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SHAKING TABLE TESTS OF RC SHEAR WALLS WITH OPENINGS

Julian Carrillo [1] and Sergio Alcocer [2]

ABSTRACT

This paper compares the performance of four isolated reinforced concrete (RC) shear walls with two openings: two full-scale prototype walls tested under quasi-static cyclic (QSC) loading and two lightly scaled models tested under shaking table excitation. Variables studied were the web steel ratio (0.125% and 0.25%), the type of web reinforcement (deformed bars and welded wire meshes) and the testing method. It was verified that loading history of the QSC testing ignores the foremost dynamic effects observed in structures subjected to earthquake loads. When dynamic and QSC responses were compared, it was apparent that stiffness and strength degradation properties depend on the loading rate, the strength mechanisms associated to the failure modes (crushing of concrete or plastic yielding of reinforcement), number of cycles, and cumulative parameters such as ductility demand and energy dissipated. Then, data obtained from QSC tests can not always be safely assumed as a lower bound of the performance capacity of RC shear walls with openings subjected to earthquake-type loading.

Introduction

While quasi-static (QS) tests are the simplest to perform, they are also the most limited to provide information on the true dynamic behavior of test specimens. In general, the loading history of QS method ignores many dynamic effects observed in real structures subjected to earthquake loads, mainly, the strain rate effects. Then, when the seismic behavior of a element or system is studied using QS method, incorrect interpretations of results can be generated, mostly in the following cases (Leon and Deierlein 1996, Rai 2001, Mosalam *et al.* 2008): a) the governing failure mode is strongly affected by the strain rates, b) the material that controls the behavior is brittle as concrete and masonry; c) the overstrength characteristics are a fundamental issue on the response and, d) ductility and energy dissipation capacity are important parameters. Then, it is unclear whether the data obtained from QS tests can be safely assumed as a lower bound for strength, ductility and energy dissipation capacity. For instance, the effect of the strain rate on degrading materials (concrete, masonry) on structural performance has not been adequately studied (Leon and Deierlein 1996). According to Rai (2001), neglecting the effect of

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[1]Ph.D Candidate, Universidad Nacional Autónoma de México, UNAM, México & Assistant Professor, Dept. of Civil Engineering, Universidad Militar Nueva Granada, Bogotá, Colombia, wcarrillo@umng.edu.co
[2]Research Professor, Instituto de Ingeniería, Universidad Nacional Autónoma de México, UNAM, México, salcocerm@ii.unam.mx

strain rates in QS testing, gives rise to many of its unique strengths as a testing method, i.e. the ability to detect and observe damage propagation, as well as to provide a consistent basis for comparison among test programs.

In order to study the seismic behavior of RC walls for housing, a large research program has been underway between the Instituto de Ingenieria at UNAM and Grupo CEMEX. The experimental program has included QSC tests and dynamic loading tests of RC walls with different height-to-width ratios (Carrillo and Alcocer 2008) and walls with openings (door and window). In this paper, the behavior of four isolated shear walls with two openings is compared. Two full-scale prototype walls were tested under QSC loading and two lightly scaled models were tested under shaking table excitation. The experimental response was studied for identifying the main parameters affecting the strength and stiffness degradations during dynamic and QSC testing. The performance of walls was compared using the failure modes, hysteresis curves, loading rates, number of cycles and the cumulative parameters, such as ductility demand and energy dissipated.

Experimental Program

The three-dimensional prototype is a two-story RC house with shear walls in the two principal directions. Typically, wall thickness and clear height are 100 and 2400 mm, respectively. Nominal compressive strength is 15 MPa. In the experimental program, four isolated walls with openings were studied: two full-scale prototypes tested under QSC loading and two 1:1.25 scaled models tested under shaking table excitation. Because the size of the models was very similar to the prototypes, the simple law of similitude was chosen. In this type of law of similitude, the models are built with the same material as the prototype (i.e. materials properties are not changed) and only the dimensions of the models are altered.

