Experimental Analysis of the Lateral Resistance of a Shear Critical Reinforced Concrete Frame

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SUMMARY:

Flexural behaviour of reinforced concrete (RC) frames is a well-studied topic that has been evaluated both analytical and experimentally. Contrarily, shear failure of RC frames have been studied in a lesser amount; in consequence, few experimental tests have been performed on shear-critical RC frames. Many RC structures designed before seismic considerations were included in design codes may present an undesirable shear failure when subjected to seismic loading. The described fallacy is common in countries in which the first seismic code dates from the mid 80's; therefore, an important percentage of the current building stock was built without adequate consideration for shear-critical behaviour under seismic loading and are susceptible to fail in a brittle manner. This paper presents results of a full scale, single-span, one-story RC frame with a shear-critical beam, tested under an increasing lateral load. Experimental behaviour agrees well with analytical predictions based on the Modified Compression Field Theory.
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Keywords: concrete, shear, seismic loading, design code.

1. INTRODUCTION

Nowadays design and construction of an earthquake resistant structure is possible thanks to the current knowledge on earthquake engineering. A different situation took place few decades ago: seismic design was introduced on the decade of the 60s, and, in several countries as Colombia, it was not mandatory until the beginning of the 80s. Furthermore, seismic knowledge has increased as seismic events have taken place; as a consequence, current design provisions present differences when compared to those used years ago.

A reinforced concrete frame structure built without proper seismic design may present the following situations: insufficient amount of transverse reinforcement, inadequate location and insufficient length for lap splices of longitudinal reinforcement, insufficient embedment length for longitudinal reinforcement, lack of strong column/weak beam design approach. The combination of the mentioned issues may yield to the impossibility of the development of the structural full flexural capacity, and, as a consequence, a non-ductile behavior.

Shear resistant is an important issue on structures built without seismic provisions or with early seismic codes; a shear-critical structure is mainly associated to insufficient ductility and energy dissipating mechanisms, concepts that were not well understood on early seismic provisions. A shear failure implies a brittle failure that could yield to both human and economic losses, losses that should not take place by today seismic design philosophy.

An important percentage of current building stock corresponds to structures built without properly seismic provisions, and may be susceptible to exhibit a shear failure if an important seismic load is imposed. It becomes necessary to asses those structures according to present seismic knowledge in order to take corrective measures if needed. For the particular case of reinforced concrete structures, the correct seismic assessment of shear concrete structures is not as simple as the flexural-critical situation. Flexural design procedures for reinforced concrete structures are based on the simple “plane sections remain plane” theory and the use of stress block factors. Prediction of flexural capacity is remarkably similar when different design codes are used (Collins *et al*., 2008) and aggresses well with experimental tests. Shear capacity, on the other hand, involves many parameters such as member depth, materials properties, and maximum aggregate size, among others. Due to the complexity on the shear behavior of reinforced concrete structures, many codes as Eurocode 2 and ACI have adopted empirical or semi-empirical procedures for the computation of shear capacity. A more rational theory, the *Modified Compression Field Theroy (MCFT)*, has been developed in the last 40 years, mainly at the University of Toronto, Canada. Collins *et al*. (2008) presents the shear prediction of four slab-strip specimens when using four different codes, as well as experimental results. The ratio of the highest to lowest predicted shear load reached a value of 2.56, which illustrates the high uncertainty associated to the shear capacity prediction by different codes.

The MCFT was first included on the 1994 edition of the LRFD (load and resistant factor design) bridge design specifications by the American Association of State and Highway and Transportation Officials (AASHTO); the shear design procedures of the 2004 Canadian code are also based on the MCFT, as well as it latest version of 2006. The MCFT, —which is based on equilibrium, compatibility and stress strain relationships—, has been proved to present a good agreement between analytical an experimental results (Collins *et al*., 2007; Collins *et al*., 2008; Acevedo *et al*., 2009) and was used in this research. It is out of the scope of this paper to present the development and the equations of the MCFT; the reader is referred, in addition to the already mentioned literature, to Vecchio and Collins (1986), Collins and Mitchell (1991) and Bentz and Collins (2006).

