An Incremental Dynamic Analysis on RC Bridge Columns Designed for P-Delta Effects According to Caltrans SDC

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***Abstract*—Caltrans Seismic design criteria (SDC) provides guidelines for designing bridge columns considering the P-Delta effects in order to prevent destabilizing moments to become dominant. Caltrans SDC controls the P-Delta effects using a conservative limit for lateral displacement due to axial load, which is enforced by limiting the design target ductility demands on structural components. For columns with high P-Delta induced moments Caltrans requires the column to be analyzed using more advanced nonlinear time history analysis or the column should be redesigned to comply with Caltrans SDC guideline for ignoring the P-Delta effects. The intention of this research is to study the accuracy of the Caltrans SDC method in detecting the point which P-Delta effects can be ignored. When P-Delta effects can't be ignored engineering firms tend to redesign the columns (use bigger section sizes or increase the reinforcement ratio) in order to prevent performing more time consuming and computationally demanding nonlinear time history analysis or Incremental Dynamic Analysis (IDA) to justify the structural behavior with inclusion of the P-Delta effects. This research performs IDA on two similar columns which one of them comply with Caltrans Criterion and the other one fails to satisfy the maximum cap for P-Delta induced moment in order to ignore the P-Delta effects. This research provides suggestions on possible improvements for the Caltrans SDC criteria for ignoring the P-Delta effects.**

***Index Terms*—P-Delta effects, Nonlinear time history analysis, Incremental Dynamic Analysis (IDA), Fragility analysis, Caltrans SDC**

I. INTRODUCTION

P–Delta effects are the result of gravity loads acting through the structure's lateral displacement [1]. The lateral displacement will enlarge as the gravity loads acting on them, and increased gravity load enlarges the lateral displacement [2]. This cycle is against the stability of structure, and may cause collapse. The complexity of this problem increases as structure gets into inelastic deformation [3]. Although for smaller levels of nonlinearity, it is perfectly acceptable to neglect the P-Delta effect, for cases with high level of nonlinearity it is crucial to accurately capture the P-Delta effects[4],[5].

P-Delta effects have long been addressed in design by stipulating a reduction in both the shear capacity and initial stiffness of RC bridge columns [6],[7]. Reduction in the initial stiffness leads to an increase in the natural period of the system with a likely increase in the design displacement demand and conversely a decrease in the spectral acceleration. However, studies by Jennings and Husid [1] have also shown that depending on the profile of an earthquake response spectrum the reverse may actually occur when analyzing or testing a RC bridge column to ground motions. These conclusions highlight some of the needs in predicting within a reasonable degree of accuracy the seismic response of RC bridge columns under P-Delta effects. Since design codes are nowadays progressing towards a performance based design there is a further need to quantify the destabilizing effect of gravity loads and its effect on the response of bridge columns from within metrics of a performance based seismic response.

II. BACKGROUND

The Caltrans Seismic Design Criteria (SDC) [4] provides the minimum requirements for seismic design of ordinary bridges. These requirements ensure that the bridge will meet the performance goals of the design. Caltrans SDC controls the P-Delta effects using a conservative limit for lateral displacements due to axial load. This goal is met by limiting the ductility demands on structural components. According to the Caltrans SDC [4] if Eq (1) is satisfied, P-Delta effects can be ignored, and structural components can be designed based on predefined ductility demands.

$$P\,\Delta_r=0.2\,M_p \qquad (1)$$

Where Δ_r is the lateral offset between the point of contra-flexure and the base of the plastic hinge, and M_p is the idealized plastic moment capacity of a column

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calculated by M-φ analysis. In cases which Eq (1) is not satisfied it is required to perform a nonlinear time history analysis to study the P-Delta effects on the column. Incremental Dynamic Analysis (IDA) is one of the most sought after methods which incorporate nonlinear time history analysis to study instability of columns with consideration of the P-Delta effects[8].

IDA is a series of nonlinear dynamic analyses of a particular structure subjected to a set of ground motions of varying intensities. IDA intends to provide information on the performance of a structure at various stages, such as, elastic response, inelastic response, and collapse of the structure [9]. IDA curves are presented in terms of Damage Measures (DM) and Intensity Measures (IM). IDA requires a series of nonlinear-time history analyses at increasing intensity levels of the same ground motion. IM is used to express the intensity of the earthquake load. Commonly used IMs are first mode spectral acceleration, peak ground acceleration, and first mode period of vibration. DM is the maximum response or damage to the structure corresponding to the intensity measure, and typical DMs used are the maximum base shear, total acceleration, nodal displacement, inter-story drift, damage index[10].