Geometry and Reinforcement

Variables studied were the web steel ratio (0.125% and 0.25%) and the type of web reinforcement (deformed bars and welded wire meshes). The geometry and the reinforcement layout of the models tested under shaking table excitation are illustrated in Fig. 1. The wall models were built in a foundation beam bolted to the platform of the shaking table. Thickness of the models was 80 mm. Areas of door and window openings were equivalent to 32% of the wall area. Longitudinal and transverse reinforcement at the boundary elements were the same in both specimens: 4 No. 4 longitudinal deformed bars (12.7 mm diameter = 4/8 in.) and No. 2 smooth bar stirrups (6.4 mm diameter = 2/8 in.) at 180-mm spacing. This reinforcement was designed in order to prevent flexural failure. Specimen MVN100D was reinforced for web shear with a single layer of No. 3 vertical and horizontal deformed bars (9.5 mm diameter = 3/8 in.) with spacing of 320 mm. The amount of web reinforcement corresponds approximately to the minimum web steel ratio prescribed in ACI-318 (2008) building code, which is equal to 0.25%. Specimen MVN50mD was reinforced for web shear with a single mesh (6x6-8/8) of No. 8 wires (4.1 mm diameter) with spacing of 150 mm (~ 6 in.). The web steel ratio was approximately 50% of the minimum ratio prescribed in ACI-318. This test was aimed at examining the performance of walls reinforced with a steel percentage smaller than the minimum prescribed by the code. A lower steel ratio is supported by the fact that lower concrete compressive strength and higher yield strength for the web steel require, in theory, smaller percentages than the 0.25% minimum prescribed ratio. Prototype walls (MVN100C and MVN50mC) were tested under QSC loading.

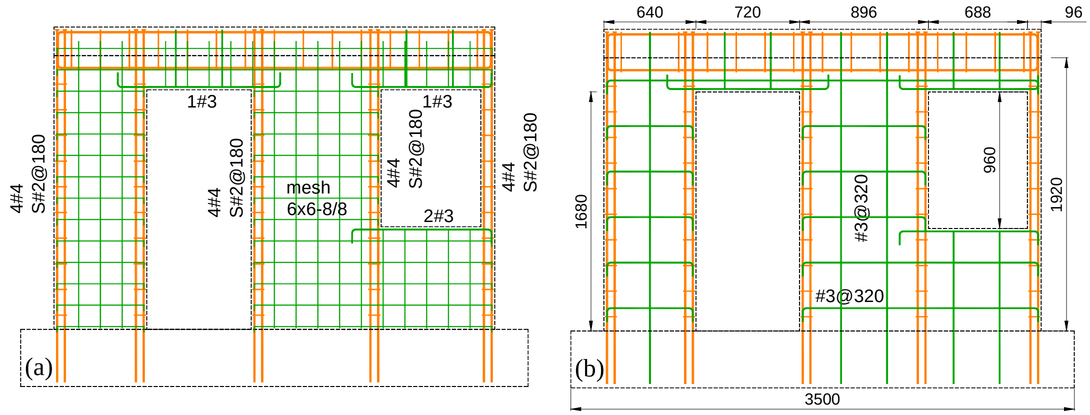


Figure 1. Geometry (mm) and reinforcement layout of models: a) MVN50m0D, b) MVN100D.

Mechanical Properties of Materials

For design, nominal concrete compressive strength was 15 MPa, and nominal yield strength of bars and wire reinforcement were 412 MPa (mild steel) and 491 MPa (cold-drawn wire reinforcement), respectively. Mean value of the measured compressive strength was 24.7 MPa for prototype walls and 16.0 MPa for wall models. Mean value of the measured yield strength of No. 3 (9.5-mm diameter) bars and wire reinforcement (4.1-mm diameter) were 435 MPa and 630 MPa, respectively. For concrete, properties were obtained at the time of testing.

Loading Histories

Aimed at studying wall performance under different limit states, from onset of cracking to collapse, models were subjected to three earthquake hazard levels using both natural and artificial acceleration records. An earthquake record from an epicentral region in Mexico (Mw=7.1, CA-71), was used for the seismic demand in the elastic limit state. The earthquake was recorded in Caleta de Campos station, in January 11, 1997. This record was considered as a Green function (basic event) to numerically simulate larger-magnitude events, i.e. with larger instrumental intensity and duration. Two earthquakes with Mw magnitudes 7.7 (CA-77) and 8.3 (CA-83) were numerically simulated for the strength and ultimate limit states, respectively. Time history accelerations for prototype walls are presented in Fig. 2-a. According to the law of similitude, acceleration and time scale factors were applied to these records for testing the models. Models were tested under progressively more severe earthquake actions, scaled up considering the value of peak acceleration as the reference factor until the final damage stage was attained.

