AN EXPERIMENTAL STUDY OF DEFECT῾SEFFECTS IN DETAILING ON THE AXIALBEHAVIOR OF EXISTING CONCRETE COLUMNS

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ABSTRACT

In many countries, Reinforced concrete structures are traditionally designed without seismic provision. In particular, non-seismic detailing is specified in the columns. Seismic rehabilitation of old buildings has been a major challenge in recent years. The first step in seismic rehabilitation is evaluation of the existing capacity and the seismic behavior.

The purpose of this research is the study of defect῾s effects in the transverse reinforcement details such as: bend type of transverse reinforcement, anchorage of transverse reinforcement, spacing and volume of transverse reinforcement, on the axial behavior of reinforced concrete columns in the existing buildings. On this basis, for investigation of the seismic behavior of RC columns of a real old building in Iran which has been designed and constructed by European engineers in 1940,four concrete specimens with half-scale with the cross section dimension of300 x 300 mm^2 and the length of 1000 mm, have been tested on the effect of vertical axis in the structures laboratory of faculty of engineering of University of Tehran. It should be noted that, all specimens are 28-day compressive strength of 25 Mpa. Samples are made with similar characteristics of old buildings and reinforced with plain bars by 90 and 135 degree bending of transverse reinforcement. Tests results indicated that the column specimens with non-seismic reinforcement detailing, possesses low ductility capacity. In addition, ratio, bending and anchorage of transverse reinforcements are the most important beneficial effect on the stress-strain behavior of concrete .

INTRODUCTION

Recent earthquakes, have been proved the lack of sufficient strength and ductility of old buildings, which can cause extensive damage and even collapse of these buildings. Seismic repair of existing concrete structures is the major challenge in the past three decades. The first step in seismic rehabilitation of existing concrete structures is evaluating seismic behaviour of these structures. To investigate the response of concrete structures, knowing the response of involved material from seismic point of view (i.e. the behaviour of concrete and rebars under cyclic reversal and monotonic loading) is the first important stage. Using the behavior of material, the predicting behavior of members is performed. Columns are the main members of the structures, that realize the actual seismic behavior of these members, give us a major help in estimation of ductility capacity and energy dissipation capacity which are important for seismic evaluation.

A large number of existing RC columns located in the most actively seismic areas have not been designed to fulfil the requirements of modern seismic design codes. Vital deficiencies in such columns include the typical reinforcement details such as plain bars, lightly, widely-spaced and poorly-anchored transverse reinforcement. These are generally termed as non-seismically detailed RC columns. Some of the older cases of such building were probably designed just for gravity loads and do not have special seismic detailing for structural members (e.g. beams, columns, joints, etc.) because the old codes did not include special seismic provisions at that time. Recent post-earthquake investigation indicate that these columns are vulnerable to earthquake-include collapse, posing a threat to public safety in future earthquakes. However, there is relatively limited data available in literature for non-seismically detailed columns with respect to ductile-detailed columns. In addition, most tests involving RC columns subjected to axial loading have been terminated shortly after loss of load resistance. Only few tests on RC columns have been carried out to the point of axial failure, resulting in limited understanding of failure and collapse mechanisms of non-seismically detailed structures. Therefore to attain a better understanding of the seismic behavior of non-seismically detailed columns, a study is being undertaken at University of Tehran (UT), Iran. Four half-scale RC columns with light transverse reinforcement were tested. The specimens were tested to the point of axial failure under constant axial load. Experimental results obtained include crack patterns, stress-strain curve and axial failure. The test results are useful in understanding the axial failure mechanisms of non-seismically detailed RC columns under axial loading. Knowing the deficiencies, it should be possible to improve the survivability of such columns during earthquakes.

