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SEISMIC DESIGN CHARACTERIZATION OF RC SPECIAL MOMENT RESISTING FRAMES IN PAKISTAN-FIELD SURVEY TO LABORATORY EXPERIMENTS

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ABSTRACT

This study was undertaken to study the influence of construction deficiencies on hysteretic behavior of exterior beam-column connections. Building stock survey was conducted in five major cities of Pakistan for identification and quantification of material and detailing disparities between design specifications and construction practices. The effect of these disparities was studied using quasi-static cyclic testing on two exterior beam-column connections; EJ-1A (code-compliant) with no deficiencies and EJ-2A (non-compliant) incorporating all identified deficiencies. Damage patterns and hysteretic force-deformation behavior of these models is presented and the performance of the two specimens is compared. The study found that moderate to high deficiencies exist between design specifications and construction practices for the construction of RC buildings in Pakistan. The study also concluded that significant loss of strength and ductility is observed between code-compliant and non-code-compliant exterior beam-column connections subjected to reverse cyclic loading.

Key Words: *Building stock survey; Quasi-static testing; Beam-column connections.*

INTRODUCTION

Pakistan is located in moderate to high seismic zones[1]. The past two decades have seen a rapid growth in construction of reinforced concrete buildings in almost all major cities of Pakistan, particularly the federal capital and provincial metropolises. This has led to shift from construction of traditional masonry buildings to the more accommodative reinforced concrete buildings. This trend of shifting from low-rise masonry construction to mid-to-high rise reinforced concrete buildings continues because of shift of population from rural to urban areas. Unfortunately, the construction industry, particularly in small cities, has not been able to cope with the ever growing pressure of maintaining quality control procedures for construction of reinforced concrete buildings. This has resulted in construction of reinforced concrete buildings being constructed far below the design standards. This fact was manifested in the October 8, 2005 Kashmir earthquake wherein many public sector reinforced concrete buildings suffered moderate to severe damage. Several studies have been conducted for characterization of damage patterns during the October 8, 2005 Kashmir earthquake[2-4]. Damage patterns of reinforced concrete buildings included strong beam-weak column phenomenon, separation of infill panels from the lateral framing system and development of soft-story mechanism, amongst others. These studies pointed out deficiencies in construction practices adopted for construction of reinforced concrete buildings, particularly the quality of concrete and reinforcement detailing of beam-column connections.

In order to portray a true picture of the construction of reinforced concrete buildings constructed in Pakistan, this study was carried out to identify and quantify the construction deficiencies in construction of reinforced concrete buildings in Pakistan. Furthermore, the effect of these deficiencies on the seismic performance of buildings was studied by subjecting experimental models of exterior beam-column connections to reverse-cyclic loading and studying the hysteretic force-deformation behavior of code-compliant and non-code-compliant models.

CONSTRUCTION DEFICIENCIES

In order to identify and quantify the disparities that exist between design specifications and actual construction, a building stock survey was carried out in five major cities of Pakistan. These cities included the four provincial capitals Peshawar, Lahore, Quetta and Karachi and the national capital, Islamabad. The idea behind selection of these cities for survey was that these cities have experienced a steep trend of construction of reinforced

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concrete buildings. Moreover, construction of reinforced concrete buildings in these cities represent the median, not the worst-case scenario for this class of buildings. Several construction sites were visited in each city for observation of the prevalent construction practices. Since reluctance was shown by building site owners in allowing site visits, it was decided to supplement the survey data by interviewing construction professionals involved in construction of reinforced concrete buildings in Pakistan. These included site engineers, structural designers, private contractors and government employees representing clients. Survey forms were developed for this purpose seeking professionals' opinion regarding various parameters pertinent to quality of concrete and detailing of reinforcement, particularly seismic detailing.

After a thorough analysis of the authors' observations during the survey and response accrued from construction professionals, the following deficiencies were identified and quantified.

1. Compressive strength of concrete is about 30% less than the specified values. This implies that the minimum specified concrete compressive strength of 3000 psi, when cast in field, is in tune of 2000 psi (Figure 1).

