ANALYTICAL STUDIES IN SUPPORT OF THE 1994 NEHRP PROVISIONS FOR NONSTRUCTURAL COMPONENTS

Mark E. Schroeder[1] and Robert E. Bachman[2]

Abstract

An analytical investigation was performed to evaluate the effects of primary structure period, primary structure inelastic behavior, secondary system period, and secondary system location on secondary system seismic response. This provided input to support the development of the proposed 1994 NEHRP provisions for nonstructural components. To validate the proposed provisions, a representative structure was evaluated using a nonlinear dynamic analysis computer program. By varying several key input parameters, a wide range of response possibilities were accounted for. In particular, the variation of nonstructural component response at different height levels throughout the structure was emphasized and evaluated. These results and other data considered for the development of the equations were summarized and discussed.

Introduction

Nonstructural systems, as defined by the National Earthquake Hazards Reduction Program (NEHRP) provisions, consist of architectural, mechanical and electrical components (FEMA, 1991). They include non-bearing walls, exterior cladding, suspended ceilings, ventilation systems, mechanical equipment, elevators, electrical equipment, and anything else permanently mounted to the structure that is not meant to assist in its primary load resisting functions. It has been observed that the equations used to establish seismic design loads on these elements depend on the particular code from which they come. The NEHRP provisions consider a wider range of structure and component types than the more limited requirements of the Uniform Building Code (UBC) (ICBO, 1991). However, the current 1991 NEHRP provisions do not consider soil effects, location in structure, and reductions due to inelastic structural behavior. A comparison between these and other codes has shown significant differences (Bachman, 1994).

[1] Engineer, Fluor Daniel, Inc., 3333 Michelson Drive, Irvine, CA 92730
[2] Manager, Civil/Structural Engineering, Fluor Daniel, Inc., 3333 Michelson Drive, Irvine, CA 92730

The accuracy of static methods for the prediction of dynamic loads is limited, but suitable for most structural engineering needs. To develop rational static equations, appropriate dynamic behavior must be considered. Many design codes assume that the behavior of nonstructural components in buildings is independent of the primary structural behavior. In a dynamic analysis, the behavior of the primary system can be incorporated through the use of in-structure response spectra. However, a decoupled analysis of this kind often ignores dynamic interaction between the primary and secondary systems. This interaction may be significant in the resonant frequencies if the mass of the secondary system is significant relative to the mass of its supporting system. In this investigation, it was assumed that the interaction effect was small.

To incorporate inelastic structural response, the Drain-2DX nonlinear computer program was used (Powell, 1992). Comparisons between a relatively stiff and flexible frame along with elastic and inelastic response were investigated. The properties of the secondary systems were selected so that a full range of dynamic behavior could be modeled in the horizontal direction. Under these assumptions, general response trends with height were investigated. Maximum response was not emphasized. When the period of the structure and component reside in the peak spectral range, higher amplifications may result. This is especially true for structures in which one mode represents the entire response. Simple models in addition to a 7 story building were evaluated for the selected input motions to examine this behavior.

Model Development

Structural systems come in many shapes, sizes and complexities. However, typical building codes have only one set of force equations to describe the behavior of them all. Therefore, these equations must be versatile to accommodate a wide range of behavior, yet simple to use. To capture the behavior of nonstructural components at various locations within a system, a 7 story steel building was modeled. This building was designed according to the "Recommended Lateral Force Requirements and Commentary (Blue Book)", by the Structural Engineers Association of California (SEAOC) (SEAOC, 1990). The selected example is found in a publication by the Steel Committee of California to assist engineers to properly interpret the SEAOC code provisions (Becker, 1988). The building has a braced frame system in one direction and a moment resisting frame system in the other. These frames represent moderately stiff and moderately flexible structural systems, respectively.

