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SEISMIC PERFORMANCE OF A FULL-SIZE TWO STOREY REINFORCED HIGH-PERFORMANCE CONCRETE BUILDING: EXPERIMENTAL STUDY AND NUMERICAL PREDICTION

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ABSTRACT

Full scale tests provide valuable information on the characteristics of building structures that can be used to evaluate design methods and to determine damage levels. These tests are scarce due to the enormous requirements in testing space and specialized testing equipment. The seismic behaviour of a full-scale two-storey reinforced high-performance concrete building designed with moderate ductility detailing is evaluated by pseudo-dynamic test, during which increasing seismic loads are applied and with resulting increasing levels of permanent damage to the structure. The paper presents the design of the test structure according to the new edition of the Design of Concrete Structure Standard A23.3-04, the series of pseudo-dynamic tests simulating different level of earthquake excitation consistent with the National Building Code of Canada 2005, the evaluation of the performance of the building and the comparison of the predictions of the non-linear response with the test results. It is shown that the detailing requirements of the CSA A23.3-04 are adequate to provide the ductility and the overstrength expected.

Introduction

Figure 1(a) shows the two-storey reinforced high-performance concrete (HPC) building specimen in the foreground. This building was subjected to repeated pseudo-dynamic tests as part of a research project on the seismic behaviour of HPC structures. The main objective of this experimental investigation is to carry out pseudo-dynamic tests on a two-storey reinforced HPC building to: (i) determine the performance under increasing seismic loads, (ii) investigate the force modification and overstrength factors suitable for this structure designed and detailed to exhibit moderate ductility, (iii) investigate whether the 55MPa compressive strength limit of the CSA-A23.3-94 (CSA 1994) is too conservative, (iv) investigate the adequacy of the design and detailing requirements for moderate ductility, (v) provide experimental evidence for predicting damage under increasing seismic loads and, (vi) evaluate the performance of different inelastic time-history dynamic analyses.

High-performance concrete in the Canadian Standard

HSC is more fragile than normal-strength concrete (NSC) and columns made of HSC need to be properly confined to display the same level of ductility as NSC columns in seismic zones. Because of the limited experimental results at the time, the 1994 Canadian Standard CSA-A23.3 limited the specified concrete

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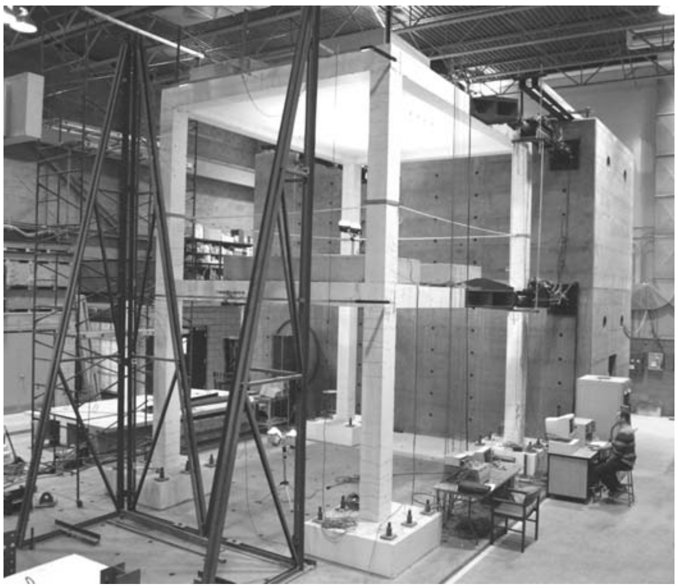
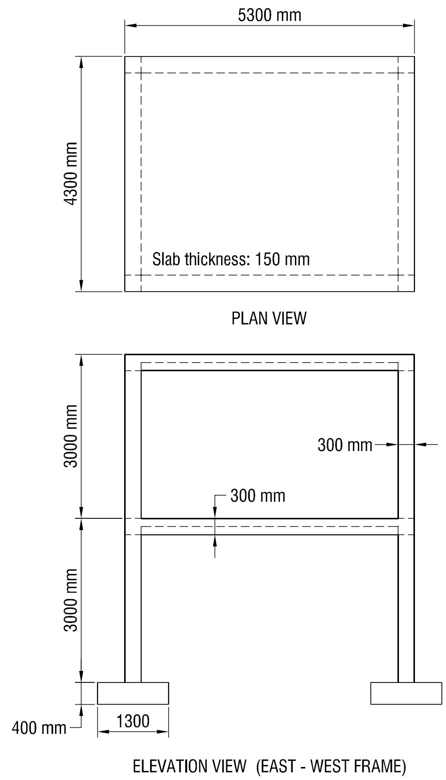
compression strength to 55MPa in seismic zone. A large number of experimental work have shown that HSC columns and beams, if properly detailed, can exhibit sufficient ductility. Most, if not all, tested specimens where single cantilever columns or beams or beam-column-slab sub-assemblages. This research program was developed to study the behaviour of HSC members in a full-size reinforced concrete structure designed according to the new Canadian Standard CSA-A23.3 (CSA 2004) under realistic earthquake excitation.





