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ASSESSMENT OF SEISMIC PERFORMANCE OF A CODE DESIGNED REINFORCED CONCRETE BUILDING

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ABSTRACT

This paper studies the inelastic seismic performance of a 12-story reinforced concrete building. The building utilizes a structural system with moment-resisting frames in the longitudinal direction and a dual structural system consisting of coupled shearwalls and moment-resisting frames in the transverse direction. The frame elements, the shearwalls and the coupling beams are sized and detailed on the basis of the current UBC for Zone 4 (regions of high seismicity). The global and local inelastic behavior of the building in the two orthogonal directions is studied under several earthquake ground motions. The nonlinear concrete behavior, with stiffness degradation and strength loss caused by cracking, crushing of concrete and yielding of steel is simulated by using the Fiber Beam-Column element of the DRAIN-2D program. Pushover analysis is used to determine the global ductility of the structure. The study indicates that the design strength may be inadequate for some critical earthquakes. Weak coupling between walls induces large ductility demand in them, which can be directly reduced by increasing the wall strength. Coupled walls are more efficient and economical than isolated or weakly coupled walls. Optimum values of beam and wall stiffness and strength can be chosen to minimize ductility demand on the walls of a coupled wall system.

KEYWORDS

Reinforced Concrete Building, Seismic Design, Shearwall, Coupled Wall, Seismic Performance, Code Design, Ductility, Wall Design.

INTRODUCTION

The Uniform Building Code (UBC) provides analysis, design and detailing requirements for buildings subject to earthquake ground motions in the United States. The intent of the document is to minimize the life safety hazard, avoid catastrophic failure of structures, and, in the case of essential facilities, to maintain continued functionality after a major earthquake. In order to avoid catastrophic structural failure, the code recognize the ability of carefully detailed concrete structural components to dissipate significant earthquake energy through hysteresis. The design philosophy thus inherently allows damage such as cracking, crushing of concrete and yielding of steel at preferred locations, to avoid distress at structurally critical locations. Though the intent of the code is simple, the actual behavior of a structure and its members could deviate substantially from the intended behavior because of complexity of the actual ground motion, complexity of the structure, manner and extent of strength deterioration and stiffness degradation of members and the redistribution of forces in the inelastic structure. These aspects need to be studied by considering the inelastic behavior of concrete components under probable ground motions, to assess the adequacy of code provisions and identify possible future improvements.

The performance of coupled walls is critical to building safety. Both weak and very strong coupling between walls can induce large ductility demands in the walls. Optimum coupling between walls (to minimize wall ductility) depends upon the beam-to-wall stiffness and strength ratios, and the yield strength of the walls. These parameters should be considered in order to design an efficient and economical coupled wall system.

The paper investigates some of these issues and highlights the important parameters that should be considered in the design process.

EXAMPLE BUILDING

Figure 1 shows the plan and elevation of a 12-story building used for this investigation. The building utilizes a structural system with moment-resisting frames in the longitudinal direction and a dual structural system consisting of shearwalls and moment-resisting frames in the transverse direction. The frame elements, the shearwalls and the coupling beams are sized and detailed on the bases of the current UBC Zone 4 (regions of high seismicity) requirements. All beams are sized at 20x24 in. and all columns at 24x24 in. The 1st and 2nd story columns are of 5 ksi concrete and the remaining columns and all beams are of 4 ksi concrete.

The elastic building has a fundamental period of 1.8 seconds in the longitudinal direction and 0.83 second in the transverse direction. The building is assumed to have 5% damping for its first three deformation modes. The strain-hardening stiffness is assumed to be 5% of the gross uncracked elastic stiffness. Gross section properties are used throughout the analyses, to be consistent with design that was based on period computations using gross section properties.

The analyses of the building are carried out separately in the longitudinal and the transverse directions using the DRAIN-2DX program [Prakash et. al]. Beams are idealized with fiber hinges at their ends. The effects of stiffness degradation, strength loss and pinching of concrete are included in the analyses. The gravity loads due to dead and contributing live load are input for columns and shearwalls at each floor level in order to simulate the beam-column (P-M) type behavior. The analysis assumes rigid floor diaphragms, with each node having three degrees of freedom.

EARTHQUAKE GROUND MOTIONS

The ground motion records used for the analyses include the El Centro (NS) component (1940), an artificial earthquake whose spectrum characteristics are compatible with the SEAOC (1990) design spectrum, the Hachinohe earthquake (1968), the Parkfield earthquake (1966), and the Northridge earthquake components (1994) at Sylmar county and Newhall stations. The Parkfield earthquake was selected to represent a harmonic type motion critical for long-period structures, and the Northridge earthquake records were selected to study the near-field ground excitation effect [Naeim, 1995]. The eight records selected were scaled up to a spectral intensity of 1.5 times that of the El-Centro (NS) component and 1.5 times that of the Hachinohe earthquake, which are also being used for level-2 earthquake design in Japan. The spectral characteristics of the scaled earthquakes for 5% viscous damping are shown in Fig. 2.

