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RESPONSE OF REINFORCED CONCRETE BUILDING SUBJECTED TO NORTHRIDGE EARTHQUAKE

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SUMMARY

This study is developed in two parts. The first part deals with modeling a 13-story frame with 2 sublevels subjected to moderate levels of ground motion. The second part focuses on applying a simple strength-based relationship to modify the response of the 13-story reinforced concrete structure and the restrictions that were used to apply the method. Results for different models and earthquake demands were analyzed and discussed. In general, it was shown that the strength reduction factor helps to improve the inter-story drift ratios for a structure subjected to several earthquake demands. The structure with a stiff foundation benefited from a greater improvement in response when the strength reduction factor was used.

INTRODUCTION

The detailing of elements of reinforced concrete frames for seismic regions requires that the anticipated maximum response (displacement) of the elements be estimated. The yielded shape of the structure determined from nonlinear analysis provides a useful tool for estimating the locations of maximum response in the building. A tool (a strength relationship) was previously developed to modify the locations of maximum response based on a study using hypothetical reinforced concrete frames and the response of a 7-story instrumented building (Kuntz [1]). Using this tool, the yielded shape of the building is modified so that the maximum response in the elements occurs at higher locations where the axial stress is less harmful to the overall stability of the structure and the severity of damage to the elements is reduced. This paper demonstrates the applicability of the method using a moderate-rise frame.

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DESCRIPTION OF THE TARGET STRUCTURE

Location

The target structure is a commercial building located in Sherman Oaks, California that was designed in 1964 and built in 1965. The structure was located approximately 9 km (5.59 mi) southeast of the epicenter of the 1994 Northridge earthquake.

Structural Configuration

Building Layout

The building is a reinforced concrete frame structure with thirteen stories above ground and two stories below. It is currently an office building. The first sublevel (below the first floor) is 3.5 m (11.5 ft) in height while the second sublevel is 2.7 m (9 ft). The first floor is 7 m (23 ft) in height and all the other floors above the ground are 3.6 m (11.75 ft). In the N-S direction there are eight frames and the E-W direction consists of three frames. The N-S frames have two bays spanning 11 m (36 ft) each, and the E-W frames have seven bays spanning 8.2 m (27 ft) each. Each floor has a one-way concrete slab with a thickness of 114 mm (4.5 in). On the roof there is a penthouse for mechanical equipment, which contributes mass but not stiffness to that floor. There is a parking facility contiguous to the building.

The N-S direction consists of concrete moment resisting frames from the ground floor through the roof floor, and concrete shear walls in the sublevels. The structural configuration is similar for the E-W direction.

Girders and Columns

The sizes of the girders vary throughout the height of the building and between frames. The beam sizes (width and height) for the exterior frame vary between 0.6 m square (24 in.) and 0.46 m by 1.4 m (18 in by 55 in). For the interior frames the range of beam sizes was between 0.76 m by 0.47 m (30 in by 18.5 in) and 0.6 m by 0.9 m (24 in by 36 in).

The exterior frames have two cross sections for the columns. The first cross section is 0.9 m square (36 in.) with a square notch of dimension 0.3 m (12 in.) in the exterior corner. These columns are located in the corners of the structure. The second cross section has dimensions of 0.6 m by 0.9 m (24 in by 36 in.). The interior frame has exterior columns of 0.9 m by 0.6 m (36 in by 24 in) and square interior columns with dimension 0.9 m (36 in.). Perimeter columns located on the north and south sides of the building had their strong axis in the north-south direction, whereas columns located on the east and west sides of the building had their strong axis in the east-west direction. The location of the strong axis for the columns in the interior frame varies.

The non-structural elements consist of a curtain wall used as exterior cladding, partitions that are gypsum board on studs, and suspended acoustical tile acts as ceiling. Major fixed equipment is located in the basement and on the roof. The elevators are located at the east end of the building.

Subsurface Conditions and Material Properties

The soil deposits supporting the structure are alluvial deposits from the San Fernando Valley and are classified as clays, silt and sand (Naeim, [2]; Ventura [3]). Shear walls surround the two sublevels and are offset from the frame by 2.4 m (8 ft). The shear walls are connected to the sublevels by a 15.2 cm (6 in) slab. The foundation consists of concrete pile caps supporting each column line. Each pile cap groups between 12 and 23 piles. Most of the pile caps have an area of 27 m^2 (285 ft^2) and a height of 91 cm (3 ft). The length of the piles varies between 6 and 7.6 m (20 and 25 ft), depending upon the diameter of the pile. The diameter of the piles varies between 38 and 61 cm (15 and 24 inches).

The concrete used for columns and walls throughout the structure as well as floor elements below the second floor was regular weight. All other floor elements were constructed with lightweight concrete. The design strength for all columns was 34.5 MPa (5000 psi), and all girders had a design concrete strength of 25.8 MPa (3750 psi). Pile caps and piles had a concrete strength of 20.7 MPa (3000 psi).

Sensors

The structure was instrumented with 10 accelerometers distributed in 5 floors: two in the second sublevel, two in the first floor, two in the second floor, two in the eighth floor and two in the roof. The accelerometers located at the west end of the structure were uniaxial and recorded accelerations on the north-south direction. The accelerometers located at the center of the building were bi-axial and recorded accelerations in both the east-west and north-south directions. Only one of the accelerometers recorded vertical accelerations and it was located at the second sublevel.

Damage

San Fernando Earthquake (1971)

Ventura [3] describes the previous damage to the target structure caused by the 1971 San Fernando earthquake. He states that all of the columns located at the corners suffered structural damage. In addition some exterior girders in exterior frames in the second floor were cracked and one shear wall in one sublevel had a small diagonal crack. Ventura emphasizes that cracking in columns was near the spandrels in the second floor. The repairs for the damage on the second floor consisted of stiffening the joint between the columns and girders using post-tensioned tendons.

Northridge Earthquake (1994)

After the Northridge earthquake in 1994, the structure showed similar damage. Naeim [2] gives specific information about the damage suffered by the target structure and classified the damage as generally moderate. Cracks in the sublevels and the second floor were repaired with epoxy injection.