EXPERIMENTAL PROGRAM

TEST SPECIMENS AND TEST VARIABLES

In the experimental program, a total of four half-scale RC column specimens were tested under monotonically increasing concentric loading. A common geometry for all specimens was used. All the specimens have square cross sections with dimensions 300 x 300 mm^2 and a total height of 1000 mm. Concrete casting was performed in one step in horizontal position. All of the specimens' reinforcements are composed of four longitudinal bars of 16 mm in diameter and confined with ϕ8 bars as transverse reinforcements. The longitudinal reinforcement was placed into the corners of the cross section. The variables investigated include the configuration, spacing, type of bars, bend and anchored transverse reinforcement. To investigate the effect various types of old transverse-detailing practice, three of specimens with various transverse reinforcement detailing with plain steel bars for longitudinal reinforcement and lateral reinforcement were considered. The only change between them and the other specimen was the type of bars that used deformed bar stead of plain bar. The distance of the transverse reinforcement at the ends was denser to prevent damage in this region to provide extra confinement and to ensure that failure occurred near the center of the specimen. Details of the geometry, reinforcement detailing and cross section of specimens are shown in Figure 1.

CONCRETE AND REINFORCING STEEL MATERIALS

Target cylindrical compressive strength for concrete specimens at the age of 28 days was 25 Mpa. For each column specimen, five 150 x 150 mm concrete cubes were also cast and tested to determine the strength of plain concrete at the day of testing. The maximum concrete stress in cubes is termed f_c. The concrete materials were composed of normal aggregates with a maximum size of 19 mm and a water-to-cement ratio of 0.51. Concrete mix proportions per cubic meter of concrete were: 800 kg coarse aggregates with a diameter between 5 and 19 mm, 1150 kg fine aggregates with a maximum diameter of 5 mm and 245 kg of Portland cement Type-I. Additives were not used for the mix. Columns were cast in a horizontal direction. The concrete cover used was 25 mm to the face of the tie for all the test specimens.Alternatively, thin layers of grout were used as capping over the top and bottom ends of each specimen to ensure parallelism of specimen end surfaces and uniform distribution of the load during testing. The column specimens are designed with different transverse reinforcement spacing and ratios, bend type and anchorage.

However, tension test were performed on steel samples of each bar diameter in Table 1. The summary of test specimens and their properties is presented in Table 2.

Table 1 . Mechanical properties of steel

No.	Specimen	Longitudinal Bars				Transverse Reinforcement			
…	…………..	Type of Bar	f_y (Mpa)	f_u (Mpa)	Elong. (%)	Type of Bar	f_{ys} (Mpa)	f_u (Mpa)	Elong. (%)
1	P-25-90-100	Plain	299	431	43.3	Plain	313	432	42.6
2	P-25-90-160	Plain	299	431	43.3	Plain	313	432	42.6
3	P-25-90-200	Plain	299	431	43.3	Plain	313	432	42.6
4	D-25-135-100	Deform	420	637	25.9	Deform	374	585	30.8

Table 2 . Details of test specimens

No.	Specimen	Concrete f'_c (Mpa)	Longitudinal Bar		Transverse Reinforcement				
…	…………	………	No.&Size	ρ_l	No.&Size	ρ_s	Spacing (mm)	Bend (deg)	Anchoring Length (mm)
1	P-25-90-100	25.90	4ϕ16	0.89	ϕ8@100	0.80	100	90	40
2	P-25-90-160	25.96	4ϕ16	0.89	ϕ8@160	0.50	160	90	40
3	P-25-90-200	24.50	4ϕ16	0.89	ϕ8@200	0.35	200	90	40
4	D-25-135-100	26.17	4ϕ16	0.89	ϕ8@100	0.80	100	135	24