Isolated models were designed considering the fundamental period of vibration of the prototype house. For establishing such dynamic characteristic, analytical models were developed and calibrated through ambient vibration testing. The fundamental period of vibration of the two-story house was estimated at 0.12 s (Carrillo and Alcocer 2008). Taking into account the scale factor for period, S_T = 1.25, isolated wall models were designed to achieve an initial in-plane vibration period (T_e), close to 0.10 s (0.12 s / 1.25). In design, it was supposed that walls would behave as a single degree of freedom system. The dynamic weight, W_d (mass × gravity acceleration) necessary to achieve the desired design period T_e, was calculated as $(K_e T_e^2/4\pi^2)g$; where K_e is the in-plane stiffness of the wall that was calculated from measured mechanical properties of materials. To account for premature shrinkage cracking, the moment of inertia of the wall section was reduced by 25%. As a result, the dynamic weight was 188.2 kN.

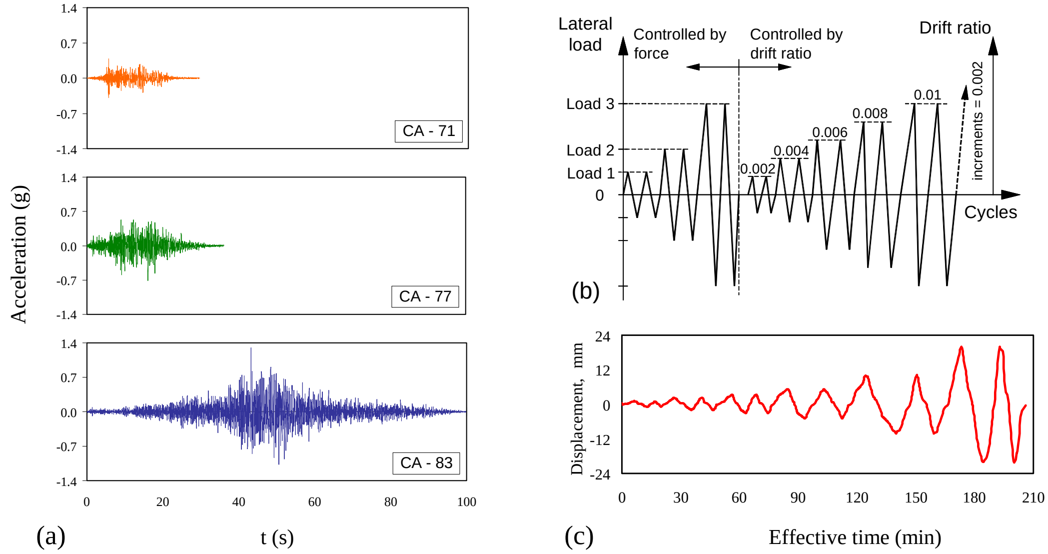


Figure 2. Loading histories: a) Dynamic testing, b) QSC testing protocol, c) QSC - Measured.

In QSC testing the loading protocol consisted of a series of increasing amplitude cycles as illustrated in Fig. 2-b. For each increment, two cycles at same amplitude were applied (I and II). The first two cycles were applied to reach the 25% of the calculated cracking load (Load 1). In the next increment (Load 2), the 50% of the calculated cracking load was reached. Then, it was applied an increment to attain the actual cracking load (Load 3). After that, the loading history was controlled by drift ratio using increments with amplitude equal to 0.002. A typical loading history during QSC testing in terms of displacement measured at mid-thickness of the top slab and the effective time between steps is depicted in Fig. 2-c.