MODEL DESCRIPTION

The target structure was modeled as two main frames (second sublevel to the roof) and an additional wall frame (2nd sublevel to the ground only). Only two of the three main frames were modeled, recognizing that the exterior frames are nearly identical. The direction of the maximum drift response (east-west) was chosen as the modeling direction. Three models were evaluated in the study:

- An as-designed model based on the actual structural properties as described in the structural plans including all stories above the ground. (Model A),
- An as-designed model based on the actual structural properties as described in the structural plans including all floors (Model B), and
- A model modified to reflect the flexibility of the walls and girders in the sublevels (Model C).

The calculated responses of the models were compared to determine the best representation of the recorded response of the building. The first model (Model A), which included all stories above the ground floor, was created because the displacement response below the ground was small. After analyzing the response of Model A, it was found that it did not adequately represent the behavior of the target structure, and it was then necessary to include the elements in the sublevels. Even though Model B had all the actual structural properties of the building, it did not adequately represent the recorded response near the base. A model (Model C) was developed and analyzed incorporating the increased flexibility of the sublevels and the foundation.

The models that included all floors of the structure (Models B and C) were modified using the strength-based relationship as described in the last section of this paper. Modifications were made to the strength of selected girders and the new response of the structure was determined for moderate and high earthquake demands.

Nonlinear Analysis Routine

The analysis routine used to calculate the nonlinear response for the target structure was called LARZ. This program was developed at the University of Illinois by Otani [4] and modified later by Saiidi [5] and Lopez [6].

Nonlinear static analysis was used to calculate the base shear response and the estimated yield mechanism. Nonlinear dynamic analysis was used to calculate the lateral drift of the model and compare it with the recorded response. Gravity effects, or P-Δ effects, were taken into account in the analyses. The equivalent viscous damping in the modeled system was assumed to be 2% of the critical damping.

Member Properties

In order to calculate the flexural deformations of the structural elements, the moment-curvature (M-ϕ) relationship for each element was calculated and provided as input. The M-ϕ relationship is a trilinear curve defined by distinct points representing cracking, yielding and ultimate moment and curvature values. A post-yielding slope of 1% of the slope to yield was used to define the final portion of the curve. The gross sectional area was used to calculate the initial uncracked stiffness for each element. The stiffness calculated for the girders included the contribution of the slab that was determined by two 45^o lines drawn from the bottom of the beam to the bottom of the slab.

Representation of the Target Structure

A nonlinear dynamic analysis was completed to compare the behavior of the model with the behavior of the instrumented structure. The input earthquake motion used for the analysis was the acceleration record obtained from the building instrumentation in the chosen modeling direction.

An estimate of the lateral stiffness of each frame was calculated to determine an appropriate distribution of mass for the modeled frames. Each frame was modeled separately and subjected to increasing lateral loads to calculate a force-deformation relationship for the frames. The lateral load distribution increased linearly with height. It was found that the exterior frame had a stiffness of approximately 2/3 the stiffness of the interior frame. Considering that only two of the frames were modeled, 71% of the total weight of the building was used in the analysis of the model.

The recorded response of the building showed that the deformations in the sublevel were not significant at the time of maximum drift (Figure 1). Initially the structure was modeled without including the sublevels to investigate the contribution of the flexibility of these floors to the overall response of the structure. Later models that were developed included the contributions of all stories to the structural configuration. For these models, the wall frame in the sublevel was represented as a one-bay frame with a wall member, a column element and a girder connecting the two vertical elements. Girder and column properties were decreased until these elements carried nearly zero load during the response. In the next section the results for this model will be discussed.

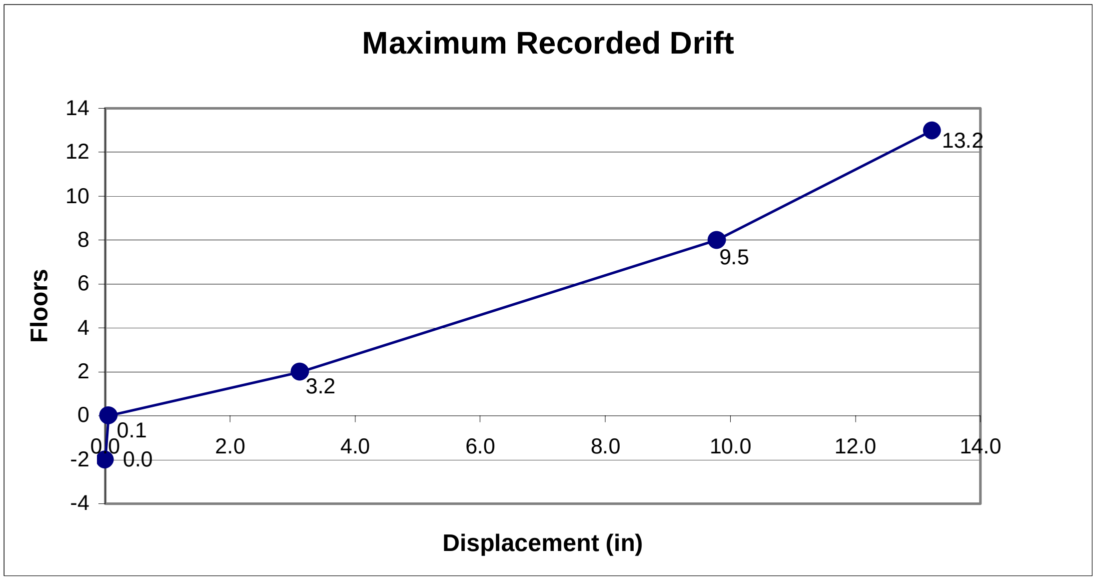

Figure 1. Maximum Recorded Drift

Results

Model Without Sublevels

The deformed shapes are shown in Figure 2 for Model A with the recorded response at two different times. The first recorded deformed shape is for the time of maximum displacement at the roof (36.54 sec). The second recorded deformed shape is for the time of maximum displacement calculated in the model (11.36 sec). Even though the maximum displacement values do not match between the model and the recorded response, the deformed shape of the model in the early portion of the response is quite similar to the recorded deformed shape, which shows how the overall response has been well represented.