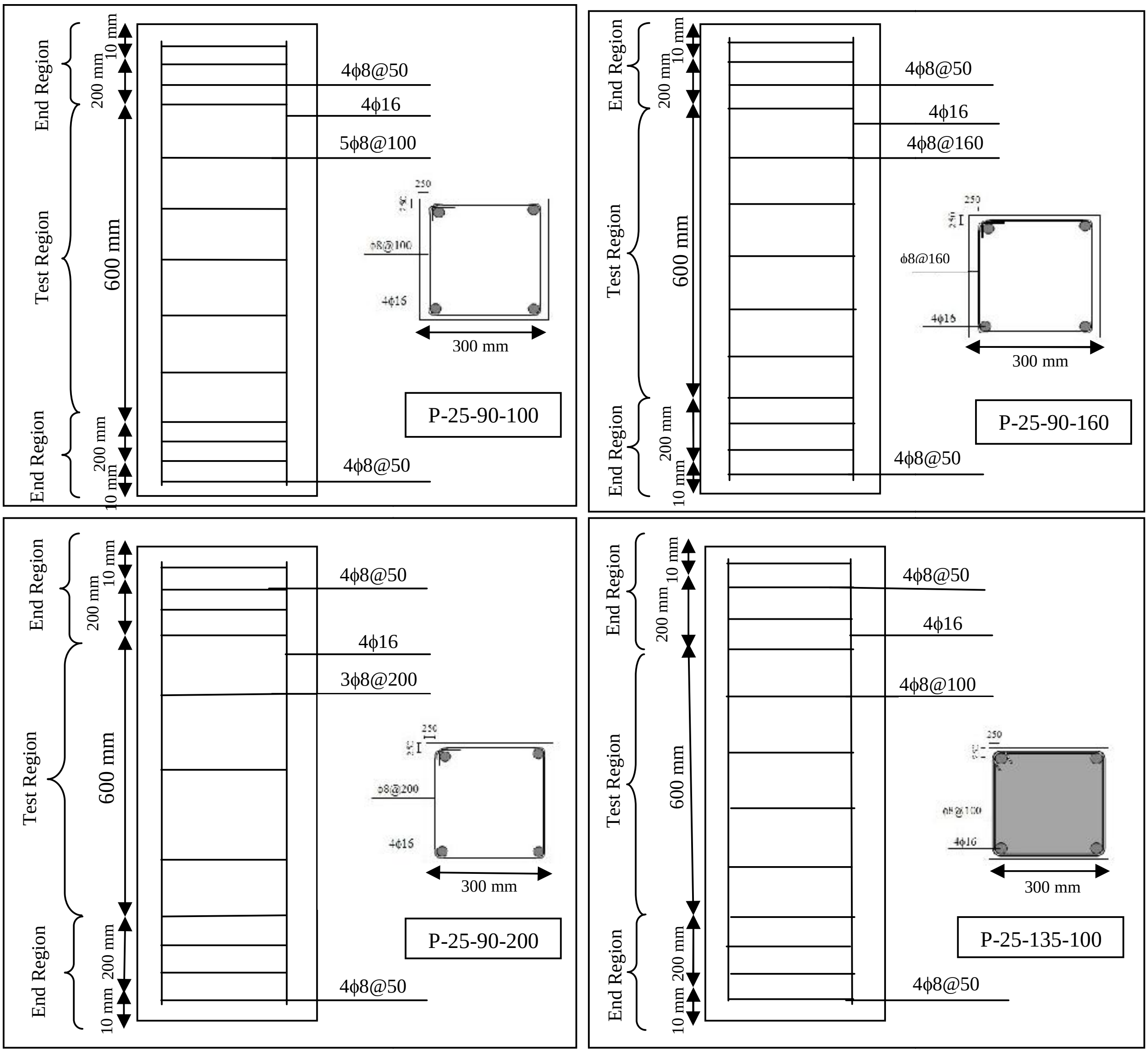

Figure 1.Cross section of specimens and reinforcement view

According to the non-seismic reinforcement detailing requirement, the spacing of ties in a reinforced concrete column should be equal to or smaller than 300 mm, 12 times the bar diameter, or the depth of the section, whichever is the least. For the test specimens, 12 times the bar diameter (192mm) becomes the control. This value is 6 times the bar diameter (96mm) in seismic reinforcement detailing. Tie spacing of 200 mm is adopted for specimen No.3 (P-25-90-200). Specimen No.2 (P-25-90-160) is designed with closer tie spacing (160mm) along the height of the column in order to achieve better ductility. For specimen No.4 (D-25-135-100), the proposed modified seismic detailing, which involves additional crossties into the cross section, is adopted.

