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An Investigation on Reliability of RC Bridge Columns Designed for Displacement Ductility Proposed by the Caltrans SDC

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Abstract

P–Delta effects are the result of gravity loads acting through the structure's lateral displacement. Typically, whenever columns have small lateral displacement, it is acceptable to neglect the P-Delta effect. Still, as columns experience high nonlinearity levels, it is crucial to capture the P-Delta effects accurately. It is of main importance to reliability detect the safe threshold for ignoring the P-Delta effects. Caltrans SDC provides a procedure that can evaluate whether P-Delta effects can be neglected in design. Whenever requirements are satisfied, predefined displacement ductility is used to design structural elements. This research intends to study the reliability of the threshold proposed by the Caltrans SDC for ignoring the P-Delta effects. Nonlinear time history analysis is used to compare the maximum reached displacement ductility with and without P-Delta effects.

Keywords - *P-Delta effects, nonlinear time-history analysis, Incremental Dynamic Analysis, Threshold for neglecting P-Delta effects, Caltrans SDC*

I. INTRODUCTION

P–Delta effects result from 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],[3],[4]. P-Delta effects can have a detrimental impact on the seismic response of bridges because of a reduction in both the shear capacity and initial stiffness of RC bridge columns [5],[6],[7]. The reduction in the initial stiffness imposes an increase in the system's natural period and a likely surge in the design displacement demand. According to the American Concrete Institute (ACI 318-14) [8], P-Delta effects can be mitigated by conducting a second-order elastic analysis with reduced stiffness values, or as an alternative, first-order elastic analysis with increased design moments using moment magnifiers (ACI 318-14) shall be used to compensate for P-Delta effects. Caltrans SDC [9] provides a methodology to determine whether P-Delta effects can be ignored in design. A main concern regarding the P-Delta effects is the threshold of safely ignoring P-Delta effects [10],[11]. This research intends to incorporate Incremental Dynamic Analysis (IDA) to determine displacement ductility at which P-Delta effects can safely be ignored. IDA performs a series of nonlinear dynamic analyses of a structure subjected to a set of ground motions of varying intensities. It provides information on the structure's performance at various stages, such as, elastic response, inelastic response collapse of the structure [12].

II. BACKGROUND

P–Delta effects result from gravity loads acting through the structure's lateral displacement [5].[7]. The lateral displacement caused due to earthquakes, wind, or blast explosions will enlarge as the gravity loads acting on them, and the increased gravity load enlarges the lateral displacement. This cycle is against the stability of the structure and may cause collapse. The complexity of this problem increases as the structure gets into inelastic deformation. Although for smaller nonlinearity levels, it is perfectly acceptable to neglect the P-Delta effect [6]. It is necessary to model P-Delta effects as accurately as possible for cases with a high level of nonlinearity and near collapse circumstances. Caltrans SDC [9] provides a procedure that can be used to evaluate whether P-Delta effects can be ignored in design. In design circumstances, not considering P-Delta effects, structural components can be designed based on predefined ductility demands. This study intends to find the range in which Eq.1 is valid. If Eq.1 is not satisfied, increasing the section size or reinforcement ratio can be used to increase the yielding moment capacity of the column. However, Caltrans SDC recommends performing nonlinear time-history analysis to verify that the column can resist the P-Delta effects.

$$P \times \Delta_r = 0.2 \times M_p^{col} \qquad \text{Eq.1}$$

The lateral offset between the point of contra-flexure and the base of the plastic hinge is the idealized plastic moment capacity of a column calculated by M-φ analysis. If Eq.1 is satisfied, predefined ductility demands limit the design of structural components. According to Caltrans, SDC target displacement ductility for single-column and multi-column bents are as follows.

