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Analysis and tests of a partially restrained braced frame subjected to seismic loadings

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ABSTRACT: A finite element model and an equivalent truss model have been developed to capture the linear and nonlinear behavior of bolted bracing connections. To validate these models, a $\frac{1}{2}$-scale plane specimen representing the lower story of a three-story concentrically braced steel frame has been tested under cyclic and pseudodynamic loadings. The latter tests have been conducted with a newly developed substructuring approach. In this paper, the modeling methods and experimental procedures are summarized, and the correlation of numerical and experimental results, in terms of global load-displacement hystereses, energy dissipation, and displacement time histories, is examined. It has been shown that the displacement response of the braced frame can be satisfactorily evaluated with the proposed connection models. Furthermore, it has been shown that the results of frame analyses in which the flexibility of moderately bolted bracing connections is neglected can underestimate the story drift by 33% under a severe seismic load.

1 INTRODUCTION

The seismic performance of concentrically braced frames (CBF) has been widely studied. Some of the results have led to design recommendations adopted in the recently developed document on Seismic Provisions for Structural Steel Buildings - LRFD (AISC 1990). In particular, stringent requirements have been proposed to avoid undesired brittle behavior of CBF under severe seismic load conditions, including the minimum strength requirements for bracing connections of the type shown in Fig. 1. Nevertheless, no specific guidelines or commonly accepted analysis methods are available to evaluate the strength and assist the design of such connections. Furthermore, in frame analysis, bracing connections are often modeled as ideal pins or rigid joints without the connection flexibility, contributed by clip angles and gusset plates, taken into consideration. In reality, the connection flexibility may affect the sway of braced frames under static and dynamic loads, and bracing connections are expected to affect the local stress distribution in beams and columns around the joints.

To evaluate the strength and stiffness of bracing connections through detailed stress analysis, a plasticity-based finite element model has been developed by Bursi (1992a) for modeling bolted clip-angle and welded fasteners that are commonly used in such connections. The model has been validated with experimental data obtained from isolated bolted and bolted-welded bracing connections (Bursi et al 1992b). Furthermore, to allow a realistic assessment of story sways in braced frames, a phenomenological model based on an equivalent truss concept has been proposed (Bursi and Gerstle 1992c). While this model is efficient for frame analysis, the strength and stiffness of the equivalent truss have to be calibrated by means of refined finite element analysis. The main objectives of this study are to examine the accuracy of these models when used in the two-stage analysis of a braced frame as mentioned above, to investigate the influence of connection flexibility on the sway of a braced frame under seismic loads, and to evaluate the performance of a CBF, including that of bolted bracing connections, designed in accordance with the LRFD provisions (AISC 1990), under severe seismic load conditions.

To this end, a $\frac{1}{2}$-scale plane frame specimen representing the bottom story of a three-story CBF has been tested under cyclic and pseudodynamic loads. The latter tests have been conducted with a newly developed substructuring approach (Vannan 1991), in which only the bottom story of a multistory frame needs to be tested, while the upper stories are modeled in an on-line computer. Numerical computations are conducted during testing to evaluate the displacement response that is to be imposed on the structural subassembly. The experimental results are compared to frame analyses conducted with an equivalent truss model using the computer program ANSR-III (Oughourlian and Powell 1982). In this paper, both the analysis methods and experimental procedures, the numerical and experimental results in terms of the global response, plastic energy dissipation, and failure mechanism of the frame specimen at various seismic load levels, are presented. Finally, the influence of connection flexibility and the overall frame performance are commented upon.

2 FRAME SPECIMEN AND ANALYSIS METHODS

In this study, a diagonal bracing system representing a single bay of a five-bay, three-story office building which has been designed with the "Equivalent Lateral Force Procedure" defined in the Uniform Building Code (UBC 1988) is considered. Because of the limited capacity of the loading apparatus, the frame has been designed for UBC Seismic Zone 3. The geometric configuration and member sizes of the prototype frame are shown in Fig. 2. The design complies with the AISC-LRFD Manual (AISC 1986) and the recently proposed seismic provisions (AISC 1990). The lateral resistance of the frame is provided by a single diagonal brace at each story. While this may not be a most desirable load resistance mechanism, in view of the limited energy-dissipation capability associated with the buckling of braces, this bracing system, nevertheless, simplifies the test procedure and specimen fabrication, and allows for a better comparison of experimental and numerical results. It will also impose severe tensile forces on the connections.

