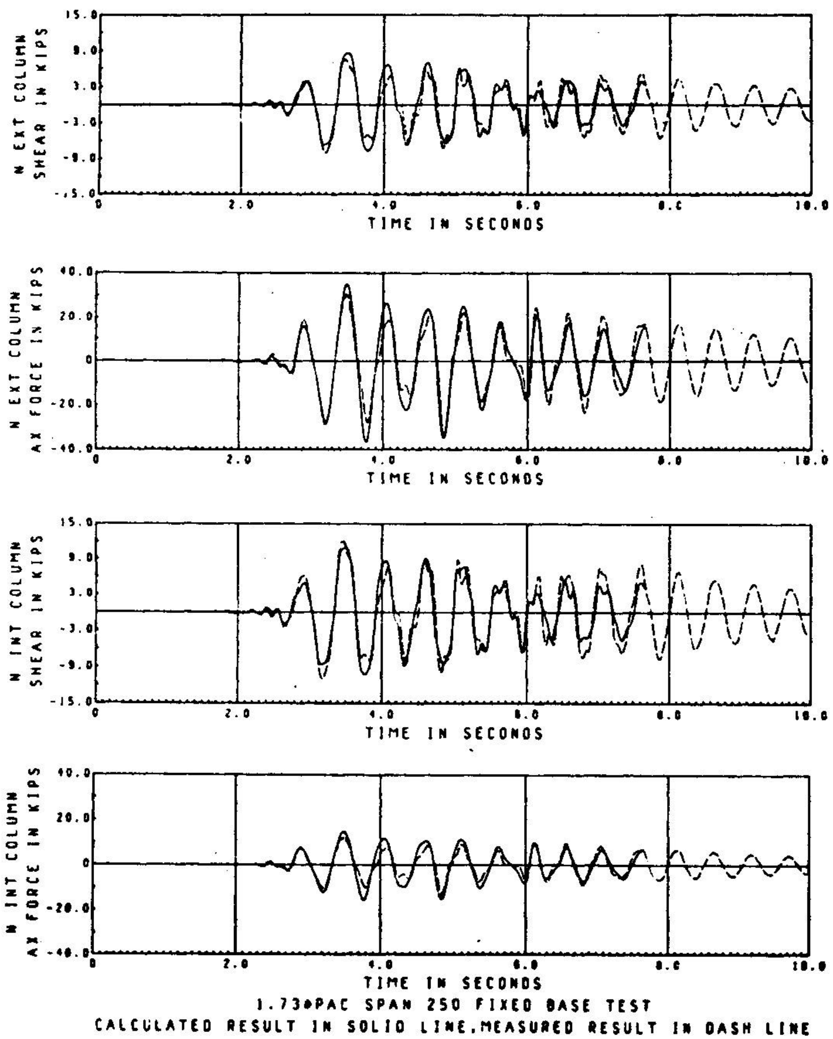


Fig. 13 Fixed Base 1st Floor Column Forces; β = .005127, Δt = .0096 sec.

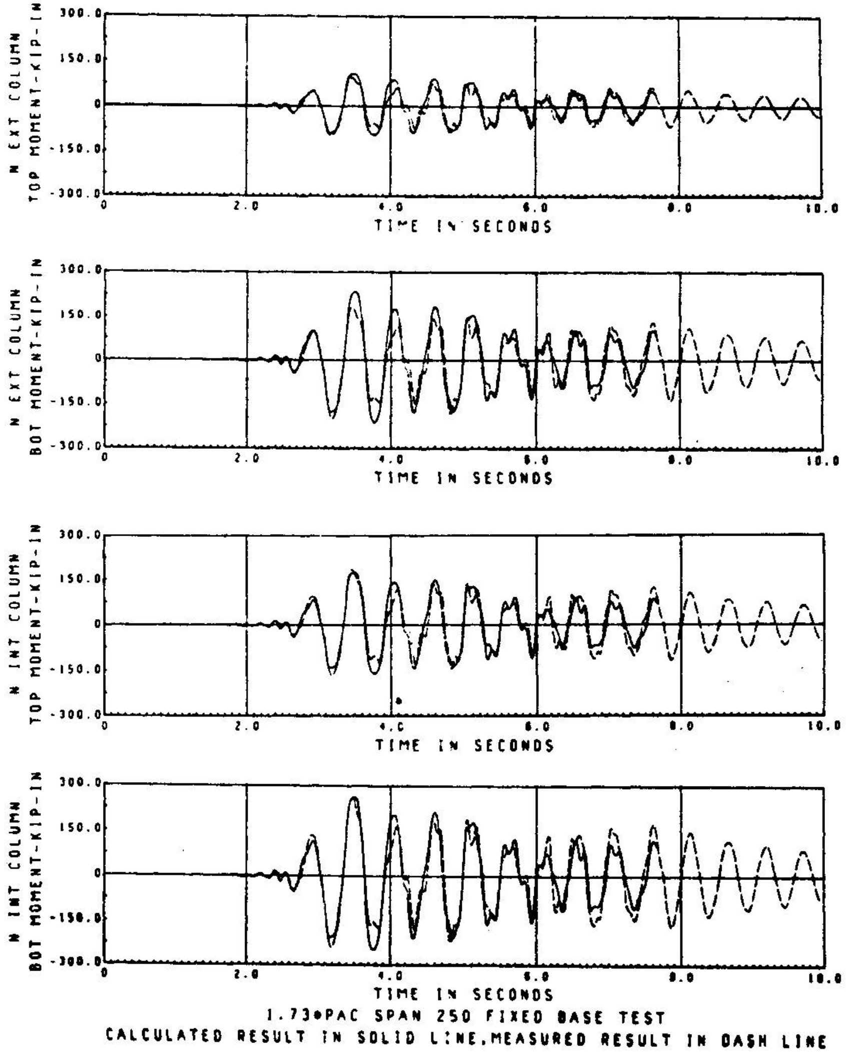


Fig. 14 Fixed Base 1st Floor Column Moments; β = .005127, Δt = .0096 sec.