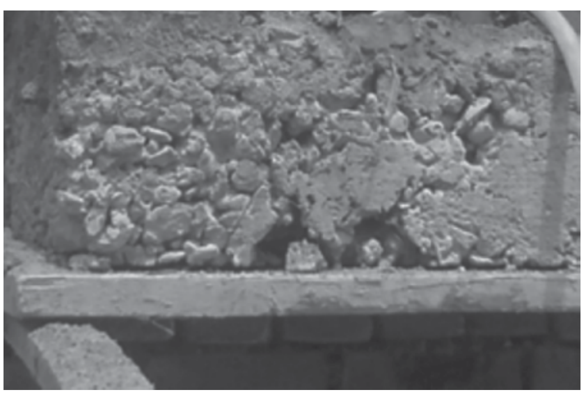


Figure 1. Honeycombed cast concrete.

2. Spacing of stirrups in beams and ties in columns is staggered (Figure 2) with an error approximately of 50%, on the higher side.

3. Majority of ties and stirrups are close-ended with 90^{o} bends instead of 135^{o} seismic hooks (Figure 3) as specified in design specifications.

Figure 2. Staggered spacing of stirrups in beams.

Figure 3. Insufficient hook length of ties in columns.

4. Lap splices are provided in columns near the beam-column connections contrary to the design specifications (Figure 4) which specify their provision at the point of contraflexure. Moreover, length of these splices is not according to the codal values but much less than what is specified by the code.

Figure 4. Improper location of lap-splices.

5. The reinforcement bars available in market have a diameter less than the nominal diameter of the rebar. This disparity is around 20% which when translated into actual bar sizes means that a 6/8 inch diameter bar in true effect has a diameter of 5/8 inch. This fact was manifested during experimental part of this work when the authors faced severe difficulties in acquiring standard size bars from local vendors.

under reverse-cyclic loading to compare their seismic resistance potential. Tests were carried out on the samples in accordance with the already published work on such models such as[5-7] amongst others.

DESCRIPTION OF EXPERIMENTAL MODELS

One part of the building stock survey included collection of structural drawings of buildings under construction. After studying the collected drawings, a low-rise, 3-story reinforced concrete building, designed according to Building Code of Pakistan-Seismic Provisions-2007 (BCP-SP 07) was selected as representative building. An exterior beam-column connection with all its detailing as regards geometry and reinforcement was decided upon to be used in experimental investigation. Two full scale models were prepared for quasi-static testing. The first model, EJ-1A, represented code-compliant model wherein the detailing was carried out in accordance with BCP-SP, 2007. Figure 5(a) shows detailing of model EJ-1A. Reinforcement in beam complied with requirements of special moment resisting frame (SMRF) of BCP SP-07 and consisted of 3, #6 bars (top and bottom) with 3/8 inch diameter stirrups spaced equally at 3 inch centre-to-centre. Reinforcement in column was 8, #6 bars distributed along the perimeter of the section and bound by 3/8 inch diameter stirrups spaced equally at 3 inch centre-to-centre. All the ties and stirrups were close-ended with 135° seismic hooks. Concrete used for casting the model had a compressive