The tests have been carried out on a $\frac{1}{2}$-scale frame specimen representing the bottom story of the prototype frame, as shown in Fig. 3. Similitude rules have been applied to the frame members as well as to the bracing and beam-to-column connections. To allow using standard member sizes, the similitude rules have been slightly violated. In spite of this, it has been shown by numerical simulations that a reasonable dynamic similitude can be attained between the prototype frame and the scale model (Vannan 1991). A typical bracing connection used in the scale model is shown in Fig. 4. This connection consisted of a $\frac{1}{4}''$ (6 mm) gusset plate fastened to the members with $3 \times 2\frac{1}{2} \times \frac{3}{16}''$ (76 x 63 x 4.8 mm) double bolted clip angles characterized by a yield stress F_y equal to 53 ksi (365 MPa). To attain the strength of the bracings (AISC 1990), three bolts were used for each gusset plate-to-member fastener (AISC 1986). The gusset plate buckling strength was computed with a conservative procedure based on the Whitmore section (Gross 1989). This type of connection is designated as 3x3 bracing connection in the following. Furthermore, to provide additional data for validating the analytical models, 2x2 bolted fasteners have also been introduced in a few tests. As shown in Fig. 3, the beam at the top of the first story has been rigidly bolted to the columns by means of Extended End-Plates (EP) $\frac{1}{2}''$ (12.7 mm) thick, to enhance the torsional resistance of the columns. The braced frame has also been investigated with simple beam-to-column connections by adopting bolted clip Web Angles (WA) of the same size of the gusset plate-to-member fasteners. A-36 steel has been used for all members. The actual yield stress F_y of the diagonal brace is found to be about 48 ksi (331 MPa).

The bracing connection used has been analyzed with a refined finite element model, in which the nonlinear behavior of bolted clip-angle fasteners is simulated with an interface element (Bursi 1992a). The element is developed within the framework of hardening plasticity with the constitutive laws based on the stiffness and strength values obtained from physical tests (Bursi 1992a). This analysis has been used to simulate the behavior of bolted bracing connections (Bursi et al. 1992b) and to provide data for calibrating the phenomenological model used for frame analysis. As shown in Fig. 5, the bracing connection model is based on an equivalent truss (Bursi and Gerstle 1992c), which can capture both the rotational constraint and axial flexibility introduced into beam-to-column joints and diagonal bracings, respectively. A bilinear axial force-axial deformation relation with a kinematic hardening rule is used for each truss member to simulate the nonlinear behavior of bracing connections. The model, however, cannot capture the buckling of the gusset plate. The ratio $\frac{F}{\frac{L}{EI}}$ of the rotational connection flexibility F to the beam flexibility $\frac{L}{EI}$ was computed for the WA ($\frac{F}{\frac{L}{EI}} = 5.7$) and the EP ($\frac{F}{\frac{L}{EI}} = 0.039$) beam-to-column connections. The values obtained are well within the range for which a pin-ended or a rigid connection can be assumed for the joint model (Fig. 5) (Gerstle 1988).

To check the validity of the phenomenological model in the context of frame analysis, the bracing connection model is calibrated in the fashion described above and incorporated into the static and dynamic analyses of the frame specimen using the computer program ANSR-III (Oughourlian and Powell 1982). While only a single-story has been tested by means of the pseudodynamic method (Fig. 6) using a substructuring approach, the dynamic analyses are conducted with the entire three-story, single-bay scale model. Elastic-plastic beam-column elements based on the plastic hinge concept (lumped plasticity beam-column element without stiffness degradation) are used to model the beam and column members. Four beam-column elements with distributed plasticity and nondegrading stiffness (Chen and Powell 1982) are used to discretize every bracing member in order to simulate the post-buckling hysteretic behavior. The depth of a column is simulated by a rigid beam extending from the center line to the flange of the column.

The masses are lumped at nodes 3, 7 and 9 of the frame model which are numbered in the same fashion as shown in Fig. 2. The masses at the first and second stories are assumed to be 0.204 kip-sec^2/in (35.7 kN-sec^2/m) and that at the third story is 0.08 kip-sec^2/in (14 kN-sec^2/m).