Single Column Bents supported on a fixed foundation $\mu_D \leq 4$

Multi-Column Bents supported on fixed or pinned footings	$\mu_D \leq 5$

Reliability is defined as the probability that a system will perform its intended function for a specific period under a given set of conditions [12]. In the context of structural engineering, failure has been defined using limit states. A limit state is a boundary between the desired and undesired performance of a structure [13]. Limit states are usually categorized as ultimate limit states, serviceability limit states. Ultimate limit states (ULSs) are mostly related to the loss of load-carrying capacities, such as forming a plastic hinge, crushing concrete in compression, shear failure of the web in a steel beam, and loss of the overall stability, and buckling of the flange. Serviceability limit states (SLSs) may or may not directly relate to structural failure, and these criteria are established to ensure the user's comfort or control the maintenance costs [12]. For RC bridge columns, the onset of cover concrete crushing or the development of crack widths of a size that require injection grouting after an earthquake is an appropriate serviceability limit. The first criterion can be related to extreme compression fiber concrete strain, while the second relates to strain in the tension reinforcing bars at maximum distance from the neutral axis [14]. The 'Ultimate' conditions may be taken as corresponding to a 'damage control' limit state beyond which structural repair is not economically feasible or a true 'collapse' limit state [14]. The substitute Structure approach is a common method used for displacement-based design, which models an inelastic system as an equivalent elastic system [15].

III.METHOD

This research intends to study the Caltrans SDC criterion for ignoring the P-Delta effects. This goal is achieved by performing nonlinear time history analysis on RC bridge columns with and without P-Delta effects. The maximum ductility ratio achieved without P-Delta effects to the maximum ductility achieved with P-Delta effects was the measure to compare the significance of the P-Delta effects. The earthquake records are amplified. The columns reach ductility level four; the Caltrans SDC suggests the design target ductility for single bent columns supported on a fixed foundation. The algorithm behind the research procedure is as depicted in Figure 1.

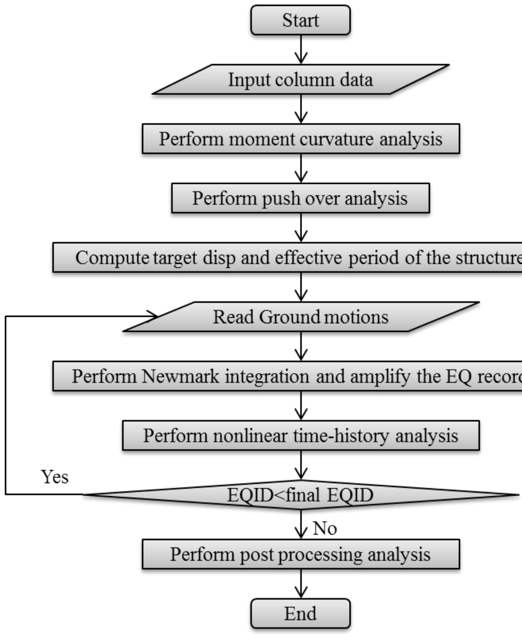


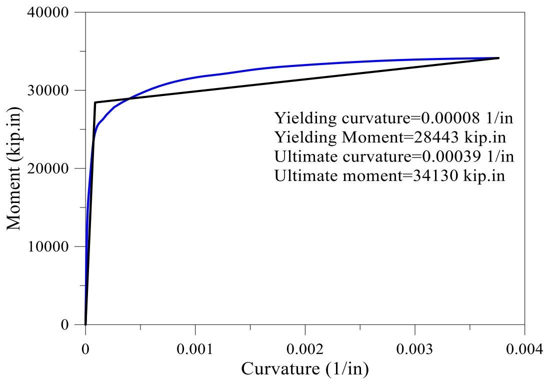
Fig. 1: The Research Method Flowchart

Table 1 illustrates the properties of all the columns subjected to this study. 180 different columns ranging between 389 to 1168 kips applied axial load, with column height to diameter ratio between 4 to 12 have been subjected to nonlinear time history analysis to study the effects of the P-Delta effects. The ratio of FL/ PD obtained using pushover analysis is considered the measure for the intensity of the P-Delta effects. As this ratio gets bigger, the P-Delta induced moment becomes less significant.

