17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020
Paper N° C001616
Registration Code: A01823

SEISMIC PERFORMANCE OF TRANSFER STORY REINFORCED CONCRETE BUILDINGS

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Abstract

This investigation involves evaluation of the behavior of a six-story five-bay reinforced concrete building with a transfer story located at the third level subjected to synthetic ground motions corresponding to Los Angeles, California. The buildings lateral load resisting system consists of special moment frames with vertical irregularity Type 4 as specified by the ASCE 7-16 [1] due to the second and the fifth column discontinuity in the third story. The transfer story girders and its supporting columns are designed using special load combinations of the ASCE 7-16 [1] provisions that include amplifying the seismic demand by using the over-strength factor ($\Omega = 3$). This resulted in impracticality of either meeting the strong-column weak-beam requirement at the column-beam joints located in the mid-span of the transfer girders or not including the corresponding columns in the lateral load resisting system, as specified by the ACI 318-14 [3]. Thus, nonlinear dynamic time-history analysis was performed by using ETABS software [2] to evaluate the seismic response of the building. The results show that the rotational ductility demands in all plastic hinges in columns in the story above the transfer girders may exceed the expected limits for the collapse prevention performance, resulting in a weak story failure mechanism. The unfavorable behavior in the building is due to not having the ACI 318-14 strong-column-weak-beam requirement satisfied at joints located at the mid-span of the transfer girders.

To address this concern, the columns located at mid-span of the transfer girders (in the story above the transfer story) are strengthened by the over-strength factor ($\Omega = 3$), as it was done with the transfer girders and its supporting columns. The result of nonlinear time-history analysis of the proposed strengthened building shows that rotational ductility demands of the column plastic hinges in the story above the transfer girders meet the life safety performance objectives; thus, the proposed strengthen procedure is found satisfactory.

In summary, for the reinforced concrete buildings having a transfer story where all continuous columns supporting transfer girders, a concern for possible collapse mechanism at the story above the transfer story level is addressed by strengthening the columns located at the mid-span of the transfer girders by the over-strength factor. Subsequently, the adequacy of the proposed strengthened building is verified by conducting nonlinear time-history analysis, where the results indicate acceptable plastic hinge patterns with corresponding rotational ductility demands in the building.

Keywords: Reinforced concrete building; Seismic response; Transfer story; Plastic hinges; Earthquake resistant design