PERFORMANCE OF MOMENT RESISTING FRAME

Inelastic Dynamic Response

The moment resisting frame in the longitudinal direction was analyzed for the selected ground motions (Fig. 2). The results for 4 earthquakes, El Centro, Hachinohe, Northridge at Sylmar county (EW), and Parkfield earthquakes are shown here. Fig. 3(a) showing the roof deformation history indicates that the building is likely to develop a biased motion, particularly more severe under the Parkfield earthquake. Fig. 3(b) shows that the Parkfield and Northridge (Sylmar-EW) earthquakes produce critical and biased drift concentrations. The maximum displacement reaches about 16 in., while the interstory drifts are more than 2% under the Parkfield earthquake. The results indicate that the Parkfield and the Northridge earthquakes have the potential of causing more critical damage than the El Centro and Hachinohe earthquakes commonly used for similar analyses. The analyses indicated that stiffness degradation, strength loss and pinching of well designed and detailed concrete components do not critically change the global response of the building.

Global Ductility by Pushover Analysis

The global yield displacement of the moment resisting frame was obtained by pushover analysis as shown in Fig. 3(c) Considering the global yield displacement of 4 in. (Fig. 3(c)) and the maximum roof displacements of 10 in., 15 in., 16 in., and 10 in., global displacement ductilities of 2.5, 3.75, 4, and 2.5 are obtained under the El Centro, Hachinohe, Parkfield, and Northridge (Sylmar-EW) earthquake components, respectively. It may be noted that these ductility values could actually be lower if effective stiffness properties rather than gross section properties were used. The ductility values seem to be within the maximum anticipated ductility of a moment resisting frame under such design earthquakes.

PERFORMANCE OF DUAL SHEAR WALL-FRAME SYSTEM

Inelastic Dynamic Response

The dual shear wall-moment resisting frame system in the transverse direction (Fig. 1) is analyzed for the selected ground motion records of Fig. 2. The results for El Centro, Hachinohe, Northridge at Sylmar county (EW), and Northridge at Newhall station (NS) ground motions are shown here. Figs. 4(a) and (b) compare the building responses in the transverse direction under the four selected earthquakes. The Northridge earthquake components induce a severe deformation pulse resulting in biased motion that could lead to instability or collapse of the building. The maximum displacement reaches about 12 in., while the maximum drift ratio is about 0.7% under the Northridge (Newhall-NS) earthquake component. Figs. 4(a) and (b) show that shearwalls reduce the displacement and drift envelopes of the building in the transverse direction.

Global Ductility by Pushover Analysis

The global displacement ductility of the building in the transverse direction is obtained by using the yield displacement determined through pushover analysis. Fig. 4(c) shows that the yield displacement of the wall-frame structure is very similar to that of the wall system alone, due to the large stiffness of the walls. Considering the global yield displacement of about 2 in. (Fig. 4(c)) and the maximum displacements of 6 in., 7 in., 10 in., and 12 in. under the El Centro, Hachinohe, Northridge (Sylmar-EW), and Northridge (Newhall-NS) motions, respectively, global ductilities of 3, 3.5, 5, and 6 result under the four selected records. Note that the actual ductility values would be lower, because of the effective stiffness properties of the concrete members. The ductilities, however, still seem to be excessive, especially in the case of the Northridge earthquake. It may be noted that this is despite the wall overstrength of about 2.1, which results with 0.8% reinforcement for the wall section under the design axial load. The overstrength factor is defined as the ratio of actual wall strength to that required by the code-prescribed seismic forces. The walls usually have substantial overstrength since wall location and proportions are more often governed by functional rather than structural considerations.

Ductility of Weakly Coupled Walls

The large ductility demand on the walls is attributed to weak coupling between the walls, due to relatively long coupling beams. In such situations, walls behave primarily as isolated cantilever walls. The ductility demand on such walls can be directly reduced by increasing their strength. This is shown in Fig. 5, which compares the response history and drift envelope of the building with the original wall strength (0.8% steel) with those of the same building with strengthened walls (1.5% steel) under the Northridge (Newhall-NS) components which produced the critical response. The strengthened wall (overstrength = 2.85) reduces the maximum displacement to 8 in. and maximum global displacement ductility to 3.2, while eliminating the biased response of the building (Fig. 5).

