ANALYSIS OF METHOD FOR IMPROVING THE PERFORMANCE OF REINFORCED CONCRETE FRAME BUILDINGS DURING EARTHQUAKES

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ABSTRACT

This thesis describes a method for improving the performance of reinforced concrete frame buildings during seismic events. The criteria used to assess performance were the location of plastic hinges and the controlling mechanisms, the displacement characteristics of the frames including the interstory distortion, and the deformation of the structural elements. The goal for improved performance was to reduce hinging in columns and force the yielding into the girders, resulting in the formation of the structural mechanism. A limit analysis was used to develop the method for a strength based relationship to ensure that the structural mechanism would be the controlling mechanism for any set of frame parameters. The relationship to improve performance reduces the flexural strength of upper floor level girders in the frames a prescribed amount. Non-linear static and dynamic analyses were used to test the method on several regular reinforced concrete frames. The results indicate that the performance of the frames improved when using the method discussed in this thesis.

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CHAPTER 1. INTRODUCTION

The goal of this project was to determine a method to improve the performance of reinforced concrete frames in earthquakes. The yielding in columns of frame buildings result in large levels of interstory drift and increased damage to structural and non-structural elements. The reduction in column hinging leads to a decrease in interstory drift, especially in lower stories, which is essential to improving the performance of the frames. The member flexural strengths, member proportions, and building geometry were determined as the major factors that affect the building response in an earthquake.

1.1 Background

To provide satisfactory performance of a frame building with earthquake loading, A.C.I. requires that column flexural strengths exceed girder flexural strengths at all joints in a frame by a factor of 6/5 (A.C.I. 318-99 Equation 21.1). While this requirement aids the performance of frames, it does not guarantee that plastic hinges will not form in the columns. Other factors, such as frame geometry and loading characteristics, also affect whether or not hinging will form in the columns, in addition to member flexural strengths.

The yielding of columns in reinforced concrete frame buildings can result in major damage to structural and non-structural elements. The design of a building in high seismic zone should be proportioned to limit the amount of column yielding or eliminate it completely. When all columns of a story experience hinging, an intermediate story sway mechanism will form, allowing unrestrained drift and eventually collapse due to lateral instability. The formation of the structural mechanism is beneficial to the overall performance of a reinforced concrete frame in an earthquake. If hinging is eliminated in columns and forced to occur only in girders, the structural mechanism will form, resulting in the columns acting as a stiff, unyielding spine over the height of the building.

The performance of a frame can be assessed by evaluating the interstory drift of each story and the rotational ductility of each column. The interstory drift of a story is the difference in displacement of the floor level above and below the prescribed story. The rotational ductility of a column is the amount of rotation a column undergoes beyond the point of yielding.

Large values of interstory drift result in large levels of distortion in the affected stories. These large distortions cause damage to the structural members and are the main factor in causing damage to the non-structural elements. The damage to a building is especially severe if large distortions occur in the lower stories of the building. The lower story columns support a larger portion of the building weight and are critical to the performance of the building.

When an intermediate story mechanism forms, the building drift will be concentrated in the lower stories. This results in large values of interstory drift in the lower stories compared to the upper stories. The maximum interstory drift will occur in the lower portion of the building. The formation of the structural mechanism results in an approximately linear building displacement. The interstory drift of every story will be relatively small and approximately uniform. The maximum interstory drift of a building decreases overall and is located at a higher story if the structural mechanism controls over an intermediate mechanism.

The yielding of the columns in a reinforced concrete frame results in large values of rotational ductility in the columns. These large levels of ductility in the columns indicate excessive plastic deformation has occurred. Whenever large deformations occur in columns, damage to these structural members is significant and may result in building collapse. If large column deformations occur in lower stories the damage to the building increases.

Hinging in columns is prevalent whenever an intermediate story mechanism forms. The column deformations are large in stories at or below the location of the mechanism as a result of the yielding. The overall maximum ductility of the columns is also large when an intermediate mechanism forms. The formation of the structural mechanism forces hinging into the girders and out of the columns. This results in the plastic deformation occurring primarily in the girders and not the columns. The column deformations are relatively small, especially in the critical lower stories. The overall maximum column ductility decreases and is located at a higher story if the structural mechanism controls over an intermediate mechanism.

The performance of a reinforced concrete frame will improve when the structural mechanism controls over an intermediate story mechanism. The A.C.I. provisions do not guarantee the formation of the structural mechanism. The structural and non-structural elements in buildings are at risk of significant damage in earthquakes if the structural mechanism does not control. The structural mechanism causes a reduction in column yielding which will reduce interstory drifts and column deformations, resulting in improved frame response.

1.2 Past Research

The seismic behavior of reinforced concrete frames with hinging columns was studied using strong-beam-weak-column (SBWC) frame models (Shultz, 1991). Schultz discussed the difficulty in preventing plastic hinging in columns by requiring column flexural strengths to be greater than beam flexural strengths. The paper documented the problems that arise when all columns of a story experience hinging and a story mechanism forms. Schultz analyzed whether allowable drift levels were maintained for a SBWC building with a 1st story mechanism if it were adequately proportioned.

Yielding and non-linear deformation were initiated in the 1st story, but other stories yielded at later times. The results of the experiments indicate that at 1% allowable drift (stiff members), the performance of a SBWC frame with a 1st story mechanism performed satisfactorily. At 1% drift, all stories responded in a stable fashion well beyond yield (hinging of columns) when loaded laterally. If the frame was proportioned to allow for 1.5% drift, the performance of the frame (column hinging) was not as good.

The seismic performance of structural frames and the safety margins against damage and collapse were studied (Bracci et al., 1995). The authors concluded that structures designed for gravity loads (GLD structures) are dominated by SBWC behavior even though current design considerations implicitly require the formation of a desirable beam side-sway collapse mechanism. The authors determined that the mode of failure under strong ground motion for a GLD structure was an undesirable column side-sway (soft story) mechanism. Bracci et al. also determined that the strength of GLD structures can be accurately predicted with a limit analysis and a static pushover analysis.

Bracci also conducted a related study on the seismic retrofit of reinforced concrete buildings designed for gravity loads (Bracci et al., 1995). The authors concluded that the seismic performance of soft story prone GLD frames may be effectively enhanced by strengthening limited columns and thus, changing the failure mode to a beam side sway mechanism with acceptable story deformation levels. It was determined that any level of retrofit should be considered to reduce the risk for structural collapse (Bracci et al., 1995).

An additional study was performed on the behavior of gravity load designed (GLD) reinforced concrete buildings subjected to earthquakes (El-Attar et al., 1997). Experimental tests on three-dimensional GLD reinforced concrete buildings using a shaking table were conducted. The first mode of vibration was dominant for all the seismic tests. At failure, plastic hinges developed in the columns, producing a soft-story failure mode. The authors also concluded that even though reinforcing details in GLD reinforced concrete structures may form a potential source of damage, they are probably not sufficient to develop a failure mechanism (El-Attar et al., 1997).

A study was performed in order to explain the numerous upper floor collapses during the 1985 Mexico City earthquake (Villaverde, 1991). Villaverde concluded that upper floor failures were possibly the result of the second or third modes of vibration influencing the behavior of the frames. Villaverde also suggests that the upper floor collapses could have been caused by a failure mechanism generated by the formation of plastic hinges in upper columns which lead to inelastic deformations that exceeded the design limits.

There have been several analyses performed on the 7-story Holiday Inn building in Van Nuys, California. An analysis of the building was done by John A. Blume & Associates after the 1971 San Fernando earthquake (Blume, 1973). An analysis was made to study the response of the Holiday Inn during the 1994 Northridge earthquake (Ventura et al., 1995). The authors investigated the recorded ground motions to determine the cause of the damage in the building. A performance assessment study was performed on the Holiday Inn building in the 1994 Northridge earthquake (Browning et al., 2000). The authors analyzed three separate models of the building and concluded it was near collapse due to the shear failures that occurred.

1.3 Project Scope

The major goal of this study was to determine a method to encourage the formation of the structural mechanism in reinforced concrete frame buildings during earthquakes. The formation of the structural mechanism reduces plastic hinging in the columns of the buildings. The reduction in column yielding results in smaller levels of interstory drift and column ductility, especially in the critical lower stories. The performance of the reinforced concrete frames improves whenever the structural mechanism is encouraged to form.

A limit analysis was performed on several regular reinforced concrete frames to determine the response under static constant and linear load distributions. The frames were proportioned by varying the building geometry and the member strengths. The number of stories (N_s) and the ratio of column flexural strength to girder flexural strength (α) were the main parameters. The controlling mechanisms were calculated to determine the values of α that were necessary in order to form the structural mechanism. The results were compared to the requirements of A.C.I. Equation 21.1.

Three strength based methods that would force the structural mechanism to control were investigated by using a limit analysis with a linear load distribution. The three methods that were investigated were: increase the flexural strength of all columns, increase the flexural strengths of columns at certain stories, and decrease the flexural strengths of girders at certain floor levels. Several 4-16 story frames were similarly proportioned and analyzed to determine the girder and column flexural strengths necessary for the structural mechanism to form. The method of reducing girder flexural strengths in upper floor levels was chosen as the preferred method due to simplicity of application and cost feasibility.

A parametric analysis was used to test and refine the method to improve performance. Frames with varying number of stories (N_s), number of bays (N_b), column width (h), column reinforcement (ρ), number of floor level with reduced girder strengths (β*N_s), and ratio of column to girder flexural strength (α) were analyzed using a limit analysis with a linear load distribution was used on. A relationship of these parameters was derived to calculate the amount of reduction in girder strength (R) necessary to encourage the formation of the structural mechanism. The initial girder strengths were divided by the factor R to determine the necessary modified girder flexural strengths.

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2} \qquad 1.1$$

Non-linear static and dynamic analyses were performed on 4-16 story regular frames to test the method for improved performance (Equation 1.1). The frames were proportioned initially and then modified by reducing the girder flexural strength in the upper floor levels. A linear load distribution was used in the static analysis and the ground motions of ten earthquakes were used in the dynamic analysis. The response of the unmodified and modified frames were compared to determine if the method of reducing girder strengths (Equation 1.1) improved the response of the frames.

The building response was calculated at the formation of the controlling mechanism in the static analysis. The maximum building response during the earthquake was calculated in the dynamic analysis. The performance of the frames was based on the controlling mechanism, the interstory drift and the column ductility. The results of the analyses indicated that the performance of the frames improved when the girder flexural strengths were reduced. The structural mechanism controlled over the intermediate story mechanisms resulting in improved levels of drift and ductility.

The method for improved performance was applied to a 7-story Holiday Inn building that suffered damage in the 1971 San Fernando and 1994 Northridge earthquakes. The non-regular building was analyzed as it was designed and modified by Equation 1.1. Non-linear static and dynamic analyses were used with an approximately linear static load distribution and the ground motion recorded from the 1994 Northridge earthquake. The results of the analyses indicated that reducing the girder strengths (R) was successful in improving the performance of the Holiday Inn building by forming a structural mechanism instead of the intermediate mechanism which caused significant damage during the actual seismic event.

CHAPTER 2. DEVELOPMENT OF RELATIONSHIP TO IMPROVE FRAME PERFORMANCE

The main goal of this study is to reduce column yielding and eliminate intermediate mechanisms from forming in reinforced concrete frames during earthquakes. When column hinging leads to the formation of an intermediate sway mechanism, drift and damage will be concentrated on the stories with yielding members. Figure 2.1 illustrates the formation of an intermediate sway mechanism. The filled circles represent locations of plastic hinges. A relationship to improve performance of frame buildings by reducing column hinging and redistributing drift over the height of the structure was developed. The focus of the method to reduce damage due to column yielding is to encourage the formation of the structural mechanism. The structural mechanism forms when all the girders in the frame yield resulting in a linear drift distribution. Figure 2.2 illustrates the behavior of the structural mechanism.

First, a general understanding of the hinging behavior of reinforced concrete frames in earthquakes was investigated. This included determining the adequacy of current code requirements for reducing yielding in columns. Then, an initial hypothesis was developed using limit analysis of frames subjected to lateral loads. A parametric analysis was then performed to refine the hypothesis into a working relationship to reduce column hinging for a variety of regular frames. Finally, the proposed equation was modified for simplicity.

2.1 Investigation into Reinforced Concrete Frame Behavior in Earthquakes

In order to achieve the project goal of improved performance of reinforced concrete frames, a better understanding of how and why certain mechanisms formed in structures was sought. An initial task of the study was to determine the effectiveness of existing code requirements for reducing yielding in reinforced concrete columns. A limit analysis of frames subjected to lateral loads was used in these investigations.

2.1.1 Description of Limit Analysis

For the limit analysis, virtual work principles were used to calculate the base shear required to form all possible intermediate mechanisms and the structural mechanism (Sozen, 1996). The mechanism that required the smallest base shear strength (total lateral force) was considered the limiting case for the selected load distribution. For example, if for a 3 story building, the limiting base shear corresponds to the formation of a 2^{nd} story mechanism (Figure 2.1), then that is the mechanism that will form under the prescribed distribution of load.

In a limit analysis, virtual work principles are used to calculate the required base shear in order to form all possible sway mechanisms and determine the limiting case. External work is calculated as the product of an assumed force distribution and a virtual displacement. The actual force is the applied lateral load to the frame, which is the same for all mechanisms. For this study, a linear and constant lateral load were considered for use in the analysis. Figures 2.3 and 2.4 illustrate both the linear and constant assumed lateral loading patterns. Ultimately, the linear loading pattern was chosen because it better corresponds to the typical distorted shape of a regular reinforced concrete frame subjected to earthquake loading.

The virtual displacement is the assumed deformed shape of the structure for a particular mechanism The frame is assumed to displace linearly from the ground until the prescribed intermediate mechanism is reached wherein the displacement remains constant to the top of the building. Deformation of elements between yielding points is neglected. Therefore, for a structural mechanism (Figure 2.2) the virtual displacement will be linear over the entire height of the structure. Figure 2.1 shows the assumed virtual displacement for a 2nd story intermediate mechanism for a three story frame with an applied linear load.

The internal work is calculated by summing the product of the flexural capacity and the rotation of all structural members that have yielded. The rotations take into account the thickness of the members. The girder rotations (θ_g) were calculated as:

$$\theta_g = \frac{L_s}{H_c * L_c} \qquad 2.1$$

L_s = Centerline to centerline span length

H_c = Height to the highest level of hinging

L_c = Clear span length (L_s – column width)

The column rotations (θ_c) were calculated as:

$$\theta_c = \frac{1}{H_c} \qquad 2.2$$

The internal work will vary depending on the assumed mechanism. For every specific mechanism there will be a specific number and type of structural member that will be assumed to have yielded. For example, a 2nd story mechanism will have the base and 2nd story columns yielded, in addition to all the girders up to the 2nd story (Figure 2.1). The columns can either yield at the bottom or the top of the story. Therefore, there are actually two intermediate mechanisms that can form for each story because of the difference in rotation. Since column strengths were assumed to be uniform over the story height, the intermediate mechanism at the top of the story always controlled over the intermediate mechanism at the bottom of the story.

A structural mechanism is defined with all the girders yielded, with columns yielding only at the base. (Figure 2.2) The required base shear to form each mechanism is then computed as the ratio of the total internal work to the total external work when the total applied force is equal to one. The limiting value of required base shear indicates the controlling mechanism for the frame under a given lateral load.

2.1.2 Mechanism Formation in Frames with Constant Column and Girder Strengths Throughout the Building

A limit analysis of frames subjected to lateral load was used to evaluate the controlling mechanisms for several theoretical frames and to determine a strength relationship that could cause the structural mechanism to be the controlling mechanism. The limit analysis is based on the flexural yield capacities of the structural members. Initially, the column and girder yield capacities were assumed to be uniform over the height of the building, with the member flexural capacities related by a factor, α. The factor α is defined as the ratio of column yield strength to girder yield strength:

$$\alpha = \frac{M_c}{M_g} \qquad 2.3$$

Several theoretical frames were analyzed. The buildings all had four 20-foot bays and 10-foot uniform story heights and ranged from 4-16 stories. Linear and constant load distributions were applied to the frames. Member sizes were chosen using a uniform girder depth of 24 inches and a uniform column width of 24 inches. Values of α were varied by increasing the strength in the columns. All of the flexural yield capacities of the columns in a frame were increased uniformly.

Figure 2.5 shows the required value of α is necessary to form the structural mechanism for frames of varying number of stories and having constant girder and column flexural strengths throughout the building. Linear and constant applied loading distributions were considered.

Figure 2.5 shows that for constant strength columns and girders, an α value between 2 and 8 is required in order to form a structural mechanism. Therefore, the column strength would have to be 2 to 8 times greater than the girder strength. The increase of the α value necessary for the structural mechanism with increasing total number of stories is approximately linear. Finally, Figure 2.5 illustrates that an α value necessary for the structural mechanism is higher for a constant load distribution as compared to a linear load distribution.

2.1 Mechanism Formation in Frames with Realistic Column and Girder Strengths

The use of constant girder and column strengths in a building is simple but not realistic. While the girder strengths may be similar throughout a frame, the column strengths with constant reinforcing details will vary due to the increasing axial load on each column. Limit analysis was used in conjunction with actual column and girder flexural strengths to investigate the mechanism formation in frames the required α to form a structural mechanism for reinforced concrete frames.

2.2.1 Frame Geometry and Member Properties

The frames used in the analysis were considered as one line of columns with girders spanning the supports. The frames were considered as two dimensional structures. Therefore, for the joint on the top floor at the exterior column, there is exactly one column and one girder framing into it. Figure 2.6 illustrates the single column line frame that was used in the limit analysis.

Yield strengths of columns and girders were calculated using a moment curvature relationship. The stress and strain constants for concrete used in the moment-curvature calculations were as follows:

f'_c (Concrete compressive strength) = 4000 psi

E_c (Young's modulus) = 3600 ksi

f_t (Concrete tensile strength) = 0 psi

ε_{cu} (Concrete ultimate strain) = 0.0038

The stress-strain relationship of the concrete was approximated based on the model by Hognestad (Hognestad, 1951) using the following relationship:

$$\varepsilon'_c \text{ (Concrete strain at } f'_c = \frac{2f'_c}{E_c} \tag{2.4}$$

An elastoplastic steel stress-strain relationship was assumed for the moment-curvature calculations. The constants were assumed to be the following:

f_y (Steel yield stress) = 60 ksi

E_s (Steel modulus of elasticity) = 29000 ksi

ε_{sh} (Strain at start of strain hardening) = 0.0075

ε_u (Ultimate strain) = 0.1200

f_u (Ultimate strength) = 100 ksi

The shape and size of the member as well as the amount and location of the reinforcement were varied in the calculation of moments and curvatures. Also, the axial load on a member was specified as 160 psf on the column tributary area to allow for a member to be in both compression and in flexure. The moments and curvatures at increasing strain intervals were calculated and plotted. The flexural capacity or yield moment of a member was specified at a point of substantial change in slope of the moment-curvature plot. The location on the curve where the slope changed by a factor of five or greater was the point at which the yield moments were obtained for the study.

The girder reinforcing details and sizes were considered uniform throughout the structure, and therefore had the same yield moment. It was assumed that a portion of the slab would contribute to the strengths of the girders. The top flange width of the girder was taken as the girder width plus twice an effective girder depth. The effective girder depth was calculated as the total girder depth minus the slab depth.

Different girder depths and reinforcements were used to vary the girder flexural strength (M_g) to be used in the limit analysis. For a specific frame, the columns were assumed to be the same size and have the same amount of reinforcement throughout the structure. However, the column yield moment (M_c) changed with increasing axial load at each floor and between interior and exterior columns of the same floor.

2.2.2 Analysis

A limit analysis with applied lateral load was used to determine the controlling mechanisms. The number of stories, the loading condition and α were varied. For this analysis, the factor α was specified to equal the ratio of the yield moment of the top exterior column (M_{ctext}) to the uniform yield moment of the girders (M_g):

$$\alpha = \frac{M_{ctext}}{M_g} \tag{2.5}$$

The top story exterior column has the least axial load applied to it of any column. Therefore, the flexural strength of the top exterior column was the minimum value possible and α of the top exterior column was also the minimum throughout the frame (α_{min}).

The girder used in this analysis had a total depth of 24 inches and was kept uniform throughout the frames for all number of stories. The girder strengths were uniform throughout each individual frame. However, sometimes the uniform girder strength had to decrease when the α value necessary to form the structural mechanism became so large that the columns had reached a maximum ρ of 8%. (A.C.I 318-99 Section 10.9.1) The column sizes varied depending on the number of stories in the frame as shown in Table 2.1. Therefore, the columns had varied flexural strengths between frames.

The flexural capacities of the girders and columns were adjusted by varying the amount of steel (ρ) in the members. The columns used the same ρ for all stories in the frame, therefore, the only variation in column flexural strengths was to increase from top to bottom due to the increasing axial load. The axial loads were based on 160 psf loading on square tributary areas. The flexural strengths of the columns were adjusted to determine the value of α. Table 2.2 shows a summary of the parameters used in this analysis.

The value of α was varied from a minimum of 1.20 for each frame up to the value at which the structural mechanism forms. The minimum α of 1.20 was used because that is the minimum ratio of column strengths to girder strengths at a joint prescribed by American Concrete Institute (A.C.I.) Equation 21.1 (A.C.I. 318-99) which states:

$$\sum_{joint} M_c \geq (6/5) \sum_{joint} M_g \qquad 2.6$$

$\sum_{joint} M_c$ = The sum of the flexural strengths of all columns framing into a joint

$\sum_{joint} M_g$ = The sum of the flexural strengths of all girders framing into a joint

2.2.3 Results

Figure 2.7 shows that for an α of 1.20 an intermediate mechanism forms for all the frames. In general, the mechanism that forms for a linear load is approximately 60% of the building height and for a constant load is 50% of the building height. The lower story mechanism indicates that there would be increasing interstory drift ratios in the lower story columns. Also, it appears that the location of the mechanism is almost independent of the total number of stories in the building.

Figure 2.8 shows the value of α necessary to form a structural mechanism using an applied linear load. The girder strengths are uniform, but the column strengths vary depending on the amount of axial load that is applied. The α values range from 1.90 to 3.90. The required value of α increases as the total number of stories increase. However, the increase of α with number of stories is not linear. This is due to the fact that column flexural strengths are not uniform, and the rate at which the strengths increase due to an increase in axial load varies.

2.3 Comparison of Results to Code Requirements

Equation 2.6 maintains that for any given joint in a building the sum of the column flexural strengths should exceed the sum of the girder strengths by a factor of 6/5. This is intended to reduce the likelihood of yielding of any reinforced concrete frame building with earthquake loading. A comparison can be made of the A.C.I. requirement in Equation 2.6 and the α value needed to form a structural mechanism because the top exterior joint has exactly one column and one girder framing into it. At this joint the ratio of the sum of the column strengths to the sum of the girder strengths is equivalent to the value α_{min} defined above.

Figure 2.8 shows that the requirement maintained in Equation 2.6 does not guarantee the formation of the structural mechanism and therefore may exhibit poor performance for a reinforced concrete frame. As shown in the previous section, α values of at least 1.90 were needed to form the structural mechanism for buildings with 4 to 16 stories.

The α required to form the structural mechanism ranges from 1.90 to 3.90 for 4 to 16 story buildings. This value of α is calculated based on the strength of the top story exterior column. These α_{min} values required to form the structural mechanism are significantly greater than the 6/5 column strength to girder strength ratio prescribed by A.C.I. The α values at other joints will be greater due to the increase in column strength from higher axial loads.

2.4 Development of Initial Hypothesis

A relationship to reduce the likelihood of yielding in columns of reinforced concrete frames was sought. The two variables that were first considered in attempting to improve the performance of the frames were stiffness and strength. In the limit analyses, the size of a member affected only the rigid lengths used to calculate member rotations. This was found to have negligible impact on the limit analysis results. Therefore, varying stiffness has little effect on the determined limiting mechanism and member strength was considered the significant factor in eliminating column yielding.

Three options were investigated: global increase in column strength, increase of column strengths at specific stories, and decrease of girder strength at specific floor levels. A limit analysis was used to evaluate these three options.

2.4.1 Global Increase in Column Strength

The first option that was investigated was to globally increase the flexural strength in the columns. The term "global" means that the structure as a whole is considered, not individual stories or members The factor α was incrementally increased for all joints for a specific frame until the structural mechanism was reached. The girder strength was held constant while the column strength was increased, resulting in an increased value of α. In this case, α is based on the top exterior column-girder connection. The column strengths were increased by increasing the reinforcement of the columns.

Increasing the flexural strength in columns can be used to form a structural mechanism as demonstrated by the calculations used in the limit analysis. For a particular intermediate mechanism, the greater the value of the column yield capacities at the level of the prescribed mechanism, the greater the base shear required to form that mechanism. In order for the structural mechanism to become the limiting mechanism for a frame, the base shear required to form the structural mechanism has to be less than the base shear required to form any other possible intermediate mechanism.

When column strengths are increased, the base shear required to form intermediate mechanisms is also increased. The base shear required to form the structural mechanism is largely unaffected because its required base shear is mostly affected by the girder flexural strengths. The base of the bottom story columns will slightly affect the base shear required to form the structural mechanism since a hinge is assumed at the column base for all mechanisms. When the column strengths are increased to the point that the required base shear to form all intermediate mechanisms is greater than the required base shear to form the structural mechanism, than the structural mechanism is allowed to form.

Figure 2.9 shows the base shear required to form the limiting mechanism and the structural mechanism for a four story frame at given values of α. The column strengths were increased in order to incrementally increase α. As α is increased, the base shear required to form the mechanisms increases. The base shear required to form the limiting intermediate mechanisms increases at a higher rate than the base shear required to form the structural mechanism, until eventually the structural mechanism becomes the controlling mechanism for the selected force distribution.

The limit analysis that was discussed in section 2.1.3 was used in assessing the global increase in column strengths. The α necessary for the structural mechanism to form ranged from 1.90 for a four story building to 3.90 for a sixteen story building, meaning that the column strength in the top exterior column had to be 1.90 to 3.90 times as strong as the uniform girder strength. The column flexural strengths in this case were increased globally so that all columns were affected.

This method was not considered in the hypothesis because even though it is a simple approach, it is not practical. These values of α shown in Figure 2.8 are too high to be used and increasing the strength of all columns of a building by this degree would be expensive.

2.4.2 Increase in Column Strength at Specific Stories

Another option to consider was to increase the column strength at only specific stories. For a given frame and load distribution, the base shear required to form the structural mechanism can be greater or less than the base shear required to form all the intermediate sway mechanisms. An increase in column strength is only needed at those stories where the base shear required to form an intermediate mechanism at that level is less than the base shear required to form the structural mechanism. Therefore, increasing the column strength for only certain prescribed stories can be used to ensure the formation of the structural mechanism. This would be cheaper than increasing the strength of all columns in a frame.

The column and girder strengths were considered uniform as in section 2.1.2. A limit analysis with a linear load distribution was used on 4-16 story frames to determine what increase in strength would be needed at each story to make the structural mechanism the limiting mechanism. The increase in column strength is represented as a factor that when multiplied by the initial column strength ensures the base shear required to form that particular intermediate mechanism is greater than the base shear required to form the structural mechanism. The column strengths are uniform initially. The initial global value of α for each frame can be calculated using Equation 2.3 because the initial column strengths and girder strengths are uniform in this analysis.

Figure 2.10 shows the increase in column strength needed for the structural mechanism to be the limiting mechanism for 4-16 story frames using a linear load distribution and an initial global α of 1.20. The story levels were normalized into a percent building height by dividing the story number by the total number of stories in the building. For example, the 4^{th} story of a 4 story building is at a percent building height of 100% and the 1^{st} story of the same 4 story building is at a percent building height of 25%.

Figure 2.10 shows that for a building with initially uniform column strengths, a specific increase in column strengths is needed at almost every story. This means that nearly every intermediate mechanism has a smaller required base shear than the base shear required to form the structural mechanism. The stories that do not need an increase in column strength show a value of 1.00. The maximum increase in column strength necessary to form the structural mechanism occurs in stories at approximately 65% of the building height. The column strengths would have to be increased at each of the other stories by a lesser and varying amount. The maximum increase in columns strength necessary to form the structural mechanism ranges from 1.50 to 4.80. A larger increase in column strength is needed as the number of stories in the frame increases.

The column strengths were than varied by applying appropriate axial loads to each column as described in Section 2.2.1. This results in a more realistic model to analyze. A limit analysis with a linear load distribution was used to determine how much the column strengths at each story needed to be increased in order for the structural mechanism to be the limiting mechanism. The increase in column strength is represented as a factor that when multiplied by the initial column strength ensures the base shear required to form that particular intermediate mechanism is greater than the base shear required to form the structural mechanism. The parameters shown in Table 2.1 and Table 2.2 were used in this analysis. The only difference is that in this case Equation 2.5 was used to calculate the initial global α of 1.20. The column strengths were increased at each story by a varying factor to force the structural mechanism to become the limiting mechanism.

Figure 2.11 shows the increase in column strength at each story needed for the structural mechanism to be the limiting mechanism for 4-16 story frames. This is for a linear load distribution, an initial global α of 1.20 and varying initial column strengths due to increasing axial loads. An increase in column strengths is needed at almost every story. The stories that do not need an increase in column strength show a value of 1.00. All stories require an increase in column strength for a percent building height of greater than about 20%.

The maximum increase in column strength necessary to form the structural mechanism occurs in stories at approximately 70% of the building height. The column strengths would have to be increased at each of the other stories by a lesser and varying amount. The maximum increase in columns strength necessary to form the structural mechanism ranges from 1.80 to 3.90. These maximum values are approximately equal to the global α values necessary to form the structural mechanism shown in Figure 2.8. A larger increase in column strength is needed as the number of stories in the frame increases.

The column strength does not have to be increased globally to cause the structural mechanism to be the limiting mechanism. Each frame has a critical story where the maximum amount of increase in column strength is required. The structural mechanism will form once the column strengths of this critical story have been increased enough such that the base shear required to form this critical intermediate mechanism exceeds the base shear required for the structural mechanism.

The maximum necessary increase in column strengths at the critical story is generally large in value. The necessary increase in column strengths is smaller at other stories of the frames. The maximum necessary increase in column strength is approximately equivalent to the necessary global α for the same parameters. Therefore, increasing the strength of columns by varying amounts at different stories would be cheaper than increasing the column strengths uniformly throughout a building. The location of the critical story, where the maximum increase in column strength is needed, can vary between frames of varying parameters. Also, because nearly every story requires some column strength increase, it is complicated to determine how much to increase column strength at each particular story for a given set of parameters. There is no discernible pattern for calculating the necessary increase in column strengths at each story.

2.4.3 Decrease in Girder Strength at Specific Levels

The final option considered was to decrease the girder strengths at certain levels of a frame. This strategy is significantly different than the other options discussed where column strengths were increased to force the structural mechanism into the controlling mechanism. The reduction of the strength in girders will significantly reduce the base shear required to form the structural mechanism. The base shear required to form the structural mechanism is almost totally dependent on the girder strengths. Since the strength in girders below the story where the intermediate mechanism occurs affects the base shear for that mechanism, the base shear required to form the intermediate mechanisms also will be reduced. The structural mechanism will control as the base shear required to form the structural mechanism is reduced below the base shear required to form all intermediate mechanisms.

It does not matter which girders are reduced in strength in order to reduce the base shear required to form the structural mechanism. However, the base shear required to form the intermediate mechanisms is dependent on which girders have reduced strengths. This is because all of the girders below a prescribed yielded story affect the base shear required to form that mechanism. The locations of reduced girder strengths should be at levels so that the base shear required to form the intermediate mechanisms is reduced at a rate much less than the reduction of the base shear required for the structural mechanism.

The best location to reduce the strength in girders is in the top levels of the building. If girder strengths are reduced in only the top few levels, then most of the intermediate mechanisms are unaffected (i.e., the base shear required to form intermediate mechanisms will only be reduced for the upper story mechanisms). The impact of reducing the girder strengths in the upper levels will not be significant to the upper story mechanisms because the rest of the unaffected girders in the frame are still included in the base shear calculations. Thus, the base shear required to form the structural mechanism will be reduced by the reduction in girder strength at the top levels while the base shear required to form the intermediate mechanisms will remain largely unchanged.

A limit analysis with a linear load distribution was used with parameters listed in Table 2.1 and Table 2.2 with an initial global α of 1.20 calculated by Equation 2.5. The only difference is that in this analysis the girder strengths were reduced starting with the top level and continuing down the building until the structural mechanism was the limiting mechanism. A maximum girder strength reduction of 4.00 was specified for each floor. The column strengths were unchanged during the analysis.

Figure 2.12 shows the reduction in girder strength needed in the upper floor levels of 4-16 story frames to make the structural mechanism the controlling mechanism for a linear load distribution. A maximum reduction of 4.0 was used per floor level because it was estimated that any reduction in strength greater than 4.0 would cause the girders to be inadequate in resisting gravity loads. The number of required girders with reduced strength in order for the structural mechanism to form increases as the total number of stories increases. Therefore, the number of floor levels with girders that require reduced strength also increases as the total number of stories increases.

Figure 2.13 shows the lowest floor levels in each frame with reduced girder strength required to form the structural mechanism. The floor levels were normalized into a percent building height by dividing the floor level number by the total number of floors in the building. In section 2.4.2 it was determined that for the specific set of parameters used in the analyses, the critical intermediate mechanisms occurred at about 65-70 % of the building height. This critical intermediate mechanism is the mechanism that initially requires the least base shear to form. Figure 2.13 shows that if the girder strengths are reduced at levels above this critical location, then the structural mechanism will become the limiting case.

An easier way of applying the method of reducing girder strength to form the structural mechanism was to reduce the girder strengths a uniform amount in each floor level. The uniform reduction in girder strength will vary depending on the number of floor levels with girders of reduced strength. As the number of floor levels with reduced girder strengths increases, the uniform reduction factor necessary to make the structural mechanism the controlling mechanism decreases. However, once the lowest floor level with reduced girder strengths approaches the height of the critical story level, the uniform girder strength reduction factor becomes nearly constant. Therefore, reducing the girder strengths in levels below the critical story height has little impact in the formation of the structural mechanism. A limit analysis with parameters from Table 2.1 and Table 2.2 were used with an initial global α of 1.20 from Equation 2.5.

Figure 2.14 shows the uniform girder strength reduction necessary in certain levels of each 4-16 story frame so that the structural mechanism is the controlling mechanism for a linear load distribution. The same reduction in girder strength is applied at each floor level. The uniform reduction in girder strength ranges from 1.50 to 2.70 and varies depending on the number of floor levels with reduced girder strengths. In Figure 2.14, the number of bars for each 4-16 story frame represents the number of floor levels that have uniform reduced girder strengths. For example, the 4 story building has 2 floor levels that need girder strengths reduced while the 16 story building has 6 floor levels that need girder strengths reduced in order to form the structural mechanism. The floor levels that have uniform reduced girder strengths start at the top floor level and continue down the height of the building.

Figure 2.15 shows the lowest floor level where a uniform reduction in girder strengths is needed to make the structural mechanism the limiting case for each 4-16 story frame. The floor levels above the line on the Figure represent those levels that need reduced girder strengths for the structural mechanism to form. All of the floor levels with reduced girder strength are above the critical story for these parameters of about 65-70% of building height. The combination of the amount of uniform reduction in girder strength and the percent of the building with these reduced girder strengths is relatively uniform. In Figure 2.14 the uniform reduction in girder strength needed to form the structural mechanism decreases linearly from an 8 story building to a 12 story building. Figure 2.15 shows that over this same range, the number of floor levels with reduced girder strengths increases at the same linear rate.

The necessary uniform strength reduction factors are reasonable (approximately 1.50 to 2.70 for this set of parameters). The method of reducing girder strengths a uniform amount also is simple and practical because only the top floor levels will be affected. A general hypothesis can be obtained by using Figures 2.14 and 2.15. The structural mechanism will be the controlling mechanism when the girder strengths in the top one third of floor levels are reduced by a factor of 3.00. These values were chosen because they are conservative limits for all of the cases analyzed. This evaluation applies only for the set of parameters used in this analysis, but it gives a good idea of an effective method to improve frame performance. This method was chosen as the option for improving the performance of reinforced concrete frames under lateral loads. A more detailed analysis using many different parameters was needed to finalize and simplify the hypothesis.

2.5 Parametric Analysis of Initial Hypothesis

A parametric analysis was performed on the initial hypothesis for improving the performance of reinforced concrete frames in earthquakes by reducing column hinging. The initial hypothesis was to decrease the strength in girders uniformly at the upper floor levels of buildings. In the previous section, it was shown that uniformly reducing the girder strengths by a factor of 3.00 in the upper one third of the frames will guarantee the structural mechanism as the limiting mechanism.

However, this was for a specific set of parameters defined in Table 2.1 and Table 2.2. The necessary amount of reduction in girder strength will vary.

A parametric analysis using a limit analysis with a linear load distribution was performed to quantify a relationship between a given set of parameters and the formation of the structural mechanism through the reduction in girder strengths. First, the parameters used in the analysis were selected. The limit analysis was made using this set of parameters. The desired result was a relationship between these parameters and the initial hypothesis that would improve the performance of all frames considered.

2.5.1 Parameters

The parameters that were used in the limit analysis were those which affected the controlling mechanism for a reinforced concrete frame under lateral loads. The parameters were divided into two categories: Constant parameters and variable parameters. Constant parameters are those properties that remained the same for all of the frames analyzed. They were necessary to perform the limit analysis, but are not included in deriving the final parametric relationship to reduce column hinging in the frames. The constant parameters and their values are summarized in Table 2.3.

A concrete compressive strength (f'_c) of 4000 psi was used because it represents an average of typical values found in concrete frame buildings. A linear load distribution was used because it closely represents the displacement profile assumed in the frames and is generally considered representative of actual earthquake loading. The 10 foot story heights and 20 foot bay lengths were not varied because the controlling mechanism was not affected by these dimensions as long as they are uniform throughout the building. Also, the span length is directly proportional to both the column strengths and girder strengths. Therefore, the ratio of column to girder strengths remains relatively unchanged as the span length varies.

Variable parameters are those properties that were varied in order to determine the impact on forcing the structural mechanism to control for a set of buildings. These parameters are used in deriving the final relationship to reduce column hinging in the frames. The variable parameters and the range of values of them that were used are summarized in Table 2.4.

Earlier limit analyses showed that the total number of stories in the building had a significant impact on the controlling mechanism and how much girder strengths needed to be reduced in order for the structural mechanism to become the controlling mechanism. Earlier analysis also showed that the number floor levels with uniform reduction in girder strength ($\beta * N_s$) also has an impact on having the structural mechanism be the limiting case. It also stands to reason that the initial ratio of column strength to girder strength (α) in the frame also has a large impact on how much uniform reduction in girder strengths is necessary. If the column strengths are significantly greater initially than the girder strengths (large value of α) than the uniform reduction in girder strength needed to form the structural mechanism will be relatively small. The α values were determined using Equation 2.5.

The column size (h) and column strength (from ρ) impact the formation of the structural mechanism in a more subtle manner. As axial loads increase in a building the flexural strengths in the columns also increase. The size and amount of reinforcement of the column affect the rate at which the strengths increase for an increasing set of axial loads. This is important because the ratio of column strengths to girder strengths (α) is based on the top exterior column (Equation 2.5) only. Therefore, the yield strengths for the rest of the columns under increasing axial load will vary depending on the size and the reinforcement in the column. The strength in these columns will have an impact on how much the girder strengths need to be reduced in order to form the structural mechanism.

Figure 2.16 shows that the rate at which flexural strength increases with increasing axial load is affected by the column size and reinforcement. The number of bays also affects mechanism formation in a limit analysis. The number of yielded girders will increase as the number of bays increase. Therefore, the base shear required to form the structural mechanism and the intermediate mechanism will be increased. The affect the number of bays has on the mechanism formation decreases significantly as the number of bays increases.

2.5.2 Analysis

A limit analysis with a linear load distribution and the parameters described in the previous section was used to determine a relationship for reducing column hinging by reducing the strength in the upper floor level girders of reinforced concrete frame buildings. The frame geometries were selected by varying the number of stories, the number of bays, the initial α, and the column size and reinforcement. These parameters were varied as shown in Table 2.4. A limit analysis was used on the unmodified frames to determine the controlling mechanism and if modification through the reduction in girder strengths was necessary in order to form the structural mechanism. The frames were not modified if the structural mechanism already was the controlling mechanism.

The frames were modified for the set of parameters if an intermediate mechanism was the limiting mechanism for the unmodified building. The frames were modified by reducing the strength in the girders in the upper floor levels. The girder strengths were reduced at a uniform rate for each floor level with a maximum strength reduction of 4.0.

The girder strengths were reduced in floor levels starting with the top floor only. The reduction in girder strength in the top floor level that was necessary to form the structural mechanism was determined. Then, girder strengths were reduced uniformly in the top 2 floor levels and the reduction in strength necessary to form the structural mechanism was calculated. The uniform reduction factor for girder strength was applied to an increasing number of floor levels and the corresponding necessary reductions in girder strengths were calculated.

Table 2.5 shows a typical portion of the parametric analysis and the determined uniform girder strength reduction factors necessary to make the structural mechanism the controlling mechanism for a given set of parameters. The case shown is for an 8-story 4-bay frame with 18 inch columns and with a column reinforcement ratio (ρ) ranging from 1% to 2%. The α as calculated by Equation 2.5 was varied from 1.0 to 2.0.

In Table 2.5, for the case of α equal to 1.0 and ρ equal to 1%, the girder strength must be reduced in the top 2 floors ($\beta = 0.25$) by a factor of 2.60 in order for the structural mechanism to form. If 3 floor levels ($\beta = 0.375$) are used, then the girder strength reduction necessary to form the structural mechanism drops to 2.0. For the case of $\alpha = 1.0$ and ρ equal to 2%, the girder strength must be reduced in the top 3 floors ($\beta = 0.375$) by a factor of 2.20 in order for the structural mechanism to form. If $\alpha = 2.0$ and $\rho = 1\%$, the girder strength must be reduced in the top 3 floors by a factor of only 1.10 to ensure the structural mechanism as the limiting case. When α was ≥ 3.0 the structural mechanism was already the limiting case and no modifications were necessary.

The limit analysis with linear load distribution was carried out for all combinations of variable parameters. The necessary reductions in girder strengths and the corresponding number of affected floor levels were calculated and tabulated as in Table 2.5. When the limit analysis was completed and the data recorded, a relationship between the parameters that would ensure the reduction in column hinging using a uniform reduction girder strength was investigated.

2.5.3 Results

A relationship between the parameters that would guarantee the reduction of column hinging through the uniform reduction in girder strength was determined.

The variable parameters were tabulated along with the uniform strength reduction factor (R) and the number of affected floor levels ($\beta * N_s$) necessary to form the structural mechanism. Table 2.6 shows the complete tabulated results. The number of stories (N_s), number of bays (N_b) and column size (h) are given. In this case, ρ is represented as a decimal and not as a percent. The value of R is the uniform reduction in girder strength factor. The girder strengths must be divided by the factor R over the corresponding number of floor levels in order to guarantee the structural mechanism as the limiting mechanism. Cases where the unmodified frame already had the structural mechanism as the limiting case were ignored.

A relationship among these parameters (Equation 2.7) was derived to ensure the formation of the structural mechanism for any set of parameters. A linear relationship between the parameters and values of R was estimated. The relationship of the parameters was termed the parameter function. The parameter function is defined as follows:

$$\text{Parameter Function} = \frac{N_s * N_b * \sqrt{\rho * h}}{(N_b + 1) * \beta * \alpha^2} \qquad 2.7$$

N_s = Number of stories

N_b = Number of bays

ρ = Column reinforcement ratio (as a decimal)

h = Column width (inches)

β = Ratio of number of floor levels with reduced girder strength to total number of stories

α = Ratio of top exterior column strength to girder strength

The parameter function was plotted against the uniform girder strength reduction factor (R) necessary to ensure the formation of the structural mechanism as the controlling mechanism. Figure 2.17 shows that the relationship between the parameter function and R is approximately linear for the scattered data points. Each data point is for 1 set of parameters (Table 2.6) and the corresponding necessary uniform reduction in girder strength to reduce column hinging in reinforced concrete frames. Because cases where the combination of parameters already ensured the structural mechanism to form were ignored, the minimum R plotted is 1.1.

A linear upper bound of the data is also shown on Figure 2.17. The linear upper bound is a conservative bound for all the data points of uniform reduction in girder strength. The structural mechanism is guaranteed to be the controlling mechanism for any possible combination of parameters when applying this linear upper bound relationship.

An equation (Equation 2.8) was derived to find a relationship that would guarantee the structural mechanism will be the limiting mechanism for any given set of frame parameters. The equation is the linear upper bound approximation shown in Figure 2.17. Column hinging should be effectively reduced if this equation is applied to girder strengths a requisite amount at a specified number of floor levels. The equation (Equation 2.8) solves for the uniform reduction in girder strength (R) necessary to form the structural mechanism and is calculated as follows:

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2} \qquad 2.8$$

A method for improving the performance of reinforced concrete in earthquakes was developed using limit analysis and a linear lateral load distribution. The strength based relationship developed was to reduce the flexural strength in upper floor girders to encourage the formation of the structural mechanism. The girder flexural strengths in a frame must be reduced by the factor R (Equation 2.8) for a set of parameters in order to eliminate column hinging.

CHAPTER 3. PARAMETRIC ANALYSIS OF RELATIONSHIP TO IMPROVE PERFORMANCE

A relationship for reducing the hinging in columns of reinforced concrete frames under seismic loading by uniformly reducing the strength in girders was developed through the use of a limit analysis in Chapter 2. Equation 2.8 represents the reduction in girder strength necessary in order for the structural mechanism to be the limiting case for any variation of frame parameters. This is a hypothesis that required testing with more complex analyses and for a variety of building geometries.

A series of frames were initially proportioned and then modified using Equation 2.8. Non-linear static analyses and non-linear dynamic analyses were used to evaluate the effectiveness of reducing girder strengths to improve frame performance. The structural analysis program LARZ (Saiidi, 1979; Lopez, 1988) was used for both the static and dynamic non-linear analyses. In the dynamic analysis, ground motion records of 10 earthquakes were applied to the frames. The earthquake records were scaled in order to create a uniform drift response in the frames.

The performance of the frames when modified by reducing girder strength were evaluated by investigating two things: the performance of the columns and the maximum building displacement profile. The performance of the columns was evaluated by identifying possible yielding locations in the columns and determining the controlling mechanism of the structure. The building displacement profile is evaluated by calculating the interstory drift ratios and plotting the displaced shape of the frames. The interstory drift ratio in a building is the relative interstory displacement of a specific story divided by the story height.

3.1 General Analysis Assumptions

The non-linear static and dynamic analyses were calculated using the computer program, LARZ. LARZ was developed at the University of Illinois and can be used to analyze complex reinforced concrete structures under a variety of loading conditions. LARZ has non-linear static and dynamic analysis capabilities.

