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GRAVITY LOAD-DESIGNED CONCEALED WIDE BEAM-NARROW COLUMN CONNECTIONS: EXPERIMENTAL ASSESSMENT OF SEISMIC RESPONSE

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Abstract. *Reinforced concrete (RC) wide concealed beam-narrow column connections represent the predominant structural system of building structures in Lebanon and the region. For a long time, engineers have paid little attention to the seismic design of these connections, despite their critical role in transferring internal forces when subjected to lateral earthquake load. A two-part experimental study is carried out for evaluating the seismic behaviour of these connections. The first part concentrates on the seismic performance of the connections when designed and detailed for gravity loads (as built) in accordance with local design and construction practices, while the second part focus on the seismic behaviour of the same connections when the reinforcement is detailed more properly against earthquake loads. This paper presents the results of two as built full-scale concealed beam-narrow column connections tested in the first part of the investigation. The connections experienced shear failure within the joint core at relatively small drift ratios varying between 1.5% and 2.0%, and before yielding of the beam or column reinforcement. At drift ratios of about 4.0%, corresponding to drift demands of building structures in regions of high seismic hazard, the connections experienced excessive and un-reparable damage within the joint core. The findings of this first part of the investigation guided the second part for introducing more adequate reinforcement detailing for improving the seismic performance of such type of connections.*

1 INTRODUCTION

The predominant building structural system in Lebanon and neighbouring countries is composed of a monolithic flexible ribbed (joist) one-way slab supported on wide concealed (shallow) beams. The beams frame into narrow columns having section aspect ratio between two to four so as to be merged with the infill masonry block walls. The long side of the column could be either parallel or normal to the axis of the beam. Being shallow or concealed the beams have to be quite wide for resisting the shear forces and bending moments produced by the applied loads. The corresponding structural system offers several advantages over the conventional slab-drop beam system in that the concealed beams and narrow columns are preferred by architects and interior designers as they are less obstructing and provide more flexible space. Also, concealed beams (having the same depth as the slab) lead to a more economical formwork, they minimize story heights, and, being wide, they reduce reinforcing steel congestions.

Unfortunately, despite being a region of moderate to high seismic hazard, [1, 2], the majority of building structures in this part of the world, particularly those between 5 and 12 stories in height, are designed with no regards to the seismic activities in the region. Accordingly, the concealed beam–narrow column connections, which constitute an important part of the building structural system, are designed to resist gravity load only. That is, the columns in these connections are designed to resist pure axial compression, with little or no attention to the moment transfer between the floor beams and the columns, and at each floor level, the column reinforcement is spliced with the reinforcement extending from the column below and tied with minimum ties required for gravity load design only. Also, the concealed beams in these connections are designed to resist flexure induced by gravity loads, where the beam positive (bottom) reinforcement on either side of the connection is terminated within a very short length inside the joint core or beyond the beam-column interface section. Furthermore, the core of these joints is constructed with no confinement reinforcement usually required in seismic design.

Little research has been conducted on shallow or wide beam - narrow column connections, [3, 4]. Consequently, most design codes place restrictions on the use of wide beam framing systems in seismic regions because there are insufficient information about their behaviour under the effects of earthquake loadings. A limited number of studies have been carried out on wide shallow beam-column connections, [5, 6, 7, 8], but most of these studies have concentrated on connections that are already designed for earthquake loads and which to a large extent satisfy the dimensions limitations set forth in recognized codes of practice. Consequently the experimental observations and conclusions reported in these studies may not be applicable to the connections under investigation.

A two-part experimental research program is being carried out at the American University of Beirut for evaluating the seismic response of concealed wide beam-narrow column connections. The first part deals with joints that were designed and constructed for gravity load (as built) in accordance with local design and construction practices. The second part, the testing of which is still underway, focus on the seismic behaviour of the same connections when detailed more properly for lateral earthquake loads. In the first part of the investigation, two types of as built beam-column sub-assemblages, representing local construction practices, were tested under lateral earthquake loads. One type in which the long dimension of the narrow column is normal to the direction of lateral load (or beam axis), while the second type is when the long dimension of the column is oriented parallel to the direction of lateral load (or beam axis). This paper presents the experimental results of the first type.

2 EXPERIMENTAL PROGRAM

The dimensions and reinforcement details of two joint specimens, one interior (IJ-F2) and one exterior (EJ-F2), tested in this investigation, are provided in Fig. 1 and are summarized for convenience in Table 1. The connections are composed of 800 mm wide by 250 mm deep beams framing into 700 mm wide by 250 mm deep column for Specimen IJ-F2 and 650 mm wide by 200 mm deep column for Specimen EJ-F2. The connections represent the frame of actual building structures that were designed for gravity load and detailed in accordance with local construction practices.

