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CYCLIC BEHAVIOR OF SLENDER R/C COLUMNS WITH INSUFFICIENT LAP SPLICE LENGTH

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ABSTRACT :

Performance of slender reinforced concrete columns with insufficient splice length and widely spaced transverse bars has been discussed in this paper. Reinforced concrete columns which constructed according to ACI (pre-1971) provisions often have common deficiencies associated with inadequate detailing of the reinforcing steel. One such deficiency is the detailing of lap splices at the base of columns, which are often short and poorly confined as a result of widely spaced ties with 90-degree hooks, and located immediately above the floor or foundation where large inelastic demands can be expected. Poor splice behavior has been observed in recent earthquakes, may have significant role in partial or total collapse of buildings. Although there are many modern codes that offer detailing for high prone seismic regions, due to poor implementation such defective detailing is constructed in some developing countries such as Iran.
Four half-scale columns representing exterior columns were constructed and tested under reversed cyclic lateral deformation with either constant axial load. Three specimens conformed to ACI (pre-1971) and a specimen was built according to ACI (318-02). Specimens consisted of cantilever columns with strong stubs. Parameters which covered in this study include axial load, widely spaced transverse bars, and lap splice length.
In conclusion, seismic behavior of the columns in term of the hysteretic response was presented and compared with various evaluation methods. Moreover, results of experimental tests were compared with a numerical study. In general, the common mode of failure for the specimens was a brittle failure owing to bond deterioration of the lap-spliced longitudinal reinforcements.

KEYWORDS: Reinforced Concrete Columns, Lap Splice Deficiencies, Experimental, Numerical

1. INTRODUCTION

It is well known that columns have a vital function in the structural concept. Therefore, any defects in the construction or design of these elements can bring about instability or collapse of whole or part of buildings under earthquake excitations.
In many high prone seismic countries numerous reinforced concrete frame structures exist that were designed and constructed conforming to earlier codes without considering the seismic loads. Many of the construction details used in the structures are now recognized to be associated with nonductile failure modes when subjected to seismic excitement. As a result, performance of the structures is considered suspect under moderate to severe seismic loading. In addition, due to poor construction practice and inspection in such countries, defective and flaw detailing has been implementing yet without meeting seismic demands. A significant deficiency in these nonductile frames is often in the columns.

1.1 Existing Nonductile Column Details and Behavior

To establish typical details of existing reinforced concrete columns, a review of the ACI 318 requirements from

1941 through 1995 was undertaken. There were no specific provisions in ACI 318 on seismic design before the 1971 edition, and seismic design provisions constituted only an appendix until 1989. Columns design characteristics used and judged to be potentially critical include:

1. Large spacing of often small-diameter transverse reinforcement resulting in poor confinement of core concrete and insufficient lateral support of longitudinal bars;
2. Insufficient shear capacity resulting from inadequate transverse reinforcing steel;
3. poor detailing of transverse reinforcement bring about loss of confinement and longitudinal bars after cover spalling; and
4. Lap splice with insufficient length and poor confinement located in regions of high bending moment when the frame is subjected to lateral loading.

These details, particularly the inadequate size, spacing, and detailing of column ties will result in inadequate column ductility. Depending on the nature of the column deficiencies, the frame geometry and the loading on the column, several different modes are summarized as follows:

Axial-Flexural failure in the hinge region- occurs when the column develops its plastic flexural strength; nevertheless, owing to inadequate confinement of the hinge region, column is not capable of developing considerable flexural ductility. Insufficient confinement of the concrete in the compression zone of the hinge brings about undesirable crushing of the concrete. Load reversal further compound the deterioration of concrete in the hinge region. Furthermore, inadequate lateral support from transverse bars allows the longitudinal reinforcing to buckle.

Ductile shear failure in the hinge region- occurs when the column develops its plastic flexural strength; however, ultimately fails in shear. Widening of the flexural-shear cracks in the hinge region under cyclic load after yielding of longitudinal reinforcing causes rapid deterioration of the shear capacity in the hinge region.

Brittle shear failure- takes place when the shear capacity of the column is insufficient to develop the plastic flexural strength of the column. Such a failure is often featured by single inclined crack (or X crack under reverse loading) accompanied by rapid reduction in lateral load resistance. Brittle shear failures are typically observed in short column.