(a) (b)

Figure 1. Two-storey reinforced HPC building and plan and elevation views of the building.

Moment-resisting frame with nominal ductility

The 1984 version of the Canadian Standard CSA-A23.3 (CSA 1984) on the design of concrete structures introduced new requirements for structures in seismic zones. New design and detailing requirements were added to the standard for the design of frames with what was called at that time nominal ductility. This new ductility level is between the ductile structures built according to capacity design procedures to dissipate energy in specially detailed members and all the other structures.

The suitability of the new requirements for nominal ductility was confirmed by results from reversed cycling loading tests on full-scale beam-slab-column sub-assemblages and by inelastic time-history dynamic analyses (Paultre et al. 1989; Paultre and Mitchell 1987). Ductility level was accounted for by using a force modification factor, R, varying from 1 to 4 for concrete structures in the 1985 version of the National Building Code (NBCC 1985). For a frame with nominal ductility, R was assigned a value of 2.0 which did not change up to the 1995 version of the NBCC (NBCC 1995). One of the objectives of this research program was to study the possibility of increasing the force modification factor for moment resisting frames with nominal ductility from 2.0 to 2.5 to make justice to this type of structure which, it was judged by many designers and members of the CSA A23.3 committee, posses more ductility than the factor 2.0 implied. It is important to mention that when this research program started, the change of the force modification factor from 2.0 to 2.5 was not made yet and that this work was part of the process of making that change.

Design and Description of the Building

The two-storey reinforced concrete building has a 5-m bay in the E-W direction and a 4-m bay in the N-S direction. The storey height measured to top of slab is 3-m. The columns are all $300 \times 300\,\text{mm}$. The two-way slab floor system consists of a slab $150\,\text{mm}$ thick supported by beams $300 \times 300\,\text{mm}$ on all four sides. Specified concrete strength was $70\,\text{MPa}$ and specified steel yield strength was $400\,\text{MPa}$. Plan and elevation views of the building are shown in Fig. 1(b) while Fig. 2 illustrates the reinforcing details.

The design forces used are according to the 1995 National Building Code of Canada (NBCC 1995) for a site located in Montreal. The moment resisting frame concrete structure has been designed for nominal ductility with a seismic force modification factor $R = 2.0$. Specified loads, as prescribed by the NBCC, were used to calculate the design forces (Mousseau and Paultre 2005).