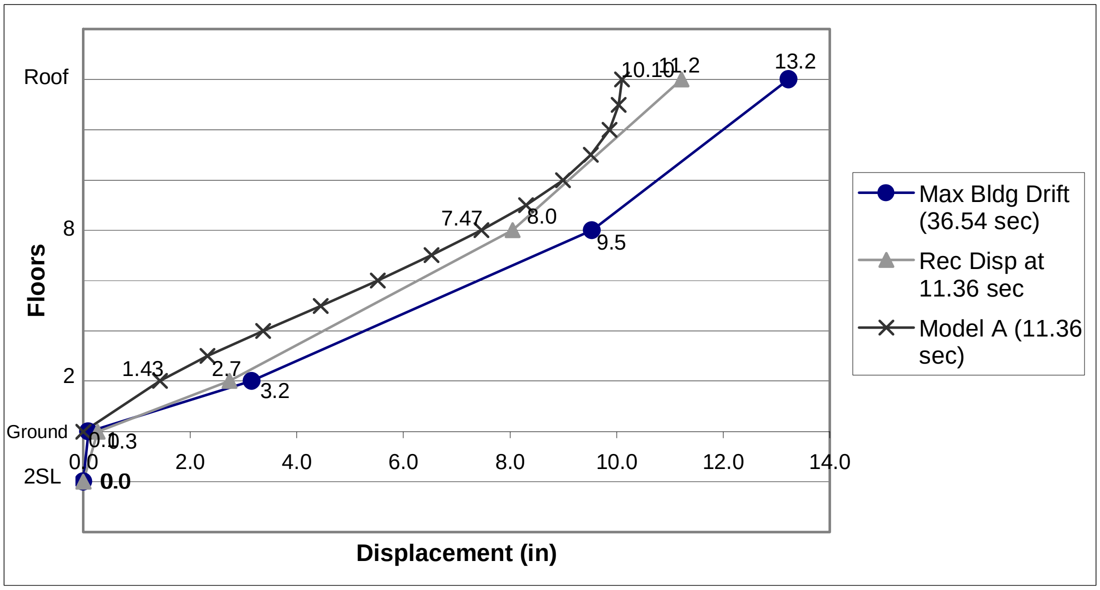

Figure 2. Comparison of Response for Model A (without Sublevels)

Model With Sublevels

The response of Model B indicated that the unmodified foundation model is stiff compared to the actual foundation (Figure 3). The actual maximum building drift occurred at 36.54 sec whereas the model maximum drift occurred at 11.36 sec. In order to simulate a softer foundation, Model C was created with more flexible lower-level structural properties. Because LARZ does not easily allow for the modeling of soil-structure flexibility, it was necessary to change the structural properties of the walls to provide a more flexible response at the sublevels as described in the following section.

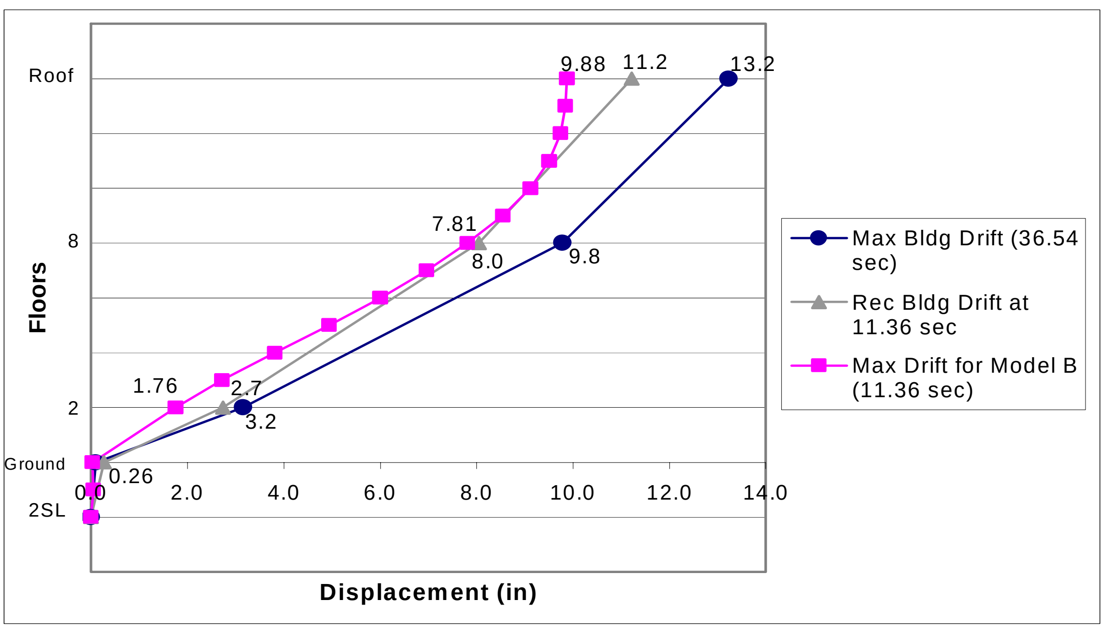

Figure 3. Deformed Shape for Model B

Model With Modified Properties

Properties for girders and walls at the sublevels were modified in Model C to investigate means for amplifying the lateral response at the top of the first sublevel. The uncracked moment of inertia of the girders was reduced by 50%. The percentage of reduction of stiffness for the wall from Model B that was implemented in Model C was determined from a combination of the values suggested in FEMA 273 [7] and Finn et al [8]. FEMA 273 guidelines recommend using 50 % of the uncracked stiffness for walls with cracks. The damage report after the San Fernando earthquake stated that there were some cracks in the walls and the stiffness was reduced to take into account that effect in the model. Finn et al (1997) suggest that the properties should be decreased additionally by 50 – 70%. The total reduction percentage value used for Model C was 82.5%.

The modeled response improved substantially in the sublevels using Model C. The deformed shape (Figure 4) and the displacement history (Figures 5 and 6) follow the record very closely. As can be seen in Figure 5, the periodicity of the model is nearly the same as the periodicity of the record. It is also noted from Figure 5 that the maximum recorded roof drift occurred in the latter portion of the earthquake record. This feature may indicate a degree of resonance between the soil surrounding the building foundation and the structure for the accelerations experienced during the 1994 Northridge earthquake. The models created in the study were unable to capture this behavior. From Figure 6 it is evident that the greatest improvement of Model C compared to Model B is that the response at the 2nd Floor is better represented by Model C.

