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Effect of confining pressure on horizontal load carrying capacity of circular RC Bridge pier

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Abstract— *An analytical model is used to find the effect of axial load and confining reinforcement on horizontal load carrying capacity of circular RC bridge pier. This model used the confining pressure generated by axial load and confinement reinforcement, and the resulting improvements in strength and ductility of column. A large No. of parameters including poorly confined and well-confined concrete was analysed and the comparisons include the effect of confining pressure on horizontal load carrying capacity of column.*

Keywords- *Circular column; Axial load; Confining pressure; Strength; Ductility;*

I. INTRODUCTION

Efficient seismic design of bridge piers required adequate section capacity in strength and deformation at critical section, especially in the case of monolithic construction, where piers should transfer not only gravity, but also horizontal forces from the superstructure to the foundations.

The basic philosophy for confining concrete in earthquake resistant design is to increase in the strength of the core due to confinement must be sufficient to offset the loss of the unconfined cover, and the confinement should enable the column to sustain large deformations without loss of the load carrying capacity.

In purely concentric column subjected to uniaxial loads, the maximum capacities of both the confined core and the plain unconfined concrete are reached at about the same axial strain. However, if the confined core attains its maximum resistance at strain beyond those at which the cover concrete has reached its ultimate, there may be a drop in the load carrying capacity of the column. For columns subjected to eccentric loading, the section capacity is depend on postpeak, moment-curvature behavior under the amount of axial load in both cases passive confinement mechanism is based on the activation of transverse reinforcement, which restrains the physical lateral expansion of concrete, induced by compressive loading. The ensuring triaxial stress state in the confined material finally leads to a significant increase in the overall strength and deformation capacity of the structural element.

The purpose of the present paper is to report the latest results of a parametric study of confining pressure generated by the axial load and confinement reinforcement on horizontal load carrying capacity of circular column.

II. DEFINITION AND MODELING OF DEFORMATION MECHANISMD IN RC MEMBERS

For adequate seismic performance, strength and deformation capacities of a structure must be greater than the demands imposed by a design earthquake. Performance evaluation of a structure is done using several methods, for example, linear static methods specified in most of the design codes, or lately using more involved non-linear methods (i.e. Static pushover analysis, Time history analysis) Time History Analysis required more complex input quantities and highly time consuming and cumbersome if used for all structures for example, cyclic load-deformation behavior of structural element. Therefore, a simpler and effective option for most of the structure is to use approximate procedures of performance evaluation of structures, such as nonlinear static pushover analysis.

Static pushover analysis is a powerful tool to predict the lateral response of structures by considering non-linearity in material and geometry (P-Δ effects). This procedure is generally considered to be more realistic in evaluating seismic vulnerability of new or existing structures than the linear procedure. The procedure of the pushover analysis involves subjecting a structure to a monotonically increasing the prescribed lateral force or displacement which would be experience when structure subjected to ground motion. Under incrementally increasing load or displacement various structural elements would yield, consequently, at each increment, the structure experiences a lost in stiffness. In the present study, SAP2000 Advanced 14 (CSI 2009) is used for displacement-controlled pushover analysis of structure. Base shear at the base of structure plotted against corresponding displacement at the top of pier is known as Pushover Curve.

2.1. Material Modeling

In the implementation of the pushover analysis, modeling is one of the most important steps. It requires the determination of the non-linear properties of each component in structures, quantified by strength and deformation capacities, which depends upon the modeling assumptions. Stress- Strain model of confined concrete developed by Mander et. al. (1988) and stress-strain curve for the reinforcing steel developed by Park et al. (1982) as shown in Figure1.

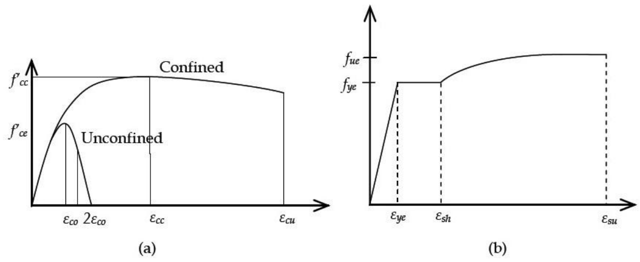

Figure 1. Stress-strain model for (a) Concrete (b) Reinforcing Steel used in the Pushover Analysis by SAP 2000 [CSI 2009]

The initial ascending curve is represented by same expression for both confined and un-confined concrete since the confining steel has no effect in this range. As the curve approaches the compressive strength of un-confined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the spalling strain ε_{sp} which is 0.005. The confining concrete model continues to ascend until the confined compressive strength f'_{cc} is reached. The ultimate compressive strain ε_{cu} is defined as the point where strain energy equilibrium is reached between concrete and the confining steel. The model is developed assuming the concrete columns under uniaxial compressive loading and confined by transverse reinforcement. The model also accounts for cyclic loading and the effect of strain rate.