The example building was designed for a ground acceleration of 0.4g and was assumed to lie on S_2 soil. The floors and roof are 76mm (3") metal deck with 83mm (3-1/4") lightweight concrete fill. Typical story height is 3.51m (11'- 6"). The lateral load resisting frames lie on the perimeter of the building, allowing two frames for each direction. Since inelastic behavior was examined extensively, member capacity was realistically defined. Plastic moments were assigned to all of the beams and columns. The buckling load of the bracing members was taken as the Euler buckling load. All primary members were assigned a strain hardening ratio of 5% and a damping ratio of 5%.

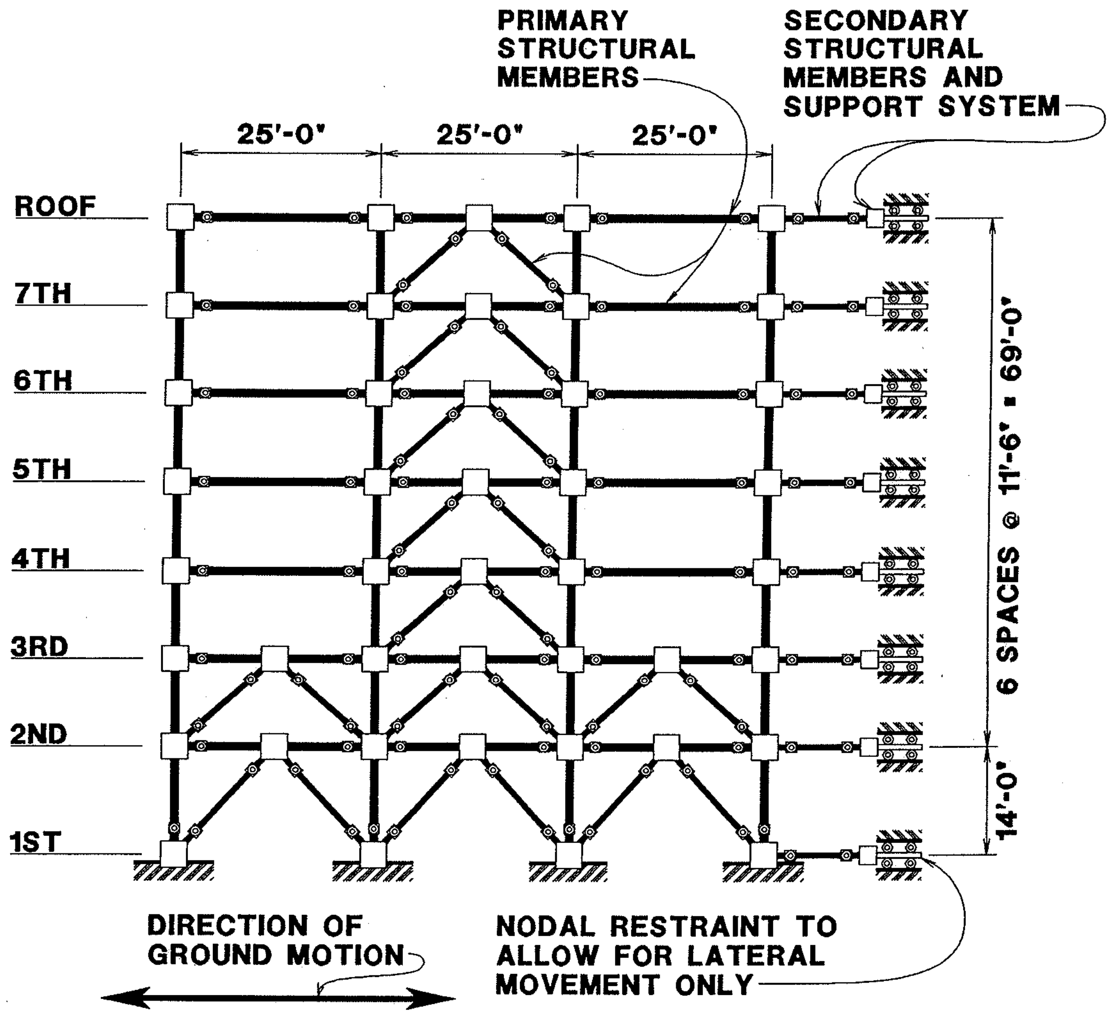

Figure 1. Braced frame model

Strength design controlled the columns in the braced frame, while allowable stress design controlled the girders and bracing members. For the design loads, stress levels in this frame ranged from 50% to 95% of allowable. The analytical computer model of the braced frame is shown in Figure 1. This structure has a fundamental period of 0.78 seconds. Its design is based on a base shear of 0.112W (W = structure weight). Drift limitations governed the selection of almost all member sizes in the moment resisting frame. A strong column, weak girder approach was also implemented. These design requirements produced stress levels from 41% to 64% of allowable. The model of the moment resisting frame is shown in Figure 2. Its fundamental period is 1.93 seconds and its design base shear is 0.047W for stress and 0.034W for drift.

Nonstructural Components

There are many factors to consider in the design of nonstructural components. Architectural, mechanical, and electrical components have different properties that must be

