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THE EFFECT OF PLAIN BARS IN R/C BEAM-COLUMN JOINTS UNDER CYCLIC LOADING

Yılmaz YUVA
Assistant Professor
Department of Civil Engineering
Istanbul University, Avcılar
Istanbul, TURKEY.

received his Civil Engineering degree from Yıldız Technical University, Istanbul in 1985. He received his MS and PhD degrees from Boğaziçi University in 1989 and in 1996, respectively.

Gülay Altay AŞKAR
Professor and Head of Department
Department of Civil Engineering
Boğaziçi University, Bebek
Istanbul, TURKEY.

received her PhD in Civil Engineering in 1977 from Istanbul Technical University. Head of Civil Engineering Department since 1992 and Vice Dean of Engineering Faculty since 1995 in Boğaziçi University.

Summary

The research study presented herein is a part of the activities of Boğaziçi University involved in the (COST- C1) project. In scope of a collaborative research program between Boğaziçi University and CEC-JRC-ELSA, eight external beam-column joint subassemblages were tested in Ispra, Italy. The test results of three full-scale subassemblages fabricated using plain bars are presented herein. The results from the specimen tests were used to calibrate the hysteretic parameters of the code IDARC and a non-linear dynamic analysis of a low-rise reinforced concrete frame structure was performed under various effects using IDARC.

1. Introduction

The experimental part of the study is aimed to investigate the influence of the use of plain bars that have been used as main reinforcement in building construction in Turkey, to investigate the influence of joint transverse reinforcement on beam-column joint behavior and to investigate the behavior of adhesively bonded external steel plates under cyclic loading for the purpose of using this technique for structural members need to be strengthened in seismic regions.

In the analytical part, the results from the specimen tests were used to calibrate the hysteretic parameters of IDARC [1] which has been developed for inelastic damage analysis of reinforced concrete frame structures. Based on the results from the tests and other studies, non-linear dynamic analysis of an prototype reinforced concrete five-story frame structure, designed in accordance with the Turkish Codes, was performed using IDARC. Inelastic dynamic analysis for El Centro and Taft earthquakes were performed under various peak ground accelerations of 0.2g, 0.3g and 0.4g. The other variables are pinching parameter and P-Δ.

2. Experimental Program

Specimen configuration and testing methodology were similar for the entire full-scale component testing program involving experiments on bare and strengthened beam-column joints. Detailed experimental results are provided in [2][3].

All the specimens were so detailed that failure would occur in the beam. One of the specimens was detailed in accordance with the ACI "Recommendations" [4] and it will be named as "**ductile**". The specimen detail is shown in Figure 1. The second specimen was detailed as not conforming seismic detailing procedures on beam-column joints such as no transverse joint reinforcement, larger spacing of beam and column end region ties. The specimen will be named as "**non-ductile**" regardless of its behavior. The specimen detail is shown in Figure 2. The third one was detailed same as the "non-ductile" one and additionally, a strengthening scheme using adhesively bonded external steel plates was applied on the specimen to investigate behavior of this type of strengthening method under cyclic loading. The specimen will be named as "**strengthened**". The specimen detail is shown in Figure 3. The word "bare" is used as common for the "ductile" and "non-ductile" specimens.

2.1 Instrumentation

For all specimens, applied beam tip displacement and corresponding load, column axial load, eight transducer readings (four at beam end regions and four at column ends just above and below the joint region) were measured. In addition, 10 strain readings at "**non-ductile**" specimen, 12 strain readings at "**ductile**" specimen and 24 readings at "**strengthened**" specimen were measured.

Strains in the longitudinal, transverse and external reinforcements were measured at locations indicated in Figures 4, 5, and 6 for the "ductile", "non-ductile", and "strengthened" specimens, respectively. For the "strengthened" specimen, embedded strain-gauge locations are the same as the "non-ductile" specimen.

The testing system is shown in Figure 7. Displacement history applied at the beam end is shown in Figure 8. The locations of the LVDTs are shown in Figure 9.

