

MULTI-PURPOSE EARTHQUAKE SIMULATION TESTING SET-UP FOR SEISMIC FORCE RESISTING SYSTEMS OF MULTI-STOREY BUILDINGS

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ABSTRACT

The paper describes an experimental setup that was designed for the earthquake simulation testing of multi-storey building models at the Structural Engineering Laboratory of École Polytechnique of Montréal, Canada. The 10 m tall test apparatus has been designed to fully exploit the capacity of the existing shake table facility. It is capable of accommodating various seismic force resisting system models for buildings up to 10 storeys in height including braced steel frames, reinforced concrete shear walls, and steel frame/wood panel shear walls. The paper first presents three different test programs which will be carried out in the coming years to study the dynamic stability and higher mode response of multi-storey frames under strong seismic ground motions. Details of the experimental setup are then presented, including the lumped mass system that is designed to simulate the seismic floor weights located at every level of the prototype buildings. These masses can be positioned vertically to reflect the number of storeys and storey height of the prototype building studied. They are vertically supported by columns that are independent of the test structure, simulating the gravity load frame typically found in actual buildings. Dedicated braced steel frames are used to provide elevated working areas for the researchers and to laterally brace the test model and the floor mass assembly. Numerical studies are performed to verify the appropriateness of the proposed test system.

INTRODUCTION

Nonlinear time step dynamic analysis has become common practice to study the inelastic seismic response of structural systems and to assess their seismic performance. This analytical procedure has been proven to adequately predict the behaviour of seismic force resisting systems that have well defined inelastic mechanisms with stable hysteretic response, such as beam flexural hinging or axial buckling and yielding. The analysis of structural systems that involve complex interaction between various structural components or multiple load paths with different response characteristics still represents a challenge that requires careful experimental validation. In many cases, the validation tests must be performed on the whole structure, or on large portions of the structure, to adequately capture overall structural response including the various interrelated effects that take place under strong ground motions. Although novel pseudo-dynamic and hybrid test methods have been recently developed, shake table testing still remains an efficient and appropriate tool to perform such experimental studies.

Three such structural systems are currently being investigated through a joint research program at École Polytechnique and McGill University in Montréal, Canada. The three projects deal with: i) the dynamic inelastic stability of multi-storey braced steel frames under seismic ground motions, ii) the prediction of the seismic storey shear demand and higher mode effects in slender reinforced concrete shear walls used in multi-storey buildings, and iii) the seismic response of multi-storey buildings braced with shear walls constructed of light gauge steel frames that are sheathed with wood based panels.

The research program involves numerical studies and component testing, in addition to the preliminary design of a multi-purpose setup to perform earthquake simulation testing of large specimens to validate analysis results.

An experimental apparatus has been developed which will be installed in the new Structural Engineering Facility under construction at École Polytechnique of Montréal. This paper briefly presents the three ongoing research projects and describes the proposed shake table test setup. The last section of the paper presents numerical simulations that have been performed to verify the feasibility of the test program with the proposed experimental arrangement.

RESEARCH PROGRAMS

Seismic stability of braced steel frames

Several numerical studies have shown that braced steel frames in multi-storey building applications with non-moment-resisting beam-to-column connections have limited capacity to redistribute the inelastic demand over their height (e.g., Sabelli et al., 2003; Tremblay 2000, 2003; Tremblay and Robert, 2001). The analyses indicate that such structures are prone to storey response and that P-delta effects can lead to instability and, eventually, structural collapse. This phenomenon is more pronounced in taller frames, typically 8 storeys or more, when subjected to long duration or pulse type earthquake ground motions. This behaviour is illustrated in Fig. 1a for a braced frame built with conventional steel braces that buckle in compression. Various structural systems have been proposed to mitigate this phenomenon and achieve stable inelastic response under maximum considered earthquakes (e.g., Tremblay, 2003; Tremblay and Merzouq, 2004; Tremblay and Poncet, 2004). Figure 1a shows one such system. In this case, the braced frame includes an elastic truss system as well as buckling restrained brace (BRB) members with stable and symmetrical hysteretic response.