For the non-linear static analysis, a selected load distribution is applied to the structure at constant increments. The loading is applied to the building until sufficient member yielding has occurred to form a mechanism and the frame is "pushed" over. The building deformations and yielded members are recorded at each loading increment to determine the formation of the mechanism and the deformed shape of the structure at that point.

Table 3.1 lists some of the assumptions that were used in the analyses. The stiffness and strength of each structural element were determined. The moment of inertia (I) and the shear area (A_v) of each member were calculated. The product of the moment of inertia and the elastic modulus (EI) constitute the elastic stiffness of each element. The shear area of each column was approximated as 5/6 of the gross area. The shear area of the girders was approximated as the area of the rectangular part of the girders.

A tri-linear strength relationship between moment (M) and curvature (ϕ) was assumed. Figure 3.1 shows an example of the points on the tri-linear strength graph for a typical girder. The example moment curvature points on Figure 3.1 are summarized in Table 3.2. The first point of the tri-linear relationship was at the cracking moment of each element (M_{cr}). The cracking moment was assumed by the following equation:

$$M_{cr} = \frac{7.5\sqrt{f'_c}I}{c} \qquad 3.1$$

f'_c (psi) = Compression Strength of Concrete

I (in^4) = Moment of Inertia of Element

c (in) = Distance from Neutral Axis to Face of Element

The curvature at cracking (ϕ_{cr}) is calculated internally by LARZ as M_{cr} divided by the elastic stiffness of the member (EI). The M_{cr}-ϕ_{cr} point is the first of the three points needed for the tri-linear graph (Figure 3.1-A).

The next point in the tri-linear relationship is the moment at yield (M_y) and the curvature at yield (ϕ_y). These values are calculated by the equations in Section 2.2.1. The third point in the tri-linear strength relationship is located at a point anywhere beyond the point of yielding (Figure 3.2-C). These values of moment (M_{ult}) and curvature (ϕ_{ult}) are calculated so that the post yield slope is equal to 1% of the yield slope. The post yield slope is the slope of the line between the yield and ultimate moments. The yield slope is the slope of the line between the cracking and yield moments. The post yield slope is calculated by the following equation:

$$\text{Post Yield Slope} = \frac{M_{ult} - M_y}{\phi_{ult} - \phi_y} = 1\% \frac{M_y - M_{cr}}{\phi_y - \phi_{cr}} \qquad 3.2$$

The yield moment for the girders was assumed to be the average of the positive and negative moment flexural strengths. The tri-linear strength relationship for each structural element was assumed to be the same at any point along the member.

Most of the user input is the same in the dynamic and static version of LARZ. The frame geometry, material properties, member stiffness, and member strength relationships are required. Rayleigh damping factors based on the first and second mode periods of the structure were calculated. The first and second mode periods were calculated using uncracked sections. The Rayleigh damping factors A and B were calculated by the following equations (Clough & Penzien, 1993):

$$A = \frac{2\omega_1\omega_2(s_2\omega_1 - s_1\omega_2)}{(\omega_1^2 - \omega_2^2)} \qquad 3.3$$

$$B = \frac{2(s_1\omega_1 - s_2\omega_2)}{(\omega_1^2 - \omega_2^2)} \qquad 3.4$$

ω_1 = Natural frequency of the 1st mode = $2\pi / T_1$

ω_2 = Natural frequency of the 2nd mode = $2\pi / T_2$

T_1 = Natural period of 1st mode

T_2 = Natural period of 2nd mode

s_1 = Damping Coefficient for 1st mode = 0.02

s_2 = Damping Coefficient for 2nd mode = 0.02

The hysteretic model that was used in this analysis was the Takeda hysteretic model (Takeda, 1970). The exponent of the unloading slope for the Takeda hysteretic was 0.4. For this study, the stiffness was recalculated at every response point. Accelerations recorded during ten earthquakes were used to determine the maximum response of the frames for the duration of the earthquakes.

3.2 Parameters

The parameters used in the non-linear static and dynamic analyses were selected. These parameters were selected because they will have the largest impact on the behavior of the frames during earthquakes. The parameters pertaining to the general geometry of the frames were determined to be the number of stories (N_s) and the span length (L_s). The size of the columns (h) was selected as the stiffness parameter. The ratio between the column strengths and girder strengths (α) was selected as the strength parameter. The column size and strength varied as the number of stories varied. The girder size and strength varied as the span length varied.

3.2.1 General Frame Geometry

There are sixteen frame cases that were used in the non-linear static and dynamic analyses. The sixteen cases vary depending on the number of stories, the span lengths, and the column stiffness factor, K, which will be described in Section 3.2.3. The study included 4, 8, 12, and 16 story buildings with either 20 or 30 foot spans and a column stiffness factor of either 0.35 or 0.20. Therefore, there are 4 separate frame cases for the 4, 8, 12, and 16 story buildings. Table 3.3 summarizes the general parameters and coding of the sixteen frame cases. There are also some additional frame geometry parameters that are the same for all of the sixteen frame cases. The non-linear static and dynamic analyses included 4 bays ($N_b = 4$), 10 foot uniform story heights, and a compressive strength of concrete (f'_c) of 4000 psi for each building case.

3.2.2 Girder and Column Dimensions

The structural elements consist of T-shaped girders with an effective slab width and square columns. The girders have dimensions illustrated in Figure 3.2. The total girder depth (D_g) was calculated by the following equation:

$$D_g = \frac{L_s}{10} \qquad 3.5$$

L_s = Bay Span Length (inches)

The total girder depth includes the slab portion (flange) of the girder. A slab depth (D_s) of 6 inches was used in the analysis. The bottom width of the girder (W_b) was calculated by the following equation:

$$W_b = \frac{L_s}{20} \qquad 3.6$$

The top width of the girder (W_t) is equivalent to the effective slab width of the girder. This effective top flange width is based on a 45 degree projection and can be calculated by the following equation:

$$W_t = 2(D_g - D_s) + W_b \qquad 3.7$$

The non-linear static and dynamic analyses included frames with span lengths (L_s) of 20 and 30 feet. These span lengths were chosen to be representative bay lengths of typical reinforced concrete frame buildings. Table 3.4 summarizes the dimensions of the girders used for each of these spans.

The columns used in the non-linear static analysis were square sections. The column size was based on the maximum axial service load ($P_{s,max}$) which is the axial load applied to the bottom story interior column for a particular frame. The dead plus live load service weight of each floor is 160 psf. The column area (A_{col}) is calculated by the following equation:

$$A_{col} \geq \frac{P_{s,max}}{Kf'_c} \qquad 3.8$$

N_s = Number of Stories

L_s = Span Length

$$P_{s,max} = 160 N_s L_s^{\;2} \quad \text{(lbs)} \tag{3.9}$$

$f'_c = 4000$ psi

K = Constant of Stiffness

Values for K of 0.35 for flexible columns and 0.20 for stiff columns were chosen to be used in the non-linear static and dynamic analyses. Since the columns are square, the column width (h) can be calculated as:

$$h = \sqrt{A_{col}} \tag{3.10}$$

The non-linear static and dynamic analyses included 4, 8, 12, and 16 story buildings with 20 and 30 foot spans. The 12 and 16 story buildings have a change in stiffness at the mid-height of the building (the column size was reduced). The column sizes were changed so that the frames were more economically sized for gravity loads. For example, the 12 story building has one column size for the bottom 6 stories of the building based on a 1st floor $P_{s,max}$ and a second column size for stories 7-12 based on a 7th floor $P_{s,max}$. The change in column size was similar for the 16 story buildings. Table 3.5 summarizes the column dimensions used in the proportioned frames. The number of stories (N_s), span length (L_s), and column stiffness factor (K) for each frame case can be found in Section 3.2.1.

3.2.3 Girder and Column Strength Parameters

The sixteen frame cases with their given column dimensions along with the appropriate girder dimensions for both span lengths were used in the non-linear static and dynamic analyses. Strength requirements had to be checked in the girders and columns. The steel reinforcement was designed to ensure that the members were satisfactory under factored gravity loads. The girders were designed for flexure and the columns were designed for both axial and flexural limit states. The girders were designed to resist factored dead and live loads. The dead loads included the weight of the 6 inch slab, the weight of the girders, and a superimposed dead load of 10 psf. The superimposed dead load takes into account the dead load due to nonstructural elements.

The design live load was based on the requirements of the American Society of Civil Engineer's Code, Minimum Design Loads for Buildings and Other Structures (A.S.C.E., 1995). The reinforced concrete frame buildings in this analysis were assumed to be office or hotel type structures. Table 4.1 of the A.S.C.E. code requires that lobbies of office buildings have a design live load of 100 psf while the actual offices have a design live load of 50 psf. Table 4.1 of the A.S.C.E. code also requires that the public rooms and corridors of hotels have a design live load of 100 psf while the private rooms and corridors of hotels have a design live load of 40 psf. An approximate average of these values was taken and a design live load of 80 psf was used in this analysis.

The 80 psf live load was reduced using the equations of Section 1607.6 of the Uniform Building Code (U.B.C., 1997). The dead load and reduced live load were factored into the ultimate load (W_u) by using Equation 9.1 of the A.C.I. building code (A.C.I. 318-99):

$$W_u = 1.4DL + 1.7LL \tag{3.11}$$

DL = Total Dead Load

LL = Reduced Live Load

The design moments were calculated based on the moment coefficients given in Section 8.3.3 of the A.C.I. building code (A.C.I. 318-99). Maximum negative (M_{max-}) and positive (M_{max+}) design moments were calculated by the following:

$$M_{max-} = -\frac{W_u L_c^{\ 2}}{10} \tag{3.12}$$

$$M_{max+} = \frac{W_u L_c^{\ 2}}{14} \tag{3.13}$$

L_c = Clear Span Length = $L_s - h$ (Column Width)

These design moments were used to determine the required amount of reinforcement necessary in the girders. The top or negative moment reinforcement ratio (ρ_{top}) was selected to be 1.00% and the bottom or positive moment reinforcement ratio (ρ_{bottom}) was selected to be 0.75%. This amount of girder reinforcement is sufficient to resist the factored gravity loads in the 2 girder cases used in this analysis. The yield moment was calculated for both girder cases by using the equations of Section 2.2.1. Table 3.6 shows the strength parameters for the 2 sizes of girders used in the analysis.

The columns were designed for axial loads and flexural loads. The columns were designed for the maximum axial load which occurs at the bottom story of the building. The moment applied to columns from unbalanced live loads was considered by increasing the axial load by an additional 10%. The total axial dead load and reduced axial live load were combined and factored into the ultimate axial load (P_u) by applying Equation 3.11. The required column steel ($A_{s,col}$) was designed to resist the ultimate axial load by using A.C.I. Equation 10.2 which assumes the columns have tied transverse reinforcement (A.C.I. 318-99):

$$A_{s,col} \geq \frac{\frac{P_u}{(0.8\phi)} - 0.85f'_c A_{col}}{f_y - 0.85f'_c} \qquad 3.14$$

$\phi = 0.70$ for Tied Columns

$f_y = 60$ ksi

$f'_c = 4$ ksi

A_{col} = Gross Area of Column

The column reinforcement ratio (ρ_{col}) was determined to be 2.00% distributed equally in the top and the bottom of each column section. This amount of column reinforcement is sufficient to resist the factored gravity loads in the sixteen frame cases used in this analysis (Equation 3.14). The column yield moment capacity was calculated for each case by using the equations of Section 2.2.1. The column yield moment capacity varied depending on the axial load. The reduction in girder strength equation (Equation 2.8) was derived based on the yield moment of the top story exterior column, which is the minimum column capacity in a frame. Therefore, the column yield moment (M_c) is based on the top story exterior columns. Table 3.7 shows the column strength parameters for the sixteen frame cases used in the analysis.

3.3 Non-Linear Static Analysis

The hypothesis to improve performance of reinforced concrete frames in earthquakes is to reduce the strength in some girders in order to encourage the formation of the structural mechanism through the use of Equation 2.8. The formation of the structural mechanism as the limiting mechanism improves performance by reducing column yielding and creating a linear displaced building shape. The first analysis that was used to test the hypothesis was a non-linear static analysis. The non-linear static analysis is an effective way of measuring the performance of a building that is independent of ground motions. LARZ was used to calculate the structure and member response under a static linear load for the proportioned frames.

The frames were analyzed as unmodified and then modified by reducing the girder strengths using Equation 2.8. The flexural girder strengths in the unmodified frames are divided by the factor, R, to determine the necessary modified girder strengths. The response of the structures was calculated and compared for both the unmodified and the modified conditions. The interstory drift ratios and the controlling mechanism were used to determine if the performance in the frame was improved by applying Equation 2.8.

3.3.1 Analysis

A non-linear static analysis was completed for sixteen frames with the parameters discussed in Section 3.2. The dimensions and non-linear strength parameters of each element were included in LARZ along with the frame geometry and loading condition. The girder flexural strength in the unmodified buildings was uniform for every floor level. The column strength varied due to the different axial load applied to each column. These unmodified buildings were expected to exhibit substantial column yielding and an intermediate mechanism as the controlling mechanism.

The sixteen frames were modified to improve performance. The flexural strengths in upper floor level girders were reduced by a factor calculated by Equation 2.8, while the other frame parameters remained the same. A non-linear static analysis was performed on these modified buildings. The response of the unmodified and modified frames were compared to determine if the hypothesis of reducing girder strengths was successful in improving the performance of reinforced concrete frame buildings in earthquakes.

The uniform reduction in girder strength (R) necessary to improve the performance of reinforced concrete frame buildings in earthquakes by ensuring the formation of the structural mechanism was calculated for each of the sixteen frames by applying equation 2.8 which follows:

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2} \qquad 3.15$$

The flexural girder strengths in the unmodified frames were divided by the factor R to determine the necessary modified girder strengths. The reduction factor (R) depends on several parameters. In this analysis, the number of bays (N_b) = 4 and the column reinforcement ratio (ρ) = 0.02 for all sixteen frames in order to minimize the number of different frames that were analyzed. The number of stories (N_s), the column width (h), the ratio of number of stories with reduced girder strength to the total number of stories (β), and the ratio of top exterior column yield strength to girder yield strength (α) varied for the sixteen frame cases. Therefore, the reduction in girder strength (R) will vary for each case depending on these parameters.

Equation 2.8 was derived for frames with uniform column dimensions. In this analysis the 12 and 16 story frames have a change in column size at mid-height. Therefore, the reduction in girder strength equation was modified for buildings with a change in column stiffness.

$$R = 1 + \frac{N_{s,eff} * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2} \qquad 3.16$$

The effective number of stories ($N_{s,eff}$) is the number of stories from the top of the building to the point where the stiffness has changed. In this analysis, the columns change size at the mid-height of the building. Therefore, the $N_{s,eff}$ will be 6 for the twelve story frames and 8 for the sixteen story frames. The column size (h) and reinforcement ratio (ρ) of the top story exterior column are used in the equation. The value of α is based on the flexural strength of the top story exterior column and β is the ratio of the number of floor levels with reduced girder strengths to the total number of stories (N_s).

Table 3.8 shows the amount of reduction in girder strength (R) necessary to ensure that the structural mechanism will be the controlling mechanism for all sixteen frame cases by using equations 2.8 and 3.16. The flexural girder strengths in the unmodified frames were divided by R to determine the necessary modified girder strengths. M_c is the top story exterior column flexural strength and M_g is the girder flexural strength for 20 and 30 foot spans. The girder flexural strength is uniform for the unmodified frames.

The girder strengths were reduced at floor levels starting at the top level of the building and proceeding down. For example, a 4 story frame will have the flexural strengths reduced in the girders of the 3rd and 4th floors. The values of β were adjusted so that the reductions in girder strength were as reasonable as possible. The girder strength reduction factor (R) decreases as β increases. A maximum β of 0.50 was selected so that as few girders as possible would have to be affected by the reduction in their flexural strengths.

Table 3.8 indicates that the parameter with the greatest impact on the amount of reduction in girder strength (R) necessary to form the structural mechanism is the ratio of column strength to girder strength (α). In general, as α increases the value of R decreases. The reduction factor (R) is the same for the 8 story frame cases and the 16 story frame cases. This is due to the sixteen story building having a change in column stiffness at the mid-height. The sixteen story building is essentially treated as an 8 story building on top of another 8 story building in Equation 3.16. The strength and stiffness parameters for the 8 story building and the top half of the 16 story building are equivalent.

The modified girder strengths for the upper floor levels were calculated by applying the appropriate reduction (R) from Table 3.8. The number of floor levels where the girder strengths are reduced are summarized in Table 3.8. The unmodified girder strengths (M_g from Table 3.6) were divided by R to determine the modified girder strengths. The girder strengths in the lower floor levels were unchanged. Table 3.9 shows the reduced girder strengths ($M_{g,red}$) and the reinforcement necessary ($A_{stop,red}$ and $A_{sbott,red}$) to achieve the reduction in girder strength for each of the sixteen cases. The reinforcement in the unmodified girders is summarized in Table 3.6. These reduced girder strengths should force the structural mechanism to be the limiting mechanism.

The reinforcement in the modified girders had to be checked to see if they were adequate for gravity loads. Using the procedure described in Section 3.2.3, the minimum amount of girder reinforcement ($A_{s,min}$) necessary to resist factored gravity loads was calculated for the cases of 20 and 30 foot bay span lengths. Table 3.10 shows the results of this analysis. The minimum amount of girder reinforcement for the 20 foot spans was 1.57 in^2 and 1.07 in^2 for the top and bottom steel respectively. The minimum top and bottom steel necessary for the 30 foot spans was 3.29 in^2 and 2.25 in^2 respectively.

The girder strengths are reduced by R to ensure that the structural mechanism will control for each of the frames. In four of the sixteen frame cases, the reinforcement in the modified girders ($A_{s,red}$) is less than the reinforcement necessary ($A_{s,min}$) to resist factored gravity loads. These cases are 1an, 1bn, 3an, and 3bn. The modified girders are those with strengths reduced by the R values in Table 3.8 The modified girders in these 4 cases used the minimum reinforcement shown in Table 3.10. Therefore, the reduction in girder strength used in these cases will be less than the reduction (R) prescribed by Equation 2.8 and 3.16. Table 3.11 summarizes the modified girders used for each of the 16 frame cases including those with the minimum steel for gravity loads

The non-linear static analysis was performed on the unmodified and modified frames using LARZ. The parameters described in Section 3.2 were entered into the static version of LARZ as described in Section 3.1.1. A linearly applied lateral force was used in the analysis. The unmodified and modified frames have the same parameters except the modified frames have upper floor girders with a reduced flexural strength. The building response of the unmodified and modified cases were calculated and compared to determine whether reducing the girder strengths improves the performance of reinforced concrete frame buildings during earthquakes.

3.3.2 Results

Non-linear static analyses were completed for the sixteen frames. The frames were analyzed as unmodified with uniform girder strengths and as modified with the girder strengths in the upper floor levels reduced by the factor R. The building response of the unmodified and modified cases were calculated and compared to determine if reducing the strength in upper floor level girders improves the performance of reinforced concrete frame buildings under lateral loads. The reduction in girder strength by a factor of R should ensure that the structural mechanism will be the controlling mechanism for each of the sixteen modified frame cases.

The performance of the frames was based on the determined controlling mechanism and the interstory drift ratios. The interstory drift ratio (Δ_{idr}) is calculated by the following equation:

$$\Delta_{idr,i} = \frac{100(\Delta_{i+1} - \Delta_i)}{H_i} \qquad 3.17$$

$\Delta_{idr,i}$= Interstory Drift Ratio of Story i (%)

Δ_i = Displacement at Level i

H_i = Height of Story i

Large values of Δ_{idr} values will lead to significant story distortion and damage to structural and non-structural elements. When the structural mechanism is the controlling case, the Δ_{idr} values should be approximately constant. The performance of the frames are improved when the Δ_{idr} values are minimized. The Δ_{idr} values in the lower stories of the frames are especially important. If the lower stories have large Δ_{idr} values then the frame will experience more damage than if the large Δ_{idr} values were located in the upper stories because of the larger axial stresses on the columns. The value and location of the maximum interstory drift ratios ($\Delta_{idr,max}$) for each frame were calculated and compared.

The controlling mechanism, interstory drift ratios, and other aspects of the building response were calculated for the unmodified and modified cases. Equation 2.8, the reduction factor formula, was derived to encourage the structural mechanism to control for any frame case. The non-linear static analysis will determine if forming the structural mechanism will improve the performance of the frame through lower values of Δ_{idr} in the lower stories. The building response was calculated at each loading increment. The building displaced shape, roof displacement, base shear and interstory drift ratios were recorded at the level of force that caused the first mechanism formation.

Table 3.12 shows the controlling mechanism for the sixteen unmodified and modified frame cases. The location of the intermediate mechanism is shown as a percent of the total building height. The table shows that when the girder strengths were reduced by R (modified frames), the controlling mechanisms change from an intermediate mechanism to the structural mechanism in every frame case under a linear lateral load. The structural mechanism formed in all of the cases where the minimum girder steel was used instead of the amount required by Table 3.9. This is due to Equation 2.8 being derived to be a conservative reduction in girder strength (R) that will guarantee the formation of the structural mechanism.

The results of the non-linear analysis agree with the results of the limit analysis. When the girder flexural strengths in the upper floor levels of the frames were reduced by the amount prescribed in Table 3.8 (R) the structural mechanism formed with an applied linear lateral load. The formation of the structural mechanism should lead to an improved building response in the modified frames. Therefore, the performance is improved in the modified frames by applying the hypothesis.

Figure 3.3 (a-d) shows plots of relative base shear ($V_{b,rel}$) versus the mean drift ratio (Δ_{mdr}) for each of the sixteen unmodified and modified frame cases. The relative base shear is the total base shear strength of the frame divided by the total weight of the building as a percent. The mean drift ratio is the roof displacement divided by the total height of the frame as a percent. Equations 3.18 and 3.19 show how $V_{b,rel}$ and Δ_{mdr} were calculated.

$$V_{b,rel} = \frac{100V_b}{W_{tot}} \qquad 3.18$$

$V_{b,rel}$ = Relative Base Shear (%)

V_b = Base Shear

W_{tot} = Total Weight of the Frame

$$\Delta_{mdr} = \frac{100\Delta_{roof}}{H_{tot}} \tag{3.19}$$

Δ_{mdr} = Mean Drift Ratio (%)

Δ_{roof} = Roof Displacement

H_{tot} = Total Building Height

The plots of relative base shear versus the mean drift ratio indicate that the modified frames reach a yield point at a lower base shear than the unmodified frames. This means that a mechanism will form at an earlier time in the cases where the girders strengths are reduced. All sixteen frames begin to yield and form a mechanism at mean drift ratios of less than approximately 1.00%. The performance of the frames can not be determined by these plots. They are just an indicator of the general frame response of the unmodified and modified frames.

Figure 3.4 (a-d) shows the relative building displacement profile for each of the sixteen frame cases. These graphs plot the normalized drift (Δ_{norm}) for each floor level in the frame. The normalized drift is the displacement at a given floor level divided by the total height (H_{tot}) of the building as a percent. The drifts at each floor level were recorded at the level of forced that caused the first sway mechanism to form for the unmodified and the modified cases. The plots illustrate the displacement profile of the frames at the formation of the controlling mechanism.

The displacement profiles of the modified frames are approximately linear. This is because the structural mechanism controls for all sixteen modified cases and the buildings should displace with columns acting as a stiff, unyielding spine. The unmodified frames have drift profiles that correspond to the intermediate mechanism that controls. For example, the unmodified frame case 1bn has a 3rd story intermediate mechanism as the controlling mechanism (Table 3.12). The building displacement profile of the unmodified frame 1bn (Figure 3.4a) illustrates a displacement profile that matches the expected profile for a 3rd story mechanism. The frame is significantly displaced from the bottom of the building to the 3rd story and then displaced at an approximately constant rate from the 3rd story to the top of the building.

In general, the frames that were modified by having reduced girder strengths displaced more desirably than the unmodified frames. The slopes of the drift profile for the modified cases are less than the slopes of the drift profile for the unmodified cases. This indicates that the floor levels are being distorted less for the modified cases, especially in the bottom floor levels. The performance of the frames are being improved by reducing the strength in girders in the modified cases.

Figure 3.5 (a-d) shows the interstory drift ratios (Δ_{idr}) for all sixteen modified and unmodified cases. The Δ_{idr} is the most important measurement of frame performance under lateral loads. The interstory drift ratios are calculated using Equation 3.17. The displacements at each floor level were recorded at the time when the first sway mechanism forms for the unmodified and the modified cases. The graphs show the Δ_{idr} for each story of the frames. The larger the Δ_{idr} value the more distortion exists in the story between the two prescribed floor levels. The actual absolute values of Δ_{idr} are insignificant for the non-linear static analysis because the values are recorded at the formation of the first sway mechanism. This may occur at a very large loading increment that will never be present in response to an actual ground motion.

The interstory drift ratios (Δ_{idr}) are approximately uniform for the modified frame cases with reduced girder strengths. This is because the structural mechanism controls for the modified cases resulting in approximately uniform values of Δ_{idr} over the height of the building. The interstory drift ratios of the unmodified frames are influenced by the intermediate sway mechanism that forms. In general, the values of Δ_{idr} for the unmodified frames is the greatest in the lower stories of the buildings. The Δ_{idr} values begin to decline from the level of the intermediate mechanism to the top of the building.

In general, when the frames are modified by reducing the strength in girders, the values of Δ_{idr} in the lower stories decrease and the values of Δ_{idr} in the upper stories increase. This is a result of the structural mechanism causing a more uniform distribution of interstory drift over the building height for the modified frames. The reduction of Δ_{idr} in the lower stories will greatly improve the performance of the buildings.

Figure 3.6 (a-b) shows the change in Δ_{idr} for all of the sixteen frame cases. The change in Δ_{idr} (%) is equal to the unmodified value of Δ_{idr} (%) minus the modified value of Δ_{idr} (%) for a given story. Therefore, a positive change in Δ_{idr} means that the value of Δ_{idr} is larger in the unmodified case compared to the modified case and a reduction in Δ_{idr} has taken place. These figures illustrate that the interstory drift ratios (Δ_{idr}) are being significantly reduced in the lower stories whenever the girder strengths are reduced (modified cases). This will improve the performance of these frames in earthquakes.

The results shown in Figure 3.6 (a-b) illustrate an improvement in the performance of reinforced concrete frame buildings under a static linear load. The Δ_{idr} values are greatly reduced in the lower stories of the frames modified by reducing the strength in girders by the factor R. The results of Figure 3.6 (a-b) for each of the individual sixteen frame cases are important, but it is also necessary to form a general performance evaluation of all the frames cases together.

A method to form a general performance evaluation of all the frames is to consider the maximum interstory drift ratio ($\Delta_{idr,max}$) of each frame. The maximum interstory drift ratio is the maximum story distortion in the building. The maximum interstory drift ratios of the unmodified and modified frames were calculated and compared. The difference in the amount and location of the $\Delta_{idr,max}$ is important in determining if the frame's performance is improved by reducing the girder strengths in the upper floor levels. The performance of the frames has improved if the values of $\Delta_{idr,max}$ decrease and the location of $\Delta_{idr,max}$ more up the building for the modified cases. The values of $\Delta_{idr,max}$ for the bottom half stories of the frames were also calculated and compared for the unmodified and modified frame cases because the interstory drift ratios of the lower half of the building are especially important to estimate the amount of damage expected to a building during response.

Figure 3.7 shows the change in the maximum interstory drift ratio for the sixteen frame cases. The change in $\Delta_{idr,max}$ (%) is equal to the $\Delta_{idr,max}$ (%) of the unmodified cases minus the $\Delta_{idr,max}$ (%) for the modified cases. A positive change in $\Delta_{idr,max}$ means that the maximum interstory ratio is larger for the unmodified case. Figure 3.7 illustrates that the maximum interstory ratio ($\Delta_{idr,max}$) decreases for every case except for one 4-story frame (1an) with slim columns. Therefore, reducing the strength in girders will reduce the maximum interstory drift ratios of most reinforced concrete frames under a linear lateral load.

Figure 3.8 shows the change in maximum interstory drift ratio for the stories in the bottom half of the frames. The response of the bottom stories of a frame are especially important to the performance of the building. Large values of Δ_{idr}, max in the bottom stories of a building can cause significant damage when the sway mechanism is reached. The difference in the values of $\Delta_{idr,max}$ located in the bottom half of the unmodified and modified frames were calculated. Figure 3.8 shows that the maximum interstory drift ratios in the frame's bottom half stories is reduced for all cases when the girder strengths are reduced. This reduction in $\Delta_{idr,max}$ for the bottom stories is critical in improving the performance of reinforced concrete frames under a linear lateral load.

Table 3.13 summarizes the location of the maximum interstory drift ratios for each of the sixteen frames. The location of the maximum interstory drift ratio is presented as a percentage of the total building height. The difference in location is calculated by subtracting the location of the $\Delta_{idr,max}$ in the unmodified frames from the location of the $\Delta_{idr,max}$ in the modified frames. Therefore, a positive change in location means that the location of the $\Delta_{idr,max}$ was higher in the building for the modified case. Table 3.13 shows that the location of the $\Delta_{idr,max}$ either moved up the building or remained unchanged when the frames were modified by reducing the girder strengths. It is beneficial to have the Δidr,max occur as high in the building as possible in order to reduce damage.

3.3.3 Summary of Static Analysis

A non-linear static analysis was performed on sixteen different frame cases using LARZ. The frames were analyzed as unmodified and modified by reducing the girder strengths in the modified frames. The results of the static analysis show that when the flexural strength in the upper floor girders are reduced by R, the performance of the frame is improved under a linear lateral load. The structural mechanism formed for all of the sixteen modified cases. An approximately linear drift distribution occurs as a result of the formation of the structural mechanism. The interstory drift ratios (Δ_{idr}) decrease significantly in the lower stories when the frames are modified. The maximum interstory drift ratios ($\Delta_{idr,max}$) decrease and the location of the $\Delta_{idr,max}$ is higher whenever the frames are modified by reducing the girder strengths.

3.4 Non-Linear Dynamic Analysis

The hypothesis to improve performance of reinforced concrete frames in earthquakes is to reduce the strength in some girders in order to guarantee the formation of the structural mechanism through the use of Equation 2.8. The formation of the structural mechanism as the limiting mechanism improves performance by reducing column yielding and creating a linear displaced building shape. The second analysis that was used to test the hypothesis was a non-linear dynamic analysis.

The non-linear dynamic analysis is an effective way of measuring a building response under the loading of actual recorded ground motions. The frames were analyzed as unmodified and then modified by reducing the girder strengths using the procedure described in Section 3.3.1. The flexural girder strengths in the unmodified frames are divided by the factor R to determine the necessary modified girder strengths. The unmodified and modified frames have the same parameters as used in the non-linear static analysis (Section 3.3).

The ground motions of ten earthquakes were used in the analysis. The ground motions of these earthquakes were applied to the sixteen frames as unscaled and scaled accelerations. The earthquake records were scaled in order to encourage similar drift responses in the frames. The 1st and 2nd mode periods of the sixteen frame cases were calculated and used in the input to the LARZ dynamic program.

LARZ was used to calculate the structure and member responses to the applied ground motions. The maximum response of the structures was calculated and compared for both the unmodified and the modified conditions. Since the building is loaded in both directions (unlike the incrementally loaded static analysis) the maximum building response can occur at any time during the loading. The maximum interstory drift ratios that occur during the ground motions were calculated for the unmodified and modified frames to determine if the performance had improved.

The maximum column and girder rotational ductilities were calculated and used to determine if the structural members had yielded and if a sway mechanism had formed at any time during the ground motion. The change in the maximum column rotational ductilities in the modified frames was calculated to determine if the performance of the frames had improved through a reduction in column yielding.

3.4.1 Analysis

The non-linear dynamic analysis uses the ground motion records of ten earthquakes. The dynamic version of LARZ was used to calculate the maximum building and member response under the seismic loads. The sixteen frame cases were analyzed for all ten earthquakes. The frames were first analyzed as unmodified using the parameters detailed in Section 3.2. The frames were then modified by reducing the girders strengths in upper floor levels by the factor R as described in Section 3.3.1. The frame parameters are the same in the non-linear dynamic analysis as they were in the non-linear static analysis

The non-linear dynamic analysis was used to determine the building responses under actual ground motions. The hypothesis of reducing the strength in girders in order to guarantee the formation of the structural mechanism was derived for a linearly applied lateral load. The lateral forces calculated using the ten earthquakes are not necessarily equivalent to a linear load distribution. The demands created by the ten earthquakes vary and may or may not even cause the formation of a mechanism in the buildings. The maximum building response was used to determine whether the hypothesis of improving performance in reinforced concrete frames by reducing the girder strengths by R was successful under actual earthquake loads.

Table 3.14 lists the ten earthquakes and the three letter codes used to identify each seismic event that were used in the non-linear dynamic analysis. The table also shows the location and total duration of each earthquake. The time step is the time increment that the ground motions were recorded for the ten earthquakes. The peak ground acceleration (P.G.A.) is the maximum acceleration recorded for each earthquake. The peak ground accelerations are given as a fraction of the acceleration of gravity (g).

Ten earthquakes were applied to the sixteen frame cases as both unscaled and scaled ground motion records. The unscaled records are the actual recorded ground motions for each of the ten earthquakes. The earthquake records were scaled in order to create similar drift responses in the frames (Figure 3.9). The records were scaled by multiplying the accelerations of each earthquake by a scale factor. Table 3.15 shows the scale factors and resulting P.G.A. values of the ten scaled earthquakes. The P.G.A. values are given as a fraction of the acceleration of gravity (g) which equals 386 in / sec^2.

The damping factors A and B which are calculated by Equations 3.3 and 3.4 were determined for each of the sixteen frame cases. The calculation of the damping factors requires the determination of the 1st and 2nd mode natural frequencies (ω) or periods (T) of the frames. The first and second mode periods were calculated for the frames by using uncracked section properties.

Table 3.16 shows the 1st and 2nd mode periods and the Rayleigh damping factors A and B for each of the sixteen frame cases. The calculations are based on the stiffness properties of the structural elements and the geometry of the frames. The unmodified and modified frames had the same periods because their only difference is in girder strengths in the upper stories. The damping coefficient for the 1st and 2nd modes was assumed to equal 0.02 of critical damping.

3.4.2 Results

A non-linear dynamic analysis was performed on the sixteen unmodified and modified frame cases using ten unscaled and scaled earthquakes as the applied loading. The building response was calculated throughout the duration of each earthquake. The maximum building response during the duration of the earthquakes was used to assess the performance of the frames. The maximum interstory drift ratios (Δ_{idr}) were calculated and compared for each of the sixteen frames to determine if the performance was improved by reducing the girder strengths in the modified frames. The interstory drift ratios were calculated by Equation 3.17.

The reduction in girder strengths by the factor R was derived to guarantee the formation of the structural mechanism under a static linear load. The location and amount of yielding in a frame varies according to the frequency content, duration, and intensity of every ground motion. The performance of the frames was based on the maximum ductilities of each element. The column ductility is a ratio of the maximum rotation a column undergoes during the duration of the earthquake to the rotation the column experiences when it yields. Equation 3.20 shows the calculation of column ductility:

$$\mu_{col} = \frac{\theta_{col,max}}{\theta_{col,yield}} \qquad 3.20$$

$\theta_{col,max}$ = Maximum Column Rotation During Earthquake

$\theta_{col,yield}$ = Column Rotation at Yield

The maximum ductilities give an indication of the amount of deformation each member has undergone for the duration of the earthquake. A reduction in column ductilities would indicate that the columns are less likely to be yielded, which is an important parameter for considering frame performance during earthquakes. Even if no columns have yielded, the ductilities will indicate how close the columns are to reaching their yield point. The change in column ductilities between the unmodified and modified cases will determine if the performance in the frames is being improved.

The maximum building response was calculated for each of the sixteen frame cases which were subjected to ten unscaled and ten scaled earthquake ground motions. The building displacement profile and the interstory drift ratios were calculated at each time step during the duration of each earthquake. The maximum normalized drifts (Δ_{norm}) and maximum interstory drift ratios (Δ_{idr}) recorded during the duration of the earthquakes were calculated for each floor level and story. These values were compared for all ten unscaled and scaled earthquakes to determine what effect the reduced girder strengths had in affecting the performance of the modified frames.

Figure 3.10 (a-d) shows the maximum building response for one of the 4, 8, 12, and 16 story frames loaded with the ground motion record of the unscaled and scaled Loma Prieta earthquake. Similar calculations and plots were made of every frame under every earthquake. The building response of every frame under every earthquake are not presented in this report for brevity. Instead a sample of the calculations is presented for the 4, 8, 12, and 16 story buildings with 30 foot bays and a column stiffness factor of 0.35 using the accelerations of the Loma Prieta earthquake. The frame case codes for these frames are 1bn, 2bn, 3bn, and 4bn.

Figure 3.10 (a) shows the maximum displacement profile for the frames with the unscaled Loma Prieta earthquake. The graphs plot the normalized drift (Δ_{norm}) versus floor level. The normalized drift is equal to the maximum drift that occurs at a floor level for the duration of the earthquake divided by the total height of the building. Figure 3.10 (c) shows the same maximum building displacement profile for the scaled Loma Prieta earthquake. The maximum displacement profiles of these frames indicate that the unmodified frames are displacing as if an intermediate mechanism controls and the modified frames are displacing linearly, suggesting the structural mechanism has formed. There is larger displacements for the 12 and 16 story frames under the scaled records as compared to the unscaled records.

Figure 3.10 (b) shows the maximum interstory drift ratios (Δ_{idr}) of each story for the frames with the unscaled Loma Prieta earthquake. The maximum values of Δ_{idr} for the duration of the earthquake were calculated using Equation 3.17. Figure 3.10 (d) shows the maximum interstory drift ratios for the scaled Loma Prieta earthquake. The maximum interstory drift ratios are less in the lower stories and larger in the upper stories for the modified frames as compared to the unmodified frames. In general for these frames, the interstory drift ratios are larger for the scaled earthquakes as compared to the unscaled earthquakes.

The response of these frames for the Loma Prieta earthquake is similar to the response under the static linear load. This indicates that the performance of the frames was improved in the modified frames for these cases in particular. The maximum building displacement profile is approximately linear, which suggests a structural mechanism controls. Also, the reduced interstory drift ratios in the lower stories will cause less significant damage during response.

This, however, is only for these particular frames under one of the ten earthquake loadings. These frames will respond differently to the other nine earthquakes and the other twelve frames will respond differently than the 4 shown in

Figure 3.10. The reduction in girder strength by the factor R has a varying impact on the responses of all the possible cases that are not illustrated in Figure 3.10.

The performance of all the frames under all the earthquakes will be based on the change in the (Δ_{idr}) from the unmodified case to the modified case. Figures 3.11 (a-h) and 3.12 (a-h) show the change in the maximum interstory drift ratio (Δ_{idr}) at each story for all sixteen frames under all ten unscaled and scaled earthquakes. The change in the maximum interstory drift ratio (%) is the maximum Δ_{idr} (%) of the unmodified frame minus the maximum Δ_{idr} (%) of the modified frames. A positive change in maximum Δ_{idr} means that the Δ_{idr} of the unmodified frames is higher than the modified frames and has been reduced by reducing the girder strengths. Figure 3.11 shows the results from the unscaled earthquakes and Figure 3.12 shows the results from the scaled earthquakes.

The change in maximum Δ_{idr} varies depending on the frame case and the earthquake. In some cases the reduction in girder strengths in the modified frames greatly improves performance by significantly reducing the interstory drift ratios. In other cases, there is no significant improvement. In general, however, the graphs of Figures 3.11 and 3.12 show a trend in the change in Δ_{idr}: the change in maximum Δ_{idr} is positive in the lower stories and negative in the upper stories. This indicates that the interstory drift ratios are being reduced in the lower stories at the expense of the interstory drift ratios in the upper stories. The decrease in distortion due to lower Δ_{idr} values in the lower stories will decrease the potential for catastrophic damage in buildings. These plots are similar to Figure 3.6 for the static linear load, which indicates that the performance of the modified frames is being improved under dynamic loading also.

The ten earthquakes all have a unique load distribution. Therefore, a better way to assess the performance of the frame was to average the response of all ten earthquakes on each of the frames. This will give a general response to a wide range of earthquakes. A way to form a general performance evaluation of the frames is to consider the average maximum interstory drift ratio ($\Delta_{idr,max}$). The average maximum interstory drift is the maximum story distortions in each building averaged for the ten earthquakes. The $\Delta_{idr,max}$ values of each frame were calculated for the ten earthquakes. These ten values were then averaged to determine the average $\Delta_{idr,max}$ values, which gives a general response to a range of earthquakes.

The average maximum interstory drift ratios of the unmodified and modified frames were calculated and compared for the entire frames and for the bottom half of the frames. The average location of the maximum interstory drift ratios were also calculated in a similar manner. The difference in the amount and location of the average $\Delta_{idr,max}$ is important in determining if the frame's performance is improved by reducing the girder strengths in the upper floor levels. The performance of the frames has improved if the values of the average $\Delta_{idr,max}$ decrease and the location of the average $\Delta_{idr,max}$ increase up the building for the modified cases. The interstory drift ratios of the lower half of the building are especially important to the amount of damage to a building during response.

Figure 3.13 shows the change in average maximum interstory drift ratios for the sixteen frame cases using the average maximum response of the ten unscaled and scaled earthquakes. This is for the maximum interstory drift ratio of all the stories in the frame. The change in average $\Delta_{idr,max}$ (%) is equal to the average $\Delta_{idr,max}$ (%) of the unmodified frames minus the $\Delta_{idr,max}$ (%) of the modified frames. Therefore, a positive change means that the average $\Delta_{idr,max}$ has been reduced in the modified frames. Figure 3.13 shows the average maximum interstory drift ratio increases for most of the modified frames using the unscaled ground motions. Figure 3.13 shows that about half of the modified frames have a decreased value of the average $\Delta_{idr,max}$ for the ten scaled earthquakes when all stories are considered.

Figure 3.4 shows the change in average $\Delta_{idr,max}$ for the stories in the bottom half of the frames. The building response from the ten unscaled and scaled earthquakes was averaged for the sixteen unmodified and modified frames. In every case the change in average $\Delta_{idr,max}$ is positive for the lower stories. The average maximum interstory drift ratio ($\Delta_{idr,max}$) decreases for the all of the modified frames. Therefore, the distortion in the lower stories is reduced when the frames are modified by reducing the strength in girders using the average response of the ten scaled and unscaled earthquakes.

Figure 3.15 shows the average change in location of the $\Delta_{idr,max}$ for the sixteen frame cases using the ten unscaled and scaled earthquakes. The location of where the $\Delta_{idr,max}$ occurred was calculated as a percentage of the total building height for each frame and then averaged for the ten earthquakes. The average change in location of the $\Delta_{idr,max}$ (%) is equal to the average location of the $\Delta_{idr,max}$ (%) for the modified frames minus the average location of the $\Delta_{idr,max}$ (%) for the unmodified frames. Therefore, a positive average change in the location of $\Delta_{idr,max}$ indicates that the average location of the $\Delta_{idr,max}$ has risen in height for the modified frames.

Figure 3.15 illustrates that the average location of $\Delta_{idr,max}$ has risen for the frames modified with reduced girder strength. As the location of the largest distortion (percent of building height) increases the performance of the frame improves. The maximum story distortions in the bottom stories were reduced and the location of the maximum story distortions is higher when the frames are modified by reducing the girder strength.

Occasionally there is a problem in determining the controlling mechanism for the non-linear dynamic analysis. Sometimes the earthquake demand is low so that none or very few of the columns and girders have yielded during the event. Therefore, no collapse mechanism has formed for the frame case. The controlling mechanism is not always apparent unless the progression of yielding is determined by checking member ductilities at every response point.

The maximum column ductilities ($\mu_{col,max}$) will be used to evaluate the performance of the frames for the non-linear dynamic analysis. The column has experienced yielding when the column ductility is greater than or equal to 1.00. If the column ductility is less than 1.00, the column has not yielded. The non-linear dynamic analysis performed with LARZ calculates the column ductilities for all of the columns in the sixteen frame cases.

The performance of the frames was evaluated by comparing the average maximum column ductilities ($\mu_{col,max}$) of the unmodified and modified frames. The ductilities of all the columns in each frame were compared and the column with the maximum ductility ($\mu_{col,max}$) was determined. This maximum column ductility ($\mu_{col,max}$) in the frames was calculated for each of the ten unscaled and scaled earthquakes. The maximum column ductility ($\mu_{col,max}$) was averaged for all ten earthquakes to determine the average maximum column ductility for each frame. The maximum girder ductility of each frame was also calculated to determine if the girder ductilities were excessive.

Figures 3.16 and 3.17 show the percent difference in the average maximum column ductility for all sixteen frames. The average maximum ductilities of the ten unscaled and scaled earthquakes were calculated. A positive change means that the average maximum column ductility has been reduced in the modified frames. A reduction in the average maximum column ductilities indicates that the columns are less likely to yield under the dynamic loading due to a decrease in the maximum rotation. Figure 3.16 shows the maximum column ductilities of all columns while Figure 3.17 shows the maximum column ductilities of the columns in the lower half of the frames.

Figure 3.16 shows the average maximum column ductility was reduced in most of the modified buildings when considering all of the columns in the frames for the unscaled and scaled earthquakes. Figure 3.17 shows the average maximum column ductility was reduced in all of the modified buildings except one when considering the columns in the lower half of the frames. The reduction in maximum ductility indicates that the columns are less likely to yield. This is especially important for the columns in the lower half of the frames.

Figure 3.18 shows the average change in location of the maximum column ductilities for the ten unscaled and scaled earthquakes. The location of the maximum column ductilities was averaged for the ten earthquakes for the unmodified and modified frames as a percent of total building height. The average change in location (%) is equal to the average location of the maximum column ductility in the modified frames (%) minus the average location of the maximum column ductilities in the unmodified frames (%). Figure 3.18 shows that the average change in location of the maximum column ductility is positive for all of the frames. This indicates that the location of the maximum column ductility is higher in the modified frames.

The maximum overall ductilities were calculated to determine if the girders had experienced excessive amounts of rotation during the earthquakes. The girder ductilities are calculated similarly to the column ductilities. Girder ductility is the ratio of the maximum girder rotation to the girder rotation at yield. The main criteria in evaluating the performance of a reinforced concrete frame in earthquakes is to force yielding into the girders and out of the columns. The ductility values in the girders should be greater than 1.0. However, it is important that the girders do not exhibit excessive rotations beyond the yielding.