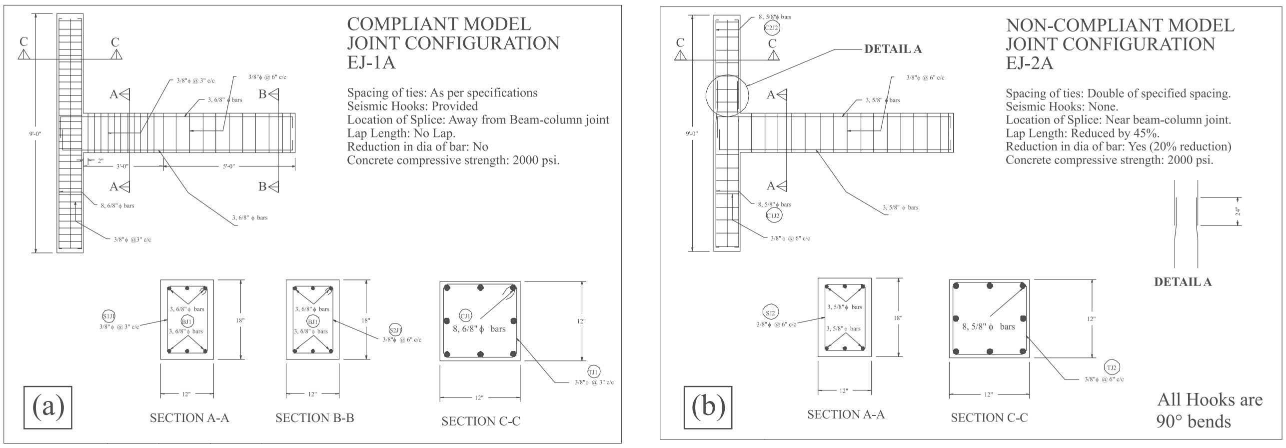


Figure 5. Reinforcement detailing of code-compliant beam-column connection EJ-1A (a) and non-code-compliant beam-column connection EJ-2A (b).

QUASI-STATIC TESTS ON EXTERIOR BEAM-COLUMN CONNECTIONS

Aim and Objectives of Quasi-Static Tests

Reinforced concrete special moment resisting frames in Pakistan are designed to the requirements of American seismic design code, Uniform Building Code-1997 (UBC-97). However, it was observed during the building stock survey that construction of RC buildings is not always executed according to the structural design drawings, especially the detailing of beam-column connections is not carried out according to the specifications, which is essential to the performance of special moment resisting frames. The main objective of quasi-static cyclic testing on exterior beam-column connections was to evaluate the true seismic resistance potential of reinforced concrete buildings with detailing deficiencies mentioned in the previous section. In this regard, two exterior beam-column connections, one fully compliant to the design specifications and the other completely non-compliant, based on results of building stock survey, were tested strength of 2000 psi reflective of the observations in the building stock survey. Reinforcement provided in the sample had a yield strength of 40000 psi.

The second model, EJ-2A, was termed as non-compliant model in which all deficiencies (as mentioned in building stock survey section) that were observed and quantified in the building stock survey were incorporated. Detailing of model EJ-2A are presented in Figure 5(b). Reinforcement in beam consisted of 3, #5 bars (top and bottom) representing a 20% reduction in the bar size based on results of building stock survey. 3/8 inch diameter stirrups were provided spaced equally at 6 inch centre-to-centre. Reinforcement in column consisted of 8, #5 bars distributed along the perimeter of the section and bound by 3/8 inch diameter ties spaced equally at 6 inch centre-to-centre. The ties and stirrups provided in column and beam of the sample were close ended with 90^{o} bends instead of 135^{o} seismic hooks. Concrete used to cast model EJ-2A had a compressive strength of 2000 psi. Grade 40 steel was used in construction of model EJ-2A having a yield strength of 40000 psi.

CONSTRUCTION OF SPECIMENS

All the models were constructed keeping in view the prevalent construction practices for reinforced concrete buildings in Pakistan. Timber formworks were prepared for casting of the model, however, to avoid loss of slurry during placement of concrete, the inner side of the formwork were lined with steel sheets. This also helped in achieving straight faces of concrete without any honeycombing or any other undesirable effect on quality of concrete. Subsequent to preparation of the models, concrete was cured for a period of 14 days by wrapping them up in hessian cloth and keeping them moist. Various stages of the specimen construction are portrayed in Figure 6 and Figure 7.

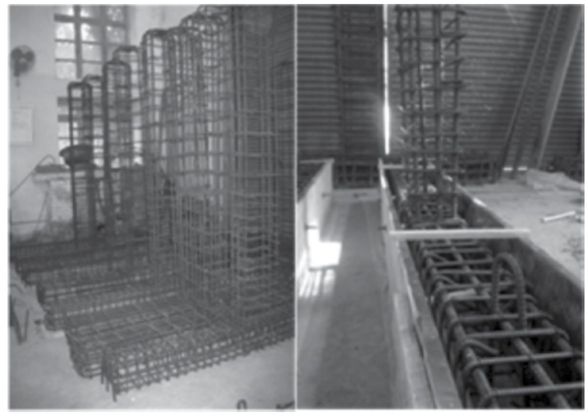


Figure 6. Preparation of reinforcement and placement in formwork.

