COLLAPSE ASSESSMENT OF RC BUILDING COLUMNS THROUGH MULTI-AXIS HYBRID SIMULATION

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Biography:

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ABSTRACT

One of the major challenges in collapse assessment of RC structures has been the lack of realistic data obtained from reliable experimental loading protocols that are capable of accurately quantifying the reserve capacity of RC structures beyond the design level to the state of complete collapse. Until now, quasi-static (QS) symmetrically cyclic or monotonic tests with constant axial load, have been commonly used, which are not adequate to accurately capture the actual response of a collapsing RC structure in real earthquake events. Hybrid
simulation (HS) can be considered as an attractive alternative to realistically simulate more complex boundary conditions and improve response prediction of a structure from elastic-range to collapse. This paper presents a comparative experimental study on two identical large-scale limited-ductility RC columns that are tested to collapse through QS and HS, respectively. The RC columns serves as the first-story corner-column of a half-scale symmetrical 5×5 bay 5-story RC ordinary moment frame building structure. A state-of-the-art facility, referred to as Multi-Axis Substructure Testing (MAST) system, is used that is capable of controlling all six-degrees-of-freedom (6-DOF) boundary conditions in mixed load and deformation modes. The load protocol in QS test includes constant axial load combined with bidirectional lateral deformation reversals, while in HS more realistic boundary effects including fluctuation in axial load and the ratcheting behavior (i.e. asymmetrical lateral deformation prior to collapse) are simulated. The hysteretic response behaviors obtained from the QS and HS tests are then used, respectively, for calibrating the analytical models employed in a comparative collapse risk assessment. The results show that the improved interface boundary effects lead to significant changes in hysteretic response and the calibration parameters and as a result estimating the probability of collapse. This highlights that the credibility of collapse assessment results relies to a great extent on the application of correct boundary interface on RC columns.

**Keywords:** Hybrid simulation, collapse assessment, ratcheting behavior, axial load variation, limited-ductility RC buildings

**INTRODUCTION**

Collapse prevention of structures and life safety of occupants are the primary goals of earthquake resistant design. Assessment of collapse risk, however, poses major challenges as it requires credible and accurate prediction of the structure’s response from linear-elastic range
through collapse. One of the primary problems is that currently available experimental data, obtained from conventional symmetric cyclic or monotonic load protocols, are insufficient to accurately capture the response of a collapsing RC structure in real earthquake events. There are two major reasons: 1) Recent research findings indicate that the variation in axial load combined with horizontal cyclic actions could drastically change the hysteretic characteristics of RC sections [1-3], and 2) Recent shake table collapse experiments of structures [4] have demonstrated that during earthquake shaking, a structure deforms asymmetrically with large monotonic pushes and a few small inelastic cycles prior to collapse (also defined as ratcheting behaviour [5, 6]). The aforementioned factors could highly influence the hysteresis properties; specifically, the plastic deformation capacity, ultimate drift and force degradation including in-cycle (post-peak negative stiffness) and between-cycle or cyclic (reduction of strength due to large number of cycles) degradations [7, 8].

In order to capture the realistic collapse behavior of RC columns and evaluate phenomena that are not represented adequately in simplified quasi-static load protocols, hybrid simulation [9, 10] can be used as an attractive alternative to study collapse [11-13]. In this method, the physical elements of the structure are embedded into the finite-element code of the numerical model of the full structure. During the simulation, while the response of the structure at the system level is numerically simulated online in the computer, the experimental elements act as part of the full structure that approaches collapse and therefore generate more realistic data as opposed to conventional cyclic tests [14]. However, it is also important to note that in hybrid simulation of structures, especially those with distributed damage, a large portion of a structure is modeled numerically and as a result the response of the specimen could be influenced by the simplifications in analytical modeling and computations. Yet, when compared to experimentally testing the entire structure on a shake table, which can have scaling issues and
could be very expensive and potentially dangerous, hybrid simulation is a better available alternative [15].