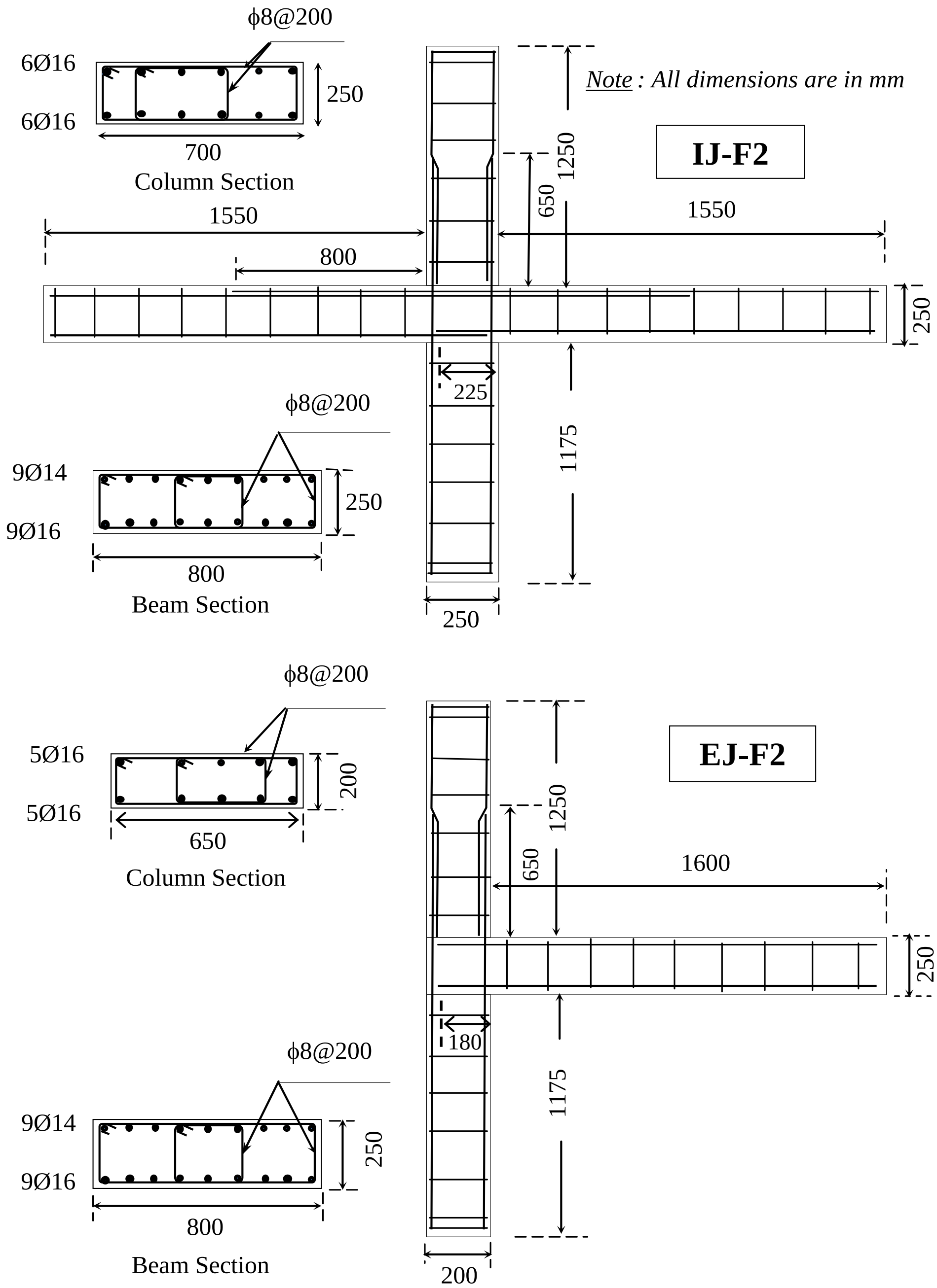

Figure 1: Dimensions and reinforcement details of the specimens

Joint Component	Dimension and steel reinforcement	**IJ-F2**	**EJ-F2**
Column	Size ($b_c x h_c$) mm	700x250	650x200
	Reinforcement & Reinforcement ratio (%)	12ϕ16 (1.37%)	10ϕ16 (1.53%)
	Transverse reinforcement	4 legs ϕ8@200	4legs ϕ8@200
Beam	Size($b_w x h_b$) mm	800x250	800x250
	Top reinforcement & Reinforcement ratio	2x9ϕ14 (1.65%)	9ϕ14 (0.83%)
	Bottom reinforcement & Reinforcement ratio (%)	9ϕ16 (1.08%)	9ϕ16 (1.08%)
	Transverse reinforcement	4 legs ϕ8@200	4 legs ϕ8@200
Joint	Shear reinforcement	None	None