Figure 7. Placement of concrete in formwork with proper compaction through mechanical vibration.

MODEL SETUP AND INSTRUMENTATION

Schematic of the specimen setup and constructed sample are shown in Figure 8. The specimen was tested in a vertical position with the column and beam lengths equal to the inflection point of the bay length and story height of the actual building.

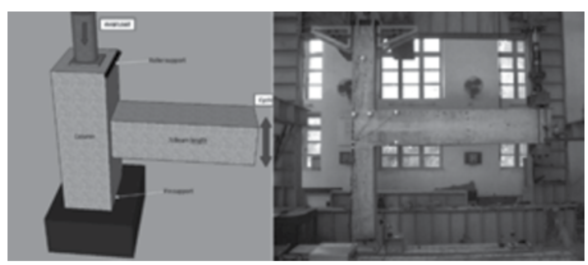


Figure 8. Schematic diagram of beam-column connection (left) and prepared and mounted beam-column connection sample ready for testing (right).

The bottom of the column was supported with a hinge, whereas the top of the column was restrained with side rollers to restrict lateral movement while allowing for vertical movement. Due to limitations in available testing equipment, specially designed side rollers and hinge support were fabricated and mounted on the model as shown in Figure 9. The column was loaded with axial load amounting to 15% of its axial capacity so as to ensure that it would act as a compression controlled section.

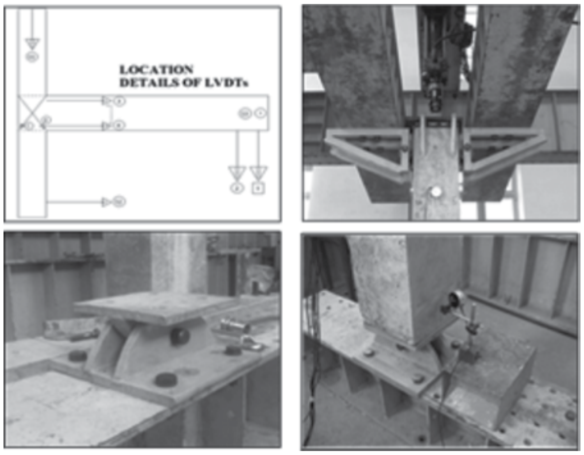


Figure 9. Specimen Instrumentation & support conditions

The specimen was tested under cyclic loading applied to the extreme end of the beam using a hydraulic jack which was pinned to the beam end to allow for rotation. The applied load was measured using load cell. The beam end rotation was measured using a displacement sensor. In order to record various parameters of interest, five linear variable displacement transducers (LVDT) were mounted on the model.

LOADING PROTOCOL

The seismic demand on the beam-column connection was simulated by cyclically loading the free end of the beam in a saw-tooth pattern as presented in Figure 10. Initially a force-controlled protocol was adopted till the theoretical yielding of the specimen. This was followed by displacement controlled till failure of the specimen. Three consecutive cycles were applied for each load or displacement increment.

DISCUSSION OF TEST RESULTS

The results of cyclic testing on the exterior beam-column connections are presented in the form of global force-deformation hysteresis loops and force-displacement envelope curves. The behaviour of specimens till failure is presented in the following section.