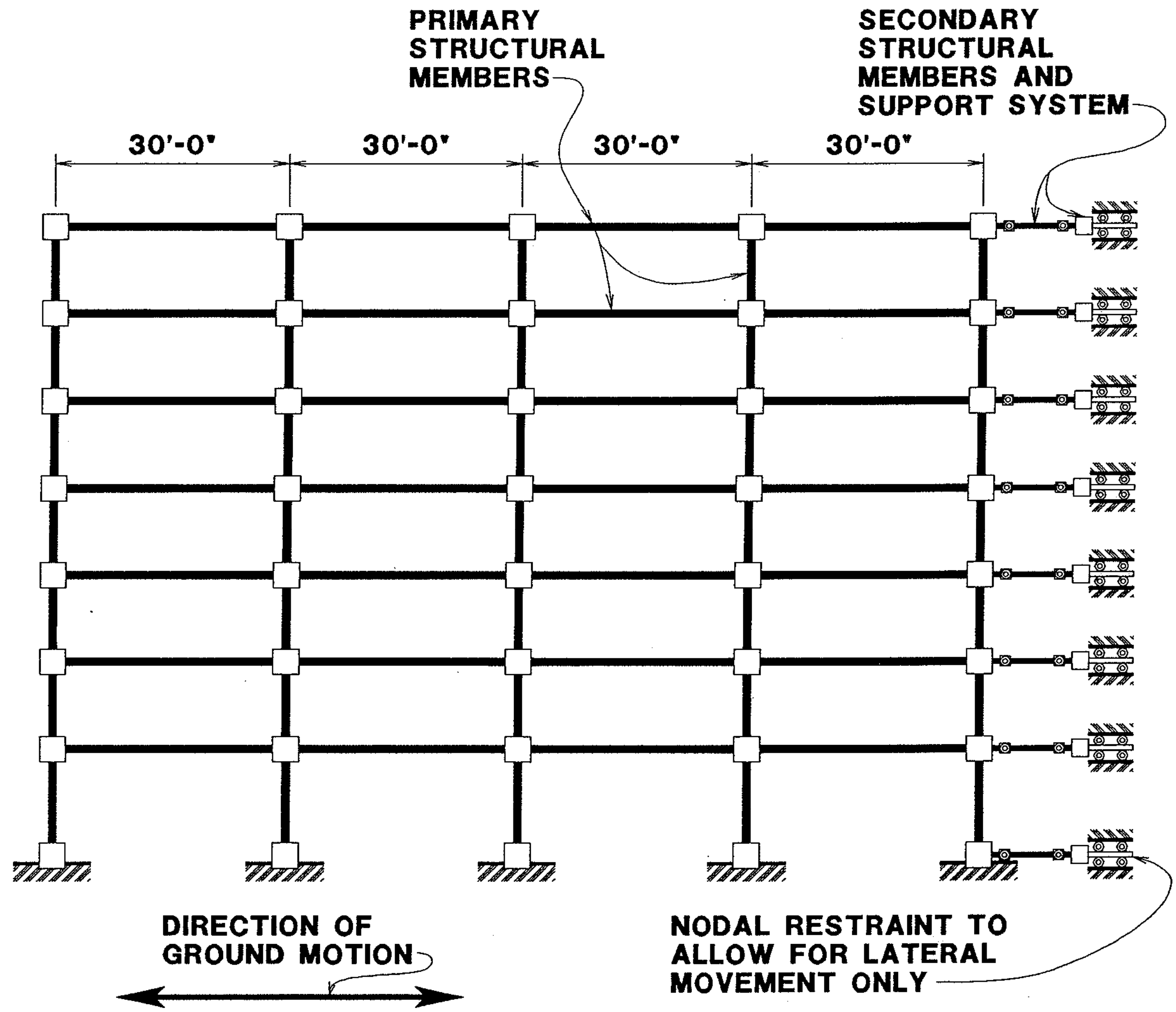

Figure 2. Moment resisting frame model

accounted for. For example, equipment mounted rigidly on floors will respond quite differently than flexible piping systems suspended from ceilings. In addition, the ductility of components may limit the load that can be produced by high acceleration values. For this study, only elastic components and attachments were investigated. Furthermore, damage resulting from differential movements of adjacent floor levels may also control the design. Forced deformation of floor to floor partitions, cladding, and piping are typical cases. These components must be capable of accommodating inter-story drift without failure. This type of damage would be amplified if the primary structure were allowed to undergo large inelastic deformations. Such nonstructural damage, as evidenced by past earthquakes, can be very significant and may endanger occupants (EERI, 1984). Although the proposed 1994 NEHRP provisions account for these differential movements critical for design, this investigation only considered the design of components governed by lateral accelerations.

The smallest lateral load on a component can occur if it has a long period compared to that of the governing mode of the primary structure. On the other hand, rigid components and anchoring systems will experience accelerations equal to that of the supporting structure. Dynamic amplification will result if the component period is very close to that of the supporting structure. For this study, three cases were evaluated. The components, 1) were essentially rigid, 2) had a period very close to that of each structure, and 3) had a period twice that of each structure. A damping ratio of 5% was assumed for the components.