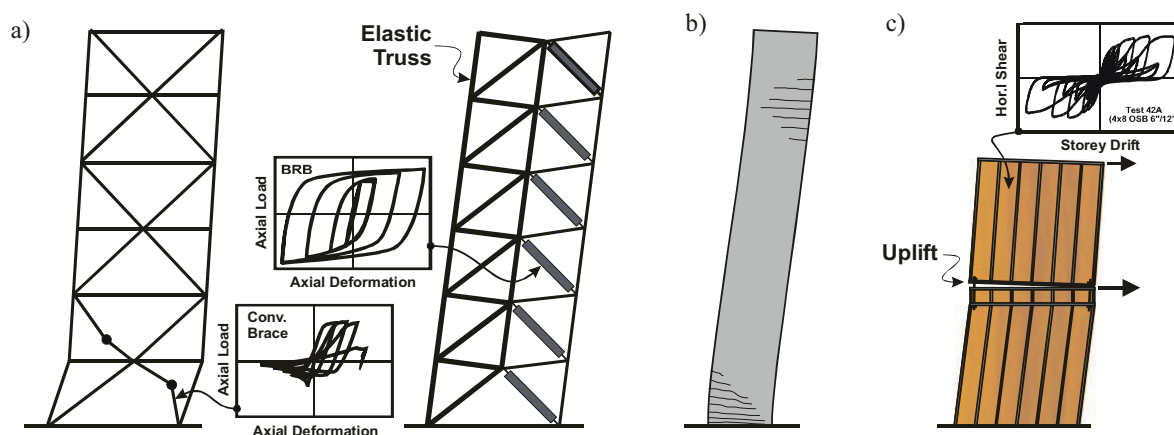


Fig. 1 Research topics for earthquake simulation testing: a) Seismic stability of braced steel frames; b) higher mode effects in slender R/C walls; and c) Seismic response of steel frame/wood panel shear walls.

Large displacement inelastic dynamic analyses are conducted to characterize the conditions leading to dynamic instability and to evaluate the efficiency of the structural schemes that are proposed to maintain stability. The results of such analyses have been found to be sensitive to modeling assumptions, and hence, experimental validation is essential to provide guidance for further analytical studies. The test setup must accommodate multi-storey model structures and permit the reproduction of the destabilizing effects of the building gravity load carrying structure (leaning columns). The examination of instability failure modes also requires the development of large drift angles of up to 10% at any storey or 5% for global collapse mechanisms.

Higher mode effects in slender R/C walls

Ductile cantilevered reinforced concrete shear walls designed according to modern building codes are typically detailed to undergo plastic hinging at their bases. Both the design moment envelope for the remaining portion of the wall and the design shear forces are evaluated based on the probable flexural resistance of the wall in the plastic hinge region (Paulay and Priestley, 1992). Several analytical studies (e.g., Filiatrault et al., 1994; Tremblay et al., 2001; Priestley and Amaris, 2002; Panneton et al., 2005) have shown that so-designed structural walls can be subjected to shear forces in excess of the design values. Plastic hinging can also develop in the upper portion of the walls, as illustrated in Fig. 1b. These effects are mainly attributed to higher mode response and, hence, are more severe in taller or slender walls with long fundamental periods.

Typical analytical models used to investigate this problem only account for inelasticity related to flexural hinging, where shear stiffness degradation due to cracking in plastic hinge regions is generally ignored. Testing of wall

models under dynamically applied ground motions should allow for the validation of the shear and bending moment demand on structural walls, including higher mode effects and the interaction between shear and flexure inelastic responses. Scaling effects on the nonlinear response of reinforced concrete structures can be significant and large scale physical modeling is a necessary condition to develop reliable results for this particular problem. Assessment of the shear and flexural force demand also requires a continuous monitoring of the lateral force demand over the height of the model during testing. Finally, the experimental setup should allow for the study of the response of walls for which rocking is permitted at their base. This behaviour is seen by many as an efficient avenue to reduce the force demand in structural walls, and it is planned to address this problem in future studies.

Seismic response of multi-storey light gauge steel frame/wood panel shear walls

Light gauge steel stud construction represents a promising alternative for housing and small building constructions in view of the increasing cost of wood products. A research project is currently underway at McGill University to develop seismic design provisions for shear walls made with wood based panels (plywood or oriented strand board (OSB)) screwed to light gauge steel stud and track assemblies (Rogers et al., 2004). Extensive quasi-static cyclic testing has been performed on shear walls to acquire knowledge of their inelastic seismic behaviour and to develop a reference database for the calibration of numerical models used to predict the response of actual buildings.