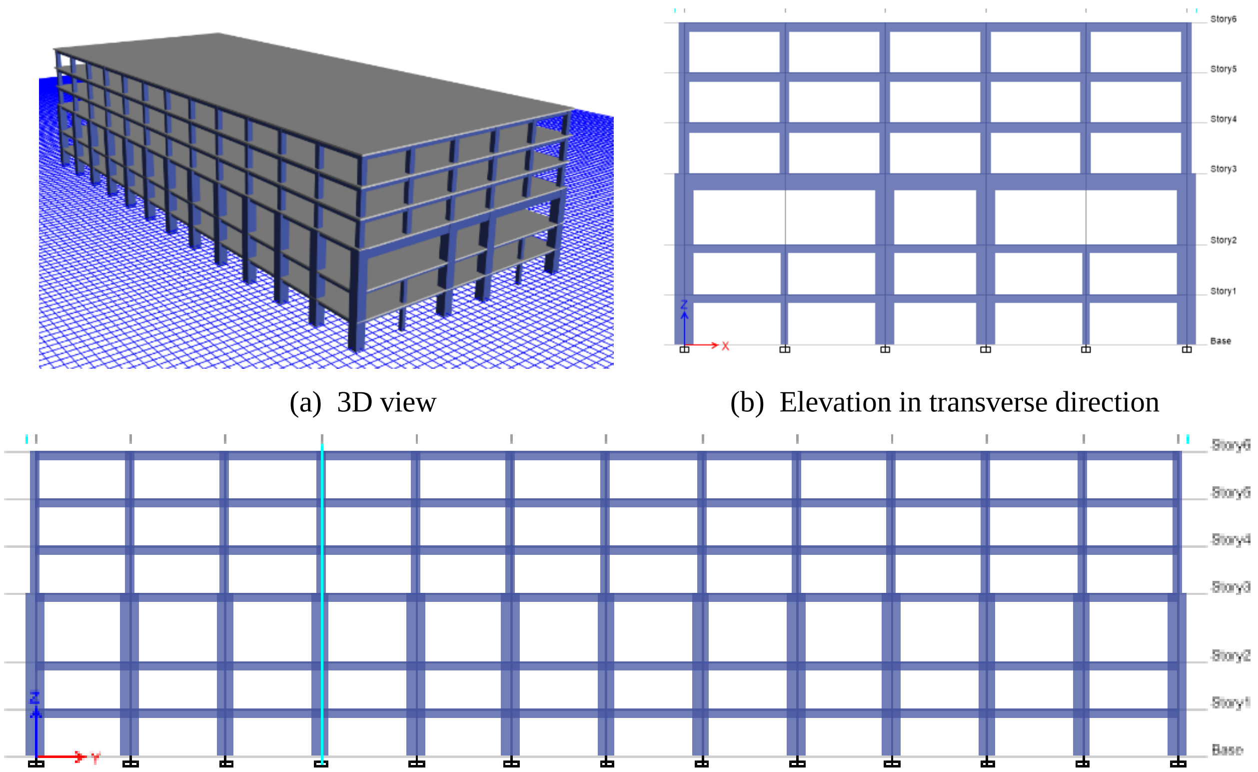
1. Introduction

Due to the architectural requirements reinforced concrete buildings with special moment resisting frames as the lateral force-resisting system may have in-plane discontinuity in some of the columns, classifying it as a Type 4 vertical irregular structural system by the ASCE 7-16 [1]. The equivalent lateral force (ELF) procedure is permitted to design such buildings with a structural height of less than 160 ft., while the transfer story girders and its supporting columns are designed using special load combinations that include amplifying the seismic effects by using the structural over-strength factor. This is done to protect the gravity load-carrying systems from possible overloads caused by the over-strength of the lateral force-resisting system according to the commentary of Section 12.3.3.3 of the ASCE 7-16 [1]. This may result in most members of the transfer story (the transfer girders and the supporting columns) to be designed using the over-strength factor, potentially changing the anticipated pattern of plastic hinges in that story. Also, it is observed that it is impractical either to meet the strong-column weak-beam requirement at joints located in the mid-region of the transfer girders, or to not include the corresponding columns in the lateral load resisting moment frame, as specified by the

ACI 318-14 [3]. Thus, nonlinear response of such a building needs to be investigated to verify the adequacy of their performance when subjected to strong ground motions.

2. Description of the designed building with a transfer story

In this research a six story reinforced concrete building with special moment resisting frame systems with Type 4 structural irregularity due to discontinuous columns in one direction is designed according to the ASCE 7-16 [1] and the ACI 318-14 [3], and subsequently it is analyzed using nonlinear time-history dynamic procedure to evaluate the seismic response of the building by ETABS software [2]. The building is symmetric in both directions with one transfer story at the third level as shown in Fig. 1. The typical frame bay spacing of 24 ft is used in both directions, except in the transfer story where two transfer girders with 48 ft spans are used in the short direction (see Fig. 1). The typical story height is 12 ft, and the transfer story height is 17 ft. The floor diaphragm consists of beam supported 9 in. thick concrete slab at all levels. The building has 12 bays in the longitudinal direction and 5 bays in the transverse direction, with a floor plan aspect ratio of 2.4 < 3; therefore, rigid diaphragms are permitted to be used in the analysis [1]. Thus, to simplify the calculation, a 2D moment frames are used for the design of the building.



(a) 3D view (b) Elevation in transverse direction

(c) Elevation in longitudinal direction

Fig. 1 - Building model

A concrete compressive strength of 4000 psi and reinforcing steel grade of 60 are used. A service floor live load of 40 psf (residential) and a roof live load of 20 psf are use in addition to building members' self-weight and superimposed dead load of 20 psf.

The building is assumed to be located in Los Angeles, California, with a 0.2 sec. spectral acceleration of S_s = 1.853g, a 1 sec. spectral acceleration of S_1 = 0.651g, and a long-period transition period of T_L = 8 sec. The earthquake load corresponding to Soil Class Type "D" (resulting in a Seismic Design Category of D), Response Modification Coefficient (R) of 8, Deflection Amplification Factor (C_d) of 5.5, and Over-strength factor (Ω) of 3 is used in the design of the building per ASCE 7-16 [1].

The ACI 318-14 [3] specified effective member cross-sectional properties are used to account for their stiffness reduction when using the equivalent lateral force procedure to design the frame members. For the transfer story building, special load combinations where the over-strength factor is included, $(1.2 + 0.2\ S_{DS})D + \Omega Q_E + L$ and $(0.9 - 0.2\ S_{DS})D + \Omega Q_E$, are used to design the transfer girders and the corresponding supporting columns. The summary of the designed beam and column dimensions and reinforcing steel areas are shown Figs. 2, 3, and 4.