If the seismic capacity of a structure has to be computed, different methodologies can be performed using nonlinear analysis procedures that typically require computer-based applications. There are some available applications such as SAP2000 (CSI 2005), RUAUMOKO (Carr 2005), DRAIN-2DX (Prakash et al. 1993), among others. All these programs ignore shear mechanisms by default and therefore, unconservative estimates of both strength and ductility are obtained. Some of these programs have the option to consider the shear behavior using automatically generated shear hinges or defining manually through user-defined shear hinges. The last option requires expert knowledge and usually takes a significant amount of time, which severely limits the use of such procedures in practice. Güner (2008) proposed a simple procedure to evaluate the seismic capacity of critical shear frame. This method has been used in this work in order to obtain the numerical prediction.

Shear behavior of reinforced concrete frames have been studied mainly at the University of Toronto (Canada). Experimental tests have been performed by Vecchio and Balopoulou (1990), Vecchio and Emara (1992), and Doung (2006). From the mentioned tests, only the latest one presented a shear failure. A more general test on reinforced concrete frames was performed by Ozden *et al*., (2003); the specimen included several structural deficiencies that caused its premature failure. The authors of this work have no knowledge of other tests performed with the aim of analyze the shear behavior of shear-critical reinforced concrete frames.

2. STUDY CASE

The full scale, single-span, one-story reinforced concrete frame with a shear-critical beam shown in Figure 2.1 was tested under an increasing lateral loading applied at the top of one of the columns. The frame was designed with material properties typically used on Colombia in the decades of the 70s and 80s and representative detailing conditions from pre-code structures on the beam element. Columns were designed with up-to-date codes in order to concentrate shear failure on the beam element. Although the situation of a one-story frame is not as common as a multiple-story frame, the first situation was selected due to laboratory limitations. No axial load was applied to the columns, once again, due to laboratory limitations. The former situation had an influence on the upper joint close to the point of lateral load application, as will be explain in Section 5. The presented experimental test constitutes an important contribution to the scarce data set of shear-critical frames tested under lateral loading.

The frame was designed with a center to center span of 2350 mm (7.7 ft), a story height of 1850 mm (6.1, ft), and an overall height of 2400 mm (7.9 ft); a 400 mm (15.7 in) heavily reinforced concrete base was integrated to the frame. The beam was 250 mm (9.8 in) and 300 mm (11.8 in), with an effective depth of 250 mm (9.8 in); columns were 250 mm (9.8 in) width and 350 mm (13.8 in) high, with an effective depth of 310 mm (12.2 in) on the upper column and 297.3mm (11.7 in) on the lower column.

Materials used correspond to a 42.5 MPa (6164 psi) compressive strength concrete (on the day of testing), and two types of steel reinforcement: smooth bars whit a yield stress of 325 MPa (47 ksi) for stirrups on the shear-critical element (beam) and corrugated bars with a yield stress of 454 MPa (66 ksi) for the shear-critical element and longitudinal reinforcement of both beam and columns.

Beam longitudinal reinforcement corresponds to 4 No. 6 corrugated bars (1136 mm^2; 1.76 in^2), for a ratio of longitudinal reinforcement of ρ = 1.82% on both sides of the element. Two legged stirrups with No. 2 smooth bars (total area of 64 mm^2; 9.9 in^2) spaced at 187.5 mm (7.4 in) were used as transverse beam reinforcement. Top column longitudinal reinforcement corresponds to 2 No.6 and 2 No. 7 corrugated bars (1342 mm^2; 2.08 in^2), ρ = 1.73%, on both sides of the element, while bottom columns included two extra 2 No. 6 corrugated bars for a total area of 1910 mm^2 (3.0 in^2), ρ = 2.57%, on both sides of the element. Both top and bottom columns had two legged stirrups with No. 6 corrugated bars (total area of 568 mm^2; 0.88 in^2) spaced at 100 mm (3.9 in). Reinforcement details are presented in Figure 2.2.