As indicated in Fig. 1c, steel frame/wood panel shear walls typically exhibit severely pinched hysteretic response, which encourages storey response characterized by large storey drift angles at the bottom level of multi-storey buildings. Numerical prediction of such response is found to be sensitive to the selection of model parameters used in the analysis, and hence testing under strong ground shaking is needed to confirm current assumptions. As for concrete structures, size effects in steel frame/wood panel construction can be of significance, which requires that testing preferably be conducted on full-scale wall specimens. In addition, the performance of multi-storey shear walls can be strongly influenced by the behaviour of the floor connections and vertical hold-down devices that are used to transfer the overturning forces from one storey to the next. Uplift is likely to develop at the base of the wall units and such response also needs to be characterized in the test program for inclusion in numerical models.

EARTHQUAKE SIMULATION TESTING SETUP

Earthquake Simulation Facility

The new Structural Engineering Laboratory currently under construction at École Polytechnique of Montreal will become operational in 2005. The facility will house a 606 sq. m. strong floor area, an L-shaped, 10 m tall reaction wall, and a multi-purpose 12 MN load frame with 3 m wide x 8 m tall test space (see Fig. 2a). The existing uniaxial earthquake simulator will also be installed in the new testing facility. The simulator has a payload capacity of 15 tons and 3.4 m x 3.4 m plan dimensions. The multi-cellular shake table box is mounted on four frictionless linear hydrostatic bearings, as illustrated in Fig. 2b. The system features a fully digital three-variable control system with delta pressure stabilization, amplitude phase control, online iteration, and adaptive inverse control capabilities. The controller also has real-time hybrid testing capabilities for up to three degrees of freedom.

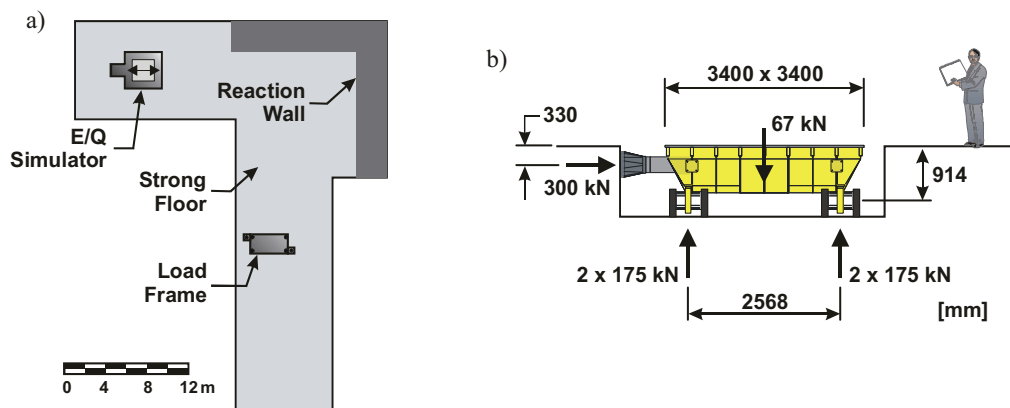


Fig. 2 Structural engineering testing facilities at École Polytechnique:
a) Plan view of the new test area; b) Characteristics of the earthquake simulator.

The actuator is powered by a hydraulic power supply with a total flow capacity of 1400 l/min. The frequency

range is 0-50 Hz and the system can achieve peak displacements of ± 125 mm, peak velocities of 1.0 m/s and peak accelerations ranging between 3.0 g (table empty) and 1.0 g (at full payload capacity). The earthquake simulator is serviced by a 15 ton overhead crane and the vertical clearance between the test platform and the crane hook is 11 m. Figure 2b gives the geometry of system, the weight of the shake table, the service capacity of the actuator, and the vertical capacity of the linear bearings. The latter governs the overturning bending moment that can be resisted by the test apparatus.

Proposed Test Setup

Each of the three research projects requires testing of individual multi-storey braced frame or wall models loaded in their plane, which can be efficiently accomplished with a uniaxial shake table facility having large height clearance such as the one described earlier. Figure 3 shows a plan view and an elevation of the test setup that is proposed for this test program, with a braced steel frame specimen mounted in the test rig. The same experimental setup can be equally used for testing concrete shear wall specimens or steel frame/wood panel shear wall specimens, as shown in Fig. 4.