Test Setups and Instrumentation

In dynamic testing, models were subjected to a series of base excitations represented by the selected earthquake records. Records were reproduced by a shaking table where the foundation beam of the models was bolted. If the dynamic weight were to rest at the top of models, the risk of lateral instability would have been a major concern. Then, an alternative method for supporting the mass and transmitting the inertia forces was required. An external device for a mass-carrying load system which is allowed to slide horizontally on a fixed supporting structure located outside the shaking table was designed (Carrillo and Alcocer 2010). In QSC testing, the lateral loads were applied directly at the top slab level through double-action hydraulic actuators (see Fig. 3). An axial compressive stress of 0.25 MPa, which roughly corresponds to 2% of the nominal concrete compressive strength, was applied on top of the walls. The axial load was kept constant during testing. In dynamic testing, it was exerted through the weight of the load and connection beams, and lead ingots bolted to the load beam. Although lead ingots resulted in a triangular load distribution, the addition of the weight of the connection beam provides for a uniform distribution of the axial load on the walls. In QSC testing, the axial load was achieved using a lever system arranged by a weight hanged at side walls which caused the vertical force on the load beam.

To measure the specimens' response, walls were instrumented internally and externally. Internal instrumentation was designed to acquire data on the local response of reinforcement through strain-gages bonded to the steel reinforcement. External instrumentation was planned for learning about the global response through displacement, acceleration and load transducers. Also, an optical displacement measurement system (with Light Emitting Diodes - LED's) was used. In the tests, 59 strain-gages and 64 external transducers were used (Carrillo 2010).

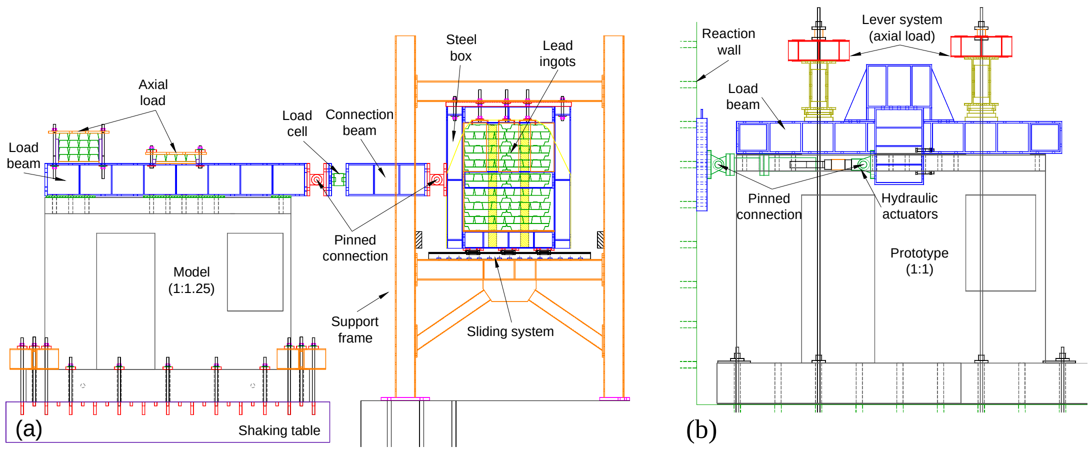


Figure 3. Test setups: a) Shaking table testing, b) QSC testing.

Test Results

The experimental response was studied for identifying the main parameters affecting the strength and stiffness degradations during dynamic and QSC testing. Initially, the overall performance of walls was compared using failure modes and hysteresis curves. Then, detailed behavior was assessed through the loading rate, number of cycles and the cumulative parameters, such as ductility demand and energy dissipated (Carrillo 2010).

Crack Patterns and Failure Modes

In dynamic testing, walls reinforced with welded wire mesh and with 50% of the minimum code prescribed steel ratio exhibited diagonal tension failures, "DT". Failure mode was governed by plastic yielding of reinforcement and subsequent fracture of wires. Failure was brittle because of the limited deformation capacity of the wire mesh itself. In contrast, walls reinforced with deformed bars and with 100% of the minimum steel ratio exhibited a mixed failure mode, where diagonal tension and diagonal compression, "DT-DC" (i.e. plastic yielding of some bars of reinforcement in the web and noticeable crushing of concrete), was observed (see Fig. 4). Although walls tested under QSC loading exhibited comparable failure modes and cracking patterns, the number and length of cracks was larger than in walls tested under dynamic loading. The differences are related mainly, with the strain rate. When marking cracks during QSC testing, it was not rare to observe crack propagation during some minutes while the target load was maintained during crack marking. It is evident that this type of crack propagation could not have occurred in short time interval during dynamic testing.