Pseudo-Dynamic Tests Program

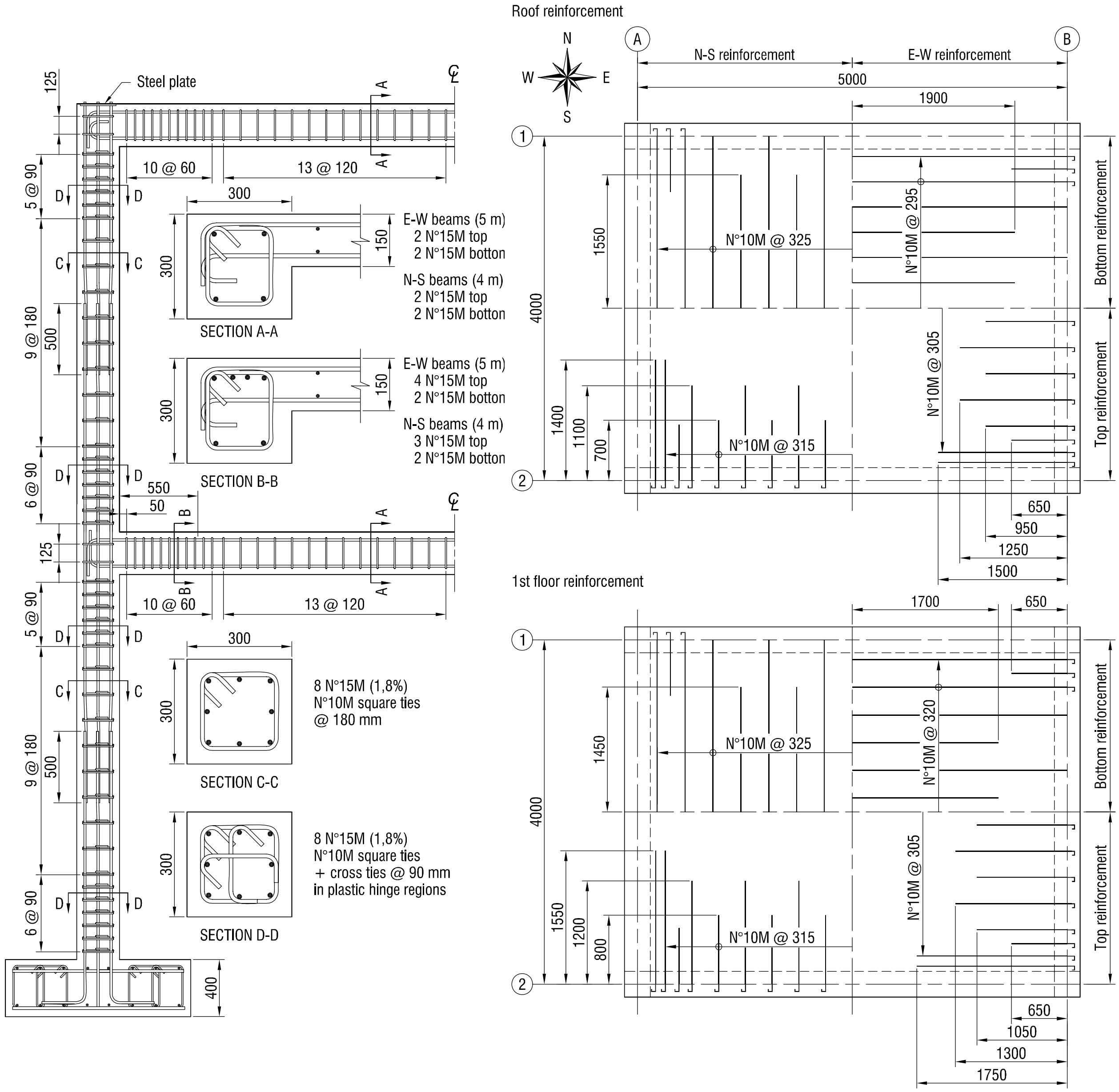
The analysis of buildings subjected to seismic loadings has always proved to be a delicate task. The inelastic behaviour of these structures is often complex to model despite the availability of different structural analysis software. Therefore, experimental investigation remains the most reliable method to predict seismic performance of civil engineering structures and to calibrate numerical models. Shake table tests make it possible to obtain the response of a structure subjected to seismic loadings. The structures are usually small scale specimens due to limitation of the shaking tables. Size effects, therefore, cannot be reproduced and detailing of reinforced concrete structures is problematic. Consequently, shake table studies are generally limited to very simplified models or scaled structures requiring scaling laws that represent poorly reinforced concrete buildings behaviour. The fast development of computer-assisted computations over the last 30 years as well as the possibility to realize a complete on-line computer controlled acquisition systems and data processing allowed the development of new hybrid experimental techniques such as pseudo-dynamic testing method (PSD) (Takanashi 1975).

Instrumentation

PSD tests require essentially the same equipment as conventional quasi-static tests, in which prescribed histories of load or displacement are imposed on specimen structures by means of hydraulic actuators. As shown in Figs. 1(a), the lateral loads were applied to the building by four double-acting servo-hydraulic actuators with a stroke of $\pm 400\,\text{mm}$ and 500-kN capacity. Two actuators were placed at each floor level and were attached at mid-span of the slabs and spandrel beams running in East-West direction. The imposed displacements were measured with respect to two independent steel frames using displacement transducers. The two-storey building was fully instrumented with strain-gauges and displacement transducers to measure the deformations in the longitudinal reinforcement in the beams, columns and slabs and in the transverse reinforcement in the columns and beams.

Test Procedure

Repeated pseudo-dynamic tests were carried out in order to observe the behaviour of the structure under increasing simulated earthquake loading. At the end, a push over test was performed to evaluate the overstrength and the structural ductility of the structure. After each level of seismic load applied to the structure, the dynamic characteristics of the building were obtained by forced-vibration tests. The structural signature is therefore obtained for each loading step and was related to level of damages in the structure (Paultre et al., 2007). The chronology and complete test procedure of the tests performed on the building have been presented by Mousseau and Paultre (2005).



(a) (b)

Figure 2. Reinforcement details of beams and columns and reinforcement details of slabs.