2.2 Experimental Results

In this section, a brief summary of the observations recorded during the tests are presented.

2.2.1 Specimen BC⁻ "Ductile"

The specimen has the folowing characteristic details : Column reinforcement : 8ϕ14 continuous bars (not spliced), positive beam reinforcement : 3ϕ14 continuous bars, negative beam reinforcement : 3ϕ14 continuous bars, ties within the joint region : 6ϕ10, constant axial load on the column : 160 kN, measured average material strengths were f'_c= 19.3 MPa and f_y = 342 MPa, number of cycles applied : 27, number of recorded loading steps : 2700.

The first crack developed at the interface of the beam and joint panel. During the next cycles, the crack originating at beam end progressed further as reversing cyclic load was applied. The

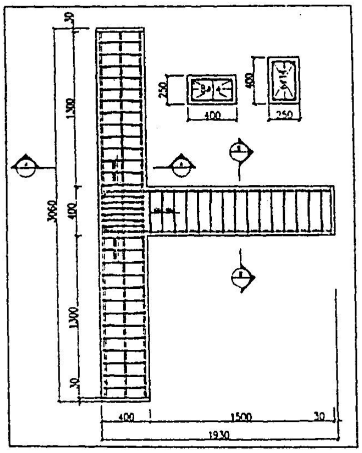

Figure 1. Details of Ductile Specimen

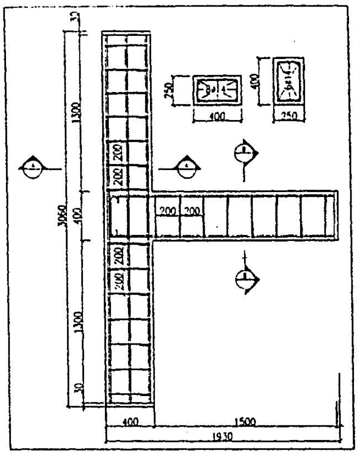

Figure 2. Details of Non-Ductile Specimen

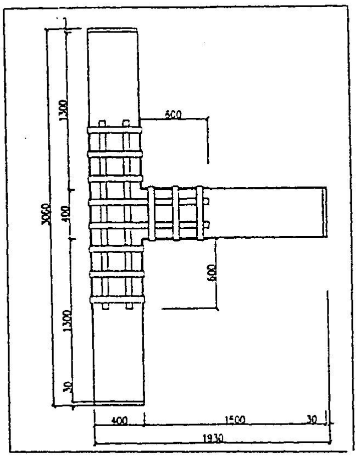

Figure 3. Details of Strengthened Specimen

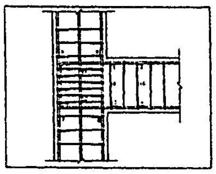

Figure 4. Strain-gages Ductile Specimen

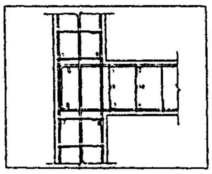

Figure 5. Strain-gages Non-Ductile Specimen

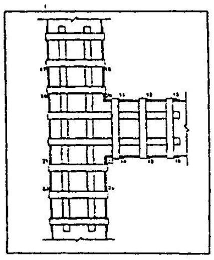

Figure 6. Strain-gages Strengthened Specimen

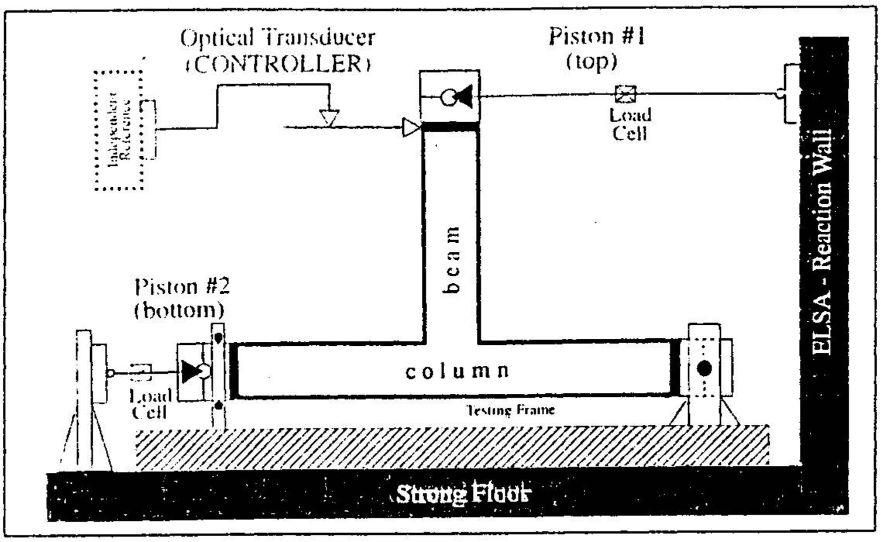