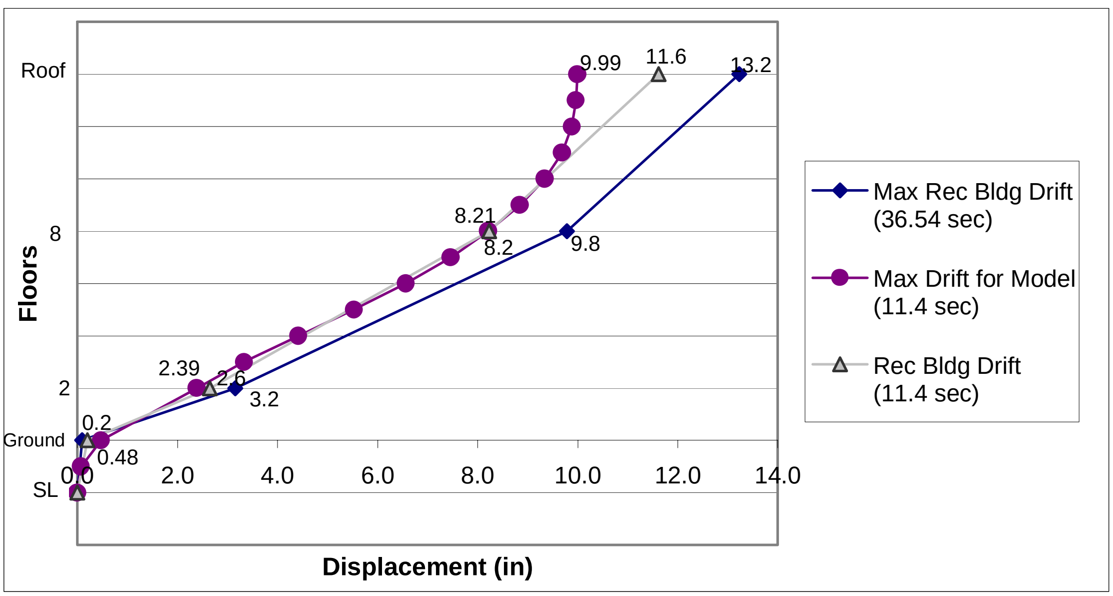

Figure 4. Deformed Shape for Model C

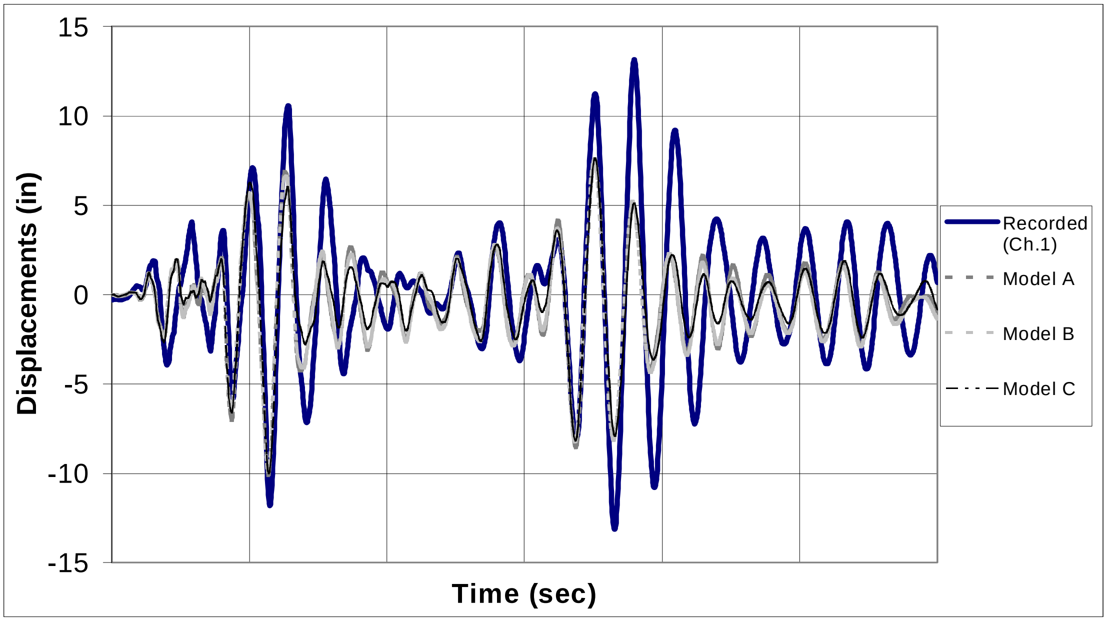

Figure 5. Displacement History for all Models at the Roof

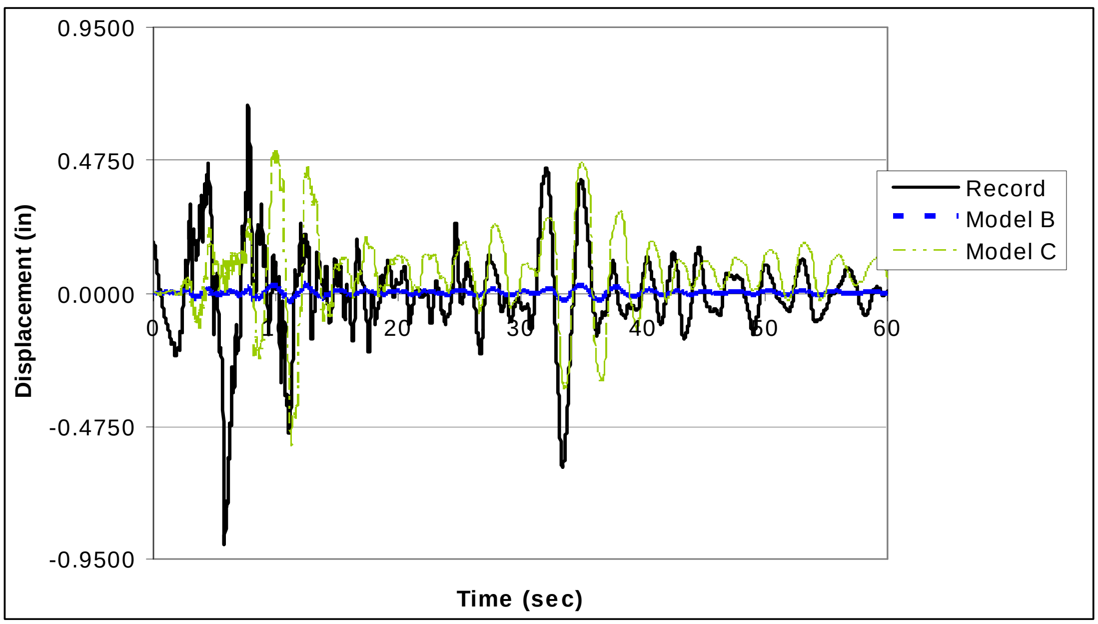

Figure 6. Displacement History for all Models at the 2nd Floor

Model Selection

This section discusses the results obtained for each model to select the best representation of the recorded response of the structure. Table 1 and Table 2 show the comparison of the periods and maximum displacements obtained from the displacement responses of the models and the recorded data. The average period of motion for the building at the roof is estimated for six time ranges and the values are shown in Table 1. Table 2 shows a comparison of the drift at the instrumented floors for the three models at the time of their maximum drift response with the drifts values from the recorded motion.