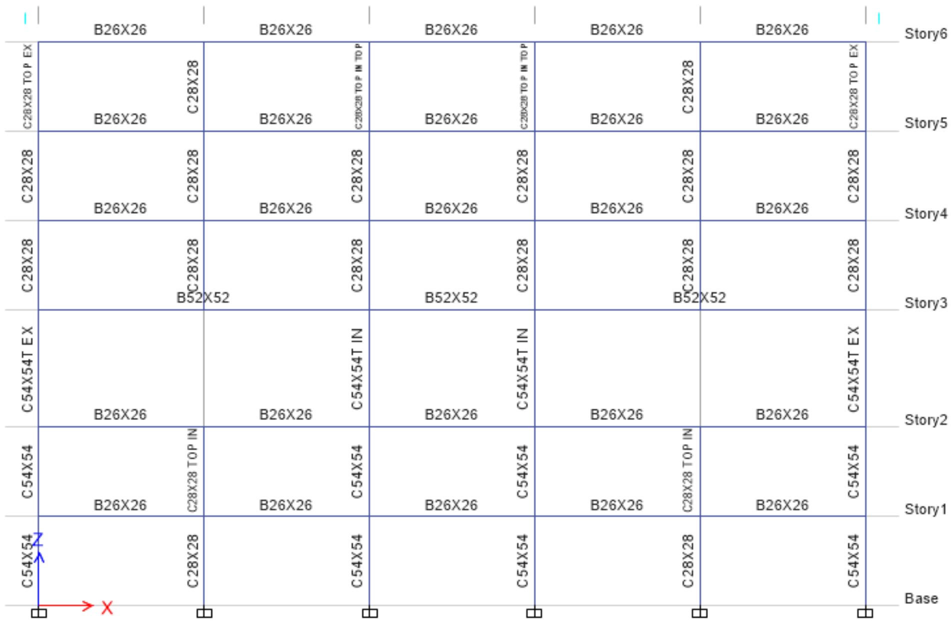


Fig. 2 – Frame member's cross-sectional dimensions (in.)

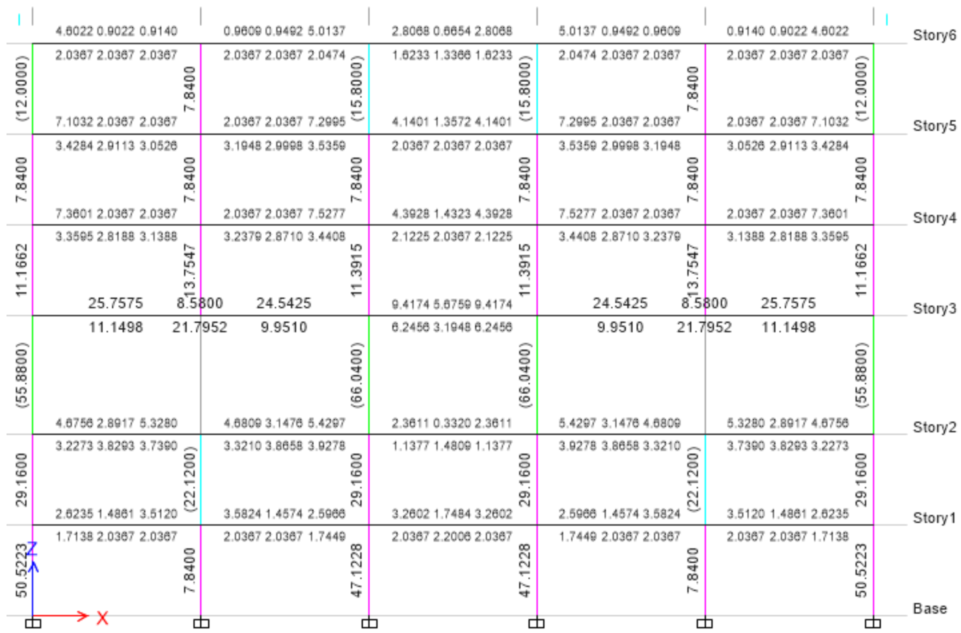


Fig. 3 – Beam and column longitudinal reinforcing steel areas (in^2)

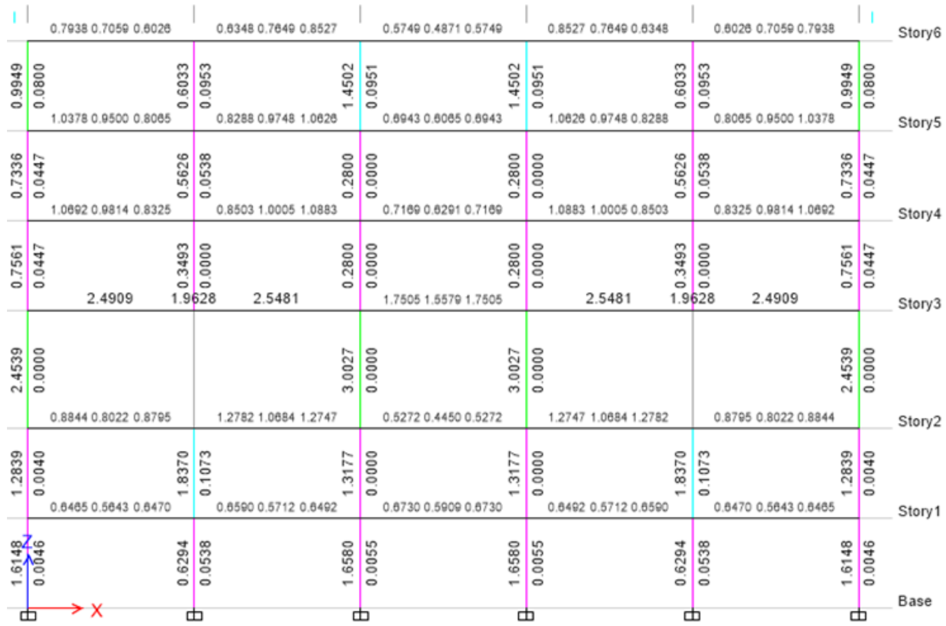


Fig. 4 – Beam and column shear reinforcing steel areas (in^2)