Lap splice failure- takes place when the longitudinal bars of the column develop more force than lap splice can carry. A splitting failure occurs in the concrete along the interface between lapped bars. The failure results from insufficient confinement in the lap-splice region and from lap splices that are not long enough to develop the tensile strength of the longitudinal bars. Lap splice failure is featured by splitting cracks and relative sliding of lapped bars.
This paper focuses on the lap splice failure. However, because of the different level of axial loads other failure modes could be happened in the specimens.

1.2 Objective

Buildings that were constructed before the 1970s can have significant deficiencies in their overall structural configuration and detailing. The longitude bars were spliced 20 reinforcement diameters independent of column size, strength, or deformation demands. Besides, the often small-diameter transverse reinforcement was typically located widely, with 90-degree bends for anchorage. It would get worse if column was slender owing to lack of design knowledge that leading to extremely high lateral demands in these columns.
San Fernando Earthquake in 1971 and following earthquakes demonstrated that columns were initially identified as the most vulnerable to earthquake damage. Prior to these events, most of the existing reinforced concrete columns were typically designed primarily for gravity loads and lateral forces may much smaller than that prescribed by current codes. These columns were generally deficient in two respects; low transverse steel causing either premature shear failures or small ductile response, and short lap spliced longitudinal steel at the

base of the columns causing premature bond failure.
Two columns damages caused by lap splice deficiency and lack of confinement in plastic hinge zone are displayed in Fig.1 and Fig.2. Fig.2 demonstrates that such detailing deficiency how can cause extremely damage in R.C building.

Figure 1 Damage in lap splice point caused by absence of adequate confinement and improper lap splice (Bingol, Turkey, 2003)

Figure 2 Collapsed column with inadequate lap splice length and widely spaced transverse bars (Gujarat, India, 2001)

Furthermore few researches exist on performance of slender columns with defective lap splice. This lack of knowledge on how the cyclic-load manner of columns splice especially when accompanied by other defects such as erroneous geometry bring about remarkably uncertainty for seismic rehabilitation of these columns. Thereby knowing the response of these columns to earthquake is very crucial issue, which can aid researchers to innovate new techniques to remedy these deficiencies and achieve to acceptable performance level.
The objective of this paper is to investigate the performance of pre-1971 columns to lateral cyclic displacement and constant axial loading. To meet this objective four flexural dominated slender R/C columns were tested. Afterward the results of a numerical study were compared with results of the experimental one.

2 RESEARCH SIGNIFICANCE

Sparse data exist on the performance of columns with short lap splices. This lack of knowledge on how the lateral load behavior is influenced by important parameters such as axial load, shear, and load history leads to considerable uncertainty for seismic rehabilitation, and ultimately, conservative and costly rehabilitation measures. To fulfill this need, a research project was undertaken to provide vital data on the performance of slender column lap splices.

3 EXPERIMENTAL PROGRAM

As an experimental work, four half-scale column specimens were built to investigate performance of R.C columns with detailing deficiencies. The specimens were divided into two groups (group I and II) of three specimens and one specimen. For three specimens detailing were provided to cover ACI (pre-1971) provisions. A rest specimen was detailed in accordance with ACI (318-02). All the specimens were slender to represent specific classification of columns which exist in Iran.

3.1 Specimens Geometry and reinforcing layout

All the columns had the square cross-section of 150×150-mm and 800-mm height with a 350×550×350-mm foundation block in order to provide full anchorage. However, the reinforcing details of the specimens were completely different. The group I (SP-C1,2,3) was reinforced by eight 8-mm vertical bars which confined by 4-mm ties with 90-degree hooks spaced at 160-mm. The lap splice length was 160-mm (20 bar diameters), and the group II (SP-C4) had eight 8-mm vertical bars with 320-mm lap splice length enclosed by 4-mm ties with 135-degree hooks at 60mm. Specimen designation is summarized in Table1.