$b = \sec tion\ \ width, h = \sec tion\ \ depth$

Table 1: Dimensions and reinforcement of the joint specimens

Some of the specific reinforcement details that are pertinent to local construction practices include: (i) the lap splice length (of 650 mm) at which the column reinforcement is spliced above the floor level; (ii) the embedment length of the beam bottom reinforcement inside the joint core of 225 mm for the interior and 180 mm for the exterior joint, respectively; (iii) the distance at which the beam top reinforcement is extended on either side of the interior connection beyond the beam-column interface (of 800 mm); and (iv) the anchorage length (of 180 mm) of the beam top and bottom reinforcement inside the exterior connection core beyond the beam-column interface section.

Commercially available Grade 60 steel bars (design yield strength of 415 MPa) having 14 mm and 16 mm diameter were used as the main beam and column longitudinal reinforcement. The measured yield strengths of the steel bars were 627 MPa for the 14 mm and 545 MPa for the 16 mm bars, respectively. The transverse steel reinforcement in both the beams and the columns consisted of plain 8 mm diameter bars having yield strength of 284 MPa.

Both specimens were cast using one batch of Ready Mix concrete. The concrete mix was prepared using Portland cement type I, coarse aggregate with maximum aggregate size of 19 mm and natural sand. The mix was designed for producing a target 28-day cylindrical concrete compressive strength of 20 MPa. The actual compressive strength, measured using standard 150 x 300 mm cylinders, was 21.0 MPa.

2.1 Testing setup and measurements

A schematic view of the test setup is given in Fig. 2. Each connection sub-assemblage was supported on steel fixtures to simulate a statically determinate structure with hinge points to represent points of inflection for the actual building structure when subjected to lateral loads. A 500 kN capacity hydraulic actuator with a maximum stroke of ±127 mm in tension and compression was used to generate cyclic displacements. Axial force was applied at the top of the column using a hydraulic jack. The jack force was transmitted to the column through four steel rods tied to steel plates at the top and bottom of the column. The magnitude of the column axial force was approximately $0.1 f'_c\, A_g$ for Specimen IJ-F2 and $0.15 f'_c\, A_g$ for Specimen EJ-F2 where A_g is the gross area of the column section. The loading history for all specimens was composed of a sequence of displacement-controlled cycles as shown in Fig. 3, given in *percent* of story drift or drift ratio *DR*. The drift ratio *DR* is expressed as $DR,\% = \frac{\Delta_\ell}{h_o} x100$ where Δ_ℓ is the lateral drift at the point where the actuator load is applied, and h_o is the story height or height of the column (measured from the point where the lateral load is applied to the bottom pin support of the column) which is equal to 2.8 m for both specimens. The loading history is composed of two cycles at each drift ratio which varied between 0.5% and 4.5%. Notice that while a maximum drift ratio of 4.5% is considered as a satisfactory story drift in regions of high seismic hazard, the stroke limit of ±127 mm of the hydraulic actuator used to apply the lateral load and the size of the specimens (column or story height) did not allow symmetrical drift history beyond 4.5%. Nevertheless, specimen IJ-F2 was tested beyond 4.5% using two stages of applied load: in the first stage (Stage 1), the specimen was subjected to cyclic displacements up to a maximum drift ratio of 4.5% (lateral drift of ±127 mm) in accordance with loading history of Fig. 3. In the second stage (Stage 2), the specimen was subjected to half-cycle displacement protocol ranging between a drift ratio of 5.0% (lateral drift of 140 mm) and a drift ratio of 8.5% (lateral drift of 240 mm). This was achieved by readjusting (retracting) the position of the actuator for utilizing its full stroke capacity in one direction.