The reinforcing steel is modeled with stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain. The length of yield plateau is a function of the steel strength and bar size. The strain hardening curve is modeled as non-linear relationship and terminates at the ultimate tensile strain, ε_{su}.

Plastic hinge length L_p is used to obtain ultimate rotation values from ultimate curvatures. Simplest form of plastic hinge length is obtained by following expression developed by the Paulay and Priestley in 1992:

$$L_p = 0.08L + 0.022\, f_{ye}\, d_{bl} \geq 0.044\, f_{ye}\, d_{bl}$$

Where, H is the section depth, L is the distance from the critical section of the plastic hinge to the point of contraflexure, and f_{ye} and d_{bl} are the expected yield strength, and diameter of longitudinal reinforcement, respectively. The plastic hinges are assumed to be form at a distance $L_p/2$ from the support.

2.2. Plastic Hinge Properties in Members

In SAP2000 (CSI 2009), non-linearity in members is not distributed along their whole length; instead, lumped plasticity is to be modeled at desired location on structural members. A two dimensional cantilever model is created in SAP2000 (CSI 2009) to carry out non-linear static analysis. RC pier is modeled as non-linear element with lumped plasticity by defining plastic hinge at fixed support shown in Figure 2. Non-linear material properties of all the structural members are require for specifying properties for plastic hinges in pushover analysis.

In RC piers, plastic hinges that generally develop are those corresponding to axial force– bending moment (P-M hinges), bending moment–bending rotation (M-θ hinges), and shear force-shear deformation (V-Δ). Typical P-M, V-Δ, and M- θ hinge properties for RC pier are shown in Figure 3.

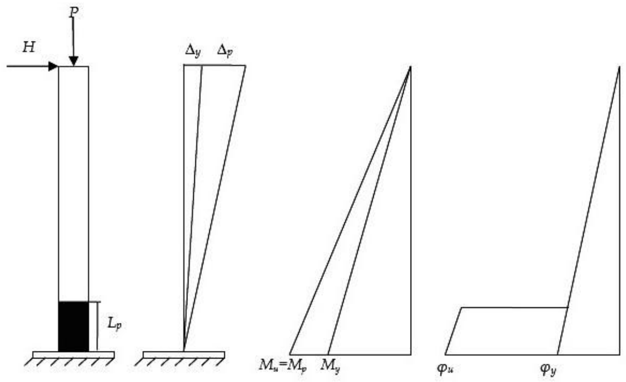

Figure 2. Lumped plasticity idealization of a cantilever and analysis model

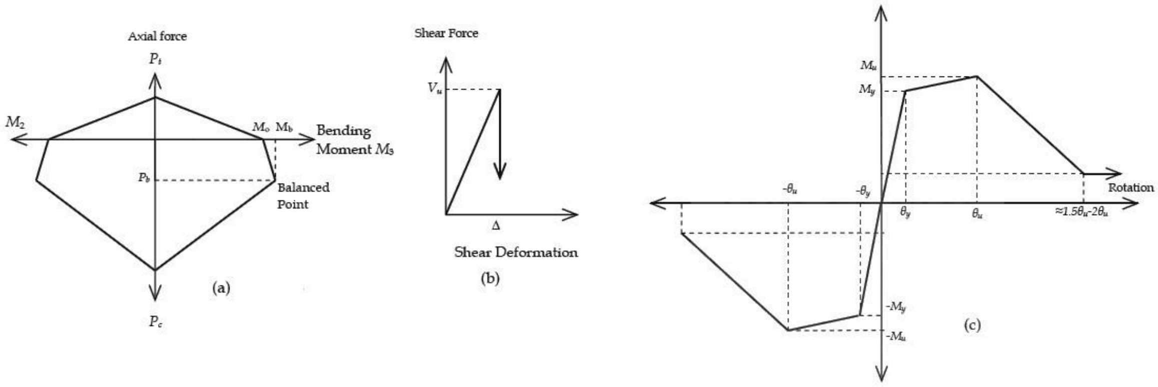

Figure 3. Typical plastic hinge propertied assigned to RC members (a) P-M (b) V-Δ, and (c) M-θ

In this study, Caltrans flexural hinge are used. The M-θ relationship for the designed sections is obtained using the moment-curvature (M-φ) relationship. The ultimate curvature φ_u at the failure limit state is defined as the concrete strain, or the confinement reinforcing steel reaching the ultimate strain. The displacement capacity Δ_{cap} of a member is on its rotation capacity, which in turn is based on its curvature capacity φ_u. The curvature capacity is determined by M-φ analysis. As per Caltrans, the plastic rotation θ_p is obtained by following Eq.:

$$\theta_p = L_p(\varphi_u - \varphi_{iy})$$

Where, φ_u and φ_{iy} are the ultimate curvature and idealized yield curvature, respectively.