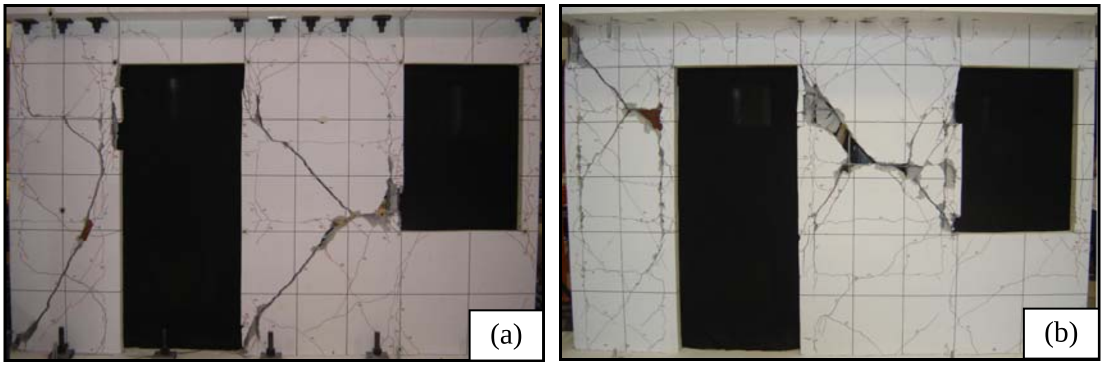


Figure 4. Final cracks patterns in dynamic testing: a) MVN50mD, b) MVN100D.

Hysteresis Curves

The hysteresis curves were expressed in terms of the normalized shear strength, V/V_{normal} and ductility demand. The shear strength predicted using equations proposed by Carrillo *et al.* (2009a), V_{normal}, was utilized to normalize the measured lateral force, V. Predicted shear strength was calculated using measured wall dimensions and mechanical properties of materials. The ductility demand was calculated dividing the drift ratio (R) by a conventional yield drift ratio (R_y), corresponding to the development of 80% of the peak strength. The drift ratio, R, was obtained by dividing the relative displacement measured at mid-thickness of the top slab, by the height at which such displacement was measured. The envelopes of the hysteresis curves and the failure mode are shown in Fig. 5. According to the loading history, two envelopes for QSC testing are drawn: using the data associated with the first (I) and second (II) cycle of each increment (see Fig. 2-b).

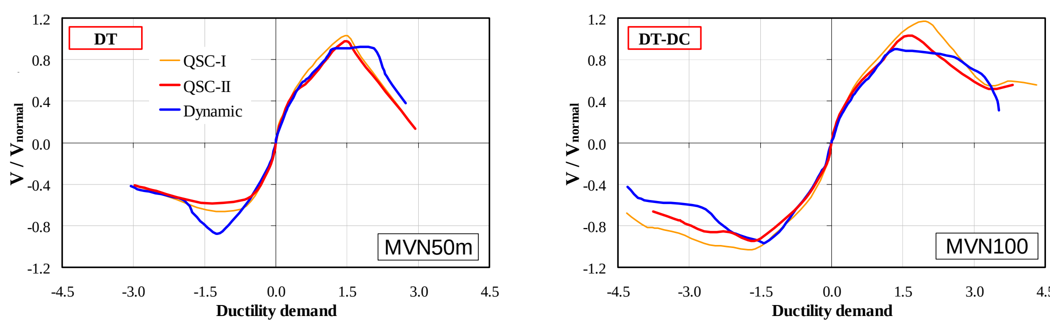


Figure 5. Envelopes of hysteresis curves.

As it was expected, differences were observed between the general performances of the specimens tested using real dynamic actions and the specimens tested under QSC loading. Foremost differences are associated with the shear strength capacity. For example, in specimen MVN50m that failed by diagonal tension, the dynamic envelope was slightly higher than the

QSC envelope associated with the second cycle (QSC-II). Moreover, the dynamic envelope was associated to a more ductile response. In contrast, in specimen MVN100 that exhibited a mixed failure, the two envelopes (Dynamic and QSC-II) were, in general, comparable.