Since the ACI 318-14 strong-column weak-beam requirement in joints located at the mid-span of the transfer girders was not satisfied due to the practicality limit of the moment frames, nonlinear response of the building needs to be investigated to verify the adequacy of their performance when subjected to strong ground motions. It was also observed that at face of supports at the end of few beams the positive moment strength was less than half of the negative moment strength of the beam at that location (Fig. 3) as required by the ACI 318-14 [3] for special moment frames. Thus, the bottom steel area of such beams was manually increased to half of the top steel area when specifying beam plastic hinge properties in ETABS prior when conducting nonlinear time-history analysis.

3. Nonlinear time-history analysis of the designed building using ETABS

3.1 General

ETABS model of a typical 2D frame in the short direction (see Fig. 1b) is constructed and used to conduct nonlinear time-history analysis of the structure. The initial gravity load for both types of nonlinear analysis consists of 1.05 x (Dead Load) and 0.25 x (Live Load) per FEMA P695 [6]. P-Δ effects are included in nonlinear analyses as specified by the ASCE 41-13 [5] to account for gravity loads acting through the lateral deformation of a structure resulting in an increased story drifts.

3.2 Plastic hinge locations and properties

Plastic hinge locations in ETABS model are defined at the center of hinge regions on beams and columns. In the 2D frame used in this study hinge regions are expected to occur at both ends of the beams and columns. The ACI 318-14 [3] recommends that the length of the hinge region (l_o) is computed as: (a) in beams as 2 times the depth of the flexural member, and (b) in columns as

$$\text{the } maximum\ of \begin{cases} 18\ in. \\ \frac{1}{6}\ clear\ span\ of\ the\ column \\ The\ depth\ of\ the\ column\ at\ the\ joint\ face \end{cases}$$

To perform nonlinear analysis, the bending moment-rotation properties of the plastic hinges with pivot hysteresis characteristics are defined automatically from the element material and section properties for the beams in ETABS [2] as shown in Fig. 5. Isotropic hysteresis model of ETABS was used for plastic hinges in columns where the corresponding bending moment-rotation properties with axial load-moment diagram are shown in Fig. 6.

The structural energy dissipation in the nonlinear dynamic analysis occurs in the plastic hinges due to their hysteresis behavior. The Rayleigh damping matrix, where a linear combination of mass matrix and stiffness matrix is used for nonlinear analysis providing additional 2.5% viscous damping for reinforced concrete members as recommended by the ASCE 7-16 [1]

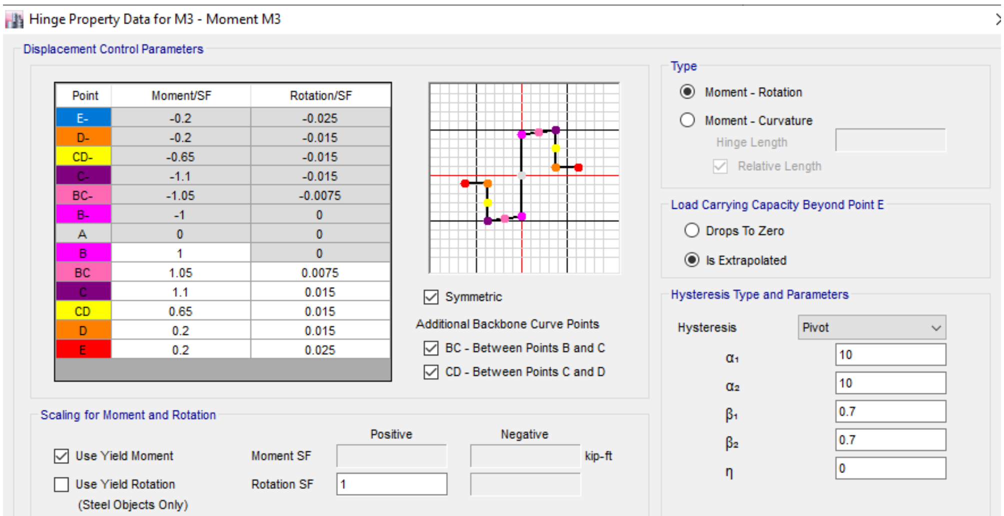


Fig. 5 – ETABS model hinge property for beams (M3)