Ground-motion time histories

Two different accelerograms were used for the pseudo-dynamic test. The first ground motion was the S00E component of the accelerogram recorded in El Centro, California, during the May 18, 1940, Imperial Valley earthquake. The first 30-second time-history for the input motion is shown in Fig. 3(b). This ground motion was scaled to different intensities to meet the objectives of the investigation. The second time-history used to carry out the PSD tests, was the M7R70A1 accelerogram generated for Montreal and having a probability of exceedance of 2% in 50 years (Atkinson and Beresnev 1998). This ground-motion is compatible with the uniform hazard spectra (UHS) used in the 2005 NBCC (NBCC 2005, Adams and Halchuk 2003). This time history (Fig. 3(a)) was generated for a moment magnitude 7.0 earthquake at hypocentral distance of 70 km. The peak ground acceleration (PGA) is 0.271g. Fig. 3 presents 5% damped pseudo-acceleration response spectra (PSA) for the ground motion used in the tests. This last figure also illustrates the uniform hazard spectra (UHS) for very dense soil and soft rock (Class C) site located in Montreal according to the 2005 NBCC.

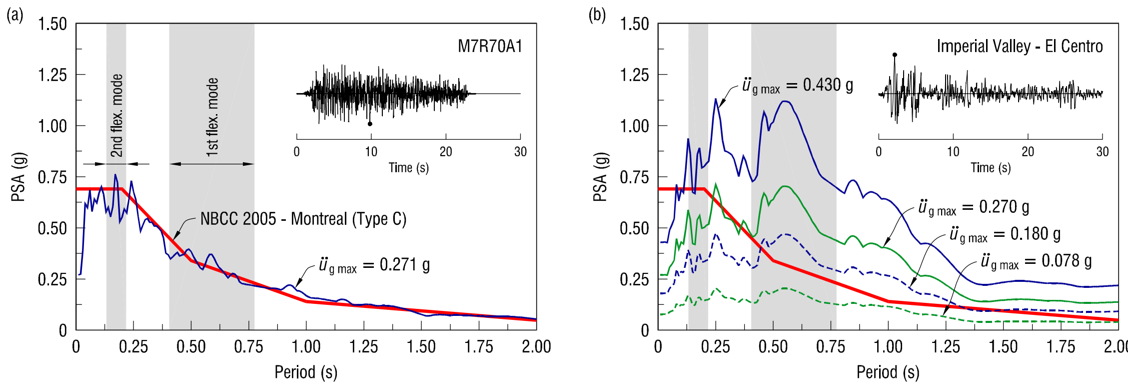


Figure 3. Spectral accelerations and time histories of M7R70A1 (Atkinson and Beresnev 1998) and El Centro Earthquake N-S component scaled at different levels.

Pseudo-Dynamic Test Results and Numerical Prediction

The earthquake responses of the structure, in terms of storey displacement and base shear forces are presented in Figs. 4(a) and 4(b), respectively, for different seismic excitation levels of the Imperial Valley earthquake. Fig. 6(a) presents the response to the M7R70A1 accelerogram. The force-displacement hysteretic curves for all seismic excitation are shown in Figs. 5 and 6(b), where the global hysteretic behaviour of the building is depicted in terms of base shear versus roof displacement. Table 1presents maximum response obtained during the pseudo-dynamic tests carried out on the building. The results are presented in chronological order of the tests.

Table 1. Maximum responses recorded during pseudo-dynamic tests.