The primary objective of this paper is to conduct a comparative experimental study to demonstrate the application of hybrid simulation (HS), as an alternative to conventional quasi-static (QS) test, in collapse assessment of RC structures. For this purpose, two identical large-scale limited-ductility RC columns are tested using a state-of-the-art loading system, referred to as the Multi-Axis Substructure Testing (MAST) system. The facility is capable of controlling all six-degrees-of-freedom (6-DOF) boundary conditions in mixed load and deformation modes [16]. In the multi-axis QS test, the specimen is subjected to constant gravity load combined with bidirectional deformation reversal that followed a hexagonal orbital pattern. In the multi-axis HS test, the RC column serves as the first-story corner-column of a half-scale symmetrical 5×5 bay 5-story RC ordinary moment frame building that is subjected to bidirectional sequential ground motions with increasing intensities. Further, a collapse risk assessment study is conducted to investigate the influence of the improved boundary effects in the HS test, as opposed to the simplified load protocol in QS test, on the calibration parameters of the analytical models and as a result the outcomes of collapse risk assessment.
RESEARCH SIGNIFICANCE

It is becoming increasingly important to quantify the reserve capacity of RC structures beyond the design level to collapse. This experimental study aims to use hybrid simulation (HS), as an improved alternative to conventional quasi-static (QS) tests, for more in-depth understanding of the collapse behavior of RC columns. Specifically, hybrid testing allows for simulation of axial load variation and the ratcheting behavior prior to collapse; thus, imposing more realistic boundary effects on the RC columns. The study compares the hysteretic responses of two identical large-scale limited-ductility RC columns tested through QS and HS tests and shows that the applied boundary effects could considerably change the response behavior of RC columns. Also, through a simplified collapse risk assessment, it is demonstrated that these changes could significantly influence the calibration parameters of the analytical models and as a result estimates of collapse probability. This highlights that the credibility of collapse assessment results relies to a great extent on the application of correct boundary interface on RC columns.
DESCRIPTION OF EXPERIMENTS

Multi-Axis Substructure Testing (MAST) System

The experiments were conducted at Swinburne University of Technology using Australia’s first and only 6-DOF hybrid simulation facility, the Multi-Axis Substructure Testing (MAST) system. The state-of-the-art facility uses four ±1MN (220kips) vertical hydraulic actuators, two pairs of ±500kN (110kips) horizontal actuators in orthogonal directions and a 9.5tonne (21kip) steel crosshead that transfers the forces from the actuators to the specimen. The reaction system comprises an L-shaped strong wall, 5m (16.4ft) tall × 1m (3.3ft) thick, and a 1m (3.3ft) thick strong floor. The test area under the crosshead is approximately three cubic meters (106ft³). The control system is capable of imposing simultaneous 6-DOF states of deformation and load in switched and mixed mode control and suitable for large-scale quasi-static (QS) cyclic tests and local/geographically-distributed hybrid simulation (HS) experiments [16]. An overview of the MAST system and the actuator assembly is shown in Fig.1. The non-concurrent capacity of the MAST system in each DOF domain is presented in Table 1.

Fig.2(a) and (b), respectively, show the design details of the limited-ductility RC columns used in the experiments and the 6-DOF axes of crosshead movements. The specimen is attached to the strong floor from the base and to the crosshead from the top through the rigid concrete pedestals. The RC column is 2.5m (8.2ft) high and has a square 250mm (9.8in) × 250mm (9.8in) cross-section. The longitudinal column reinforcement (reinforcement ratio = 1.28%) consisted of four N16 bars (normal ductility, 16mm (0.63in) diameter) that were lapped over a length of 800mm just above the bottom joint to represent typical construction practices. The column contained transverse reinforcement throughout the entire length consisting of R6 (6mm (0.24in) diameter) closed stirrups at 175mm spacing with 30mm (1.2in) cover thickness. The
stirrups were anchored using 135° bent hooks with a development length of 75mm (2.95in). The material properties of the specimen, obtained from laboratory tests, are also presented in Table 2.

**Quasi-Static Cyclic Test**

The first experiment conducted on the RC column was a three-dimensional mixed-mode quasi-static cyclic test. The loading protocol consisted of simultaneously applying a constant 189.3kN (42.5kips) gravity load (equal to 8% of ultimate compressive load capacity) in force control while imposing bidirectional lateral deformation reversals in displacement control that follows the hexagonal orbital pattern suggested in FEMA 461 [17], and shown in Fig.3.