The yield deflection Δ_y and plastic deflection Δ_p is obtained using Eqs.:

$$\Delta_y = \varphi_{iy} L^2/3$$

$$\Delta_p = \theta_p(L - L_p/2)$$

Where, L is the length of the member.

The total deflection capacity Δ_{cap} of section is obtained using Eq.:

$$\Delta_{cap} = \Delta_y + \Delta_p$$

The lateral load capacity obtained using M-θ relationship; it is given by following expression:

Lateral Load Capacity $= M_p/L$

Where, M_p is the plastic moment of the section obtained using the M-θ relationship.

The lateral load capacity (M_p/L) should be less than the shear strength V_{cap} to avoid brittle shear failure. Shear strength of the RC members were calculated using the IS 456:2000. If shear strength V_{cap} exceeds the lateral load capacity (M_p/L), then the brittle shear failure will occur, and shear hinge will be developed in the sections. Thus for no shear failure following condition should be satisfied:

$$M_p/L < V_{cap}$$

Shear failure of the members should be taken into consideration by assigning shear hinges in RC piers. Shear hinge properties are defined in such a way that when shear force in member reaches its capacity, the member fails immediately.

III. ANALYSIS PROCEDURE

Load patterns have been defined as dead load or live load, etc., and then load cases corresponding to non-linear static analysis were defined. Firstly, the Gravity Load Case is defined, which corresponds to the gravity load as well as other permanent loads acting on the structure. Secondly, in the Final Pushover Case, the stiffness of the members of structures at the end of non-linear Gravity Load Case has been considered as initial condition. More than one pushover cases are run in the same analysis. Pushover analysis cases can either be force controlled, i.e., structure is pushed at certain defined force level, or they can be displacement controlled, i.e., structure is pushed to a certain target specified displacement. In this study, Gravity Load Case is force controlled and Final Pushover Case is displacement controlled, same is used in the present study.

Analysis model is run after necessary inputs, such as material properties, plastic hinge properties are given. SAP2000 (CSI 2009) allows increasing the maximum number of steps by modifying the non-linear parameters for the analysis. There are three methods of hinge unloading, namely, unload entire structure, apply local distribution, and restart secant stiffness. Any of three methods can complete analysis which is based on the trial and error. Unload entire structure method is used for hinge unloading to complete the analysis.

IV. PARAMETRIC STUDY AND RESULTS

Attempt has been made to study the effect of confining pressure using nonlinear moment curvature analysis by an iterative process for circular column with concrete strength ranging from 40 Mpa to 70 Mpa, Axial load level ranging from 10% to 30 % of concrete crushing strength, longitudinal reinforcement ratio 1% and confinement reinforcement varied (10mm, 12mm, 16mm, 20mm, 25mm) and spacing of the confinement ring also varied (50mm, 100mm, 150mm, 200mm, 250mm ,300mm) in following cases as shown in Table-1

Table 1: Details of circular Section

	Dia of section	Grade of concrete	Long reinforcement details		Pt%
			DIA. (mm)	NO.	
Case -A	2400	M40	32	1x57	1
Case -D	2400	M50	32	1x57	1
Case -E	2400	M60	32	1x57	1
Case -F	2400	M70	32	1x57	1

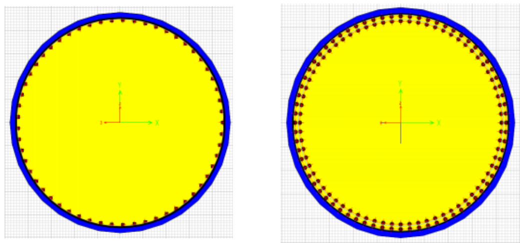

Figure 4. Typical Cross Section of Circular Section

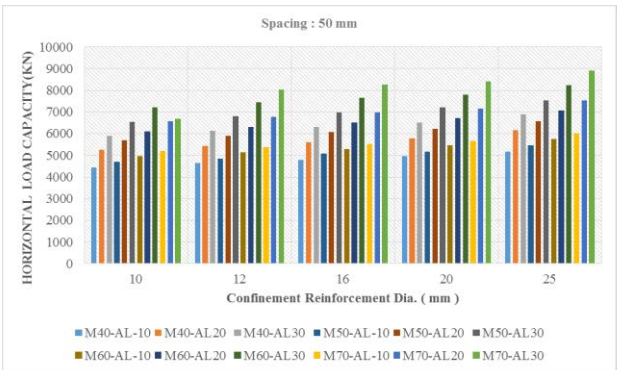

Figure 5.1. Horizontal load carrying capacity for confining reinforcement spacing 50 mm