In seismic design, the ties are required to be anchored 135 degree with at least 6 or 8 bar diameter extension. This value can 90 degree with at least 5 bar diameter extension in one side and 135 degree with at least 6 bar diameter extension at the other side.

EXPERIMENTAL SETUP

View of the experimental setup is presented in Figure 2(b). A concentric vertical load was applied by a vertical hydraulic actuator with a maximum force capacity of 3000 KN. The steel reaction columns of the machine used in this research are stiff enough to meet such requirements and permit the machine to load at a rate as low as 0.01 mm/min. The test specimens were placed in the press with steel guides to ensure adequate alignment with the axis of the applied load. The tests continued until complete after loss of load resistance of the columns.

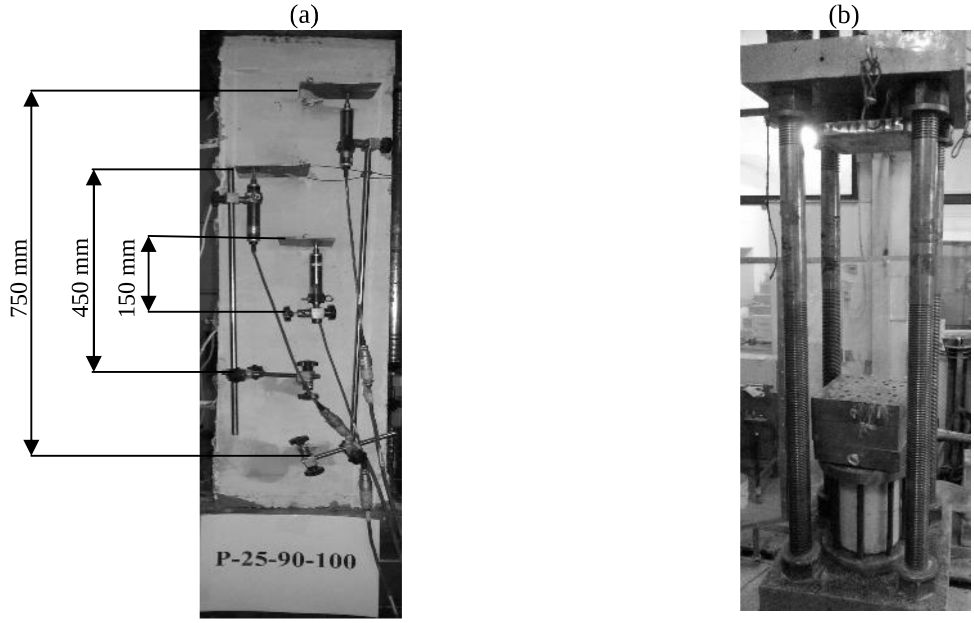

Figure 2.Typical LVDTs arrangement on specimens (a). Test setup for specimens (b)

INSTRUMENTATION

The axial deformation of the specimens was recorded using six linear variable displacement transducers (LVDT) located at the mid region of the North and South sides of the specimens. All of the LVDTs were fixed to the specimen by passing through bars that had contrived before concrete casting. In addition, two pairs of LVDT with a range of 25 mm are placed on each side of the column close to the base, for measuring the global deflections. The overall concrete axial strain was calculated as the average of the LVDT measurments divided by the gage length. In order to determine the strain in reinforcement steel, electrical-resistance strain gages are attached to longitudinal and transverse reinforcemens steel. One longitudinal steel bar and three of ties, located at the midlle height of each column specimens was instrumented with strain gages. Craking, buckling of axial reinforcement, and other observational data were also recorded during all tests. Figure 2(a) shows the arrangement of the LVDTs. data of each test were acquiesced digitally by a 40-channel data logger.

TEST OBSERVATIONS

During the tests, different observation were recorded, which will be discuss in this section. The crack patterns of the test specimens at axial failure are in which axial failure happened once the specimens were unable to resist a certain applied axial force. As it is not possible to describe the crack development for all the specimens in detail within this paper, only important features in crack development of specimens are highlighted.