PERFORMANCE OF COUPLED WALL SYSTEM

Coupled Walls

The wall system in the transverse direction is redesigned as a coupled wall system by reducing the wall section and coupling beam length (Fig. 1(c)) and the increasing the coupling beam depth. Coupled walls are more efficient and economical for lateral load resistance, as compared to isolated walls. Short coupling beams between walls substantially increase the stiffness of the system. The larger strength and rotations of the coupling beams increase the energy dissipation capacity of the building. The increase in coupling between walls however, also increases the magnitude of axial forces (compression and tension) in the walls. This complicates the post-yield behavior and the ductility demand on the walls. Thus, increase in coupling beyond a certain limit may not be efficient and would only increase the ductility demand on the walls. An efficient coupled wall system will have optimal coupling between walls in terms of beam to wall stiffness and strength ratios, resulting in minimum wall ductility [Saatcioglu, Park and Paulay].

Design of Coupled Wall System

A coupled wall system (Fig. 1(c)), having its stiffness and period similar to those of the original wall-frame system, is designed with a beam-to-wall stiffness ratio of 0.04. This optimum stiffness ratio is obtained by reducing the wall section and increasing the beam to wall stiffness ratio [Saatcioglu, 1981]. The resulting coupling arm is 7 ft. long and 32 in. deep. The resulting wall section has only 34% of the gross inertia of the original wall section. The design moments from lateral load analysis are substantially lower for the coupled walls (Fig. 6(a)). The shear forces (Fig. 6(b)) however, remain similar, whereas the maximum total axial forces (Fig. 6(c) increase by about 60%. Consequently, the wall section chosen results in an overstrength of 3.25 with 0.8% reinforcement.

Inelastic Response

Pushover analysis of the coupled wall system (Fig. 4(c)) showed that the stiffness and the yield displacement is similar to those of the original wall-frame system. Comparing the roof displacement history and the drift response envelope of the coupled wall system with those of the original wall-frame system under the Northridge (Newhall-NS) ground motion, Fig. 7 indicates that the deformation and ductility demands of the two systems are similar, as anticipated [Saatcioglu, 1981]. The coupled wall system results in about 25% saving in concrete and 30% saving in steel while producing responses similar to those of the original wall-frame design. The maximum moments, shears, and axial forces induced by the Northridge (Newhall-360) earthquake were also compared with the design forces of Fig. 6. Though the moments induced were consistent with the overstrength of the walls, the induced shear forces were larger (not consistent with moments) in the coupled walls, which is attributed to higher mode effect [Ghosh & Markevicius].

Ductility of Coupled Wall System

The ratio of coupling beam strength to wall strength may be varied, to result in minimal wall ductility under a given beam-to-wall stiffness ratio. Saatcioglu (1981) indicated that wall ductility can be reduced by increasing wall strength and selecting an optimal beam-to-wall strength ratio for that wall strength. Coupled wall systems with two wall strength levels of 400,000 in.-kip (0.8% steel) and 550,000 in.-kip (1.8% steel) were analyzed under the Northridge (Newhall-NS) earthquake. The latter represents a strengthened wall for reducing wall ductility. Beam-to-wall strength ratios of 2%, 1%, 0.5%, and 0.2% were considered for each level of wall strength.

Fig. 8(a) and (b) show that the maximum roof displacements vary slightly with the selected beam-to-wall strength ratios. For the wall strength of 400,000 in.-kip, the displacement ductilities of 6, 5.5, 5.5, and 5.7 result, while for 550,000 in.-kip of wall strength, the ductility values reduce to 3.5, 4.4, 3.8, and 3.5 for 2%, 1%, 0.5% and 0.2% beam-to-wall strength ratios, respectively. The results indicate that for each level of wall strength, an optimum range of beam-to-wall strength ratios exists, that would result in minimum wall ductility. The maximum displacements do not appear to be overly sensitive for this range of beam-to-wall strength ratios. In this range, wall strength thus has a more pronounced effect on wall ductility. Similar results were obtained by Saatcioglu (1981).

From this limited study and that carried out by Saatcioglu (1981), it appears that the design of coupled walls can be optimized to yield efficient and economical lateral load resisting systems. More systematic and complete parameteric studies, considering different periods, wall strengths, and beam-to-wall stiffness and strength ratios will, however, be required to establish a definite criteria for efficient design of coupled walls.

CONCLUDING REMARKS

The global ductility of the code-designed moment resisting frame seems to be well within anticipated limits under the selected earthquake intensity. The Parkfield and Northridge earthquakes, however, produce biased local and global deformation responses in the building, indicating that similar earthquakes could be critical for this building. The Northridge earthquake induced critical wall ductility and biased response of the wall-frame structure. The weak coupling between the walls in the transverse direction could induce large ductility demands in them. Optimum coupling between walls in terms of coupling beam-to-wall stiffness and strength ratios provides an efficient and economical lateral load resisting system. The wall strength of an isolated or a well designed coupled wall system has a pronounced effect on its ductility. The results from this limited study indicate that code design forces may need to be reviewed to safeguard against possible structural instability produced by some critical earthquake characteristics.