The sequence of loading in QS testing started with applying the gravity load on the specimen along Z-axis. The specimen was then pushed to the initial uniaxial drift ratio towards point ‘a’, followed by the orbital pattern ‘a-b-c-d-e-f-a’. The reversal from point ‘a’ accompanies an orthogonal drift at points ‘b’ and ‘c’ equal to one-half the maximum drift ratios at points ‘a’ and ‘d’. The entire loading cycle was then repeated at the same amplitude. Once the specimen reached point ‘a’ for the second time, the amplitude value for the next two cycles was increased and the next two biaxial load cycles were applied on the specimen. The process continued until the failure of the specimen. The remaining DOF axes (Roll, Pitch and Yaw) were controlled in zero-angle forming a double-curvature deformation of the column.

The results of the QS test, including hysteresis in X and Y axes, the axial force time history in Z-axis, biaxial deformations in terms of lateral drifts in X and Y axes and biaxial moment interactions in Rx and Ry axes are presented in Fig.4. The force relaxations (sharp drops in force during the unloading and reloading phases) observed in the hysteresis were due to pausing of the test in order to collect photogrammetry data at peak deformations in the X axis. The failure of the specimen occurred when it was subjected to the maximum of 7.0% and 3.5%
drift ratios in Y and X axes, respectively. These are large drifts for a limited-ductility column, but effective of the relatively low axial loads applied to the column. More information on the influence of axial load on the ultimate drift capacity of limited-ductility RC columns have been discussed in Wibowo et al. [18].

**Hybrid Simulation Test**

The second experiment conducted was a three-dimensional hybrid simulation that included a physical RC column element, which was identical to the one previously tested in the quasi-static cyclic experiment. For this purpose, a half-scale symmetrical 5-story (height of first story $h_1=2.5m$ (8.2ft), height of other stories $h_{typ}=2.0m$ (6.5ft)) 5×5 bay (column spacing $b=4.2m$ (13.8ft)) RC ordinary moment frame building is selected as the hybrid model. The physical specimens serve as the first-story corner-column of the building, considered as the critical element of the structure. The rest of the structural elements, inertial and damping forces, gravity and dynamic loads and second-order effects are modeled numerically in the computer. An advanced three-loop hybrid simulation architecture [19] that uses OpenSees [20], OpenFresco [14] and the xPC-Target real-time digital signal processor [21] was implemented. Fig.5 illustrates the components of hybrid simulation including numerical and experimental substructures.

The structure’s beams and columns were modeled using beam-with-hinges elements, of which the nonlinear behavior is assumed to occur within a finite-length at both ends based on the distributed-plasticity concept [22] (Fig.6(a)). The plasticity model follows a peak-ordinated hysteresis response based on the Modified Ibarra-Medina-Krawinkler (IMK) deterioration model of flexural behavior [23, 24]. This model was chosen because it is capable of capturing the important modes of deterioration that participate in sidesway collapse of RC framed structures. The model requires the specification of a range of parameters to control the tri-linear
monotonic backbone curve and different modes of cyclic deteriorations. As shown in Fig.6(b), these parameters include $M_y$, $M_c/M_y$, $\psi_p$, $\psi_{pc}$ and $\lambda$ that respectively represent yielding moment, a measure of maximum moment capacity, plastic curvature capacity, post-capping curvature capacity and cyclic deteriorations. The model captures four modes of cyclic deteriorations including strength deterioration of hardening region ($\lambda_S$), strength deterioration of post-peak softening region ($\lambda_C$), accelerated reloading stiffness deterioration ($\lambda_A$), and unloading stiffness deterioration ($\lambda_K$). Based on the studies by Haselton et al. [25], the strength deterioration of hardening region and the post-capping strength deterioration were assumed equal in the case study, while accelerated reloading and unloading stiffness deteriorations were ignored. This reduced the calibration of cyclic deteriorations to one parameter. Table 3 presents the IMK parameters for beam and column elements.

After developing the numerical model, the elastic fundamental period of vibration and the corresponding first mode shape were obtained through eigenvalue analysis. A nonlinear static pushover analysis was then performed with the lateral force distribution proportional to the fundamental mode of vibration and with the consideration of second-order $P$-$\Delta$ effects. Fig.7 presents the results of the pushover analysis that show most of the energy dissipation occurs in the lower two stories.

For the HS test, the two horizontal components of the 1979 Imperial Valley earthquake ground motions recorded at El Centro station with peak ground acceleration of 0.15g were used. Fig.8 shows the acceleration, displacement and acceleration-displacement response spectra for the two ground motion components.