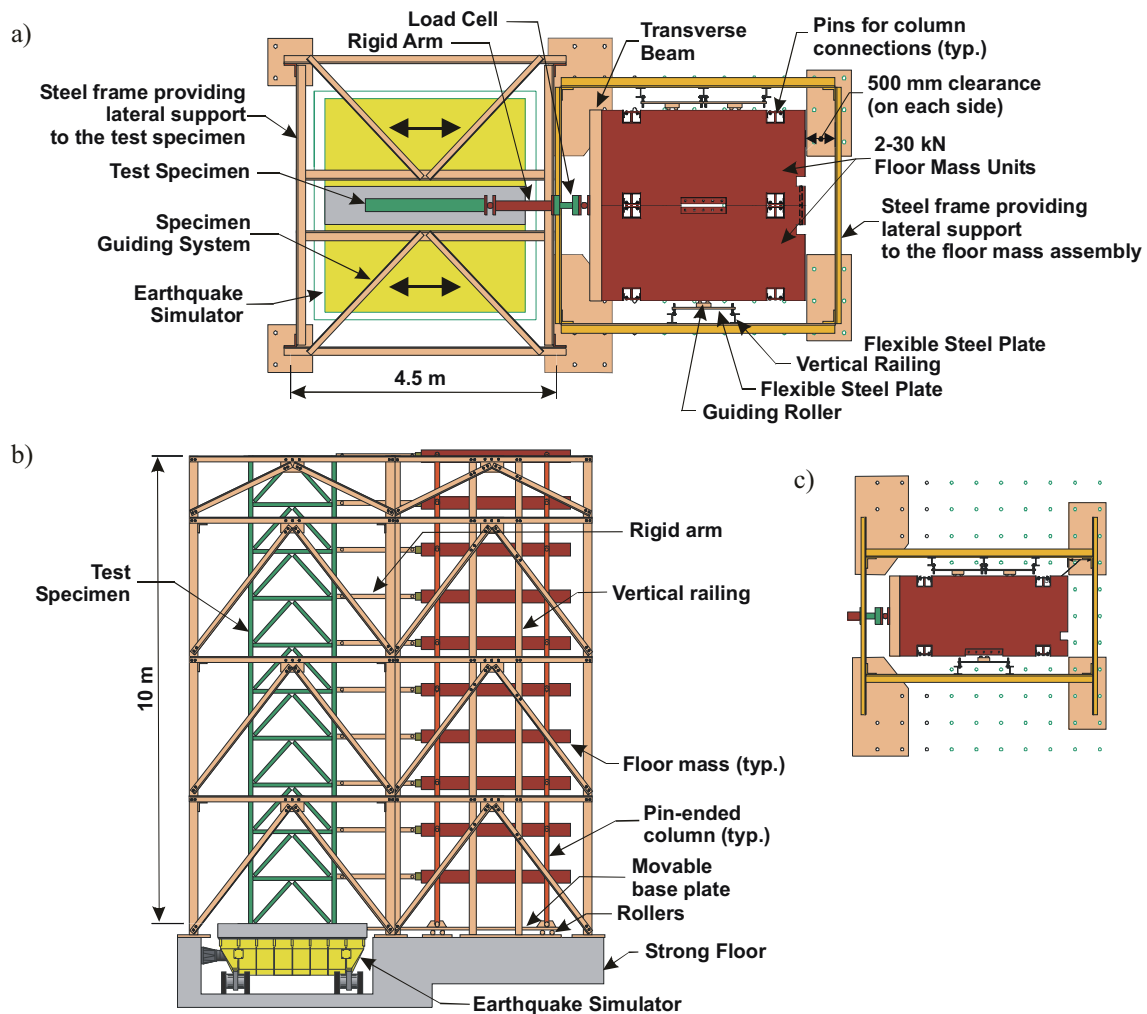


Fig. 3 Proposed test setup with a braced steel frame specimen: a) Plan view (two-mass configuration); b) Elevation view; and c) Single-mass arrangement (specimen not shown).