Figure 7. Schematic View of the Testing System

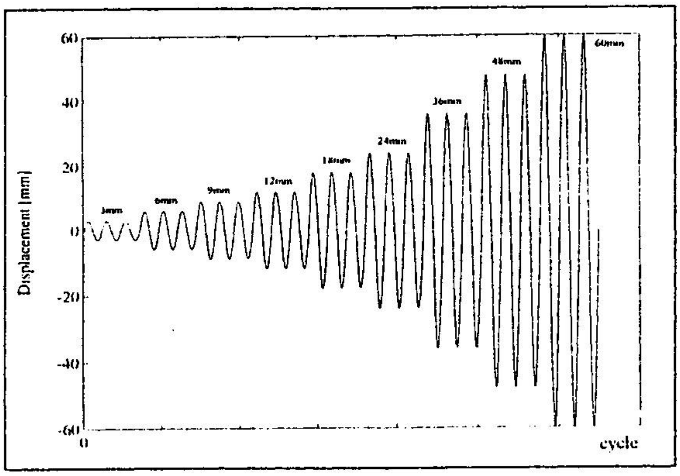

Figure 8. Loading History Applied at the Beam End

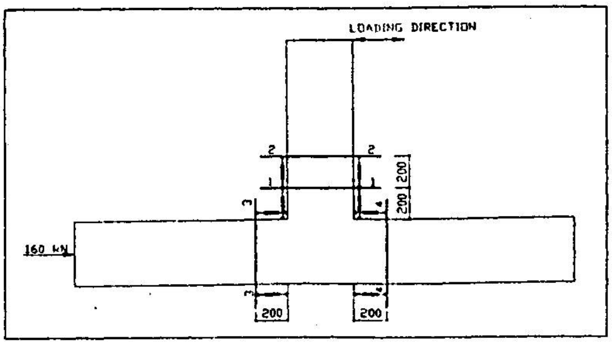

Figure 9. LVDT Locations

subsequent cycles caused further opening of this single crack. At 60 mm cycle, concrete cover spalling was initiated on the face opposite to the beam. No cracking was observed on the columns and the joint panel. The experiment was terminated when the crack was opened too much at 60 mm cycle and the LVDTs were contacted by spalling concrete cover.

2.2.2 Specimen BC6 "Non-ductile"
The specimen has the folowing characteristic details : Column reinforcement : 8ϕ14 continuous bars (not spliced), positive beam reinforcement : 3ϕ14 continuous bars, negative beam reinforcement : 3ϕ14 continuous bars, no ties within the joint region, constant axial load on the column : 160 kN, measured average material strengths were f'_c= 18.3 MPa and f_y = 342 MPa, number of cycles applied : 25, number of recorded loading steps : 2500.

Observations for the "non-ductile" specimen are almost identical to the "ductile" specimen. However, concrete spalling at the outer face of the joint was observed on "non-ductile" specimen since the beam longitudinal bars tend to open in joint region under high load levels and no transverse joint reinforcement is available in the joint region.