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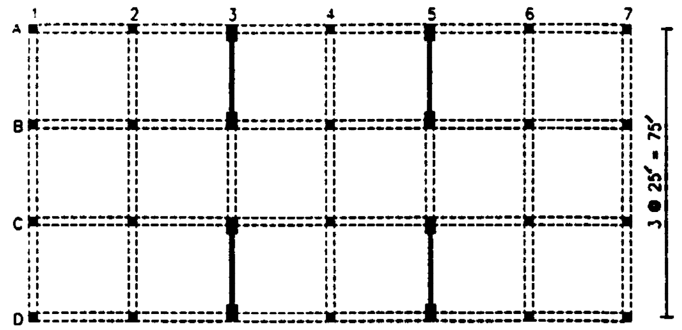
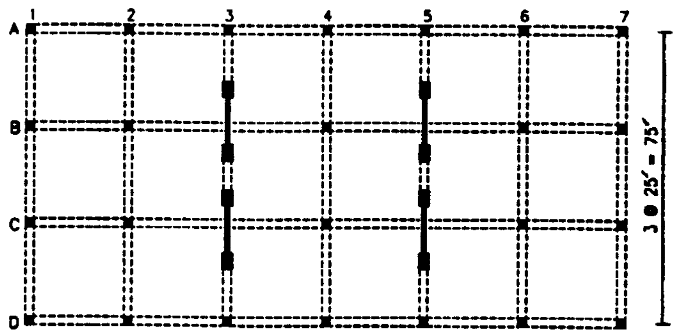
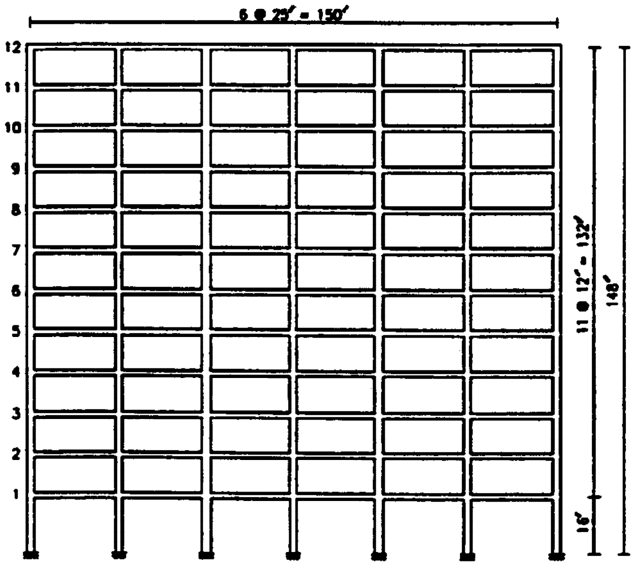


(a) Plan View



(c) New Configuration (coupled wall)



(b) Elevation

Fig. 1. Example Building.