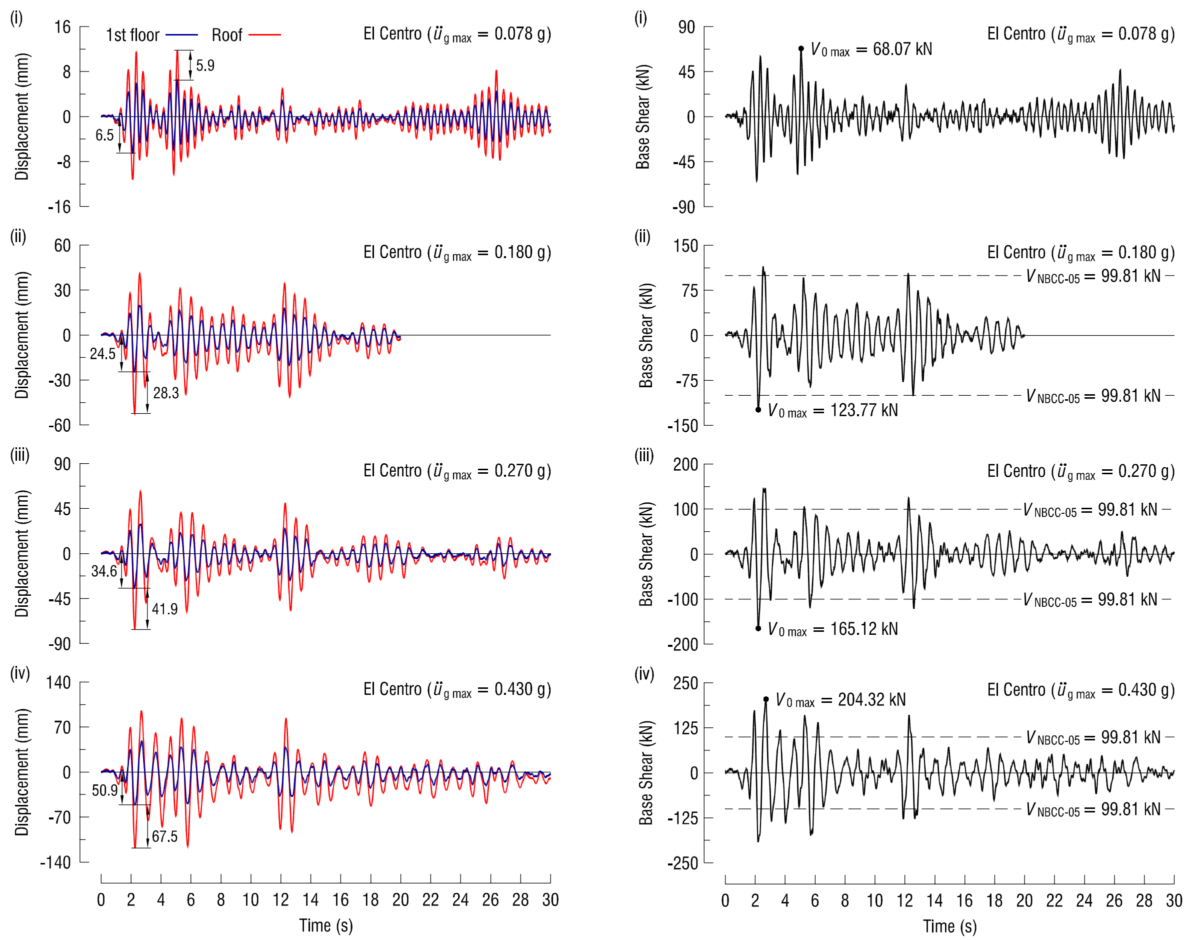
Accelerogram	u_{1max} (mm)	u_{2max} (mm)	Δ_{2max} (mm)	f_{1max} (kN)	f_{2max} (kN)	V_{0max} (kN)	M_{0max} (kN-m)	μ_Δ
El Centro ($\ddot{u}_{gmax} = 0.078$ g)	6.5	11.8	5.2	39.24	42.54	68.07	306.37	0.21
El Centro ($\ddot{u}_{gmax} = 0.180$ g)	24.5	52.4	28.3	67.97	104.08	123.77	661.54	0.93
El Centro ($\ddot{u}_{gmax} = 0.270$ g)	34.6	75.9	41.9	93.26	130.53	165.12	837.12	1.35
M7R70A1 ($\ddot{u}_{gmax} = 0.271$ g)	17.7	39.2	21.6	68.66	66.93	76.32	384.77	0.70
El Centro ($\ddot{u}_{gmax} = 0.430$ g)	50.9	118.4	67.5	110.54	168.98	204.32	1031.35	2.11

Response to low seismic excitation

The responses of the building to the El Centro ground motion scaled to 0.078 g PGA were basically within the linearly elastic range. According to the 1995 NBCC, this level of seismic loading corresponds to a 1% in 1 year probability level for Montreal. The storey displacement time history is shown in Fig. 4(a)i while the base shear versus roof displacement is plotted in Fig. 4(b)i. The maximum interstorey drifts were 6.5 mm and 5.2 mm for the first and second storey which correspond to drift ratios of 0.217% and 0.198% respectively. According to the 2005 NBCC, drift ratios shall be limited to 1%, for post-disaster buildings, 2% for schools, and 2.5% for all other buildings. Maximum base shear was 68.07 kN during the test. Global behaviour of the building during that test was excellent. The inspection of the specimen after the test did not reveal any major crack other than hairline cracks due to the preliminary forced-vibration tests which had been performed before the first PSD test to obtain initial dynamic properties (frequencies and mode shapes).

Response to moderate seismic excitation

Moderate seismic excitation was achieved by scaling the El Centro ground motion to 0.18 g PGA. The earthquake responses of the structure are shown in Figs. 4(a)ii, 4(b)ii and 5(b). A small period elongation can be observed from the response history. All floor displacement time-histories are in phase pointing out to a dominant first mode participation. Under this seismic input, the structure exhibited significant cracking in the beams, columns and slabs but performed with no measured yielding of the reinforcement and no spalling of the concrete covers. The building displayed a maximum first storey displacement of 24.5 mm and a maximum top storey displacement of 52.4 mm. Maximum storey drifts are 24.5 and 28.3 mm for the first and second storey which are about 82% and 94% of the maximum 1% interstorey drift ratio allowed by the code for post-disaster buildings. Note that the maximum base shear measured just reached the design base shear (99.81kN) of the 2005 NBCC. This moderate level PSD test indicates that HPC performs well up to this level of excitation with very little damage.