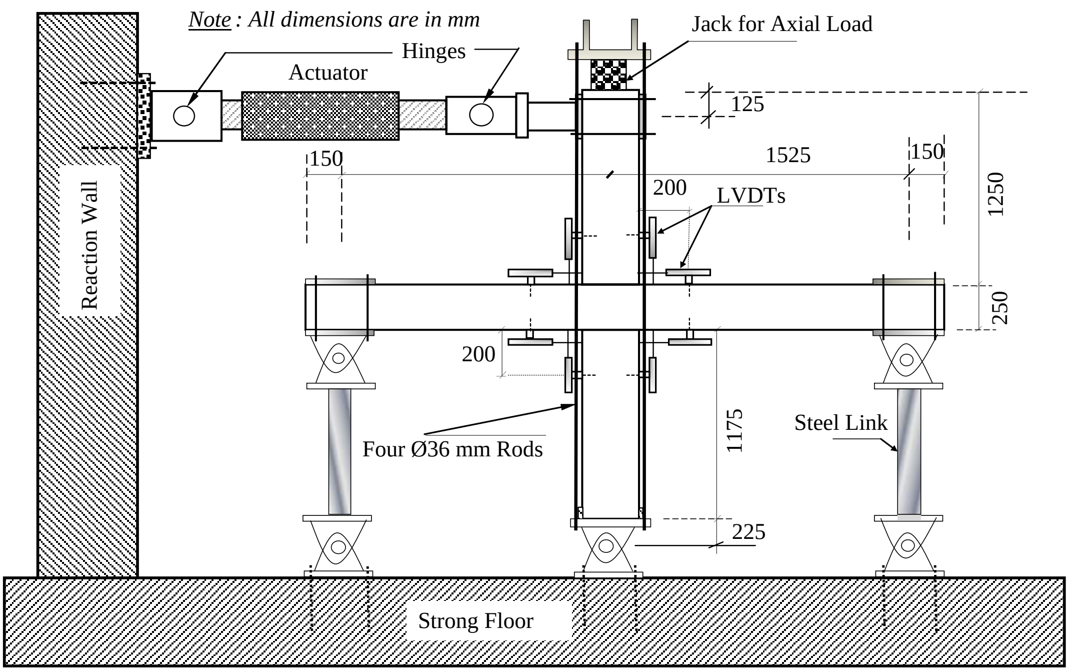

Fig. 2: Test setup

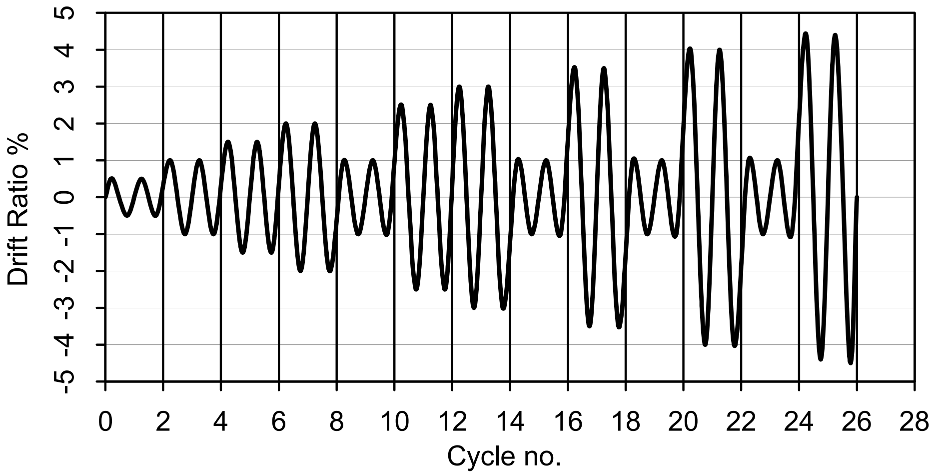

Fig. 3: Loading history

Test measurements included applied load and lateral drift, strain in the outermost column reinforcement (starter bars) at the column-beam interface, strain in the top and bottom longitudinal bars of the beams, and average curvature or rotation in the end zones of the columns and beams. The strains were measured using electric strain gauges, while curvatures were measured using Linear Variable Differential Transducers (LVDTs). The test data was recorded using a data acquisition and control system.

