

Effect of confinement reinforcement on flexural behaviour of column

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Abstract

Analytical study for the inelastic behavior and ductility capacity of confined reinforced concrete bridge piers under the horizontal loading is presented. Simple expressions for the yield curvature of circular and rectangular reinforced concrete column cross section are presented here, based on moment-curvature analysis. This study includes the material nonlinearity, tensile, compressive and shear models for cracked concrete and reinforcing steel. The analytical results reveal that the addition of confinement would enhance the flexural capacity of column.

Keywords-Bridge pier behavior; Confinement reinforcement; Flexural capacity; High strength concrete; Ductility

I. INTRODUCTION

Efficient seismic design of bridge piers required adequate section deformation capacity without significant loss of strength 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.

High strength concrete is generally use for the construction of bridge pier. This is because the use of high strength concrete can reduce the dimensions of the structural members. However, High strength concrete is more brittle than normal strength concrete. The main reason behind it is the High strength concrete stress-strain curve has a relatively steep and short post peak branch than normal strength concrete. Therefore, high strength concrete columns require proportionately more confinement to attain deformability usually expected from earthquake resistant columns. In the past a series of column tests carried out by Li. et al. and found that the high strength columns with concrete cylinder strength of 100 Mpa could be very brittle if they were not provided with adequate confining reinforcement.

Failure of the bridge piers in the past major earthquakes also shows the buckling of main reinforcement and hence it lead to rapid degradation of the section capacity. Hence, ties in columns is provide to lateral support needed for prevent buckling of reinforcing steels. For this, ties to be effective in both, the tie spacing and tie stiffness must be adequate.

The purpose of the present paper is to report the latest results of a parametric study of confinement reinforcement on the flexural behavior of the section.

II. DEFINITION AND MODELING OF DEFORMATION MECHANISM 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 Figure 1.

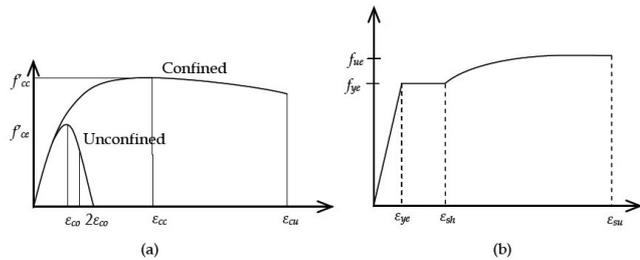


Figure 1. Stress-strain model for (a) Concrete (b) Reinforcing Steel used in the Pushover Analysis by SAP 2000 [CSI 2009]

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 contraflexure 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.

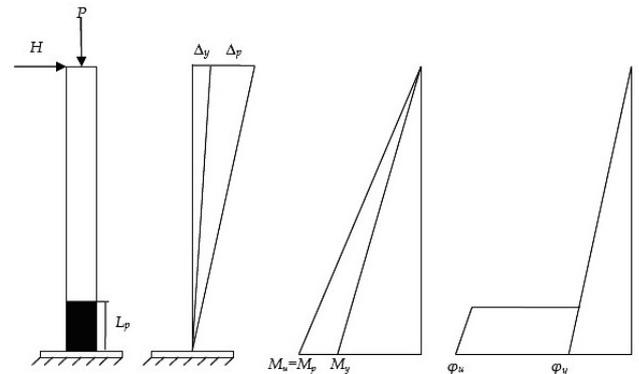


Figure 2. Lumped plasticity idealization of a cantilever and analysis model

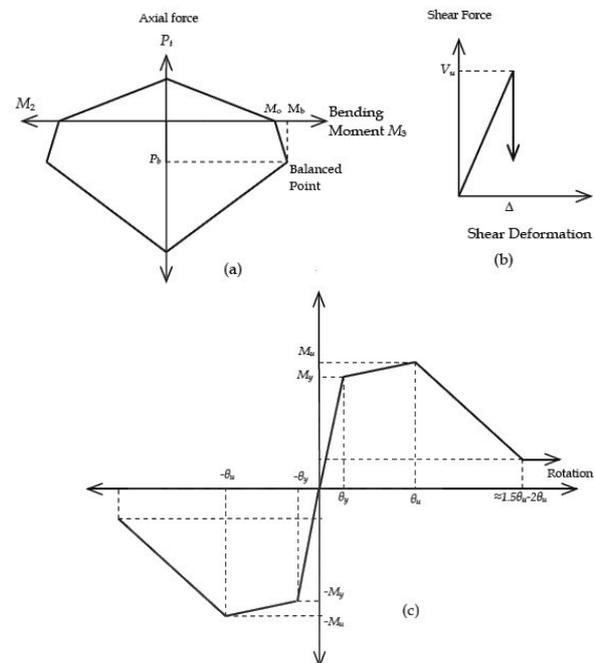


Figure 3. Typical plastic hinge properties assigned to RC members (a) P-M (b) V- Δ , and (c) M- θ