(a) (b)

Figure 4. Time histories of (a) 1st floor and roof displacements and (b) base shear for El Centro Earthquake N-S component scaled to (i) 0.078 g PGA, (ii) 0.180 g PGA, (iii) 0.270 g PGA and (iv) 0.430 g PGA.

Response to severe seismic excitation

Two different PSD tests were carried out at this level of excitation. The first test was done with the El Centro ground motion scaled to 0.27 g PGA. As it can be seen in Fig. 3(b), the pseudo-acceleration response spectra for this level of excitation at the measured periods of vibration of the building is clearly higher than the design spectra for Montreal. The responses of the structure for the first test are shown in Figs. 4(a)iii, 4(b)iii and 5(c). A vibration period of approximately 0.68 s can be observed from the storey displacement time history. Maximum interstorey drifts were 34.6 mm and 41.9 mm for the first and second storey which correspond to drift ratios of 1.153% and 1.396% respectively. These values exceed the drift ratio allowed by the 2005 NBCC for post-disaster buildings, but are lower than the drift ratios for school or for all other buildings. Maximum base shear developed during the test was 165.12 kN which is significantly higher than the design base shear (99.81 kN). This test caused the first measured yielding of reinforcement steel. Measured strains in the longitudinal reinforcement at the bottom of the first storey column reached $1.175\varepsilon_y$. At the time of this first yielding, base shear was 144.44 kN. The corresponding overstrength is $144.44/99.81 = 1.45$. More discussions will follow on this subject. The structure performed very well during this test. Few new cracks developed, but existent cracks widened. Neither spalling of the cover, nor local instabilities of reinforcement were observed.

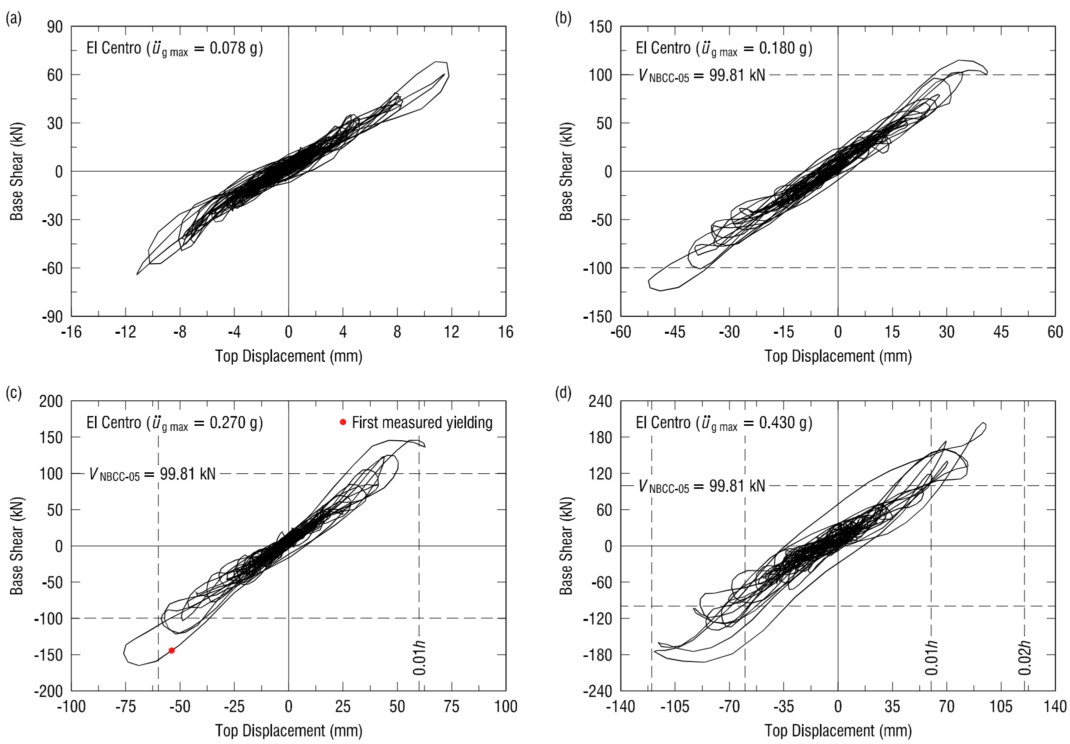


Figure 5. Base shear-roof displacement response for El Centro Earthquake N-S component scaled to (a) 0.078 g PGA, (b) 0.180 g PGA, (c) 0.270 g PGA and (d) 0.430 g PGA.