Loading Rate

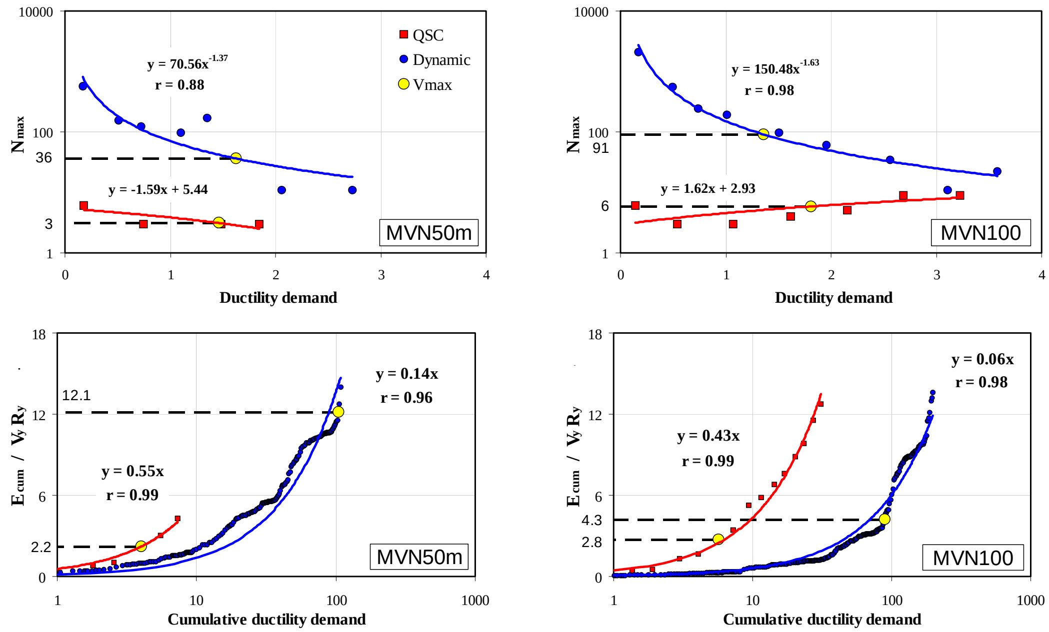
The effect of the loading rate on the structural behavior is widely recognized. However, rate effects on RC shear walls have not yet been clearly quantified. Measured maximum loading rates in terms of displacement (mm/s) are shown in Table 1. Velocities were calculated dividing the relative displacement measured at mid-thickness of the top slab by the time step. In dynamic testing, the later corresponds to the constant time step of the records (0.01 s). In QSC testing, the effective time between steps was used (Fig. 2-c). The ratio between the maximum velocities measured during dynamic (mean value) and QSC testing is included in Table 1. Such ratio is, approximately, equal to 650. Then, the loading rates are noticeably different between two methods of testing. The observed higher shear strength in dynamic testing of specimen wit "DT" failure would be associated with the loading rate effect.

Table 1. Measured loading rates during testing (mm/s).

Failure mode	**Wall model**	**Dynamic testing (D)**						**QSC testing (C)**	**D / C**
		Record					**Mean value**		
		CA-71		**CA-77**		**CA-83**			
		50%	**100%**	**75%**	**100%**	**75%**			
"DT"	MVN50m	68	121	257	279	---	181	0.30	604
"DT-DC"	MVN100	72	139	279	368	559	283	0.40	709