Prior to conducting the actual HS test with the physical subassembly in the laboratory, a series of FE-coupled [26] sensitivity numerical simulations was conducted to evaluate the
ground motion intensity levels and integration scheme parameters for the actual experiments. Accordingly, four levels of intensities were considered to capture the full range of structural response from linear-elastic range to collapse. The selected scale factors are 0.6, 4.0, 8.0 and 9.0 that pushed the structure to nearly 0.25% (elastic), 2%, 4% and 6% inter-story drift ratios, respectively. Further, Generalized Alpha-OS [14] was selected as the integration scheme and the integration time-step was optimized to preserve the accuracy and stability of the simulation, while allowing the completion of the entire test during the regular operation time of the laboratory. 5% Rayleigh damping was specified to the first and third modes of vibration, corresponding to the primary translational modes in X- and Y-directions. Additional damping was also assigned to free vibration time intervals between the forced vibrations in order to quickly bring the structure to rest.

The hybrid simulation started with applying the gravity load on the specimen, using a ramp function, followed by sequential ground motions. The entire sequence of loading was performed and automated using OpenSees. Considering the 117msec delay in the hydraulic system, 500msec was specified as the simulation time step in xPC-Target predictor-corrector to provide sufficient time for integration computation, communication, actuator motions and data acquisition. This scaled the 60sec of sequential ground motions to 6 hours in laboratory time. Similar to the QS test, the rotational axes (Roll, Pitch and Yaw) were controlled in zero-angle forming a double-curvature deformation of the column.

Fig.9 compares the responses of RC column in QS and the HS tests including hysteresis in X and Y axes, the axial force time history in Z-axis, biaxial deformations in terms of lateral drifts in X and Y axes and biaxial moment interactions in Rx and Ry axes. The specimen was pushed to a maximum of 6.4% and 2.7% drift ratios in Y and X axes, respectively. The maximum time-varying axial load applied on the specimen was 553kN (124.3kips) in
compression and 161kN (36.2kips) in tension. By comparing the hysteresis plots from the HS test, it can be seen that the column damaged as the structure progressively moved in one direction, while in the QS test the pattern of damage was symmetrical due to load reversals in cyclic deformations. Fig.10 shows the flexural failure of columns for both tests by comparing the plastic hinges developed at the top and the base of the columns from different views.
COLLAPSE RISK ASSESSMENT

Numerical Model and Calibration

In order to investigate the influence of the selected experimental method on assessing the collapse risk of structure, a simplified comparative collapse fragility analysis for a substructure of the RC building was conducted using the results of QS and HS tests, respectively. As illustrated in Fig.11, the numerical model selected for incremental dynamic analysis includes only the first-story corner-column and the overhead mass portion of the upper 5 floors, which is equivalent to a single-degree-of-freedom (SDOF) system with natural period of 0.6sec. This allows studying the response of the critical element (i.e. the first-story corner-column) purely based on experimental results and removes the influence of other elements response.

The experimental results were used to calibrate the SDOF numerical model. As previously shown in Fig.6, the moment-curvature behavior of the plastic zones follows the IMK hysteresis model. Although this model can generally simulate most of the important behaviors including strength and stiffness degradation, the effects such as the interaction between axial, flexure, and shear failure cannot be captured. Accordingly, a unidirectional numerical model of the column was selected and the hysteresis parameters of IMK model have been calibrated to the response of the specimen in the main axis (i.e. the Y axis of the MAST system), along which it experienced maximum deformation. Consequently, the influence of axial loads and out-of-plane moments in the experiments were implicitly taken into account by using the calibrated numerical models. Note that, use of fiber-based plasticity models could be an alternative, however, only the most basic aspects such as material constitutive relationships are modeled while the degradation parameters that have significant impact on collapse behaviors are not included [27].
A close view of the hysteretic responses is presented in Fig. 12. It can clearly be observed that the flexural strength, the capping point and post-capping negative tangent stiffness (in-cycle strength degradation) are significantly different. Accordingly, following the procedures given in Haselton et al. [25], the numerical SDOF model was calibrated to the QS and HS test results, with particular focus on precisely mimicking the plastic and post-capping deformation capacities as well as the cyclic deteriorations that are known to have important influence on collapse prediction (see Fig. 13). However, it is noted that some portions of unloading and reloading phases in the experiments, especially for the QS test, resulted in higher strength and larger hysteretic loops in the experiment compared to the more pinched behavior in calibrated models.