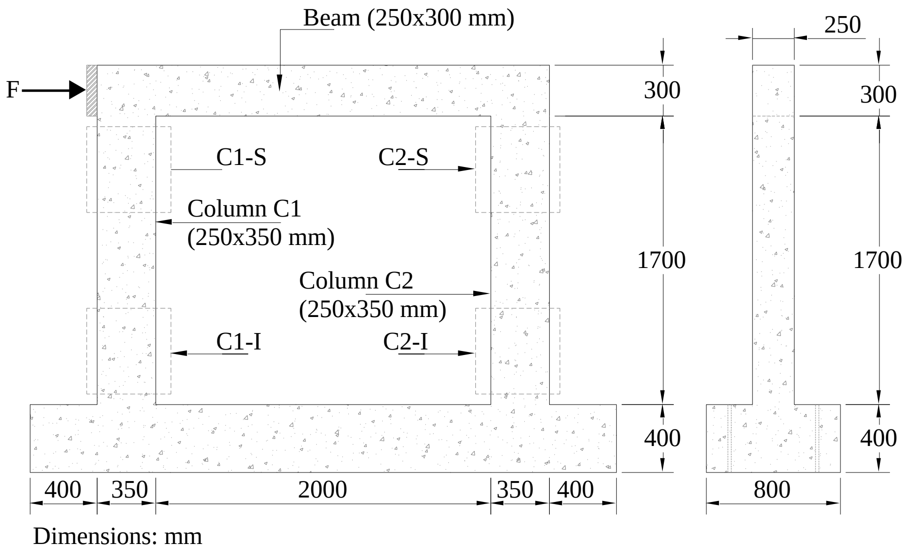


Figure 2.1. Frame geometry

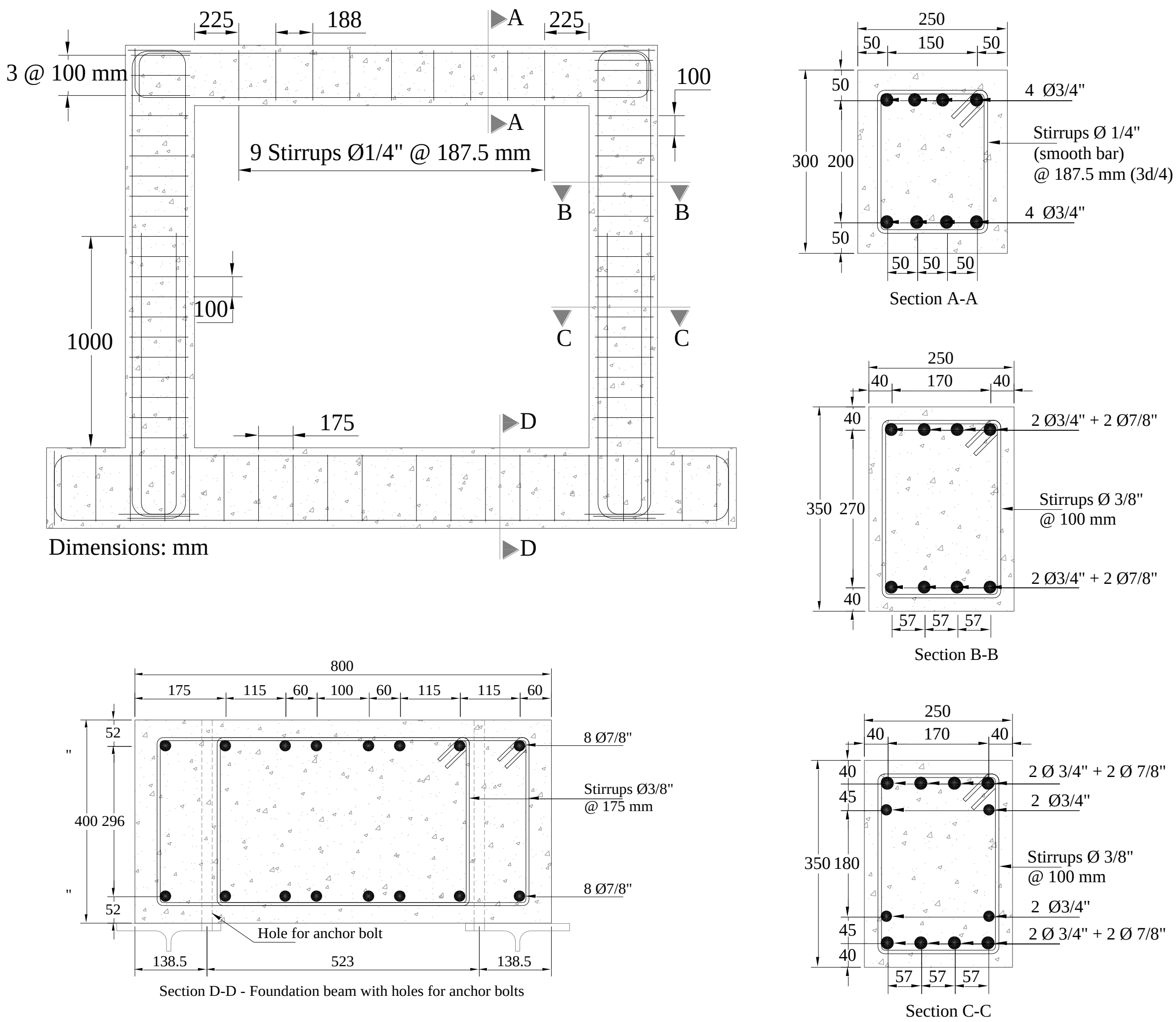


Figure 2.2. Reinforcement details

3. TESTING DETAILS

3.1. Test setup

Frame testing was conducted using the setup shown in Figs. 3.1 and 3.2; test set-up basic components include the test unit, a foundation beam, a horizontal hydraulic actuator, and a loading frame. The foundation beam was clamped to the loading frame with bolts of 5/8” diameter spaced every 200 mm (7.87 in). Horizontal loading was applied