The proposed test arrangement can accommodate structural models up to 10 storeys with a total height of 10 m. The seismic weight at each floor is simulated using one or two 30 kN concrete blocks. The maximum seismic weight for a model, excluding the specimen self weight, is then equal to 600 kN. In order to maintain maximum payload capacity for the test specimen as well as overturning bending moment capacity, the concrete blocks simulating the floor masses are located at some distance away from the test specimen and the shake table, and hence are vertically supported on the laboratory strong floor. At every floor, a rigid arm is used to horizontally attach the floor mass units to the test specimen, as described later. The weight of the floor masses is carried by means of pin-ended column elements that connect between adjacent concrete blocks. These columns are made of

steel tubes and threaded steel rods so that their height can be adjusted to match the model storey height. In the direction of loading, the columns are truly pin-connected at their two ends so that the concrete block/supporting column assembly provides no resistance to lateral displacements. At the base, the columns are pin-assembled to longitudinal steel base plates that extend horizontally up to the earthquake simulator where they are connected. Rollers are used between the base plates and the strong floor and, hence, the plates experience the imposed ground displacements. With this configuration, the test specimen must resist P-delta moments induced by the seismic weights following the relative lateral deformations of the specimen, thus replicating the influence of gravity frames in actual buildings.

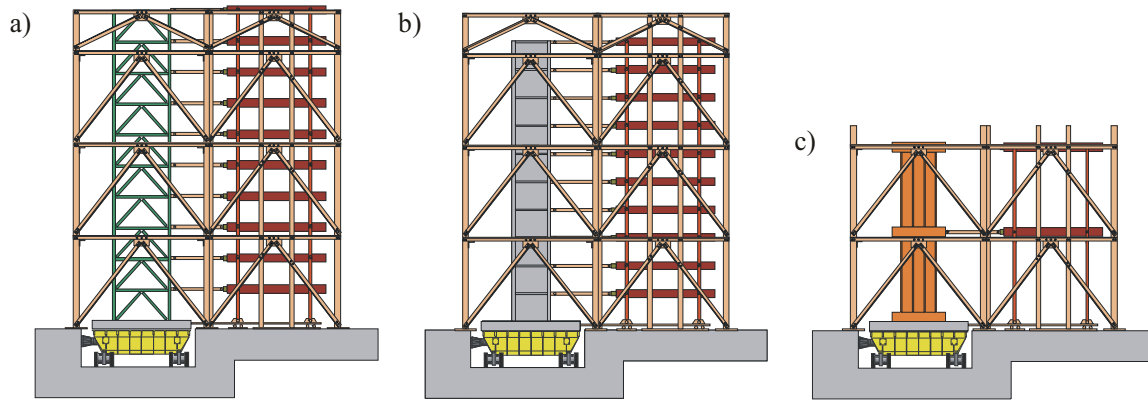


Fig. 4 Setup proposed for testing: a) 1:4 model of a 10-storey braced steel frame; b) 1:3.28 model of a 10-storey reinforced concrete shear wall; and c) 1:1 model of a 2-storey steel frame/wood panel shear wall.

The floor mass assembly is laterally supported by a 10 m tall steel frame with 3.5 m x 4.4 m plan dimensions. Chevron bracing is provided along the four sides of the bracing frame. During the construction of a test model, the seismic weight units are secured to the steel frame in the longitudinal (test) direction. During testing, these connections are replaced by safety steel cables that permit the anticipated longitudinal horizontal movements to take place. In the transverse direction, horizontal movements of the concrete blocks are restrained by means of longitudinal rollers that are mounted on flexible steel plates attached to continuous vertical rails that extend the full height of the steel frame. These rails are connected to the surrounding steel frame members and the floor masses can be positioned at any vertical location. The flexible mounting assembly for the longitudinal rollers can be tuned to provide sufficient lateral strength to resist out-of-plane forces due to imperfections or damage to the test specimen while developing low frequency transverse response of the masses to minimize the potential for resonance during a test. When two concrete blocks are used at a given floor, the blocks are joined together by means of connectors located along their common edges. A stiff transverse steel beam located along the edge facing the test specimen links the floor weights to the test specimen and also connects the two blocks together. As shown in Fig. 3c, a transverse beam is also installed when a single floor mass configuration is used. In such a case, the transverse dimension of the steel frame is reduced to provide for the lateral guidance of the floor masses.

At each level, the test model is connected to the seismic weight by means of a rigid arm made of a steel tube with high axial stiffness. As illustrated in Fig. 3, a load cell is mounted between each of the rigid arms and the transverse beams, which allows for direct measurement of the floor inertial loads at every floor during a test. The connection between the test specimen and the connecting arm will vary depending on the type of specimen. For reinforced concrete and steel frame/wood panel shear walls, the specimens would be built with floor segments extending on their two sides. A pair of longitudinal steel members connected to the rigid arm would be secured to the two floor segments by means of uniformly spaced fasteners, which would lead to uniform horizontal shear transfer to the specimen, similar to what is found in actual structures. For steel specimens, direct bolted connections between the specimen and the horizontal rigid arms will be used.