Figure 4: Failure mode of the joint specimens

3 TEST RESULTS

3.1 General behaviour and mode of failure

(i) Specimen IJ-F2

A typical photo of the specimen taken at the conclusion of the test is shown in Fig. 4. As indicated earlier, this specimen was subjected to two stages of load application (Stages 1 and 2). The first flexural crack in this specimen developed in the beam at the joint interface section at a drift ratio of 1.0%. As the lateral drift increased to somewhere between 1.5% and 2.0% diagonal shear cracks started to develop mainly in the joint core indicating the initiation of a joint shear failure. These diagonal cracks continued to spread and widen as the lateral drift increased and to propagate outside the joint core a distance equal approximately a beam depth away from the beam-column interface section (Fig. 4). However, at the end of Stage 1 of the test, corresponding to a maximum drift ratio of 4.5%, despite the development of shear cracks, the specimen did not show signs of strength degradation. In fact, during Stage 2 of the test, the specimen continued to resist lateral load steadily until a drift ratio of 7.0%, beyond which the specimen started experiencing gradual strength degradation. The width of the shear cracks at the maximum drift ratio in Stage 2 of load application reached 6 mm, causing considerable concrete damage.

During the first stage of load application, bond splitting cracks appeared along one of the outermost bottom beam bars on one side of the beam at a drift ratio of about 4.0%. During the second stage, splitting cracks developed as well along the outermost bottom bar on the other side of the beam. At the end of the test in Stage 2, these splitting cracks widened leading to concrete spalling at the bottom corners of the beam (Fig. 4).

(ii) Specimen EJ-F2

The first flexural cracks in specimen EJ-F2 appeared in the beam at the beam-column interface sections at drift ratios of about 1.0%. As the drift increased between 1.0% and 1.5%, the lateral applied load increased gradually, and at the same time, diagonal shear cracks started to develop within the joint core indicating the start of joint shear failure. These diagonal cracks continued to spread and widen as the lateral load increased until the development of a complete joint shear failure which occurred in this specimen at a drift ratio of approximately 1.75% to 2%. The width of the shear cracks at this stage reached between 2 to 3 mm. Beyond a drift ratio of 2%, the specimens experienced progressive strength degradation towards the end of the test. At maximum drift ratios of about 4.5%, the width of the shear cracks increased to about 6 mm, leading to a substantial concrete damage within the joint core (Fig. 4).

While the shear cracks concentrated mainly within the joint core, beyond a drift ratio of 2.0% the cracks tended to propagate outside the joint core similar to Specimen IJ-F2. Although few flexural cracks developed in the beam close to the beam-column interface, the number and width of these cracks stabilized as soon as the diagonal cracks in the joint core started to prevail.

During the tests, at drift levels of 3.0%, bond splitting cracks developed along the beam bottom longitudinal reinforcement outside the column core (one corner bar from each side). These cracks propagated along the full embedment length of the bars beyond the beam-column interface section, of 180 mm (Figs. 3). Eventually these cracks led to spalling of the concrete at the bottom face of the beam indicating bond splitting failure in localized regions of the specimens. This was regarded however as a secondary mode of failure as the development of these cracks did not affect the overall response of the specimens

3.2 Column shear-drift response

The hysteresis response of applied lateral load (column shear force) versus drift ratio are presented in Fig. 5 for both the interior and exterior specimens, and compared with the estimated maximum lateral load capacities H_{max} of the connections. The estimated capacities H_{max} were obtained using force equilibrium of the whole connection sub-assemblages calculated as the minimum lateral force required for developing the nominal positive or nominal negative flexural strength of the beam, respectively, or nominal flexural strength of the column. As indicated earlier, the specimens experienced connection shear failure at drift ratios varying between 1.75% and 2.0%.

For Specimen IJ-F2, despite the development of joint shear cracks at a drift ratio of 2.0%, the specimen continued to resist lateral loads at a small rate reaching a load of 72.0kN at a drift ratio of 4.5% (end of the test in the first loading stage). The corresponding lateral load resistance is 59% of the estimated static lateral load capacity H_{max} of the specimen of 122 kN. It is interesting to observe in Fig. 5 that even subjecting the specimen to 8.5% lateral drift in a half-cycle mode, it did not lead to significant strength degradation. The low strength degradation of this specimen below the load at which shear failure occurred may be attributed to the relatively low lateral load at which the specimen failed in shear (of about 56 kN) combined with relatively low shear decay associated with stabilization of the shear cracks within the joint core. The stiffness of the specimen (defined as the slope of the line that passes from the origin to the point on the envelope response) decreased to about 50% of the initial stiffness (stiffness at initial loading) at drift ratio of 4.5% (Stage 1) and 25% of the initial stiffness at drift ratio of 8.0% (Stage 2), respectively.