Furthermore, IDA is one of the highly sought after method to determine global collapse capacity [11]. The global collapse capacity is defined as when the IDA curve becomes flat. This indicates that, a small increase in the IM (e.g., ground motion) causes a large increase in the DM (e.g., deformation of target node). Results obtained by IDA can be interpreted using collapse fragility curves. The collapse fragility curve corresponds to the cumulative distribution function of collapse intensities from individual records, and provides information on how the probability of collapse increases with increasing ground motion intensity.

III. Method

The intention of this research is to study the Caltrans SDC method in detecting the point which P-Delta effects can be ignored. In this research two similar columns are subjected to a study. Both columns have same geometry, height and have same axial load applied on top of them. Column I has 1% longitudinal reinforcement ratio and fails to satisfy the Caltrans SDC requirements to ignore the P-Delta effects. Caltrans SDC suggests performing nonlinear time history analysis to study the P-Delta effects, or changing the column properties until the Eq (1) is satisfied. Typically, designers to prevent performing time consuming, nonlinear time history analyses redesign the column. Column II has 2% longitudinal reinforcement ratio and satisfies the Caltrans requirements for ignoring the P-Delta effects. This study intends to compare the structural response obtained from these two columns by looking at their IDA curves.

A. Pushover Analysis on Two Similar Columns

In this research using OpenSees[12], nonlinear static analysis (Pushover) is performed to obtain the moment-curvature and subsequently the load-deformation for the columns (Fig. 1). Table I illustrates the column properties.

TABLE I. Column Properties

Concrete Strength, f'c (MPa, ksi)	27.5 (4)
yield Strength, fy (MPa, ksi)	413 (60.0)
Reinforcement ratio (Column I)	1%
Reinforcement ratio (Column II)	2%
Modulus of elasticity, Es (MPa, ksi)	2×10^5 (29,000)
Longitudinal reinforcing steel: yield strain, εy	0.0015
Column diameter, L (m, ft)	1.21 (4)
Column aspect ratio, CAR	8
Cover concrete (cm, in)	5 (2)
Axial load (kips)	389

Pushover analysis (Fig. 1) provides the lateral load and displacement at yielding and ultimate capacity of the column which is required to create the bilinear force-displacement graph. Displacement and load at design target ductility is obtained by linear interpolation between the yielding and the ultimate point.

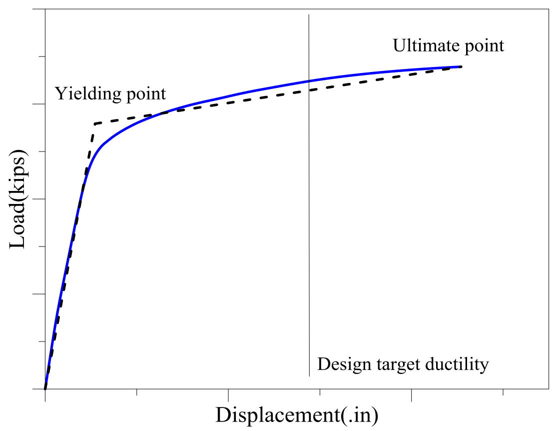

Figure 1. Load-Deformation obtained from pushover analysis.

B. Development of IDA Curves

The process to perform incremental dynamic analysis can be considered in to multiple steps. (1) Start the process with an initial scale factor for the ground motion. (2) Perform the nonlinear time history analysis. (3) Compute the corresponding intensity measure, such as, peak ground acceleration or spectral acceleration of the first mode, etc. (4) Compute the corresponding engineering demand parameter, such as, story drift, displacement of target nodes, etc. (5) Compare the engineering demand parameter with the predefined termination criteria. (6) If the termination criteria have not been met, increase the scale factor and return to step (2).

Adopting a dynamic algorithm for determination of the step size of ground motion scale factor can reduce the computation costs of development of IDA curves (Fig. 2). An efficient algorithm is developed to use bigger step sizes at less sensitive regions of the analysis, and reduces the step size as we get closer to the termination criteria.