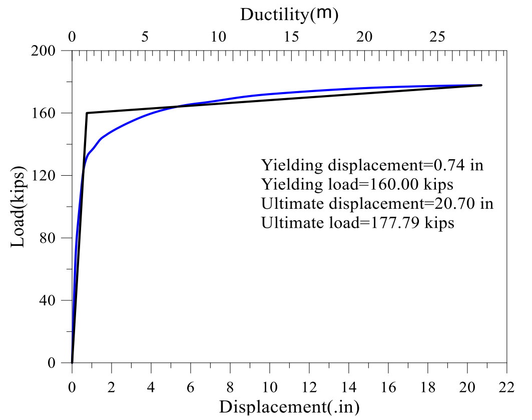
TABLE I. Column Properties

Concrete Strength, f'c (ksi)	**5.38**	**Column diameter, L (ft)**	**4**
yield Strength, fy (ksi)	**60.0**	**Column height ratio, CHR**	**4 to 12**
Reinforcement ratio	**1%**	**Cover concrete (in)**	**2**
Modulus of elasticity, Es (ksi)	**29,000**	**Axial load (kips)**	**389, 584, 778,973,1168**

Further description is provided through a numerical example. Moment curvature and pushover analysis for the sample column is presented in Figure. 2. Pushover analysis provides the load and displacement at yielding and the ultimate capacity of the column



(a)



(b)

Fig.2: Moment-curvature and pushover analysis (Col spec: Col Height=16 ft., Axial load=389kips, ρL=1%)

The coefficient X is defined as the yielding moment over the P-Delta induced moment.

$$X = \frac{F_{y0}.L}{P.\Delta_y} = \frac{160.00 \times 192}{389 \times 0.74} = 106.71 \qquad \text{Eq.2}$$

Caltrans SDC suggests design target ductility of four for single-column bent supported on a fixed foundation. Eq.3 is used to compute the effective stiffness at target ductility. The bilinear idealization of the Force-displacement analysis is shown in Figure 2(b).

The lateral load at the target ductility can be obtained using Eq.3.

$$F_{target} = F_y + \frac{F_u - F_y}{\Delta_u - \Delta_v}(\Delta_y(\mu - 1)) = 161.97 kips \qquad \text{Eq.3}$$

Effective stiffness at target ductility can be obtained using Eq.4.

$$K_{eff} = \frac{F_{target}}{\mu\, \Delta_y} = 54.71\ kips/inch \qquad \text{Eq.4}$$

The effective period of structure can be obtained using

$$T_{eff} = 2\pi\sqrt{\frac{W}{g\, K_{eff}}} = 0.87\ sec \qquad \text{Eq.5}$$

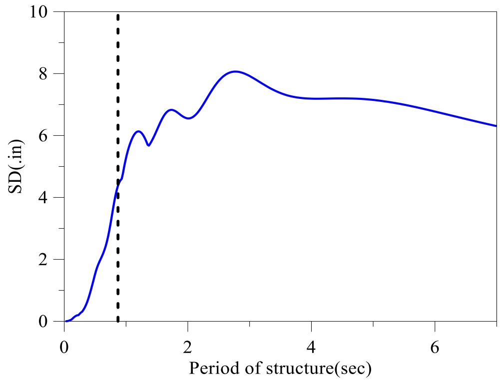
Developing the response spectrum using Newmark integration needs to compute the effective damping ratio at the design target ductility. Expression, proposed by Chopra and Goel (1999), $\hat{\zeta_{eq}} = \zeta + \zeta_{eq}$ was used for the total viscous damping ratio.

$$\zeta_{eq} = \frac{1}{\pi}(1 - \frac{0.95}{\sqrt{\mu}} - 0.05\sqrt{\mu}) \qquad \text{Eq.6}$$