Table 4 compares the IMK model parameters for QS- and HS-based numerical models. The higher flexural strength and significant reduction of the drift capacity observed in the HS test could be due to the higher levels of axial load in this test as previously addressed in Refs. [28-30]. Fig. 14 specifically shows that the rapid drop in shear strength occurred immediately after the maximum compressive axial load in the HS test, which is 2.9 times larger than the uniform axial load applied in the QS test (equivalent to 23% ultimate axial load capacity of the column). This clearly shows the significant impacts of the axial load level and its variation on the performance of RC structure and the ultimate drift capacity. Studies conducted by Wibowo et al. [18] reported similar findings based on an experimental program performed to develop a generic back-bone pushover curve for lightly-reinforced concrete columns. In terms of the calibration parameters this leads to significant reduction of $\psi_{pc}$ and higher level of in-cycle deterioration in the HS-based model.

Another observation is that the specimen showed larger cyclic (between-cycle) deteriorations in QS test due to the application of many large cycles and load reversals to the
specimen before failure. Note that the values assigned to the cyclic deterioration in the IMK model (i.e. $\lambda_S, \lambda_C$) are inversely related to the level of deteriorations.

**Fragility Analysis**

Incremental dynamic analyses (IDA) were performed using the calibrated numerical model in order to capture a range of probable dynamic response behaviors due to record-to-record variability in ground motion characteristics. For this purpose, three earthquake scenarios including M6.0R28, M6.5R40 and M7.0R90 (M and R stand for magnitude and source-site distance, respectively) have been considered. A suite of 20 recorded ground motions was selected from the PEER database [31] that are listed in Table 5 along with the values of peak ground acceleration ($PGA$) and spectral acceleration at the fundamental natural period of the numerical model ($Sa(T_1)$ and $T_1$=0.6sec for the SDOF model). The response spectra of the input ground motions are also shown in Fig.15.

Each unidirectional ground motion was individually applied to QS- and HS-based calibrated numerical models for the nonlinear simulation. The ground motions were increasingly scaled according to the value of $Sa(0.6)$, until reaching the collapse state of the building. The simulation was based on 5% mass-proportional damping and restricted to sidesway-only collapse with drift limit of 7% based on experimental results. The outcome of this assessment is a structural collapse fragility function, which is a lognormal distribution relating the structure’s probability of collapse to the ground-motion intensity, in terms of $Sa(0.6, 5\%)$. Fig.16 presents the results of nonlinear incremental time-history analyses for QS- and HS-based numerical models.

The mean (the $Sa$ level with 50% probability of collapse) and standard deviation (the dispersion of $Sa$) for each case can be derived from the following equations [5]:

\[ \text{Mean} = \mu \]  
\[ \text{Standard Deviation} = \sigma \]
\[
\ln(\theta) = \frac{1}{n} \sum_{i=1}^{n} \ln(S_a(i))
\]  
(1)

\[
\beta = \frac{1}{n-1} \sum_{i=1}^{n} \left( \ln \left( \frac{S_a(i)}{\theta} \right) \right)^2
\]  
(2)

where \(n\) is the number of ground motions considered and \(S_a(i)\) is the \(S_a\) value associated with onset of collapse for the \(i\)-th ground motion. Also, \(\ln(\theta)\) and \(\beta\) are respectively the mean and the standard deviation of the normal distribution that represents the \(\ln(S_a)\) values. Note that the mean of \(\ln(S_a)\) is corresponding to the median of \(S_a\) in the case that \(S_a\) is lognormally distributed.

The computed mean and standard deviation values for QS and HS-based numerical models show that while dispersion of \(S_a\) is similar in both cases (0.45 and 0.4 for QS and HS, respectively), the \(S_a\) level with 50% probability of collapse is significantly overestimated in QS-based model (\(S_a=1.5g\)) compared to HS-based model (\(S_a=1.2g\)). A lognormal cumulative distribution function was then used to define the fragility functions [32]:

\[
P(\text{Collapse}|S_a) = \Phi \left( \frac{\ln \left( \frac{S_a}{\theta} \right)}{\beta} \right)
\]  
(3)

where \(P(\text{Collapse}|S_a)\) is the probability that a ground motion with intensity of \(S_a\) will cause the structure to collapse, \(\Phi\) is the standard normal cumulative distribution function (CDF), \(\theta\) is the median of the fragility function and \(\beta\) is the standard deviation of \(\ln(\theta)\).