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.:

$$\theta_p = L_p(\phi_u - \phi_{iy})$$

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.:

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

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

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:

$$\text{Lateral Load Capacity} = M_p / L$$

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 the Diameter of confinement reinforcement, Spacing of the confinement Reinforcement, Grade of Concrete, with constant axial load of 20% of concrete strength on RC bridge pier section. It is studied with following variables.

4.1. Rectangle section

To study the effect of the confinement of concrete on the behavior of rectangular section, the diameter of 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 Rectangle Section

| | Size of section | | Grade of concrete | Long reinforcement details | | Pt% |
|---------|-----------------|--------|-------------------|----------------------------|-----|-----|
| | B (mm) | D (mm) | | DIA. (mm) | NO. | |
| Case -A | 1600 | 2900 | M40 | 32 | 60 | 1 |
| Case -B | 1600 | 2900 | M50 | 32 | 60 | 1 |
| Case -C | 1600 | 2900 | M60 | 32 | 60 | 1 |
| Case -D | 1600 | 2900 | M70 | 32 | 60 | 1 |

4.2. Circular section

To study the effect of the confinement of concrete on the behavior of circular section, the diameter of 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: 2.

Table 2: 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 |

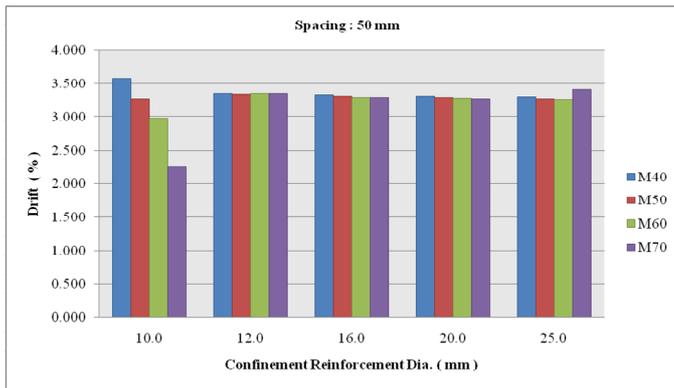


Figure 4.1. Drift capacity of rectangular section with confining reinforcing spacing 50 mm

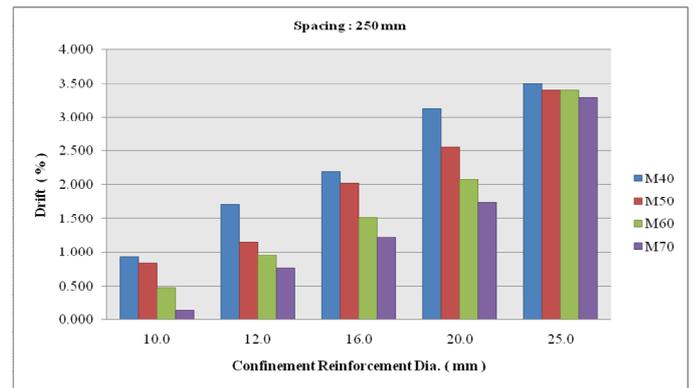


Figure 4.5. Drift capacity of rectangular section with confining reinforcing spacing 250 mm

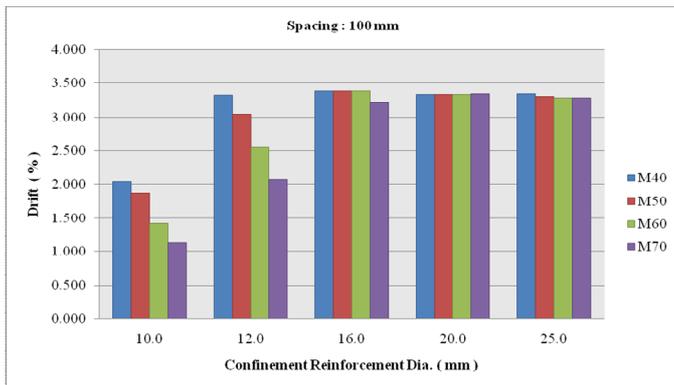


Figure 4.2. Drift capacity of rectangular section with confining reinforcing spacing 100 mm