The response of actual structures and components is typically measured by accelerometers placed at various locations. Drain-2DX does not give a time-history of nodal acceleration values in its output, but does give element force values. For this reason, an accelerometer was analytically modeled as a single degree of freedom system attached to the floors. Since the components of the investigation were assumed to be light compared to the weight of the structure, a small mass weighing 45.36 kg (100 lb) (0.00025 of the floor mass) was used. This minimized dynamic interaction and allowed acceleration values to be read directly from the axial load in the connecting member. For example, if the load was 266.9 N (0.06 k), the acceleration was 0.60g for the component.

Input Motions

Input motions from several events were evaluated to get a wide range of response values. The three ground motions used for this study were from the 1941 El Centro, 1971 San Fernando, and 1979 Imperial Valley Earthquakes. These earthquakes generated their highest response levels near 0.54, 0.35, and 0.37 seconds, respectively. In order to establish an equivalent basis for comparison, each ground motion was scaled to the design response spectrum found in the SEAOC Blue Book and UBC for 0.4g and S_2 soil. The maximum response of a first floor component tuned to each structure's fundamental period was used to find the spectral acceleration for each of the three earthquakes. At a period of 0.78 seconds, the ground motions used for the braced frame analysis were scaled by 1.765, 2.543, and 1.980 for the El Centro, San Fernando, and Imperial Valley Earthquakes, respectively. This equated dynamic amplification to that of the SEAOC, UBC response spectrum. The values were further scaled by 1.364, 2.134, and 0.734 to bring the peak ground acceleration to 0.4g. At a period of 1.93 seconds, the ground motions used for the moment frame analysis were scaled by 1.516, 3.167, and 3.190 to balance dynamic amplification for the same three earthquakes, respectively. The factors used to balance peak ground acceleration were identical to those used for the braced frame analysis. The average component response of the three ground motions was plotted for each level. All results were normalized to peak ground acceleration and given in terms of an amplification factor.