It is observed in Fig.17 that the differences in the collapse probability between fragility curves become larger as the intensity level increases. Specifically, at the intensity level \((S_a)\) of 1.5g, where the QS-based model predicts 50% probability of collapse, the HS-based model predicts 75.2% probability of collapse, which is 1.5 times larger.

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The 2012 edition of the International Building Code (IBC) [33] and the 2010 edition of the structural design standard ASCE/SEI 7 [34] have specified the performance requirement of having uniform collapse risk for structures that are designed based on the risk-targeted maximum considered earthquake (MCE_R) ground motions, with a return period of ~2500 years. Under the MCE_R ground motions, it is expected to have less than 10% probability of collapse for Risk Category I and II structures, 6% for Risk Category III structures and 3% for Risk Category IV structures. The calculations of such probability for the case study structure results in 1.3% and 3.9%, respectively, for the QS- and HS-based models. It indicates that a Risk Category IV structure is deemed safe if the numerical model is calibrated against QS results, but it may be considered unsafe based on HS results.

It is important to mention, however, while in this case study the QS test results tend to underestimate the collapse probability, it should not be generalized to all other cases. In addition, the results of HS are applicable only to ground motions similar to the one used in the experiment and more tests may be required to achieve better accuracy. Nevertheless, this method still allows for applying more realistic boundary force and deformations and better capturing collapse behavior of RC columns.

**CONCLUSIONS**

The key objective of this paper was to demonstrate the application of hybrid simulation (HS) as an alternative for conventional quasi-static (QS) test in collapse assessment of RC columns. Two experiments were conducted on identical large-scale limited-ductility RC columns by the respective testing methods using the state-of-the-art Multi-Axis Substructure Testing (MAST) system, which is capable of controlling all six-degrees-of-freedom (6-DOF) boundary conditions in mixed load and deformation modes. The RC columns served as the first-story
corner-column of a half-scale symmetrical 5×5 bay 5-story RC ordinary moment frame building structure. The load protocol in QS test included constant axial load combined with bidirectional lateral deformation reversals, while in HS more realistic boundary effects including time-varying axial load and the ratcheting of structure’s lateral deformation were simulated. The hysteretic response behaviors from the QS and HS tests were then used, respectively, for calibrating the numerical models employed in a comparative collapse risk assessment. The results show that the fluctuation in axial load could lead to higher compressive axial demands and potentially tensile demands, which results in more pronounced in-cycle force degradation and significant reduction in drift capacity. In addition, the simulation of the ratcheting behavior allows for better estimation of between-cycle force degradation, as opposed to symmetrical cyclic load protocols that overestimates the degradation rate. The fragility analysis also showed that the improved interface boundary effects lead to significant changes in calibration parameters and as a result estimating the probability of collapse. This highlights that the credibility of collapse assessment results relies to a great extent on the application of realistic boundary interface on RC columns.

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Table 1 - MAST DOFs Capacity (non-concurrent)

<table>
<thead>
<tr>
<th>DOF</th>
<th>Load</th>
<th>Deformation</th>
<th>Specimen Dimension</th>
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<tbody>
<tr>
<td>X (Lateral)</td>
<td>1MN (220kips)</td>
<td>± 250mm (+10in)</td>
<td>3.00m (10.0ft)</td>
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<tr>
<td>Y (Longitudinal)</td>
<td>1MN (220kips)</td>
<td>± 250mm (+10in)</td>
<td>3.00m (10.0ft)</td>
</tr>
<tr>
<td>Z (Axial/Vertical)</td>
<td>4MN (880kips)</td>
<td>± 250mm (+10in)</td>
<td>3.25m (10.6ft)</td>
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<td>Rx (Bending/Roll)</td>
<td>4.5MN.m (3320kips.ft)</td>
<td>± 7degrees</td>
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<tr>
<td>Ry (Bending/Pitch)</td>
<td>4.5MN.m (3320kips.ft)</td>
<td>± 7degrees</td>
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<tr>
<td>Rz (Torsion/Yaw)</td>
<td>3.5MN.m (2580kips.ft)</td>
<td>± 7degrees</td>
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Table 2 - Material properties of RC columns