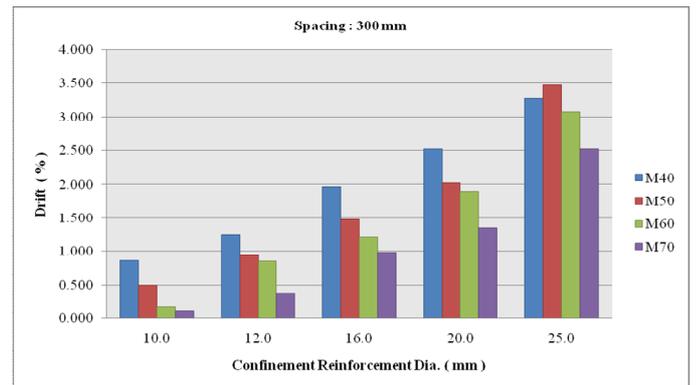


Figure 4.6. Drift capacity of rectangular section with confining reinforcing spacing 300 mm

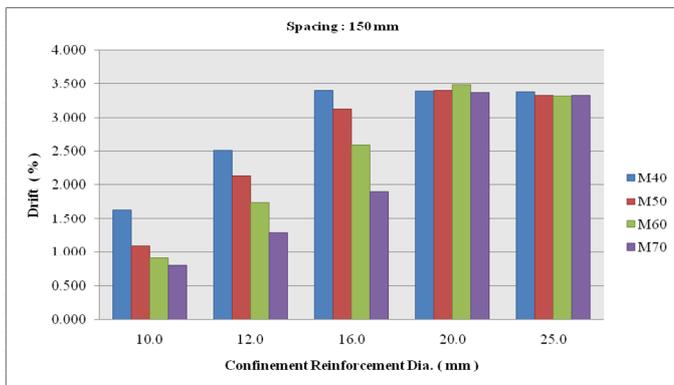


Figure 4.3. Drift capacity of rectangular section with confining reinforcing spacing 150 mm

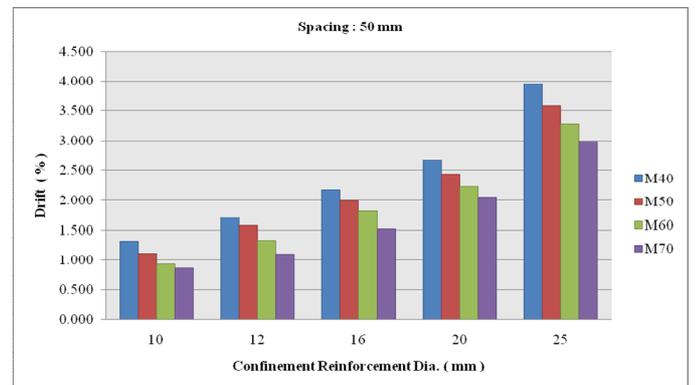


Figure 5.1. Drift capacity of circular section with confining reinforcing spacing 50 mm

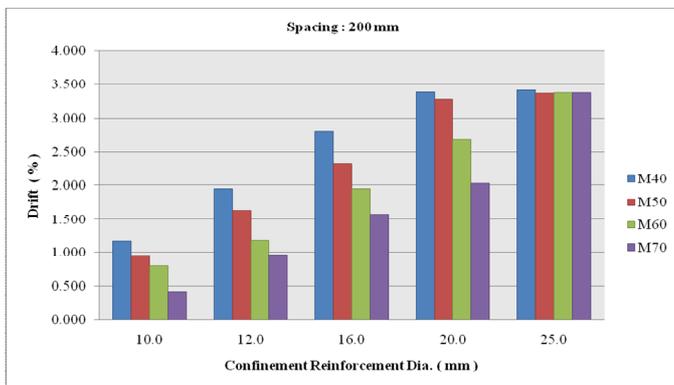


Figure 4.4. Drift capacity of rectangular section with confining reinforcing spacing 200 mm

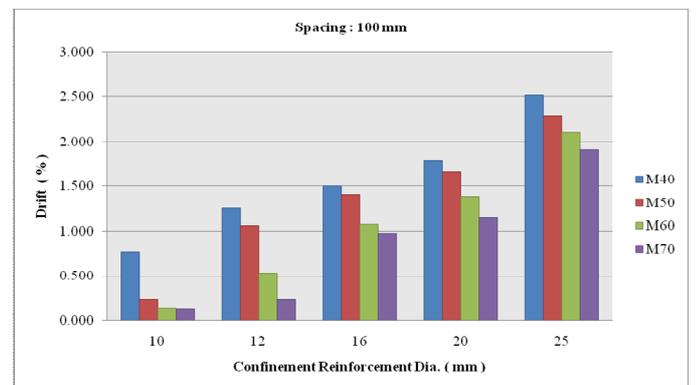


Figure 5.2. Drift capacity of circular section with confining reinforcing spacing 100 mm