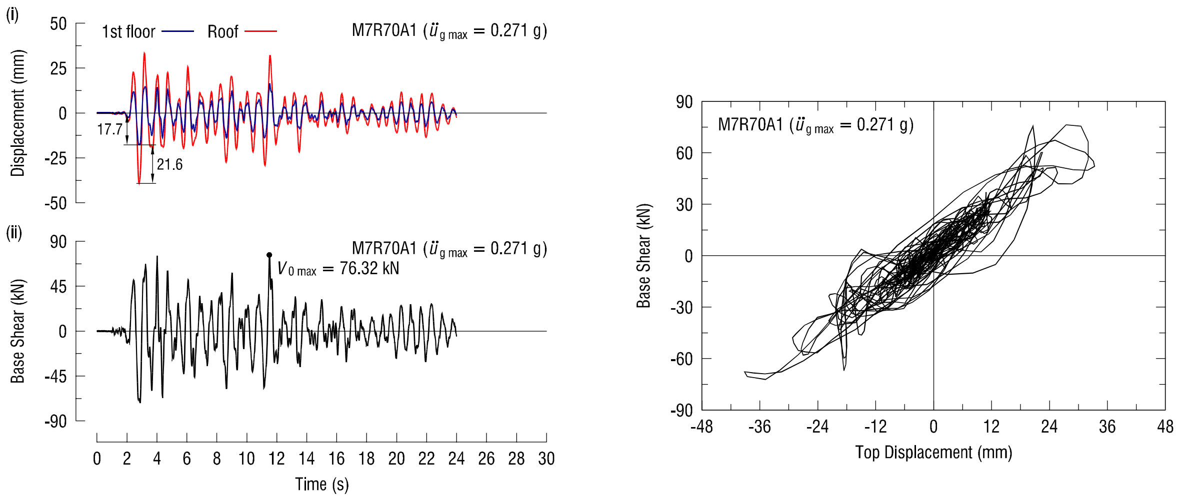
The second test was carried out with the M7R70A1 accelerogram generated for Montreal (see Fig. 3(a)). The peak ground acceleration of this accelerogram is 0.271g. The PSD test results obtained with this accelerogram are shown in Fig. 6. The maximum interstory drifts were 17.7 mm and 21.6 mm for the first and second storey, respectively. These values are less than the limits imposed by the 2005 NBCC for all types of structures. Furthermore, as shown in Fig. 6(b), the maximum base shear developed was only 76.32 kN, significantly less than the design base shear (99.81 kN) of the 2005 NBCC.

No new crack appeared during this test. Existing cracks opened (and closed) in the critical regions of the beams and the columns according to the displacements imposed to the structure by the seismic forces.

Measured displacements and storey shears were clearly lower than those measured during the PSD test with the El Centro ground motion scaled to 0.27 g PGA, despite the fact that maximum ground accelerations of these two seismic functions are almost identical. This confirms that the spectral content of an earthquake is a parameter much more important than peak ground acceleration. The spectral approach introduced in the 2005 NBCC seems to represent the regional seismic conditions better that the approach bound solely to the peak ground acceleration as it is the case in the 1995 NBCC. This result also highlight that the high frequency content of eastern Canada earthquakes are not critical for flexible structures.

Response to very severe seismic excitation

The last pseudo-dynamic test was performed with the El Centro ground motion scaled to 0.43 g PGA. This PGA level corresponds to a 2% in 50 year probability level for Montreal (Adams and Halchuk 2003). Even though this PGA value is in accordance with the prescribed acceleration for Montreal, the pseudo-acceleration response spectra for this level of excitation is clearly higher than the design spectra of Montreal for all given frequencies (Fig. 3(b)). This level was used to study the behaviour of the structure under very severe seismic excitation.



(a) (b)

Figure 6. (a) Time histories of (i) 1st floor, roof displacements and (ii) base shear and (b) base shear-roof displacement response for M7R70A1 time-history scaled to 0.271g PGA.