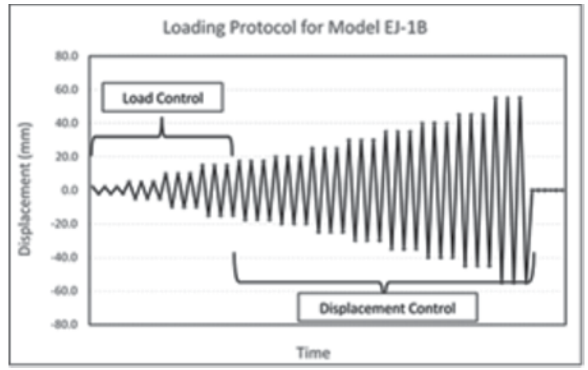


Figure 10. Loading protocol

Model EJ-1A

Cracking commenced on western face of the model at a load of 2.25 ton where a crack was observed to have started along the beam-column interface and projecting downwards along the whole length of the interface. At a load of 3.0 ton on the beam end a diagonal crack initiated at the middle of the joint region and progressed towards the right side. No further cracking was observed at this loading level. Increase in the loading level corresponding to 0.75% drift of the beam end, the beam experienced first crack at a distance of 12 inch from the beam-column interface which started at the top fibres of the beam and progressed vertically downwards till it terminated at mid-height of the beam. At the same loading level, a diagonal crack initiated at the left-most corner of the joint which progressed along the diagonal and terminated at the mid-height of the joint region. Damage pattern observed on western face of the model at end of loading level corresponding to 0.75% drift of the beam end is presented in Figure 11.

At 1.25% drift, a new crack was observed to have started in the top beam fibers at a distance of 7 inch from face of the column which progressed downwards at a sharp angle towards to the column and terminated at 10 inch from top beam fibers. The beam experienced no further cracking till a loading level of 2.0% drift. More damage was however observed in the joint region. The crack that had initiated at the diagonal at a load of 3.0 ton progressed downwards at a loading level of 1.0% and reached the bottom-right corner of the joint. The same crack progressed upwards towards the top-left corner at a load level of 1.0% drift thus completing the crack along the diagonal.

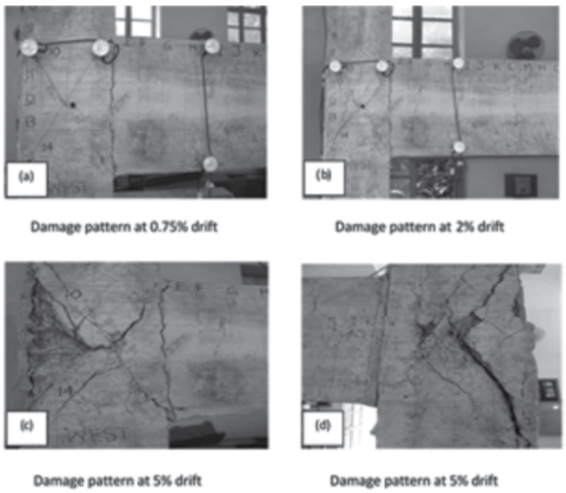


Figure 11. Damage pattern of model EJ-1A under increasing drift demand.

It can be clearly seen that damage, though initiated in the beam region, did not cause a failure mechanism in the beam but in the joint region thus signifying the vulnerability of the joint region in shear. Figure 12 and Figure 13 present the hysteretic response and cyclic envelope curve for specimen EJ-1A, respectively. It can be seen that the specimen response is severely pinched and loses its strength rapidly due to the joint shear mechanism.

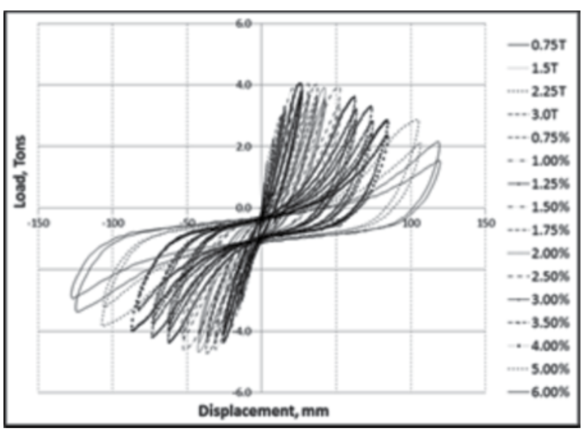


Figure 12. Hysteretic response of model EJ-1A.