Table 1. Comparison of Period values for Models and Record

Model	T(sec) 1–10 sec Roof	T(sec) 11–20 sec Roof	T(sec) 21–30 sec Roof	T(sec) 31–40 sec Roof	T(sec) 41–50 sec Roof	T(sec) 51–60 sec Roof
A	1.48	2.5	2.5	2.5	2.5	2.86
B	2.11	2.5	2.5	2.5	2.5	2.86
C	**2.11**	**2.5**	**2.67**	**3.33**	**2.5**	**2.86**
Rec.	2.67	2.86	3.03	3.33	3.33	2.86

Table 2. Comparison of Maximum Deformed Shape Drift Values

Model Floor	A (t=11.36 sec)			B (t=11.36 sec)			C (t=11.40 sec)		
	Model (in.)	Rec (in.)	Ratio	Model (in.)	Rec (in.)	Ratio	Model (in.)	Rec. (in.)	Ratio
Ground	0	0.3	NA	0.03	0.3	0.10	**0.48**	**0.2**	**2.4**
Second	1.4	2.7	0.53	1.8	2.7	0.67	**2.4**	**2.6**	**0.92**
Eighth	7.5	8.0	0.94	7.8	8.0	0.98	**8.2**	**8.2**	**1.0**
Roof	10.1	11.2	0.90	9.9	11.2	0.88	**10.0**	**11.6**	**0.86**

The analysis of the data shows that the behavior of Model C better represents the overall recorded response of the structure. The period values of Model C for all the ranges of time are closer to the values from the recorded response. The deformed shape of Model C closely follows the recorded deformed shape of the structure. The most important improvement in the modeling of the target structure is that Model C is able to simulate some displacement at the ground floor, so that it better represents the response of the lower levels of the building.

MODIFICATION USING THE STRENGTH REDUCTION FACTOR

Models B and C were analyzed using a nonlinear static analysis routine (LARZ) to estimate the yielded shape of the structure. The strength of the girders for Models B and C were modified with the relationship developed by Kuntz [1] and analyzed to evaluate the change in the structural response. The modifications and results are discussed in this section. The different models present an opportunity to compare the results for a model with a stiff foundation (Model B) and a model with a softer foundation (Model C).

The objective of the strength-based relationship (Kuntz [1]) was to improve the structural response of a building system. The manner to improve the response was to encourage the formation of a structural mechanism instead of an intermediate or story mechanism (Figure 7). By encouraging a structural mechanism to form, yielding in the columns above the base is eliminated and the deformed shape is improved. The structural mechanism is achieved by forcing the yielding to occur in the girders instead of the columns, and as a result a more uniform distribution of drift will be developed over the height of the structure.

The strength-based relationship reduces the strength of the girders by a factor that depends on several variables. There is a natural lower bound to the reduction that must be based on the minimum strength necessary to resist the gravity loads supported by the girders. The equation developed by Kuntz [1] is presented below:

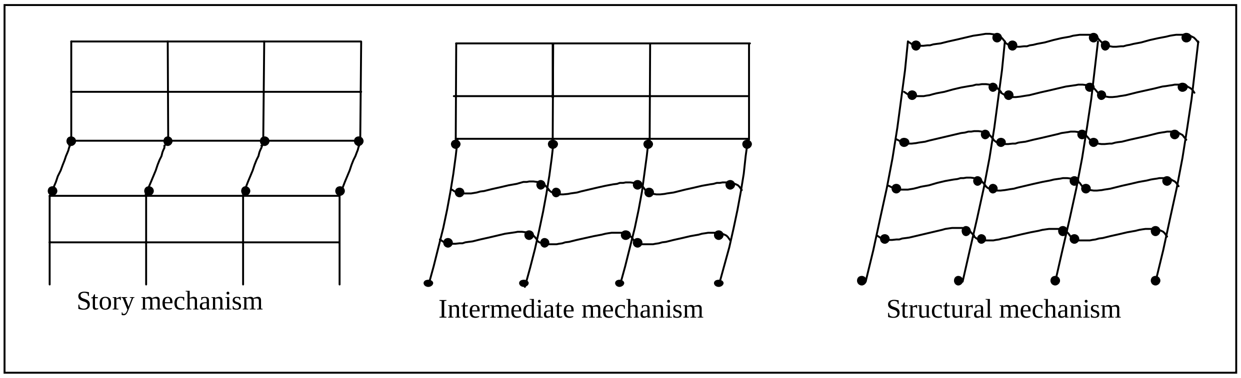

Figure 7. Yield Mechanisms for Regular Frames

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

where: N_s = number of stories above ground
N_b = number of bays
ρ = column reinforcement ratio as an average for all columns in the structure
h = column width (inches), as the square root of the cross sectional area
β = ratio of number of floors with reduced girder strength to total number of stories above ground
α = ratio of top exterior column strength to girder strength (based on average strengths)

The reduction factor obtained for the target structure was extremely large (6.41) as shown in Table 3. It was not possible to apply it to the model based on the limitations due to gravity loads. The strengths of the girders were reduced by a common factor value, 1.5, so that the modified girder strengths were approximately 66% of their original strength values. The 5 middle girders of the interior frame from the 9th through the 13th floor were modified by a lower factor due to the restrictions imposed by the gravity load demands. It is anticipated that many moderate-rise frames will experience similar restrictions when applying the proposed strength reduction factor.