Analytical Results

The response profiles shown in figures 3A, 4A, and 5A represent elastic response. Figures 3B, 4B, and 5B show the same structures under inelastic response. The overall displacement ductility demand on the structures was measured as a function of displacement of the top node. For the three ground motions, the braced frame had an average ductility demand of 1.29, while the moment frame required 1.23. These were calculated assuming that the response was elastic-plastic

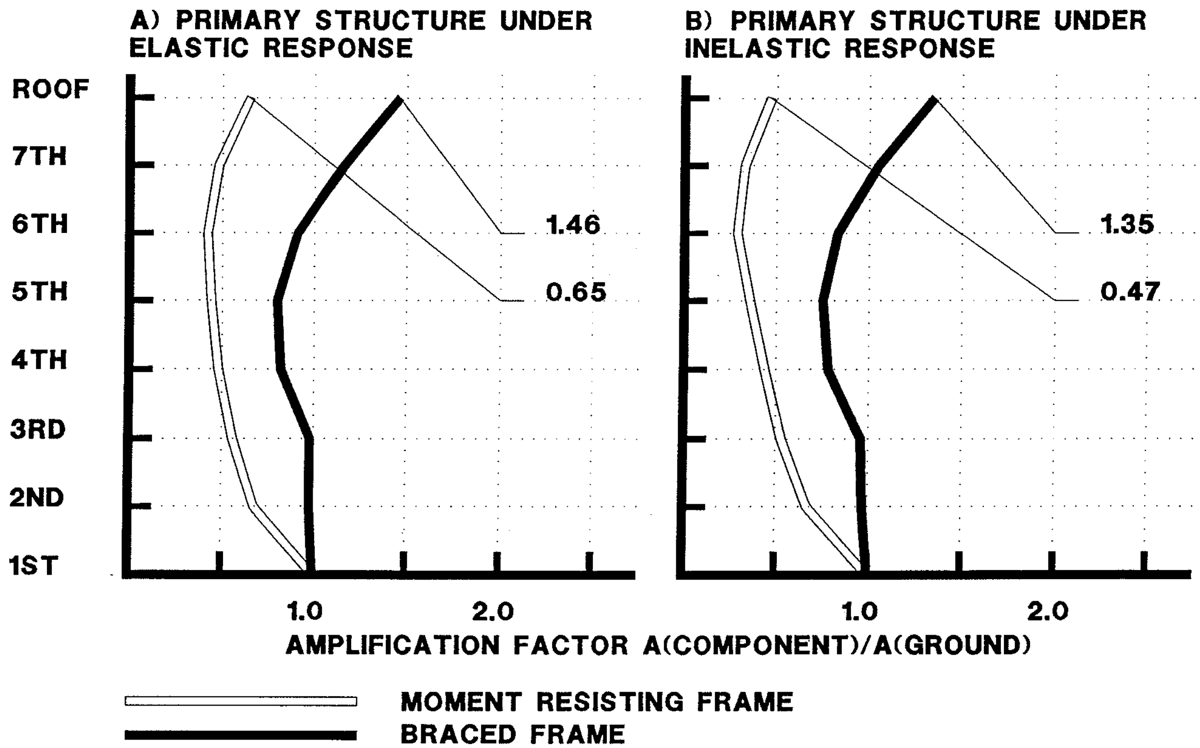

Figure 3. Response of rigid components

However, some individual members of the structures deformed extensively while others remained elastic. Actual accelerometer records from recent earthquakes were from structures that primarily experienced elastic response (Drake, 1994). Those earthquakes were not large enough to cause significant inelastic deformation that may result from a maximum credible event. In addition, the capacity of actual structures is usually enhanced by nonstructural components. The two structures in this evaluation also experienced only minor inelastic deformation.

Figure 3A shows the amplification factor at each floor level for the structures under elastic response. The response of the floor would be the same for a rigid component attached to it. It indicates an initial lack of amplification, but gradually increases towards the top of the structures. Figure 3B shows the amplification factor at each floor level for the structures under inelastic response. Note the reduction in values from the elastic structures in Figure 3A.