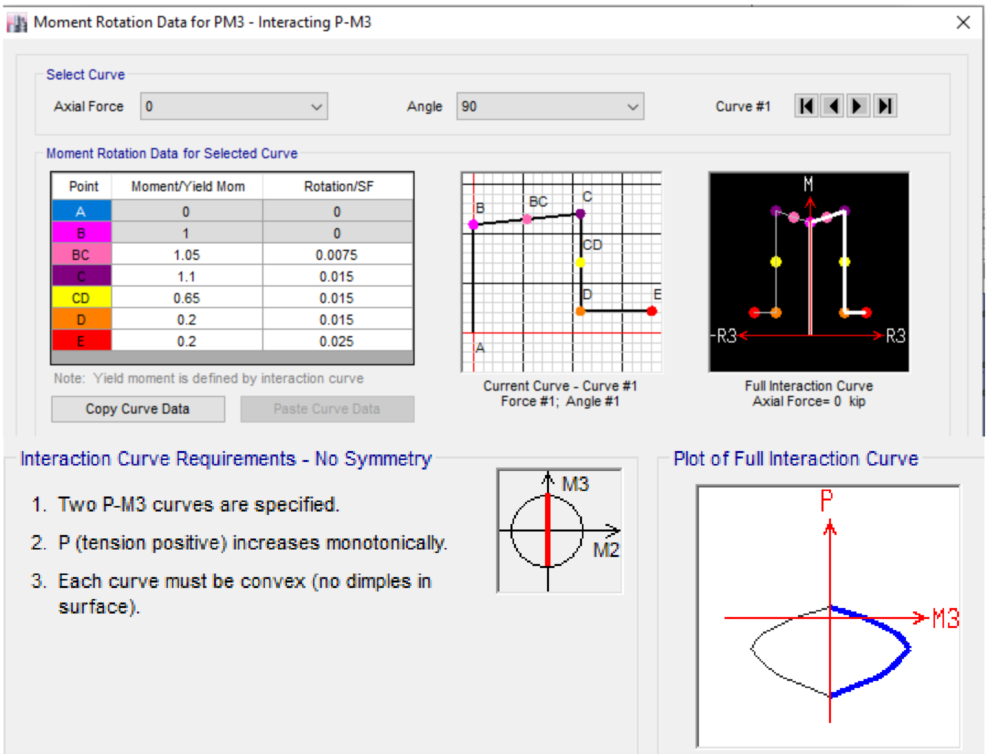
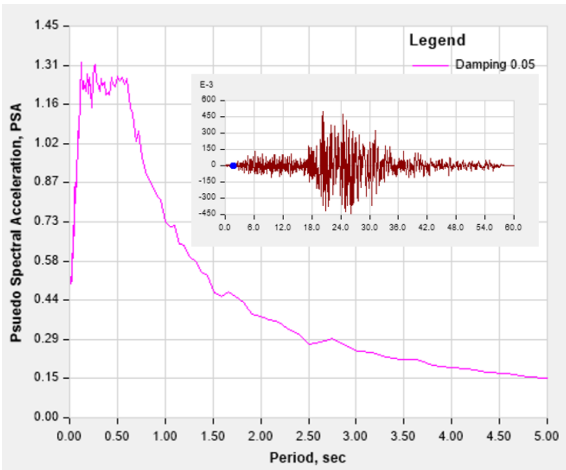


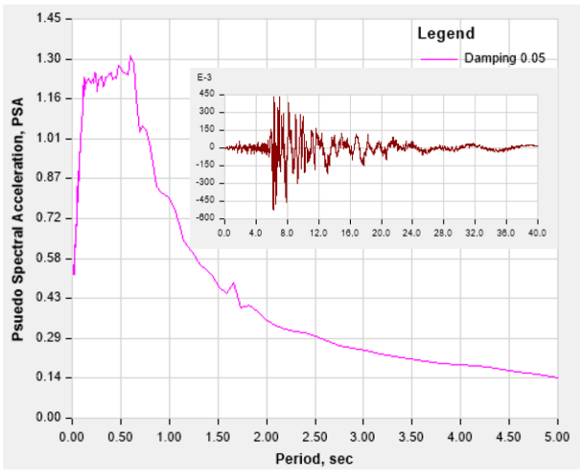
Fig. 6 - ETABS model hinge property for columns (P-M3)

3.3 Selection and scaling of ground motions

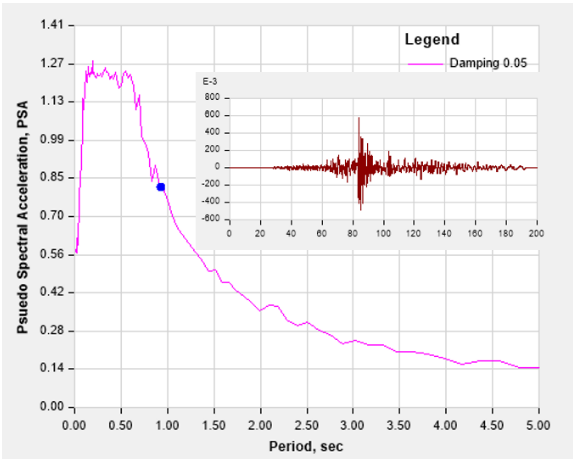
Three ground motions in Los Angeles, California, were selected and scaled to match to the target design response spectrum according to the ASCE 7-16 [1], where the average spectra of scaled ground motions is not less than 90% or more than 110% of the target design spectrum in the period from 0.2T to 1.5T (T is the fundamental period of the structure). Three synthetic time histories generated from three ground motion records Big Bear, North, and Sierra obtained from Peer Ground Motion Database website [9] using the ASCE 7-16 scaling procedure are shown in Fig. 7.