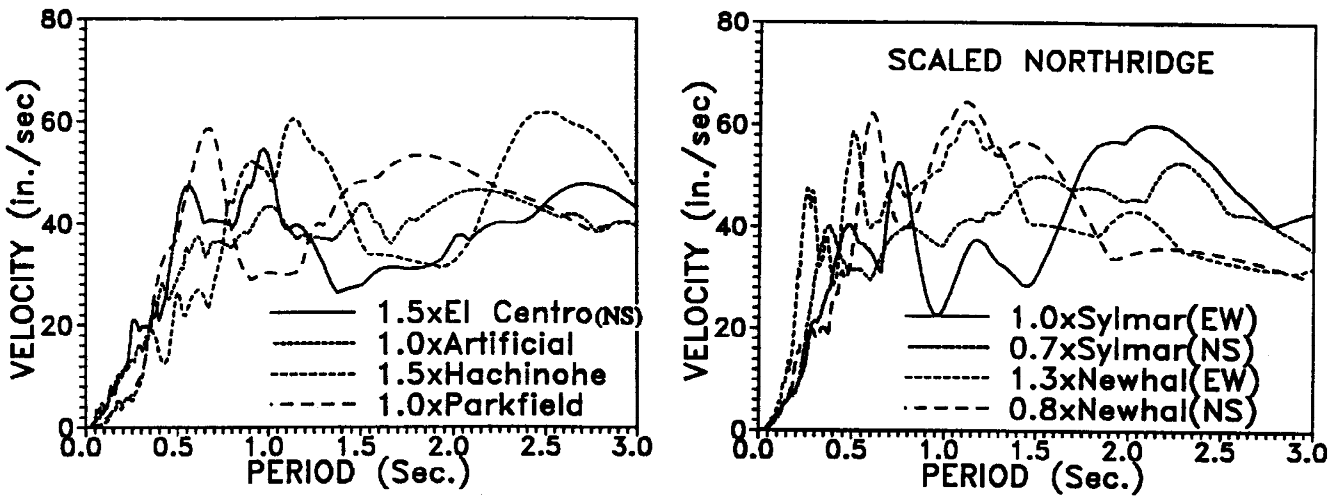
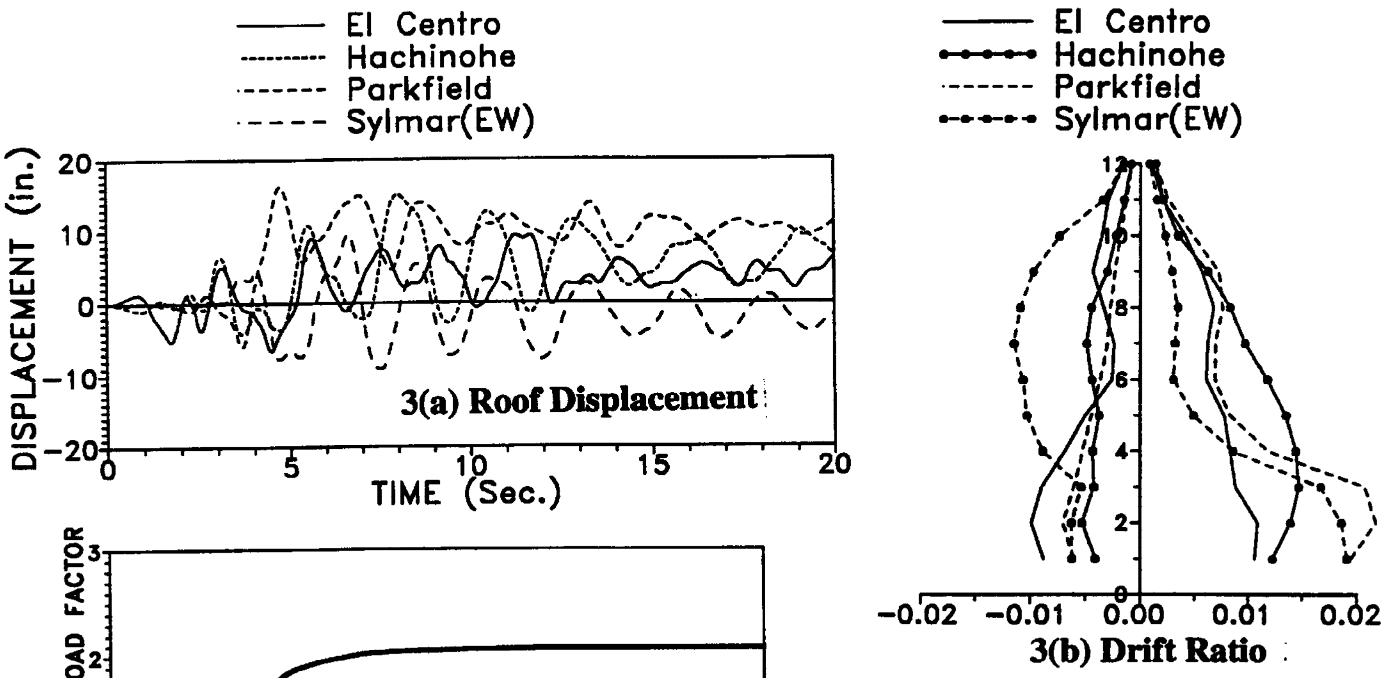


Fig. 2. Spectral Characteristics of Earthquakes. (5% Damping Ratio)



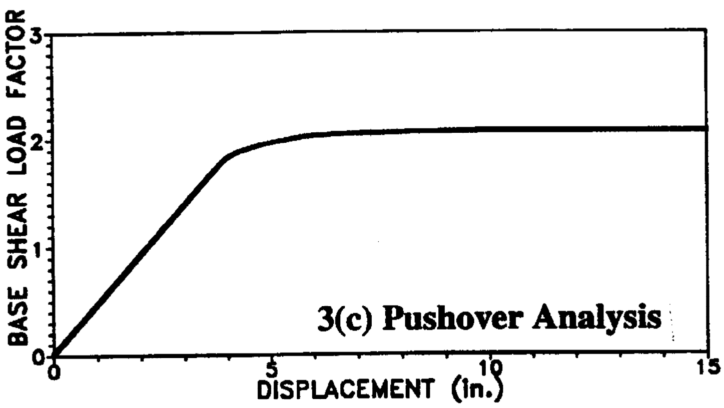
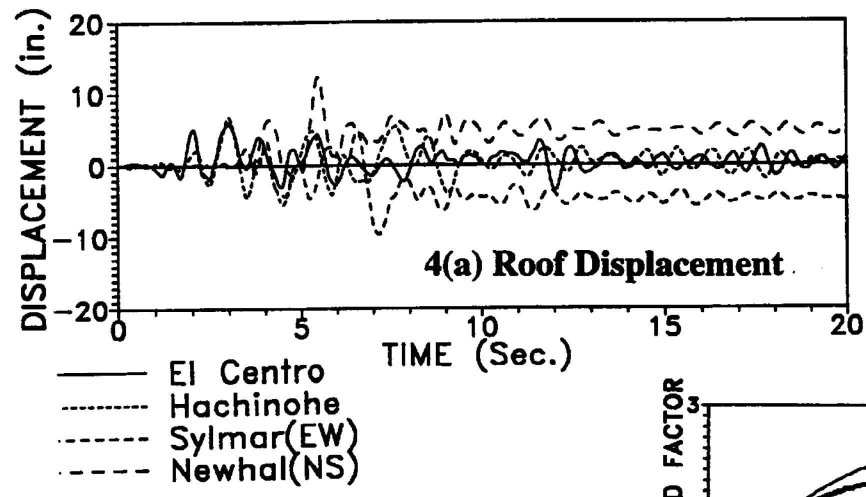