Figure 4A shows the amplification factors for resonant components of structures in the elastic range. Note the increase in response from the non-resonant elements of Figure 3A. Figure 4B shows the amplification factors for resonant components of structures in the inelastic range. Note again the reduction in amplification from the elastic structures in Figure 4A. However, this inelastic response is still greater than that of the rigid elements shown in Figure 3A. Since the structure and component periods are the same, the maximum acceleration of the x

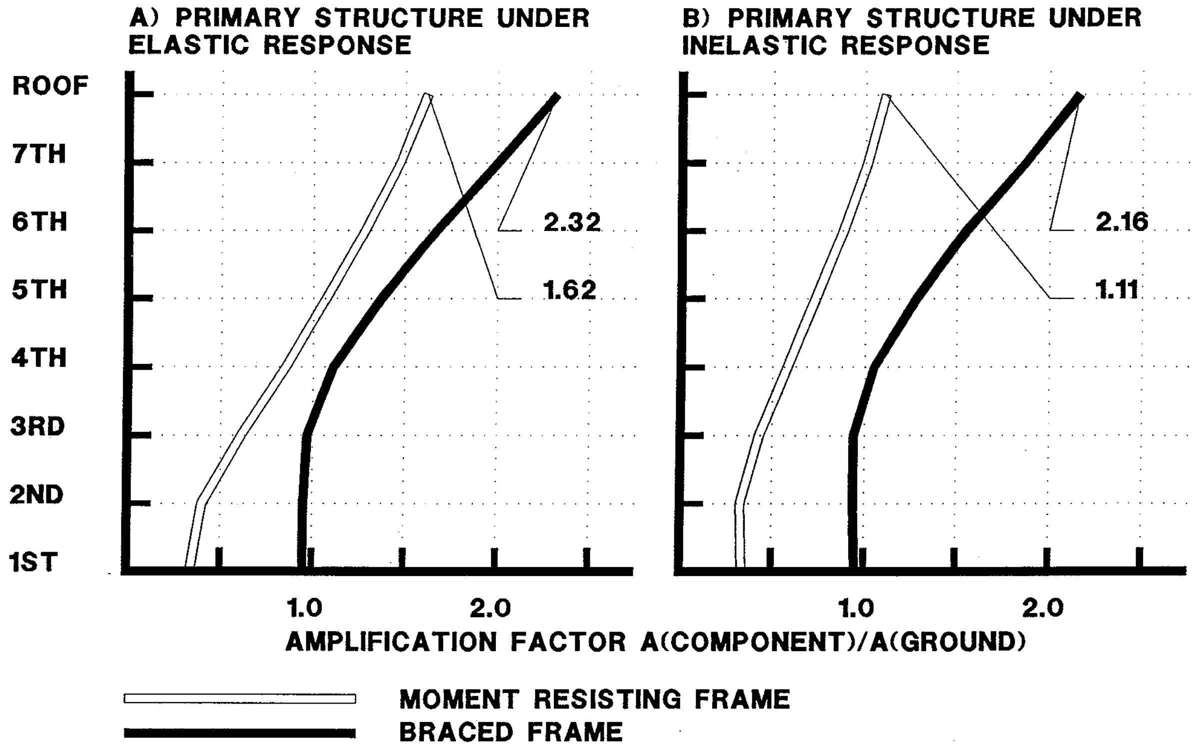

Figure 4. Response of resonant components

components at the ground is equal to the spectral acceleration of the first mode of each building. It turns out that the spectral acceleration corresponding to the fundamental period of the braced frame (0.78 seconds) is greater than the ground acceleration, while that of the moment frame (1.93 seconds) is less than the ground acceleration. This reflects the values at the base of each structure in Figures 4A and 4B.