No cracks were observed up to the measured peak load in all of the specimens which were subjected to small unintentional eccentricities during testing. By increasing load, axial cracks were observed as the extending of axial cracks form two edges in the loading direction of columns to the middle area before the columns reached maximum axial force. In specimen No. 3 cracks were observed. The specimen No. 1 experienced these cracks after this stage. At the peak load, the concrete cover suddenly spalled off explosively at the mid region of the column with larger tie spacing (P-25-90-200). Spalling of the concrete cover for the column with closer tie spacing occurred more slowly (P-25-90-100). At this stage, main axial cracks form in the body of the specimen No. 2 (P-25-90-160). Specimen No. 4 (D-25-135-100) shows stable strength and stiffness degradation during the experiment. In axial failure modes observed in the specimen No. 3 as illustrated in Figure 3. In this mode, crushing of concrete as well as buckling of main bars and fracturing of transverse reinforcement developed across

Figure 3. Modes of failure of specimens 1,2,3 and 4

a damaged zone. Specimen P-25-90-200, failed in a brittle manner and surface concrete spalls off and crushes with the corresponding bad decrease. For specimens P-25-90-100 and specimen P-25-90-135, crushing of concrete was first observed along the two surfaces with no internal links being tied to the main reinforcements. Subsequently, the main reinforcement buckled together with some degree of opening up of the 90 degree hooks.
Although the specimen No.1 cracked in the same way as the specimen No. 2, the specimen No. 2 showed a more rapid reduction in bad carrying capacity after peak strength. Relatively higher residual resistance remind after the peak load was measured for column No. 4 (D-25-135-100) with closer tie spacing and anchorage of 135 degree. This behaviour suggests that the remaining resistance of the column is highly dependent on the local buckling resistance of the individual longitudinal reinforcement. For some of the columns with closer spacing of ties, the

transverse reinforcement yielded at a later stage of loading. For specimen No. 3, the 90 degree hooks with anchorage of 40 mm, are opened and buckling of reinforcement is observed after spalling of surface concrete.

On these tests, when ties spacing is larger, the core of the specimens No. 1&2 and 3 is damaged more seriously than that of the specimen No. 4. Indeed, compared to specimens of P-25-90-100 and P-25-135-100 with the same spacing, the damage to specimen P-25-90-100 was concentrated in the narrower region between ties. A well-define axial failure plane, attaining in the full length of the specimen, was formed in the P-25-135-100 specimen. This failure pattern depends on lateral confining pressure and therefore volumetric ratio and angel of bend. Finally, the bars always buckled between transverse reinforcement, as can be seen in Figure 4.

STRESS-STRAIN RESPONSE

EFFECT OF VOLUMETRIC RATIO OF TRANSVERSE REINFORCEMENT

Figure 4 plot compares the stress-strain curves of specimens between which only volumetric ratio of transverse reinforcement (ρ_s) differs. The tie yield strength (f_{ys}) for all of these specimens is 313 Mpa. It is clear that, as volumetric ratio ρ_s increases, the peak stress and the corresponding strain of confined concrete increases. With additional lateral reinforcement, P-25-90-100 column exhibit greater strain ductility than the others and axial-resisting capacity reduced gradually. In addition, a consistent decrease in ductility is observed with increasing space of transverse reinforcement. This is because column with larger space of lateral reinforcement, exhibits less lateral confinement under axial compression than the column with closer space. Since confinement pressure is developed when transverse reinforcement restrains the lateral expansion of concrete, the confining reinforcement comes into play later when loading column with larger spacing. Consequently, passive confinement is less efficient in column with larger spacing.

As it is shown in the plot, specimen P-25-90-200 lost at least 40% of load-carrying capacity almost immediately after the peak strength. At approximately $\frac{1}{3}$ of their peak strength, specimens with lower volumetric ratios lost capacity so quickly that the hydrolic machine was unable to continue loading.