Fig. 3. Inelastic Response of Moment Resisting Frame.



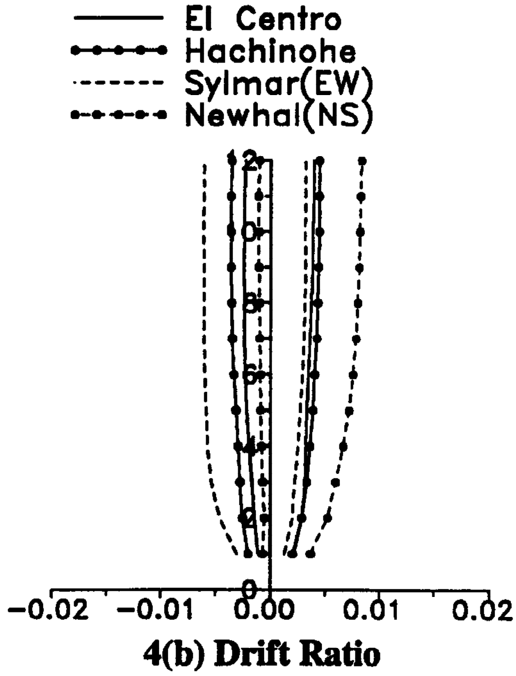
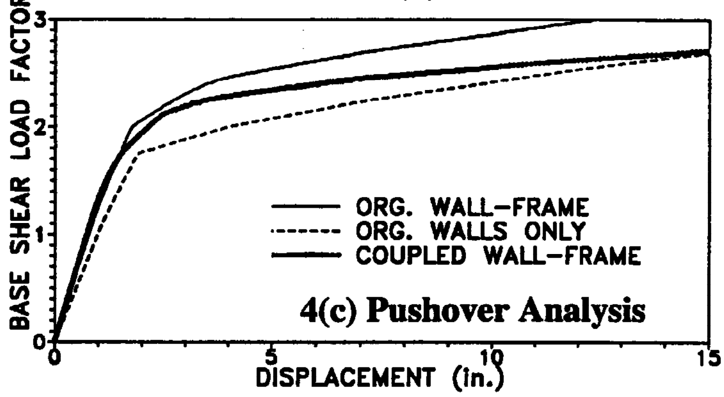


Fig. 4. Inelastic Response of Wall-Frame System.