After the initiation of shear cracks at a drift ratio of about 1.75%, Specimen EJ-F2 displayed a stable behaviour without loss in load resistance until reaching a drift ratio of 2.5%. The peak lateral load developed in this specimen was approximately 20.0 kN which is only 35% of the estimated static lateral load resistance H_{max} of 54 kN. Beyond a drift of 2.5%, the lateral load resistance degraded gradually, reaching at 4.0% drift about 50% of the peak load. Also, at a drift ratio of 4.0%, the stiffness of the specimen decreased to about 17.0% of the initial stiffness.

3.3 Reinforcement strains/stresses and bond performance

3.3.1 Beam reinforcement

Cyclic response of the beam reinforcement strain with lateral drift of the specimens is presented in Fig. 6. As a result of the premature connection shear failure, the strains developed in the steel reinforcement were well below yield. The maximum tension strains of the beam bottom and top bars at peak lateral load were 1536 $\mu\varepsilon$ and 1847 $\mu\varepsilon$ for Specimen IJ-F2; and 1435 $\mu\varepsilon$ and 1670 $\mu\varepsilon$ for Specimen EJ-F2, respectively.

As indicated earlier, both of Specimens IJ-F2 and EJ-F2 developed localized bond splitting cracks in the outermost beam bottom bars outside the column core. Some of the main reasons for the localized bond splitting failure are the short development length provided for the beam bottom bars beyond the beam-column interface section, and the lack of concrete or steel confinement for the bars outside the column core. Notice that the development