In general, the girder ductilities were larger than the column ductilites in the frames. This indicates that the girders tended to yield prior to the columns. The maximum girder ductilities in the modified frames are larger than the maximum girder ductilities in the unmodified frames because the frames have been modified in order to force the demand from the columns into the girders. Table 3.17 shows the maximum girder ductilities averaged for the ten unscaled and scaled earthquakes in the modified frames. Some of the values of maximum girder ductility are large because some of the earthquakes represented a very large demand. These large girder ductilities occurred in the upper floor levels of the modified frames. The maximum girder ductility increases as the amount of reduction in girder strength (R) increases.

4.4.3 Summary

The results of the non-linear dynamic analysis indicate the frame performance has improved when the average maximum response of each earthquake is calculated. The maximum interstory drift ratio and maximum column ductilities are reduced in the lower stories of each building. The location of the maximum interstory drift ratio is higher up in the modified frames. There will be less distortion and less column yielding in the critical bottom stories whenever the frames are modified through the reduction in girder strengths.

In general, for unscaled and scaled earthquakes, the maximum column ductility has been reduced and the location of the maximum column ductility is higher when the frames are modified by reducing the strength in the girders. A lower column ductility indicates the columns in the frame are less likely to exhibit yielding. If the chance for column yielding has been reduced then the chance for the formation of an intermediate collapse mechanism has also been reduced. This implies that the performance of the frames has been improved by reducing the strength in the girders. The higher in the building that the columns with the maximum rotations occur, the less likelihood there is for excessive damage during response.

CHAPTER 4. CASE STUDY: HOLIDAY INN, VAN NUYS, CA.

The relationship for improving performance was applied to an existing building with non-regular proportions to determine if reducing the strengths in girders improved the response. A 7-story Holiday Inn located in Van Nuys, California was selected. Non-linear static and dynamic analyses were performed on the Holiday Inn. The building was analyzed as unmodified using the actual building design and modified by reducing the strength in upper floor level girders. Ground motions recorded in sensors during the 1994 Northridge earthquake were used in the dynamic analysis. The performance was based on the criteria described in Chapter 3.

4.1 Holiday Inn Description and Parameters

The Holiday Inn is a seven-story reinforced concrete structure built in 1966 and located a few miles northeast of the epicenter of the 1994 Northridge earthquake. Figures 4.1 and 4.2 summarize the layout and framing system of the Holiday Inn (Blume, 1971). The longitudinal frame is in the east-west (EW) direction and the transverse frame is in the north-south (NS) direction. The building is rectangular with 8 bays in the longitudinal direction and 3 bays in the transverse direction. The longitudinal bay lengths are all 18'-9" and the 3 bays in the transverse direction measure 20'-1", 20'-10", and 20'-1". The 1st story is 13'-6" tall and the 2nd – 6th stories are 8'-8 ½ ", and the 7th story is 8'-8" . The building measures approximately 62 by 150 feet in plan area and is 66 feet in height.

The framing system consists of spandrel beams along the perimeter of the structure and a reinforced concrete flat slab. The slab is 10" thick at the second floor, 8 ½" thick at the third to seventh floors, and 8" thick at the roof. The spandrel beams are 30" deep (including the slab thickness) at the second floor, 22 ½" deep at the third to seventh floors, and 22" deep at the roof. The spandrel beams are 16" wide along the longitudinal perimeter and 14" wide along the transverse perimeter.

The exterior and corner columns at all stories are 20" by 14" rectangular sections. The interior columns are 20" by 20" square sections at the first story and 18" by 18" square sections at the second to seventh stories. Table 4.1 gives the properties of the regular weight reinforced concrete (150 pcf) that was used in the structure. Table 4.2 lists the properties of the reinforcing steel.

The lateral forces are resisted in each direction by the interior column-slab frames and the exterior column-spandrel beam frames. The exterior frame is significantly stiffer than the interior frames due to the added stiffness of the spandrel beams (Blume, 1971). The weight is 1830 kips at the 1st story, 1460 kips at the second to sixth stories, and 1410 kips at the seventh story. The period of the building was calculated assuming uncracked sections. The 1st mode period of the building was 0.86 seconds for the longitudinal frame (EW) and 0.93 seconds for the transverse frames (NS).

The ground motion caused by the 1994 Northridge earthquake was recorded by accelerographs located on the ground, second, third, sixth and roof levels (Ventura et al., 1995) The building suffered significant structural damage during the 1994

Northridge earthquake including cracking in beams, columns, and beam-column joints (Browning et al., 2000). The most extensive damage was in the exterior columns of the fourth story which sustained significant shear cracking. There was significant interior non-structural damage caused by large levels of story distortion. The damage during the 1971 San Fernando earthquake was concentrated on the second and third floors which suffered severe structural and non-structural damage (Blume, 1973).

4.2 Analysis

Non-linear static and dynamic analyses were performed on a model of the longitudinal frame (east-west) of the Holiday Inn building. The longitudinal frame has 8 bays with a slab-column interior frame and spandrel beam-column exterior frame. The building was analyzed as an unmodified frame by modeling the structural members as they were originally designed. The building was then analyzed as a modified frame by applying the girder strength reduction factor (R) to the original design.

An approximately linear load distribution was used in the static analyses. The lateral forces applied to each floor level varied due to story height and weight. Since the frame has neither uniform story heights or uniform story weights, the distribution is approximately linear. The distribution of lateral forces was calculated using the following equation:

$$F_i = \frac{w_i h_i}{\sum_{i=1}^{n} w_i h_i} \qquad 4.1$$

F_i = Lateral Force at i^{th} Floor Level

w_i = Weight of i^{th} Floor Level

h_i = Height to i^{th} Floor Level

n = Total Number of Stories

The ground motion recorded at one of the accelerographs was used in the dynamic analysis. The accelerations were recorded from sensor # 16 which is located at the southeast corner of the ground floor level of the building. This sensor measured the ground motions that were applied in the longitudinal direction of the building. Sensor # 16 measured a peak ground acceleration of 0.47g during the Northridge earthquake (Ventura et al, 1995). The Rayleigh damping factors were calculated using 0.02 viscous damping and a period of 0.86 seconds assuming uncracked sections.

The tri-linear moment-curvature relationships were calculated for the exterior spandrel beams, interior frame slabs, and columns. The top reinforcement in the spandrel beams included 50% of the slab reinforcement within 20" of the 2^{nd} floor beam faces and within 14" of the 3^{rd}-7^{th} floor beam faces. The reinforcement in the interior slab sections was assumed to be the steel within the column strip width defined in Section 13.2.1 of A.C.I. 318-99.

The Holiday Inn was analyzed as unmodified and modified by reducing the flexural strengths in upper floor level girders. The girder strengths were reduced by the factor R as prescribed in Equation 2.8 in order to encourage the structural mechanism to be the controlling mechanism:

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2} \qquad 2.8$$

The flexural strengths in the spandrel beams and interior slabs in the upper floor levels were divided by R to determine the modified girder strengths. Table 4.3 lists the Holiday Inn parameters that were necessary in determining the girder strength reduction factor (R). Table 4.3 also gives the value of R that was determined from these parameters by using Equation 2.8.

The longitudinal frame of the Holiday Inn building has 7 stories (N_s) and 8 bays (N_b). The column reinforcement ratio (ρ_{col}) of 0.0223 was determined as the average reinforcement ratio of all the columns. The column width (h_{col}) of 20 inches was the longitudinal column dimension of the top story corner columns. The column flexural strength (M_c) was determined as the strength of the top story corner column. The minimum column flexural strength occurs in the top story corner column.
The girder flexural strength (M_g) was the average flexural strength of the exterior spandrel beams and the interior frame slabs. The value of α is equal to the ratio of M_c of the top story corner column to the average M_g. The beams and slabs in the top 3 floor levels had their girder strengths reduced. The value of β is equal to the number of floor levels with reduced girder strengths divided by the total number of stories.

The girder strength reduction factor R was calculated to be 1.75 by using Equation 2.8. Therefore, the strength in the spandrel beams and interior slabs in the top 3 floor levels of the Holiday Inn must be divided by 1.75 in order to encourage the formation of the structural mechanism. The girder strength reduction factor must be applied to the exterior and interior frames because the M_g was based on the average of both the spandrel beams and the interior slabs.

4.3 Static Results

A non-linear static analysis was performed on the Holiday Inn building using LARZ. The 7-story building was analyzed as unmodified and modified with reduced girder strengths in the top 3 floor levels. The flexural strengths in the exterior spandrel beams and interior slabs were reduced by the factor R in order to encourage the formation of the structural mechanism. Table 4.3 showed that the necessary value of R was determined to be 1.75. The response of the unmodified and modified Holiday Inn was calculated and compared to determine if the performance of the building was improved. A static lateral load distribution using Equation 4.1 was applied to the building.

Table 4.4 shows the controlling mechanism for the unmodified and modified Holiday Inn building. The location of the intermediate mechanism is shown as a percent of total building height. The reduction in girder strength equation (Equation 2.8) was derived to guarantee the formation of the structural mechanism for any set of frame parameters. Table 4.4 shows that the controlling mechanism changed from a 5^{th} story intermediate mechanism to the structural mechanism when the girder strengths were reduced by R = 1.75 (modified Holiday Inn) under the approximately linear lateral load.

Figure 4.3 shows the relative base shear ($V_{b,rel}$) versus the mean drift ratio (Δ_{mdr}) for the unmodified and modified Holiday Inn. The relative base shear and mean drift ratio were determined from Equations 3.18 and 3.19. The final point of relative base shear and mean drift ratio shown on Figure 4.3 occurs at the formation of the controlling mechanism. The modified building reaches a yield point at a lower base shear than the unmodified frames. This indicates that a mechanism will form at an earlier time in the case where the girder strengths were reduced in the Holiday Inn building.

Figure 4.4 show the relative building displacement profile for the Holiday Inn building. This graph plots the normalized drift (Δ_{norm}) for each floor level in the frame. The normalized drift is the displacement at a given floor level divided by the total height of the building as a percent. The drift at each floor level was recorded at the time when the first sway mechanism forms for the unmodified and modified Holiday Inn building.

Figure 4.4 shows that the displacement profile of the modified Holiday inn building is approximately linear. This is because the structural mechanism is the controlling mechanism for the modified case and the building is displacing with the column acting as a stiff, unyielding spine. The displacement profile of the unmodified Holiday Inn building corresponds to the displacement profile expected for a building with a 5th story intermediate mechanism as the controlling mechanism. The displacement in the modified Holiday Inn is more desirable than the displacement of the unmodified Holiday Inn. The slope of the drift profile decreased when the girder strengths were reduced, especially in the lower floor levels which results in less distortion.

Figure 4.5 shows the interstory drift ratios (Δ_{idr}) for the unmodified and modified Holiday Inn building that were recorded at the level of force that caused the first sway mechanism forms. The interstory drift ratios for each story were calculated using Equation 3.17. The interstory drift ratios are a lot more uniform in the modified Holiday Inn because the structural mechanism is the controlling mechanism. The unmodified Holiday Inn building exhibits very large Δ_{idr} values in the 2nd, 3rd, and 4th stories which result in large distortions that cause extensive structural and non-structural damage. The modified Holiday Inn building experiences a significant reduction of the interstory drift ratio in these bottom stories. The interstory drift ratio increases slightly in the upper stories of the modified Holiday Inn because of the Δ_{idr} becoming more uniform. The reduction in the strength of the beams and slabs of the top 3 floor levels of the Holiday Inn improves the performance of the building by decreasing significantly the distortion in the bottom stories.

Figure 4.6 shows the change in Δ_{idr} for the 7 stories of the Holiday Inn building. A positive change in Δ_{idr} means the interstory drift ratio has been reduced in the modified building. Figure 4.6 shows that the interstory drift ratios are being reduced in the lower stories and increased in the upper stories when the building was modified by reducing the girder strengths. The reduction in Δ_{idr} in the lower stories indicates the performance of the building improved when it was modified.

Table 4.5 lists further results from the non-linear static analysis of the Holiday Inn building. The maximum interstory drift ratio ($\Delta_{idr,max}$) is the maximum story distortion in the building at the formation of the first mechanism. Table 4.5 gives the change in value and location of the $\Delta_{idr,max}$ between the unmodified and modified Holiday Inn building. A positive change in $\Delta_{idr,max}$ means that the maximum interstory drift ratio has been reduced in the modified frames. A positive change in location of the maximum interstory drift ratio means that the location of the $\Delta_{idr,max}$ is higher in the modified frames.

The maximum interstory drift ratio was calculated for all stories and for the critical bottom stories, which in this case are stories 1-4. Table 4.5 shows that the value of $\Delta_{idr,max}$ has been reduced and the location of the $\Delta_{idr,max}$ is higher in the modified Holiday Inn building. This is further evidence that the performance of the Holiday Inn building was improved when the strengths of the beams and slabs in the top 3 floor levels was reduced by 1.75.

4.4 Dynamic Results

A non-linear dynamic analysis was also performed on the Holiday Inn building. The ground motions recorded at sensor located in the building during the 1994 Northridge earthquake were applied to the building. The maximum response of the unmodified and modified Holiday Inn was calculated and compared to determine if the performance of the building improved when the girder strengths (spandrel beams and interior slab) were reduced in the top 3 floor levels. The factors used to assess the performance of the building under the earthquake load were the same ones used in Chapter 3.

Figure 4.7 shows the maximum displacement profiles of the unmodified and modified Holiday inn buildings. The figure plots the normalized drift (Δ_{norm}) versus floor level. The normalized drift (Δ_{norm}) is the maximum drift that occurs at a floor level for the duration of the earthquake (recorded at sensor #16) divided by the total building height. The displacement profiles of the Holiday Inn indicate that the unmodified building is displacing as if a 4th or 5th story intermediate mechanism controls and the modified building is displacing linearly, suggesting the structural mechanism has formed. The displacement of the modified Holiday Inn is preferred because there is less story distortion in the lower floor levels.

Figure 4.8 show the maximum interstory drift ratios (Δ_{idr}) of each story for the Holiday Inn building under the earthquake loading. The maximum values of Δ_{idr} for the duration of the earthquake were calculated using equation 3.17. The maximum interstory drift ratios are less in the lower stories and larger in the upper stories for the modified Holiday Inn as compared to the unmodified building. The largest interstory drift ratios occur in the 2nd, 3rd, and 4th stories of the unmodified frames. These locations of large distortions match the results of the static analysis and probably caused the damage during the Northridge earthquake. The modified Holiday Inn building exhibits less distortion in the lower stories indicating the performance of the building improved by reducing the girder strengths in the top 3 floor levels.

Figure 4.9 shows the change in the maximum interstory drift ratio (Δ_{idr}) at each story for the Holiday Inn building for the duration of the earthquake. A positive change in Δ_{idr} means that the interstory drift ratio has decreased in the building modified by reducing the girder strengths. The interstory drift ratio was reduced in the stories 1-3 and increased in stories 5-7 when the building was modified. The decrease in distortion in the lower stories will decrease the potential for large damage in the building. The increase in the interstory drift in the upper stories of the modified building was significant indicating that the girders with reduced strengths in the top 3 levels experienced large rotations.

Table 4.6 lists results from the non-linear dynamic analysis of the Holiday Inn building. The maximum interstory drift ratio ($\Delta_{idr,max}$) is the maximum story distortion in the building for the duration of the earthquake. Table 4.6 gives the change in value and location of the $\Delta_{idr,max}$ between the unmodified and modified Holiday Inn building. The maximum interstory drift ratio was calculated for all stories and for the critical bottom stories, which in this case are stories 1-4. Table 4.6 shows that the value of $\Delta_{idr,max}$ has been reduced and the location of the $\Delta_{idr,max}$ is higher in the modified Holiday Inn building. This indicates that the performance of the Holiday Inn building was improved when the strengths of the beams and slabs in the top 3 floor levels was reduced by 1.75.

The maximum member rotational ductilities (μ) were calculated for the unmodified and modified Holiday Inn building using Equation 3.20. The member ductility is the ratio between the maximum rotation (θ_{max}) of a member during the earthquake to the rotation of the member at yield (θ_{yield}). A member has yielded if the maximum ductility is greater than or equal to 1.0. The controlling mechanisms were determined for the unmodified and modified building by evaluating which members had yielded ($\mu > 1.0$) for the duration of the earthquake.

Table 4.7 lists the yielded members and the controlling mechanism for the Holiday Inn building. A 4th story intermediate mechanism formed in the exterior frame of the unmodified Holiday Inn building. Several columns in the 5th and 6th stories also yielded in the unmodified building. The structural mechanism formed in the interior frame of the modified Holiday Inn building during the earthquake. Very few columns experienced yielding in the modified building. The girders in the top floor of the exterior frame were the only girders that did not yield in the modified frame. This indicates that the structural mechanism was close to forming in the exterior frame as well. The reduction in the girder strengths changed the controlling mechanism from an intermediate mechanism to the structural mechanism. These results are similar to the results of the static non-linear analysis using an approximately linear lateral load distribution.

The value and location of the maximum column ductility ($\mu_{col,max}$) was calculated for the unmodified and modified Holiday Inn building (Equation 3.20). The change in the value and the location of the maximum column ductility was used to assess the performance of the building. Table 4.8 lists the change in maximum column ductilities for all 7 stories and for the bottom stories 1-4. A positive change in maximum column ductility means the $\mu_{col,max}$ was reduced in the modified frames.

A positive change in location of the maximum column ductility means that the location of the $\mu_{col,max}$ was higher in the modified building.

Table 4.8 shows that the maximum column ductility increased overall in the modified frames but decreased in the lower stories. The location of the maximum column ductility was much higher in the modified building. The top story columns experienced large levels of rotation in the modified building which contributed to the overall increase in maximum column ductility. The maximum girder ductility ($\mu_{girder,max}$) also increased from 3.63 to 4.21 in the modified Holiday Inn building. This is due to the increased rotations in the top floor level girders.

4.5 Summary

The performance of the Holiday Inn building improved when the strengths in the spandrel beams and interior slabs were reduced by the factor R = 1.75. Non-linear static and dynamic analyses were performed on the unmodified and modified frame. The dynamic analysis used ground motions recorded in the building during the Northridge earthquake. The results indicated that large distortions occurred in the 2^{nd} – 4^{th} stories of the unmodified frame resulting in damage at these locations. The reduction in girder strengths caused the structural mechanism to form, reduced the interstory drift ratios in the bottom stories, and raised the location of the maximum interstory drift ratio and maximum column ductility.

CHAPTER 5. CONCLUSION

A relationship was developed to improve the performance of reinforced concrete frames during earthquakes. A limit analysis was used to develop a strength based method for reinforced concrete frames that will enable the structural mechanism to be the controlling mechanism under a linear lateral loading distribution. Several methods were investigated including the global increase in column strength, the increase in column strengths at selected stories, and the decrease of girder strengths at selected floor levels. The reduction of girder strengths at particular floor levels was selected as the initial hypothesis for reducing column yielding and forcing the structural mechanism to form.

A parametric analysis was performed on the initial hypothesis using a limit analysis. An equation was developed (Equation 2.8) that calculated the reduction in girder strength necessary (R) to facilitate the formation of the structural mechanism. The girder strengths in upper floor level girders were divided by the factor R to determine the girder strengths necessary to improve the performance of the frames.

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2}$$

Non-linear static and dynamic analyses were used to test the viability of the relationship to improve performance. Sixteen different frames were proportioned using varying parameters. The frames were analyzed as unmodified frames and modified frames and the results were compared. The modified frames were similar to the unmodified frames except the girder strengths in the upper floor levels were reduced by the factor R. The minimum girder strength was used in those cases where R was excessive.

A linear loading distribution was used in the non-linear static analysis on the sixteen frames. The structural mechanism was the controlling mechanism for all of the modified frames. In general, the maximum interstory drift ratios decreased in the modified frames. The interstory drift ratios in the bottom half stories decreased significantly and the locations of the maximum interstory drift were higher in the modified frames. These indicators of improved performance occurred for all of the modified frames, including those with minimum girder strengths (girder strengths not fully reduced).

Any reduction in girder strengths in the upper floor levels will improve the performance of a reinforced concrete frame under static linear lateral load. The reduction in girder strength by the factor R will facilitate the formation of the structural mechanism and significantly improve the response. The formation of the structural mechanism under static linear load was the goal in developing the relationship. The formation of the structural mechanism in the modified frames was directly related to the decrease in maximum interstory drift ratios, the decrease in the interstory drift ratios in the bottom half stories, and the increase in height in the location of the maximum interstory drift ratios.

Ten unscaled and scaled earthquakes were used in the non-linear dynamic analysis. The relationship to improve performance was developed for a linear load distribution. The earthquake loading distributions are not necessarily linear and vary from case to case. Therefore, the response of the building under each earthquake will vary. The maximum response of the frames was calculated and averaged for the ten unscaled and scaled earthquakes. The column yielding was based on the maximum rotational ductilites of the columns.

The interstory drift ratios decreased in the bottom half stories and the location of the maximum interstory drift ratio increased in height in the modified frames. The maximum column ductilities decreased and the location of the maximum column ductility increased in height in the modified frames. These indicators of improved performance were calculated as the average response of the ten unscaled and scaled earthquakes.

The improvement of the frames in the dynamic analysis was not as dramatic as in the static analysis. In the static analysis, the linear load was applied incrementally until a mechanism formed. The dynamic analysis used the ground motions of ten earthquakes (varying load distributions) which often were insufficient to form any type of mechanism. The results of the dynamic analysis indicate that the performance of the frames will improve under seismic loading. The level of improvement will vary depending on the earthquake.

The relationship to improve performance was applied to a Holiday Inn building in Van Nuys, California. Non-linear static and dynamic analyses were performed on the Holiday Inn in its present state and as if it were modified by reducing the strength in the girders of the upper floors. Ground motions recorded from sensors in the building during the 1994 Northridge earthquake were used in the dynamic analysis. The results of the analyses showed that the performance of the building improved when using the reduction in girder strength method.

The reduction in the strength of upper floor level girders improved the performance of reinforced concrete frames under earthquakes by encouraging the structural mechanism to form. The formation of the structural mechanism results in decreased distortion and column rotation in the bottom stories of buildings. The maximum distortion decreased and the location of the maximum distortion increased in height when the girder strengths were reduced. These improvements in the response of the frames will result in decreased structural damage during a seismic event.

SUMMARY

1. Large values of α (ratio of column to girder flexural strength) are needed in reinforced concrete frame buildings to eliminate column hinging and prevent intermediate story mechanisms from forming during a seismic event.
2. A simple and cost-effective method to increase α in order to encourage the formation of the structural mechanism (girder hinging) is to decrease the flexural strength in the upper floor level girders of buildings.
3. The results of a parametric study using limit analysis indicated that the structural mechanism will form if the girder strengths in a building are reduced by dividing the initial flexural strengths by the factor R.

$$R = 1 + \frac{N_s * N_b * \sqrt{\rho * h}}{7 * (N_b + 1) * \beta * \alpha^2}$$

4. The method of reducing the flexural strength in upper floor level girders (R) to reduce column hinging and improve frame performance seemed to work as indicated in the results of the static, dynamic and Holiday Inn studies.

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TABLE 2.1
Square Column Sizes for 4-16 Story Frames

Number of Stories	Square Column Size (in)
4	16
6	18
8	20
10	22
12	24
14	26
16	28

TABLE 2.2
Parameters Used in Investigation of Mechanism Formation Analysis

Parameter	Value	Comments
Load	160 psf on tributary areas	For determining axial loads on columns
f'_c	4000 psi	Columns and girders
Number of Stories	4-16 stories	
Number of Bays	4	
Story Height	10 feet	Uniform
Span Length	20 feet	Uniform
Loading Distribution	Linear & Constant	
Column Size	16 in – 28 in	See Table 2.1
Girder Size	24 in	Total Depth
Column Strength	ρ = 1%(min) - 8%(max)	As necessary to increase α
Girder Strength	$\rho_{top} = \rho_{bottom}$ = 1% (base value)	Increase as necessary to maintain α increments
Strength Ratio (α)	1.20 (min)	As necessary to form structural mechanism. Use 0.10 increments.

TABLE 2.3
Constant Parameters Used in Analysis of Initial Hypothesis

Constant Parameters	Value
f'_c	4000 psi
Story Height	10 feet
Span Length	20 feet
Girder Depth	24 inches
Floor Loading For Axial Loads to Columns	160 psf
Load Distribution	Linear

TABLE 2.4
Variable Parameters Used in Analysis of Initial Hypothesis

Variable Parameter	Range Used in Analysis	Comments
Number of Stories (N_s)	4 8 12 14 16	
Number of Bays (N_b)	4 6	
Column Size (h)	$A_{col} = P_{max} / (0.4 * f'_c)$ $A_{col} = P_{max} / (0.3 * f'_c)$ $A_{col} = P_{max} / (0.2 * f'_c)$	P_{max} = Axial Load on Bottom Story Interior Column
Column Strength	$\rho_{col} = 1\%$ $\rho_{col} = 2\%$ $\rho_{col} = 3\%$ $\rho_{col} = 4\%$	Used to Find the Column Moments (M_c)
Girder Strength	$\alpha = 1$ $\alpha = 2$ $\alpha = 3$ $\alpha = 4$	Vary Initial Global α to Vary Initial Girder Strength Equation 2.6
Number of Floor Levels with Reduced Girder Strength (β)	$\beta = 0 - 0.5$	β = Ratio of Number of Floor Levels with Reduced Girder Strengths to Total Number of Floor

TABLE 2.5
Typical Portion of Parametric Analysis of Initial Hypothesis
Uniform Reduction in Girder Strength Necessary to Make the Structural Mechanism the Controlling Mechanism

N_s	N_b	h (in)	ρ (%)	α (initial)	# of Floor Levels with Reduced Girder Strength	β	Uniform Reduction in Girder Strength
8	4	18	1	1.0	2	0.25	2.6
					3	0.375	2.0
				2.0	1	0.125	1.2
					2	0.25	1.1
					3	0.375	1.1
			2	1.0	3	0.375	2.2
				2.0	1	0.125	1.7
					2	0.25	1.3
					3	0.375	1.2

TABLE 2.6

Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
8	4	18	0.01	1	0.125	**2.60**
8	4	18	0.01	1	0.250	**2.00**
8	4	18	0.01	2	0.125	**1.10**
8	4	18	0.01	2	0.250	**1.10**
8	4	18	0.02	1	0.250	**2.20**
8	4	18	0.02	2	0.125	**1.30**
8	4	18	0.02	2	0.250	**1.20**
8	4	18	0.03	1	0.250	**2.30**
8	4	18	0.03	2	0.125	**1.40**
8	4	18	0.03	2	0.250	**1.30**
8	4	18	0.04	1	0.250	**2.50**
8	4	18	0.04	2	0.125	**1.50**
8	4	18	0.04	2	0.250	**1.30**
8	4	21	0.01	1	0.250	**2.00**
8	4	21	0.01	2	0.125	**1.20**
8	4	21	0.01	2	0.250	**1.20**
8	4	21	0.02	1	0.250	**2.20**
8	4	21	0.02	2	0.125	**1.40**
8	4	21	0.02	2	0.250	**1.30**
8	4	21	0.03	1	0.250	**2.50**
8	4	21	0.03	2	0.125	**1.50**
8	4	21	0.03	2	0.250	**1.30**
8	4	21	0.04	1	0.250	**2.60**
8	4	21	0.04	2	0.125	**1.60**
8	4	21	0.04	2	0.250	**1.30**
8	4	26	0.01	1	0.250	**2.20**
8	4	26	0.01	2	0.125	**1.30**
8	4	26	0.01	2	0.250	**1.30**
8	4	26	0.02	1	0.250	**2.50**
8	4	26	0.02	2	0.125	**1.50**
8	4	26	0.02	2	0.250	**1.30**
8	4	26	0.03	1	0.250	**2.80**
8	4	26	0.03	2	0.125	**1.70**
8	4	26	0.03	2	0.250	**1.40**
8	4	26	0.04	1	0.250	**2.90**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
8	4	26	0.04	2	0.125	**1.80**
8	4	26	0.04	2	0.250	**1.40**
8	6	18	0.01	1	0.250	**2.00**
8	6	18	0.02	1	0.250	**2.20**
8	6	18	0.03	1	0.250	**2.40**
8	6	18	0.04	1	0.250	**2.60**
8	6	21	0.01	1	0.250	**2.10**
8	6	21	0.02	1	0.250	**2.40**
8	6	21	0.03	1	0.250	**2.70**
8	6	21	0.04	1	0.250	**2.80**
8	6	26	0.01	1	0.250	**2.30**
8	6	26	0.02	1	0.250	**2.70**
8	6	26	0.03	1	0.250	**3.00**
8	6	26	0.04	1	0.250	**3.20**
14	4	24	0.01	1	0.286	**2.60**
14	4	24	0.01	2	0.071	**2.80**
14	4	24	0.01	2	0.143	**1.80**
14	4	24	0.01	2	0.214	**1.50**
14	4	24	0.01	2	0.286	**1.50**
14	4	24	0.01	3	0.071	**1.20**
14	4	24	0.01	3	0.143	**1.10**
14	4	24	0.01	3	0.214	**1.10**
14	4	24	0.01	3	0.286	**1.10**
14	4	24	0.02	1	0.286	**2.80**
14	4	24	0.02	2	0.143	**2.40**
14	4	24	0.02	2	0.214	**1.80**
14	4	24	0.02	2	0.286	**1.60**
14	4	24	0.02	3	0.071	**1.50**
14	4	24	0.02	3	0.143	**1.30**
14	4	24	0.02	3	0.214	**1.20**
14	4	24	0.02	3	0.286	**1.20**
14	4	24	0.03	1	0.286	**3.10**
14	4	24	0.03	1	0.357	**2.90**
14	4	24	0.03	2	0.143	**2.90**
14	4	24	0.03	2	0.214	**2.00**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
14	4	24	0.03	2	0.286	**1.70**
14	4	24	0.03	3	0.071	**1.90**
14	4	24	0.03	3	0.143	**1.50**
14	4	24	0.03	3	0.214	**1.30**
14	4	24	0.03	3	0.286	**1.30**
14	4	24	0.04	1	0.286	**3.50**
14	4	24	0.04	1	0.357	**2.90**
14	4	24	0.04	2	0.214	**2.20**
14	4	24	0.04	2	0.286	**1.80**
14	4	24	0.04	3	0.071	**2.20**
14	4	24	0.04	3	0.143	**1.60**
14	4	24	0.04	3	0.214	**1.40**
14	4	24	0.04	3	0.286	**1.30**
14	4	24	0.04	4	0.071	**1.20**
14	4	24	0.04	4	0.143	**1.10**
14	4	24	0.04	4	0.214	**1.10**
14	4	24	0.04	4	0.286	**1.10**
14	4	28	0.01	1	0.286	**2.70**
14	4	28	0.01	2	0.143	**2.20**
14	4	28	0.01	2	0.214	**1.70**
14	4	28	0.01	2	0.286	**1.60**
14	4	28	0.01	3	0.071	**1.40**
14	4	28	0.01	3	0.143	**1.30**
14	4	28	0.01	3	0.214	**1.20**
14	4	28	0.01	3	0.286	**1.20**
14	4	28	0.02	1	0.286	**3.10**
14	4	28	0.02	1	0.357	**2.90**
14	4	28	0.02	2	0.143	**2.90**
14	4	28	0.02	2	0.214	**2.00**
14	4	28	0.02	2	0.286	**1.70**
14	4	28	0.02	3	0.071	**2.00**
14	4	28	0.02	3	0.143	**1.50**
14	4	28	0.02	3	0.214	**1.40**
14	4	28	0.02	3	0.286	**1.30**
14	4	28	0.02	4	0.071	**1.10**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
14	4	28	0.02	4	0.143	**1.10**
14	4	28	0.02	4	0.214	**1.10**
14	4	28	0.02	4	0.286	**1.10**
14	4	28	0.03	1	0.286	**3.50**
14	4	28	0.03	1	0.357	**3.00**
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14	4	28	0.03	2	0.286	**1.80**
14	4	28	0.03	3	0.071	**2.40**
14	4	28	0.03	3	0.143	**1.70**
14	4	28	0.03	3	0.214	**1.50**
14	4	28	0.03	3	0.286	**1.40**
14	4	28	0.03	4	0.071	**1.20**
14	4	28	0.03	4	0.143	**1.20**
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14	4	28	0.04	3	0.071	**2.60**
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14	4	28	0.04	3	0.214	**1.50**
14	4	28	0.04	3	0.286	**1.40**
14	4	28	0.04	4	0.071	**1.30**
14	4	28	0.04	4	0.143	**1.20**
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14	4	34	0.01	2	0.286	**1.70**
14	4	34	0.01	3	0.071	**1.80**
14	4	34	0.01	3	0.143	**1.40**
14	4	34	0.01	3	0.214	**1.30**
14	4	34	0.01	3	0.286	**1.30**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
14	4	34	0.02	1	0.286	**3.60**
14	4	34	0.02	1	0.357	**3.00**
14	4	34	0.02	2	0.214	**2.30**
14	4	34	0.02	2	0.286	**1.90**
14	4	34	0.02	3	0.071	**2.60**
14	4	34	0.02	3	0.143	**1.70**
14	4	34	0.02	3	0.214	**1.50**
14	4	34	0.02	3	0.286	**1.40**
14	4	34	0.02	4	0.071	**1.30**
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14	4	34	0.02	4	0.286	**1.10**
14	4	34	0.03	1	0.286	**4.30**
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14	4	34	0.03	2	0.286	**2.00**
14	4	34	0.03	3	0.143	**1.90**
14	4	34	0.03	3	0.214	**1.60**
14	4	34	0.03	3	0.286	**1.40**
14	4	34	0.03	4	0.071	**1.40**
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14	4	34	0.03	4	0.286	**1.20**
14	4	34	0.04	1	0.286	**4.30**
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14	4	34	0.04	2	0.214	**2.70**
14	4	34	0.04	2	0.286	**2.00**
14	4	34	0.04	3	0.143	**2.00**
14	4	34	0.04	3	0.214	**1.60**
14	4	34	0.04	3	0.286	**1.40**
14	4	34	0.04	4	0.071	**1.50**
14	4	34	0.04	4	0.143	**1.30**
14	4	34	0.04	4	0.214	**1.20**
14	4	34	0.04	4	0.286	**1.20**
14	6	24	0.01	1	0.286	**2.70**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
14	6	24	0.02	1	0.286	**2.90**
14	6	24	0.03	2	0.286	**1.80**
14	6	24	0.04	2	0.286	**1.90**
14	6	28	0.01	1	0.286	**2.80**
14	6	28	0.02	2	0.286	**1.80**
14	6	28	0.03	2	0.286	**1.90**
14	6	28	0.04	2	0.286	**2.00**
4	4	13	0.01	1	0.250	**1.40**
4	4	13	0.02	1	0.250	**1.50**
4	4	13	0.03	1	0.250	**1.50**
4	4	13	0.04	1	0.250	**1.50**
4	4	15	0.01	1	0.250	**1.50**
4	4	15	0.02	1	0.250	**1.50**
4	4	15	0.03	1	0.250	**1.50**
4	4	15	0.04	1	0.250	**1.60**
4	4	18	0.01	1	0.250	**1.50**
4	4	18	0.02	1	0.250	**1.60**
4	4	18	0.03	1	0.250	**1.60**
4	4	18	0.04	1	0.250	**1.60**
12	4	22	0.01	1	0.250	**2.40**
12	4	22	0.01	1	0.333	**2.40**
12	4	22	0.01	2	0.083	**1.80**
12	4	22	0.01	2	0.167	**1.40**
12	4	22	0.01	2	0.250	**1.40**
12	4	22	0.01	2	0.333	**1.40**
12	4	22	0.02	1	0.250	**2.90**
12	4	22	0.02	1	0.333	**2.60**
12	4	22	0.02	2	0.083	**2.90**
12	4	22	0.02	2	0.167	**1.80**
12	4	22	0.02	2	0.250	**1.50**
12	4	22	0.02	2	0.333	**1.50**
12	4	22	0.02	3	0.083	**1.20**
12	4	22	0.02	3	0.167	**1.20**
12	4	22	0.02	3	0.250	**1.10**
12	4	22	0.02	3	0.333	**1.10**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
12	4	22	0.03	1	0.333	**2.70**
12	4	22	0.03	2	0.167	**2.00**
12	4	22	0.03	2	0.250	**1.60**
12	4	22	0.03	2	0.333	**1.60**
12	4	22	0.03	3	0.083	**1.40**
12	4	22	0.03	3	0.167	**1.30**
12	4	22	0.03	3	0.250	**1.20**
12	4	22	0.03	3	0.333	**1.20**
12	4	22	0.04	1	0.333	**2.70**
12	4	22	0.04	2	0.167	**2.20**
12	4	22	0.04	2	0.250	**1.70**
12	4	22	0.04	2	0.333	**1.60**
12	4	22	0.04	3	0.083	**1.50**
12	4	22	0.04	3	0.167	**1.30**
12	4	22	0.04	3	0.250	**1.20**
12	4	22	0.04	3	0.333	**1.20**
12	4	26	0.01	1	0.250	**2.60**
12	4	26	0.01	1	0.333	**2.60**
12	4	26	0.01	2	0.083	**2.20**
12	4	26	0.01	2	0.167	**1.60**
12	4	26	0.01	2	0.250	**1.50**
12	4	26	0.01	2	0.333	**1.50**
12	4	26	0.01	3	0.083	**1.10**
12	4	26	0.01	3	0.167	**1.10**
12	4	26	0.01	3	0.250	**1.10**
12	4	26	0.01	3	0.333	**1.10**
12	4	26	0.02	1	0.333	**2.70**
12	4	26	0.02	2	0.167	**2.00**
12	4	26	0.02	2	0.250	**1.50**
12	4	26	0.02	2	0.333	**1.60**
12	4	26	0.02	3	0.083	**1.40**
12	4	26	0.02	3	0.167	**1.20**
12	4	26	0.02	3	0.250	**1.20**
12	4	26	0.02	3	0.333	**1.20**
12	4	26	0.03	1	0.333	**2.70**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	**Number of Bays (N_b)**	**Column Size h (in)**	ρ	α	β	**Reduction in Girder Strength Factor (R)**
12	4	26	0.03	2	0.167	**2.30**
12	4	26	0.03	2	0.250	**1.70**
12	4	26	0.03	2	0.333	**1.70**
12	4	26	0.03	3	0.083	**1.50**
12	4	26	0.03	3	0.167	**1.30**
12	4	26	0.03	3	0.250	**1.20**
12	4	26	0.03	3	0.333	**1.20**
12	4	26	0.04	1	0.333	**2.80**
12	4	26	0.04	2	0.167	**2.50**
12	4	26	0.04	2	0.250	**1.90**
12	4	26	0.04	2	0.333	**1.70**
12	4	26	0.04	3	0.083	**1.80**
12	4	26	0.04	3	0.167	**1.40**
12	4	26	0.04	3	0.250	**1.30**
12	4	26	0.04	3	0.333	**1.30**
12	4	31	0.01	1	0.250	**2.90**
12	4	31	0.01	1	0.333	**2.70**
12	4	31	0.01	2	0.083	**3.00**
12	4	31	0.01	2	0.167	**1.80**
12	4	31	0.01	2	0.250	**1.60**
12	4	31	0.01	2	0.333	**1.60**
12	4	31	0.01	3	0.083	**1.20**
12	4	31	0.01	3	0.167	**1.20**
12	4	31	0.01	3	0.250	**1.10**
12	4	31	0.01	3	0.333	**1.10**
12	4	31	0.02	1	0.333	**2.80**
12	4	31	0.02	2	0.167	**2.30**
12	4	31	0.02	2	0.250	**1.80**
12	4	31	0.02	2	0.333	**1.70**
12	4	31	0.02	3	0.083	**1.60**
12	4	31	0.02	3	0.167	**1.30**
12	4	31	0.02	3	0.250	**1.30**
12	4	31	0.02	3	0.333	**1.30**
12	4	31	0.03	1	0.333	**2.80**
12	4	31	0.03	2	0.167	**2.60**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
12	4	31	0.03	2	0.250	**1.90**
12	4	31	0.03	2	0.333	**1.70**
12	4	31	0.03	3	0.083	**1.90**
12	4	31	0.03	3	0.167	**1.50**
12	4	31	0.03	3	0.250	**1.30**
12	4	31	0.03	3	0.333	**1.30**
12	4	31	0.03	4	0.083	**1.10**
12	4	31	0.03	4	0.167	**1.10**
12	4	31	0.03	4	0.250	**1.10**
12	4	31	0.03	4	0.333	**1.10**
12	4	31	0.04	1	0.333	**2.80**
12	4	31	0.04	2	0.167	**2.70**
12	4	31	0.04	2	0.250	**1.90**
12	4	31	0.04	2	0.333	**1.70**
12	4	31	0.04	3	0.083	**1.90**
12	4	31	0.04	3	0.167	**1.50**
12	4	31	0.04	3	0.250	**1.30**
12	4	31	0.04	3	0.333	**1.30**
12	4	31	0.04	4	0.083	**1.10**
12	4	31	0.04	4	0.167	**1.10**
12	4	31	0.04	4	0.250	**1.10**
12	4	31	0.04	4	0.333	**1.10**
16	4	26	0.01	1	0.250	**3.00**
16	4	26	0.01	1	0.313	**2.80**
16	4	26	0.01	1	0.375	**2.80**
16	4	26	0.01	2	0.125	**2.50**
16	4	26	0.01	2	0.188	**1.90**
16	4	26	0.01	2	0.250	**1.60**
16	4	26	0.01	2	0.313	**1.60**
16	4	26	0.01	2	0.375	**1.60**
16	4	26	0.01	3	0.063	**1.50**
16	4	26	0.01	3	0.125	**1.30**
16	4	26	0.01	3	0.188	**1.20**
16	4	26	0.01	3	0.250	**1.20**
16	4	26	0.01	3	0.313	**1.20**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
16	4	26	0.01	3	0.375	**1.20**
16	4	26	0.02	1	0.313	**3.00**
16	4	26	0.02	1	0.375	**3.00**
16	4	26	0.02	2	0.188	**2.50**
16	4	26	0.02	2	0.250	**1.90**
16	4	26	0.02	2	0.313	**1.80**
16	4	26	0.02	2	0.375	**1.80**
16	4	26	0.02	3	0.063	**2.40**
16	4	26	0.02	3	0.125	**1.70**
16	4	26	0.02	3	0.188	**1.50**
16	4	26	0.02	3	0.250	**1.40**
16	4	26	0.02	3	0.313	**1.30**
16	4	26	0.02	3	0.375	**1.40**
16	4	26	0.02	4	0.063	**1.20**
16	4	26	0.02	4	0.125	**1.10**
16	4	26	0.02	4	0.188	**1.10**
16	4	26	0.02	4	0.250	**1.10**
16	4	26	0.02	4	0.313	**1.10**
16	4	26	0.02	4	0.375	**1.10**
16	4	26	0.03	1	0.313	**3.10**
16	4	26	0.03	1	0.375	**3.10**
16	4	26	0.03	2	0.188	**2.80**
16	4	26	0.03	2	0.250	**2.10**
16	4	26	0.03	2	0.313	**1.90**
16	4	26	0.03	2	0.375	**1.90**
16	4	26	0.03	3	0.125	**2.00**
16	4	26	0.03	3	0.188	**1.60**
16	4	26	0.03	3	0.250	**1.40**
16	4	26	0.03	3	0.313	**1.40**
16	4	26	0.03	3	0.375	**1.40**
16	4	26	0.03	4	0.063	**1.40**
16	4	26	0.03	4	0.125	**1.20**
16	4	26	0.03	4	0.188	**1.20**
16	4	26	0.03	4	0.250	**1.20**
16	4	26	0.03	4	0.313	**1.20**

TABLE 2.6 (Cont.)
Reduction in Girder Strength Factor (R) Necessary to Guarantee the Formation of the Structural Mechanism

Number of Stories (N_s)	Number of Bays (N_b)	Column Size h (in)	ρ	α	β	Reduction in Girder Strength Factor (R)
16	4	26	0.03	4	0.375	**1.20**
16	4	26	0.04	1	0.313	**3.30**
16	4	26	0.04	1	0.375	**3.20**
16	4	26	0.04	2	0.188	**3.10**
16	4	26	0.04	2	0.250	**2.20**
16	4	26	0.04	2	0.313	**1.90**
16	4	26	0.04	2	0.375	**1.90**
16	4	26	0.04	3	0.125	**2.20**
16	4	26	0.04	3	0.188	**1.70**
16	4	26	0.04	3	0.250	**1.50**
16	4	26	0.04	3	0.313	**1.40**
16	4	26	0.04	3	0.375	**1.50**
16	4	26	0.04	4	0.063	**1.50**
16	4	26	0.04	4	0.125	**1.30**
16	4	26	0.04	4	0.188	**1.20**
16	4	26	0.04	4	0.250	**1.20**
16	4	26	0.04	4	0.313	**1.20**
16	4	26	0.04	4	0.375	**1.20**

TABLE 3.1
Assumptions Used in Parametric Analysis

Compressive Strength of Concrete (f'_c)	Modulus of Elasticity of Concrete (E)	Modulus of Rigidity of Concrete (G)	P-Δ Effects in Analysis	M-φ Relationship
4000 psi	4000 ksi	1600 ksi	Included	Tri-linear

TABLE 3.2
Example Tri-Linear Moment Curvature Points (See Figure 3.1)

Point	Moment (k-")	Curvature (in^{-1})
A	$M_{cr} = 500$	$\phi_{cr} = 1.00e^{-5}$
B	$M_y = 3000$	$\phi_y = 1.00e^{-4}$
C	$M_{ult} = 3250$	$\phi_{ult} = 1.00e^{-3}$

TABLE 3.3
General Parameters and Coding of Sixteen Frame Cases

Frame Case Code	Number of Stories (N_s)	Span Length (L_s) (feet)	Column Stiffness Factor (K)
1an	4	20	0.35
1bn	4	30	0.35
1as	4	20	0.20
1bs	4	30	0.20
2an	8	20	0.35
2bn	8	30	0.35
2as	8	20	0.20
2bs	8	30	0.20
3an	12	20	0.35
3bn	12	30	0.35
3as	12	20	0.20
3bs	12	30	0.20
4an	16	20	0.35
4bn	16	30	0.35
4as	16	20	0.20
4bs	16	30	0.20

TABLE 3.4
Girder Dimensions Used in Parametric Analysis

L_s (ft)	D_g (in)	D_s (in)	W_b (in)	W_t (in)
20	24	6	12	48
30	36	6	18	78

TABLE 3.5
Column Dimensions Used in Parametric Analysis

Frame Case Code	Stories	$P_{s,max}$ (kips)	$A_{col}(in^2)$	h (in)
1an	All	256	196	14
1bn	All	576	484	22
1as	All	256	324	18
1bs	All	576	784	28
2an	All	512	400	20
2bn	All	1152	900	30
2as	All	512	676	26
2bs	All	1152	1444	38
3an	1-6	768	576	24
	7-12	384	324	18
3bn	1-6	1728	1296	36
	7-12	864	676	26
3as	1-6	768	1024	32
	7-12	384	484	22
3bs	1-6	1728	2304	48
	7-12	864	1156	34
4an	1-8	1024	784	28
	9-16	512	400	20
4bn	1-8	2304	1764	42
	9-16	1152	900	30
4as	1-8	1024	1296	36
	9-16	512	796	26
4bs	1-8	2304	2916	54
	9-16	1152	1444	38