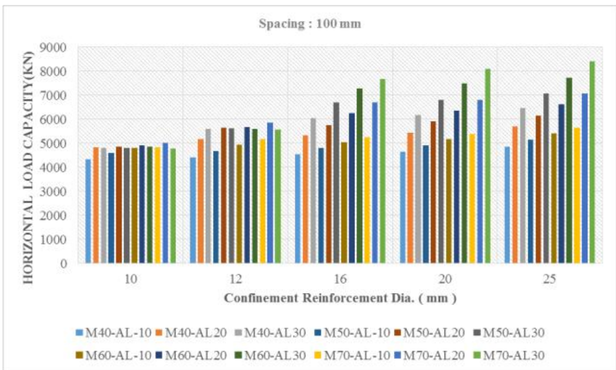

Figure 5.2. Horizontal load carrying capacity for confining reinforcement spacing 100 mm

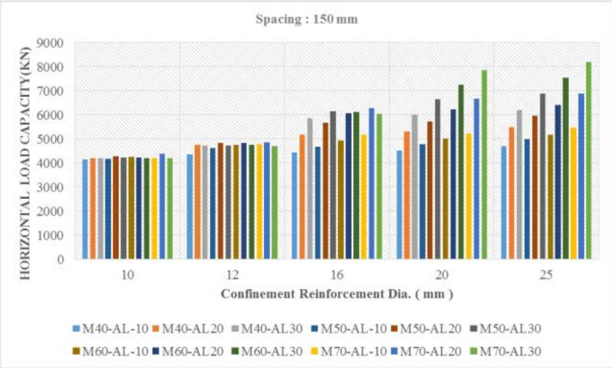

Figure 5.3. Horizontal load carrying capacity for confining reinforcement spacing 150 mm

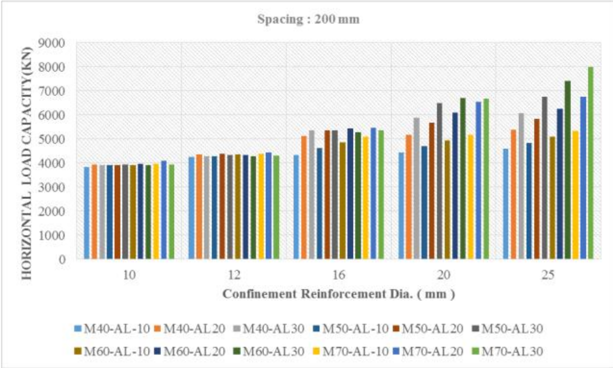

Figure 5.4. Horizontal load carrying capacity for confining reinforcement spacing 200 mm

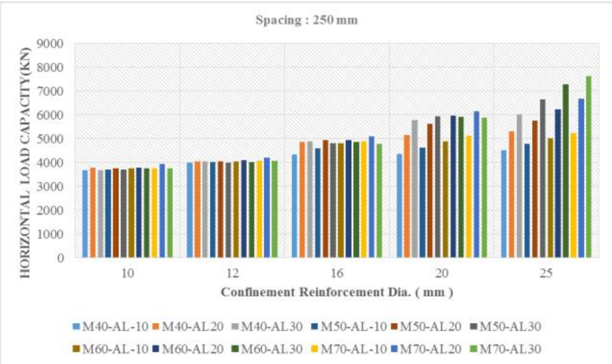

Figure 5.5. Horizontal load carrying capacity for confining reinforcement spacing 250 mm

Figure 5.6. Horizontal load carrying capacity for confining reinforcement spacing 300 mm

VII. CONCLUSIONS

From the *Figure 5.1* to *Figure 5.6* it can be seen that for all concrete grade, Effect of Axial load level in poorly confined concrete is not increasing the horizontal load carrying capacity of column but it is increasing in well confined concrete. Also poorly confined but higher grade of concrete is not improving the section capacity for horizontal load.

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