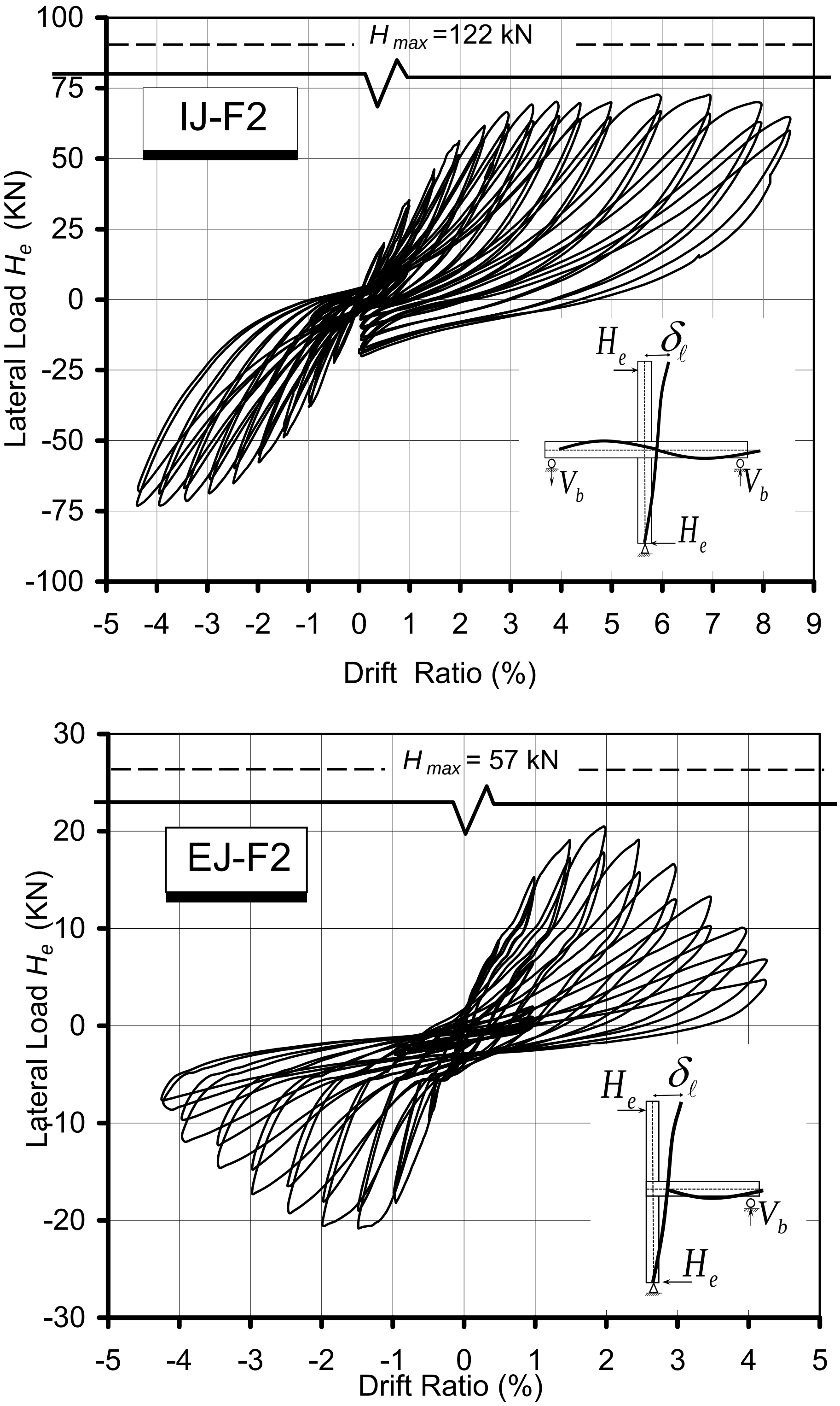

Figure 5: Lateral load-drift ratio response of the specimens

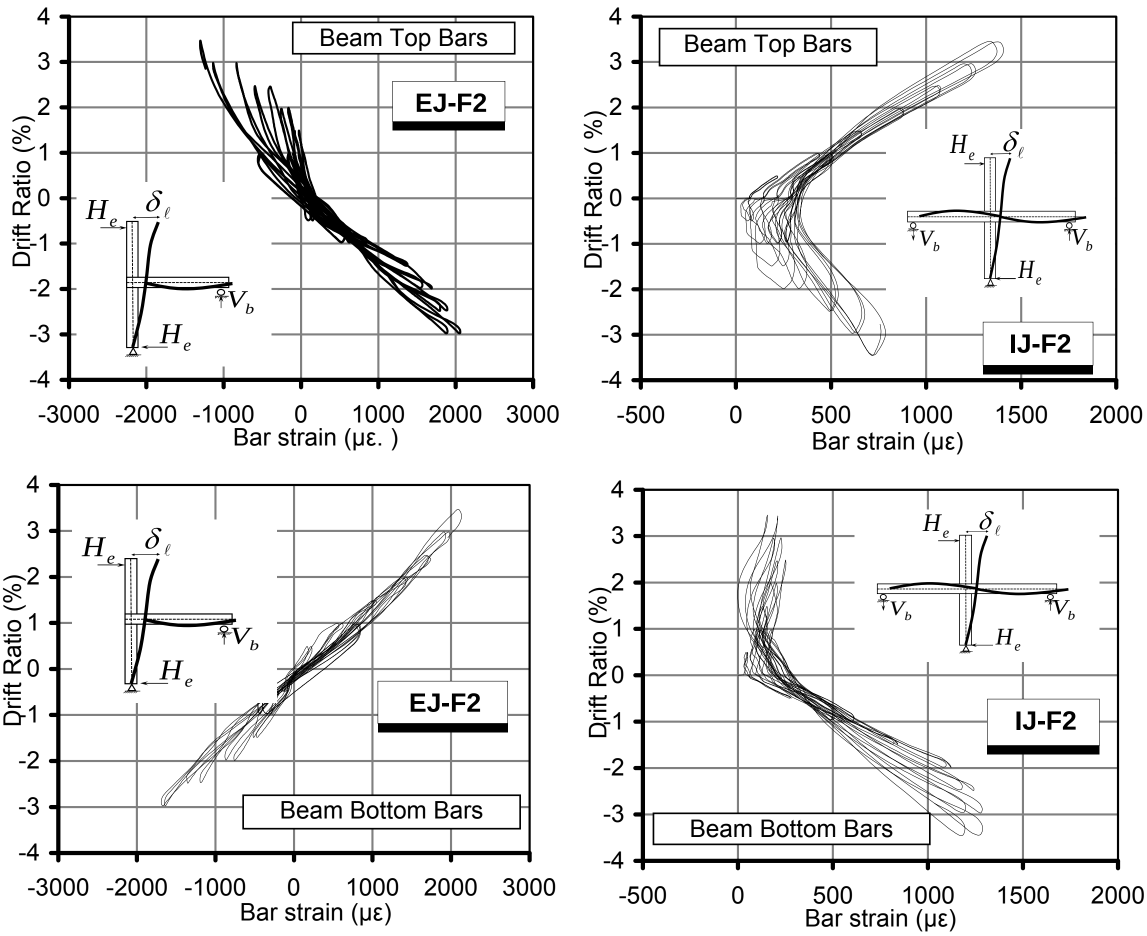

Fig. 6: Variation of beam reinforcement strain with drift ratio

lengths provided for the beam bottom bars were 225 mm or $14.0d_b$ for Specimen IJ-F2, and 180 mm or $11.3d_b$ for EJ-F2, respectively. These development lengths are less than the minimum of $27d_b$ required by ACI 318-08 for f'_c = 21 MPa.