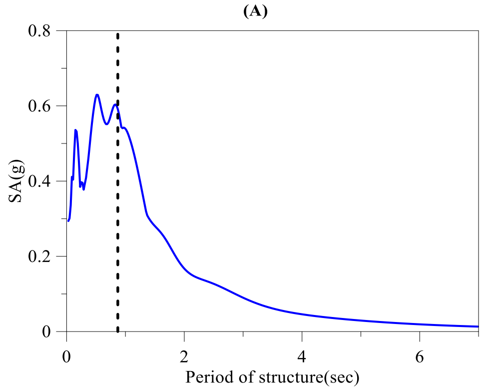
$\mu = 4, \zeta_{eq} = 0.185$

Where □ are the target ductility level, and ζ is the elastic, viscous damping and is equivalent viscous damping?

Newmark integration method is used to develop spectral displacement and spectral acceleration versus the period of the structure. Figure 3(A) shows the spectral displacement versus the earthquake record structure period with EQID=120111 (look Table3) with an 18.5% damping ratio. Figure 3(B) shows the spectral acceleration versus the period of the structure. For a structure with a period of 0.87 sec, the spectral displacement is 4.38 inches.



(A)



(B)

Fig.3: Response spectrum (EQID=120111, ζ_eq=0.185)

The earthquake scale factor is the spectral displacement ratio computed in step 4 to the spectral displacement for the non-scaled earthquake record (at the effective period) computed at step 6.

$$scalefactor = \frac{SD_{capacity}}{SD_{demand}} = \frac{2.96}{4.38} = 0.67 \qquad \text{Eq.7}$$

The amplified earthquake record is applied to the structure. The nonlinear time-history response of the column is presented in Figure 4.

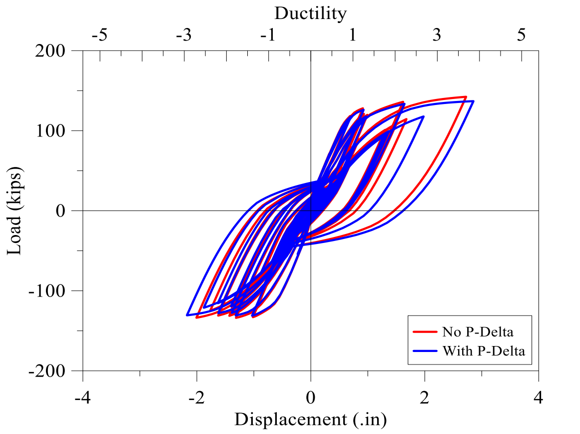


Fig.4. dynamic inelastic time history analysis (EQID=120111)

A. *Earthquake record selection*

Throughout this research, ATC Far-Field, a ground motion recordset, is used. The ground motion set is collected from the Pacific Earthquake Engineering Research Center (PEER-NGA) database. Table II and Table III tabulate 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 III: 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 3.

TABLE IIIII: Ground motion records

EQ ID	Earthquake		PGA (g)	EQ ID	Earthquake		PGA (g)
	Year	Name			Year	Name	
12011	94	Northridge	0.52	12092	92	Landers	0.42
12012	94	Northridge	0.48	12101	89	Loma Prieta	0.53
12041	99	Duzce, Turkey	0.82	12102	89	Loma Prieta	0.56
12052	99	Hector Mine	0.34	12111	90	Manjil, Iran	0.51
12061	79	Imperial Valley	0.35	12121	87	Superstition Hills	0.36
12062	79	Imperial Valley	0.38	12122	87	Superstition Hills	0.45
12071	95	Kobe, Japan	0.51	12132	92	Cape Mendocino	0.55
12072	95	Kobe, Japan	0.24	12141	99	Chi-Chi, Taiwan	0.44
12081	99	Kocaeli, Turkey	0.36	12142	99	Chi-Chi, Taiwan	0.51
12082	99	Kocaeli, Turkey	0.22	12151	71	San Fernando	0.21
12091	92	Landers	0.24	12171	76	Friuli, Italy	0.35

B. *Finite element model*

Throughout this research, nonlinear pushover, and time history analyses were performed using the open-source, object-oriented nonlinear structural analysis program, Open System for Earthquake Engineering Simulation (OpenSees) [16].OpenSees has been successfully used by other researchers in investigating the nonlinear load-deformation response of RC bridge columns. The circular cross-section was represented by a fiber-based model originally developed by Taucer et al. [17]and implemented in OpenSees by Scott and Fenves [18]. The cross-section was subdivided into fibers and assigned uniaxial stress-strain laws available in OpenSees to describe the cover and core concrete's response.