Table 1 Test specimens designation

Specimen	P/ f' c A_g	Axial load(kN)	ρ_{sh} %
SP-C1	0.05	21.2	0.258
SP-C2	0.10	42.4	0.258
SP-C3	0.15	63.6	0.258
SP-C4	0.05	21.2	0.644

Table 2 Material properties of test columns

Concrete (f'_C),MPa	Longitudinal steel(f_y), MPa	Transverse steel(f_y),MPa
18.9	400	300

3.2 Material Properties

Standard tests on the materials of the column specimens, test of cylindrical samples on concrete batch and tension test on the steel reinforcements, were carried out to obtain mechanical properties of them. Compression strength of the concrete and yield strength of the reinforcement steels are listed in Table 2. The target low compression strength of the concrete reflected the design strength of the 1960s, 20.7MPa.

3.3 Applied Loadings

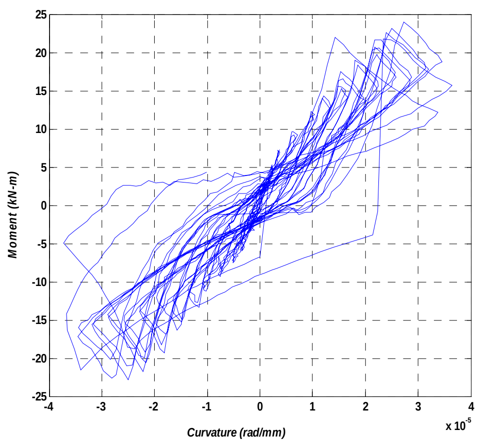
The cantilever specimens were subjected to combined shear cyclic displacement and constant axial loading at top level. Axial load was applied using a 100 kN capacity hydraulic jack through hinges at the ends of the specimens allowing in-plane end rotations. The reversed cyclic displacement was applied through a servo-controlled actuator having 60 kN load capacity and ±60 mm stroke capacity. Displacement control mode of the actuator was used to apply the predetermined displacement history. Axial load levels were approximately 5%, 10% and 15% of the assumed column gross axial capacity ($A_g f_{,c}$). The lateral reverse cyclic displacement imposed for each of the following peak drift ratios: 0.25%, 0.5%, 0.75%, 1%, 1.25%, 1.5%, 1.75%, 2.25%, 2.75%, 3.25%, 3.75%, 4.25%, 4.75%, 5.25%, and 5.75% as displacement increments with two cycles at each level of drift ratio. Photo of the test set up is displayed in Fig.3.

Figure 3 Photo of test set up

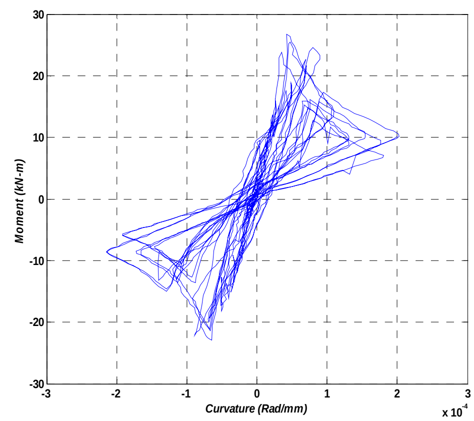
The longitudinal and transverse reinforcing bars and concrete of the specimens were instrumented with strain gauges to determine the steel and concrete strains at various locations and stages of the tests. A total of 6 linear variable differential transducers (LVDTs), 3 on one side, 3 on other side and 3 on the reaction wall, were mounted to measure the deformations of the columns.

3.4 Hysteretic response

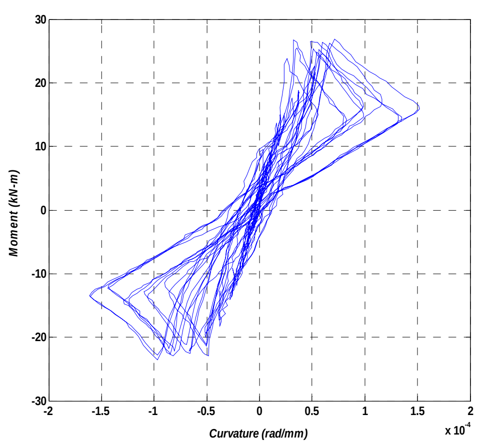
Fig.4 presents the M-versus-ϕ response of the specimens. The total moment at interface was the sum of the primary moment produced by the lateral load and the secondary moment caused by the axial load. The deflected shape of the column obtained from the LVDT readings was used to compute the secondary moment. The differential rotation angle was measured as angle made by the tangent at the column-stub interface with the tangent drawn at the column tip. The deflection values obtained from the LVDT readings were used to compute the rotation angle.