The earthquake responses of the structure are shown in Figs. 4(a)iv, 4(b)iv and 5(d). An important period elongation can be observed from the response history. The measured vibration period is about 0.85 s. The maximum interstorey drifts were 50.9 mm and 67.5 mm for the first and second storey which correspond to drift ratios of 1.70% and 2.25% respectively. These values exceed the ratio allowed by the 2005 NBCC for post-disaster buildings. The interstorey drift of the second floor also exceed the limit of 2% imposed to schools. On the other hand, limit concerning all other buildings was not exceeded. Maximum base shear was 204.32 kN during the test, which is more than twice the design base shear. The analysis of the hysteretic curve reveals important inelastic behaviour of the structure. The longitudinal reinforcement in the beams, near the column faces, and at the base of the first storey columns suffered inelastic tensile strains. The columns longitudinal bars, below the first floor beam-column joints and all the longitudinal bars of the second storey columns remained in the elastic range of the steel reinforcement. This intensity level caused some damages to the structure. Several new cracks in the beams, the columns, the slabs and the joints appeared and most of the existent cracks widened significantly. On the other hand, neither spalling of the cover, nor local instabilities of reinforcement were observed. Despite this very high level of excitation, the building preserved his structural integrity and his capacity to sustained gravity loads.

Numerical prediction

Numerical predictions were made using the program *EFiCoS*, a 2D layered beam element damage mechanics-based finite-element program. This program uses a concrete constitutive law that takes account for the following phenomena: cracking in tension; confinement effect in compression; and cyclic behaviour (Légeron et al. 2005). Earthquake effects are generated by applying acceleration history at the supports and writing the equations of motion in terms of total displacements. For this analysis, the unconditionally stable Newmark constant acceleration step-by-step integration method was used together with Rayleigh damping. The displacements at the roof and the base shear recorded during the test with the El Centro ground motion scaled to 0.43 g PGA are compared with the predicted responses using *EFiCoS* in Figs. 7(a) and 7(b), respectively. As it can be seen from Fig. 7, the predicted responses are in very good agreement with the test data.

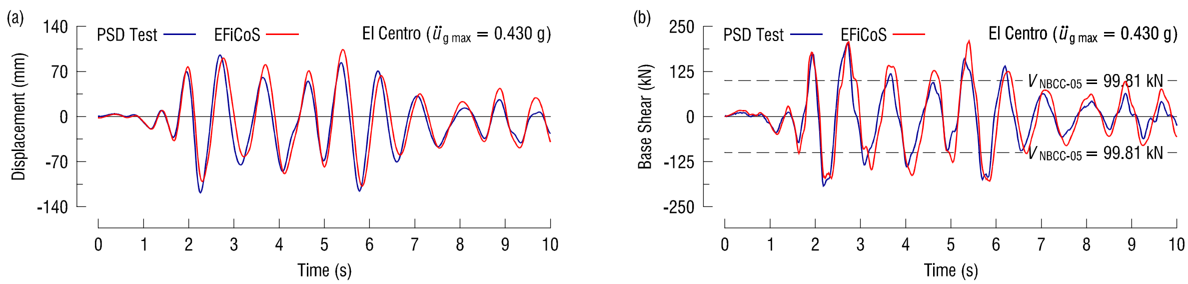


Figure 7. Prediction of roof displacement and base shear.

Analysis of Test Results

According to Table 1, the base shear increased from 68.07 kN (9.0%W) to 204.32 kN (26.9%W) during the pseudo-dynamic tests carried out on the building, what represents a 300% increase. For the same tests, roof displacement increased by approximately 1000%. Structural ductility demand for the PSD tests progressed from 0.21 to 2.11. Even at this last level of ductility, the global behaviour of the building was still very good. The 1% drift limit for post-disaster buildings was exceeded during the 0.180 g PGA test with the El Centro ground motion while the drift ratio for schools (2%) was reached at second storey during the PSD test with 0.430 g PGA. As for the 2.5% limit prescribed for all other buildings, it was never exceeded during the pseudo-dynamic tests carried out on this building.

Conclusions

Forced vibration were used to extract the structure's key dynamic properties (vibration frequencies and mode shapes, as well as modal damping ratios) to be used in PSD tests and to set the initial state of the structure. Several PSD tests with increasing seismic excitations demonstrated the excellent performance of this high-strength concrete structure designed with the new CSA A23.3 Standard under earthquake excitations. In particular, displacement ductility and overstrength are higher than the prescribed values in the new 2005 NBCC. HSC columns designed with the new requirements of the A23.3 Standard performed very well under PSD tests and push over test even at drift levels significantly larger than the limits allowed by the 2005 NBCC. These results confirm the appropriateness of the new CSA A23.3 Design of Concrete Structures Standard, in particular (i) new strong column weak beam requirements, (ii) new confinement requirements for columns. The test results also confirm the safe values of the ductility related force modification factor and the overstrength related force modification factor. These unique test results can be used for calibration of numerical models and as gauges for expected damage levels corresponding to sectional behaviour.

Acknowledgements

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