In a similar fashion, a second steel frame is used to provide lateral stability for the test specimen. This frame is also used to support working platforms on both sides of the test model at every 2.7 m. These platforms will give the researchers direct access to the specimen for the installation of the instrumentation or visual inspection after testing. The concrete floor masses have been sized such that they can also fit into the steel frame surrounding the earthquake simulator, allowing tests where the floor weights need to be directly connected on each side of the test model. In this configuration, the concrete blocks would be supported directly by the earthquake simulator and this arrangement would therefore be limited to models with relatively low seismic weight.

ANALYTICAL PREDICTION OF TEST MODEL RESPONSES

Seismic stability of braced steel frames

A 10-storey chevron buckling restrained braced steel frame was studied analytically to validate the experimental setup. The storey height in the prototype building is 4.0 m and the scale factor for length, l_r , was set to 1/4 to meet the model height limit of 10 m. A first order model with artificial mass-simulation was adopted (Monczar and Krawinkler, 1981) with the following dimensionless products: a/g , Δ/l , F_y/E , $t^2 EI/M$, and aM/EI^2 , where a is the horizontal acceleration, g is the acceleration due to gravity, Δ is the relative lateral displacement, F_y and E are the steel yield strength and Young's modulus, respectively, t is the time, l is the length, and M is the lumped floor mass. The scale factor for accelerations, g_r and a_r , were taken as 1.0, the scale factor for displacement $\Delta_r = l_r$, and steel was assumed to be used for the fabrication of the structural components of the model, i.e. $E_r = 1.0$ and F_{y_r} in the model is the same as in the prototype. The mass ratio, $M_r = l_r^2 = 1/16$ and the time ratio, $t_r = l_r^{0.5} = 1/2$.

The plan view of the prototype building is shown in Fig. 5a. The dimensions were back-calculated such that the seismic weight in the model corresponded to 60 kN per floor (2 concrete blocks). The prototype building was assumed to be located on firm ground in Victoria, B.C., Canada. It was designed according to the provisions of the upcoming 2005 National Building Code of Canada (NBCC) (Heidebrecht, 2003). The design base shear ratio, V/W , for this structure was equal to 2.7% and the structure had a fundamental period of vibration of 2.27 s (1.14 s for the model). Figure 5b shows the time history of the roof lateral displacements for both the prototype and model structures under a long duration subduction earthquake ground motion (Tremblay and Atkinson, 2001). The analyses were carried out using the Ruaumoko computer program (Carr 2002) with BRB modeling as described in Tremblay and Merzouq (2004). Collapse by instability occurs for the prototype building. Similar response is obtained for the model structure but P-delta effects are less critical. In the analysis of the prototype structure, the companion live load was included in the gravity loads, as typically required in building codes. In the model, the gravity load corresponded to the seismic weight and, hence, included only the dead load. Both structures are found to exhibit identical response when only the dead load is considered in the analysis of the prototype building, confirming the possibility of reproducing actual building response with the test model.

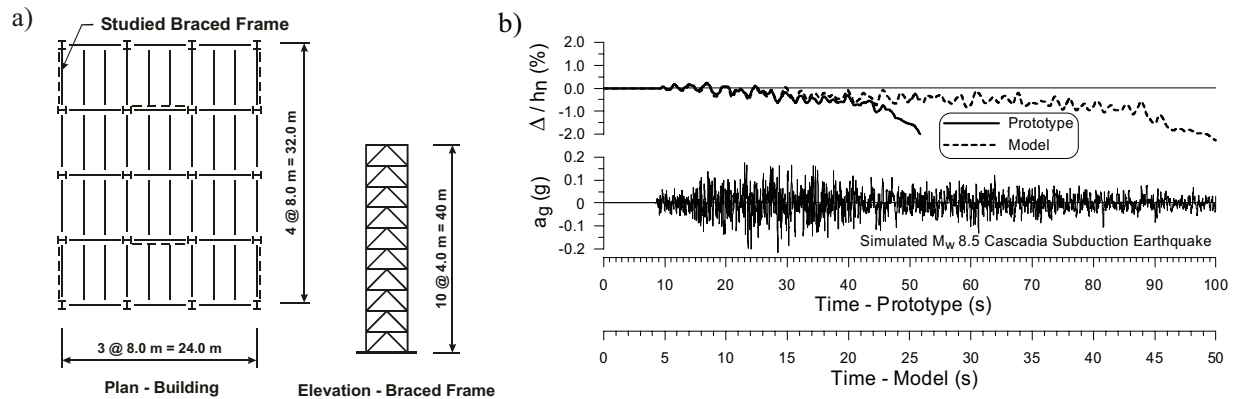


Fig. 5 Analysis of braced steel frame prototype and model: a) Prototype structure; b) Roof lateral displacement response under a simulated subduction earthquake ground motion.