by a displacement-controlled actuator positioned at the top story beam centerline at a height of 1850 mm (6.1 ft) above the foundation beam. This actuator was anchored against a strong loading frame; it has a load capacity of 400 kN (89.71 kips) and a stroke capacity of approximately ±300 mm (11.8 in) after accounting for slack in the loading system.

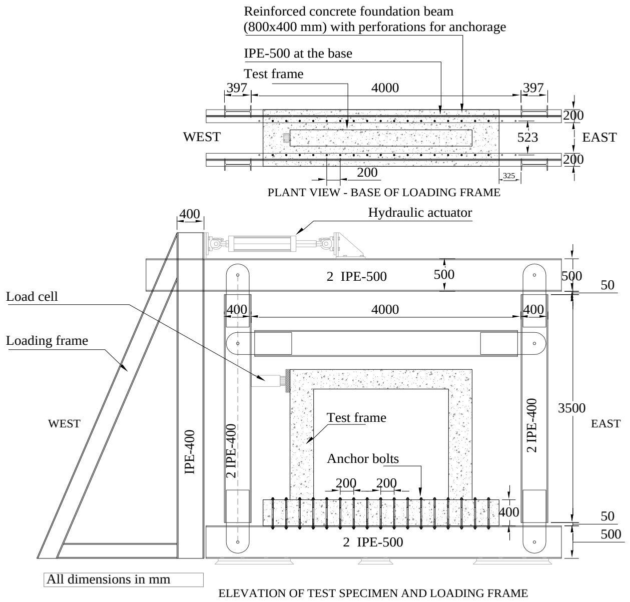
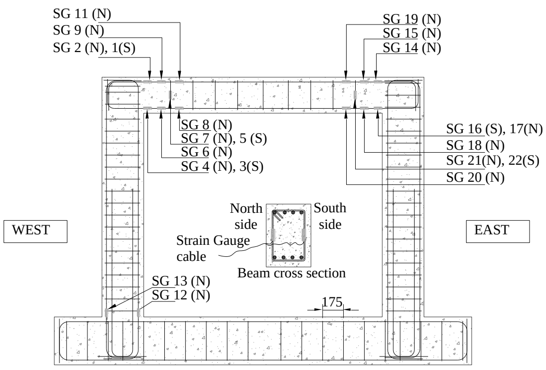


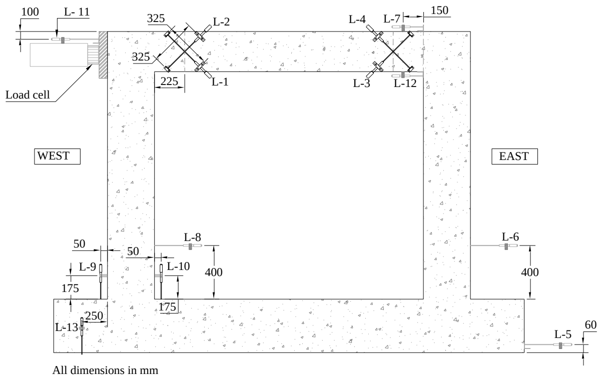
Figure 3.1. General details of loading set-up