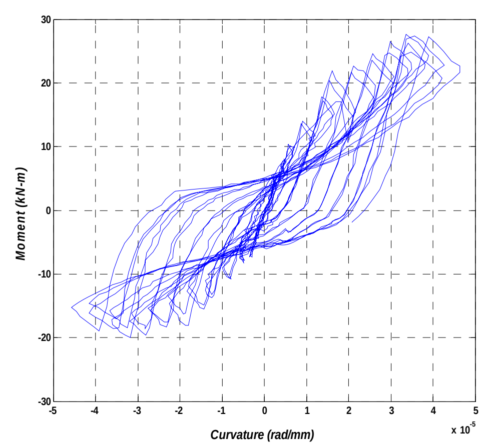
a) SP-C1



b) SP-C2



c) SP-C3



d) SP-C4

Figure 4 Moment versus curvature curves of the specimens

As shown in Fig. 4 specimens with less axial load have more stable hysteretic curves; also, rapid stiffness degradation and pinching effect in SP-C1, C2 and C3 are obvious.

4 RESIDUAL DISPLACEMENT

Response of the reinforced concrete columns to cyclic displacement is strongly dependent on the capability to sustain large inelastic deformation without significant degradation load-carrying capacity. This factor has important role in stability of the buildings. Reduced strength in the specimens can be obtained by measuring permanent displacement. Little remained displacement observed in all of the specimens. Comparison of this deflection among the columns explains residual strength and stiffness. Table 3 shows residual displacement of the specimens.

Table 3 Residual Displacement

Specimen	SP-C1	SP-C2	SP-C3	SP-C4
Residual Displacement (mm)	13.5	17.2	20.5	9.4

5 CALCULATION OF DEFLECTION OF THE SPECIMENS VIA PLASTIC HINGE METHOD

One of the methods to calculate the flexural deflection of an RC member is plastic hinge method. This method could be used to estimate deflection and curvature in R.C members under cyclic loading. In this method, the deformation is divided into elastic and plastic parts and is expressed as:

$$\Delta = \Delta e + \Delta p \quad . (5.1)$$

Where Δe. is the elastic flexural deflection or contribution of the member flexural deflection excluding the plastic hinge length, deflection at yield. This deflection may either be computed exactly based on the moment-curvature relationship or based on the assumption that the curvature distribution within the yield curvature is linear. Calculation of the elastic flexural deflection are on order of:

$$I_g = \frac{bd^3}{12} \quad (5.2) \qquad I_e = 0.7 I_g \quad (5.3)$$

$$M = \left(\frac{L - L_p}{L}\right) M_n \quad (5.4) \qquad \phi_e = \frac{M}{E I_e} \quad (5.5)$$

$$\Delta_e = \frac{\phi_e (L - L_p)^2}{3} \quad (5.6)$$

Where ϕe is elastic curvature, L is the column height, and Lp denotes the length of the plastic hinge. In this study the plastic hinge formulation of Priestly and Park was employed.

Table 4 Plastic rotation and curvature, and strain in concrete in the specimens

	Drift	$\Delta\theta_p$ (rad)	$\Delta\phi_p$ (rad/mm)	$\Delta\varepsilon_c$ (mm/mm)	ε_c (mm/mm)
SP-C1	3%	0.0127	0.000090	0.00449	0.00815
	4%	0.0227	0.000162	0.00802	0.01170
	5%	0.0327	0.000234	0.01158	0.01524
	6%	0.0427	0.000305	0.01510	0.01876
SP-C2	3%	0.0138	0.000099	0.00559	0.00925
	4%	0.0238	0.000170	0.00961	0.01327
	5%	0.0338	0.000204	0.01356	0.01722