Cumulative Parameters

For dynamic and QSC testing, the ductility demand and the maximum number of equivalent cycles at a certain range of ductility (N_{max}) were calculated. N_{max} represents the maximum value of the ratios between the cumulative energy dissipated in a cycle and the cumulative energy dissipated associated to the range of ductility. Data is plotted at the top side of Fig. 6, where fitted regression curves are also shown. The fitted curves show a very good correlation with test data as can be observed from the correlation coefficient, *r*. From the top graphs of Fig. 6, it is apparent that at a certain value of ductility demand, N_{max} was different between the two groups of specimens ("DT" and "DT-DC" failures). Then, for comparison purposes the ductility demand at shear strength (V_{max}) was used. The point is depicted in Fig. 6. It is evident from the top graphs of Fig. 6, that N_{max} is very different between dynamic and QSC testing. For example, in QSC testing, N_{max} at V_{max} was equal to 3 and 6 in specimens with "DT" and "DT-CD" failure modes, respectively. However, in dynamic testing these values were 36 and 91 in specimens with "DT" and "DT-CD" failure modes, respectively. As it was mentioned earlier, in specimen with "DT" failure, shear strength capacity in dynamic testing was higher than in QSC testing; however, in dynamic testing, N_{max} at V_{max} was approximately 12 times (36/3) higher than in QSC testing. In specimen with "DT-CD" failure mode, shear strength capacity in dynamic testing was just slightly lower than in QSC testing, but the difference in N_{max} was more remarkable, about 15 times (91/6) higher. Then, the number of cycles is a key parameter to explain the differences between observed behaviors.



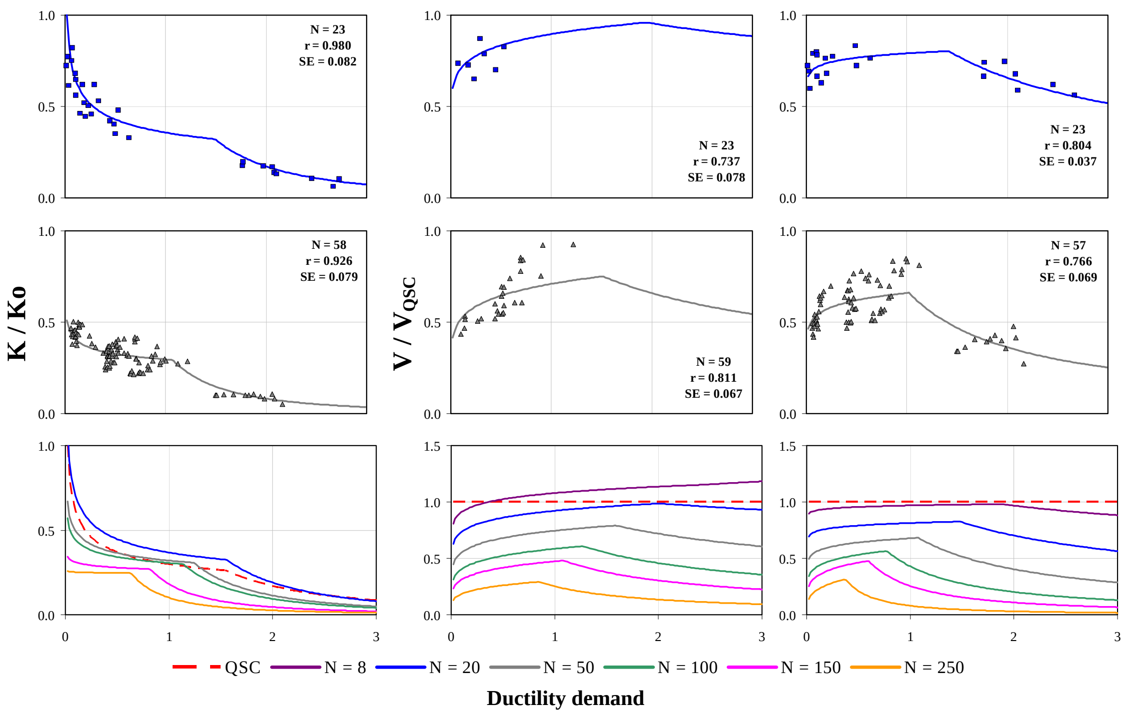
(a) "DT" failure (b) "DT-DC" failure

Figure 6. Variation of the cumulative parameters.