2.2.3 Specimen BC2 "Strengthened"
The specimen has the folowing characteristic details : Column reinforcement : 8ϕ14 continuous bars (not spliced), positive beam reinforcement : 3ϕ14 continuous bars, negative beam reinforcement : 3ϕ14 continuous bars, no ties within the joint region,external steel plates on the specimen, constant axial load on the column : 160 kN, measured average material strengths were f'_c= 22.1 MPa and f_y = 342 MPa, number of cycles applied : 22.5, number of recorded loading steps : 2250.

No crack was observed during the cycles of 3 mm and 6 mm. At the first cycle of 9 mm, a minor crack developed at the interface of the beam and joint panel, and also where the external plates are ended in the beam. The subsequent cycles caused further opening of the crack at beam plate end. For the safety of testing the experiment was terminated at one and a half cycle at 48 mm.

2.3 Load-Displacement Curves

The load-displacement curves are shown in Figure 10. The load values reached at extremes of 3 mm, 12 mm, 18 mm, 48 mm and 60 mm cycles are given averaging negative and positive displacement amplitudes as the following. At the first cycle of 3 mm for the "ductile", "non-ductile" and "strengthened" specimens, the measured load values are 12.88 kN, 14.02 kN and 21.68 kN, respectively. At the first cycle of 12 mm , the load values are 27.51 kN, 29.04 kN and 42.75 kN. For the "ductile" and "non-ductile" specimens, the load values are 29.12 kN and 29.04 kN, respectively, for the first cycle of 18 mm. For the "strengthened" specimen, a decrease in load level in tension side of the beam at the first cycle of 18 mm was observed. The measured load value at the third cycle of 12 mm is 38.95 kN while it is 38.36 kN at the first cycle of 18 mm. Instead of an increase in load level due to increase in displacement amplitude, approximately 1.5 percent decrease was measured. This can be attributed to partial seperation of the external plates from the concrete surface. Because of this decrease in tension side of the beam at the first +18 mm displacement amplitude the load values for the subsequent cycles are not given for the "strengthened" specimen. At the first 48 mm cycle, the load values are 30.34 kN and 30.42 kN. At the first cycle of 60 mm, the load values are 30.52 kN and 30.75 kN.

As seen from the load displacement curves, all three specimens are almost stabilized the load level after yielding, only slight strength degradation is observed for all specimens tested. The strength degradation is around 4 percent for each repeated cycles of same displacement amplitude. In other words, strength degradation between first and third cycle of same amplitude is around 8 percent.

2.4 Energy Dissipation

Taking the "ductile" specimen hysteretic energy dissipation as the reference (i.e. 100%), energy dissipation values for the other two specimens are given as the following. Hysteretic energy dissipated corresponds to the cumulative dissipated energy for the three repeated cycles of same displacement amplitude. For all three cycles of 3 mm, energy dissipation is 122% and 189% for the "non-ductile" and "strengthened" specimens, respectively. For all 3 mm and 6 mm cycles, the values are 86% and 151%. For the nine cycles up to 12 mm cycle, the values are 68% and 123% for the "non-ductile" and "strengthened" specimens, respectively. For the twelve cycles up to 18 mm cycle, the values are 69% and 111% for the "non-ductile" and "strengthened" specimens, respectively. It can be concluded from the dissipated energy values obtained from the tests in this study that energy dissipation in strengthening by using externally bonded steel plate technique decreases with increase in ductility. In other words, in high ductility demand, the strengthening technique does not provide significant contribution to energy dissipation capacity.

Hysteretic energy curve for the three experiments is shown in Figure 11. As shown from the figure that the most energy dissipation is occurred for the strengthened specimen. With regard to bare specimens, ductile specimen dissipated more energy than the non-ductile specimen.

3. Analytical Study

For the calibration of the load-displacement behavior of the bare specimens a parametric study was performed playing with IDARC Ver. 3.1 hysteretic parameters. The parametric study results are provided in [2]. The analytical load-displacement curve capturing the experimental load-displacement curve is shown in Figure 10.d.