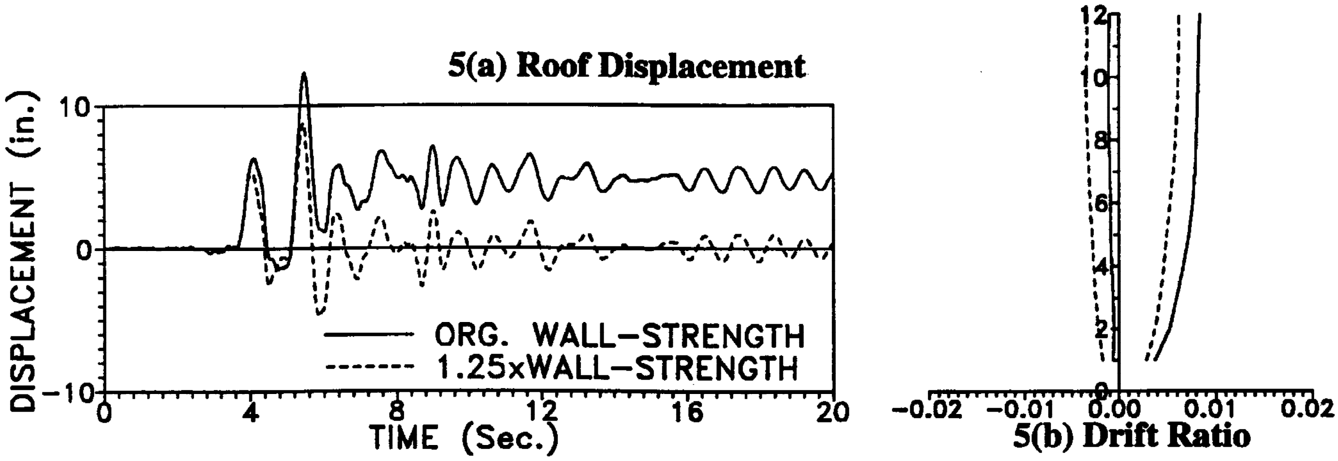


Fig. 5. Inelastic Response of Strengthened Wall-Frame System.

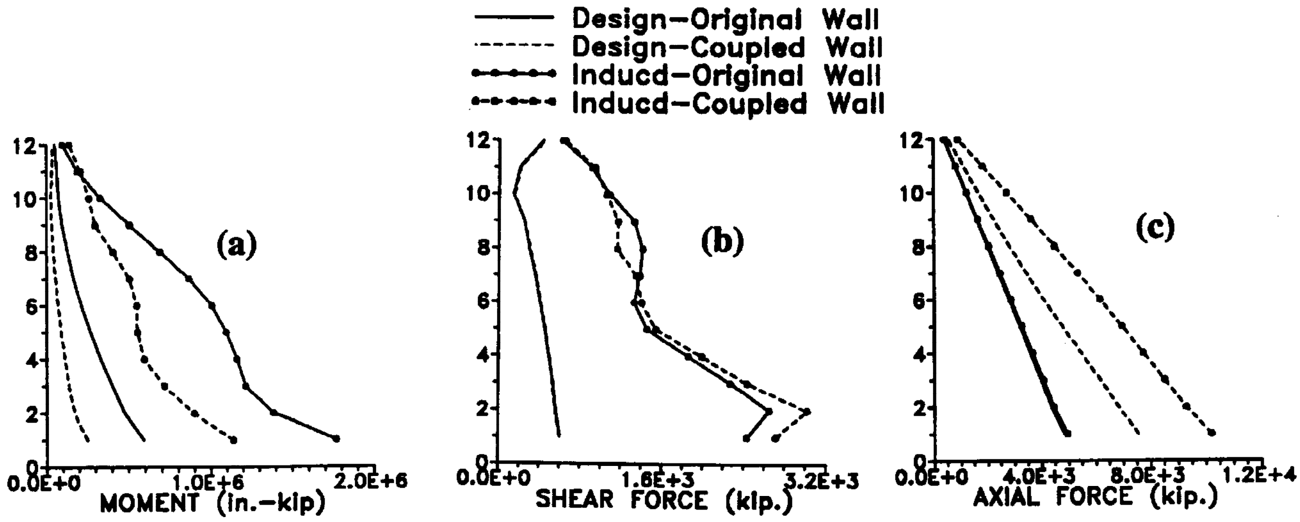


Fig. 6. Design Force Comparison of Weakly and Strongly Coupled Wall System. (2-Lumped Walls)

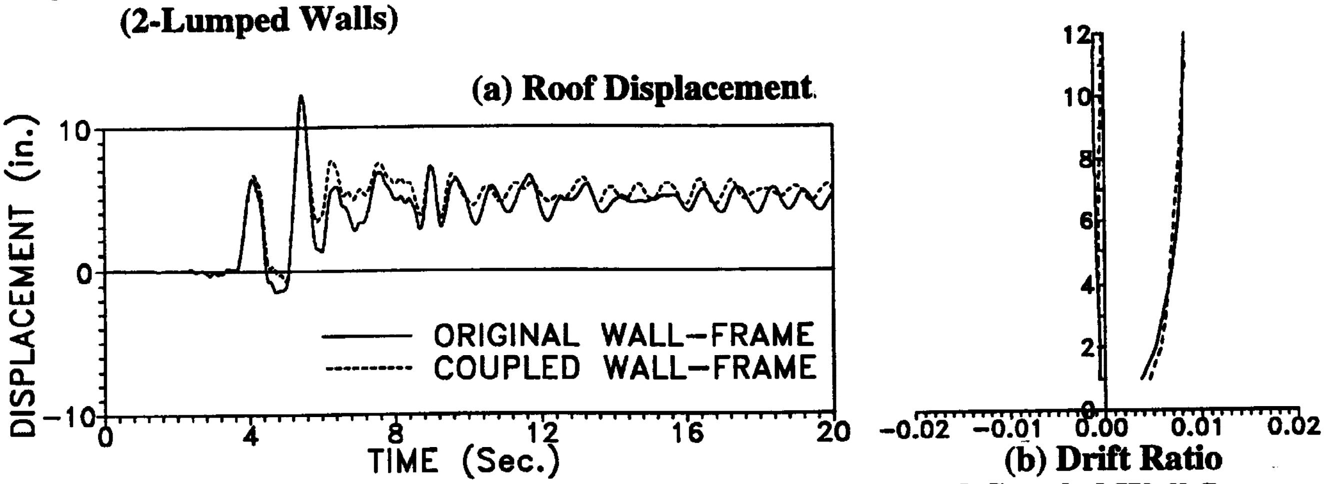


Fig. 7. Response Comparison of Wall-Frame and Coupled Wall System.