TABLE 3.6
Strength Parameters for the Girders Used in Analysis

L_s (ft)	D_g (in)	ρ_{top}(%)	ρ_{bottom}(%)	A_{stop} (in2)	$A_{sbottom}$ (in2)	M_g (K-in)
20	24	1.00	0.75	2.64	1.98	2617
30	36	1.00	0.75	6.12	4.59	9571

TABLE 3.7
Column Strength Parameters for the Sixteen Frame Cases in Analysis

Frame Case Code	Top Story Column h (in)	ρ_{col} (%)	M_c (K-in) Yield
1an	14	2.00	1405
1bn	22	2.00	5853
1as	18	2.00	3004
1bs	28	2.00	11946
2an	20	2.00	4114
2bn	30	2.00	14706
2as	26	2.00	9093
2bs	38	2.00	29741
3an	18	2.00	3004
3bn	26	2.00	9560
3as	22	2.00	5513
3bs	34	2.00	21241
4an	20	2.00	4114
4bn	30	2.00	14706
4as	26	2.00	9093
4bs	38	2.00	29741

TABLE 3.8
Amount of Reduction in Girder Strength (R) Necessary to Ensure the Structural Mechanism Controls for the Sixteen Frame Cases

Frame Case Code	N_s	$N_{s,eff}$	h (in)	M_c (K-in)	M_g (K-in)	α	# of Floor Levels with Reduced M_g	β	R
1an	4	4	14	1405	2617	0.54	2	0.500	**2.68**
1bn	4	4	22	5853	9571	0.61	2	0.500	**2.62**
1as	4	4	18	3004	2617	1.15	2	0.500	**1.42**
1bs	4	4	28	11946	9571	1.25	2	0.500	**1.44**
2an	8	8	20	4114	2617	1.57	3	0.375	**1.62**
2bn	8	8	30	14706	9571	1.54	3	0.375	**1.80**
2as	8	8	26	9093	2617	3.47	3	0.375	**1.15**
2bs	8	8	38	29741	9571	3.11	3	0.375	**1.22**
3an	12	6	18	3004	2617	1.15	4	0.333	**1.94**
3bn	12	6	26	9560	9571	1.00	4	0.333	**2.49**
3as	12	6	22	5513	2617	2.11	4	0.333	**1.31**
3bs	12	6	34	21241	9571	2.22	4	0.333	**1.34**
4an	16	8	20	4114	2617	1.57	6	0.375	**1.62**
4bn	16	8	30	14706	9571	1.54	6	0.375	**1.80**
4as	16	8	26	9093	2617	3.47	6	0.375	**1.15**
4bs	16	8	38	29741	9571	3.11	6	0.375	**1.22**

TABLE 3.9
Reduced Girder Strengths and Reinforcement for the Sixteen Frame Cases

Frame Case Code	R	M_g (K-in)	$M_{g,red}$ (K-in)	$A_{stop,red}$ (in^2)	$A_{sbott,red}$ (in^2)	$\rho_{top,red}$ (%)	$\rho_{bott,red}$ (%)
1an	2.68	2617	**976**	**1.00**	**0.75**	0.379	0.284
1bn	2.62	9571	**3640**	**2.30**	**1.75**	0.376	0.286
1as	1.42	2617	**1837**	**1.84**	**1.40**	0.701	0.530
1bs	1.44	9571	**6649**	**4.25**	**3.15**	0.694	0.515
2an	1.62	2617	**1610**	**1.65**	**1.25**	0.625	0.473
2bn	1.80	9571	**5309**	**3.40**	**2.50**	0.556	0.408
2as	1.15	2617	**2274**	**2.25**	**1.75**	0.852	0.663
2bs	1.22	9571	**7852**	**4.95**	**3.75**	0.809	0.613
3an	1.94	2617	**1344**	**1.35**	**1.05**	0.511	0.398
3bn	2.49	9571	**3848**	**2.40**	**1.85**	0.392	0.302
3as	1.31	2617	**1977**	**2.05**	**1.50**	0.777	0.568
3bs	1.34	9571	**7131**	**4.50**	**3.35**	0.735	0.547
4an	1.62	2617	**1610**	**1.65**	**1.25**	0.625	0.473
4bn	1.80	9571	**5309**	**3.40**	**2.50**	0.556	0.408
4as	1.15	2617	**2274**	**2.25**	**1.75**	0.852	0.663
4bs	1.22	9571	**7852**	**4.95**	**3.75**	0.809	0.613

TABLE 3.10
Minimum Girder Reinforcement Necessary to Resist Factored Gravity Loads

Span Length (ft)	Reinforcement Layer	$A_{s,min}$
20	Top Steel	**1.57**
	Bottom Steel	**1.07**
30	Top Steel	**3.29**
	Bottom Steel	**2.25**

TABLE 3.11
Strength and Reinforcement of Modified Girders for the Sixteen Frame Cases

Frame Case Code	R	$A_{stop,red}$ (in^2)	$A_{sbott,red}$ (in^2)	$M_{g,red}$ (K-in)
1an	1.71 (max)	1.57	1.07	1527
1bn	1.85 (max)	3.29	2.25	5174
1as	1.42	1.85	1.40	1837
1bs	1.44	4.25	3.15	6649
2an	1.62	1.65	1.25	1610
2bn	1.80	3.40	2.50	5309
2as	1.15	2.25	1.75	2274
2bs	1.22	4.95	3.75	7852
3an	1.71 (max)	1.57	1.07	1527
3bn	1.85 (max)	3.29	2.25	5174
3as	1.31	2.05	1.50	1977
3bs	1.34	4.50	3.35	7131
4an	1.62	1.65	1.25	1610
4bn	1.80	3.40	2.50	5309
4as	1.15	2.25	1.75	2274
4bs	1.22	4.95	3.75	7852

TABLE 3.12
Controlling Mechanism for the Sixteen Frame Cases
Static Analysis

Frame Case Code	N_s	Controlling Mechanism Unmodified Frames	Location of Controlling Mechanism Unmodified Frames (%)	Controlling Mechanism Modified Frames
1an	4	2nd Story Intermediate	50.00	Structural
1bn	4	3rd Story Intermediate	75.00	Structural
1as	4	3rd Story Intermediate	75.00	Structural
1bs	4	3rd Story Intermediate	75.00	Structural
2an	8	5th Story Intermediate	62.50	Structural
2bn	8	6th Story Intermediate	75.00	Structural
2as	8	7th Story Intermediate	87.50	Structural
2bs	8	7th Story Intermediate	87.50	Structural
3an	12	8th Story Intermediate	66.67	Structural
3bn	12	8th Story Intermediate	66.67	Structural
3as	12	9th Story Intermediate	75.00	Structural
3bs	12	10th Story Intermediate	83.33	Structural
4an	16	11th Story Intermediate	68.75	Structural
4bn	16	11th Story Intermediate	68.75	Structural
4as	16	12th Story Intermediate	75.00	Structural
4bs	16	13th Story Intermediate	81.25	Structural

TABLE 3.13
Location of the Maximum Interstory Drift Ratio for the Sixteen Frame Cases
Static Analysis

Frame Case Code	Location of $\Delta_{idr,max}$ (%) UNMODIFIED	Location of $\Delta_{idr,max}$ (%) MODIFIED	Change in Location of $\Delta_{idr,max}$ (%)
1an	50.0	75.0	**25.0**
1bn	50.0	75.0	**25.0**
1as	50.0	50.0	**0.0**
1bs	50.0	·50.0	**0.0**
2an	37.5	37.5	**0.0**
2bn	37.5	37.5	**0.0**
2as	37.5	50.0	**12.5**
2bs	37.5	50.0	**12.5**
3an	41.7	41.7	**0.0**
3bn	41.7	58.3	**16.6**
3as	58.3	58.3	**0.0**
3bs	50.0	58.3	**8.3**
4an	37.5	43.8	**6.3**
4bn	43.8	56.3	**12.5**
4as	50.0	50.0	**0.0**
4bs	50.0	56.3	**6.3**

TABLE 3.14
Ten Earthquakes Used in Dynamic Analysis

Earthquake Code	Location of Event	Duration of Event (sec)	Time Step (sec)	P.G.A. (*g)
ELC	Imperial Valley, CA	25	0.02	0.35
KOB	Hyogo-Ken-Nanbu, Japan	7	0.02	0.83
LLO	Chile	48	0.005	0.71
LOM	Loma Prieta, CA	5	0.02	0.37
NAH	Nahanni, Canada	8	0.005	0.98
SEN	Miyagi-Ken-Oki, Japan	12	0.02	0.26
TAR	Northridge, CA	20	0.02	0.99
TURK	Turkey	3	0.005	0.48
VAL1	Valparaiso Central Chile	8	0.005	0.35
VAL2	Valparaiso Central Chile	11	0.005	0.47

TABLE 3.15

Scale Factors and Peak Ground Accelerations for the Earthquakes

Earthquake Code	Scale Factor	Scaled P.G.A. (*g)
ELC	2.4	0.84
KOB	0.7	0.58
LLO	1.2	0.85
LOM	1.5	0.55
NAH	1.8	1.76
SEN	0.9	0.24
TAR	0.8	0.79
TURK	1.5	0.72
VAL1	1.3	0.46
VAL2	1.2	0.57

TABLE 3.16

First and Second Mode Periods and Damping Factors for the Sixteen Frame Cases

Frame Case Code	T_1 (sec)	T_2 (sec)	ω_1 (rad / sec)	ω_2 (rad / sec)	A	B
1an	0.66	0.22	9.52	28.56	**0.286**	**0.00105**
1bn	0.38	0.13	16.53	48.33	**0.493**	**0.00062**
1as	0.48	0.16	13.09	39.27	**0.393**	**0.00076**
1bs	0.29	0.09	21.67	69.81	**0.661**	**0.00044**
2an	0.87	0.28	7.22	22.44	**0.219**	**0.00135**
2bn	0.55	0.18	11.42	34.91	**0.344**	**0.00086**
2as	0.71	0.23	8.85	27.32	**0.267**	**0.00111**
2bs	0.47	0.15	13.37	41.89	**0.405**	**0.00072**
3an	1.21	0.43	5.19	14.61	**0.153**	**0.00202**
3bn	0.91	0.32	6.90	19.63	**0.204**	**0.00151**
3as	1.01	0.35	6.22	17.95	**0.185**	**0.00165**
3bs	0.78	0.27	8.06	23.27	**0.239**	**0.00128**
4an	1.48	0.52	4.25	12.08	**0.126**	**0.00245**
4bn	1.13	0.39	5.56	16.11	**0.165**	**0.00185**
4as	1.27	0.44	4.95	14.28	**0.147**	**0.00208**
4bs	1.00	0.34	6.28	18.48	**0.188**	**0.00162**

TABLE 3.17
Average Maximum Girder Ductilities for the Sixteen Frame Cases

Frame Case Code	Maximum Girder Ductility Average of Unscaled EQs	Maximum Girder Ductility Average of Scaled EQs
1an	4.82	4.65
1bn	5.67	3.88
1as	3.25	3.73
1bs	2.28	3.07
2an	3.38	4.66
2bn	2.85	3.48
2as	2.44	3.27
2bs	2.07	2.76
3an	4.93	4.54
3bn	5.33	4.98
3as	2.76	3.25
3bs	1.98	2.55
4an	4.08	4.45
4bn	3.11	3.54
4as	2.56	3.01
4bs	2.03	2.48

TABLE 4.1
Concrete Properties of Holiday Inn

Location in Structure	Minimum Specified Comp. Strength (f'$_c$) (psi)	Modulus of Elasticity (E) (psi)
Columns, 1st to 2nd floors	5000	4.2×10^6
Columns 2nd to 3rd floors	4000	3.7×10^6
Beams and slabs, 2nd floor	4000	3.7×10^6
All other concrete, 3rd floor to roof	3000	3.3×10^6

TABLE 4.2
Reinforcement Properties of Holiday Inn

Location in Structure	Minimum Specified Yield Strength (fy) (ksi)	Modulus of Elasticity (E) (psi)
Beams and slabs	40	29×10^6
Column bars	60	29×10^6

TABLE 4.3
Holiday Inn Building Parameters Used to Determine the Girder Strength Reduction Factor (R)

N_s	N_b	ρ_{col}	h_{col} (in)	M_c (K-in)	M_g (K-in)	α	# of Floor Levels with Reduced M_g	β	R
7	8	0.0223	20	1175	1324	0.89	3	0.429	**1.75**

TABLE 4.4
Controlling Mechanism for Holiday Inn Building
Static Analysis

Building	**N_s**	**Controlling Mechanism Unmodified Building**	**Location of Controlling Mechanism Unmodified Building (%)**	**Controlling Mechanism Modified Building**
Holiday Inn	7	5^{th} Story Intermediate	71.43	Structural

TABLE 4.5
Value and Location of Maximum Interstory Drift Ratio
Unmodified and Modified Holiday Inn Building
Static Analysis

	$\Delta_{idr,max}$ (%) **All Stories**	$\Delta_{idr,max}$ (%) **Stories 1-4**	**Location of $\Delta_{idr,max}$ (% of Building Height)**
Unmodified Holiday Inn	2.67	2.67	42.86
Modified Holiday Inn	1.94	1.94	57.14
CHANGE (%)	**+0.73**	**+0.73**	**+14.29**

TABLE 4.6
Value and Location of Maximum Interstory Drift Ratio
Unmodified and Modified Holiday Inn Building
Dynamic Analysis

	$\Delta_{idr,max}$ (%) **All Stories**	$\Delta_{idr,max}$ (%) **Stories 1-4**	**Location of $\Delta_{idr,max}$ (% of Building Height)**
Unmodified Holiday Inn	2.20	2.20	42.86
Modified Holiday Inn	2.19	1.91	100.00
CHANGE (%)	**+0.01**	**+0.29**	**+57.14**

TABLE 4.7
Yielded Members and Controlling Mechanism
Unmodified and Modified Holiday Inn Building
Dynamic Analysis

Building	Frame	Yielded Columns	Yielded Girders	Controlling Mechanism
Unmodified Holiday Inn	Exterior	1st Story Base 4th Story All 2nd, 5th-7th Stories Interior	1st – 5th Floors	4th Story Intermediate
	Interior	1st Story Base 6th Story Interior	1st - 4th Floors	N/A
Modified Holiday Inn	Exterior	1st Story Base 2nd, 6th, 7th Stories Interior	1st – 6th Floors	N/A
	Interior	1st Story Base	All	Structural

TABLE 4.8
Value and Location of Maximum Column Ductility
Unmodified and Modified Holiday Inn Building
Dynamic Analysis

	$\mu_{col,max}$ All Stories	$\mu_{col,max}$ Stories 1-4	Location of $\mu_{col,max}$ (% of Building Height)
Unmodified Holiday Inn	1.91	1.91	57.14
Modified Holiday Inn	2.24	1.72	100.00
CHANGE (%)	**-17.54**	**+9.89**	**+42.86**

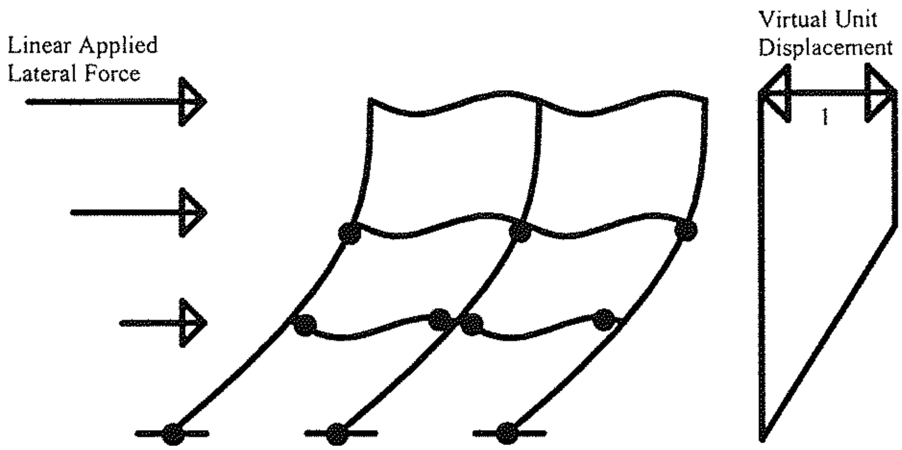


FIGURE 2.1
2nd Story Intermediate
Sway Mechanism

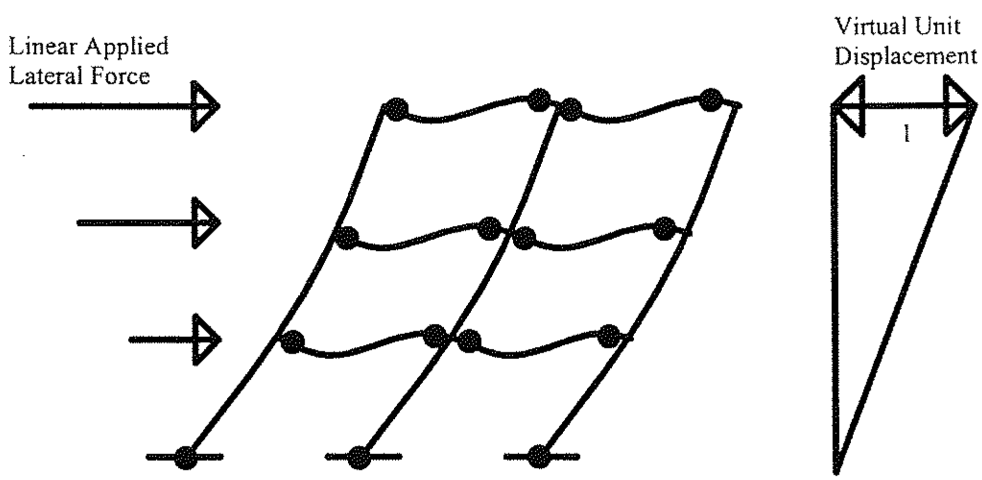


FIGURE 2.2
Structural Mechanism

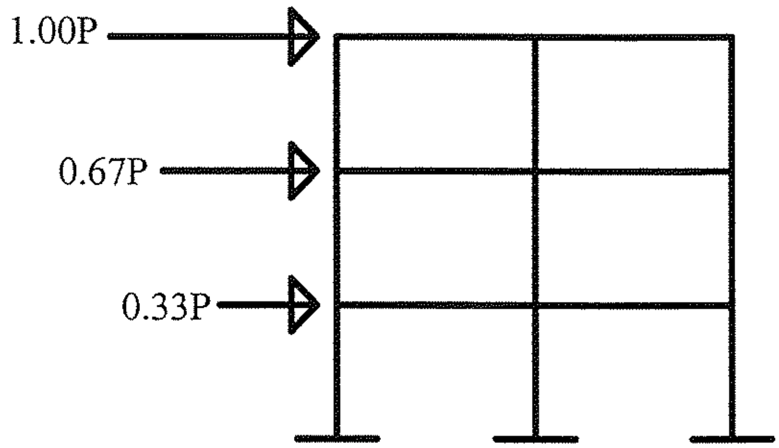


FIGURE 2.3
Linear Applied Lateral Force

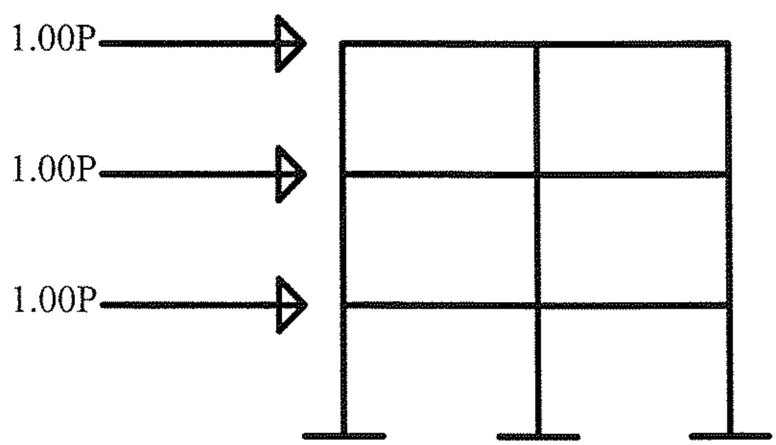


FIGURE 2.4
Constant Applied Lateral Force

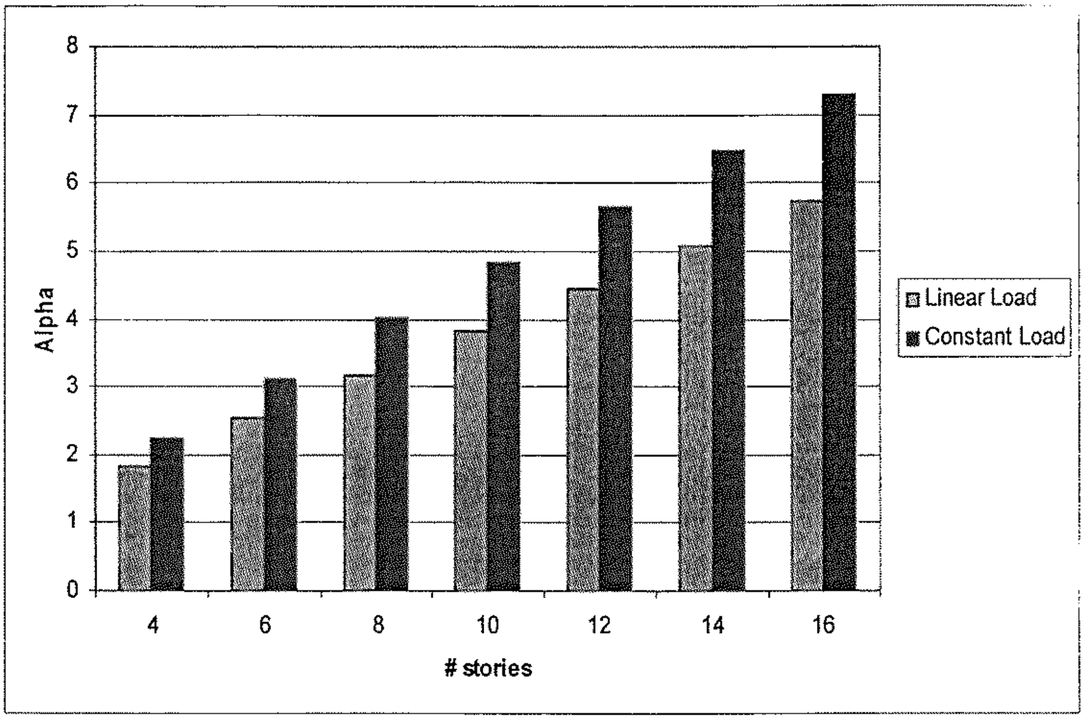


FIGURE 2.5
Necessary α to Form Structural Mechanism

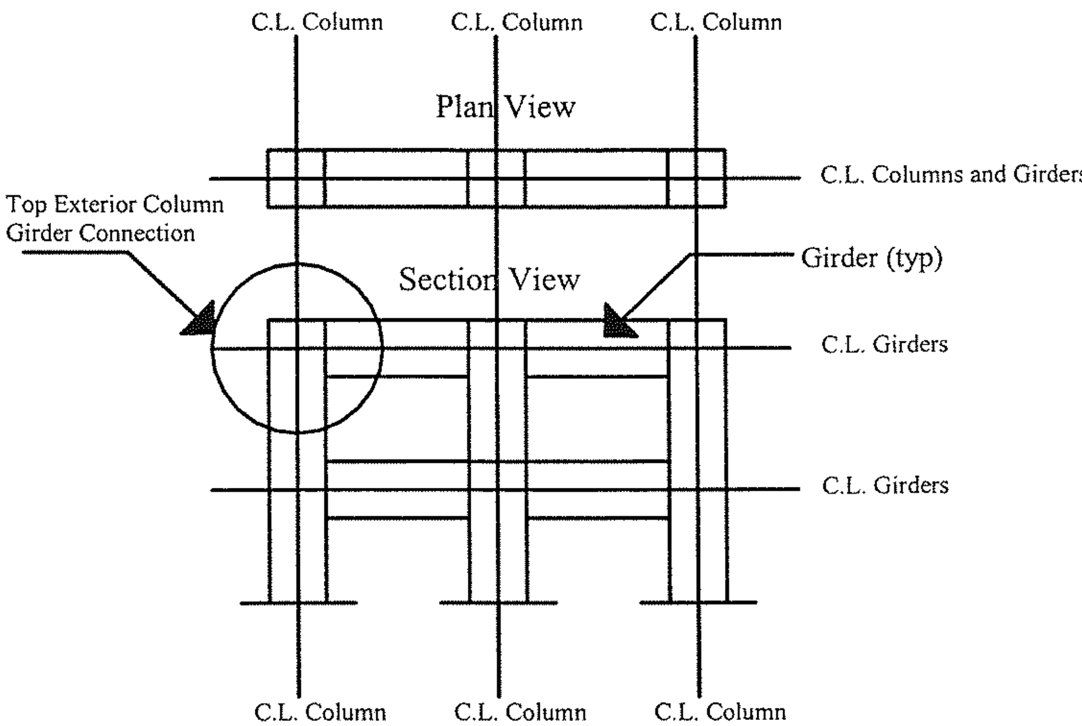


FIGURE 2.6
2 Story 2 Bay Single
Column Line Frame

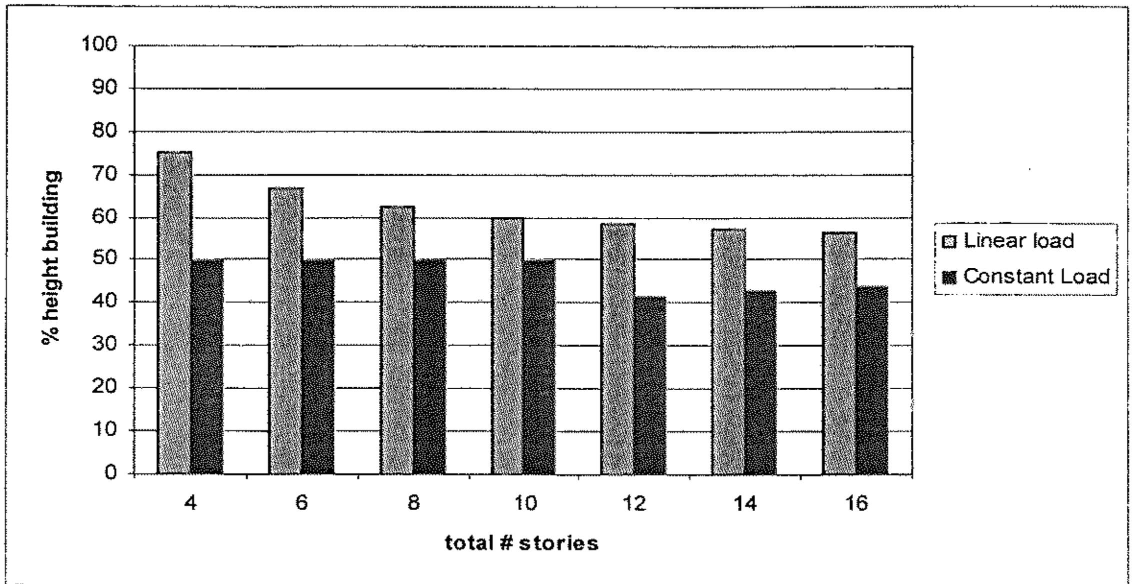


FIGURE 2.7
Location of Intermediate Mechanism Formation at $\alpha = 1.20$

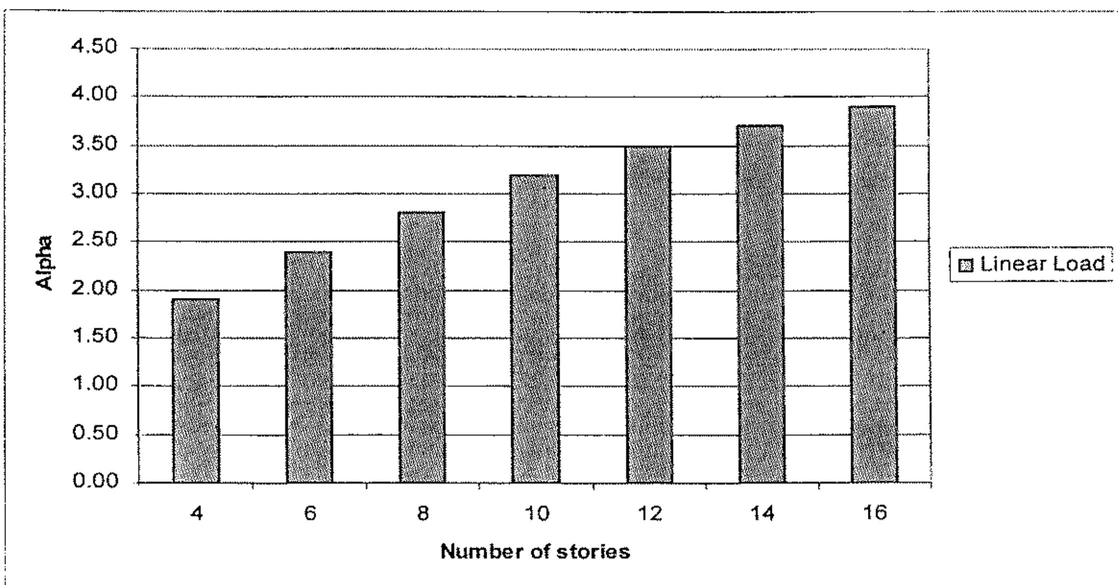


FIGURE 2.8
Necessary α to Form Structural Mechanism
$\alpha = M_{ctext} / M_g$

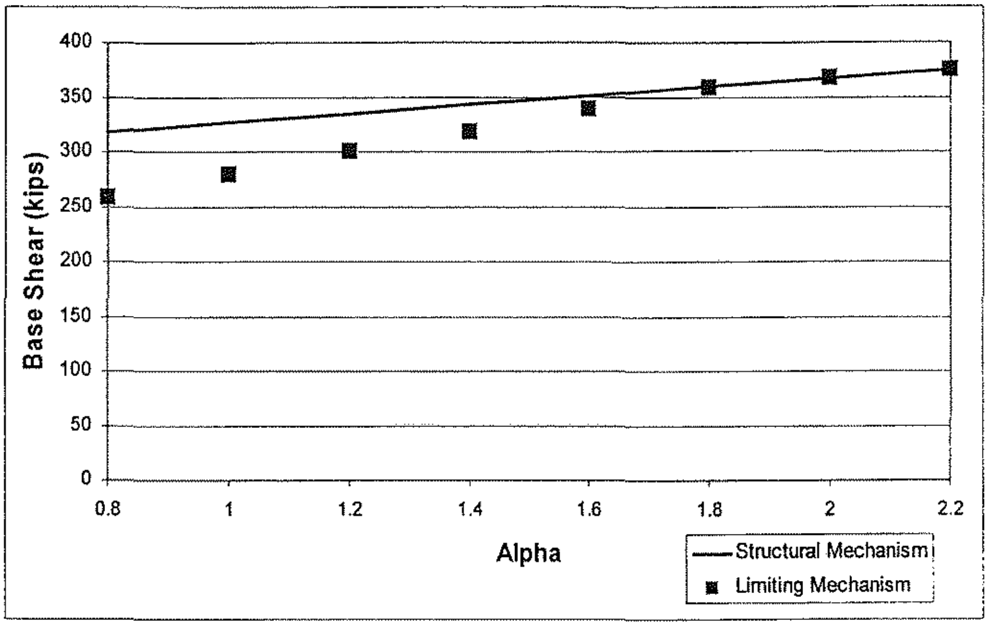


FIGURE 2.9
Comparison of Base Shear Required to Form Mechanisms for a Set of α Values

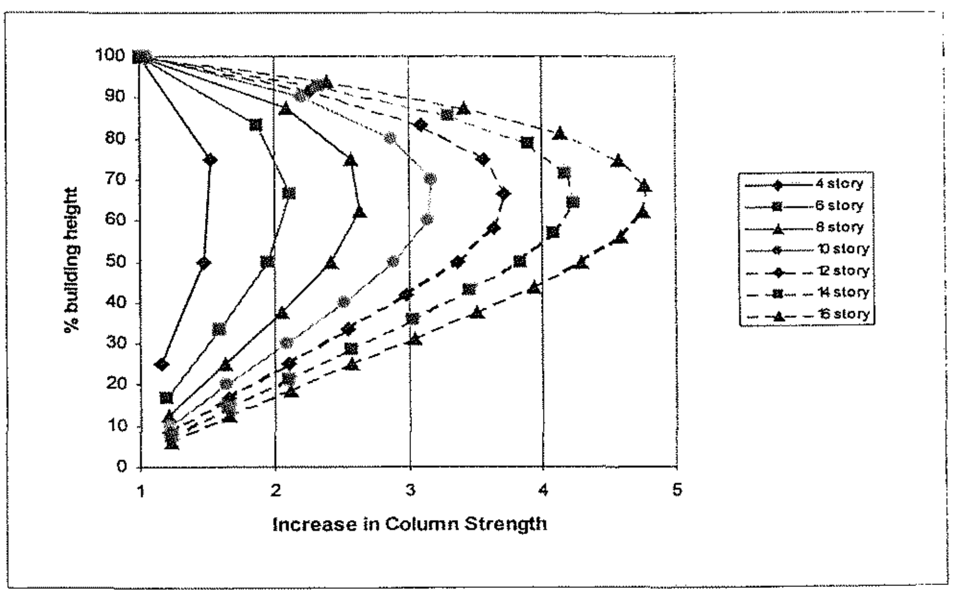


FIGURE 2.10
Increase in Column Strength Necessary at Each Story to Form Structural Mechanism. Uniform Column and Girder Strengths. Linear Load Distribution and Initial Global α = 1.20

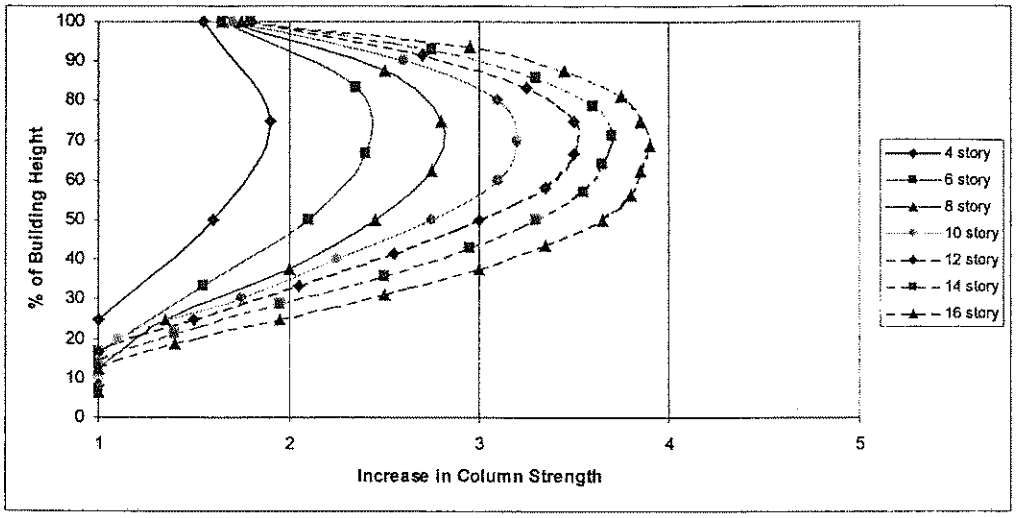


FIGURE 2.11
Increase in Column Strength Necessary at Each Story to Form Structural Mechanism.
Linear Load Distribution and Initial Global $\alpha = 1.20$

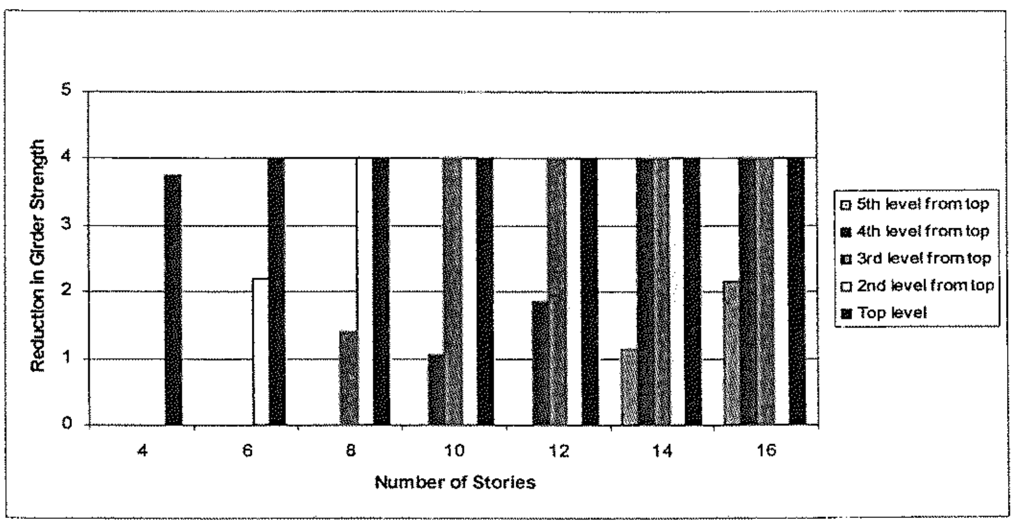


FIGURE 2.12
Reduction in Girder Strength in Upper Levels of Building Necessary to Form Structural Mechanism
Linear Load Distribution and Initial Global $\alpha = 1.20$

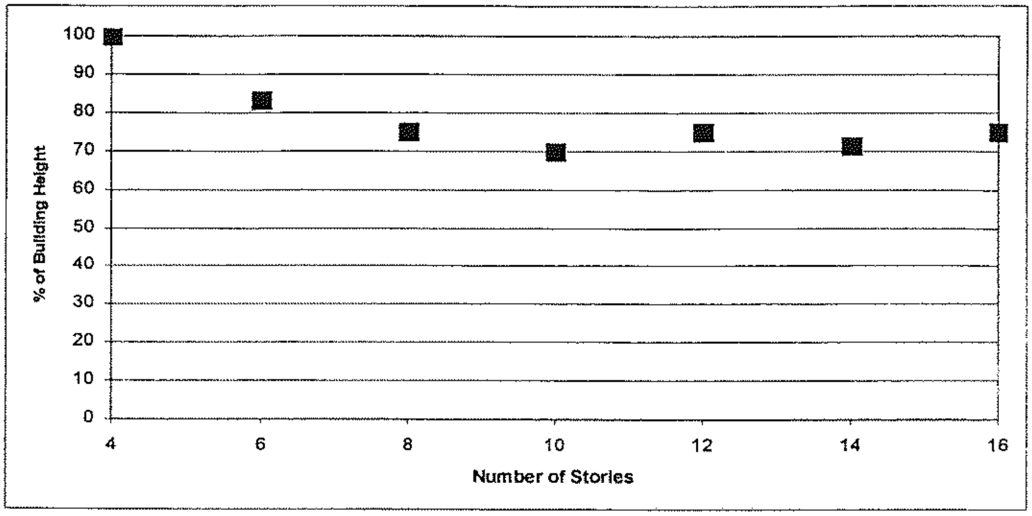


FIGURE 2.13
Lowest Floor Level Where Strength Needs to be Reduced in Girders for Structural Mechanism to Form

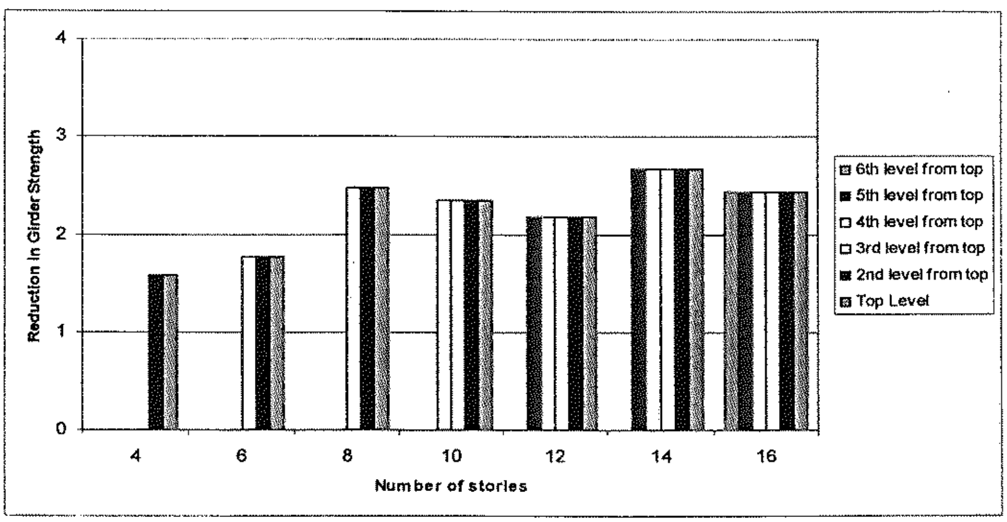


FIGURE 2.14
Uniform Reduction in Girder Strength in Upper Levels of Building Necessary to Form Structural Mechanism
Linear Load Distribution and Initial Global $\alpha = 1.20$

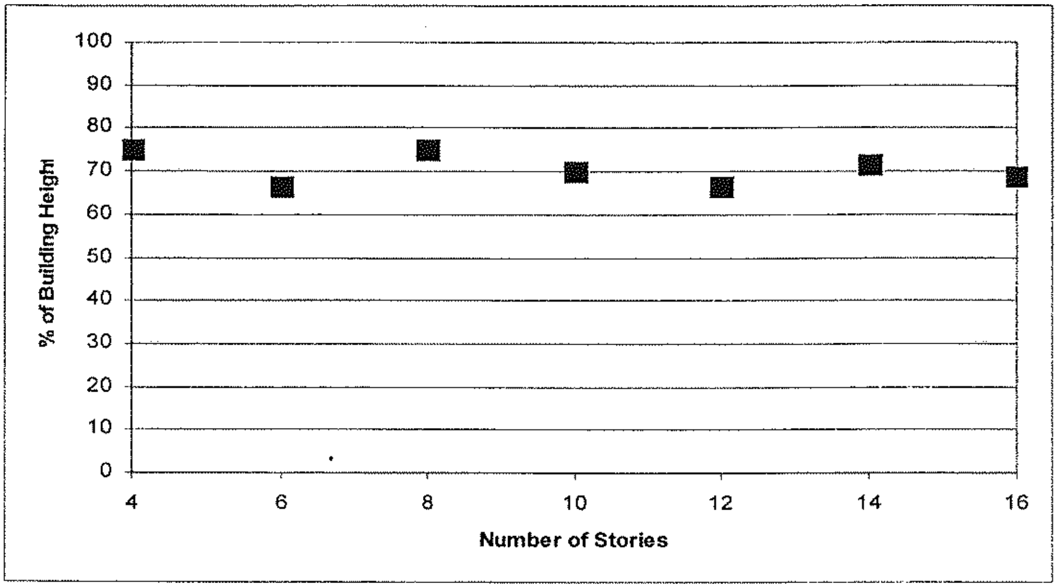


FIGURE 2.15
Lowest Floor Level Where Strength Needs to be Reduced in Girders for Structural Mechanism to Form

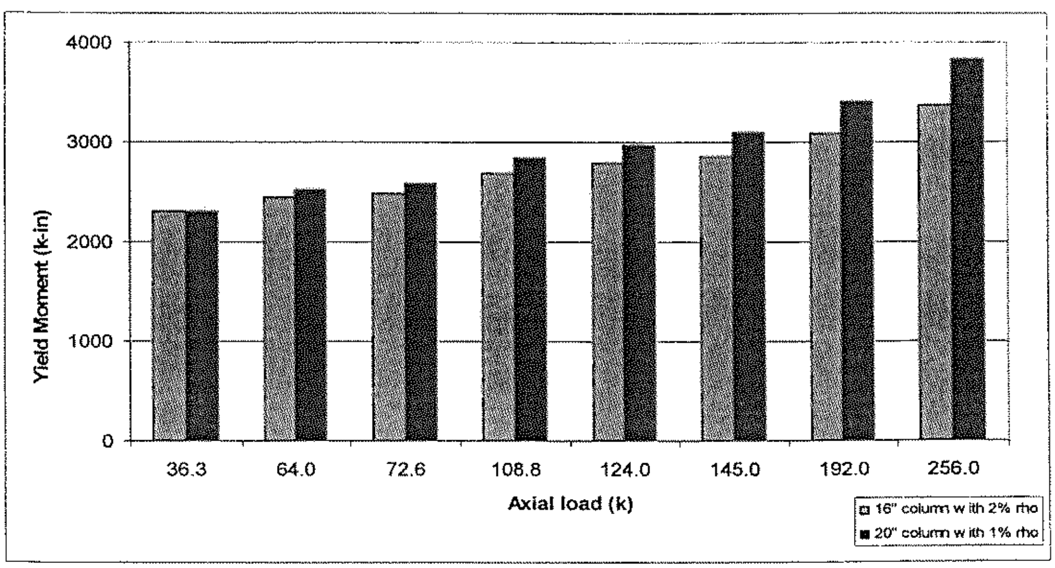


FIGURE 2.16
Increase in Yield Moment with Increasing Axial Load for a 16" and 20" Column

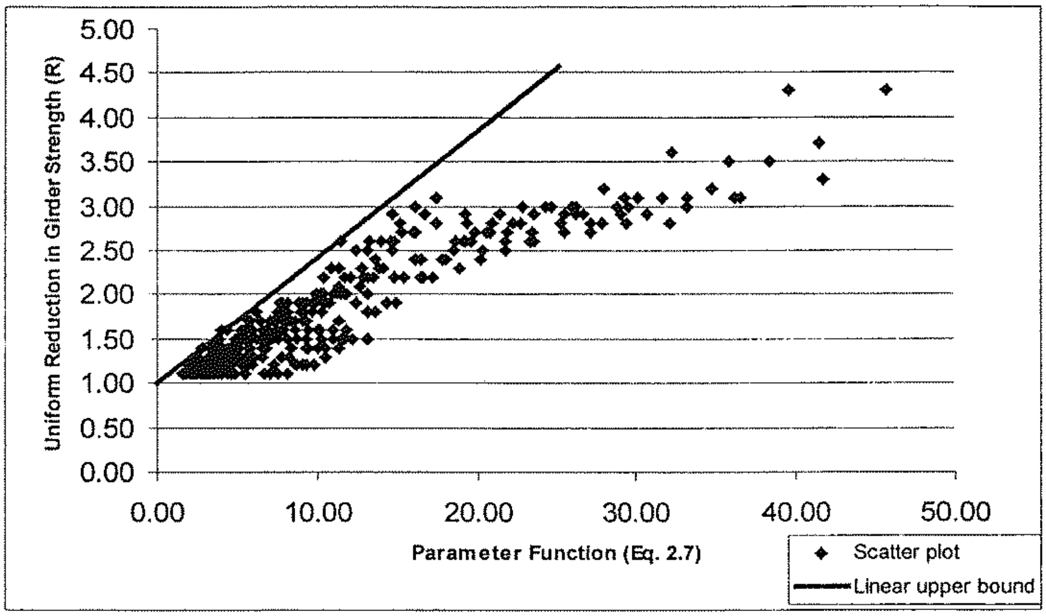


FIGURE 2.17
Plot of Parameter Function Vs. Uniform Reduction in Girder Strength Necessary to Form the Structural Mechanism