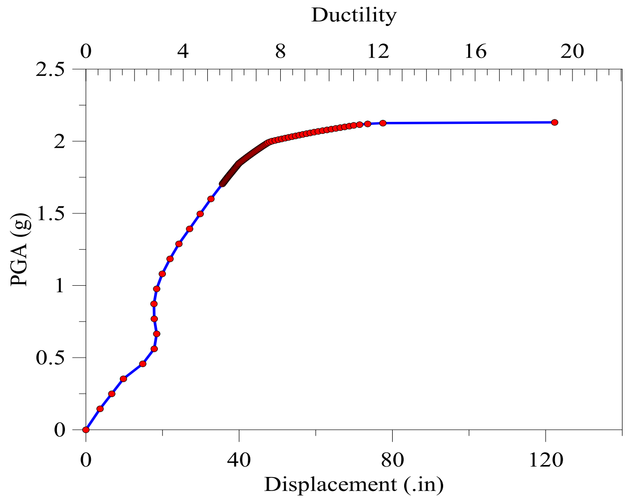

Figure 2. Single record IDA (Column spec: Col Height=48ft, Axial load=586kips, ρ=2%, EQID=120111).

Each point on the IDA curve corresponds to a nonlinear time History analysis. Fig. 3 depicts four different nonlinear time history analysis at different peak ground accelerations. It is evident that, as earthquakes intensity increases the column moves further into nonlinear range.

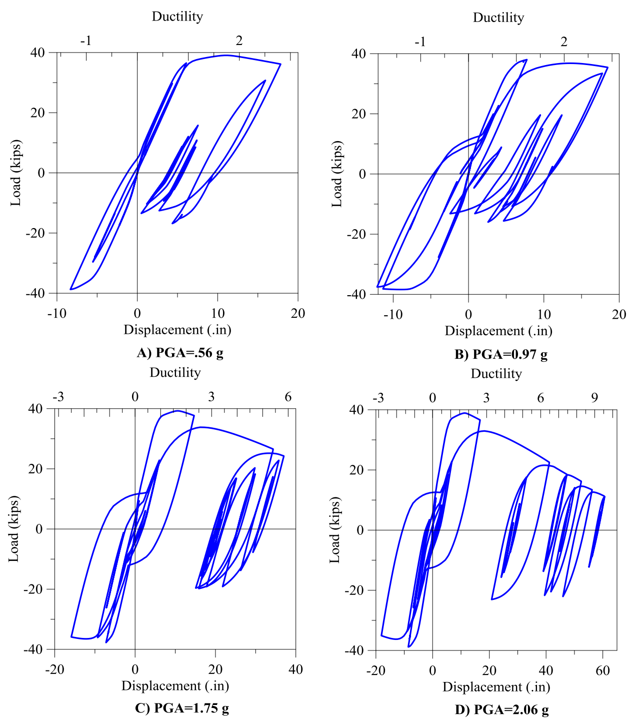

Figure 3. Nonlinear time-history analysis (Column spec: Col Height=48ft, Axial load=586kips, ρ=2%, EQID=120111).

C. Ground Motion Selection

Throughout this research ATC Far-Field, ground motion record set is used. The ground motion set is collected from Pacific Earthquake Engineering Research Center (PEER-NGA) database. Table II and Table III tabulates the characteristics of the ground motion set. Fig. 4 illustrates the response spectrum for the earthquake records (damping ratio of 5%). Following characteristics are common among all these ground motion records.

TABLE II. GROUND MOTION PROPERTIES

Distance R	R > 10 km
Large Magnitude Events	M > 6.5
Equal Weighting of Events	⩽ 2 records per event
Strong Ground Shaking	PGA > 0.2g /PGV > 15 cm/sec
Source Type	Both Strike-Slip and Thrust Fault Sources
Site Conditions	Rock or Stiff Soil Sites Vs > 180 m/s
Record Quality	Lowest Useable Frequency < 0.25 Hz

Far-Field earthquake record set specifications are tabulated in Table III.