Table 3. Factor R Variables Values

Ns	Nb	ρ	H (in)	Mc (k-in)	# girders	Mg (k-in)	α	β	**R**
13	7	0.02	33	6105	182	7889	0.77	0.38	**6.41**

The models were analyzed for different earthquake demands. The selection of various acceleration records demonstrated the influence of earthquake frequency content and intensity on the response of the models. The additional records were chosen to create greater demands to the structural elements than the original recorded accelerations (i.e., to create a yield mechanism for the structure). The analyses were evaluated based on calculated yielding locations, deformed shapes, and inter-story drift ratios (SDR, the ratio of inter-story drift to story height).

The three earthquakes chosen were:

1. San Fernando – Pacoima Dam record (1971)
2. Kobe (1995)

3. Northridge (1994) – Original Building record

San Fernando Earthquake

The San Fernando earthquake caused the exterior frame of the Model B to have an intermediate mechanism, i.e., with the bases of all of the columns in the ground floor and the top portion of all of the columns in the 7th floor yielding. The results for Model B after being modified by the strength reduction factor showed that the intermediate mechanism moved up the structure with columns now yielding between the 8th and 12th floors. As expected, the interior frame for the unmodified model did not develop a yield mechanism (because it had greater relative stiffness than the exterior frame). It is noted that the modified model response had yielded girders in higher floors than the original model. This is the expected behavior for the modified models because it will be the path to follow if a structural mechanism is to form.

Model C did not reach a yield mechanism under the influence of the San Fernando earthquake. Although all of the girders reached yielding during response, the columns did not yield. Therefore, no improvement was demonstrated in the modified model response.

Figure 8 shows the deformed shapes for unmodified and modified Models B and C. Upon inspection of the deformed shapes, the change between the response of the unmodified and modified Models B and C appears to be nearly the same. Model C experiences greater maximum drift because of the decreased stiffness in the lower levels of the frame.

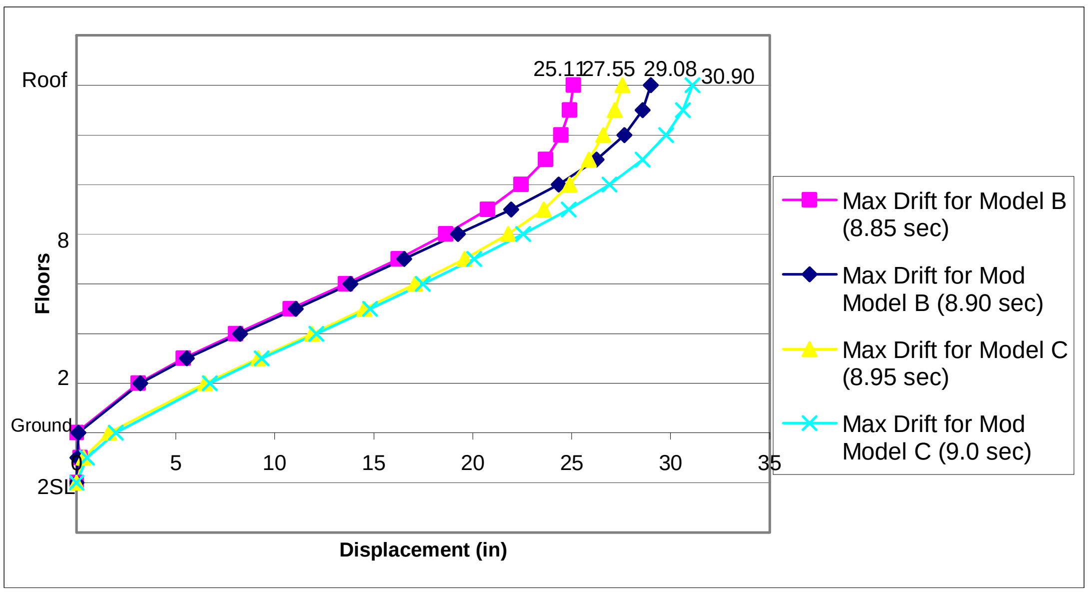

Figure 8. Deformed Shapes for all Models Subjected to Pacoima Dam Record

When Model B is subjected to the San Fernando Earthquake record the SDR values for the modified model are more uniform from the 3rd through the 10th floors than the SDR calculated for the unmodified model (Figure 9). It is also observed that there was a reduction in the SDR values for the two sublevels where there was a wall with a large stiffness value. The top stories showed an increase in SDR because the reduction of the strength of the girders induced the structure to have a more evenly distributed drift over the total height of the structure. Although the strength reduction factor does help the inter-story drift ratio of modified Model C to have a more even distribution than the unmodified Model C, the improvement was not as much as that obtained for Model B. The SDR increased for all of the floors of the modified Model C, which may be because the softer foundation induced larger displacements in the structure (Figure 10).

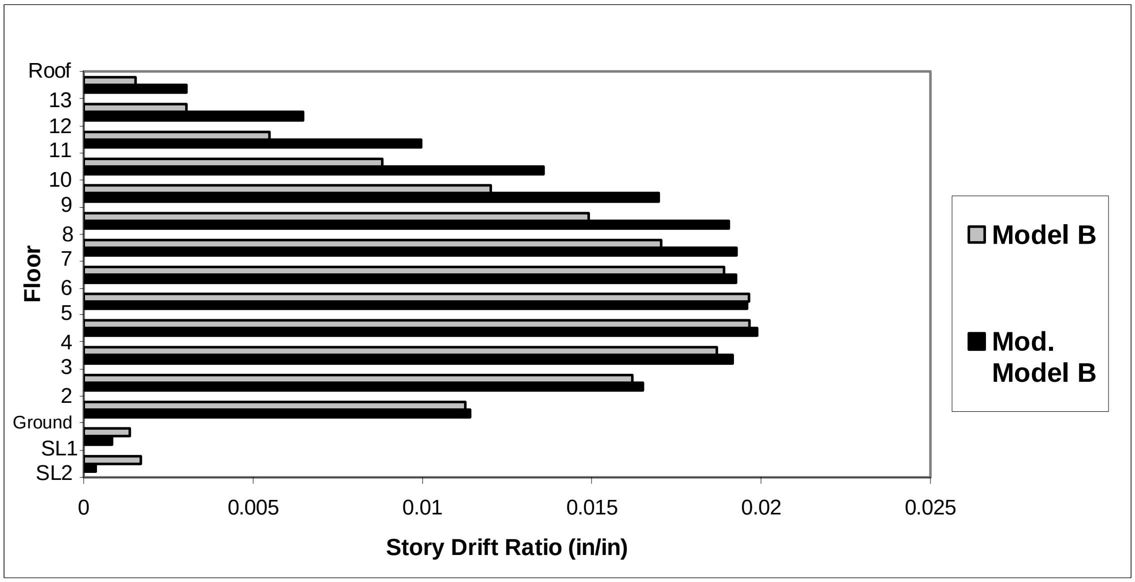

Figure 9. Comparison of SDR Values for Unmodified and Modified Model B, San Fernando Earthquake