Model EJ-2A

Specimen EJ-2A was tested in the same loading pattern as specimen EJ-1A. When the beam's end was loaded, no damage was observed during the expected yielding load in either the beam, column or the joint region. At loading level corresponding to 1.0% drift of the beam's end, first

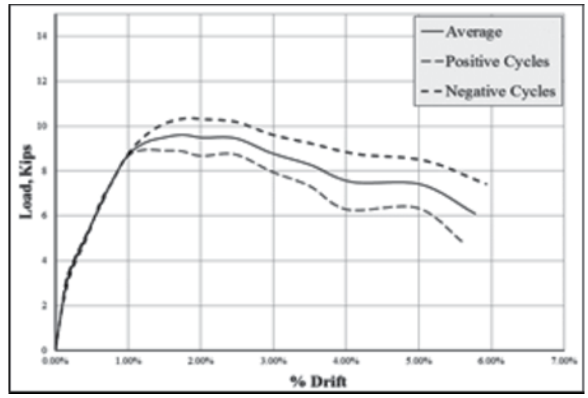


Figure 13. Force-Deformation backbone curve of model EJ-1A.

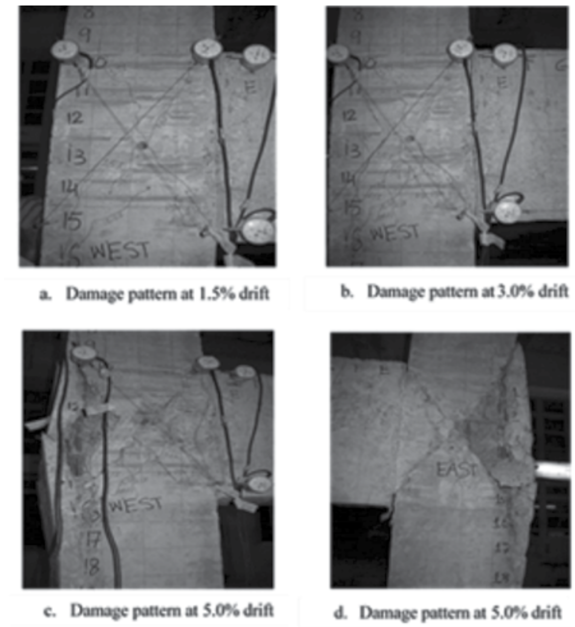
cracks appeared along the beam-column interface and at the diagonals in the joint region on western face of the model. Similar cracks were observed on eastern face of the model with cracks projected along the diagonals. The next significant damage in the model was observed at loading corresponding to 3.0% drift of the beam's end. On western face of the model, a number of new cracks initiated in the bottom part of the joint region with orientation of around 45° with the horizontal. Relatively less cracking was observed in top portion of the joint on western face of model EJ-2A. Damage on western face shows that no further cracks were developed at loading stages subsequent to loading at 3.0% drift of the beam's end. The cracks that were created in the joint region till 3.0% drift expanded in width.

The diagonal shear cracking formed a concrete wedge at free end of the joint region which eventually detached from the model. On eastern face, similar damage was observed. It can be seen that damage was limited to the joint region. Apart from the beam-column interface crack, no significant damage was observed in the beams. Damage patterns at various loading levels during testing of model EJ-2A are presented in Figure 14.

Figure 15 and Figure 16 present the hysteretic response and cyclic envelope curve for specimen EJ-2A. It can be seen that the cyclic response is characterized by severe pinching and rapid strength degradation.

Comparison of Test Results

The main objective of these tests was to quantify the difference between the seismic performance of



a. Damage pattern at 1.5% drift
b. Damage pattern at 3.0% drift
c. Damage pattern at 5.0% drift
d. Damage pattern at 5.0% drift

Figure 14. Damage pattern of model EJ-2A under increasing demand.

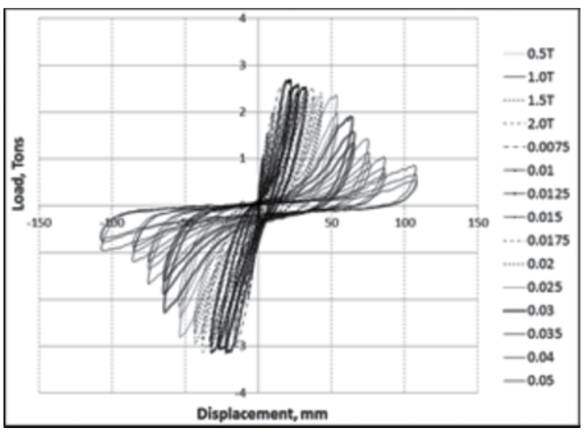


Figure 15. Hysteretic response of model EJ-2A.