TABLE III. GROUND MOTIONS RECORDS

EQ ID	Earthquake			PGA_{max} (g)	PGV_{max} (cm/s.)
	M	Year	Name		
12011	6.7	1994	Northridge	0.52	63
12012	6.7	1994	Northridge	0.48	45
12041	7.1	1999	Duzce, Turkey	0.82	62
12052	7.1	1999	Hector Mine	0.34	42
12061	6.5	1979	Imperial Valley	0.35	33
12062	6.5	1979	Imperial Valley	0.38	42
12071	6.9	1995	Kobe, Japan	0.51	37
12072	6.9	1995	Kobe, Japan	0.24	38
12081	7.5	1999	Kocaeli, Turkey	0.36	59
12082	7.5	1999	Kocaeli, Turkey	0.22	40
12091	7.3	1992	Landers	0.24	52
12092	7.3	1992	Landers	0.42	42
12101	6.9	1989	Loma Prieta	0.53	35
12102	6.9	1989	Loma Prieta	0.56	45
12111	7.4	1990	Manjil, Iran	0.51	54
12121	6.5	1987	Superstition Hills	0.36	46
12122	6.5	1987	Superstition Hills	0.45	36
12132	7.0	1992	Cape Mendocino	0.55	44
12141	7.6	1999	Chi-Chi, Taiwan	0.44	115
12142	7.6	1999	Chi-Chi, Taiwan	0.51	39
12151	6.6	1971	San Fernando	0.21	19
12171	6.5	1976	Friuli, Italy	0.35	31

SA-T response spectrum for earthquake records are shown in Fig. 4.

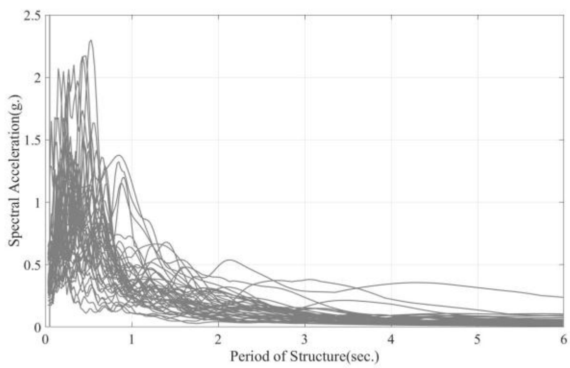

Figure 4. Far-Field record set response spectrum.

IV. RESULTS

According to Caltrans SDC if the ratio of bending moment induced by P-Delta effects to the yielding moment capacity of column is less the twenty percent, then structural components shall be designed based on predefined displacement ductility demands. Table IV presents the results obtained from pushover analysis, and checks the Caltrans SDC criteria for ignoring the P-Delta effects.

TABLE IV. INVESTIGATED COLUMNS

	Unit	Column I	Column II
Yielding displacement	in	3.51	5.05
Yielding load	kips	74.62	126.31
Yielding Moment	Kip.in	26253	45670
Ultimate Load	in	84.71	154.13
Ultimate displacement	in	78.23	63.17
Ultimate Ductility	N/A	22.29	12.51
Load at ductility 4	kips	76.04	133.57
P-Delta induced bending moment	Kip.in	5466	7864
$\frac{P\,\Delta_r}{M_p^{col}}$		0.21	0.17
Ignore P-Delta		NOT-OK	OK

Fig. 5 and Fig. 6 show the result obtained from pushover analysis for column I and column II.

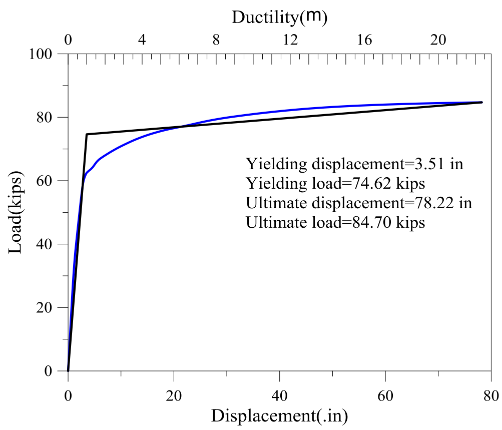

Figure 5. Pushover analysis for Column I.

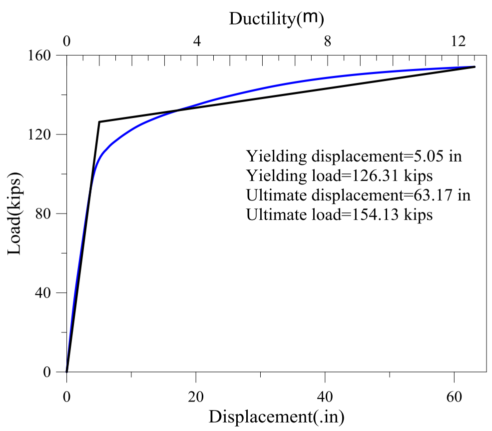

Figure 6. Pushover analysis for Column II.