Figure 5A shows the amplification with height of components with periods of vibration two times that of each supporting structure. Note the reduction from the resonant elements of Figure 4A and rigid elements of Figure 3A which also have support structures under elastic response. Figure 5B shows the amplification with height of these long period components attached to structures under inelastic response. The reduction in response from the elements depicted in Figure 5A is due to inelastic deformation. The reduction in response from the elements in Figures 3B and 4B is due to the difference in period associated with the components. The reductions due to inelastic behavior in Figure 5 are less pronounced than those in Figures 3 and 4. This is partially due to the fact that the periods of the yielding structures tend to approach that of long period components, exciting a resonant response. The values at the base of each structure are less than unity, resulting from the low spectral acceleration associated with the long period components.

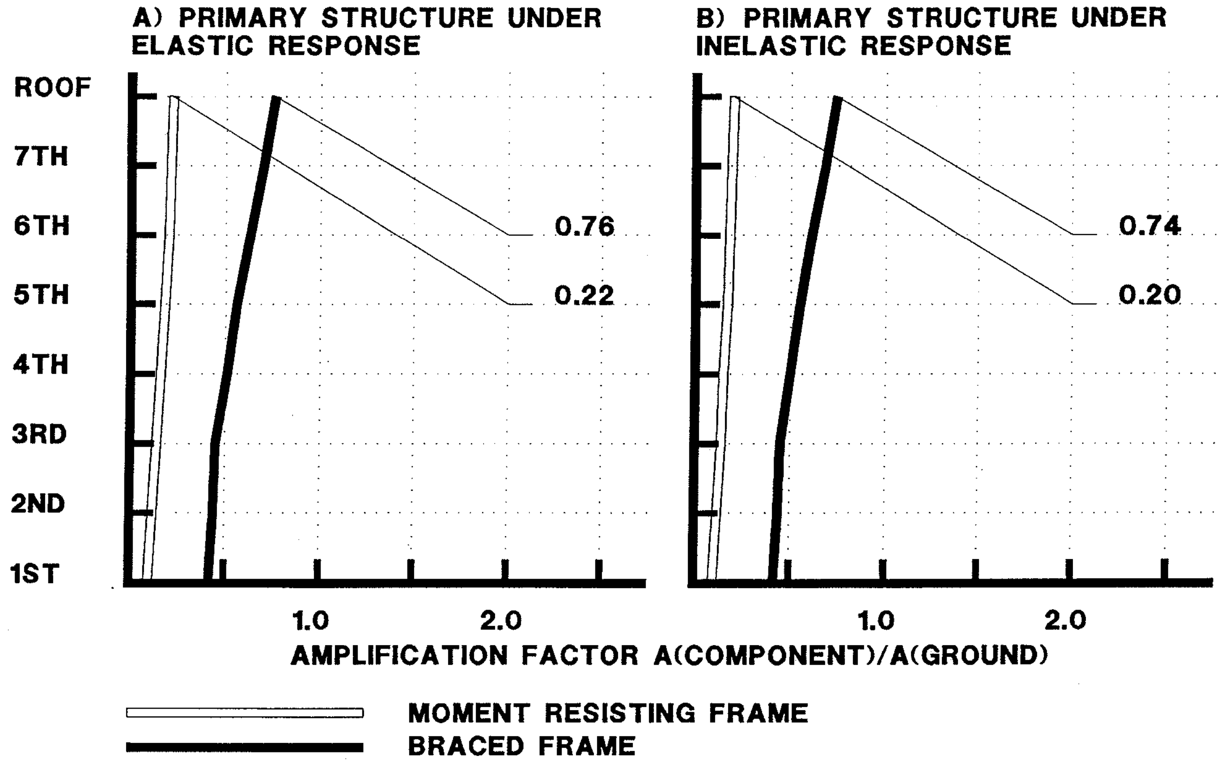

Figure 5. Response of long period components

The effect of inelastic behavior on the support structure was not included in the proposed force equations for the 1994 NEHRP provisions (Bachman, 1994). The response modification factor (R) was left out for several reasons. First, the degree of inelastic behavior is usually minor for structures designed by modern building codes. Second, nonstructural components must often be designed for a structure without knowledge of its composition. Third, it is slightly conservative. However, the propagation of plastic hinges in a structure will eventually limit the load that can be transferred to higher levels and components. The equations recommend a maximum amplification factor at the roof of 4.00 for all elements. As a structure approaches collapse, the increase in floor response becomes significantly smaller than the increase in ground motion. A graphical representation of each structure's roof response as a function of increasing ground motion is shown in Figure 6.