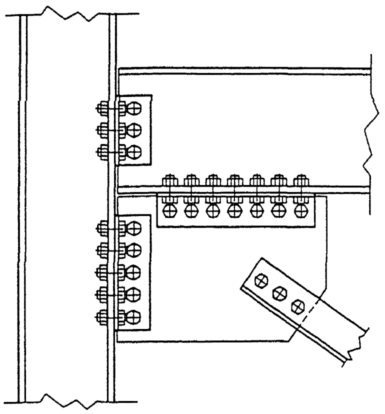

Fig. 1. Bolted beam web-to-column and bracing connections.

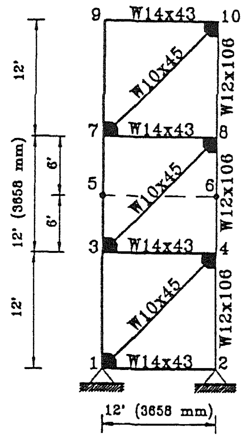

Fig. 2. Prototype structure.

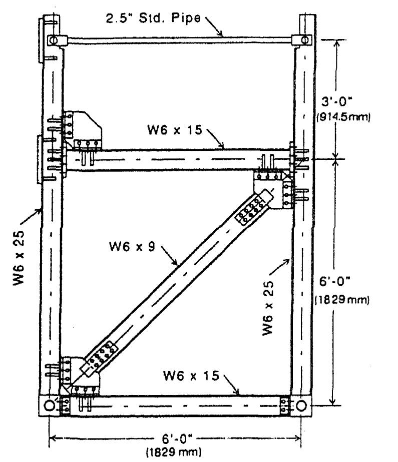

Fig. 3. Test structure.

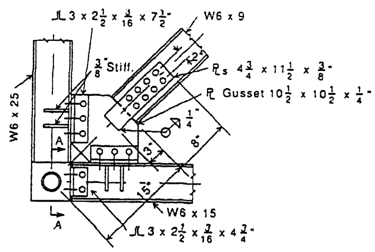

Fig. 4. Details of base connection.

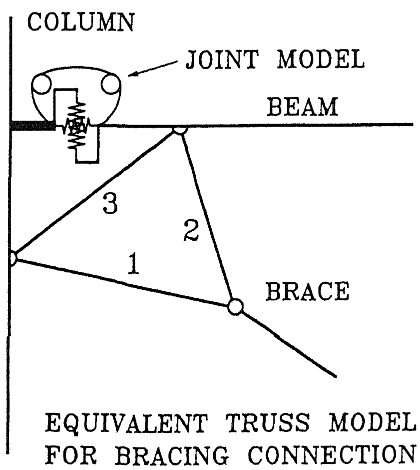

Fig. 5. Beam-to-column joint and bracing connection phenomenological models.

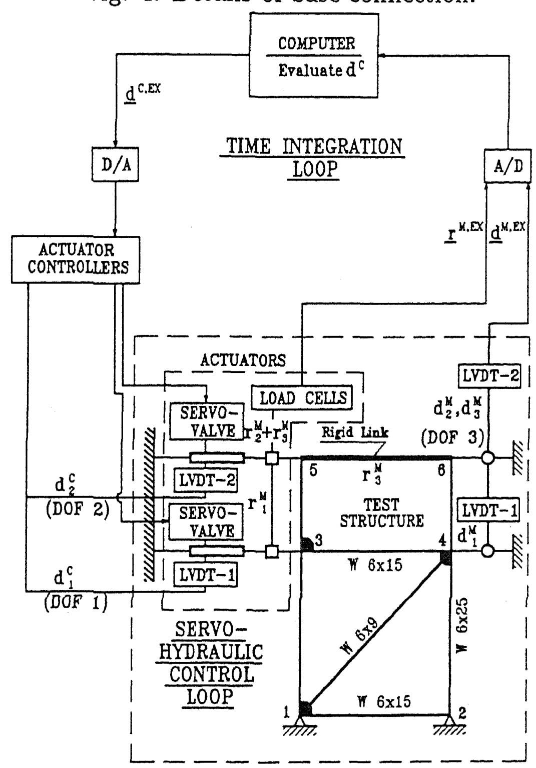

Fig. 6. Schematic of pseudodynamic test setup.

Rayleigh damping is selected and the damping ratio for the fundamental mode is based on the actual damping measured in pseudodynamic free-vibration tests.