<table>
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<tr>
<td>$\varepsilon_c$</td>
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<td>$f'_c$</td>
<td>35.1MPa (5.1ksi)</td>
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<tr>
<td>$\varepsilon_{cu}$</td>
<td>0.0063</td>
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<td>$f'_{cu}$</td>
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Table 3 - IMK model parameters for RC framed building structure

<table>
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<tr>
<th>Story No.</th>
<th>Element location</th>
<th>$M_c/M_y$</th>
<th>$M_y$ (kN.m)*</th>
<th>$\psi_p$ (1/m)</th>
<th>$\psi_p$ (1/m)</th>
<th>$\lambda_S, \lambda_C$</th>
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* 1kN.m = 0.74kips.ft

Table 4 - Comparison of IMK model parameters calibrated to QS and HS tests

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<th>Experiment Method</th>
<th>$M_c/M_y$</th>
<th>$M_y^+$ (kN.m)*</th>
<th>$M_y^-$ (kN.m)*</th>
<th>$\psi_p^+$</th>
<th>$\psi_p^-$</th>
<th>$\psi_p$</th>
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<td>0.9</td>
<td>1.0</td>
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* 1kN.m = 0.74kips.ft
<table>
<thead>
<tr>
<th>No.</th>
<th>Record Name</th>
<th>Scenario</th>
<th>$PGA(g)$</th>
<th>$S_0(T_1)(g)$</th>
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**Fig. 1 - MAST system and actuators assembly**

a) Actuators assembly, plan view  
b) Actuators assembly, side view  
c) Overview of the MAST system

**Fig. 2 - Experimental setup for QS and HS experiments**

a) Design details of RC column (1mm = 0.04in)  
b) 6-DOF movements of RC column
a) Hexagonal orbital pattern for bidirectional lateral deformation reversals

Fig. 3 - QS loading protocol
a) Cyclic response in X and Y axes (1kN = 0.220 kips)

b) Axial load time history (1kN = 0.220 kips)

c) Biaxial lateral drifts

d) Biaxial moment interactions (1kN.m = 0.74 kips.ft)

Fig. 4 - Response of the RC column in QS cyclic test
Fig. 5 - Hybrid simulation substructures

Fig. 6 - Nonlinear analytical model for beam and column elements
a) Story shear versus story drift ratio

**Fig. 7 - Pushover analysis of RC framed building structure**

b) Story drift ratio

a) Acceleration response spectra  
b) Displacement response spectra  
c) Acceleration-displacement response spectra

**Fig. 8 - Response spectra for the two horizontal components of the 1979 Imperial Valley earthquake ground motions (recorded at El Centro station) used in HS test**
a) Comparison of the responses of RC column in X and Y axes (1kN = 0.220kips)

b) Axial load time history (1kN = 0.220kips)

c) Biaxial lateral drifts

d) Biaxial moment interactions (1kN.m = 0.74kips.ft)

Fig. 9 - Comparison of responses of the RC columns in QS and HS tests
Fig. 10 - Comparison of plastic hinges in QS and HS tests

Fig. 11 - Numerical substructure selected for collapse risk assessment
a) Calibration of SDOF model to response of RC column from QS test

b) Calibration of SDOF model to response of RC column from HS test

Fig. 12 - Calibration of SDOF numerical model to QS test results and comparison with HS test results (1kN = 0.220kips)

Fig. 13 – Close view of the hysteretic responses with emphasis on in-cycle force degradation (1kN = 0.220kips)
Fig. 14 - Maximum in-cycle negative tangent stiffness corresponding to maximum compressive axial load in HS test (1kN = 0.220kips)

Fig. 15 - Response spectra for ground motions used in IDA

a) Acceleration response spectra  b) Displacement response spectra  c) Acceleration-displacement response spectra

(1m = 3.3ft)
a) IDA results based on numerical model calibrated to QS test

b) IDA results based on numerical model calibrated to HS test

Fig. 16 - Comparison of IDA results for the RC column based on results from QS and HS tests

Fig. 17 - Comparison of fragility curves for the RC column based on results from QS and HS tests