	6%	0.0438	0.000310	0.0175	0.02116
SP-C3	3%	0.0137	0.000097	0.00612	0.00978
	4%	0.0237	0.000169	0.01056	0.01422
	5%	0.0337	0.000241	0.01506	0.01872
	6%	0.0437	0.000312	0.02130	0.02496
SP-C4	3%	0.0078	0.000055	0.00269	0.00809
	4%	0.0178	0.000127	0.00613	0.01153
	5%	0.0278	0.000199	0.00961	0.01501
	6%	0.0378	0.000270	0.01304	0.01840

Priestley and Park (1987) proposed a plastic hinge length that considers the strain penetration into the footing for columns, and is dependent on the rebar diameter and column length. The plastic hinge length proposed is:

$$L_p = 0.08L + 0.022\, d_{lb} f_y \tag{5.7}$$

Where d_{lb} denotes the bars diameter and f_y is the yield strength of the longitudinal bars. Fig. 5 is a typical illustration of the plastic hinge method (Priestley and Park, 1987). The length of the plastic hinge region will be 140 mm for the columns.

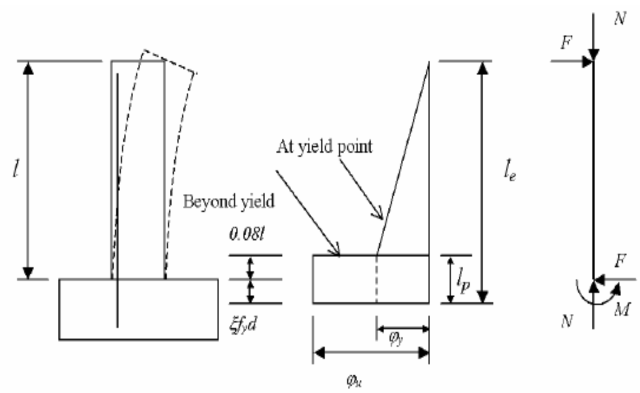


Figure 5 Typical plastic hinge method

Δp in Eqn. 5.1 is the postyield deflection resulting from the plastic hinge and is computed as follows:

$$c = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s} d \tag{5.8}$$

$$\phi_p = \frac{\varepsilon_c}{c} \tag{5.9}$$

$$\theta_p = \phi_p \times L_p \tag{5.10}$$

$$\Delta_p = \theta_p (L - L_p / 2) \tag{5.11}$$

Where ε_c= compressive postyield strain of the concrete in extreme fiber; ε_s=steel strain in tension; d= distance from extreme compression fiber to centroid of tension reinforcement; ϕ_p= postyield curvature of the concrete; θp= rotation in the hinge region; and c= depth of the natural axis.
Table 4 displays the plastic rotation and curvature, and strain in concrete in the specimens. In SP-C4 with sufficient splice length the values of the plastic rotations and curvatures, and concrete strain is minimum among the specimens.

6 CONCLUSION

Existing reinforced concrete (RC) columns detailed with poor lap splices and inadequate transverse confinement reinforcement with 90-degree hooks in the potential plastic hinge regions near footing-column joints, characteristic of pre-1970 design provisions, are found to be deficient for the strength and ductility demands imposed by earthquake loading. The work reported was directed toward the evaluation of the columns experimentally. To investigate the preciseness of the experimental program the plastic hinge method was employed, and the results of two methods were compared with each other. The following conclusions were made:

1. Presence of poorly detailed lap splices in the potential plastic hinge region of a column leads to significantly reduced ductility and unstable hysteretic behavior with rapid degradation of strength and pinching effect due to premature splice failure.
2. High axial load resulted in considerable reduction in the ductility and energy dissipation capacity of columns.
3. The plastic hinge method results approximately were in agreement with experimental work; however, induced concrete strain in the method was much more than experimental work, which it could be because of the splice failure in the specimens.
4. SP-C4 has the most stable hysteretic response, which it could be due to yielding the longitudinal reinforcing bars without any premature failure.
5. The contribution of bond-slip in the splice region to lateral displacements can be substantial and should not be ignored in the analysis of older reinforced concrete construction. For the cantilever columns studied here, the bond-slip effects were computed to account for about 30% of the calculated lateral curvature corresponding to the peak load.

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