3 TEST PROCEDURES

3.1 Quasi-static cyclic tests

To identify the lateral load vs. lateral displacement response characteristics of the braced frame, it was subjected to a series of quasi-static cyclic load reversals. The specimens were loaded with a servo-controlled electrohydraulic actuator at the top of the first story. Tests were conducted with both 2x2 and 3x3 bracing connections at small deformation levels. Four tests were performed under load control on the unbraced and braced frame in order to characterize the elastic response of both the structure and the connections. Other two tests were performed under displacement control, gradually increasing fully reversed lateral displacement cycles and imposing at each displacement level three equal amplitude cycles. The amplitude was progressively increased in steps of 0.1 inches in order to reach the yielding of the bolted gusset plate-to-member fasteners.

3.2 Pseudodynamic tests

A multilevel substructuring algorithm has been developed and implemented in a finite element-based pseudodynamic test program, in which part of a structure can be modeled in a computer with beam-column and truss elements while the remaining portion is tested experimentally. An unconditionally stable implicit direct time integration scheme is used to evaluate the dynamic response. A modified Newton-Raphson iterative approach based on the initial structural stiffness is adopted in the solution process. The initial stiffness can be estimated by means of an analytical model. The detailed test methodology and theory have been covered by Vannan (1991).

In this study, only the lowest story of the braced frame, in which damage is expected to be most severe, has been tested, while the two upper stories have been modeled in a computer. The schematic of the test setup is shown in Fig. 6. The experimental and analytical substructures have been partitioned at the mid-height of the second story, where the inflection points are assumed to be located. This assumption is not exactly true but has no significant influence. The partition line is shown in Fig. 2, where nodes 3, 5, and 6 are the interface nodes. It should be noted that the diagonal brace at the second story has been considered to be part of the analytical substructure and is, therefore, not shown in Fig. 6. The mass has been assumed to be distributed in the same fashion as in the frame analyses using the ANSR-III program.

Two actuators have been used to control the horizontal displacements at nodes 3, 5, and 6 (i.e. DOF 1, DOF 2, and DOF 3 shown in Fig. 6) of the experimental substructure. By means of a rigid link, the displacements at DOF 2 and DOF 3 (i.e., d_2^M and d_3^M) have been slaved. Even though all degrees of freedom at the nodal points have been retained, those corresponding to zero mass have not been controlled except for the interface degrees of freedom. This is identical to static condensation. The restoring forces corresponding to DOF 1, 2 and 3 have been measured by load transducers in the actuators and the rigid link. However, since the diagonal brace in the second story has been considered as part of the analytical substructure, its axial force has to be computed and added to the force measured at DOF 1 to obtain the total restoring force.

For this test series, the NS component of the 1940 El Centro ground motion record has been used. The peak ground acceleration has been scaled progressively from 0.04 g (minor level) to 0.30 g (severe level). This series provides two tests with elastic response.

Prior to these tests, pseudodynamic free-vibration tests have been conducted to measure the inherent damping in the structure in a presumably linearly elastic range.

4 COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

For conciseness, only the main results will be presented in the following. The hysteresis loops of story shear vs. story displacement obtained from the quasi-static cyclic test with 3x3 bracing connections and extended end-plate joints are shown in Fig. 7a. The inelastic hysteretic behavior observed is mainly due to the yielding of the gusset plate-to-member fasteners thus indicating that a substantial amount of energy can be dissipated in the bolted clip angles. The result of a numerical simulation using the phenomenological model for bracing connections is also plotted in Fig. 7a. It can be observed that the hysteretic behavior of the test frame is well captured. The analytical model overestimates the energy dissipation by about 11.3%. In order to explore the effects of bracing connections on the frame response, a rigid braced frame analysis neglecting the bracing connection flexibilities has been performed. The comparison of the analytical and experimental results (Fig. 7b) shows that the rigid connection analysis overestimates the stiffness and the strength of the test frame both in the tensile brace regime (pull) and in the compressive brace regime (push).

The displacement histories vs. the compressed times obtained from the pseudodynamic tests with 0.15 g, 0.20 g and 0.30 g peak ground accelerations and with 3x3 bracing connections are shown in Figures 8 to 10 respectively. In the same figures, the analysis results conducted with the bracing connection model are illustrated as well. The equivalent viscous damping measured from a prior pseudodynamic free-vibration test, and then used by the numerical simulations is 2.3%. As indicated in Fig. 8, the experimental frame response which is characterized by the yielding in the gusset plate-to-member fasteners correlates satisfactorily with the numerical simulation.