3.1 Prototype R/C Frame Structure

In this study, to examine the influence of beam-column joints on the overall behavior of reinforced concrete frame structures during earthquakes a typical bent of a five-story frame structure was chosen as a case study. Only one way framing scheme of the model structure was considered.

3.2 Nonlinear Analysis of Prototype Structure

An analysis was made to determine the likely mode of failure (beam or column hinging, or joint failure) at each connection region in the model structure. The purpose of the analysis was to identify the weakest element of each connection region. The code IDARC Ver. 3.1 was used for the inelastic analysis of the structure. Two sets of IDARC parameters were defined for

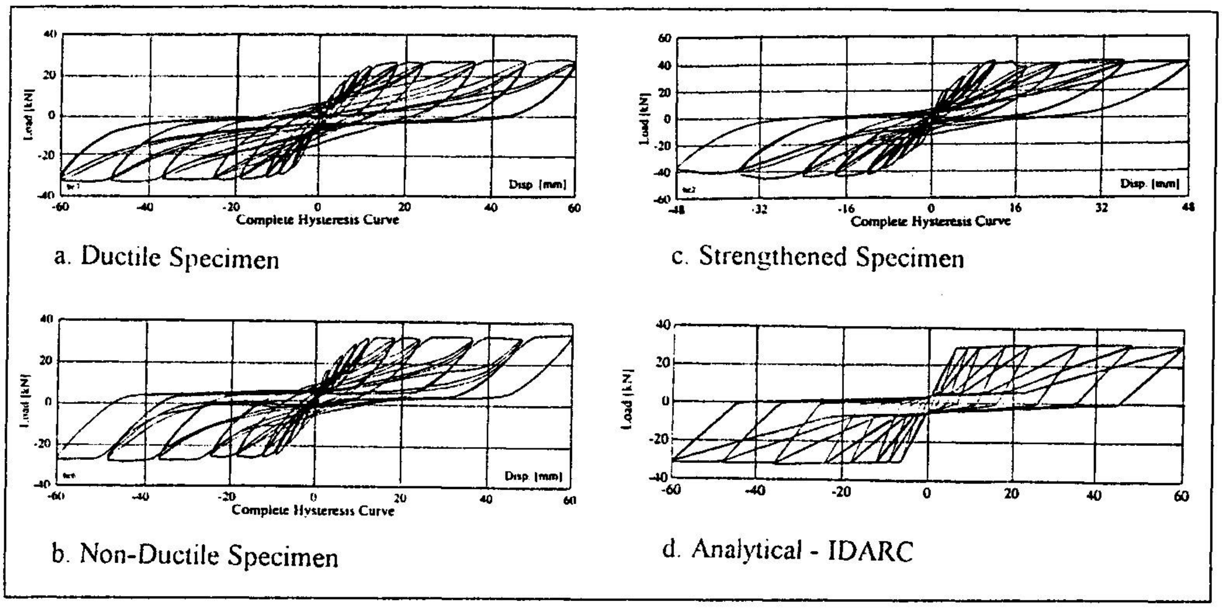

Figure 10. Load-Displacement Curves

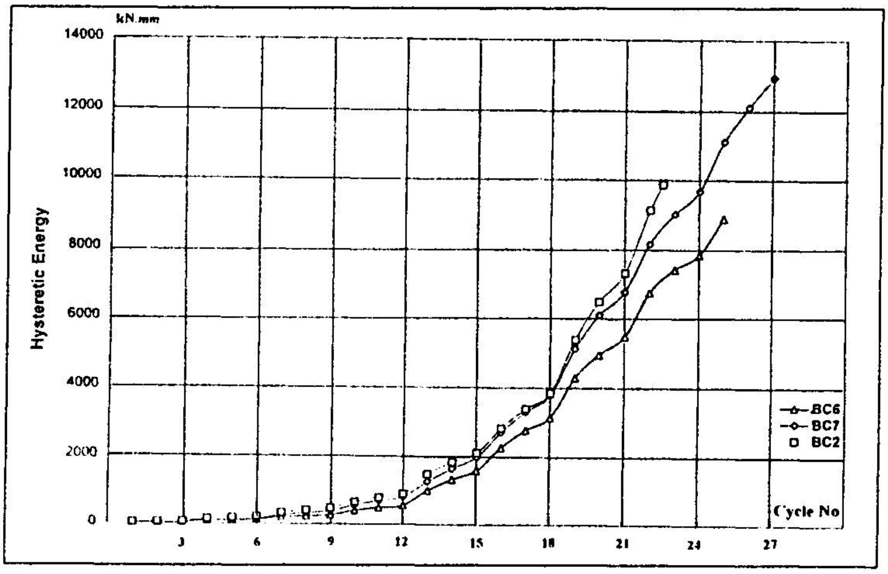

Figure 11. Hysteretic Energy Curves

the whole structure. One set of hysteretic parameters was provided for each end of beam and column members. The variables to be examined are the influences of P-Δ effect, bond characteristics of reinforcing steel and type of earthquake.

From a monotonic push-over analysis, which is an inverted triangular static loading on the structure until the drift level is achieved, based on a 2% top story drift limit of the total height of the structure a base shear capacity of 14% of the total weight W is computed.

IDARC is used to estimate the critical peak ground acceleration to scale the El Centro S00E and Taft S69E components to induce collapse on the example frame structure. The criteria used to determine the critical peak ground motion are related to the following variables: **i.)** base shear demand and capacity, **ii.)** the maximum interstory drifts, **iii.)** the damage index of the critical story columns and of the overall structure.

For this analytical evaluation, repetitive analyses were done using the El Centro 1940 S00E and Taft 1952 S69E earthquakes with the PGAs of 0.2g, 0.3g and 0.4g.