A. *Incremental Dynamic Analysis*

Incremental dynamic analysis using the selected ground motion records was performed on Columns I and II. Fig. 7 and Fig. 8 show the obtained IDA curves. Displacement ductility is used to represent the engineering demand parameter, and Peak Ground Acceleration (PGA) is used as the intensity measure.

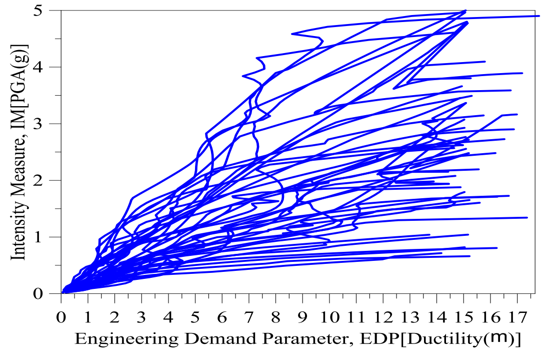

Figure 7. IDA curves for Column I.

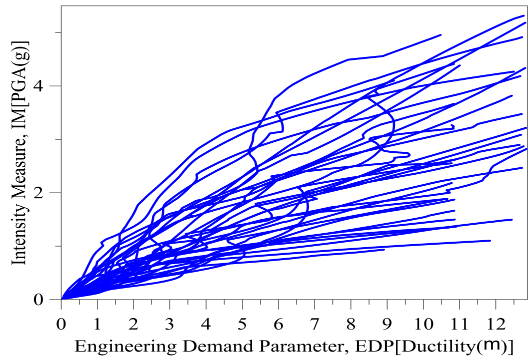

Figure 8. IDA curves for Column II.

The collapse fragility curve corresponding to the cumulative distribution function of collapse ductility provides information on how the probability of collapse increases with increasing the maximum displacement ductility experienced by the column. For single column bents supported on fixed foundation Caltrans recommends design target ductility of four which is used in this research as design target ductility. Fig. 9 and Fig. 10 show the cumulative distribution function for the collapse of column I and column II. The best curve fitted to points corresponding to instability obtained from IDA is included in Fig. 9 and Fig. 10.

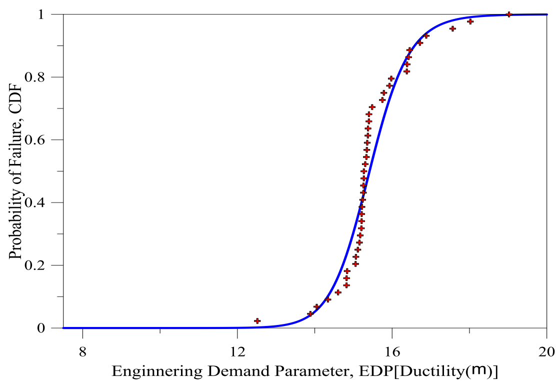

Figure 9. Fragility curves for Column I.

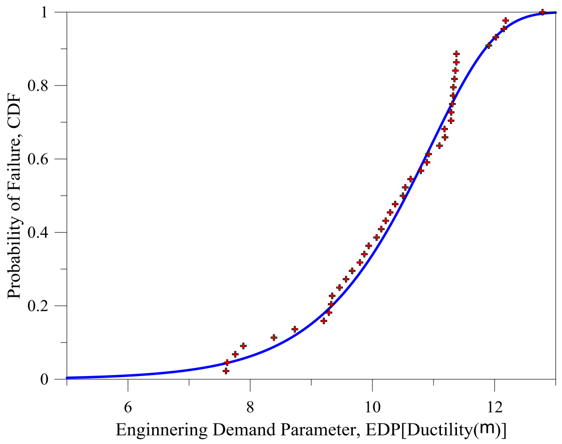

Figure 10. Fragility curves for Column II.

FEMA P695[13] suggests limiting the probability of collapse to 10 % or less as performance target under maximum considered earthquake (MCE). Considering target ductility of four both column I and II have probability of failure less than 1 % which is acceptable.

B. *Nonlinear Time-History Analysis at Target Ductility*

The result for nonlinear time-history analysis at target ductility of 4, for earthquake records 120111, 120121, 120411, and 120521 are presented below.