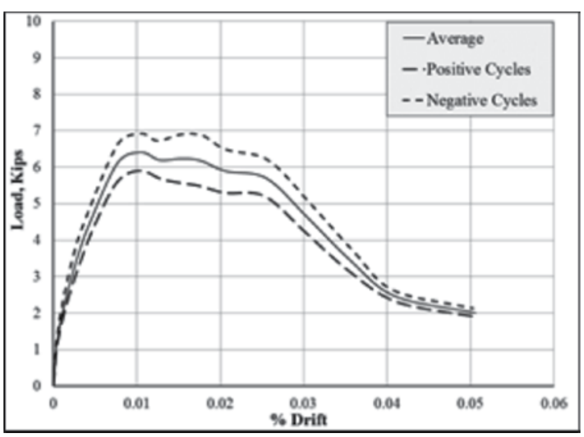


Figure 16. Force-deformation backbone curve of model EJ-2A.

specimen executed according to the design specifications (EJ-1A) and specimen with the above-mentioned construction deficiencies (EJ-2A). It is interesting to note that the observed damage pattern was similar for (EJ-1A) and (EJ-2A). In both specimens, damage started at the beam-column interface and eventually failed in joint shear mechanism which led to a severe pinching response and low energy dissipation. Minimal damage occurred in the beams till the failure of the specimens. However, the code-compliant specimen, EJ-1A, did perform better in terms of load carrying capacity due to the larger diameter of longitudinal bars and the presence of ties in the beam-column joint region. It can be seen from the force-displacement plots that the strength and stiffness degradation was rapid for the non-compliant specimen, as compared to the compliant specimen, due to the absence of the required number of ties in the joint panel. 3. At the peak displacement demand, specimen EJ-1A could sustain 53.4% of the peak load carrying capacity, whereas, specimen EJ-2A sustained 37.3 % of its peak load carrying capacity.

CONCLUSIONS

This study aimed at the seismic design characterization of RC special moment resisting frames constructed in Pakistan. The characterization consisted of a field survey of the existing RC frames in five different cities of Pakistan and the evaluation of seismic performance of RC frames proportioned and detailed based on the observations during the field survey. Following are the conclusions derived from this study:

Field Survey:

1. The compressive strength of concrete achieved on construction sites is about 30% less than the specified values.

2. Spacing of stirrups in beams and ties in columns is staggered and err approximately 50%, on the higher side from the design specifications.

3. Majority of ties and stirrups are close-ended with 90^{o} bends instead of 135^{o} seismic hooks as specified in design specifications.

4. Lap splices are provided in columns near the

beam-column connections contrary to the design specifications which specify their provision at the point of contra-flexure.

5. Length of lap-splices is not provided according to the codal values but much less than what is specified by the code.

6. The reinforcement bars available in market have a diameter less than the nominal diameter of the rebar. This disparity is around 20% which when translated into actual bar sizes means that a 6/8 inch diameter bar in true effect has a diameter of 5/8 inch.

Quasi-Static Cyclic Testing of Exterior Beam Column Connections:

1. Code-compliant specimen, EJ-1A, exhibited a higher load carrying capacity as compared to non-compliant specimen, EJ-2A.

2. The strength and stiffness degradation with increasing displacement cycles was rapid for non-compliant specimen as compared to the code-compliant specimen.

3. At the peak displacement amplitude, specimen EJ-1A could sustain 53.4% of the peak load carrying capacity, whereas, specimen EJ-2A sustained 37.3 % of its peak load carrying capacity.

The design provisions for exterior beam-column joints need a re-consideration as the code-compliant specimen failed in an unfavorable brittle joint shear mechanism as opposed to the expected beam flexure hinge mechanism which is the preferable mode of damage for code-compliant beam-column connections.

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