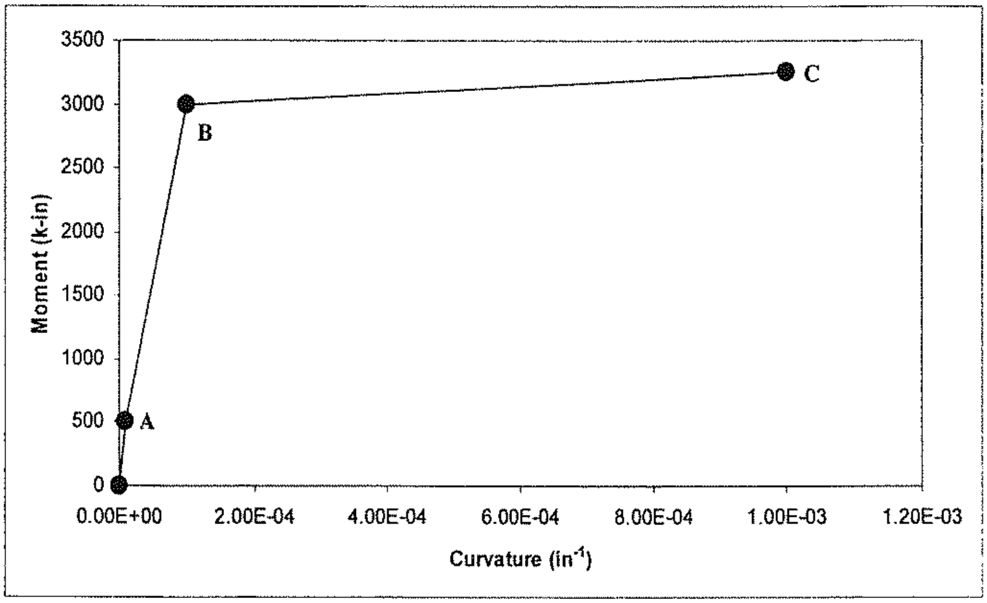


FIGURE 3.1
Example of Tri-Linear Moment Curvature Relationship

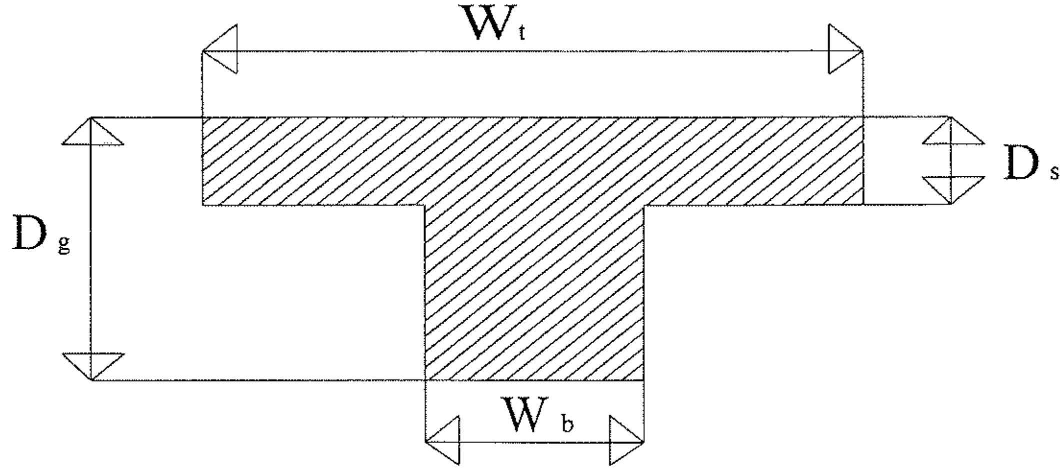


FIGURE 3.2
T-Girder Dimensions

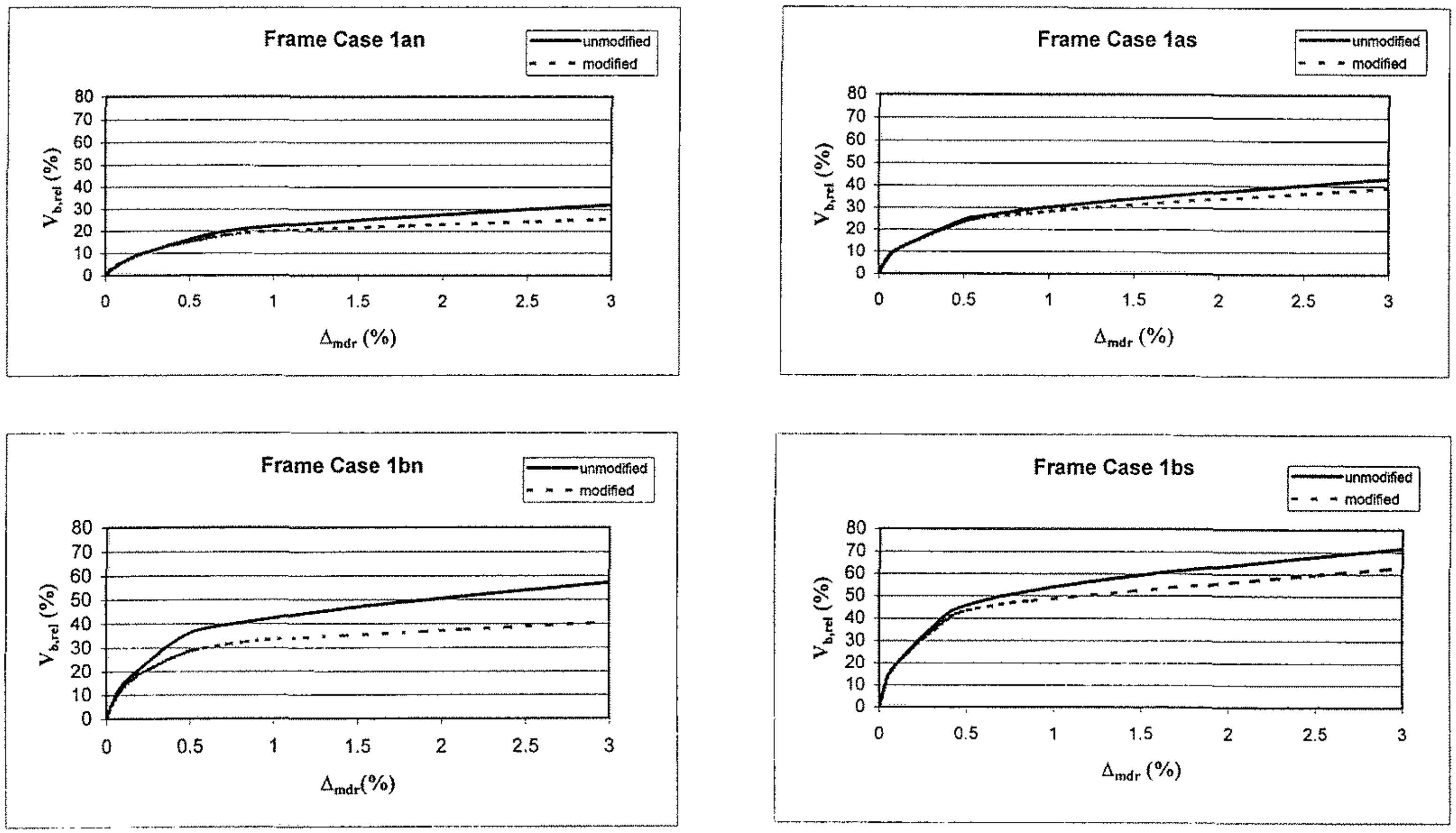


FIGURE 3.3 (a)
4 Story Frame Cases
Relative Base Shear ($V_{b,rel}$) Vs. Mean Drift Ratio (Δ_{mdr})

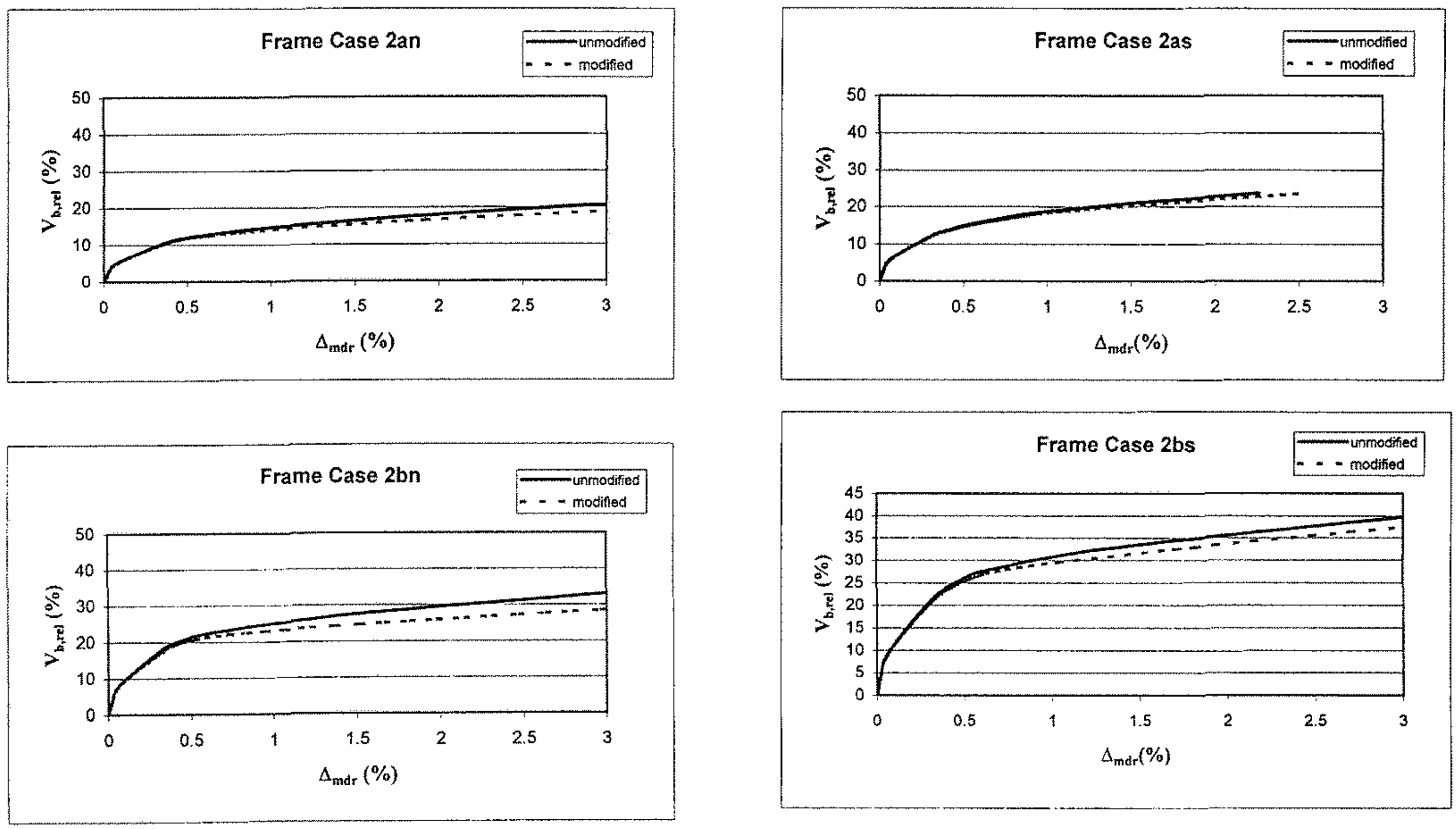


FIGURE 3.3 (b)
8 Story Frame Cases
Relative Base Shear ($V_{b,rel}$) Vs. Mean Drift Ratio (Δ_{mdr})

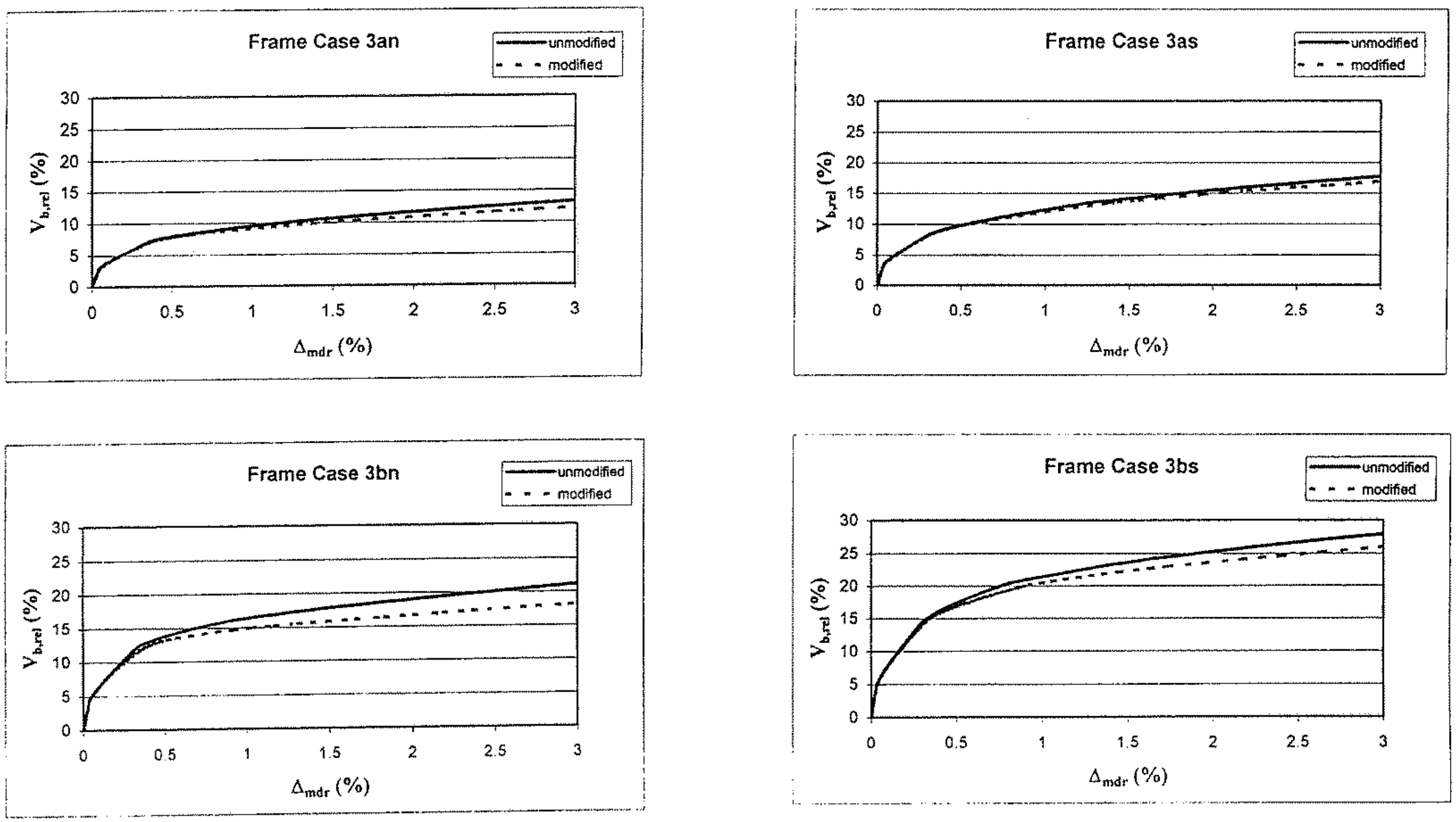


FIGURE 3.3 (c)
12 Story Frame Cases
Relative Base Shear ($V_{b,rel}$) Vs. Mean Drift Ratio (Δ_{mdr})

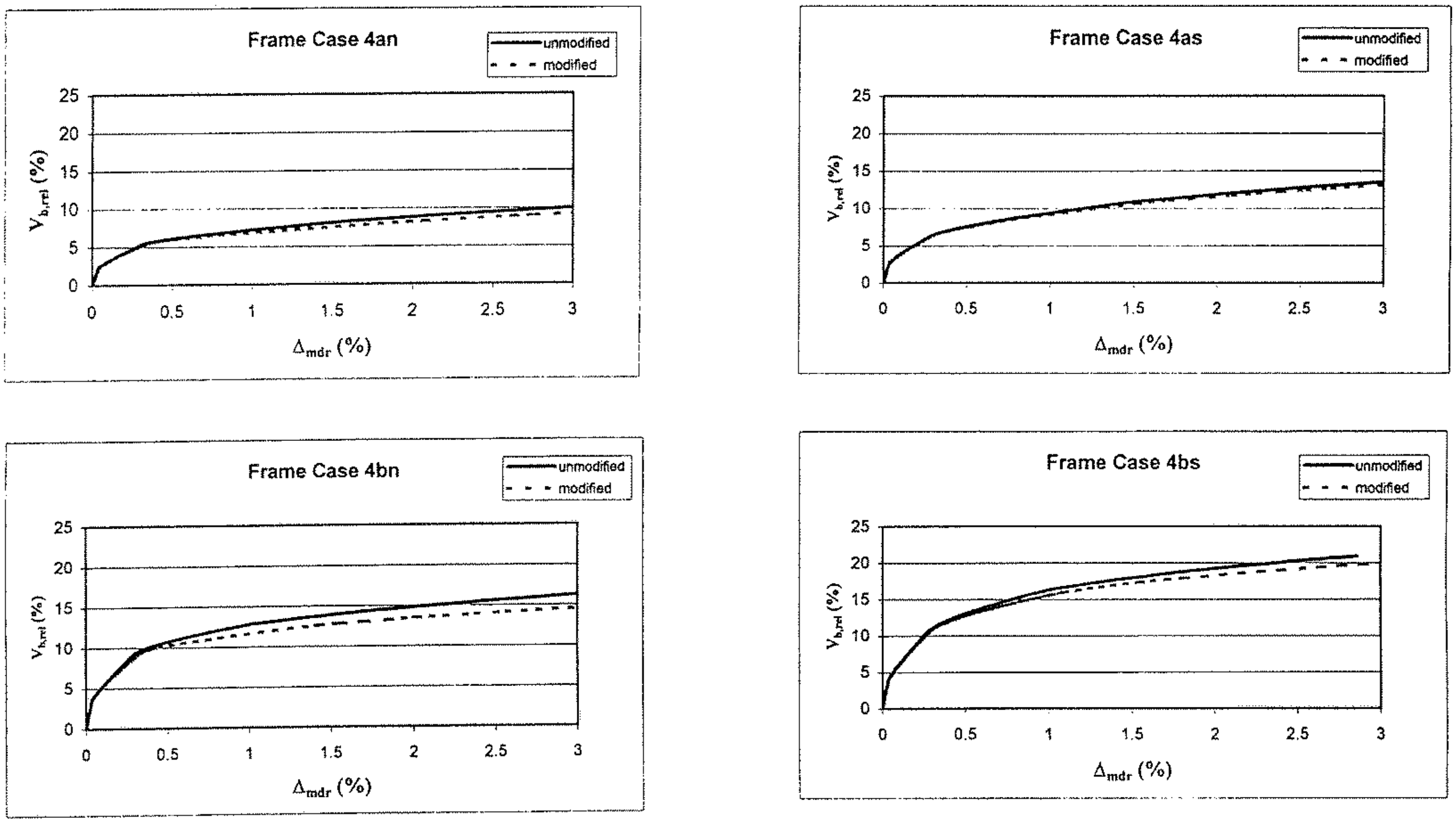


FIGURE 3.3 (d)
16 Story Frame Cases
Relative Base Shear ($V_{b,rel}$) Vs. Mean Drift Ratio (Δ_{mdr})

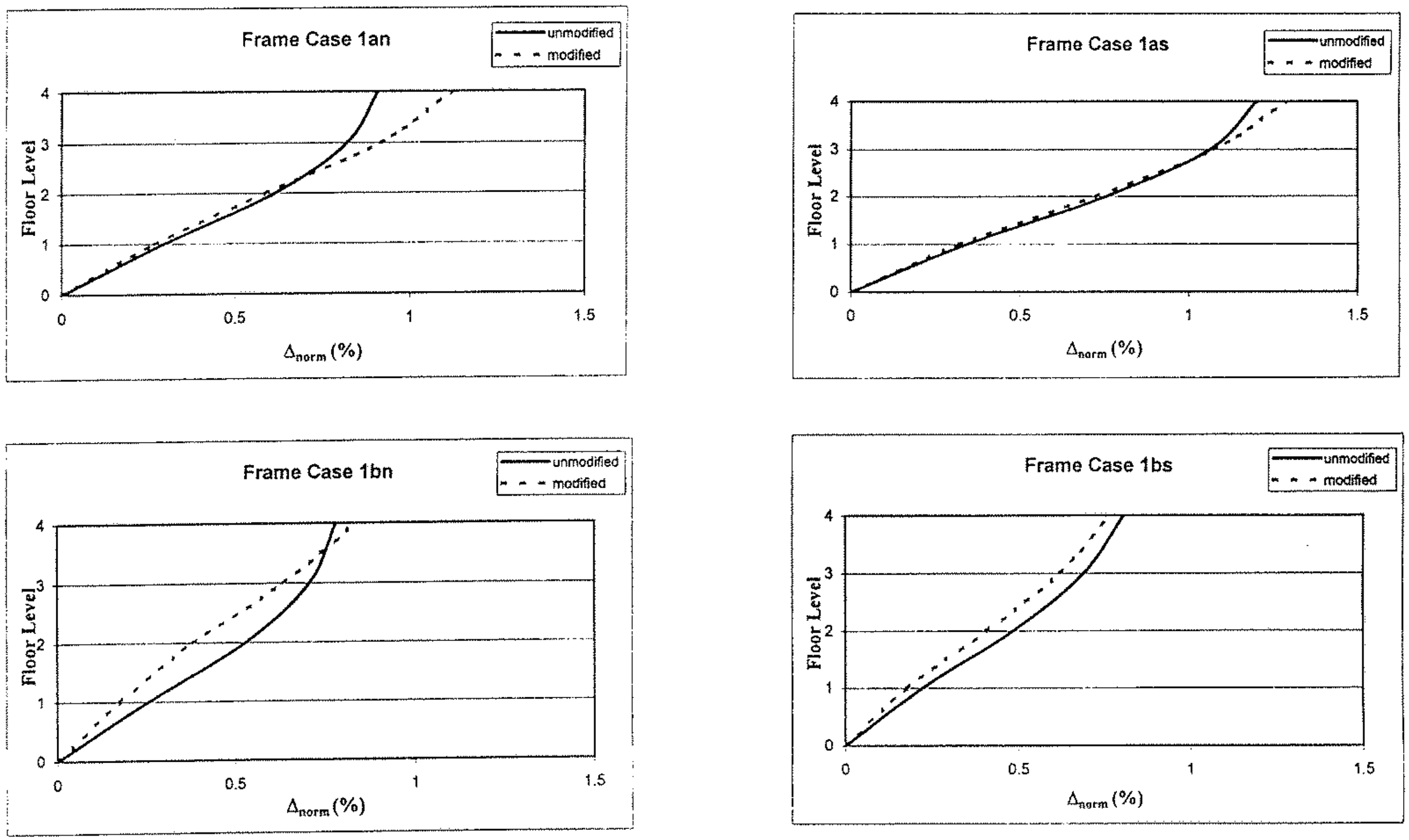


FIGURE 3.4 (a)
4 Story Frame Cases
Normalized Drift (Δ_{norm}) Vs. Floor Level

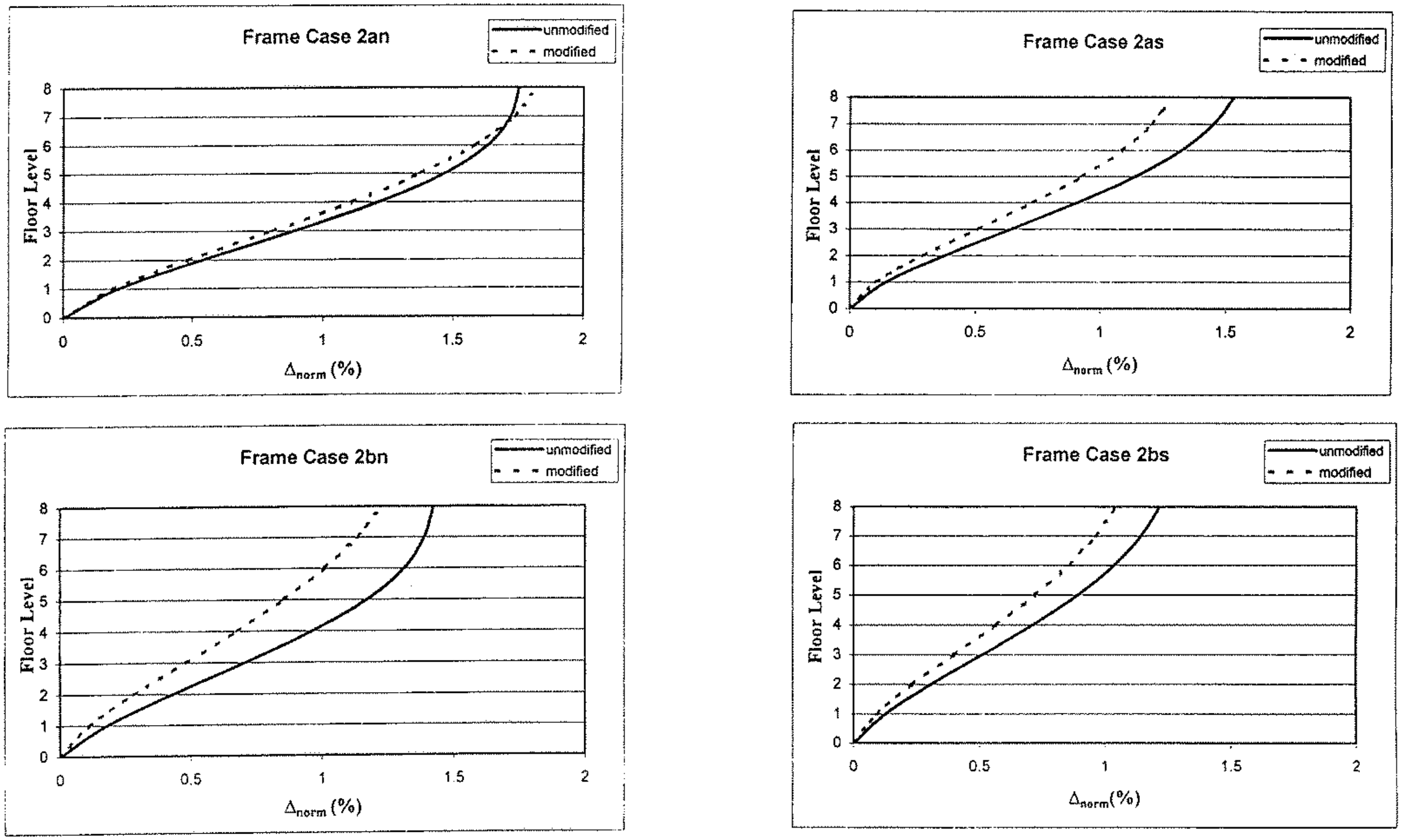


FIGURE 3.4 (b)
8 Story Frame Cases
Normalized Drift (Δ_{norm}) Vs. Floor Level

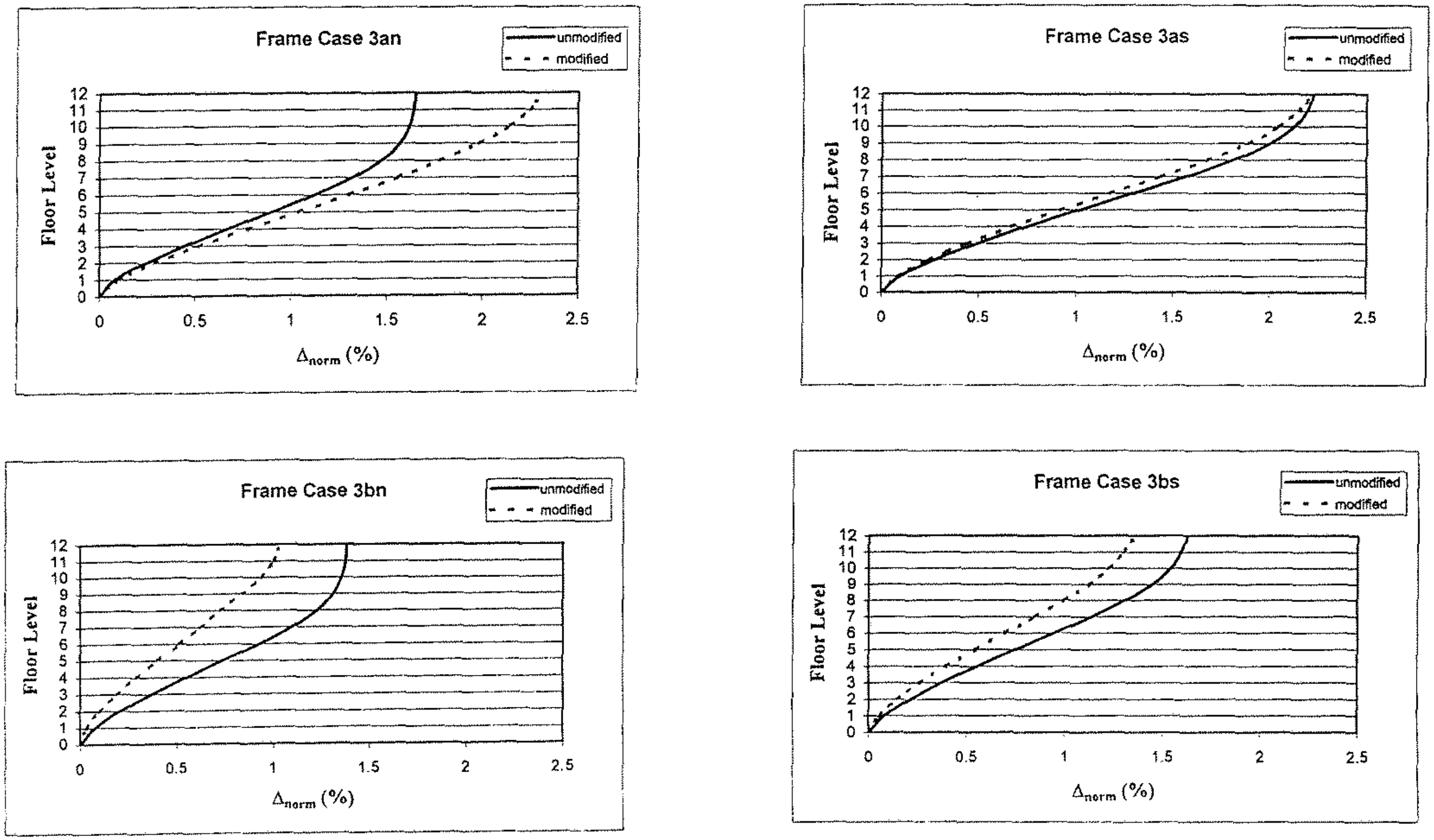


FIGURE 3.4 (c)
12 Story Frame Cases
Normalized Drift (Δ_{norm}) Vs. Floor Level

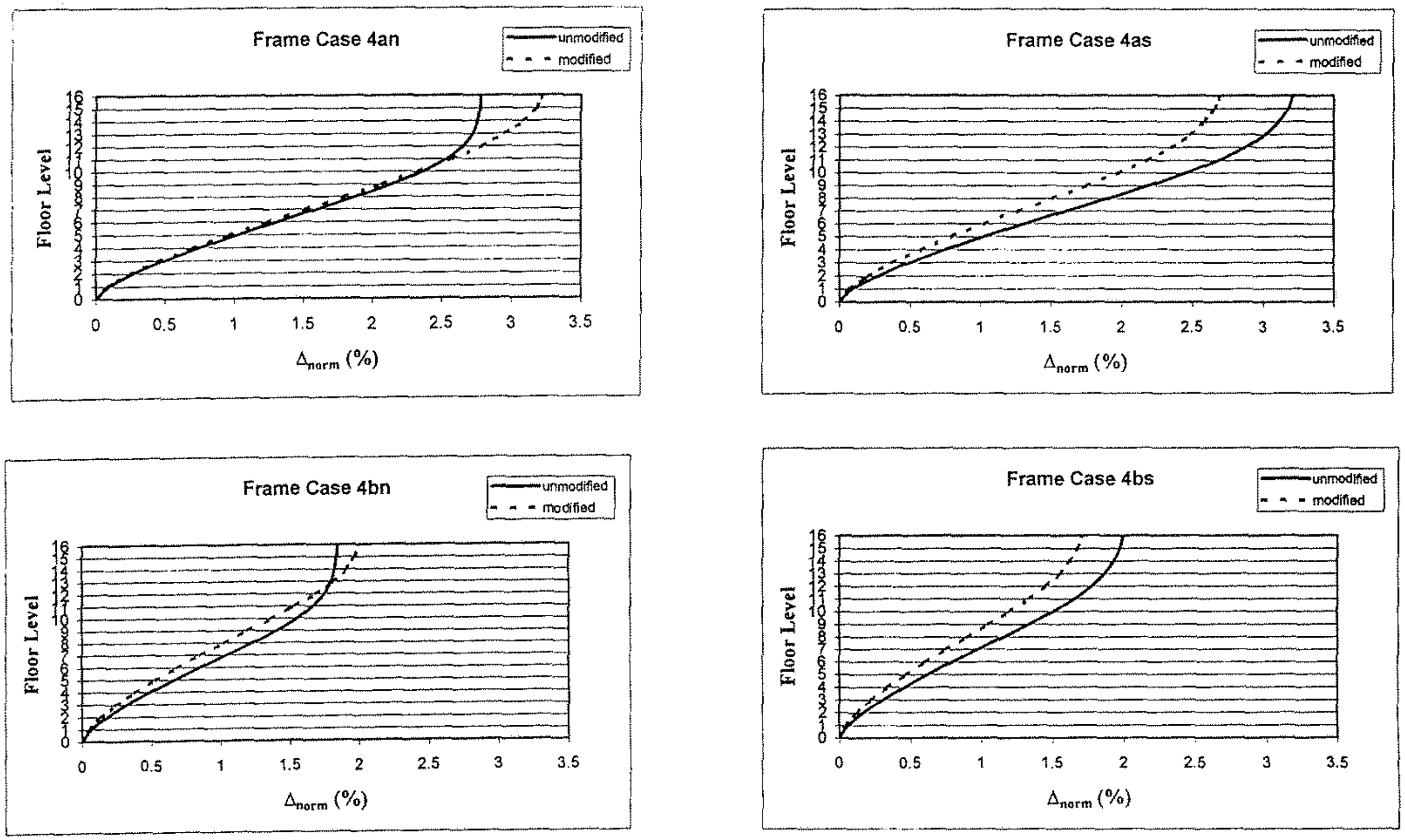


FIGURE 3.4 (d)
16 Story Frame Cases
Normalized Drift (Δ_{norm}) Vs. Floor Level

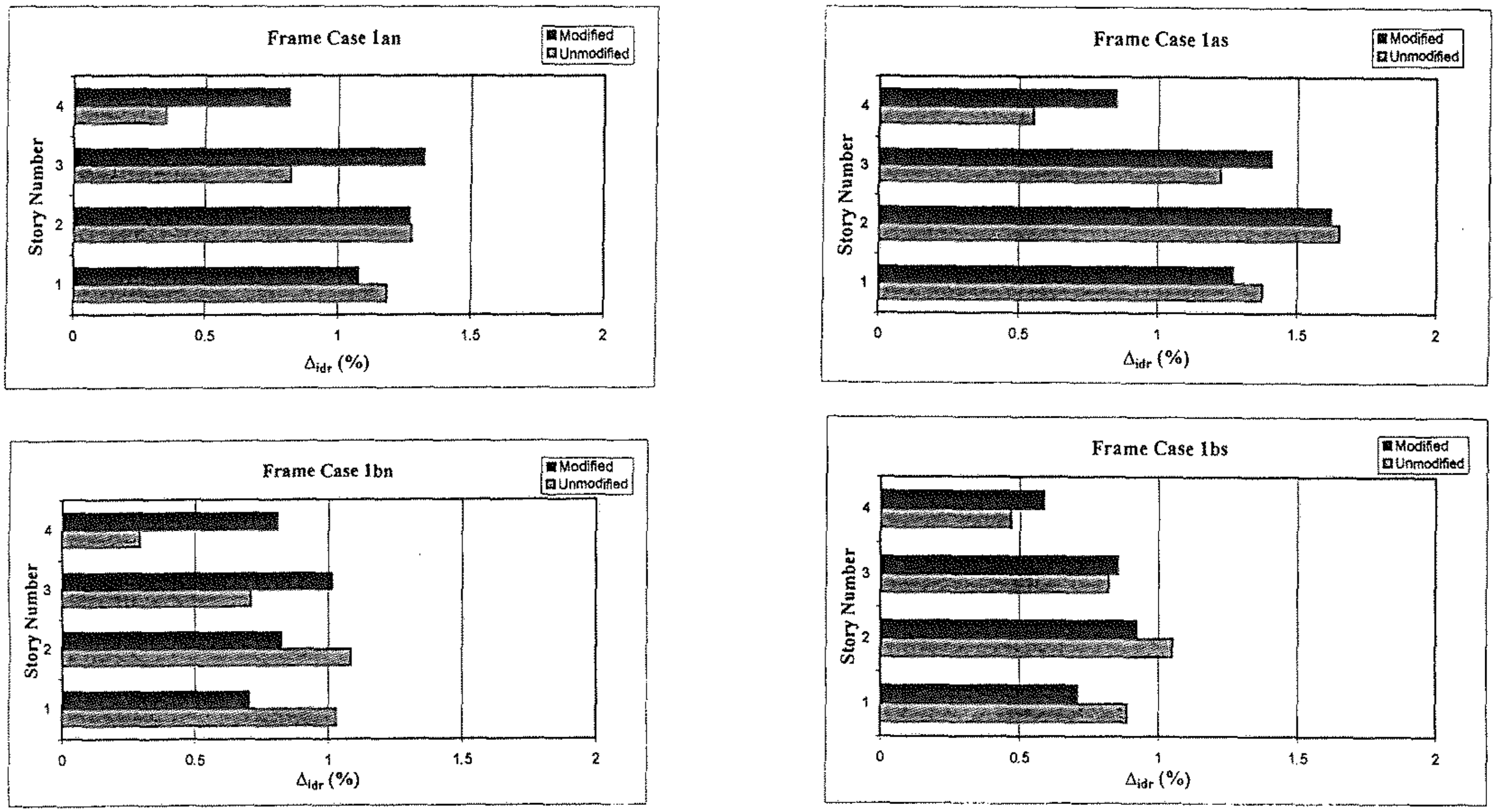


FIGURE 3.5 (a)
4 Story Frame Cases
Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

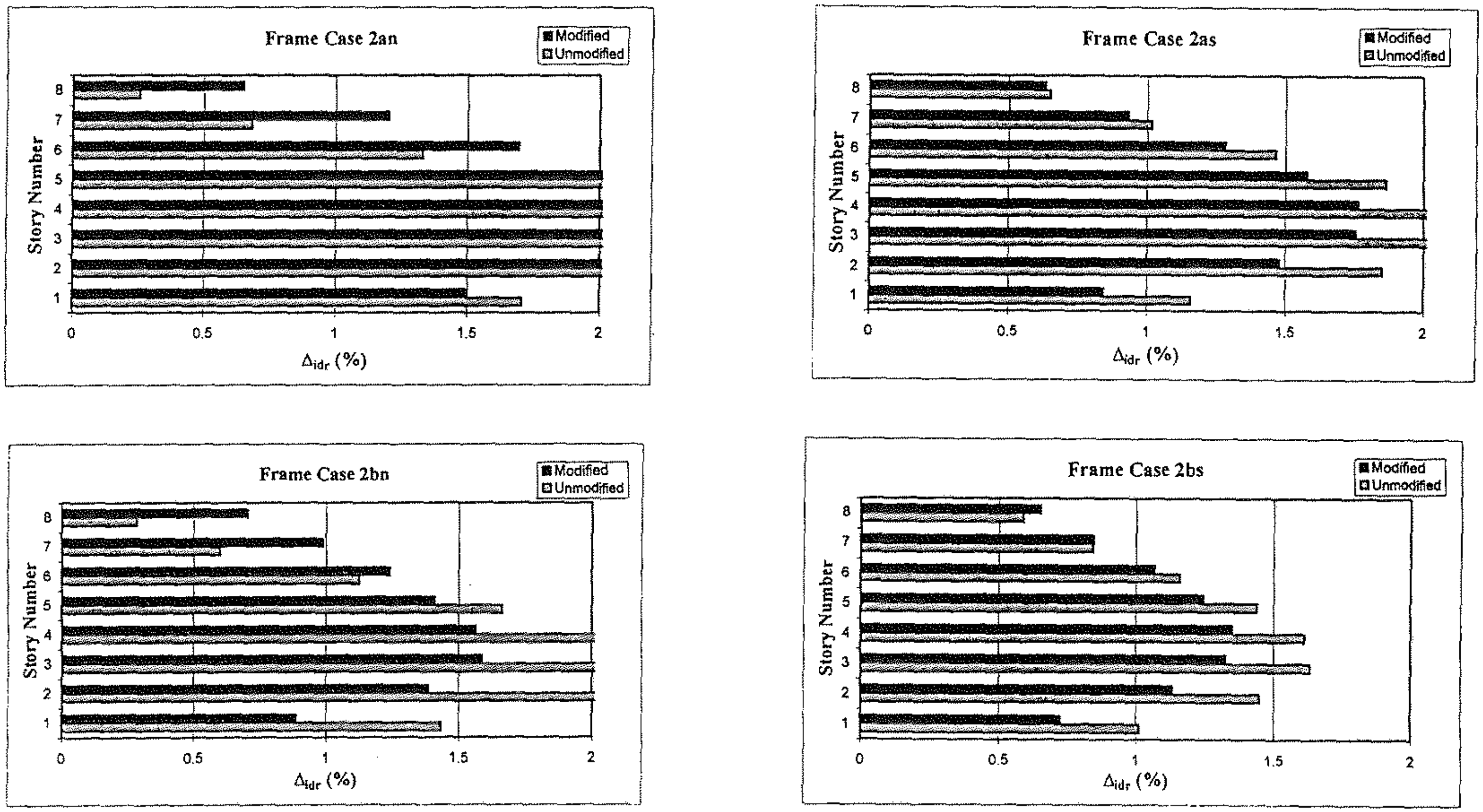
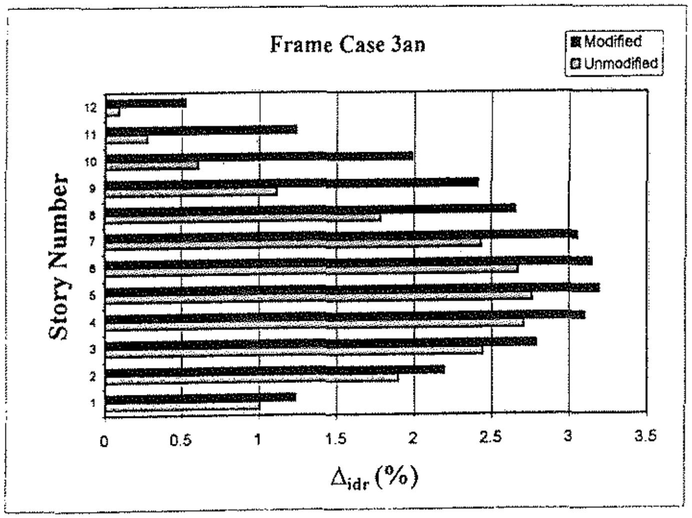
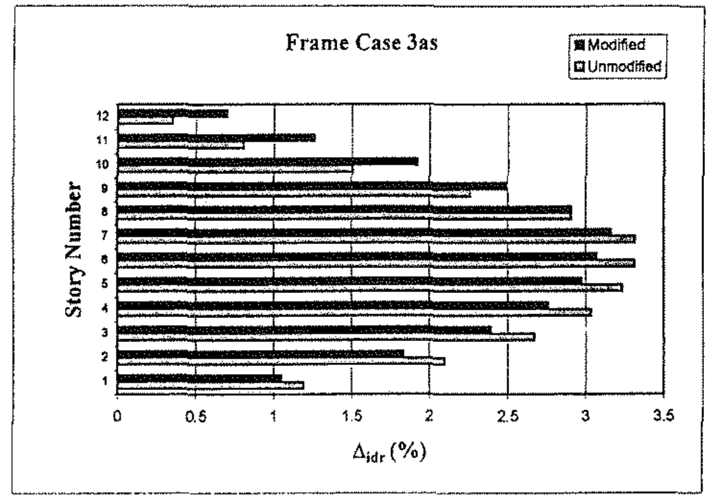
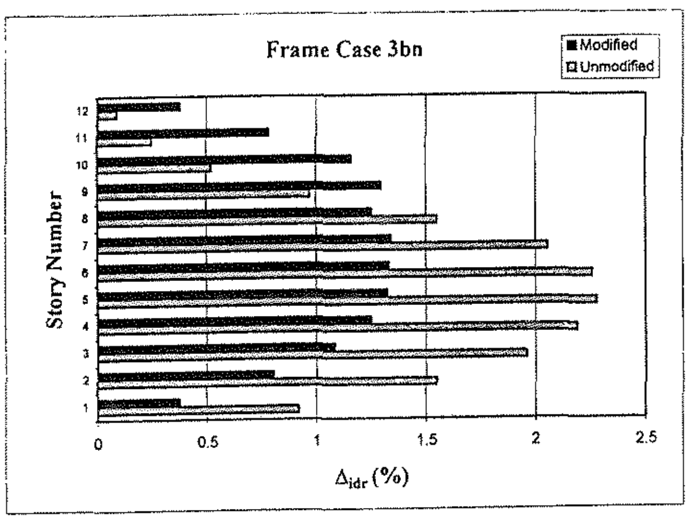


FIGURE 3.5 (b)
8 Story Frame Cases
Interstory Drift Ratio (Δ_{idr}) Vs. Story Number