3.2. Instrumentation

The frame was instrumented to monitor global and local behavior during testing. Instrumentation was used to measure displacements, loads and strains at critical locations for the frame; it is schematically showed in Fig. 3.2. Lateral displacement was measured at the beam level by mean of Linear Variable Differential Transducers, LVDT´s, and horizontal load was measured by the load cell as shown in Fig. 3.2b. A total of 22 steel strain gauges were installed on the longitudinal reinforcement, at potential locations for beam and column flexural hinging, and on the beam stirrups (Fig. 3.2a). Thirteen LVDT´s were placed at various locations, as illustrated in Fig. 3.2b to measure lifting or sliding displacements of the foundation beam, lateral top displacement of the frame and the shear deformation of the critical section on the beam.



a) Strain gauges



b) LVDT´s

Figure 3.2. Location of instrumentation

3.3. Testing procedure

The frame was subjected to quasi-static cyclic loading at increasing levels of top displacement. Lateral load was divided into five load stages (130.7 kN, 193.7 kN, 243.7 kN, and the maximum load achieved of 386.5 kN). Once the stage load was reached, the system was partially unloaded for safety by approximately 10% to mark and measure cracks, perform a visual inspection, and take photographs. Frame failure load was not achieved due to actuator capacity; even thought, the specimen was close to failure as strain gauges indicated that yielding occurred in the transversal reinforcement at the end of the beam.

4. NUMERICAL PREDICTION

The seismic behavior of the frame has been evaluated using the procedure proposed by Güner (2008), which was developed based on three fundamental aspects: First, it allows a prediction of the overall capacity of the frame, second, an iterative procedure is carried on until the member end forces are obtained from the global analysis and from the sectional analysis converge, and finally it considers the concept of secant stiffness formulation.

A pushover analysis was carried out to evaluate the seismic capacity. The procedure considers the cracked sections. An effective stiffness value of 0.5 times the uncracked gross stiffness was used according to FEMA (2000). Rigid end offsets with rigid end factors of 1.0, which correspond to a fully rigid connection, were used in order to account for the overlapping portions of the beam-column connections. The nonlinear behavior was evaluated by mean of lumped nonlinearity plasticity model. Flexural plastic hinges were defined at critical section both beam and columns. The ratio of bending moment to shear force was to take in to account to calculate shear plastic hinges. The axial force effect was included in both cases using an iterative solution procedure due to axial load dependent on the lateral load acting on the frame. Moment-curvature and shear-strain were calculated using RESPONSE-2000. This program calculates the shear capacity according to the Modified Compression Field Theory. The bending and shear hinge lengths were assumed to be equal to 0.5 and 1.5 times the depth of the cross sections. A more detailed description of the method is presented in Güner (2008).

A fixed support at the base of the columns is considered during the design of the frame. Nevertheless, a rotation at the base of columns is presented during the testing, which are recorded for the strain gauges (SG 12 and SG13) and the LVDT´s (L6, L9 and L10). From this information, rotational stiffness was calculated at the base of the columns (more detailed information can be founded in the next section). The capacity curve for both models (fixed and including the rotation at the base) is shown in Figure 4.1.

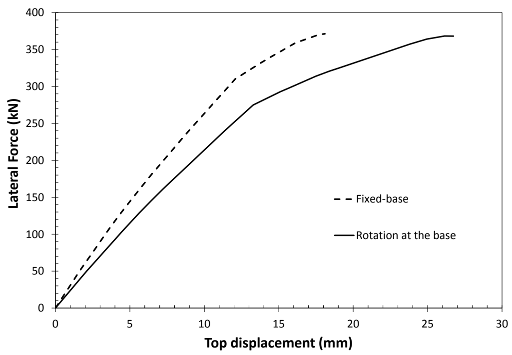


Figure 4.1. Numerical prediction of the frame

5. TEST RESULTS

Sliding and lifting displacements were recorded at the base of the frame according to information given by L5 and L13 sensors, respectively (see Figure 5.1). Numerical prediction including the rotation at the base and experimental result considering the correction for the displacement recorded at the foundation beam are shown at Figure 5.2. Damage patterns after failure are depicted in Figure 5.3 indicating that the majority of damage was concentrated in the beam, particularly at the west side.