IV. RESULTS

For each of the 180 columns subjected to this study, the nonlinear time-history analysis was performed with a properly scaled set of ground motions (Table 2). Earthquake records were amplified such that the column reaches ductility level four.

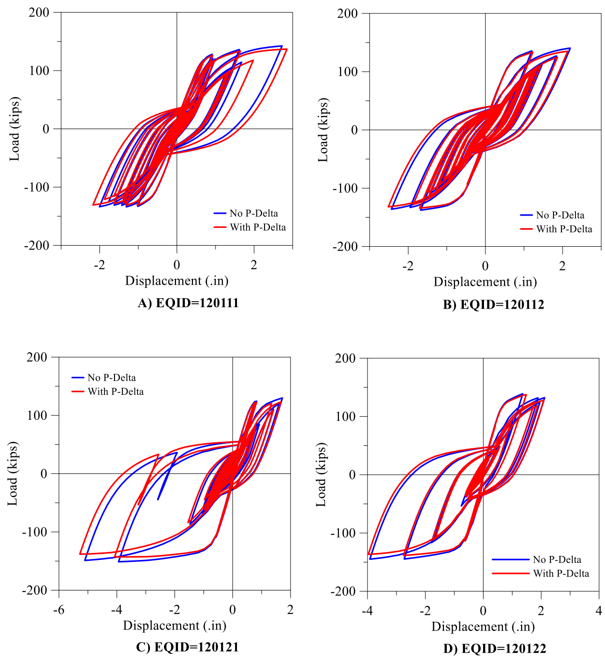


Fig.5. Nonlinear time-history analysis

Figure 6**Fi** shows the ratio of the obtained ductility level with the inclusion of the P-Delta effects over the obtained ductility level when the P-Delta effects are ignored. Since this column has a very high FL/ PD, it is expected that the inclusion of the P-Delta effects has minimalistic effects, as shown in Figure 6.

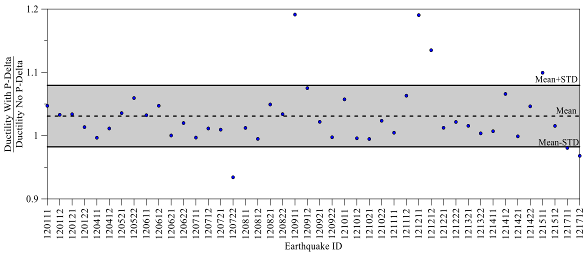


Fig 6. obtained ductility ratio for each earthquake (Col spec: Col Height=16 ft., Axial load=389kips, ρL=1%)

V. CONCLUSION

This analysis was performed on 180 different columns with 44 earthquake records for each column. As FL/ PD increases, the significance of the P-Delta effects is reduced, and the ductility ratio with P-Delta effects to ductility without P-Delta effects converges to one. Columns with a higher FL/ PD ratio are considered as safe for ignoring the P-Delta effects. For columns with smaller FL/ PD values, the standard deviation is significantly higher than columns with larger FL/ PD. The rationale behind this difference is that columns with smaller FL/ PD are more susceptible to P-Delta effects. Whenever the inclusion of the P-Delta effects causes the column to collapse, the ductility ratio with P-Delta over ductility without P-Delta surges creates a high standard deviation. On the other hand, columns high FL/ PD ratio are not susceptible to collapse due to gravity loads acting through the lateral displacement and have a very small standard deviation.

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