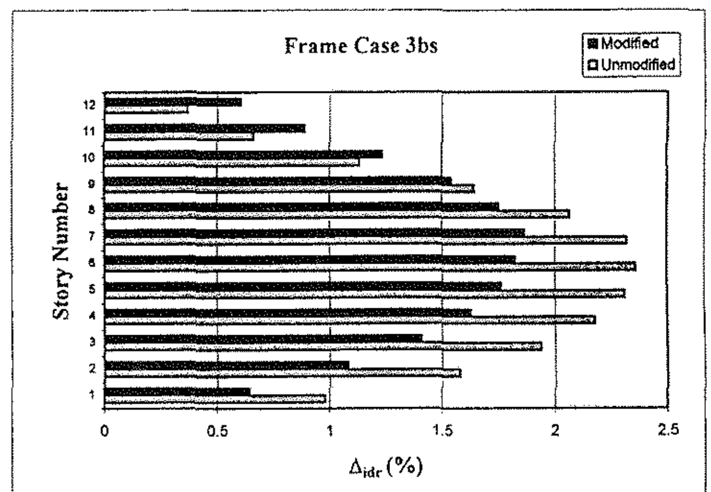


FIGURE 3.5 (c)
12 Story Frame Cases
Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

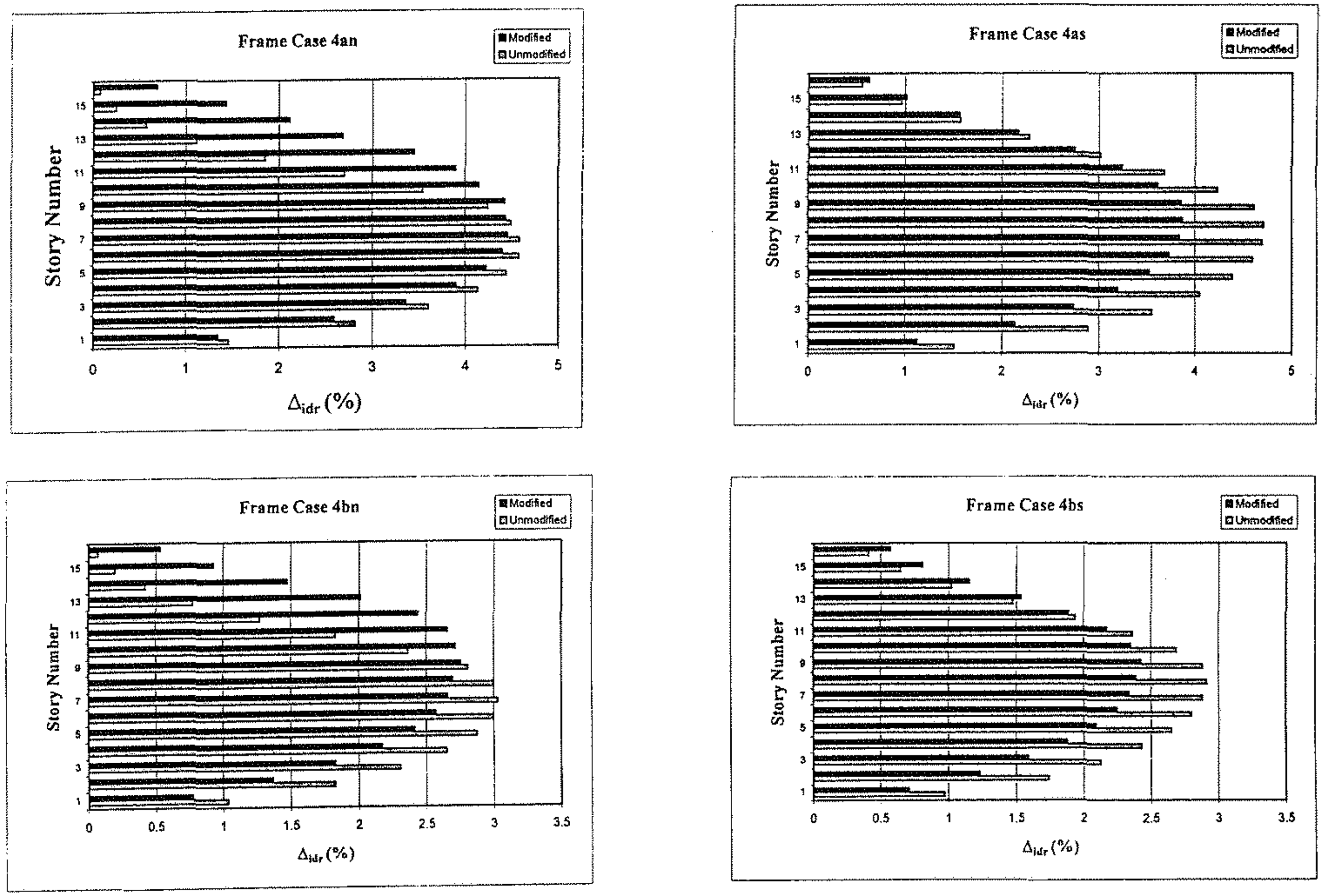


FIGURE 3.5 (d)
16 Story Frame Cases
Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

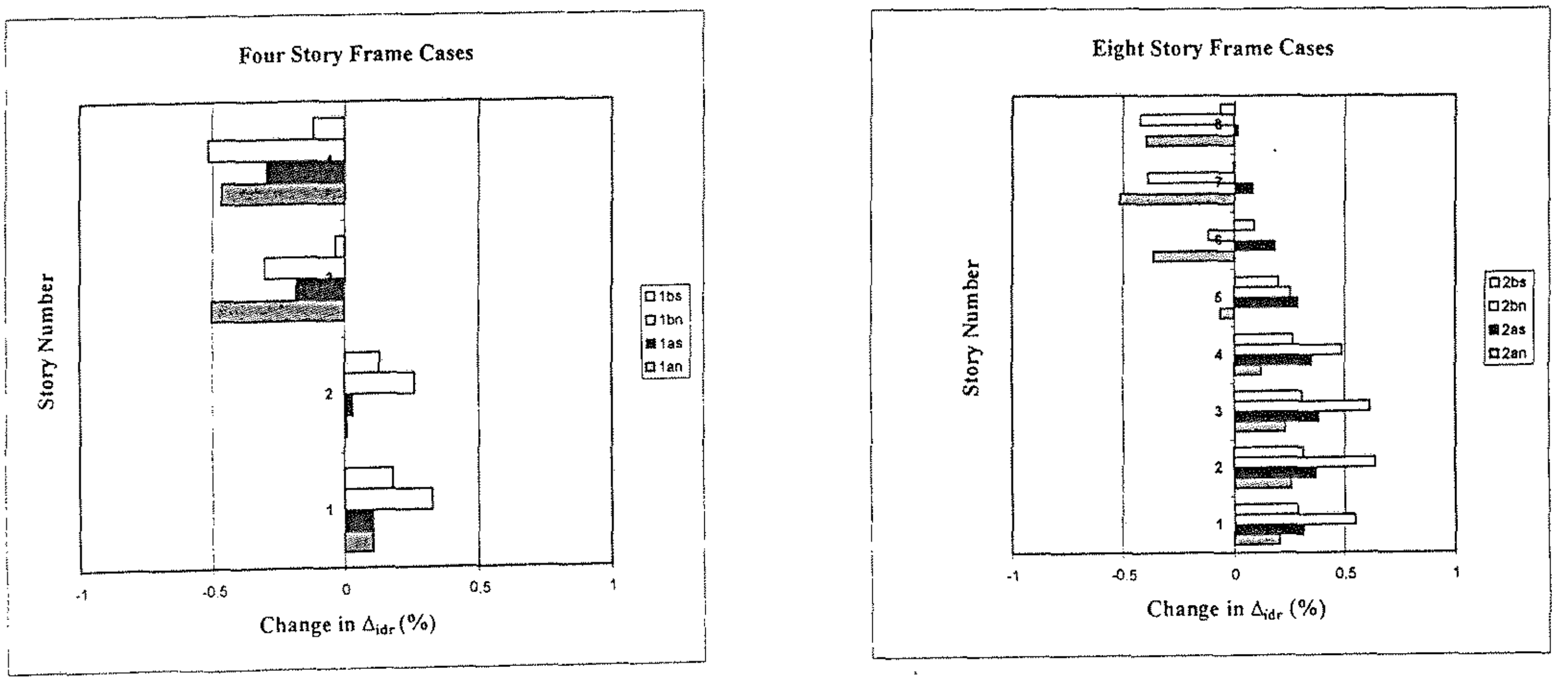


FIGURE 3.6 (a)
4 & 8 Story Frame Cases
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

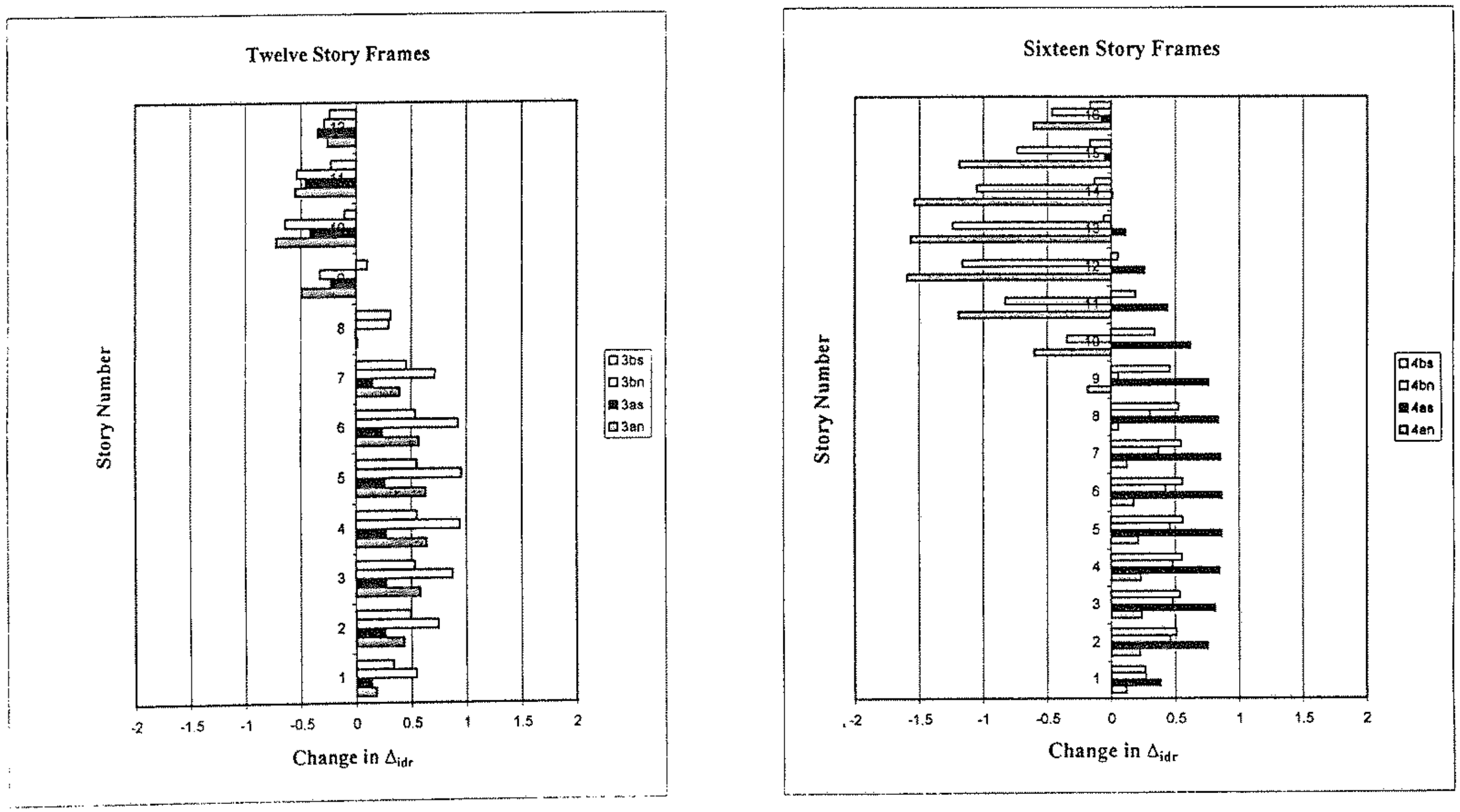


FIGURE 3.6 (b)
12 & 16 Story Frame Cases
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

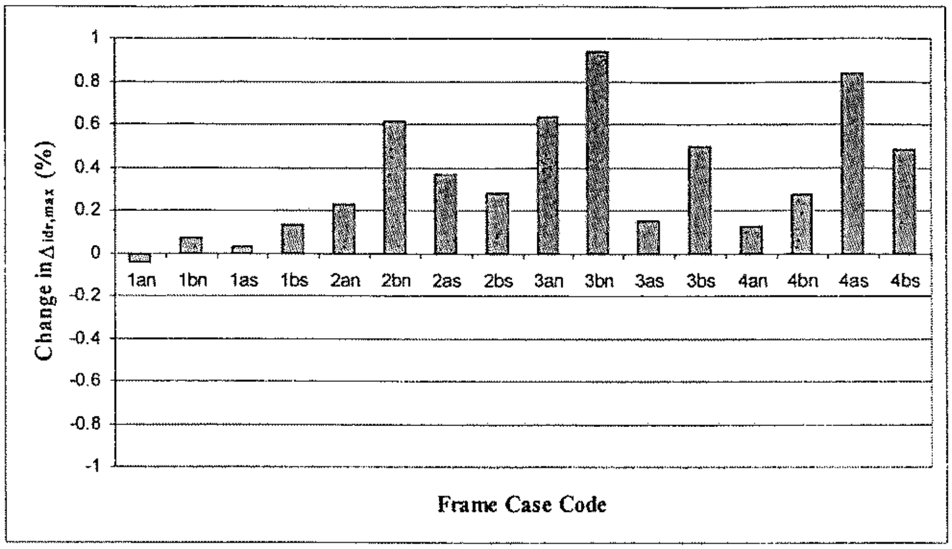


FIGURE 3.7
Change in Maximum Interstory Drift Ratio (%)
Sixteen Frame Cases
Static Analysis

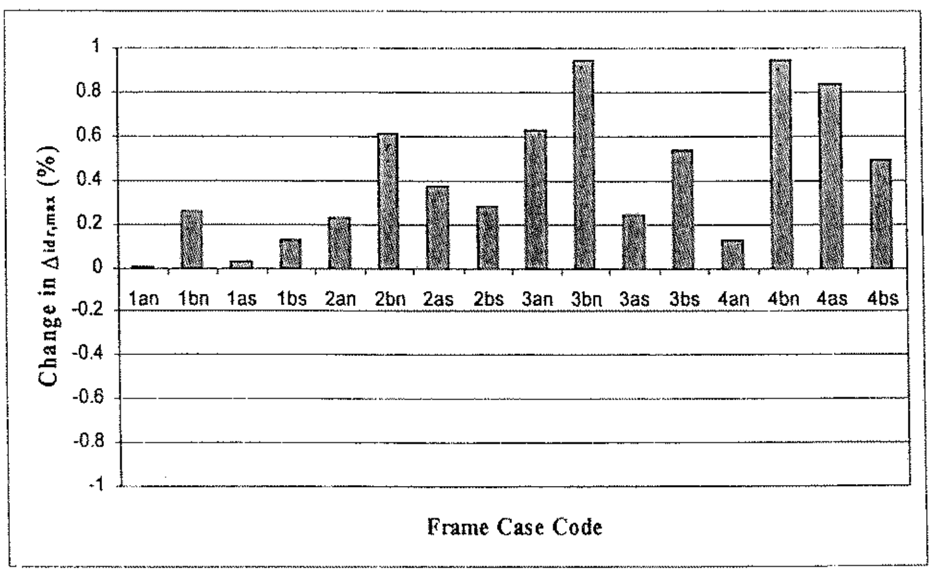


FIGURE 3.8
Change in Maximum Interstory Drift Ratio (%)
Stories in the Bottom Half of the Frames
Sixteen Frame Cases
Static Analysis

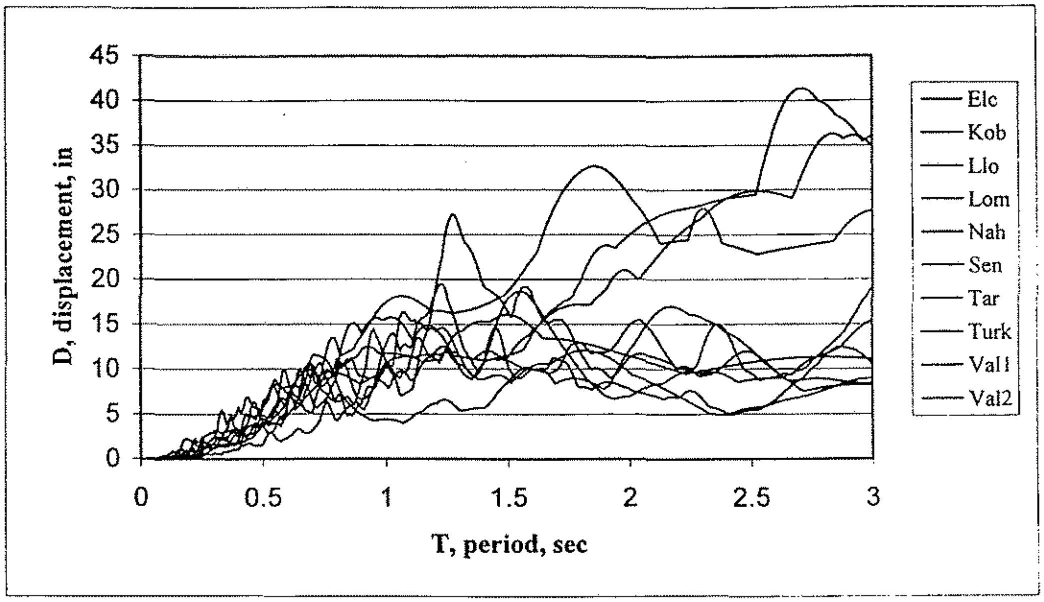


FIGURE 3.9
Displacement Response Spectra
Ten Scaled Earthquakes

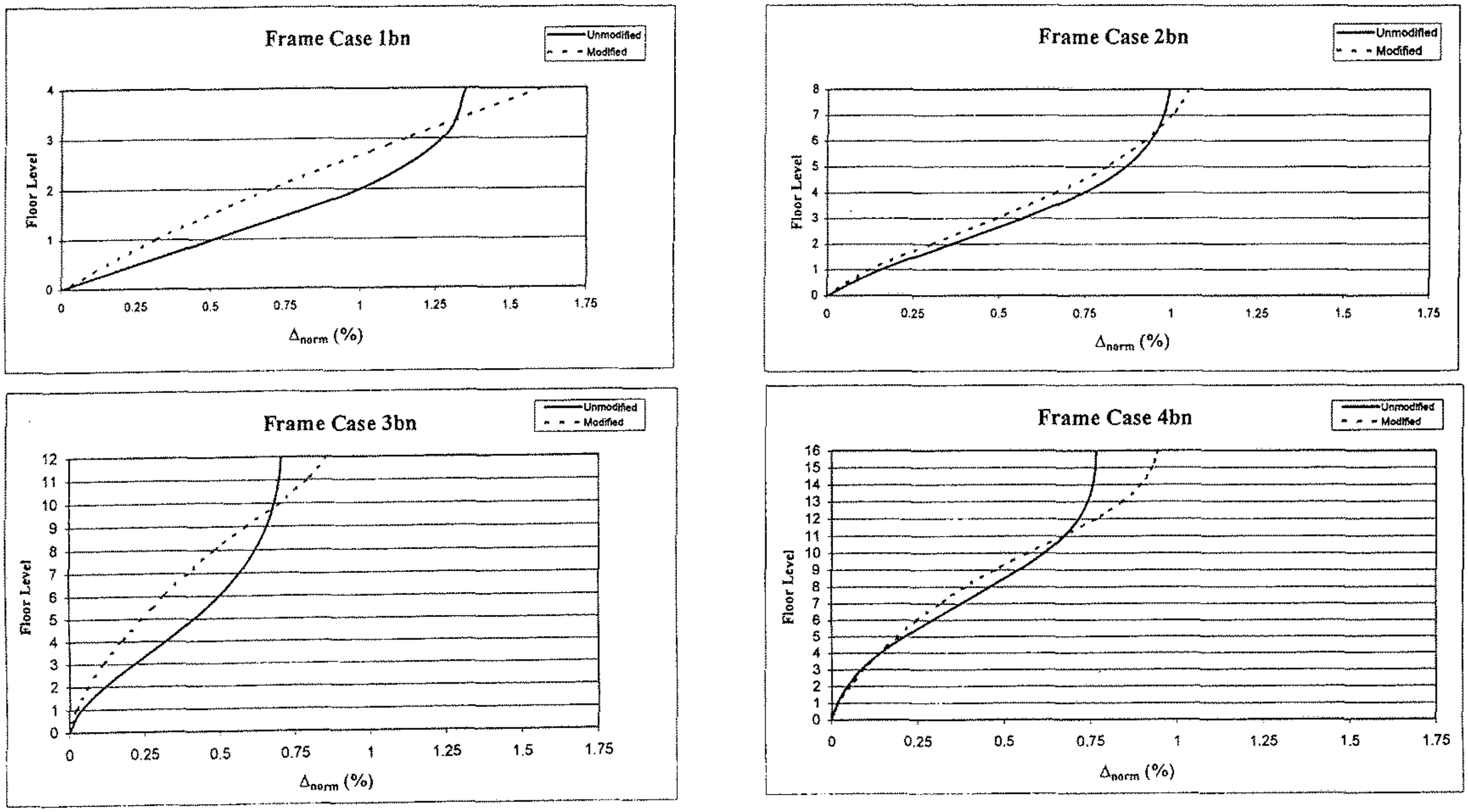


FIGURE 3.10 (a)
Unscaled Loma Prieta Earthquake
Normalized Drift (Δ_{norm}) Vs. Floor Level
4, 8, 12, and 16 Story Frame Cases (30 ft. Bays and Column Stiffness Factor = 0.35)

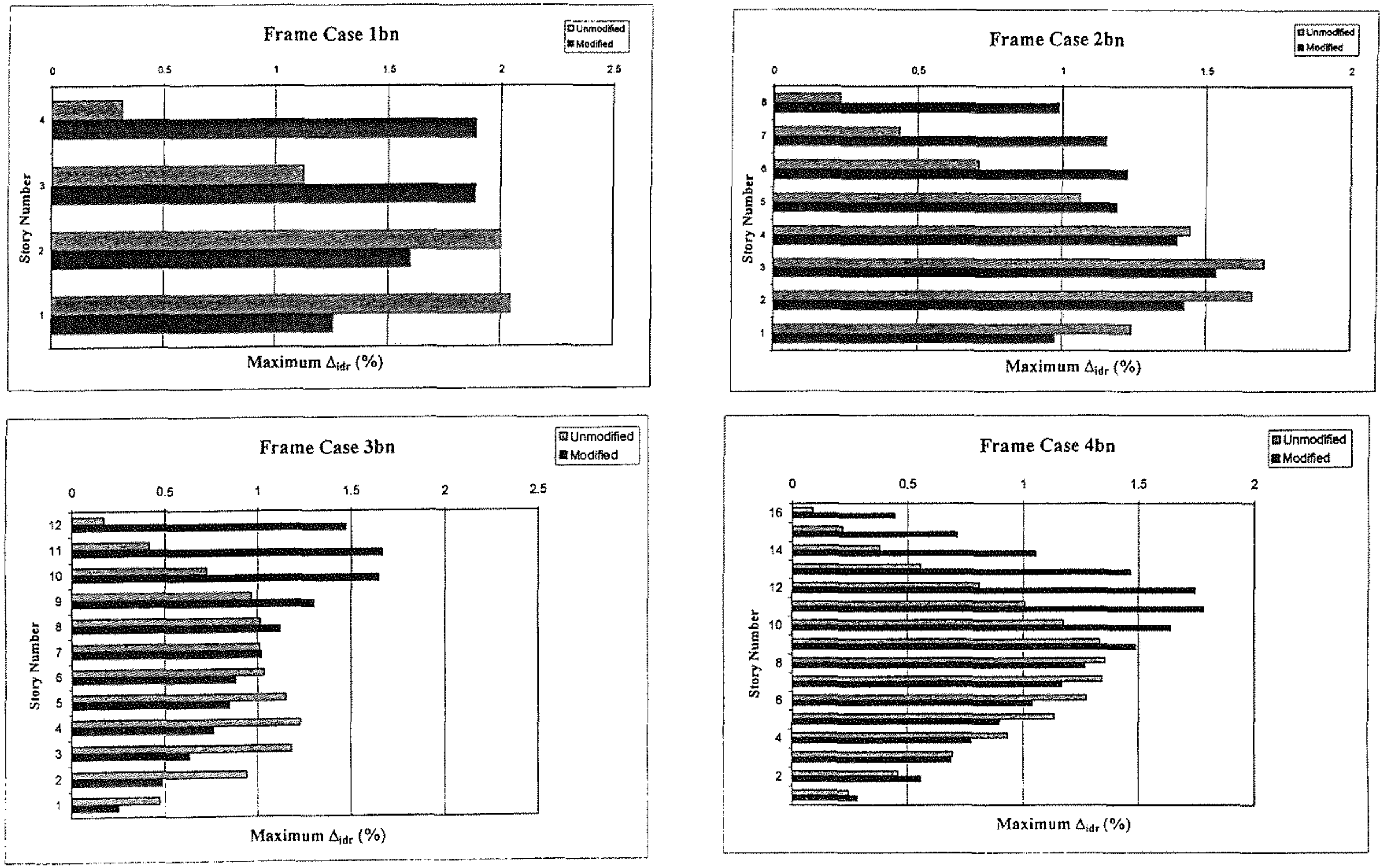


FIGURE 3.10 (b)
Unscaled Loma Prieta Earthquake
Maximum Interstory Drift Ratios (Maximum Δ_{idr}) Vs. Story Number
4, 8, 12, and 16 Story Frame Cases (30 ft. Bays and Column Stiffness Factor = 0.35)

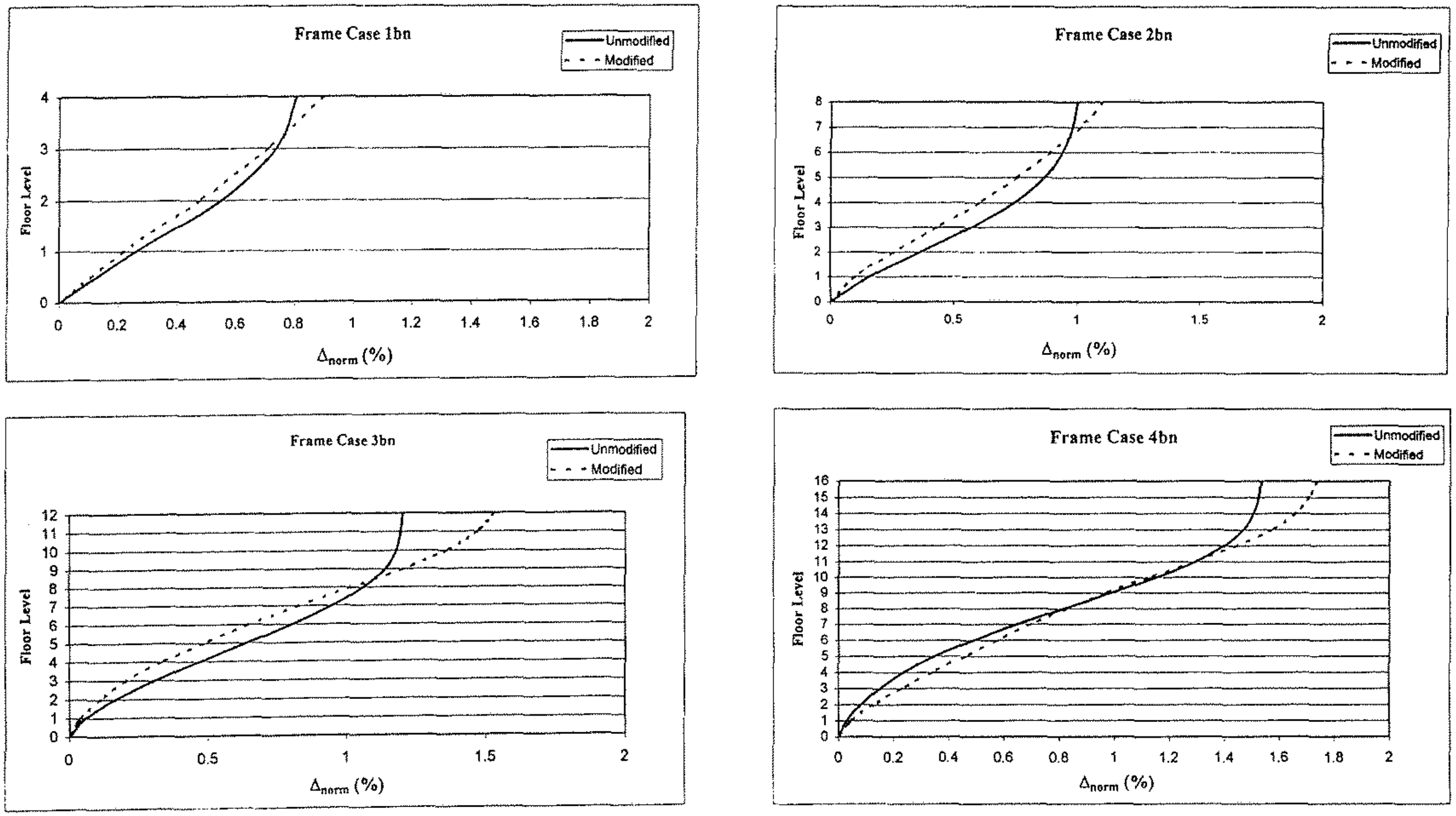


FIGURE 3.10 (c)
Scaled Loma Prieta Earthquake
Normalized Drift (Δ_{norm}) Vs. Floor Level
4, 8, 12, and 16 Story Frame Cases (30 ft. Bays and Column Stiffness Factor = 0.35)

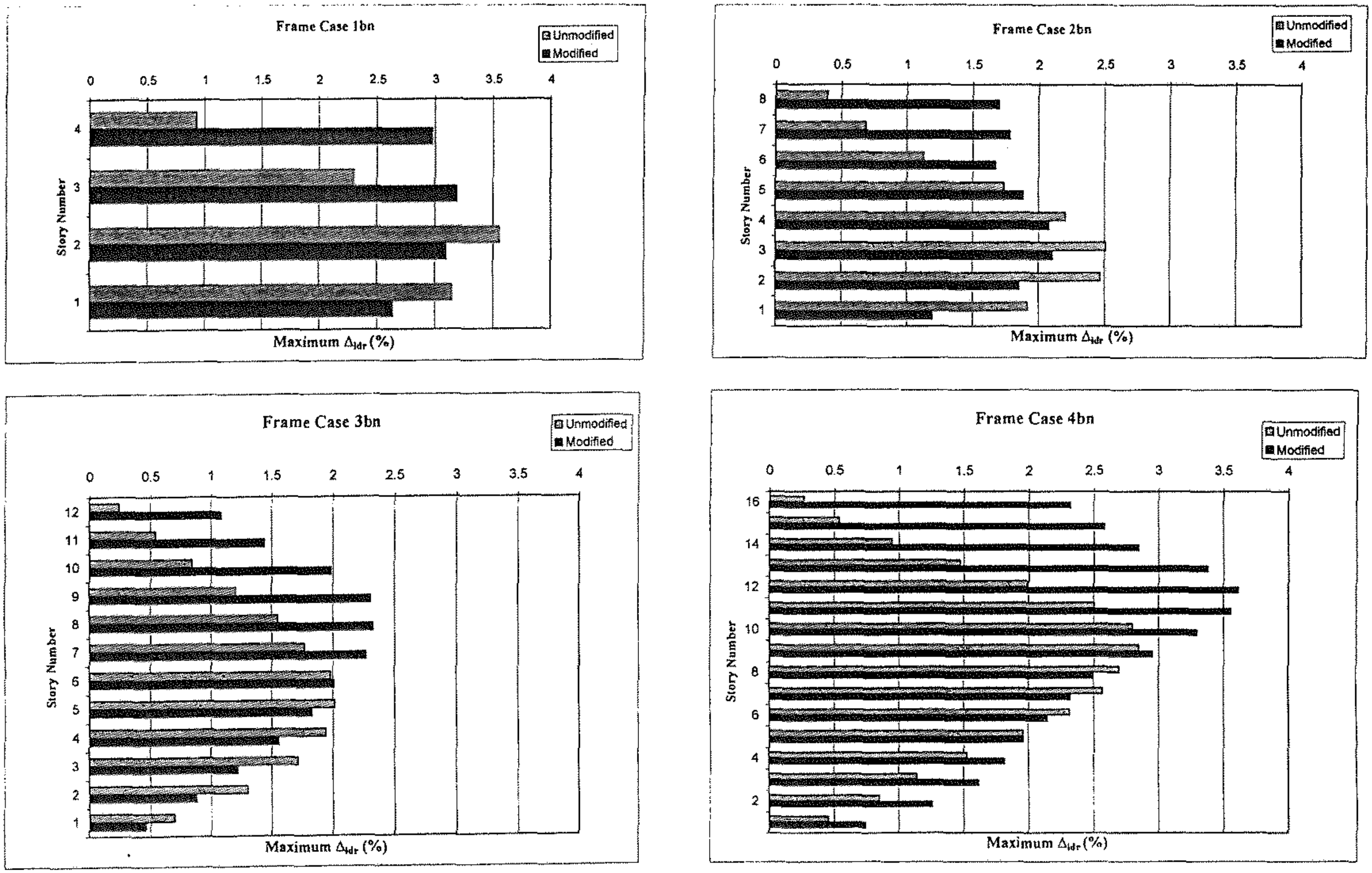


FIGURE 3.10 (d)

Scaled Loma Prieta Earthquake

Maximum Interstory Drift Ratios (Maximum Δ_{idr}) Vs. Story Number

4, 8, 12, and 16 Story Frame Cases (30 ft. Bays and Column Stiffness Factor = 0.35)

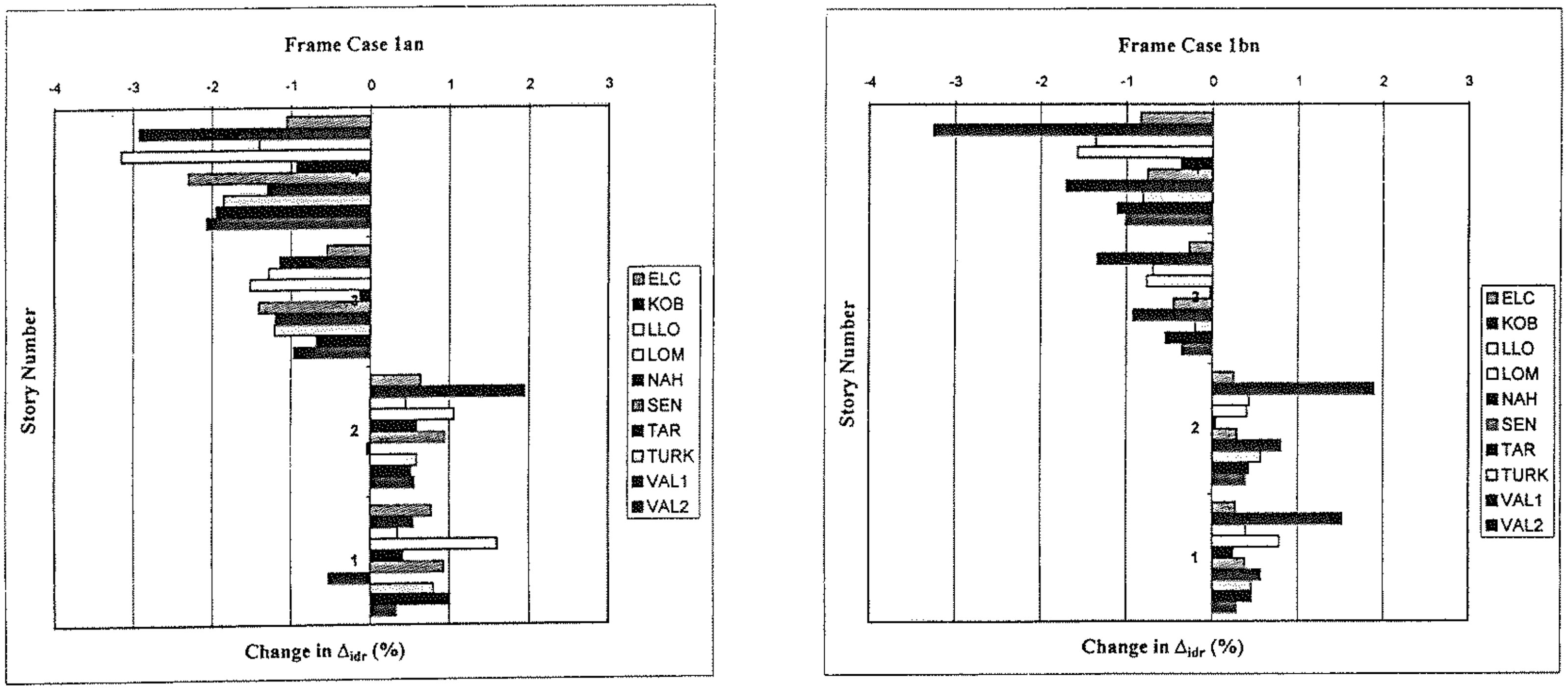


FIGURE 3.11 (a)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 1an & 1bn

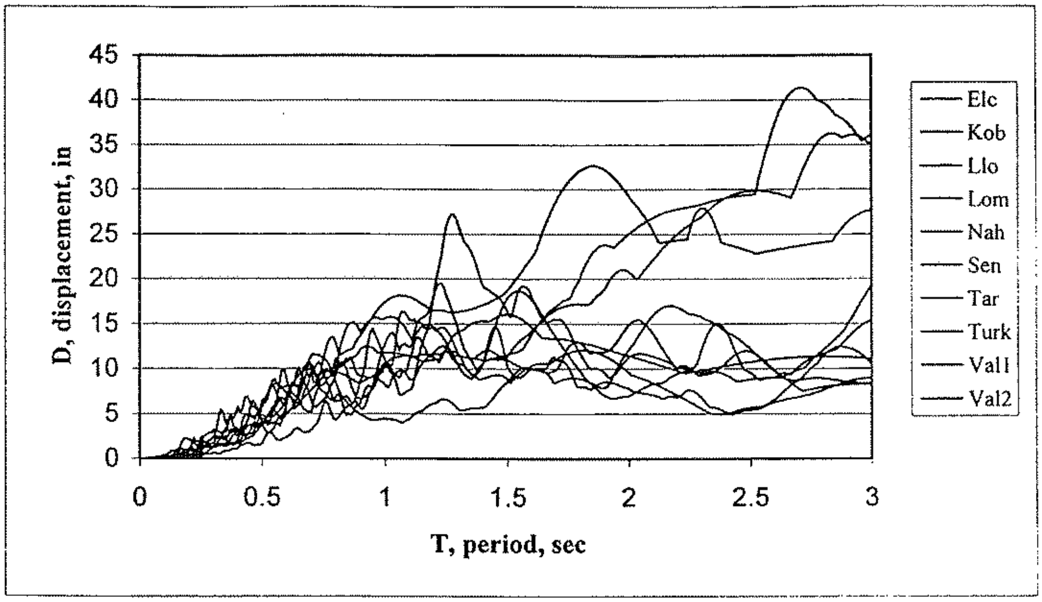


FIGURE 3.9
Displacement Response Spectra
Ten Scaled Earthquakes

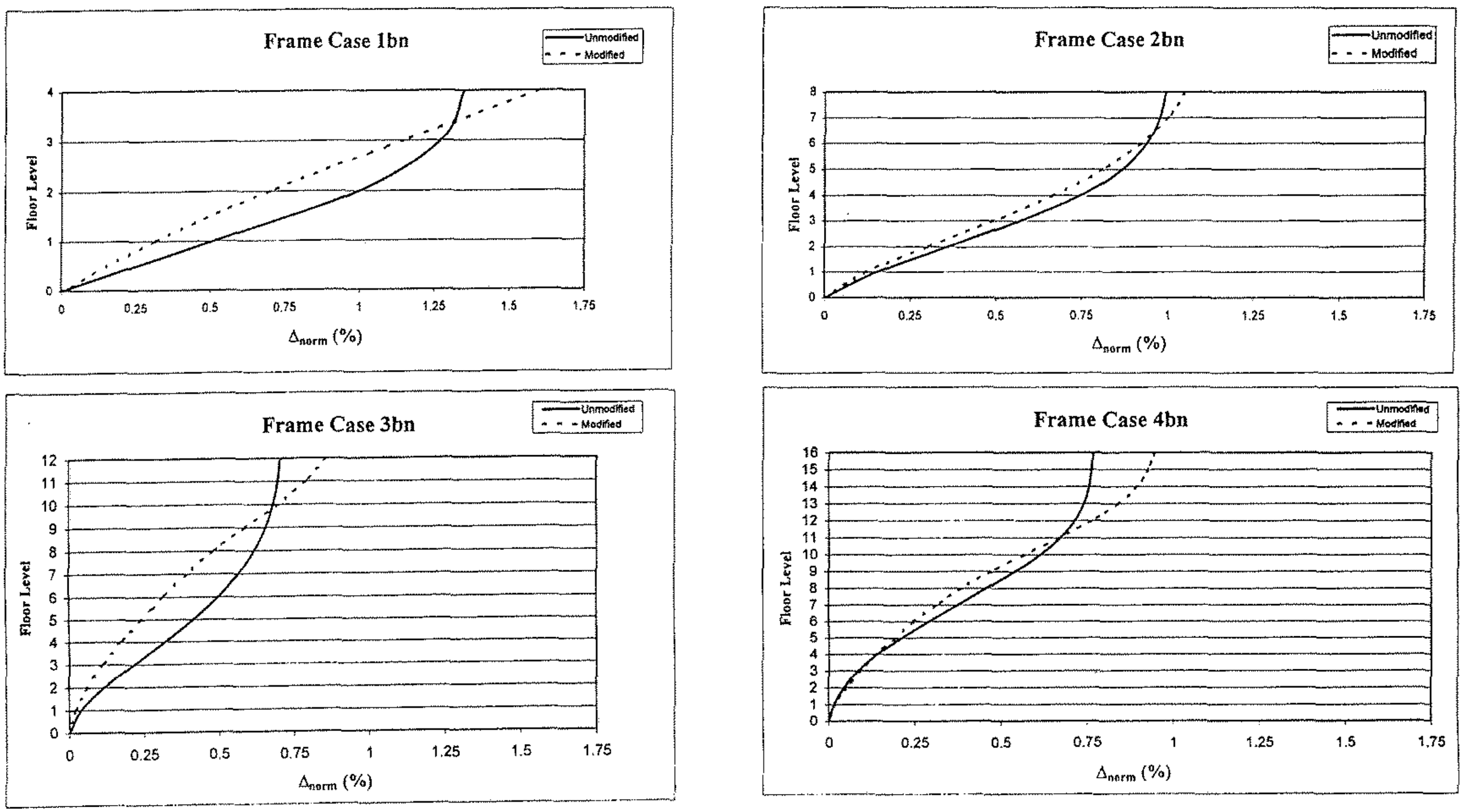
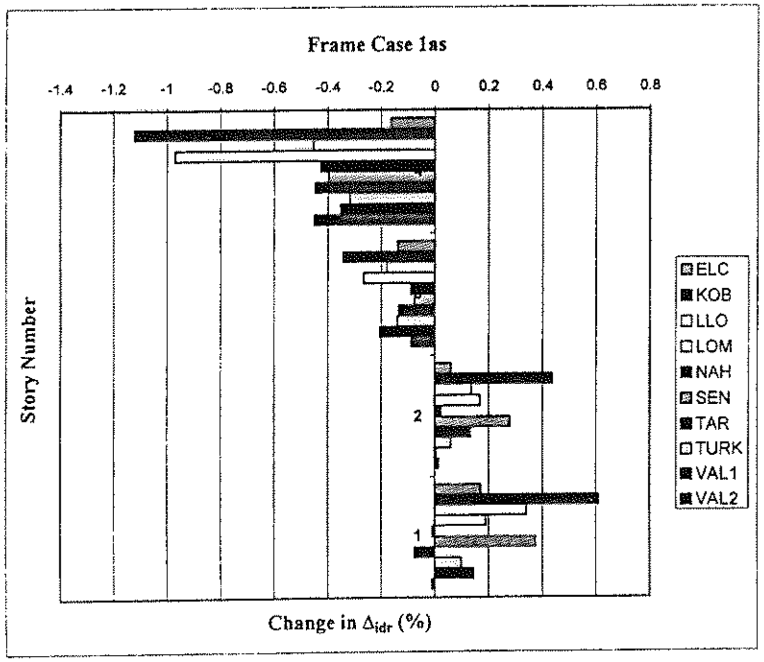


FIGURE 3.10 (a)
Unscaled Loma Prieta Earthquake
Normalized Drift (Δ_{norm}) Vs. Floor Level
4, 8, 12, and 16 Story Frame Cases (30 ft. Bays and Column Stiffness Factor = 0.35)