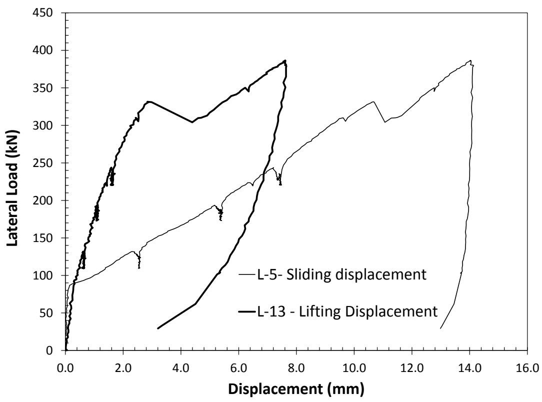


Figure 5.1. Displacement recorded at the base of the frame

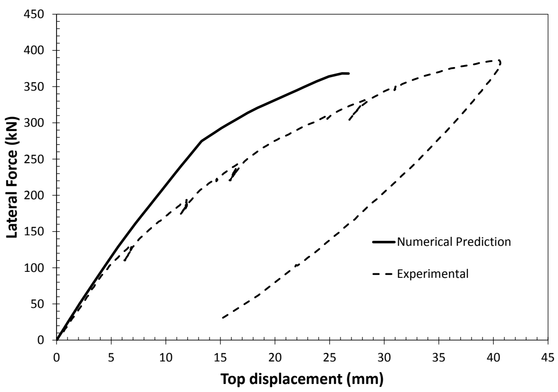
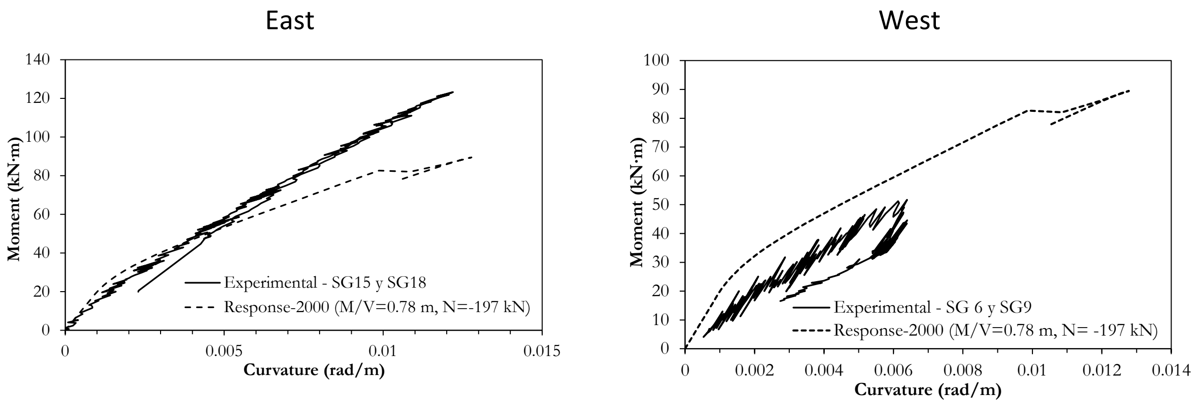


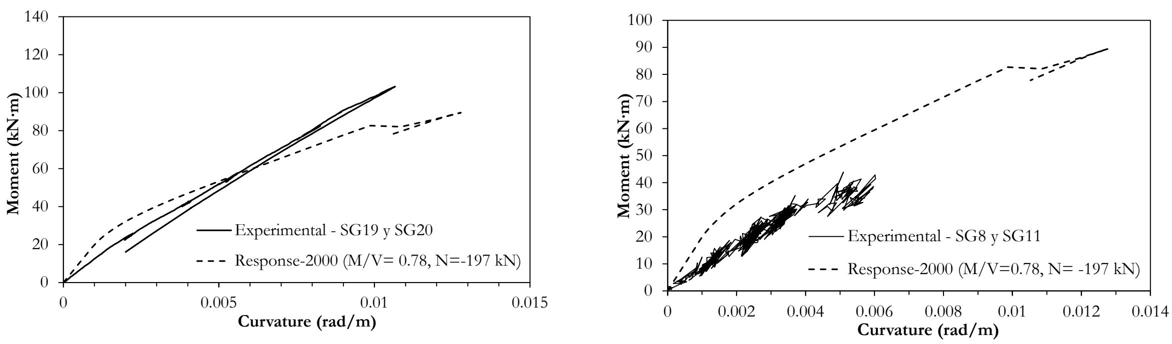
Figure 5.2. Numerical and experimental capacity curve