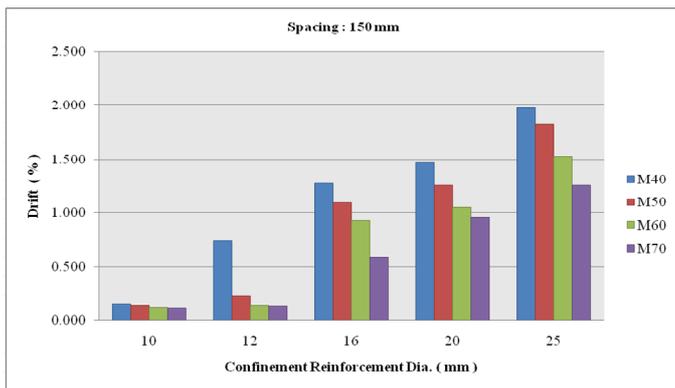


Figure 5.3. Drift capacity of circular section with confining reinforcing spacing 150 mm

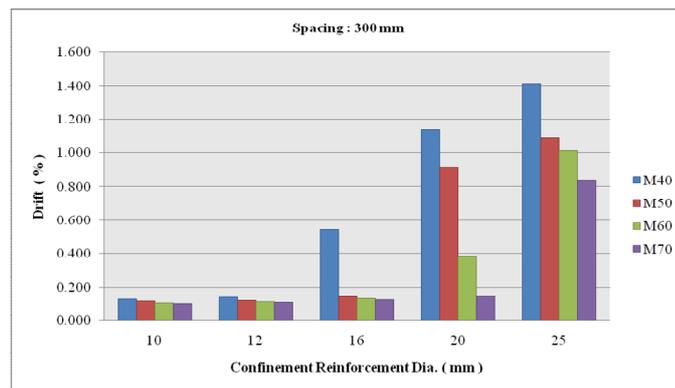


Figure 5.6. Drift capacity of circular section with confining reinforcing spacing 300 mm

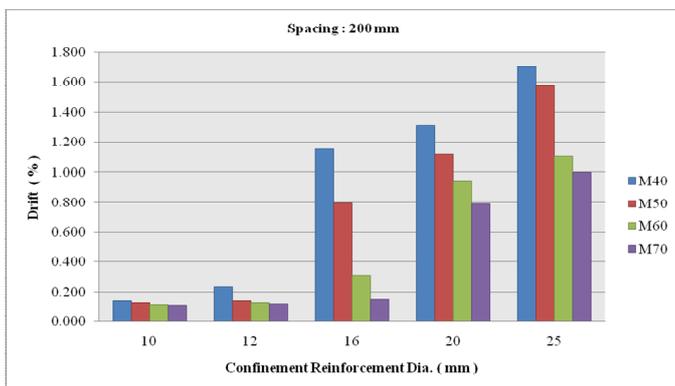


Figure 5.4. Drift capacity of circular section with confining reinforcing spacing 200 mm

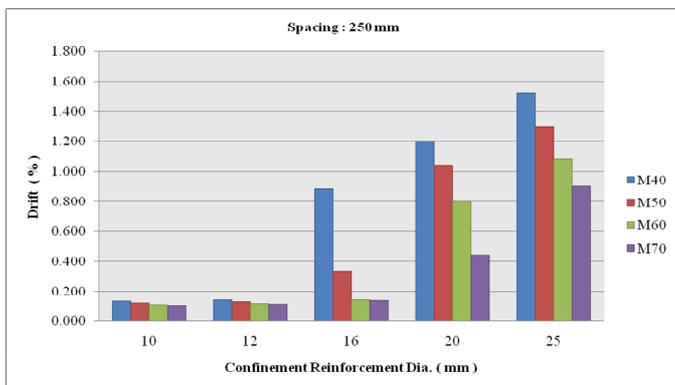


Figure 5.5. Drift capacity of circular section with confining reinforcing spacing 250 mm

V. CONCLUSIONS

The flexural behavior and ductility of confined concrete columns have been studied by nonlinear moment-curvature analysis. From the analysis results so obtained, it may be concluded that although the increasing the confining reinforcement is generally effective in improving the flexural ductility, its effectiveness rapidly decreases as the concrete strength increases. This implies that the design of High strength concrete columns to have at least the same level of flexural ductility to Normal strength concrete may require an large amount of confining reinforcement.

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