From nonlinear dynamic analyses, the story shears, story displacements and maximum interstory drifts for various PGAs of 0.2g, 0.3g and 0.4g were obtained. Including P-Δ effect, the maximum computed base shears for the El Centro earthquake are 301.2 kN (10.11% W), 388.3 kN (13.03% W), and 434.3 kN (14.57% W). For the Taft earthquake the values are 336.7 kN (11.3% W), 401.8 kN (13.48% W), and 432.6 kN (14.52% W) for the PGA values of 0.2g, 0.3g and 0.4g, respectively. Therefore, according to the model, the base shear capacity has been achieved in the 0.4g for both the El Centro and Taft Earthquakes with and without P-Δ effect.

For the El Centro S00E motion, the interstory drifts are exceeded for the 0.3g and 0.4g PGAs on the fifth floor. The interstory drift maxima for the first four stories remain within tolerable limits. The fifth floor drift values are 2.68% and 3.51% of the story height for the 0.3g and 0.4g PGAs without P-Δ effect. Including P-Δ effect, the values become 2.89% and 4.49%, respectively. The base shear demands are about 73% of capacity, 94% of capacity for the 0.2g and 0.3g PGAs, respectively. For the 0.4g PGA base shear demand reaches capacity.

Interstory drifts obtained during Taft S69E component are within reasonable limits. Only the fifth story drift value for the 0.4g PGA exceeds 2%. (2.24% and 2.17% of the story height with and without P-Δ effect, respectively). For this case, it can be said that P-Δ has no great influence on the structural performance with regard to maximum displacements, maximum interstory drifts and base shear capacity. However, the base shear force demands approach capacity very rapidly. For the 0.2g motion, the base shear demand is about 80% of capacity. For the 0.3g motion, the base shear demand is about 96% of capacity. For the 0.4g motion, the base shear demand reaches capacity. Therefore, collapse is very probable for the 0.30g PGA and inevitable for the 0.40g PGA.

The damage model in IDARC assesses the level of damage induced in the structure both at the component and story levels. This model assigns a damage index value between 0.0 and 1.0, with 0.0 indicating no damage and 1.0 representing total collapse.