Figure 5.3. Photo the specimen at the end of the testing

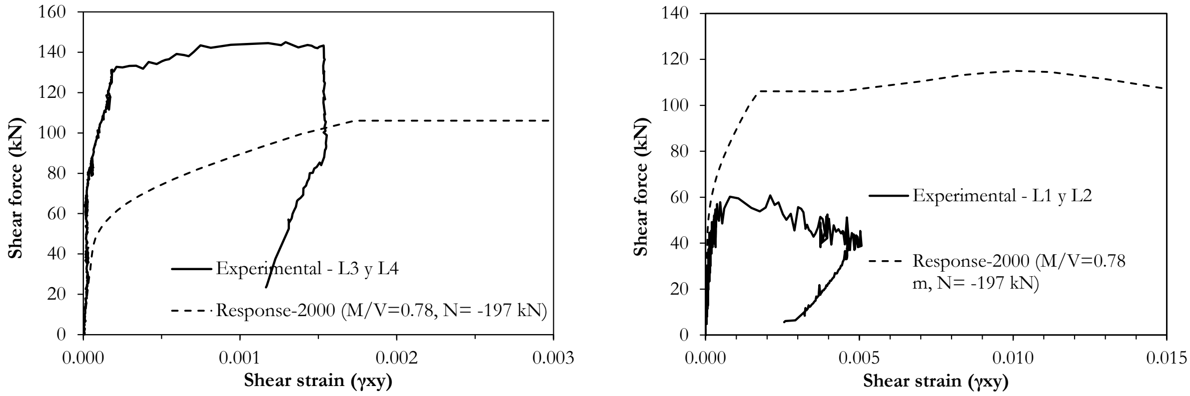
The test response of the beam and the numerical prediction obtained to Response-2000 program are compared in Figure 5.4. The first column corresponds to information recorded at the east side and the second is associated to west side of the beam. Moment-shear ratio equal to 0.78 m and axial load (N) of 197 kN ($N/f_c'A_g$ = 5.67%) were used to the numerical prediction. Moment-curvature relationships are shown in Figure 5.4a and 5.4b, respectively while shear force – strain force relationship is presented in Figure 5.4c.



a) Moment - curvature relationship - h/2 (150 mm)



b) Moment - curvature relationship – h (300 mm)



c) Shear force- shear strain relationship

Figure 5.4. Experimental and numerical results of beam

5. DISCUSSION AND CONCLUSSION

Figure 5.1 shows analytical and experimental results for the tested specimen. Although no experimental failure load was achieved during testing due to actuator capacity, instrumentation readings measured significant shear deformation for the maximum load of 386.5 kN, which indicates that the frame was close to failure. Analytical and experimental results are similar in terms of capacity: numerical prediction estimated a value of 368.15 kN as a failure load (95.3% of maximum experimental load).

Differences between experimental and analytical results can be explained by an inadequate joint reinforcement detailing: beam-column joints of the tested frame were built and designed as it was common in pre-code structures. The lack of appropriated joint detailing allowed for a considerable amount of shear deformation on the joint, deformation that affected the frame overall lateral displacement. Implications of joint local failure depend not only on the joint detailing, but also on the amount of longitudinal tension reinforcement in the members reaching the joint, as it was observed by Ingvar *et al*. (1976) and Singh and Chaturvedi (1997), among others.

Beam-column joint of the tested specimen lacked of diagonal stirrups as required in current design specifications for opening corners or knee joints. Additionally, highly longitudinal reinforced members were connected at the joints (ρ = 1.82% and ρ = 1.73% for beam and columns respectively); situation that has been reported as non-desirable by Swann (1969).

Tests results indicate initial joint cracking at a lateral load close to 100 kN; at this level of loading steel reinforcement on tension had not yet reached the yielding stress. It can be observed from Figure 5.1 that numerical and experimental curves differ from each other from a load close to 100 kN. It can be stated that differences are due to joint deformations, which produce a reduction on the frame lateral stiffness, and an increase on lateral displacements. Figure 5.3 shows the specimen at the end of testing. It can be observed significant damage ate the top-west joint.

Even though beam-column deformation was presented, joint failure did not take place due to the presence of longitudinal stirrups. The specimen was able to develop its beam shear capacity. Data from the different systems of instruments was analyzed and good agreement was found for analytical and observed beam behavior, as shown in Figure 5.4.

The authors of this article recommend for futures tests the application of axial loading on the columns and a beam-column joint detailing as recommended on state-of-the-art seismic codes. Frame joint strengthening will not induce a reduction on the lateral frame stiffness and numerical and tested values of the overall load-deformation response will be closer.

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