3.3.2 Column reinforcement

The maximum strain developed in the column reinforcement of Specimen EJ-F2 was 2630 $\mu\varepsilon$, which is below yield. However, during the second stage of loading, the outermost left column bar in Specimen IJ-F2 did indeed develop yielding and attained a measured strain of 4545 $\mu\varepsilon$ at a drift ratio of 5.0%.

It should be mentioned that, based on test observations, all of the column reinforcement for Specimen EJ-F2 did not exhibit bond deterioration associated with bond splitting and/or excessive bond slip. However, during the second stage of loading for Specimen IJ-F2, a bond splitting crack was observed along one the corner column bars, but it was not critical as it did not result in a noticeable change of the specimen behaviour. Notice that the ACI Building code prohibits splicing of column reinforcement at the base of the column in regions of high seismic hazard so as to avoid possible bond failure at the column base where plastic hinging is likely to develop. However, because of the premature joint shear failure and the consequent development of relatively small stresses in the column reinforcement at nominal lateral load capacities of the specimens, no conclusions can be drawn regarding this requirement in relation to the beam-column connections under investigation. A conclusion regarding this requirement is addressed during the second part of this investigation dealing with the seismic response of the same beam-column connections after being detailed more properly for avoiding joint shear failure.

4 CONCLUSIONS

The following conclusions can be drawn from this investigation:

- When subjected to drift reversals induced by even moderate earthquakes, gravity load designed concealed wide beam-narrow column connections of the type under investigation will develop sizable diagonal shear cracks and consequently joint shear failure at drift ratios varying between 1.5 and 2.0%. These connections are expected to experience large damage beyond repair and possibly structural collapse when subjected to large drift reversals induced by strong earthquakes.
- As a result of the premature joint shear failure, the strains in the beam and column reinforcement developed at maximum lateral load capacity were well below yield. The lateral load capacities reached by the specimens in percent of the estimated capacities should the beams or columns acquire their nominal flexural strength were 59% for the interior and 35% for the exterior connection, respectively.
- Bond splitting cracks did develop around the outermost beam bars outside the column core due to the short embedment length beyond the beam-column interface section and the lack of concrete or steel confinement, but these cracks were not as critical in controlling the performance of the joints when compared with the joint diagonal shear cracks.
- Even in regions of moderate seismic hazard, unless designed and detailed to prevent joint shear failure, the connections under investigation are significantly weak to be considered as part of the earthquake lateral load resisting system.

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RERFERENCES

[1] M. Harajli, S. Sadek, R. Asbahan, Evaluation of seismic hazard of Lebanon: implications of recent earthquakes and new technical developments. *Journal of Seismology*, Kluwer Academic Publishers, 6(2), 257-27, 2002.

[2] C. Huijer, M. Harajli, S. Sadek, *Implications of the recent mapping of the offshore thrust fault system on the seismic hazard of Lebanon*, Research Report submitted to the Lebanese National Council for Scientific Research (LNCSR), October 2010. Also, MS Thesis presented by the first author to the CEE Dept. at the American University of Beirut (AUB), Beirut, Lebanon, 100 pp.

[3] ACI Committee 318, *Building Code Requirements for Reinforced Concrete and Commentary,* American Concrete Institute, Farmington Hills, Mich., 2008.

[4] ACI-ASCE Committee 352R-02, *Recommendation for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures*, American Concrete Institute, Farmington Hills, Mich., 2002.

[5] J.M. Lafave, J.K. Wight, Reinforced concrete exterior wide beam-column-slab connections subjected to lateral earthquake loading. *ACI Structural Journal*, 96(3), 577-586, 1999.

[6] J.M. Lafave, J.K. Wight, Reinforced concrete wide-beam construction vs. conventional construction: resistance to lateral earthquake loads. *Earthquake Spectra*, 17(3), 2001.

[7] B. Li, S.A. Kulkarni, Seismic behaviour of reinforced concrete exterior wide beam-column connections. *Journal of Structural Engineering*, ASCE, 136(1), 26-36, 2010.

[8] C.G. Quintero-Febres, J.K. Wight, Experimental study of reinforced concrete interior wide beam-Column connections subjected to lateral earthquake loading. *ACI Structural Journal*, 98(4), 572-582, 2001.