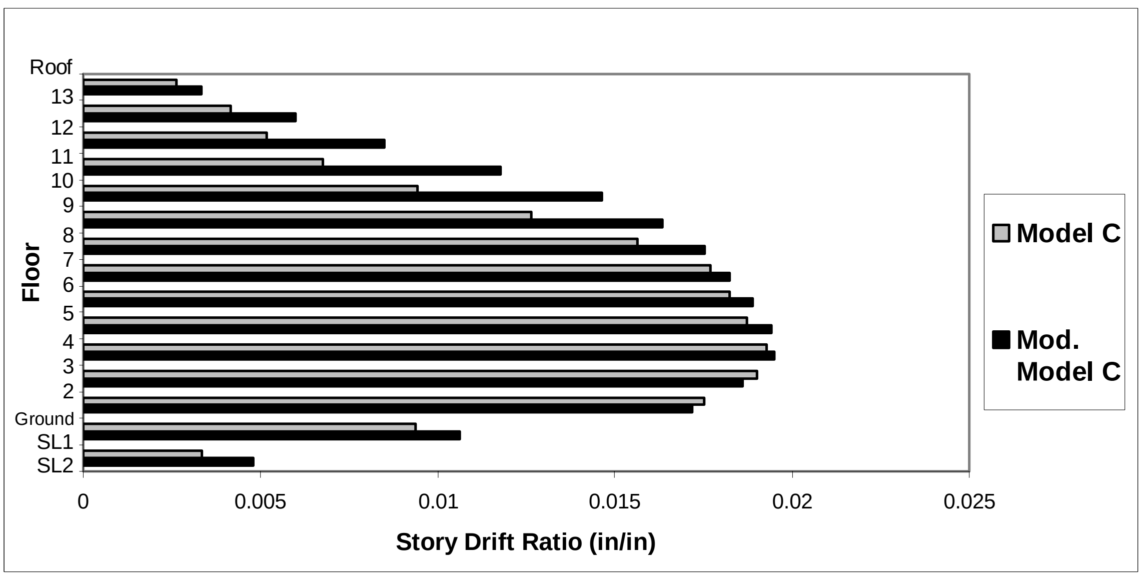

Figure 10. Comparison of SDR Values for Unmodified and Modified Model C, San Fernando Earthquake

Kobe Earthquake

The Kobe record affected Model B by creating a story yield mechanism between the 11th and 12th floors of the structure (yielding occurred in the top portions of the columns of the 11th and 12th floors and for all of the girders at the 12th floor). The yield profile for the modified model was improved as there was no mechanism present. The columns in the 11th floor did not reach yielding and additional girders on the top floors (13th and roof) yielded to improve the deformed shape of the structure. For the case of Model C, application of the strength reduction factor improved the yield profile for the exterior frame (a yield mechanism was not present after the factor was applied) but no improvement was noted for the interior frame. Application of the strength reduction factor did not significantly improve the deformed shapes of the frames when subjected to the Kobe record. The deformed shapes of the modified models followed approximately the same shape as the unmodified model responses.

Northridge Earthquake

Applying the strength reduction factor to Model B improved the yield profile in response to the Northridge earthquake. The unmodified frames had yielded girders through the 9th floor of the exterior frame and the 8th floor of the interior frame. Both frames of the modified model had yielded girders through the 11th floor, indicating a greater spread of inelastic action over the height of the structure. When yielding occurs in the top floors of a structure, the drift can be redistributed over a larger portion of the structure and improve the response.

Figure 11 shows the deformed shapes for the unmodified and modified models B and C when subjected to the Northridge earthquake. It appears that for this moderate earthquake demand the strength reduction factor does help to reduce the drift at mid-height while the drift at the upper floors is increased. The drifts calculated in the sublevels remain almost constant for Model B and Model C. Figure 12 and 13 show that the calculated inter-story displacements for the models subjected to the Northridge earthquake were reduced at mid-height and increased for the upper floors of the structures. The SDR for the first sublevel of Model B (having a stiff foundation) was increased for the modified model response, which demonstrates how the redistribution of drift is evident in all floors of the structure. For Model C, the SDR for both sublevels were reduced and SDR became more uniform for the floors above.