The cumulative energy dissipated and the cumulative ductility demand is shown at the bottom side of Fig. 6. The energy dissipated in a cycle is the area within the hysteresis loop enclosed by the shear force-relative displacement curve. To remove the scale factors in QSC results, the yield shear strength (V_y=0.8V_{max}) times yield drift ratio (R_y) was used to normalize the cumulative energy dissipated (E_{cum}/V_yR_y). In dynamic testing the cumulative energy dissipated at shear strength (V_{max}) was evidently different between specimens MVN50m and MVN100. For example, normalized cumulative energy dissipated was equal to 12.1 and 4.3 in specimens MVN50m and MVN100, respectively. The observed variations in the cumulative energy dissipated are associated essentially with the effect of the number of cycles on the strength mechanisms associated to the failure modes. For instance, when the failure mode is related to cracking or/and crushing of concrete (i.e. "DT-DC" failures), stiffness and strength degradation rates increase noticeably as the number of cycles augment. Also, pinching of hysteresis loops becomes more significant. Then, the hysteresis loops are narrower and, thus, the energy dissipated is reduced. In contrast, when the failure mode is governed by plastic yielding of reinforcement and its subsequent fracture (i.e. "DT" failures), the number of cycles slightly affects the stiffness and strength properties. In quasi-static testing, the normalized cumulative energy dissipated at V_{max} was higher in specimen with "DT-DC" failure than in specimen with "DT" failure (2.8 versus 2.2, see Fig. 6), i.e., the effect of the number of cycles on the cracking or/and crushing of concrete was not observed.

Stiffness and Strength Degradation

Aimed at establishing a stiffness degradation model using the dynamic response and at numerically correlating the dynamic and QSC strength degradation, measured cumulative parameters were studied. Ductility demand, cumulative ductility demand, cycle stiffness (K), the ratio between K and initial stiffness (K/Ko), the ratio between dynamic and QSC normalized strength (V/V_{QSC}), energy dissipated, cumulative energy dissipated and the number of equivalent cycles at a certain value of ductility demand (N) were calculated for each loading cycle for both types of tests. It was observed that failure mode did not significantly affect the stiffness degradation. The type of concrete and the geometry of the wall are the main parameters affecting stiffness behavior (Carrillo 2010). Then, test data for the two specimens tested under dynamic loading and for some values of N are plotted in Fig. 7-a. In contrast, it was observed that strength degradation is remarkably influenced by the failure mode, i.e., crushing of concrete or plastic yielding of steel reinforcement. After that, test data for specimens with "DT" failure mode and for specimens with "DT-DC" failure mode are plotted in Fig. 7-b and 7-c, respectively. Fitted nonlinear regression curves are included in Fig. 7. In this study, two commonly used measures of "goodness-of-fit" of the results of the nonlinear regression analyses were computed: the correlation coefficient (r) and the standard error of the residuals (SE). As shown in Fig. 7, the fitted curves showed a very good agreement with the test data as can be observed from r (close to one) and EE (close to zero).



(a) Stiffness (b) Strength - "DT" (c) Strength - "DT-DC"

Figure 7. Degradation behavior

Following the trends of experimental results, stiffness and strength degradation could be divided into two branches (see Fig. 7). For the stiffness degradation curve (K/K_0), degradation rate of the second branch was larger than that of the first branch. For strength degradation (V/V_{QSC}), the first branch includes pinching of the dynamic hysteresis loops regarding of QSC hysteresis envelope. Using the nonlinear regression analyses, results for the same value of N are depicted in the last row of Fig. 7. QSC results are included in these curves. As it is shown in Fig. 7-a, when N is equal to or lower than 20, dynamic stiffness is larger than that measured under QSC testing. As it was expected, when the failure mode is governed by concrete crushing, strength degradation was considerably apparent (see Fig. 7-b and 7-c). Also, when N is equal to or lower than 8, and the failure mode is governed by plastic yielding of reinforcement, dynamic strength was higher than QSC strength.

Summary and Conclusions

Results of an experimental study of RC shear walls with openings subjected to dynamic and QSC loading were presented and discussed. Shaking table tests were shown to be essential not only for assessing the dynamic characteristics of specimens, but also for verifying QSC test results. It was confirmed that loading history of the QSC testing ignored the principal dynamic effects observed in structures subjected to earthquake loads, mainly, the parameter associated with the strain rate and the number of cycles. When the dynamic and QSC responses were compared, stiffness and strength degradation properties were clearly dependent upon the loading rate, the strength mechanisms associated to the failure modes (crushing of concrete or plastic yielding of reinforcement), the number of cycles, as well as the cumulative parameters such as ductility demand and energy dissipated. It was found that data obtained from QSC tests can not always be safely assumed as a lower bound of the performance capacity of RC shear walls with openings subjected to earthquake-type loading.

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