To examine maximum possible response, a single mass with a lightweight component, both tuned to the resonant frequency of each ground motion at 2% damping, generated amplifications of 6.20, 5.32, and 6.06 for the El Centro, San Fernando, and Imperial Valley Earthquakes, respectively. These models remained completely elastic and were evaluated at the ground level. Also, only one mode contributed to their entire response. High level response values, although rare, must be considered should a similar design situation arise.

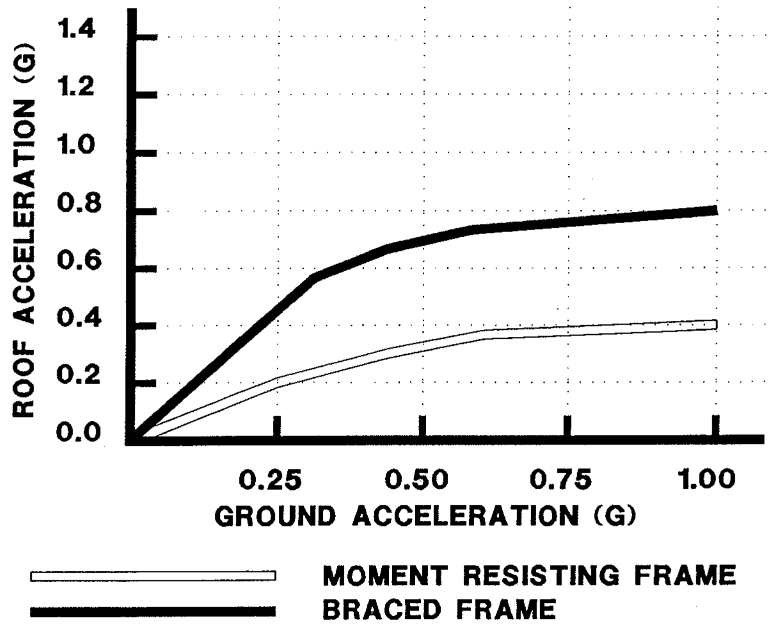

Figure 6. Component acceleration as a function of increasing ground motion

The proposed equations were checked against each structure for the elastic case with rigid components (Figure 3A). The equations produced a roof amplification of 1.57 for the braced frame (7.5% greater than the computer model) and 0.86 for the moment frame (32.3% greater than the computer model). The values produced by the equations were reduced by a factor of 1.5 (minimum) accounting for component deformation, yet the components in the analysis were modeled as rigid. If the components were modeled with limited strength and inelastic properties, the analysis would produce lower values. However, other effects, such as torsion, were not modeled and may increase the overall response. If a component amplification factor of 2.5 (maximum) was used, the results would amplify accordingly, resembling the models depicted in Figure 4A. Soil effects were not considered in the equations for this example.

Summary and Conclusions

These results and others have shown that a distinct increase in the lateral load on components is present towards the upper levels of a structure and should be reflected in design equations. Although the first mode dominated both structures, higher modes may also play a role and excite other nonstructural components. To incorporate the increase in response with height and envelope the various response levels that may result from other modes, the proposed equations use a trapezoidal response distribution that increases linearly with the ground acceleration value at the base. In addition, since a large event may generate only minor levels of inelastic behavior in modern structures, and the analytical models did not include the additional strength that can be found in a complete building system, it is likely that true response would lie between the two cases evaluated. Consequently, the primary structure response modification factor was not included in the proposed equations. The results of this effort helped in the development and verification of the proposed 1994 NEHRP equations for the seismic design of architectural, mechanical and electrical components. These equations produce more accurate load levels than previous code equations, yet are still very simple to use.

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