Higher mode effects in slender R/C walls

A 10-storey building was also considered to verify the system for a ductile reinforced concrete shear wall test. The same modeling technique was also adopted except that a scaling factor $l_r = 1 / 3.28$ was selected for length and that a scaling factor of 3.0 had to be applied to horizontal accelerations ($a_r = 3.0$) in order to obtain realistic plan dimensions for the prototype building when using the 60 kN seismic weight per floor in the model. Since $g_r = 1.0$, this test model is not exact considering that the gravity loads in the model are 3.0 times less than would be required. However, this discrepancy has negligible impact for this project as gravity load effects on the lateral response of this test specimen are very small. Moreover, the as-built test conditions will be replicated in the analytical models used to predict and reproduce the experimental measurements. Therefore, this discrepancy will not affect the ability to use the test data to validate numerical models. The same material (concrete and steel) will be used for both the prototype and the model, which means that $E_r = 1.0$. From the dimensionless products, it can be found that $\Delta_r = l_r$, $M_r = l_r^2 / a_r = 0.03098$, and $t_r = l_r^{0.5} / a_r^{0.5} = 0.3187$.

Figure 6a shows details of the prototype building. The design was performed for Vancouver, B.C., Canada, according to NBCC 2005 and the CSA-A23.3 Standard for the design of concrete structures in Canada (CSA 2004). Firm ground conditions were assumed. Nonlinear dynamic analyses were performed using the Ruaumoko computer program. The

plastic hinge flexural response was modeled with the Takeda model. Additional analyses were carried out using a shear stiffness degradation model coupled with the flexural demand in the plastic hinge. Figure 6b compares the design shear forces to the peak values obtained from the analysis of the prototype wall with and without shear stiffness degradation. For this particular ground motion, the results from both models exceed significantly the design values near the base of the walls. Shear stiffness degradation only affected the response in the plastic hinge region. The same results were obtained from analyses of the test model, as illustrated in Fig. 6c for the time history of the roof lateral displacements, indicating that the experimental setup should provide realistic estimates of the wall response.