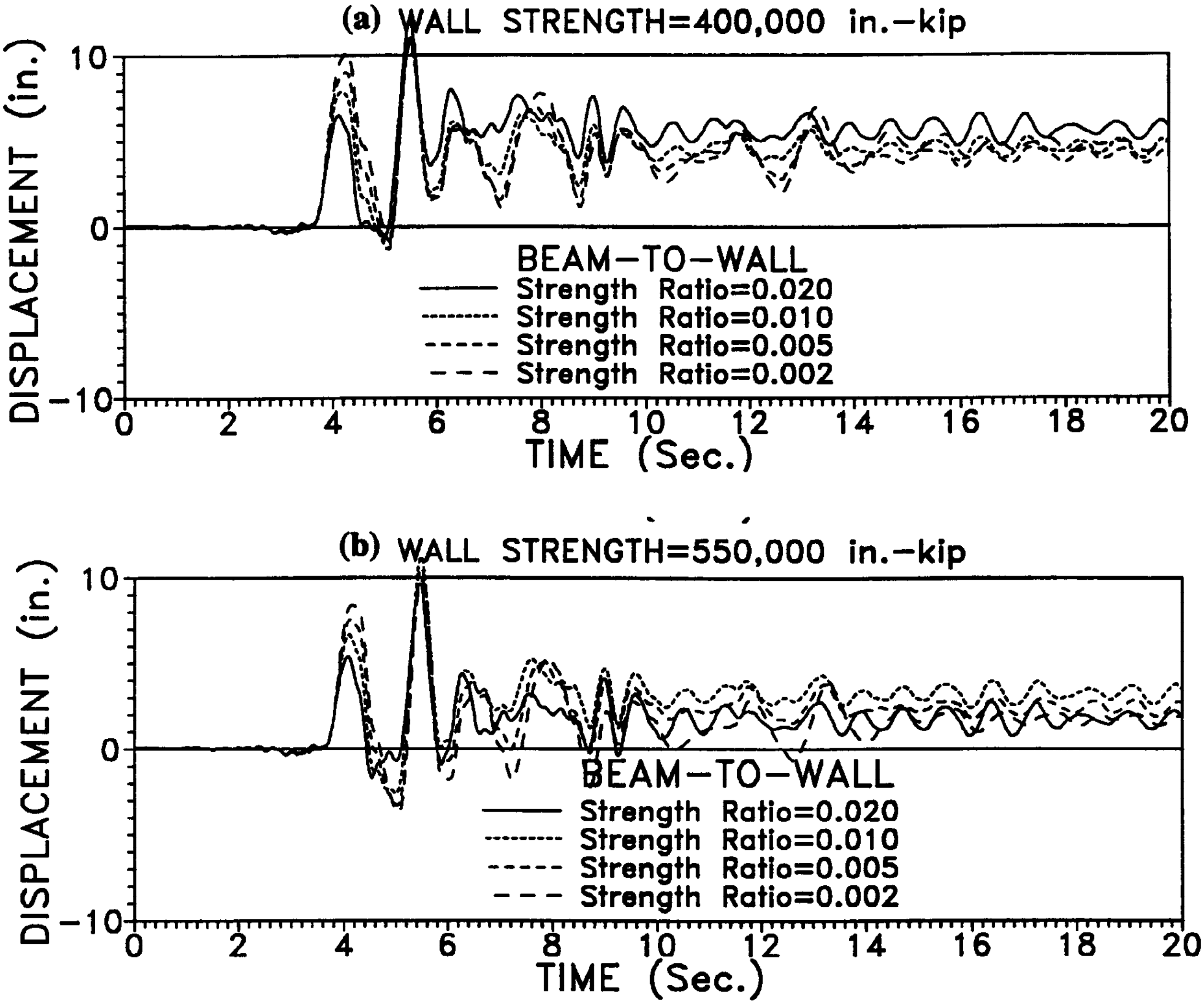


Fig. 8. Inelastic Response of Coupled Wall System.

SI CONVERSION

1 ft = 0.3048 m
1 in. = 25.4 mm
1 kip = 4.448 kN