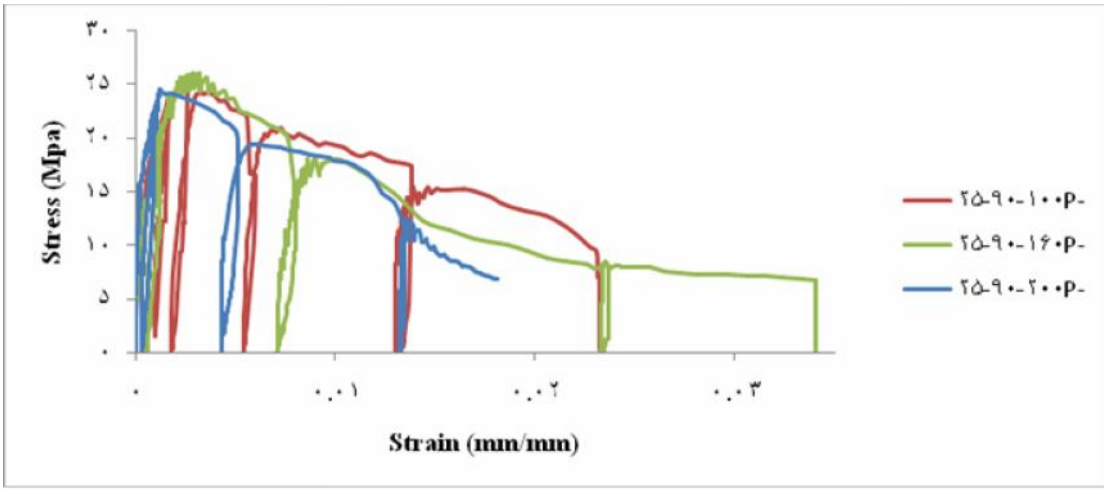

Figure 4. Effect of tie volumetric ratio

EFFECT OF YIELD STRENGTH OF TRANSVERSE REINFORCEMENT

Figure 5 shows the stress-strain curves for specimens having different tie yield strengths, but with similar volumetric ratio (ρ_s=0.5%). As it is shown in the table 1, yielding strength for deformed transverse reinforcement bars was 374 Mpa while it was 313 Mpa for plain of them. It is clear that increasing the yield strength does not significantly improve the confinement effect. Even when columns have approximately the same ρ_s and f_{ys} product, those with higher volumetric ratios of lower-grade steel performed better than those with lower volumetric ratios of higher-grade steel. Lateral confinement pressure is typically calculated using the tie yield strength, predicting higher confinement pressure when higher grade steel is used. However, these experiments show that high-grade tie bar does not yield in strength concrete. Therefore, the tie yield strength cannot be used when calculating the confinement pressure, and confinement calculating using f_{ys} will overestimate the lateral confining pressure. Thus, the behavior shown in Figure 5 is to be expected.