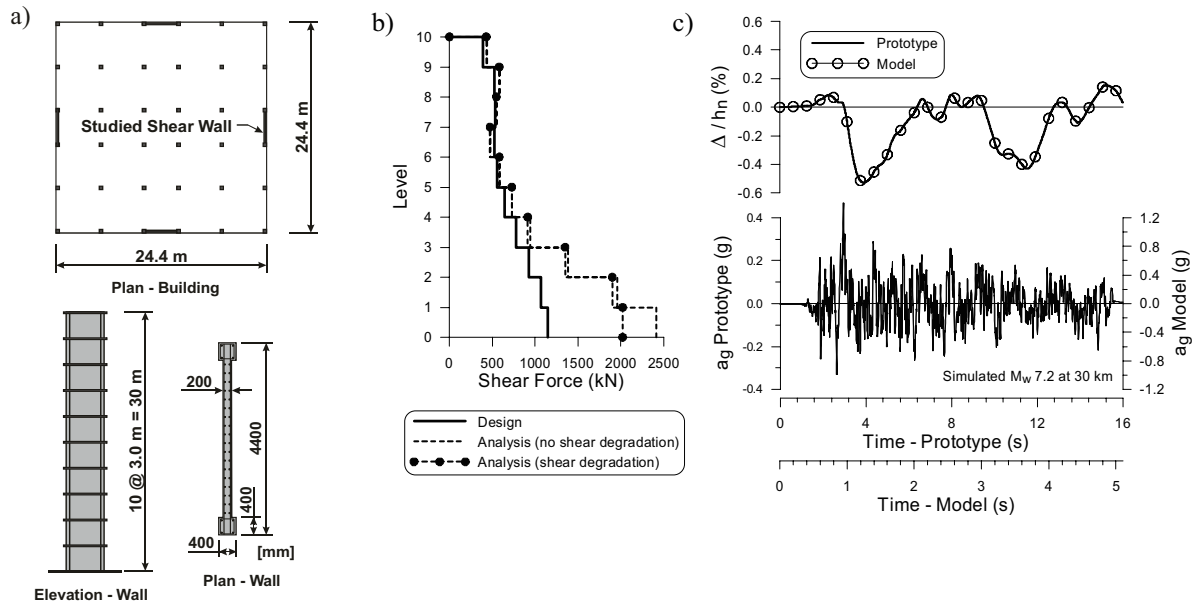


Fig. 6 Analysis of shear wall prototype and model structures: a) Prototype structure; b) Storey shear demand on the prototype wall under simulated M7.2 at 30 km; c) Roof lateral displacement response under simulated M7.2 at 30 km.

Seismic response of multi-storey light gauge steel frame/wood panel shear walls

The test specimen examined herein is a full-scale two-storey shear wall structure, as shown in Fig. 4c. The same wall construction is used at both floors, as would typically be done in practice. Each wall measures 1.22 m in length and 2.44 m in height. Two back-to-back C-shaped studs are used for the chord members together with regularly spaced intermediate single C studs. The sheathing is made of 9.5 mm thick OSB panels screwed at 152 mm o/c to the steel framing members. Only one concrete block is used to simulate the seismic weight at each level. Figure 7 shows the predicted response of the test specimen under a simulated M6.5 at 30 km ground motion event.

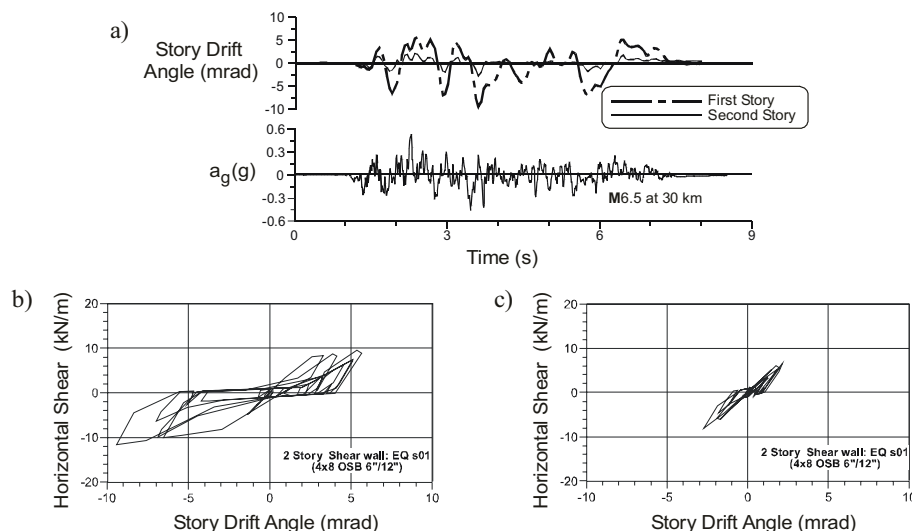


Fig. 7 Predicted response of a two-storey steel frame/wood panel shear wall specimen: a) Storey drift angle response at both floors; b) Hysteretic response at first floor; c) Hysteretic response at second floor.

Previous quasi-static tests have indicated that nonlinearity in the seismic response of this type of shear wall design mainly develops in the screwed connections between the sheathing and the metal studs. The analytical response was therefore obtained using the Wayne Stewart degrading stiffness hysteretic rule included in the Ruaumoko computer program. This model has been specifically developed to reproduce the global pinching and stiffness degradation behaviour of walls due to local fastener response observed in nailed timber shear walls. In this particular example, fully rigid hold-down response was assumed at the first floor. As anticipated, the storey deformation demand was concentrated at the lower floor.

SUMMARY AND CONCLUSIONS

An earthquake simulation test setup has been developed to study the seismic stability of multi-storey braced steel frames, higher mode effects on the flexural and storey shear force demand imposed on slender reinforced concrete shear walls, and the seismic response of multi-storey steel frame / wood panel shear walls used in housing and small buildings. The experimental setup can accommodate specimens up to 10 m or 10 storeys in height, permits a total seismic weight of up to 600 kN, and incorporates P-delta effects. Analytical prediction of the behaviour of sample structures was performed to validate the similitude laws used in the design of the test models and the possibility of performing the future test program with the proposed setup.

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