The pseudodynamic test results with 0.2 g and 0.3 g peak ground acceleration (Figs. 9 and 10) resulted in severe tensile yielding of the gusset plate-to-member fasteners and out-of-plane buckling of the diagonal brace. Therefore, the displacement time histories show a significant period elongation when compared to the test result in Fig. 8. The overall responses in the simulations are similar to the test results, but the analyses indicate smaller displacements than those obtained in the tests. The observed differences are largely due to the analytical brace model which tends to overestimates the material tangent modulus and the energy dissipation.

The effective length coefficient K was computed for the diagonal braces, based on the brace buckling strength P_{cr} of 79.6 kips (354.1 kN) and 79.9 kips (355.4 kN) measured in two tests, and the average yield stress of 47.7 ksi (328.9 MPa). Using the clear brace length (70.2 inches (1783 mm)) and the AISC-LRFD (AISC 1986) buckling strength formula, a K factor of 1 is obtained.

In summary, the frame has exhibited a relatively ductile behavior. Both the bracing connections and the diagonal brace are able to dissipate a substantial amount of energy. Nevertheless, the buckling of the diagonal brace has led to a substantial drop of lateral resistance. Lastly, the maximum story displacement in the three PSD tests (0.15 g, 0.20 g and 0.30 g peak ground accelerations) and the maximum computed displacements from rigid frame analyses in which the bracing connection flexibility is excluded are compared in Fig. 11. The analytical response appears to be 33% smaller than the test response because of the neglecting of bracing connection flexibility.

5 CONCLUSIONS

In this paper, the seismic response of a flexibly connected steel braced frame is studied by means of pseudodynamic testing and numerical simulation. The experimental study has been based on a newly developed substructuring approach and the analyses have been conducted with a phenomenological model that is capable of capturing the linear and nonlinear response of bracing connections. The conclusions reached are as follows:

1. The frame designed in accordance with the proposed AISC Seismic Provisions (AISC 1990) has exhibited a good ductile behavior. The moderately bolted bracing connections are capable to dissipate a substantial amount of energy.
2. The K factor for the out-of-plane buckling of the diagonal brace connected with bolted gusset plates has been found to be close to unity, which corresponds to a pinned-pinned end condition.
3. Satisfactory correlations have been obtained between the test results and numerical simulations using the phenomenological model for the bracing connections.
4. The rigid frame analysis of the examined structure gives safe results. But for sway-critical frames, the effects of the bracing connection flexibility should be considered, since moderately bolted bracing connections can contribute considerably to sway under seismic loads.
5. The results indicate that pseudodynamic testing is a viable alternative to testing a complete structure.

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REFERENCES

AISC 1986. *Manual of Steel Construction - LRFD* AISC, Chicago, IL.

AISC 1990. *Seismic Provisions for structural steel buildings - LRFD.* AISC, Chicago, IL.

Bursi, O.S. 1992a. Elastic-plastic modeling of structural fasteners for steel bracing connections. *J. Construct. Steel Research* (submitted).

Bursi, O.S. Gerstle, K.G., Sigfusdottir A. & Zitur J. 1992b. Behavior and analysis of bracing connections for steel frames. *J. Construct. Steel Research* (submitted).

Bursi, O.S. & Gerstle K.H. 1992c. Analysis of flexibly-connected braced steel frames. *J. Construct. Steel Research* (submitted).

Chen, P.F. & Powell G.H. 1982. Generalized plastic hinge concepts for 3D beam-column elements. Report *No. UCB/EERC-82/20*, University of California at Berkeley, CA.

Gerstle, K.H. 1988. Effect of connections on frames. *J. Construct. Steel Research* 10:241-267

Gross, J.L. 1989. Experimental Study of Gussetted Connections. *Proc. National Steel Construction Conference, Nashville, AISC*: 11.1-11.22.

Oughourlian, C.V. & Powell G.H. 1982. ANSR-III - General purpose computer program for nonlinear structural analysis. Report *No. UCB/EERC-82/21*, University of California at Berkeley, CA.

UBC 1988. *Uniform building code.* International Conference of Building Officials, Whittier, CA.

Vannan, M.T. 1991. The pseudodynamic test method with substructuring applications. *Ph. D. Thesis*, CEAE Department, University of Colorado at Boulder, CO.