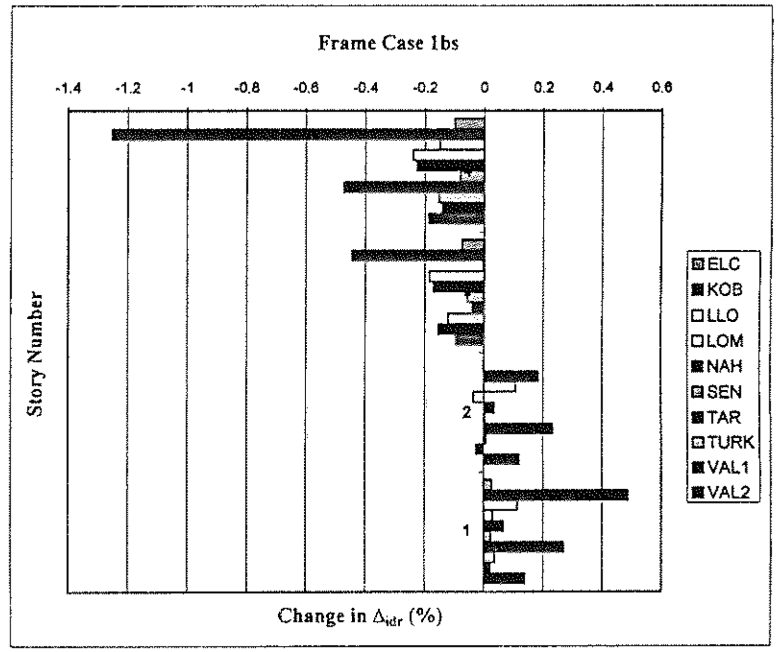
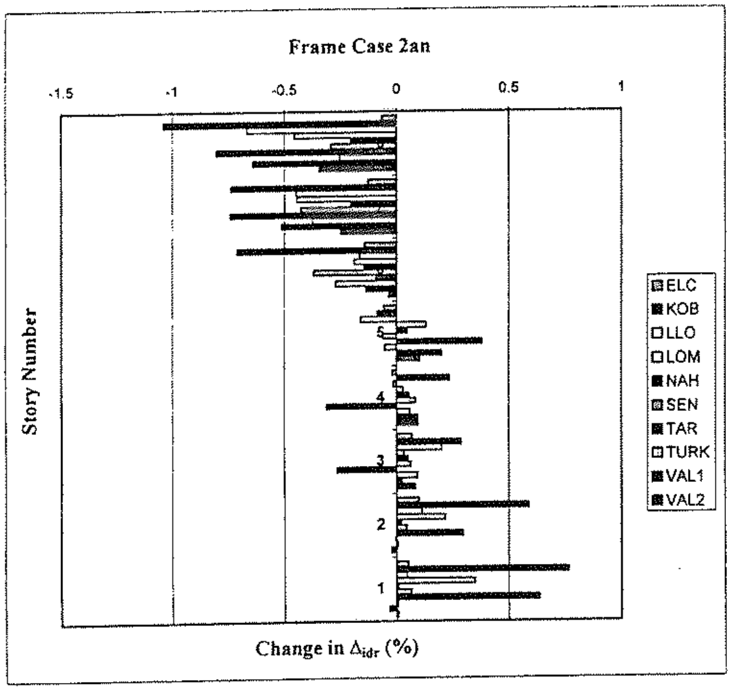


FIGURE 3.11 (b)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 1as & 1bs



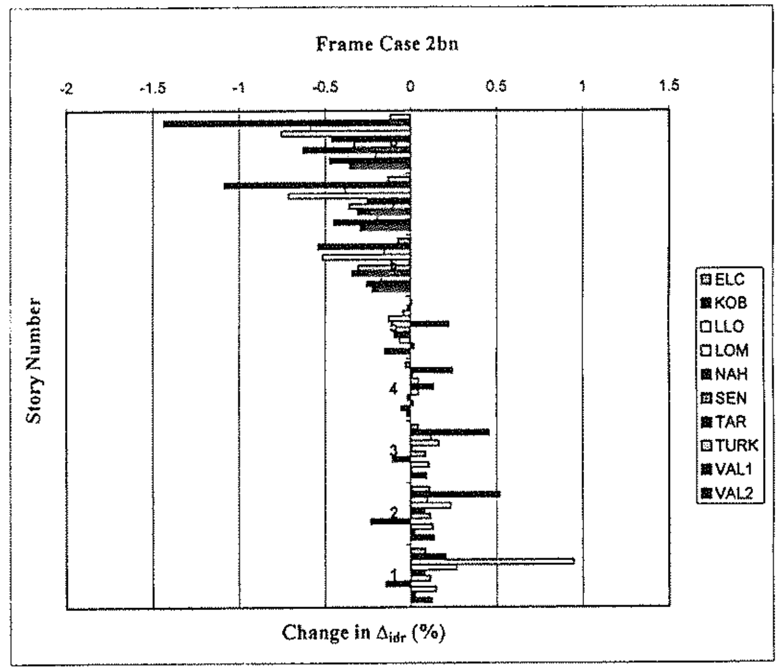
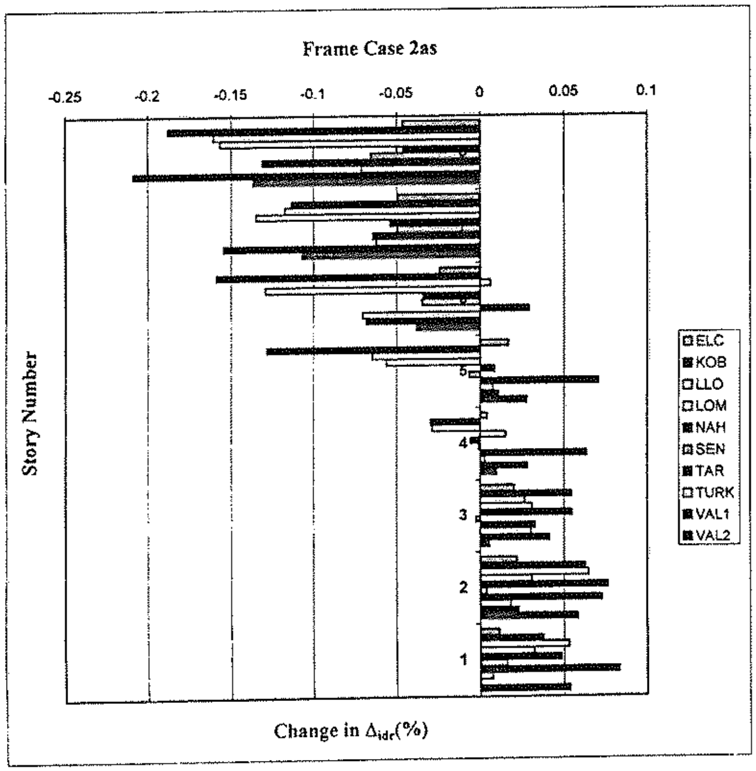


FIGURE 3.11 (c)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 2an & 2bn



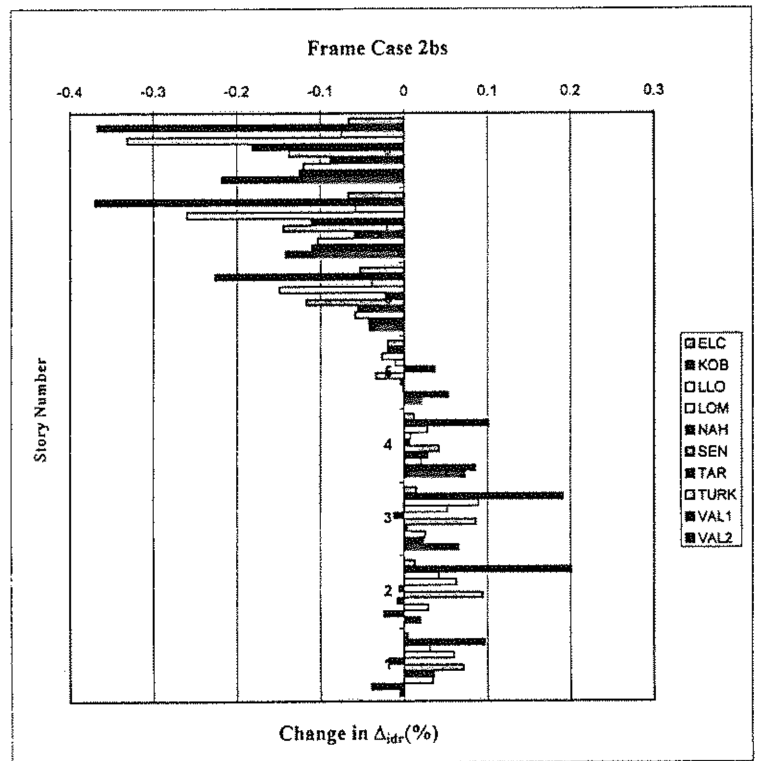
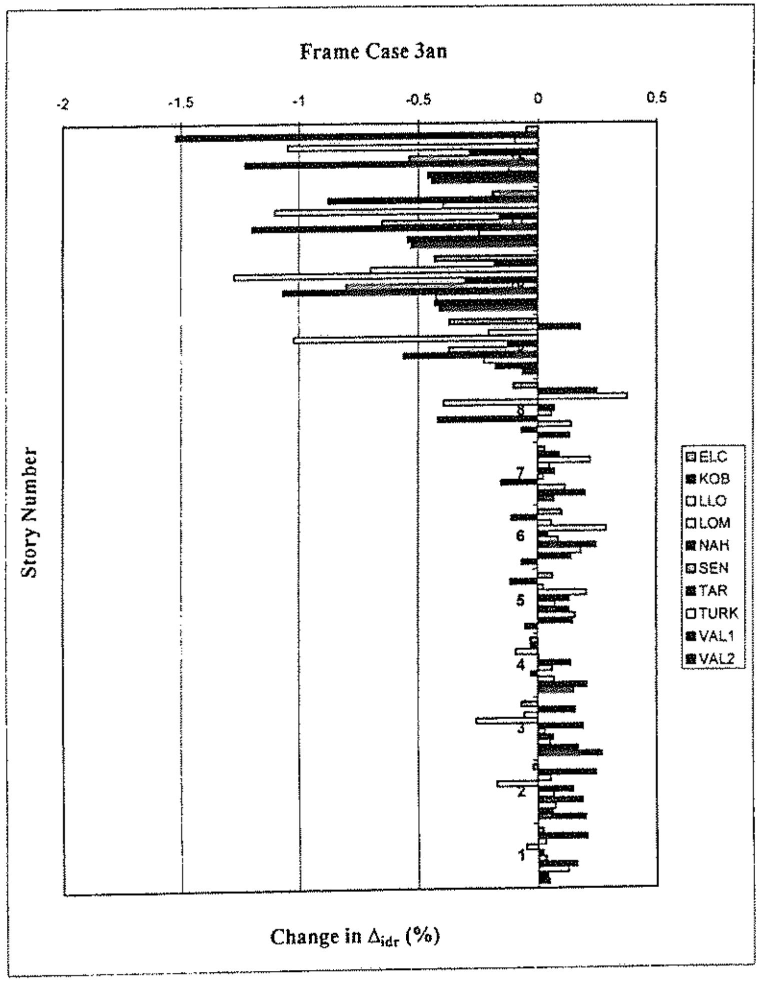


FIGURE 3.11 (d)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 2as & 2bs



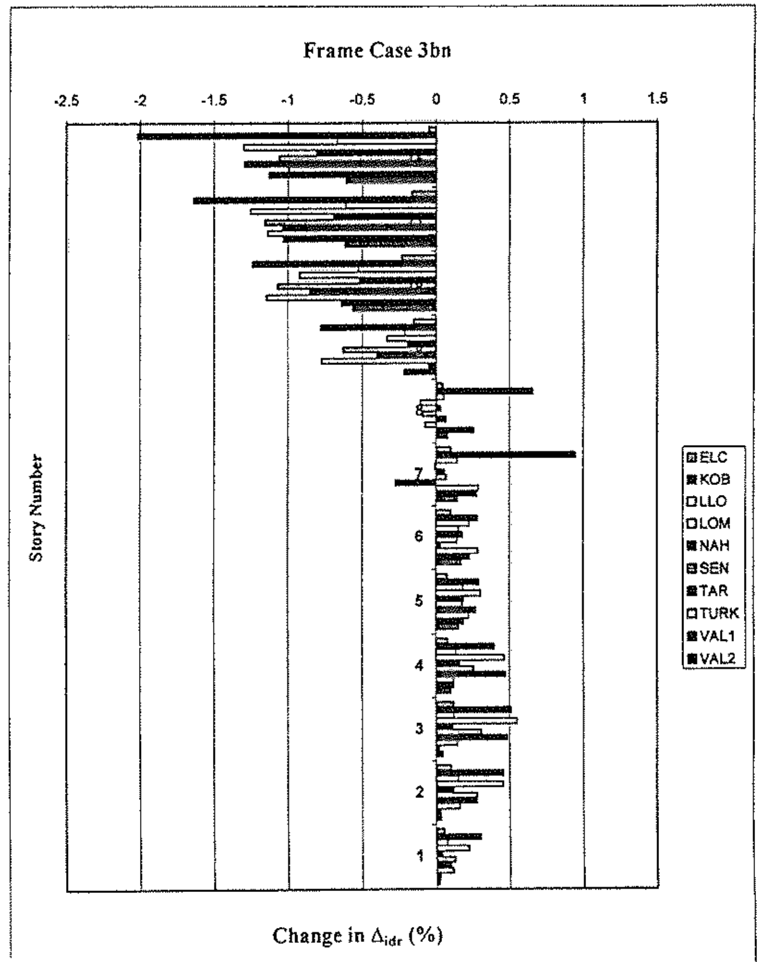
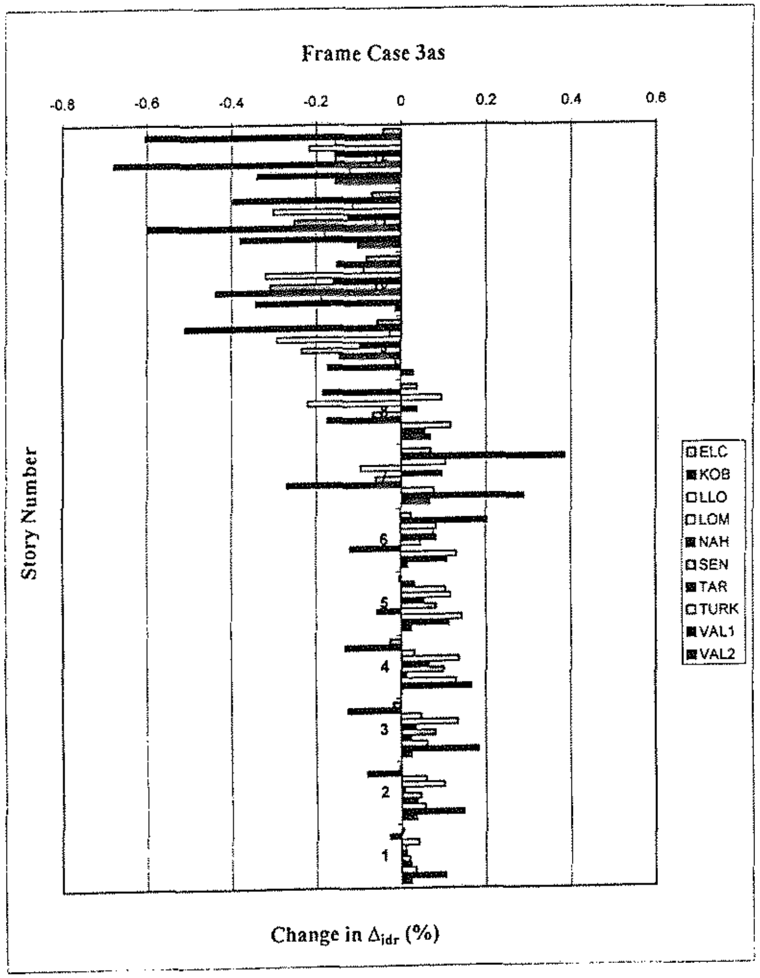


FIGURE 3.11 (e)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 3an & 3bn



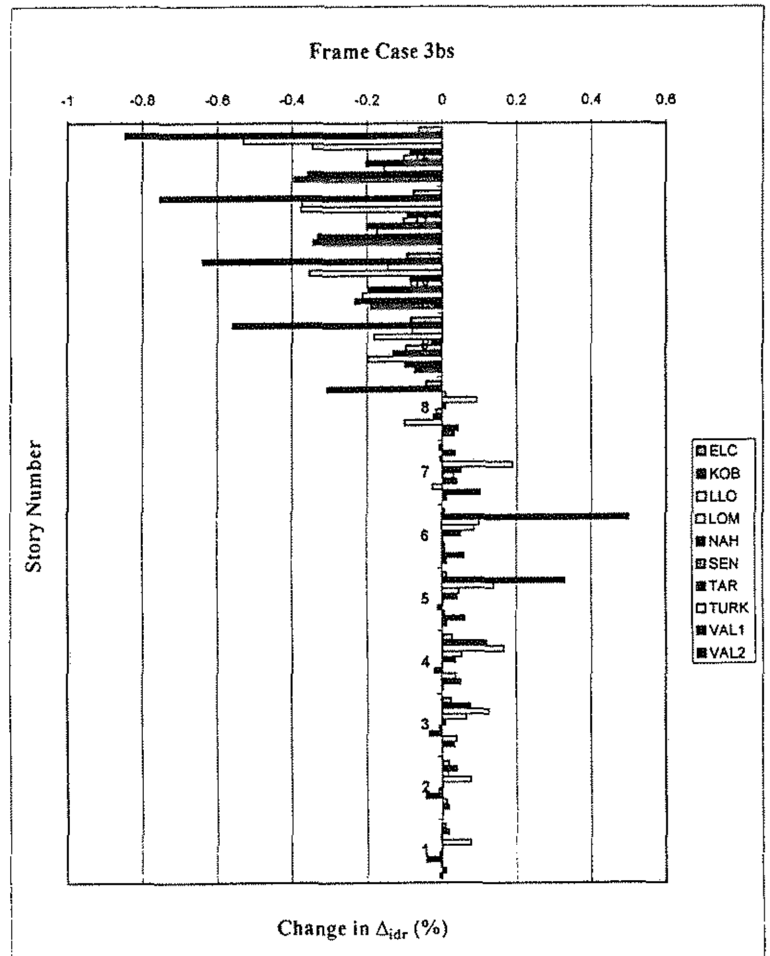


FIGURE 3.11 (f)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 3as & 3bs

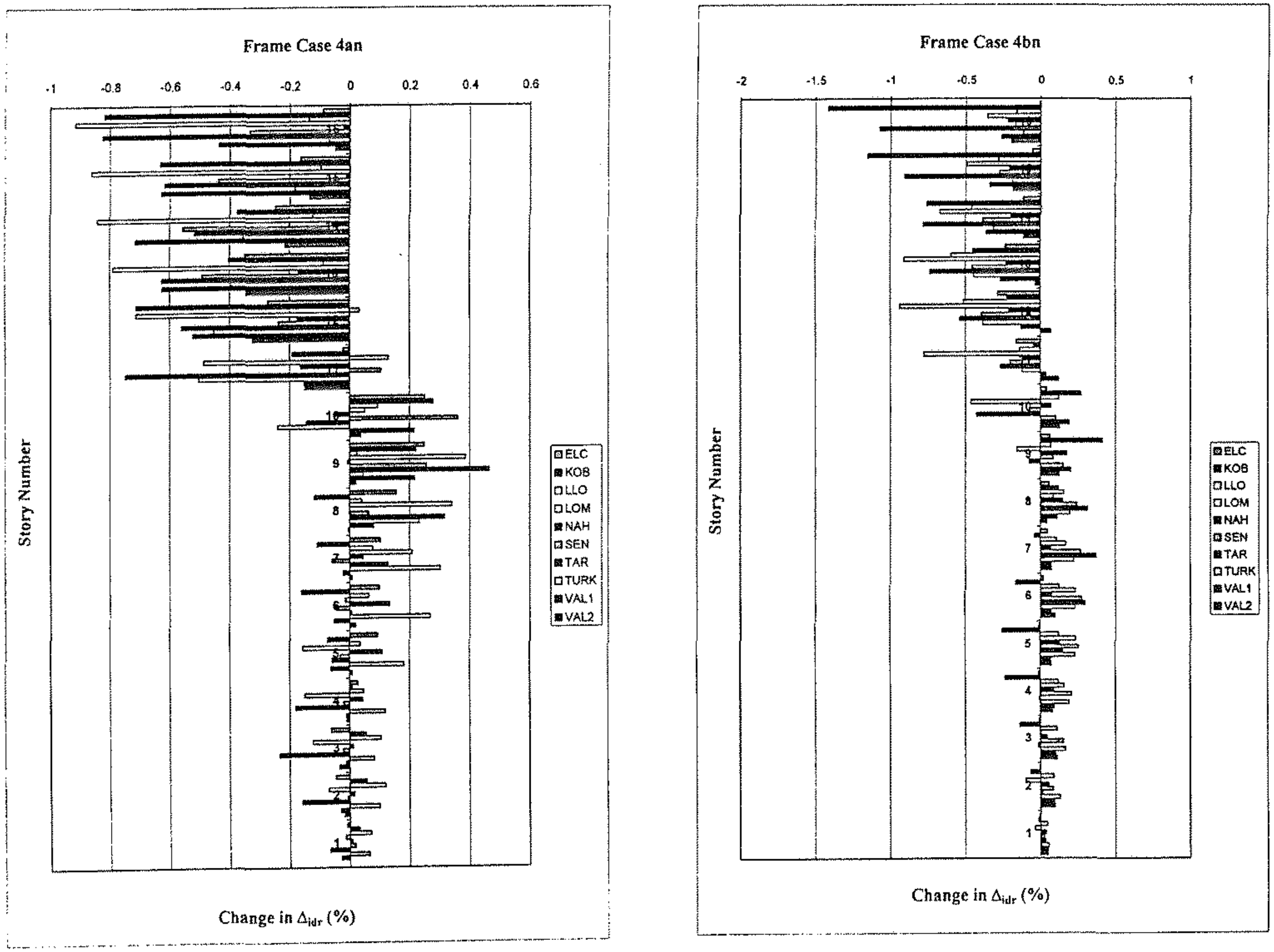
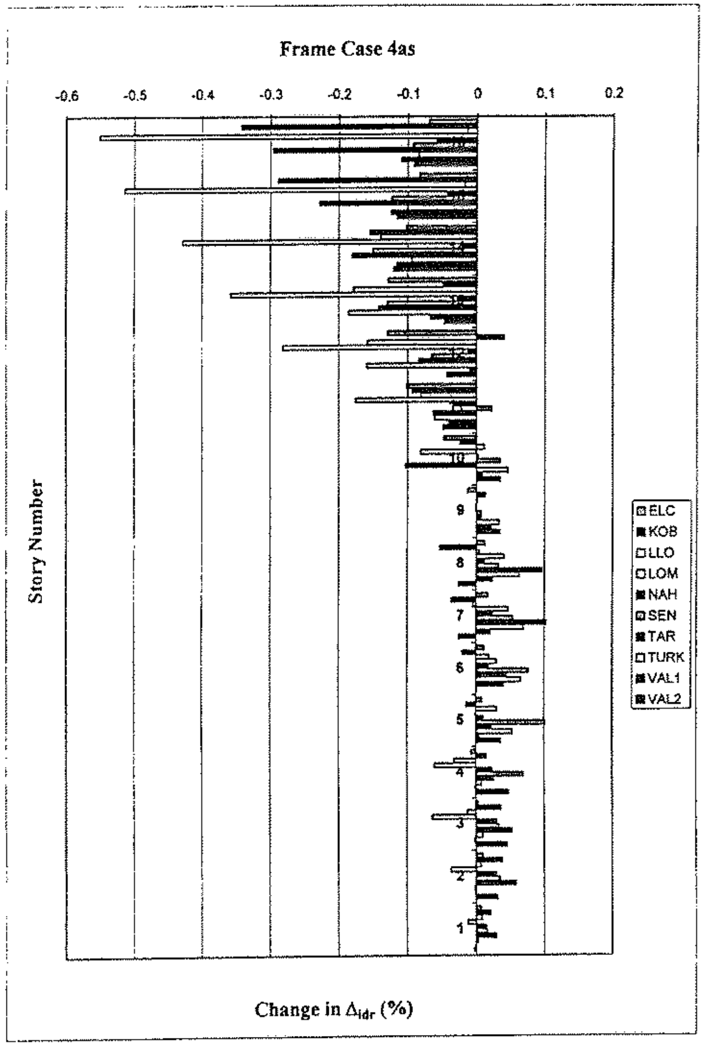


FIGURE 3.11 (g)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 4an & 4bn



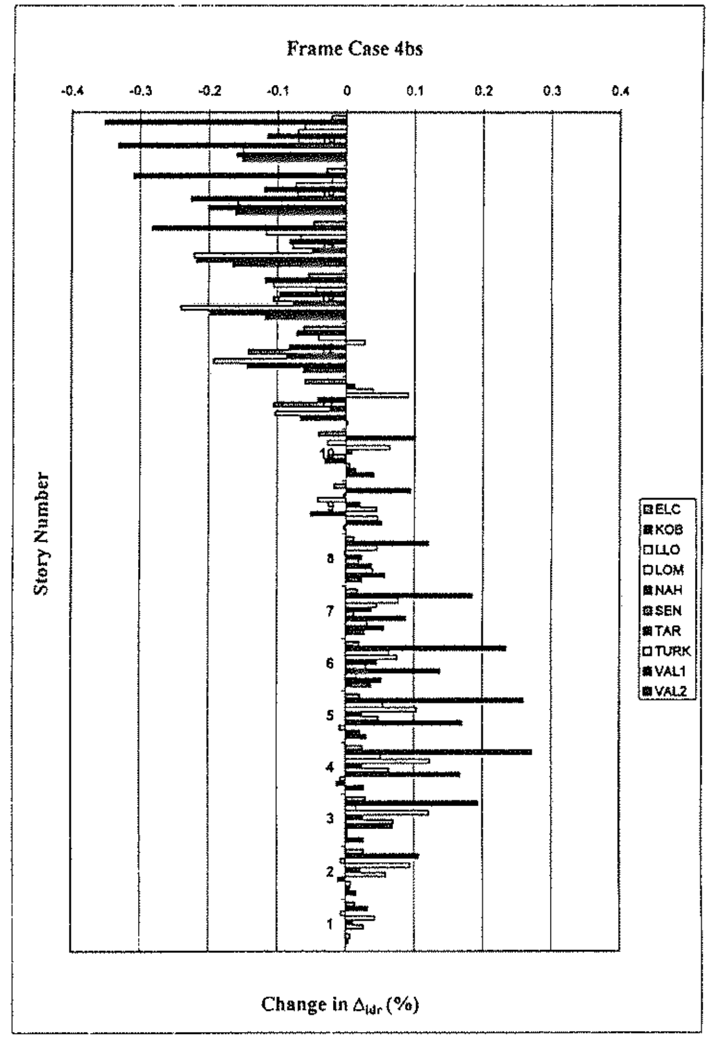


FIGURE 3.11 (h)
Ten Unscaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 4as & 4bs

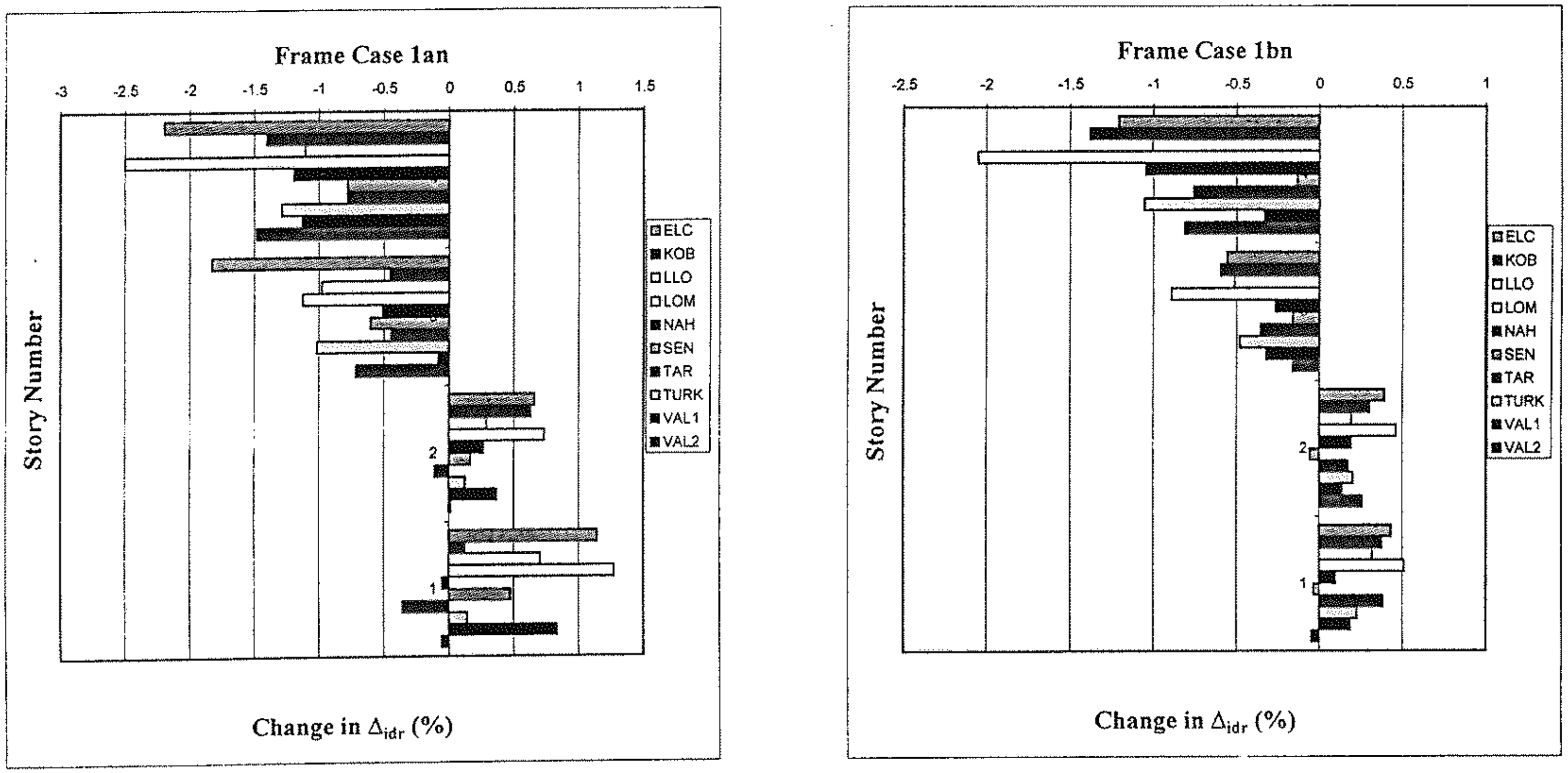
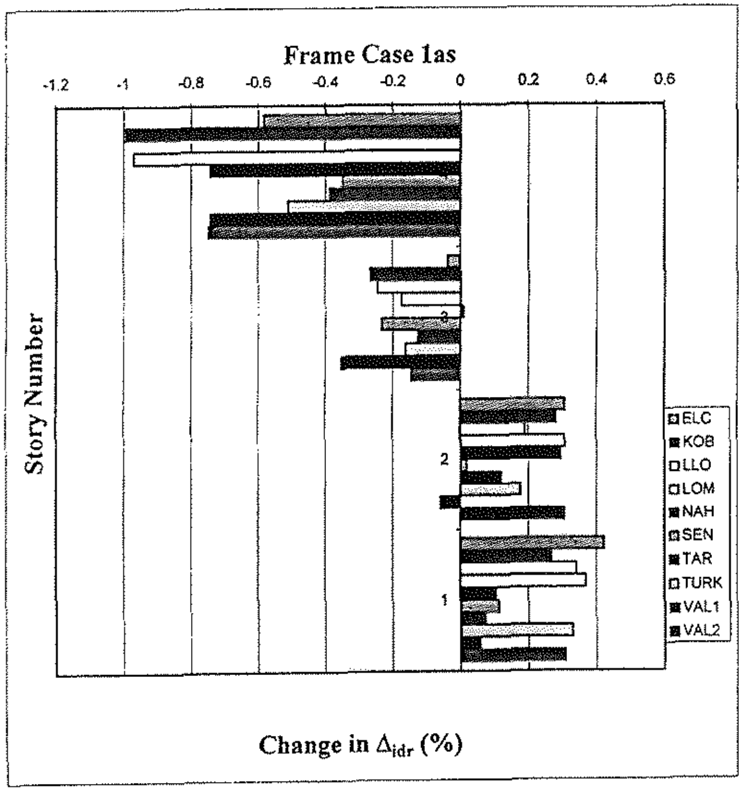


FIGURE 3.12 (a)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 1an & 1bn



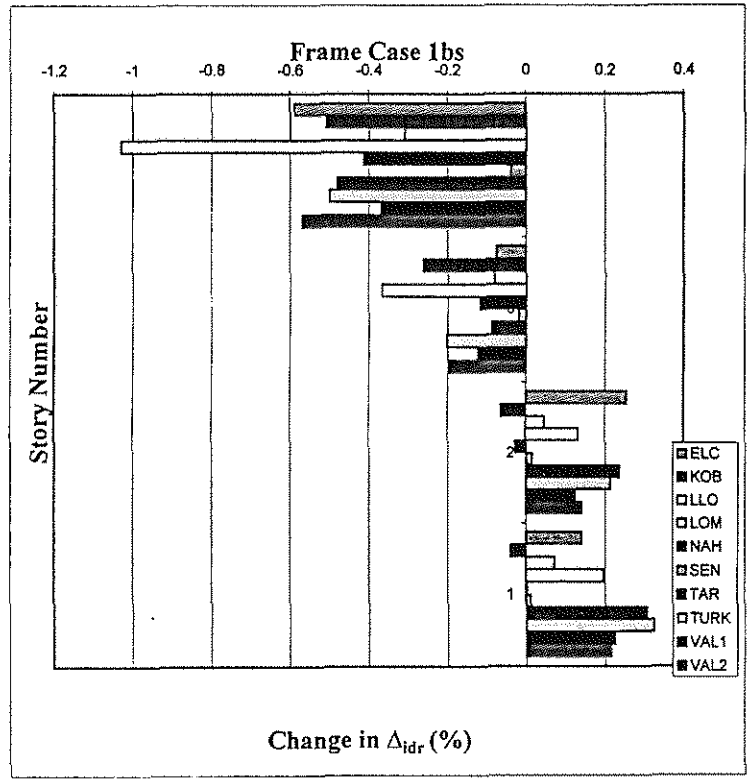
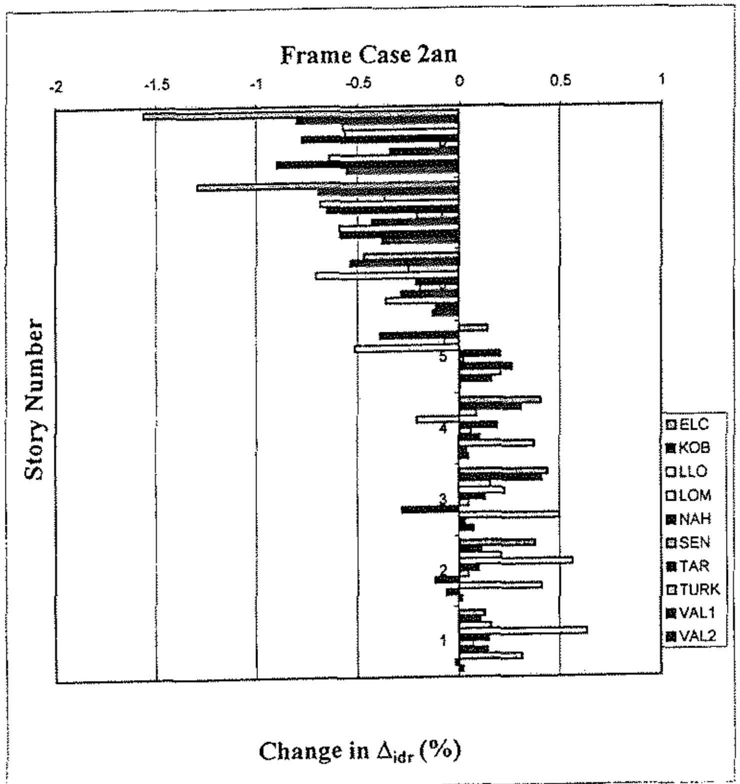


FIGURE 3.12 (b)

Ten Scaled Earthquakes

Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number

Frame Case 1as & 1bs



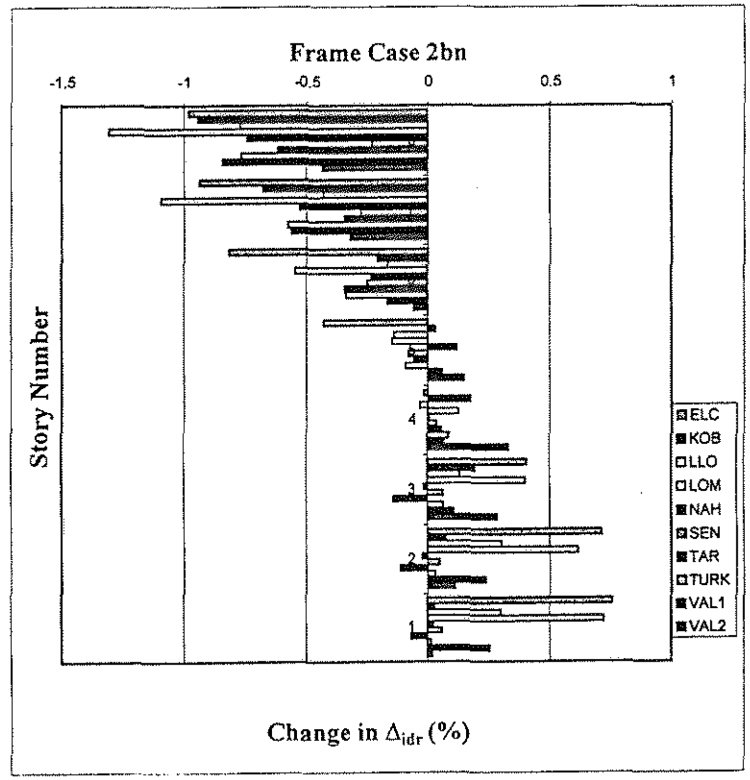
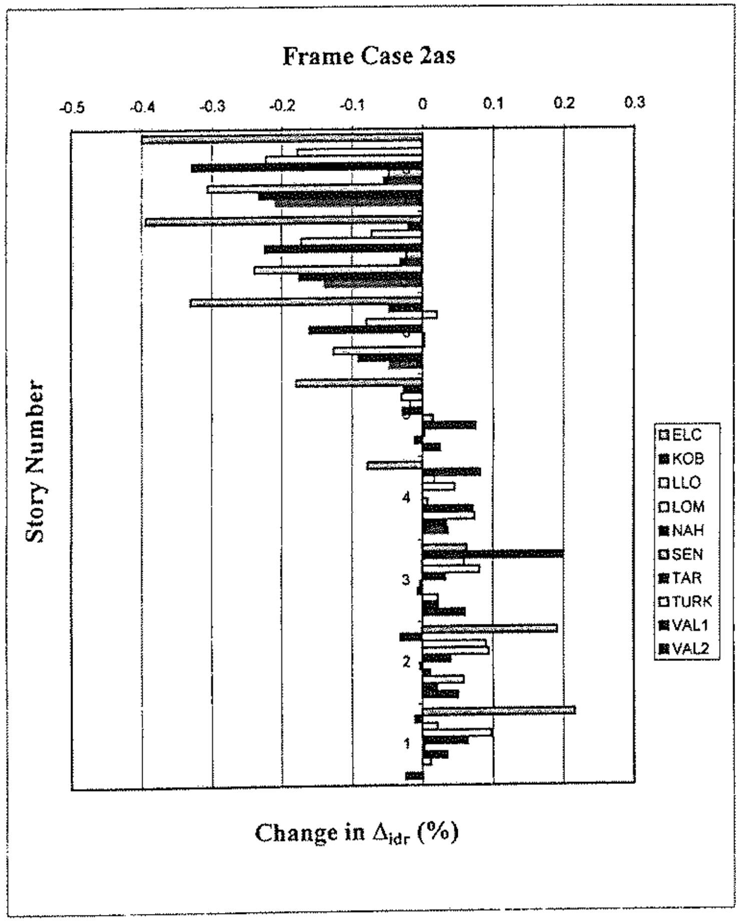


FIGURE 3.12 (c)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 2an & 2bn



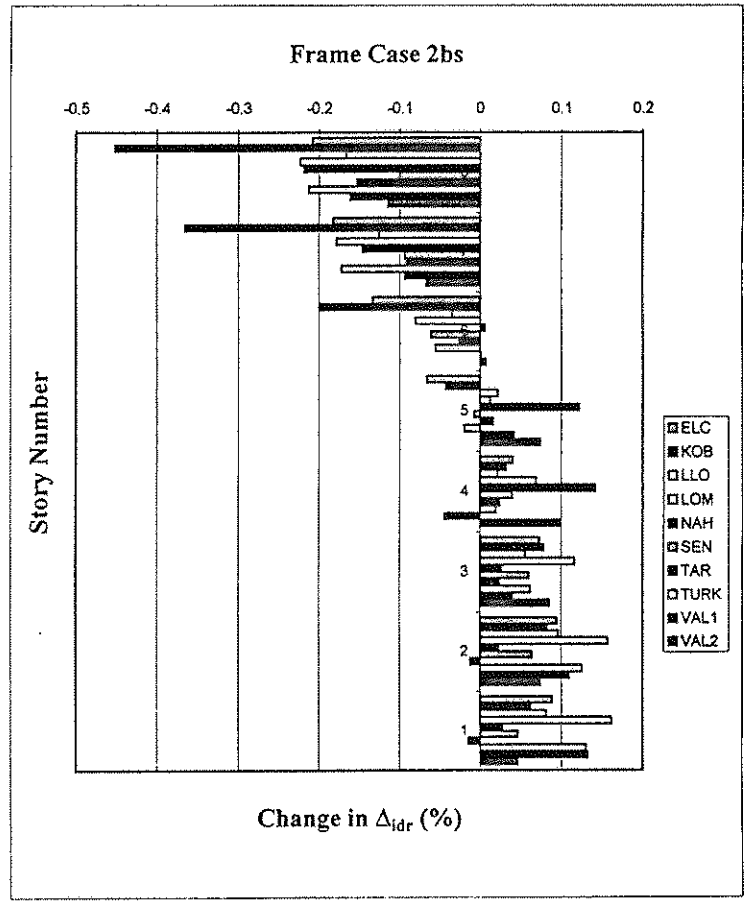
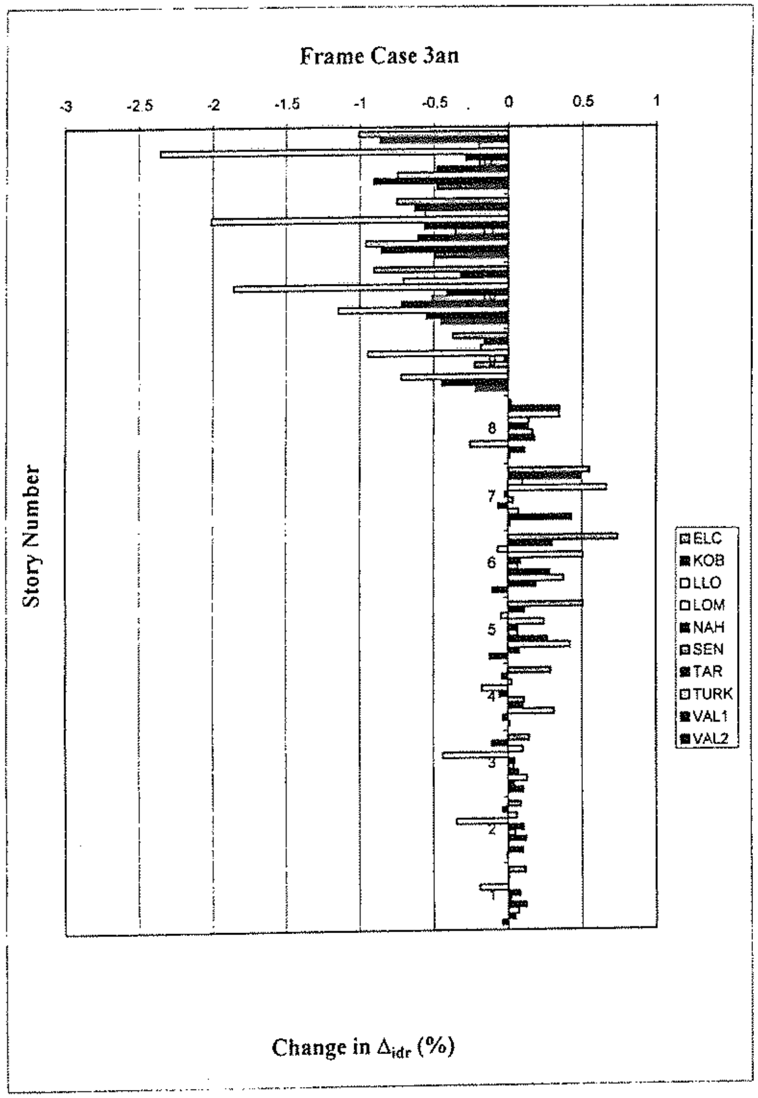