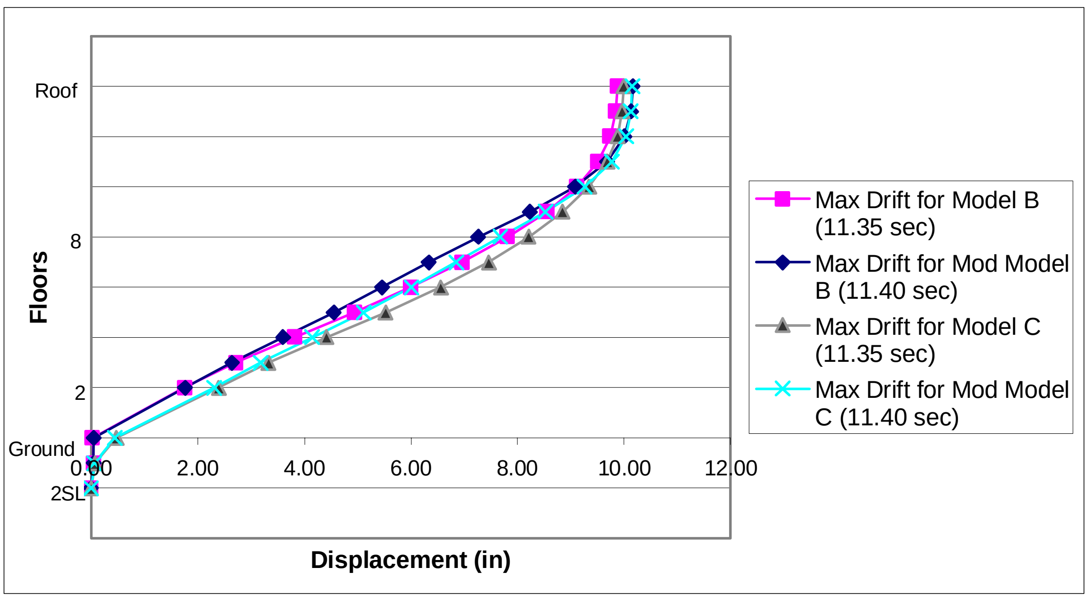

Figure 11. Deformed Shape for all Models Subjected to Northridge Earthquake

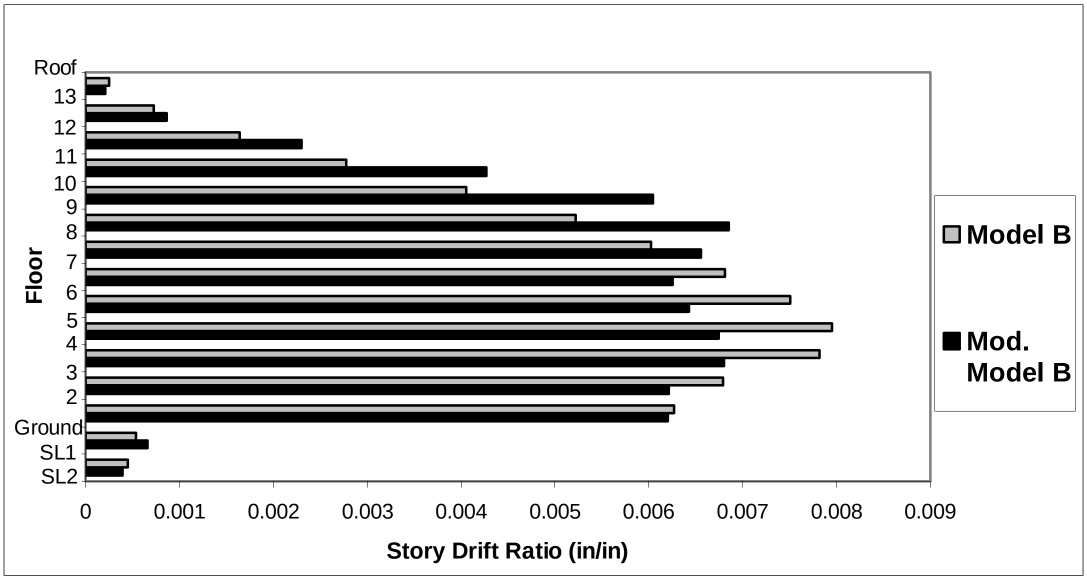

Figure 12. Comparison of SDR Values for Unmodified and Modified Model B

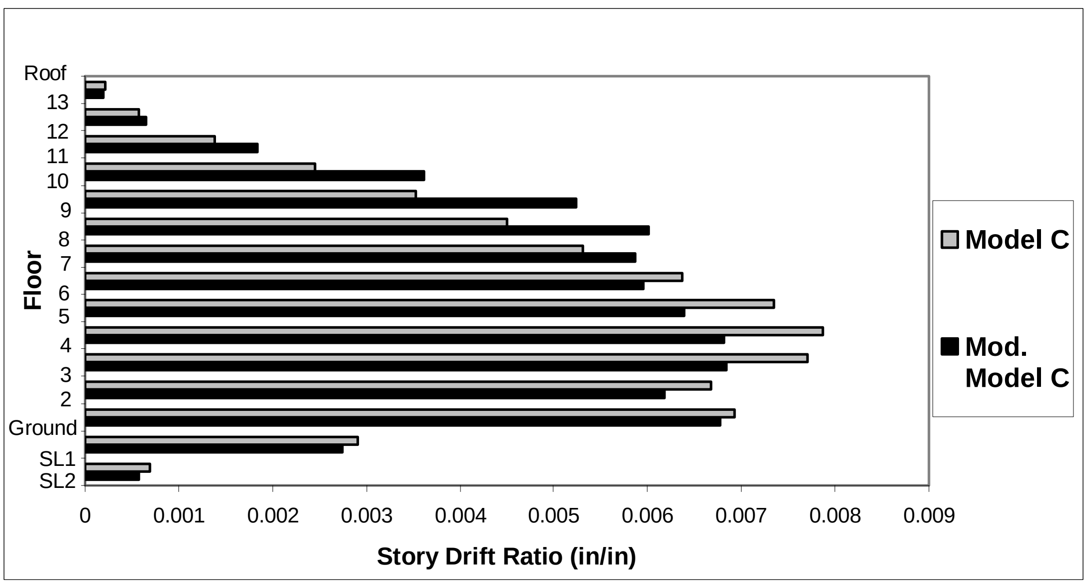

Figure 13. Comparison of SDR Values for Unmodified and Modified Model C

CONCLUSIONS

This study led to several conclusions about the modeling and model modification of a 13-story structure:

- It is not possible to ignore the influence of the flexibility of the sublevels to obtain a representative response of the target structure.
- It may not be possible to use the full calculated value of the strength reduction factor for moderate-rise frames because of gravity-load capacity requirements. Application of the largest factor possible will still provide some improvement to the overall response of the structure as measured by yield profiles, deformed shapes, and SDR.
- The strength reduction factor had more influence on the response of a structure with a stiff foundation (Model B) than with a flexible foundation (Model C). This was shown in yield profiles, deformed shapes, and SDR.
- The earthquake demand (intensity and frequency content) influences the effectiveness of the strength reduction factor in improving the response of a structure. Application of the strength reduction factor tends to increase the maximum roof drift while improving the yield profile and the distribution of the drift over the height of the structure. For large earthquake intensities, the maximum roof drift must be controlled when applying the strength reduction factor so that SDR in the upper stories do not exceed allowable limits.
- The response of the structure at the roof is adequately represented when all of the elements are modeled using a tri-linear curve for the hysteretic response and using gross-section properties to define the uncracked stiffness. When the stiffness of the sublevels is reduced, the response at the base is improved, but the response at the roof remains nearly identical to the original model.

ACKNOWLEDGEMENTS

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