a) Big Bear



b) North



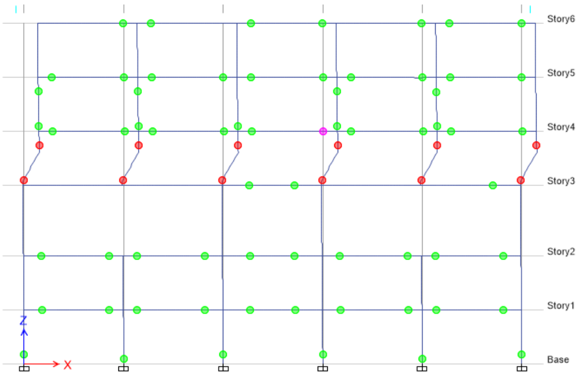
c) Sierra

Fig. 7 - Response spectrum of synthetic ground motions

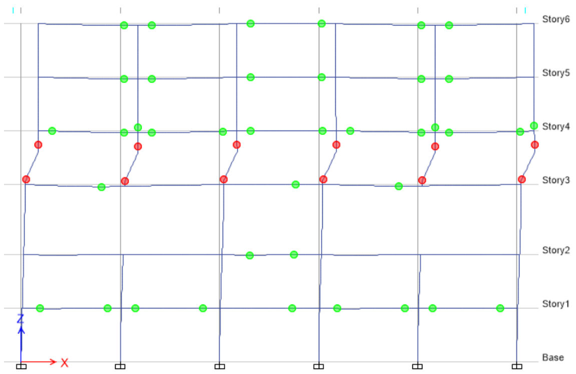
3.4 Seismic performance of the building

Nonlinear dynamic time-history analysis was performed by using ETABS software [2] to evaluate the seismic response of the building subject to three synthetic earthquakes. The overall performance of the building is illustrated in Fig. 8 where the plastic hinging pattern with the corresponding rotational ductility demands in all columns located in 4th story, shown in “red” in Fig. 8, exceed the expected limits for the Collapse Prevention (CP) performance, resulting in a weak-story collapse mechanism. It is noted that although the labels “A, B, C, D, and E” shown on the key of Fig. 8 correspond to the ones with given moment-rotation values in Figs. 4 and 5, but the colors shown on the key of Fig. 8 are different that the ones shown in Figs. 4 and 5.

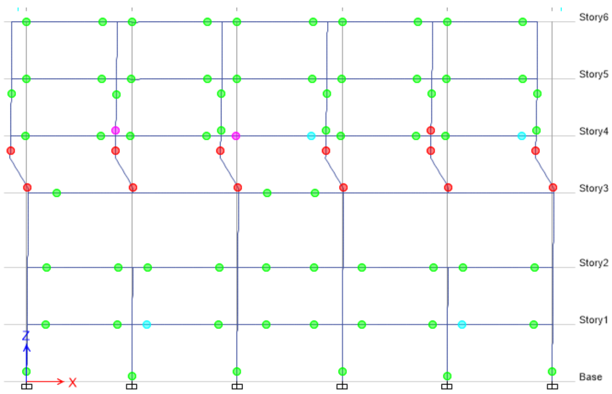
More specifically, Fig. 9 illustrates the behavior of a typical 4th story column plastic hinge (located at mid-span of the transfer girder) and a typical 4th story beam plastic hinge (located at the right end of the third span beam) when the frame is subjective to the synthetic Big Bear earthquake. It is observed that both hinges do not meet the “Collapse Prevention” performance objectives for such members due to excessive hinge rotations (as shown in Fig. 9).



a) Big Bear

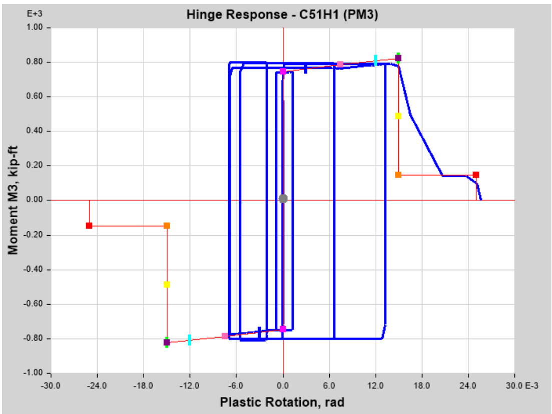


b) North



c) Sierra

Fig. 8 – Plastic hinging patterns of the designed building subjected to three synthetic ground motions



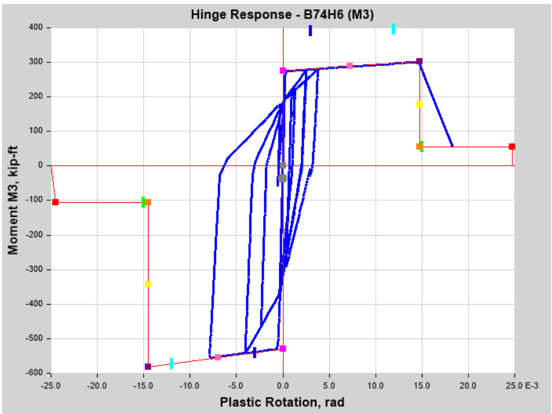


Fig. 9 – Plastic hinge performance in a typical 4th story column (left) and beam (right) when subjected to synthetic Big Bear earthquake