FIGURE 3.12 (d)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 2as & 2bs



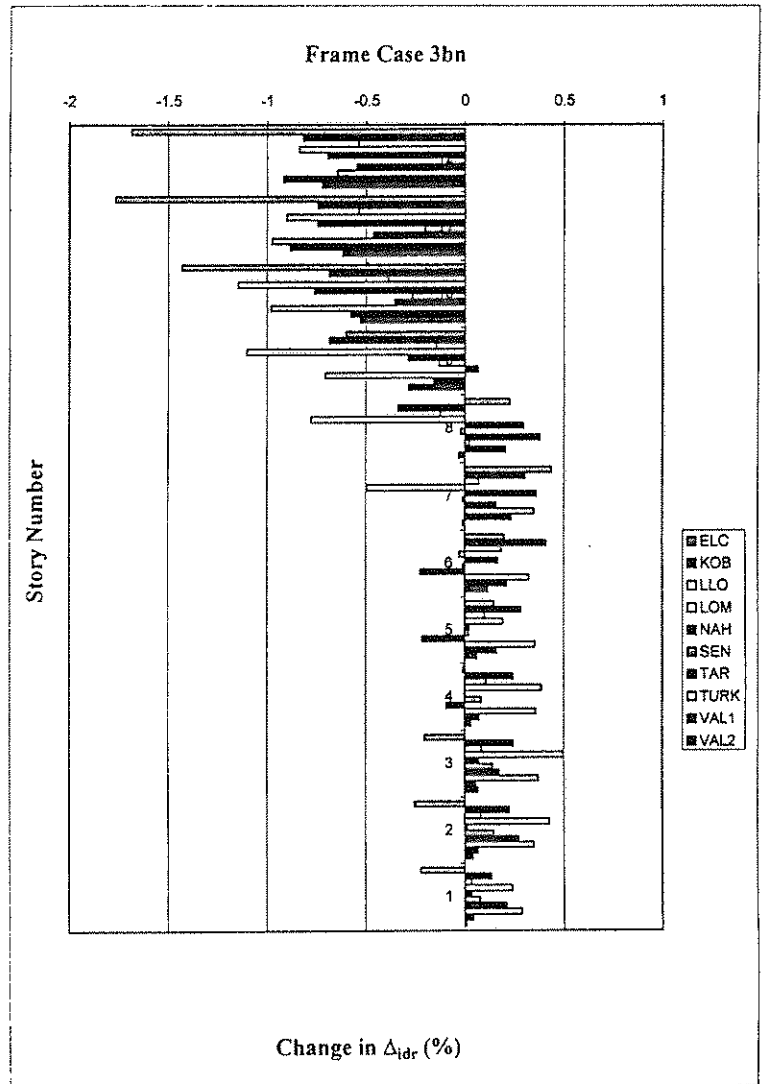
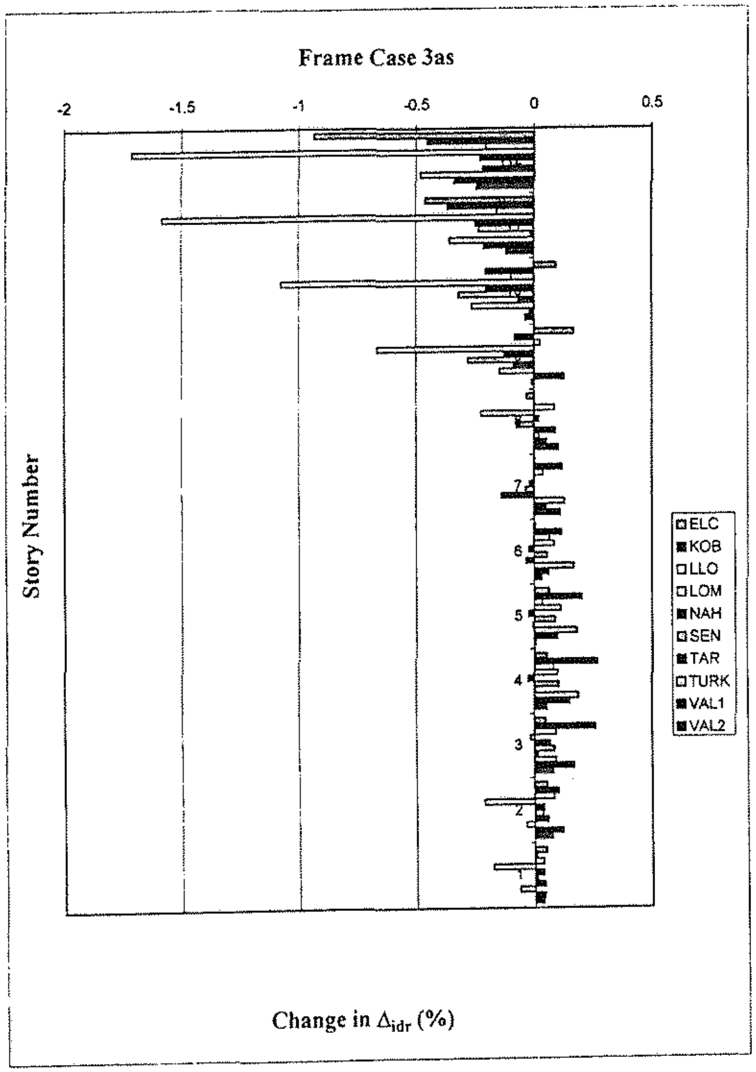


FIGURE 3.12 (e)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 3an & 3bn



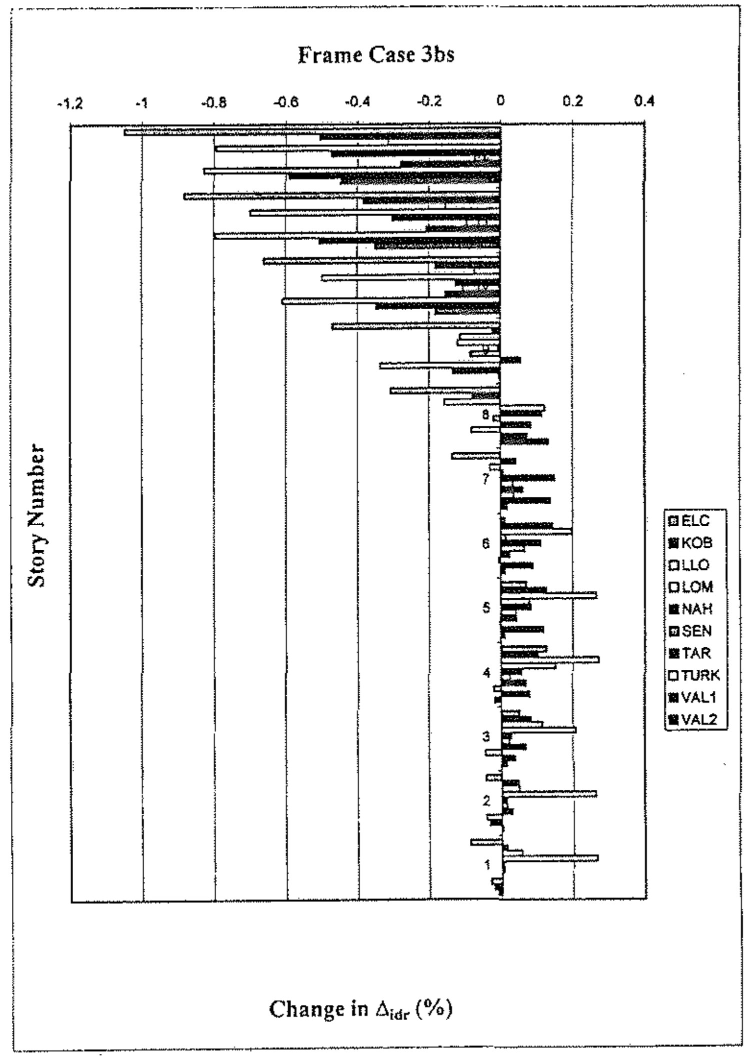


FIGURE 3.12 (f)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 3as & 3bs

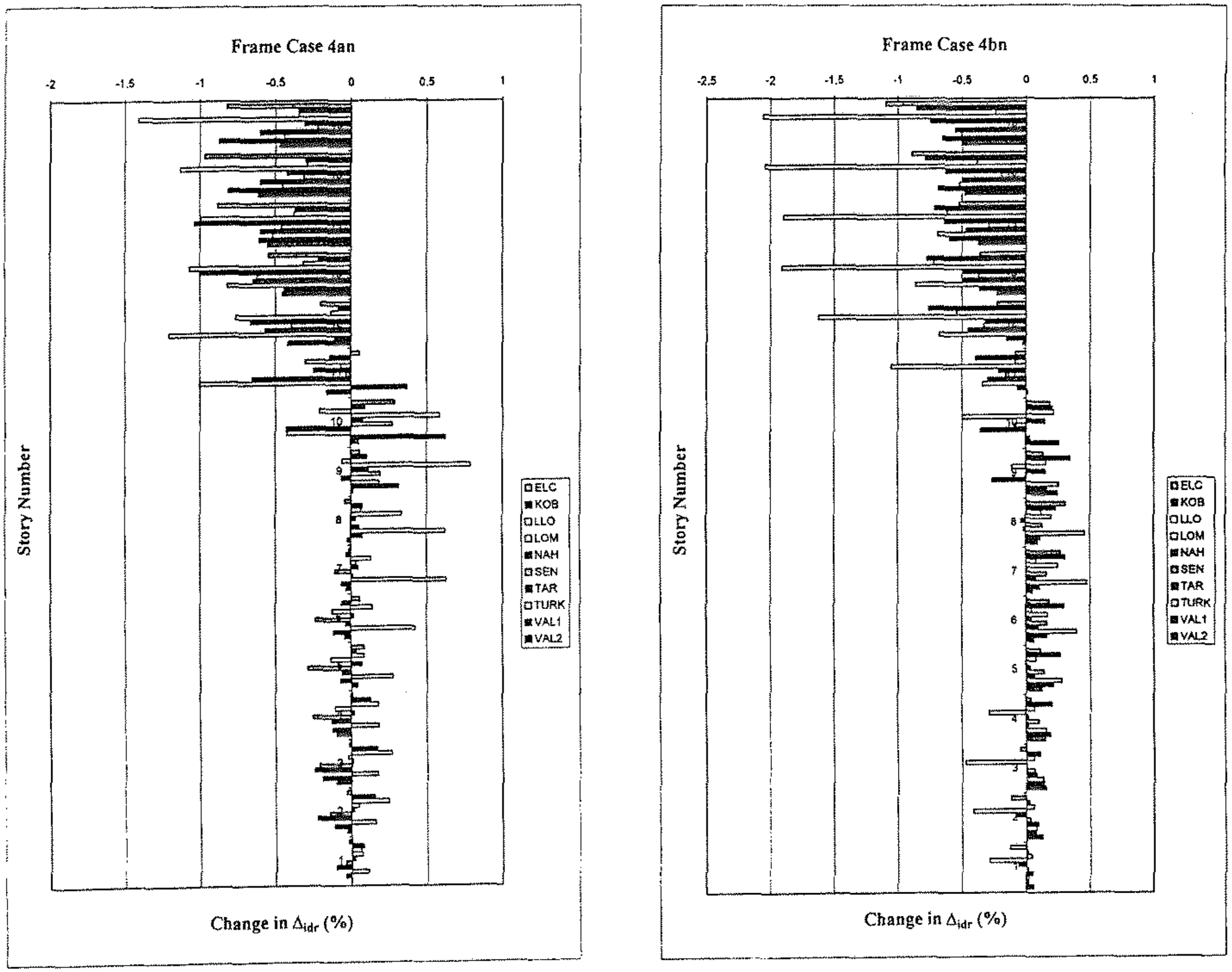
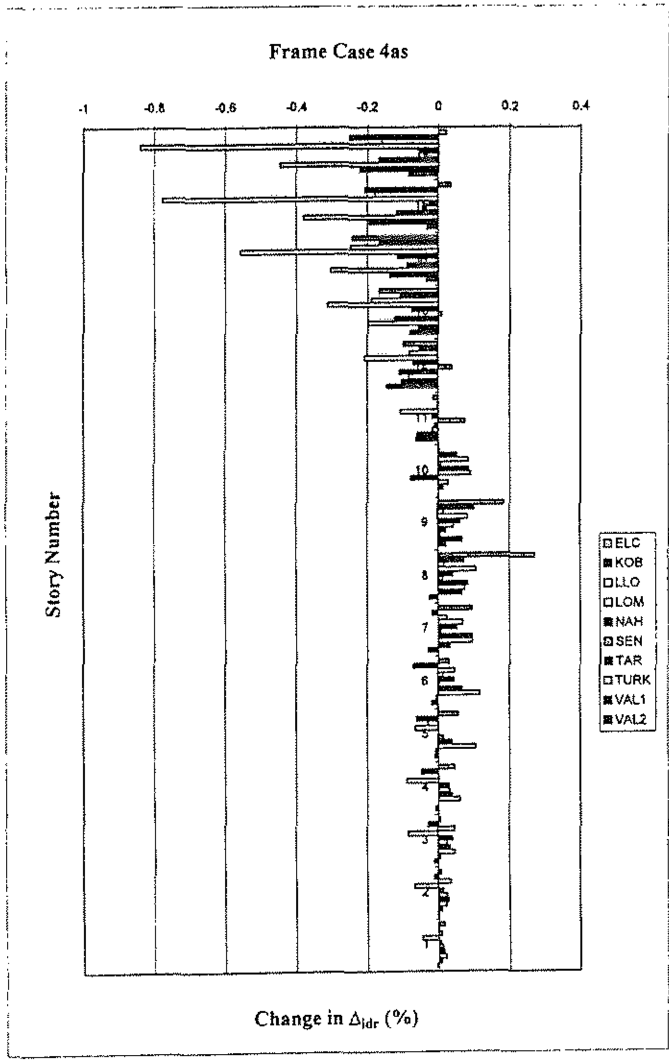


FIGURE 3.12 (g)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 4an & 4bn



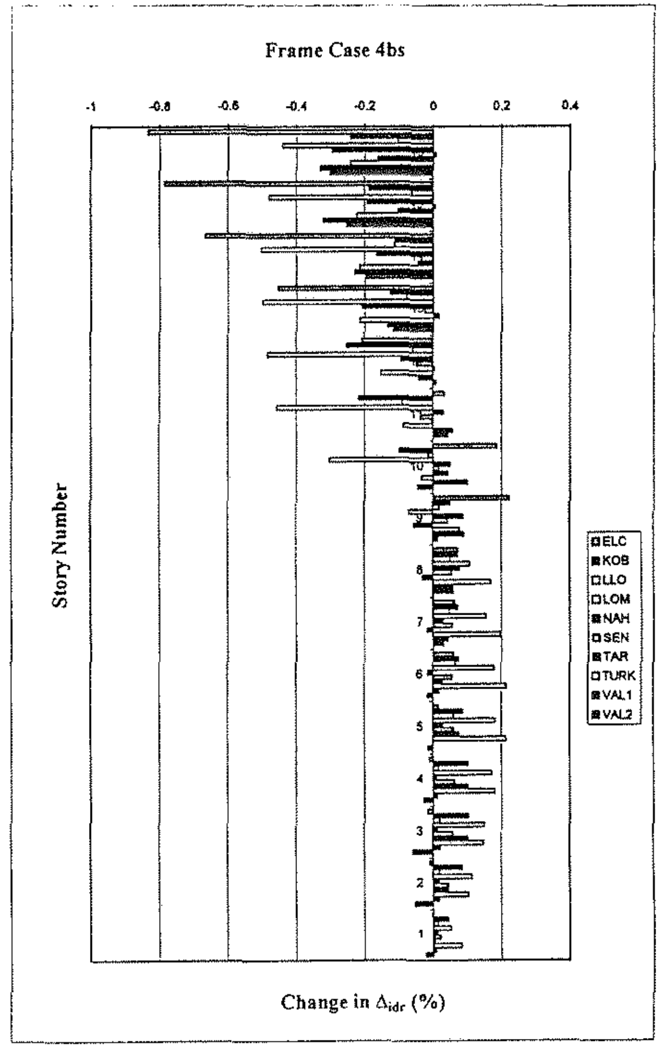


FIGURE 3.12 (h)
Ten Scaled Earthquakes
Change in Interstory Drift Ratio (Δ_{idr}) Vs. Story Number
Frame Case 4as & 4bs

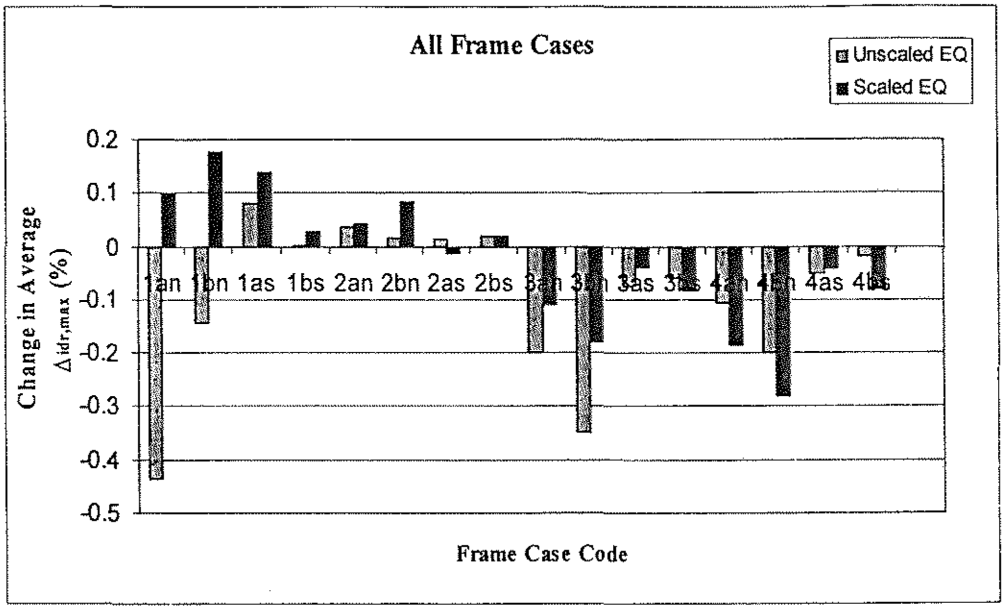


FIGURE 3.13
Change in Average Maximum Interstory Drift Ratio (%)
Dynamic Analysis

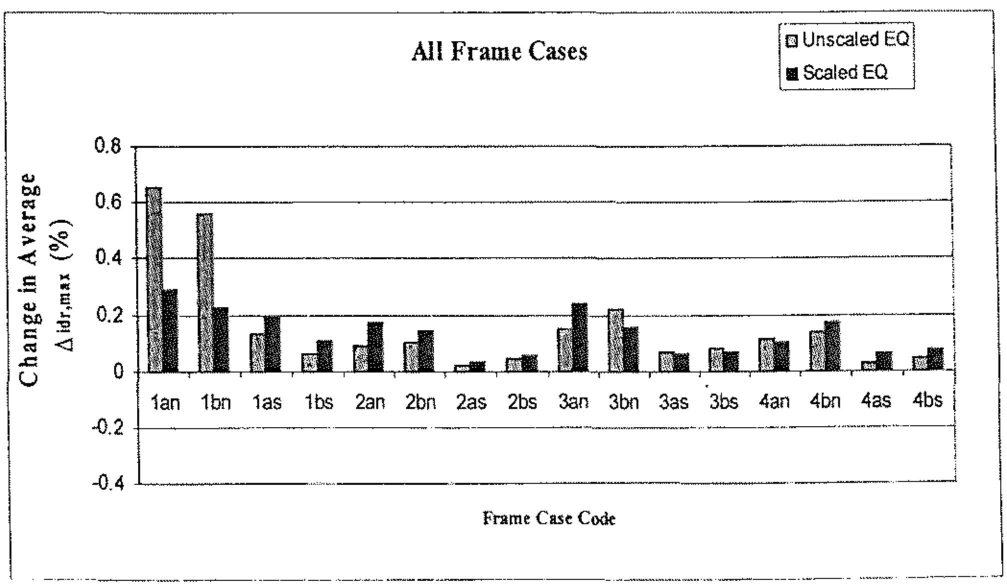


FIGURE 3.14
Change in Average Maximum Interstory Drift Ratio (%)
Stories in the Bottom Half of the Frames
Dynamic Analysis

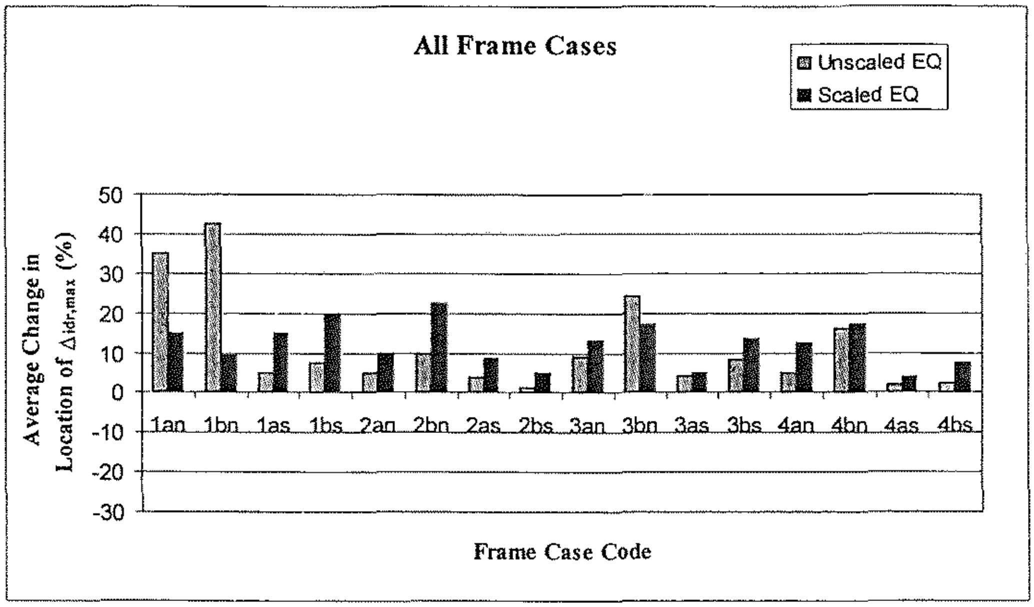


FIGURE 3.15
Average Change in Location of Maximum Interstory Drift Ratio (%)
Dynamic Analysis

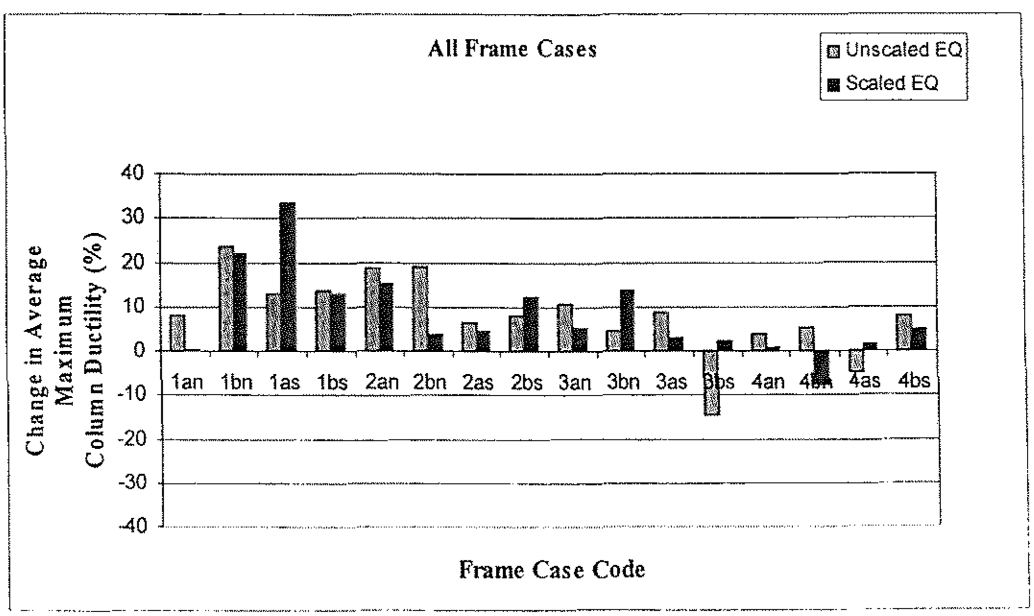


FIGURE 3.16
Change in Average Maximum Column Ductility (% Difference)
Dynamic Analysis

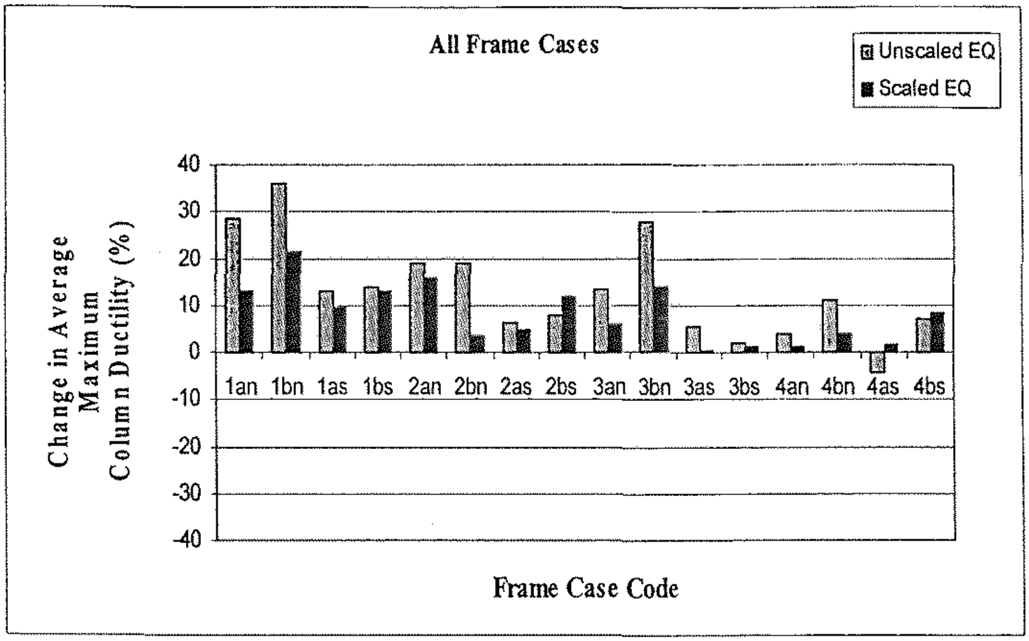


FIGURE 3.17
Change in Average Maximum Column Ductility (% Difference)
Stories in the Bottom Half of the Frames
Dynamic Analysis

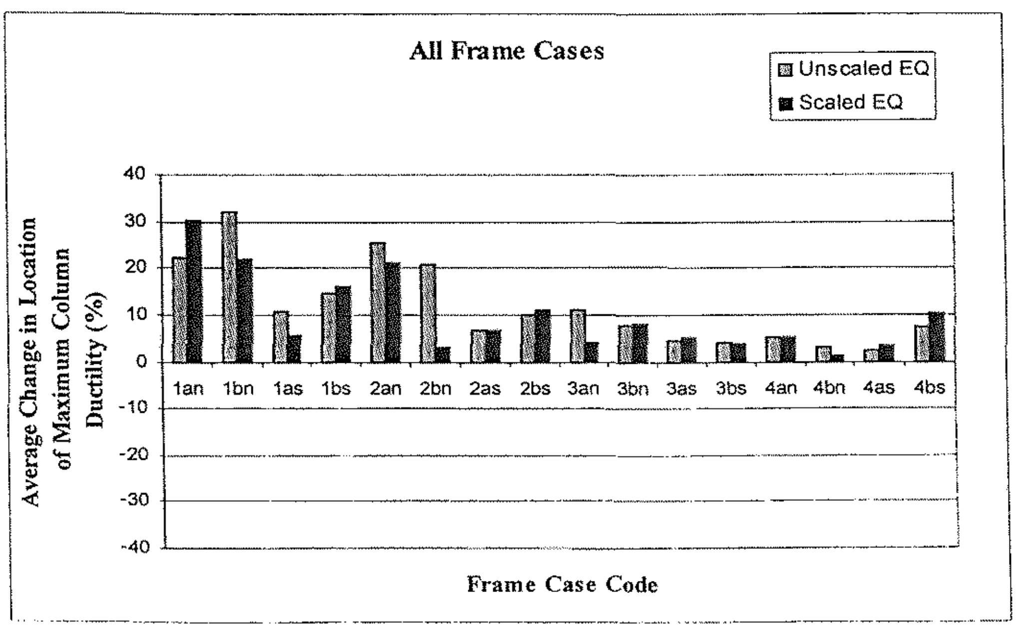


FIGURE 3.18
Average Change in Location of Maximum Column Ductility (%)
Dynamic Analysis

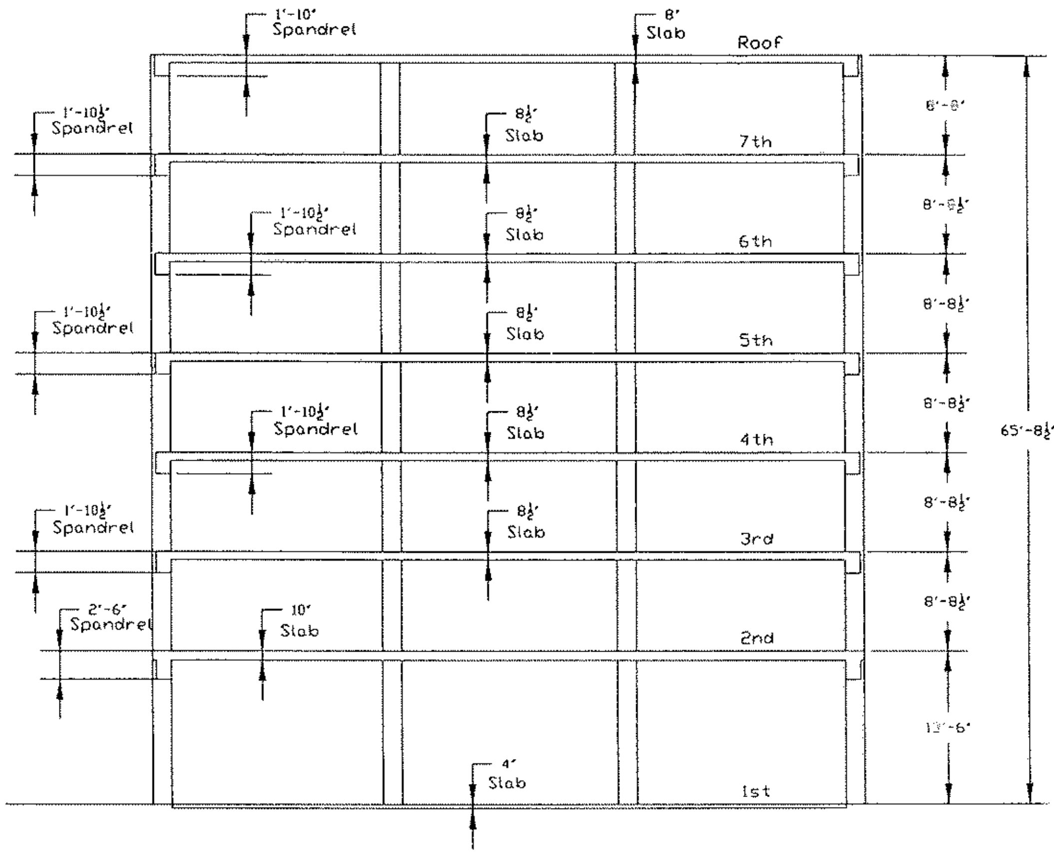


FIGURE 4.1
Typical Transverse Section
Holiday Inn

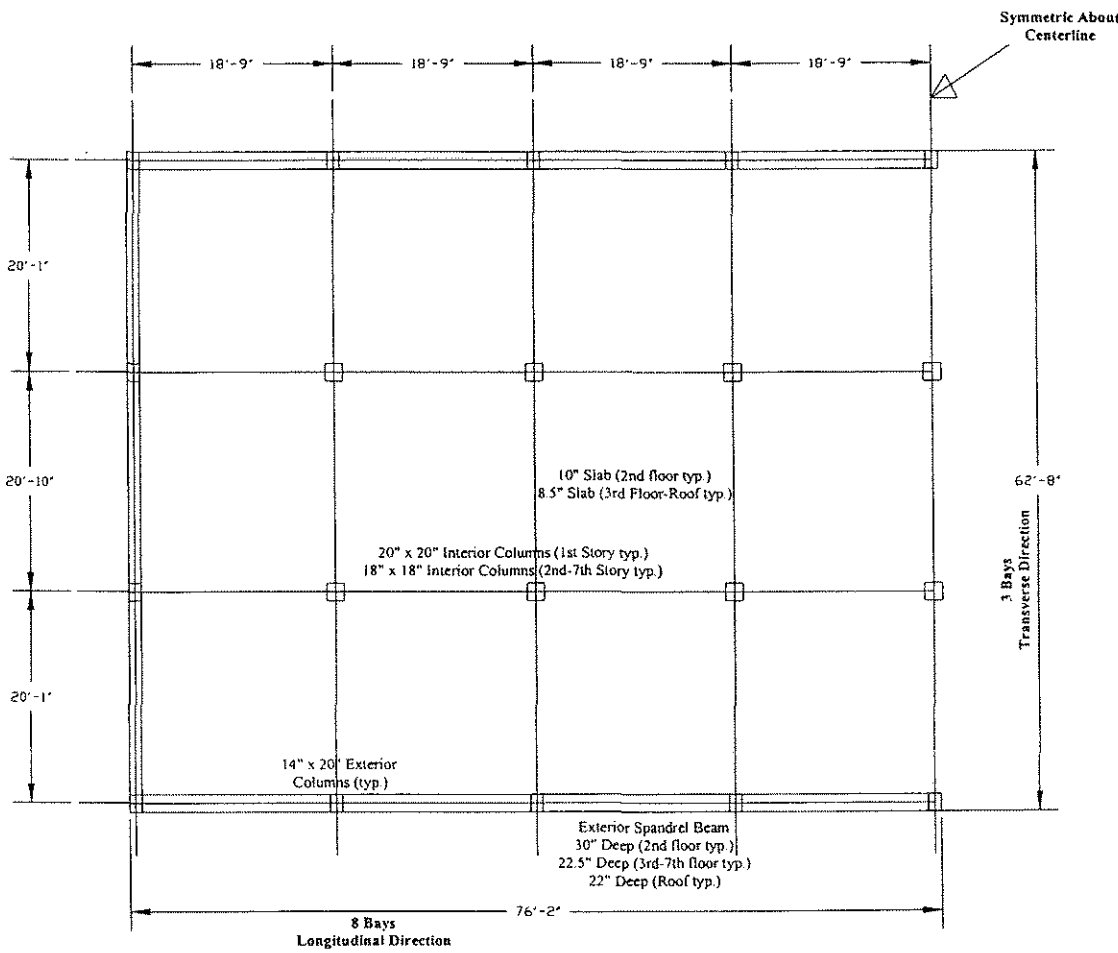


FIGURE 4.2
Typical Floor Framing Plan
Holiday Inn

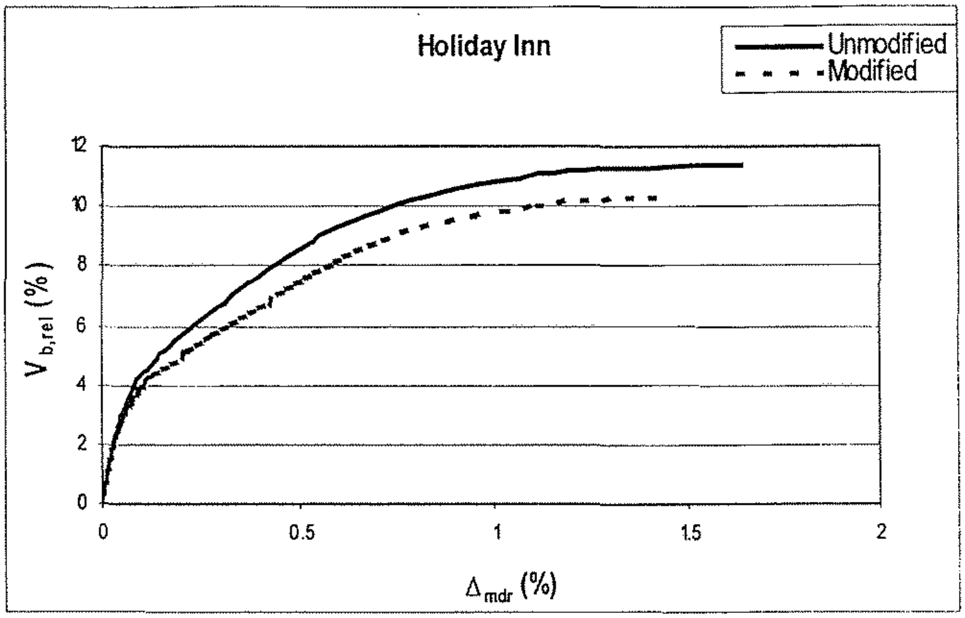


FIGURE 4.3
Relative Base Shear Vs. Mean Drift Ratio
Holiday Inn Static Analysis

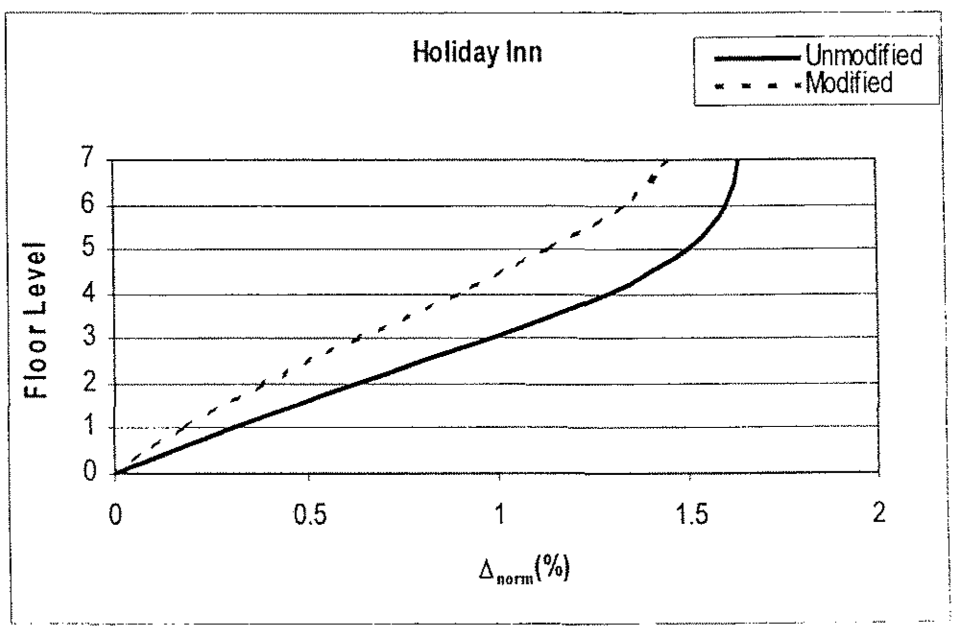


FIGURE 4.4
Building Displacement Profile
Normalized Drift Vs. Floor Level
Holiday Inn Static Analysis

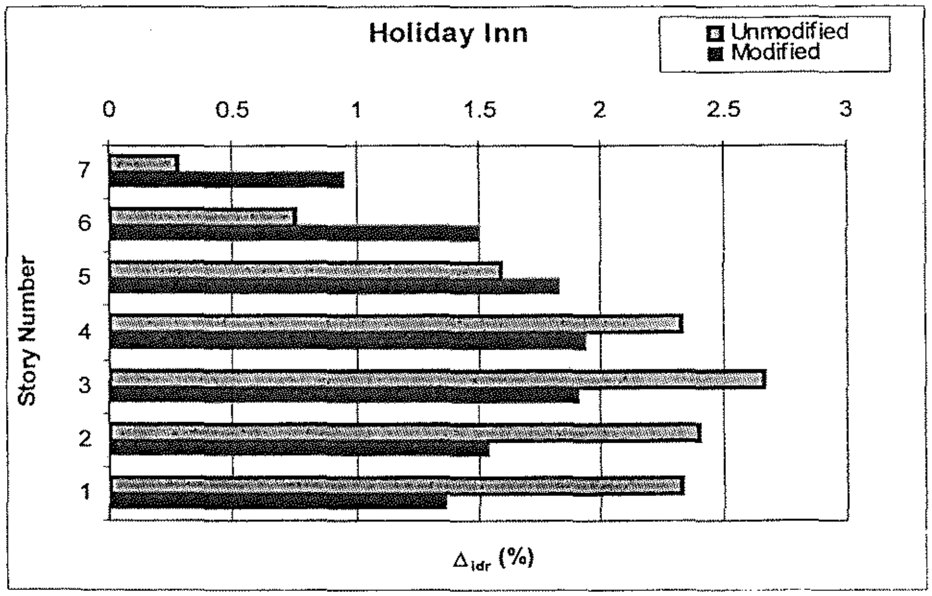


FIGURE 4.5
Interstory Drift Ratio Vs. Story Number
Holiday Inn Static Analysis

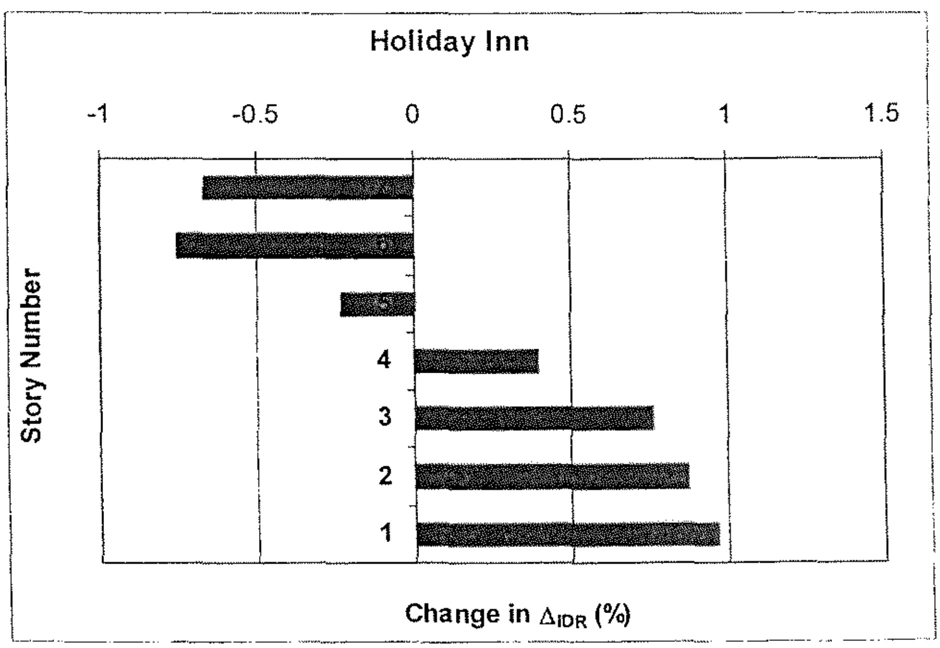


FIGURE 4.6
Change in Interstory Drift Ratio Vs. Story Number
Holiday Inn Static Analysis

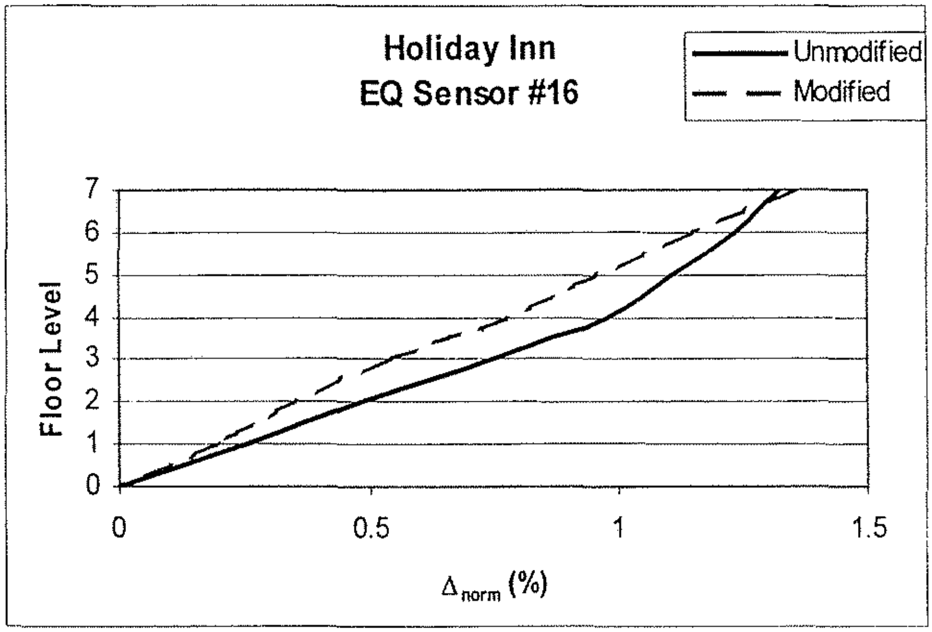


FIGURE 4.7
Maximum Building Displacement Profile
Normalized Drift Vs. Floor Level
Holiday Inn Dynamic Analysis

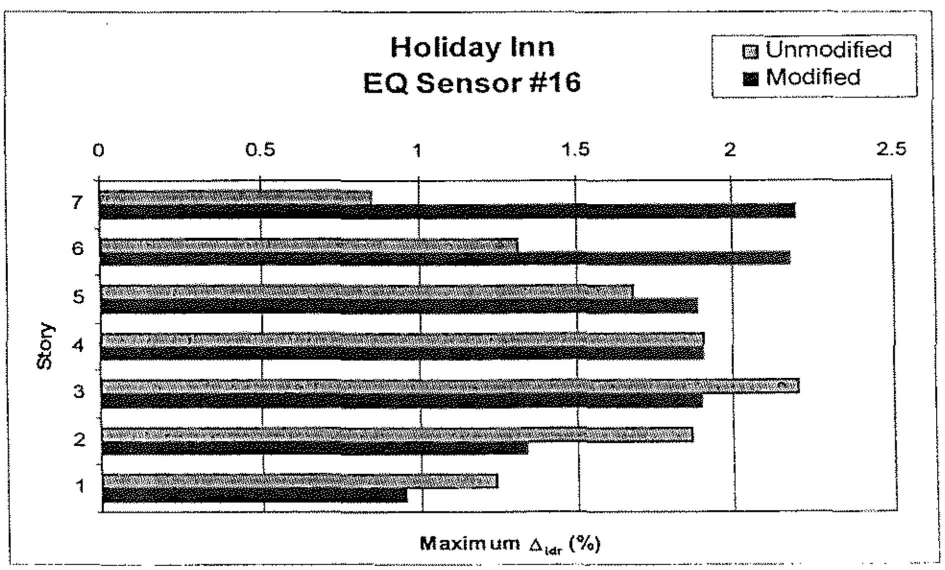


FIGURE 4.8
Maximum Interstory Drift Ratio Vs. Story Number
Holiday Inn Dynamic Analysis

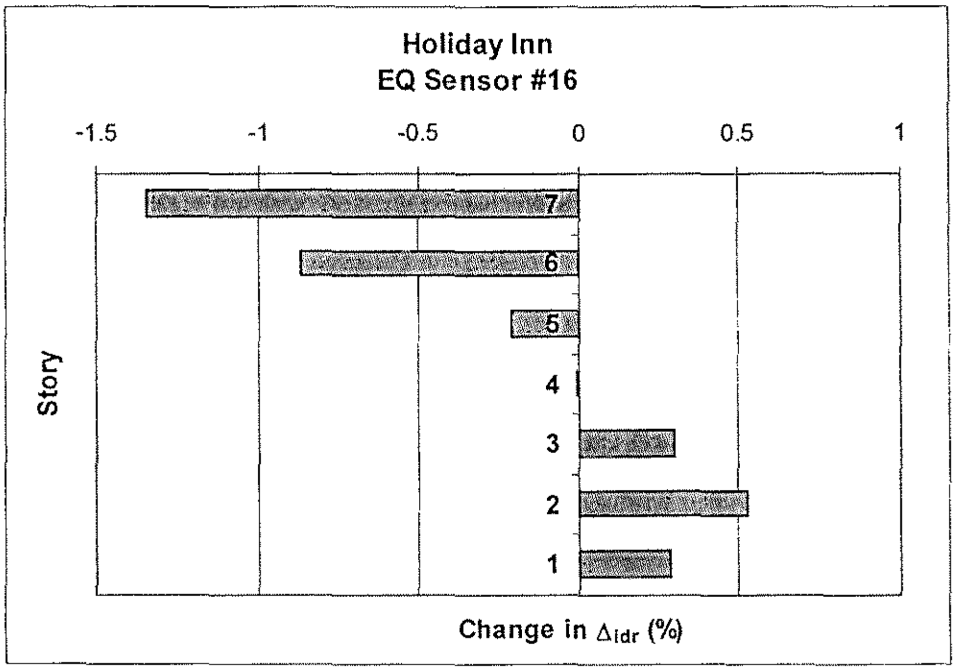


FIGURE 4.9
Change in Interstory Drift Ratio Vs. Story Number
Holiday Inn Dynamic Analysis