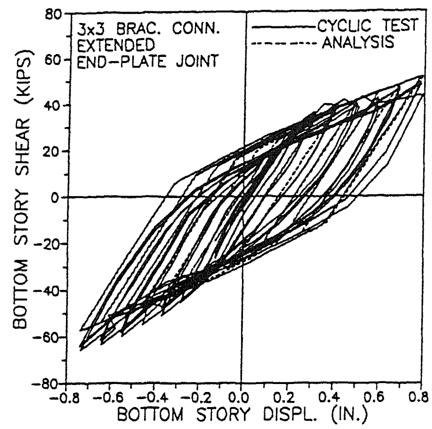

Fig. 7a. Hysteretic loops from test and analysis including the bracing connection flexibility.

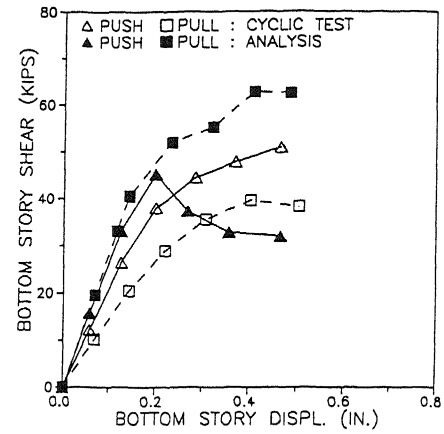

Fig. 7b. Envelopes from test and analysis neglecting the bracing connection flexibility.

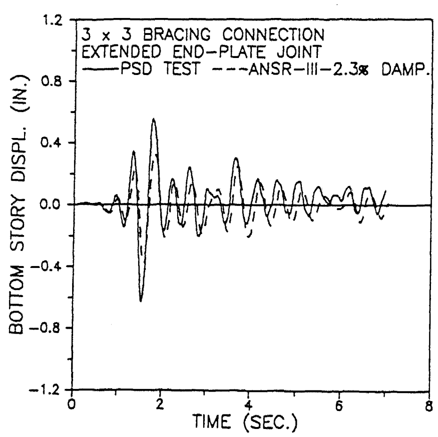

Fig. 8. Story displacement histories - El Centro 0.15 g peak acceleration.

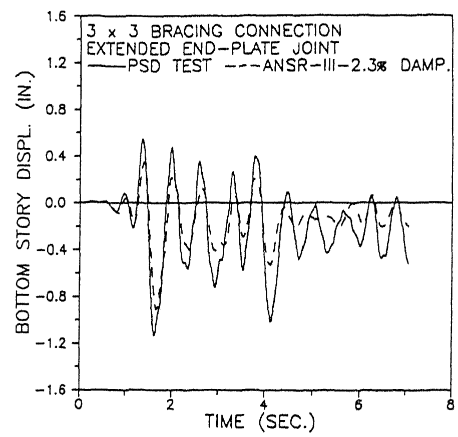

Fig. 9. Story displacement histories - El Centro 0.20 g peak acceleration.

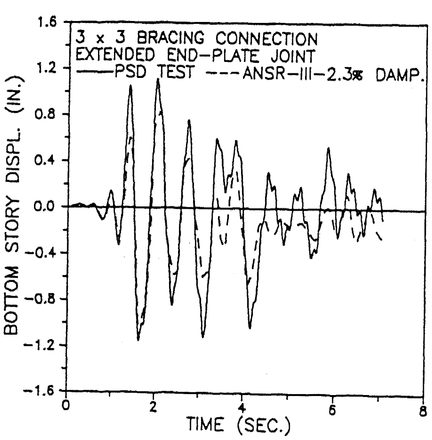

Fig. 10. Story displacement histories - El Centro 0.30 g peak acceleration.

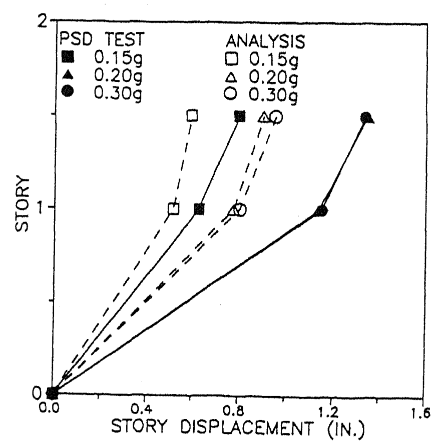

Fig. 11. Envelopes from test and analysis neglecting the bracing connection flexibility.