Most of the damage occurs to the columns of the fifth floor. The maxima damage indices are 0.28 and 0.69 for the 0.2g and 0.3g PGAs, respectively. The damage index reaches severe damage state for the 0.3g motion. Also the overall structural damage indices for the 0.2g and 0.3g motions are 0.128 and 0.249, respectively, which indicate minor damage. Finally at 0.4g, the damage index for the fifth floor columns is between 0.98 and 1.02. Therefore, collapse of the top story columns is definite with 0.4g. Also the overall structural damage index for this motion is 0.361, which indicates moderate damage. Therefore from the response results and corresponding critical members and overall structural damage indices from IDARC, it is determined that the El Centro S00E component with a PGA of 0.4g would induce collapse of the fifth story columns. The resulting collapse mechanism is a column-sidesway mechanism.

The third criteria for determination of the critical PGA for collapse of the example structure is the damage index of the members and of overall structure. For the Taft earthquake, most of the damage occurs to the fifth story columns. Thus the resulting failure mechanism is that of column-sidesway collapse mechanism. The damage index maxima for the fifth story columns are 0.21, 0.35 and 0.54 for the 0.2g, 0.3g and 0.4g PGAs, respectively. The overall structural damage index values for the 0.2g, 0.3g and 0.4g PGAs are 0.105, 0.177 and 0.269, respectively. It can be concluded that the damage indices for the member and for the overall structure remain within minor through moderate-repairable damage state.

So far, all the results from IDARC are obtained using the pinching hysteretic parameter corresponding to plain bar obtained in this experimental study. Additional results are obtained with 0.3g PGA for both the Taft and the El Centro motions. The new pinching parameters are considered to simulate bond characteristics of the deformed reinforcing steel. In that case, most of the damage occurs to the columns of the fifth floor for the El Centro S00E motion. The damage indices are between 0.53 and 0.58 for the top story columns. In comparison to the damage indices obtained in the original case (i.e. using plain bar hysteretic pinching parameter) , the damage indices are decreased. Furthermore, the overall structural damage index is 0.237 which is less than 0.249. For the Taft earthquake, the damage indices are greater than 1.0 for all fifth story columns while the maxima is 0.35 with the original hysteretic parameters. Therefore, collapse of the fifth story columns is definite for 0.3g motion.

4. Conclusions and Recommendations

1. As in the current Code, the new draft Turkish Earthquake Code expected to be in use in 1996 does not contain any provisions and recommendations pertaining to the use of deformed bars. As clearly seen from the load-displacement curves that severe pinching occurs in use of plain bars. The draft Code therefore should restrict the use of plain bars as main reinforcement in the 1^{st} and 2^{nd} degree earthquake zones in Turkey.
2. It can be concluded from the "ductile" and "non-ductile" subassemblage tests results that for the cases with high flexural strength ratio and low joint shear stress, joint transverse reinforcement could be reduced without jeopardizing the performance of the frame.
3. It was observed from the results of the tests that high pinching occurred due to the weak bond behavior of plain bars although the specimens were designed in accordance with ACI "Recommendations". For the hysteretic behavior modeling purposes, the pinching factor in IDARC may be taken as around 0.2 even if the joint is in elastic range.

4. From the analytical study performed in this study, it has been realized that damage level at member level and for the entire system is very sensitive to hysteretic parameters chosen. Structure behavior is not influenced much by the hysteretic parameters. Mechanisms that will cause damage/failure (or collapse) at member level should be clearly identified prior to make a decision for the parameters.
5. Much more experimental study should be performed to determine more efficient plate dimensions and configuration, and also to understand all aspects of externally bonded steel plate technique under earthquake type loading. One conclusion can be drawn from the strengthened specimen test in this study that the plate length extending from column face to beam should be reduced (extend between one half of beam height - beam height).
6. The failure of strengthened beam-column subassemblage was occurred in the beam section where the external plates are ended, not at the column face as in the cases of bare specimen tests. Repair and strengthening work near joint region is much more difficult and joint failure influences the behavior of a member of a frame significantly. Methods to relocate plastic hinges away from joints is getting popularity to avoid the difficulties mentioned above. The strengthening technique applied in this study may be used an alternative method to move plastic hinges to an appropriate location.

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