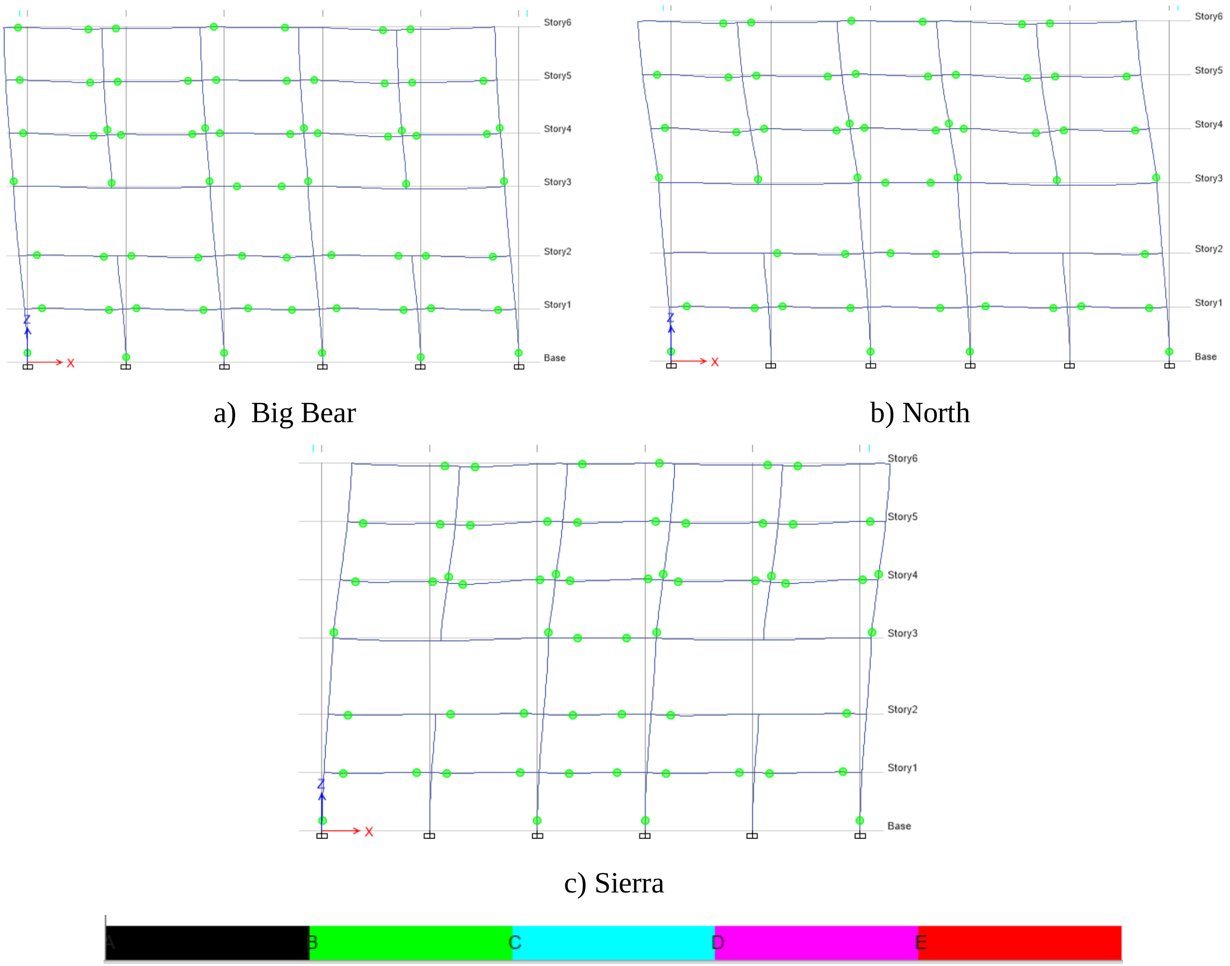
It is believed that the unacceptable performance of the building under all three synthetics earthquakes is due to not having the ACI 318-14 strong-column-weak-beam requirement met at the joints located at the mid-span of the transfer girder because of the practicality constrain of the moment frame.

4. Strengthened building and its seismic performance

To address the deficiency observed in the seismic performance of the building (see Section 3.4), the columns located at mid-span of the transfer girders (in the story above the transfer story) are strengthened by the over-strength factor ($\Omega = 3$), as it was done with the transfer girders and its supporting columns.

Although this resulted in using 3.2 times the area of longitudinal reinforcement in the strengthened columns (from 1.75% A_{gross} to 5.60% A_{gross}), the ACI 318-14 requirement for the strong-column-weak-beam was not satisfied; thus, nonlinear time-history analysis of the strengthened frame is conducted using three synthetic earthquakes to verify adequacy of the proposed strengthening procedure.