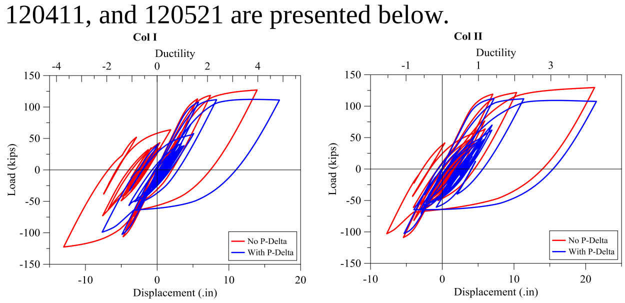

Figure 11. Nonlinear time-history analysis at target ductility of 4 (EQID=120111).

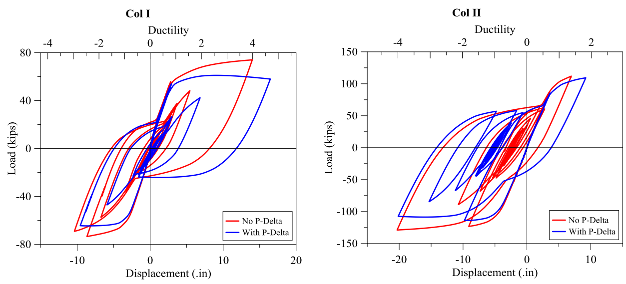

Figure 12. Nonlinear time-history analysis at target ductility of 4 (EQID=120121).

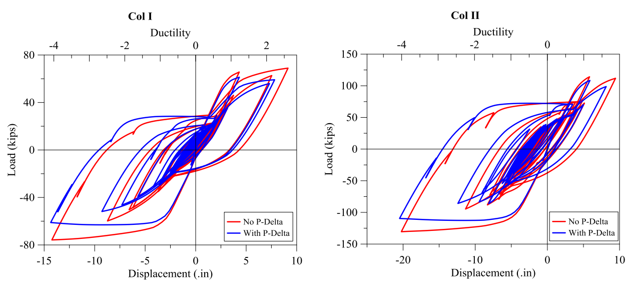

Figure 13. Nonlinear time-history analysis at target ductility of 4 (EQID=120411).

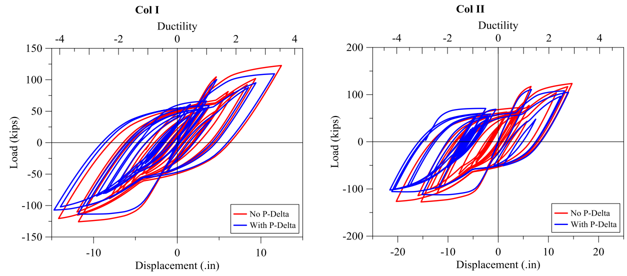

Figure 14. Nonlinear time-history analysis at target ductility of 4 (EQID=120521).

Results obtained from nonlinear time-history analysis for ground motions scaled such that the column reaches to ductility level four ignoring the P-Delta effects indicates that in none of the cases any sign of instability was seen (Fig. 11, and Fig. 12). Although in some cases Col I experienced higher ductility levels (Fig. 13, and Fig. 14) than the design target ductility with inclusion of P-Delta effects, but it never created stability issues.

V. Conclusions

The P-Delta effect under seismic loading is highly influenced by randomness of the ground motion characteristics, and only can be fully captured using nonlinear time-history analysis.

Caltrans SDC encourages designers to use higher longitudinal reinforcement ratio or bigger section sizes in order to prevent performing nonlinear time-history analysis. The case study in this research showed that the section with higher reinforcement ratio necessarily does not have a better seismic response. In many cases the original column also is sufficient for collapse prevention purpose.

Methods currently used for designing columns for P-Delta effects do not explicitly address the P-Delta effects[14]. Codes usually tend to control the P-Delta effects by imposing conservative limits on drift or stability index. For columns with column height to diameter ratios greater than ten it is suggested to incorporate nonlinear time history analysis with multiple earthquake records to investigate the P-Delta effects. Caltrans Criteria for ignoring the P-Delta effect is sensitive to the ratio of moment induced by P-Delta effects to the idealized plastic moment. An improved method also should be sensitive to Column height to diameter ratio and axial load on top as independent variables.

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