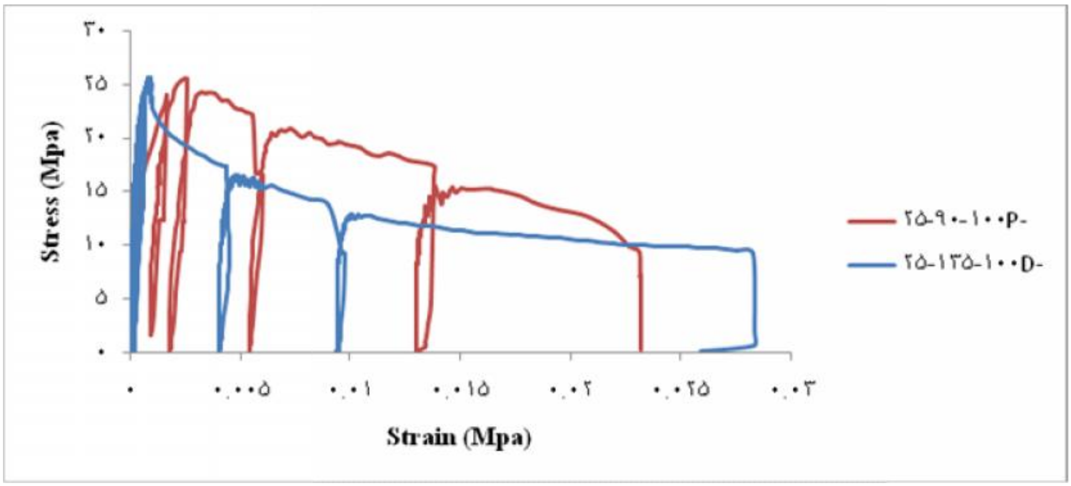

Figure 5. Effect of tie yield strength

EFFECT OF BEND AND ANCHORAGE OF TRANSVERSE REINFORCEMENT

In the final stage of experiments, all specimens have different behavior. Tests shown that, after crushing of concrete when longitudinal bars want to buckling, bend of Transverse reinforcement have a significant effect to don't permit the tie up and then buckling of longitudinal bars. It is clear that, specimen with bend of 135 degree have a great role in ductility of specimens, but in column P-25-135-100 due to the lack of anchorage of transverse reinforcement, the middle column transverse reinforcements ties up and for this reason, causing were premature buckling of longitudinal bars. Figure 6 can be shows the effects of transverse reinforcement bend and anchorage in stress-strain curve from the average of two LVDT are placed closed to the base of column specimens.

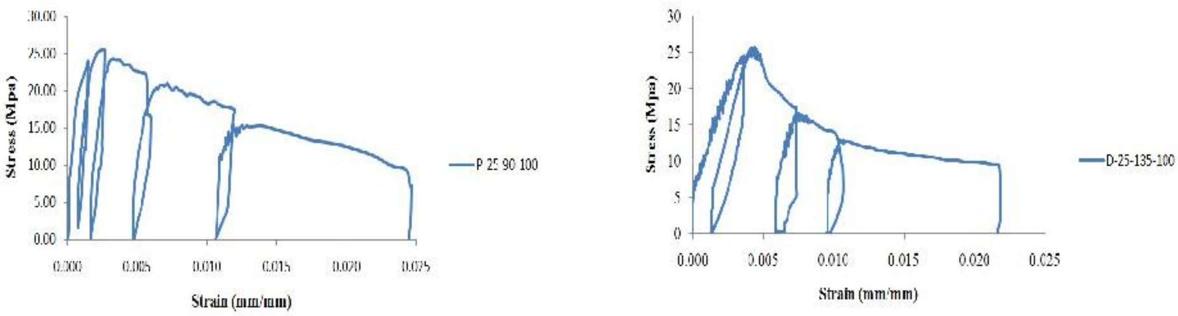

Figure 6. Effect of bend and anchorage of transverse reinforcement

This research presents results of some part an experimental research program carried out at the structural laboratory of University of Tehran on the seismic evaluation and assessment of old concrete columns reinforced by plain and deformed bars with understanding of effect of detailing on the axial capacity. According to the monotonic tests that were performed on four half-scale column specimens under axial loads, the following conclusions can be drawn from the results of these tests:

Damage pattern of concrete columns during the experiment shows that slip contribution is the major source of specimen deformation in old concrete columns with plain bar by 90 degree bending of transverse reinforcement with poorly-anchored.

Increasing the transverse reinforcement ratio significantly enhances the strength and toughness gains of the confined concrete. The transverse reinforcement ratio is the test variable with the most important beneficial effect on the stress-strain behavior of concrete.

An increase of the tie yield strength would result in an enhancement of the strength and toughness gains only for well-confined specimens with large ratios of lateral reinforcement.

The transverse reinforcements with 90 degree and 135 degree ends can effectively restrain the unsupported longitudinal reinforcement bars from buckling, which indirectly confine the core of the column.

The column specimen with non-seismic reinforcement detailing possesses low ductility capacity. Rapid strength and stiffness degradation have been observed during the experiment.

Although some research has been carried out by different researchers in recent decades on seismic structural evaluation concrete reinforced by plain bars, it seems that much more research is needed to prove the results obtained in this study, especially for concrete columns with moderate and high axial load. Comparative experimental study on the seismic test results of reinforced concrete columns with different details and bar type is recommended for future research to indicate how inferior are old columns with plain bars respect to those with deformed bars.

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