The seismic performance of the strengthened building is summarized in Figs. 10 where the plastic hinging patterns and their corresponding performance are illustrated. It is observed that the plastic hinge rotations in all the hinges, shown in "green" in Fig. 10, meet the Life Safety (LS) performance objective for all members.



a) Big Bear

b) North

c) Sierra

Fig. 10 – Plastic hinging patterns of the strengthened building subjected to three synthetic ground motions

To examine adequacy of the strengthened columns further, the moment-rotation plot of its bottom plastic hinge located at the mid-span of the transfer girder is shown in Fig. 11 when the frame is subjected to the synthetic North earthquake. It is observed that its low rotational ductility demand meets the intended objective of the ACI 318-14 strong-column-weak-beam requirement. Thus, the proposed strengthening method not only effectively addressed the concern related to this join but also prevented other plastic hinges in fourth story columns from having excessive rotational ductility demands (see Fig. 10).

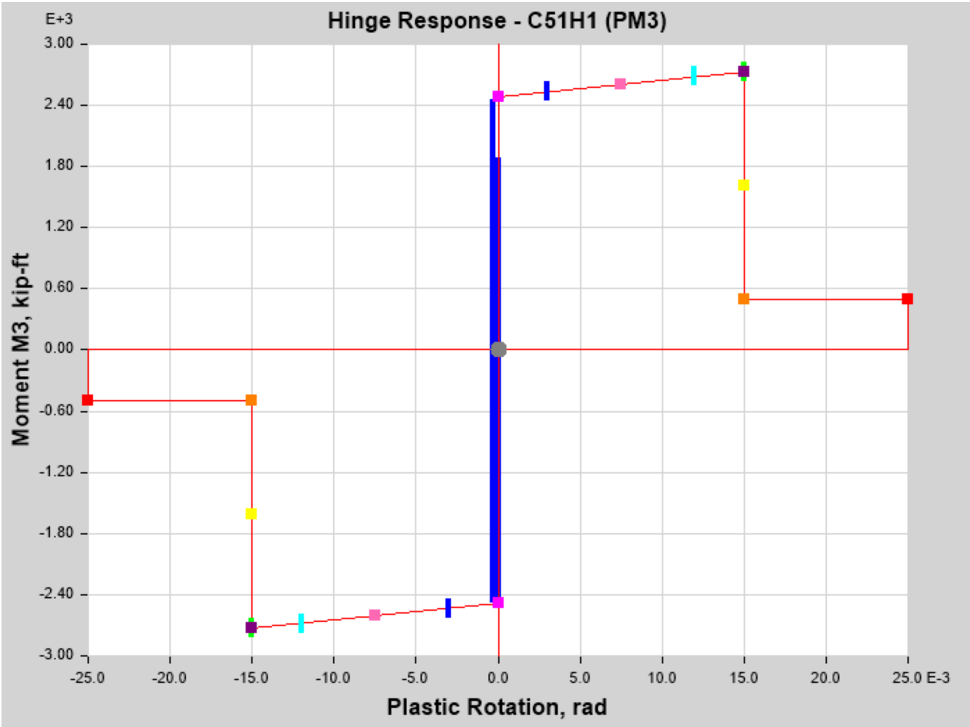


Fig. 11 - Plastic hinge performance in the strenghtened column when subjected to synthetic North earthquake

5. Summary and conclusion

This investigation involves evaluating nonlinear behavior of a six-story reinforced concrete building with a transfer story at its third level subjected to synthetic ground motions corresponding to Los Angeles, California, using ETABS software [2].

A special moment frame was designed following the ACI 318-14 [3] and ASCE 7-16 [1] standards, which classified the building as a Type 4 vertical irregular structure. Thus, special load combination requirement of ASCE 7-16 [1], which includes amplifying the seismic demands by the over-strength factor, is used to design the transfer girders and the supporting columns to avoid collapse mechanism [1].

However, the plastic hinging patterns and their behaviors obtained from nonlinear time-history analysis of the building subjected to synthetic ground motions showed a potential for a weak-story collapse mechanism in the story above the transfer girders. This was attributed to the fact that the strong-column weak-beam requirement of the ACI 318-14 [3] could not be satisfied in the joints located at the mid-span of the transfer girders (due to the practicality limit of using moment frames).

To address this concern, the columns located at mid-span of the transfer girders (in the story above the transfer story) are strengthened by the over-strength factor (as it was done with the transfer girders and its supporting columns). The result of nonlinear time-history analysis of the proposed strengthened building shows that rotational ductility demands of the column plastic hinges in the story above the transfer girders meet the life safety performance objectives [5]. Thus, the proposed strengthen procedure can be a practical solution for addressing the noted concern. However, since ETABS hysteresis models used in beam and column hinges overestimates hysteresis damping in the structure when conducting nonlinear time-history analysis by ignoring potential strength degradation and stiffness deterioration of these members under cyclic loads, further research is needed to verify the above conclusion using more accurate plastic hinge hysteresis models.

6. References

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