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Overturning of precast RC columns in conditions of moderate ground shaking

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Abstract. A simple method of assessing the risk of overturning of precast reinforced concrete columns is presented in this paper. The displacement-based methodology introduced herein is distinguished from conventional force-based codified methods of aseismic design of structures. As evidenced by results from field tests precast reinforced concrete columns can be displaced to a generous limit without sustaining damage and then fully recover from most of the displacement afterwards. Realistic predictions of the displacement demand of such (rocking) system in conjunction with the displacement capacity estimates enable fragility curves for overturning to be constructed. The interesting observation from the developed fragility curves is that the probability of failure of the precast soft-storey column decreases with increasing size of the column importantly illustrating the "size effect" phenomenon.

Keywords: rocking; risk of overturning; shake table; fragility curves; risk of failure

1. Introduction

This paper is concerned with the ultimate performance behaviour of reinforced concrete columns undergoing large displacement and their risk of overturning when subject to seismic actions. Shake table experiments were first conducted on non-deformable free-standing rectangular object specimens with symmetrical distribution of mass. Experimental results from the shaking table (Kafle *et al*. 2011a) have been augmented by computer simulations using algorithms developed in Lam *et al*. (2003) that had been verified. Observations from experiments, and from simulations, enable fragility curves to be constructed and provide insights into factors controlling the risk of overturning. Expressions which have been derived in Kafle *et al*. (2011a) to correlate the maximum estimated tilt of the overturning (rigid free-standing) object with the elastic response

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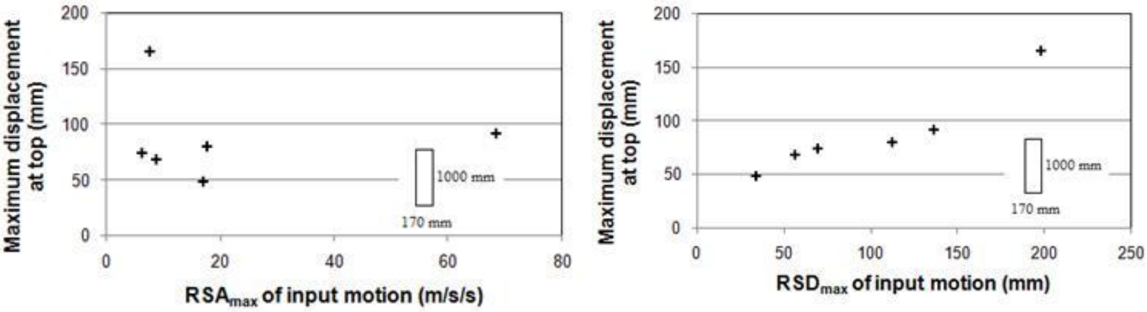
spectral parameter values are summarised in Section 2. In this paper the method developed for assessing the risk of overturning of rigid free-standing objects has been extended to precast reinforced concrete (RC) columns taking into account the deformation of the concrete. Precast RC columns possessing the potential capacity to rock have been analysed for the risk of overturning based on both recorded, and simulated, force-displacement (hysteretic) relationships. Displacement based methodology is illustrated in this paper using a case study of a four-storey and a twenty-storey building both of which were supported by precast RC columns at their ground floor level. The outcomes of the evaluation of the ultimate performance of the two buildings employing both conventional force-based and displacement-based methodologies are compared. There are potentials of adopting the methodology for the design of innovative unbonded post-tensioned precast concrete frames and retaining wall structures which can also be designed to displace in a similar fashion.

2. Overturning risk

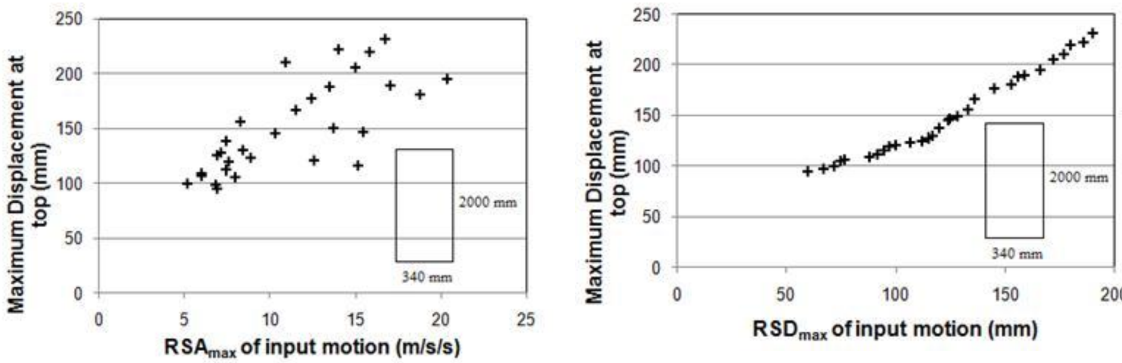
As a non-deformable free-standing object responds to base excitations and undergoes rocking motion the effective natural period of vibration is not constant but varies with the amplitude of displacement. Consequently, the (usual) response spectral acceleration, or displacement, value which corresponds to a pre-defined natural period of vibration cannot be identified. Thus, broad based ground motion parameters such as peak ground acceleration (PGA) or peak ground velocity (PGV) are normally used for predicting the risk of overturning. Alternatively, parameters that are derived from the elastic response spectrum can be used. In recent shake table experimentations of free-standing objects undertaken by the authors (Kafle *et al*. 2011a) the amount of maximum tilt of the object as indicated by the maximum displacement at the top was correlated against values of the RSA_{max} which is defined herein as the maximum response spectral acceleration; RSV_{max} which is the highest point on the velocity response spectrum; and RSD_{max} which is the highest point on the displacement response spectrum up to the limit of 5 seconds period (Lam *et al*. 2000, Kafle *et al*. 2011a). A method for predicting the probability of overturning without requiring repetitive numerical simulations has been proposed by Dimitrakopoulos and DeJong (2012a) who has provided important insights into an alternative approach for modelling this complex behaviour. The substantial mitigating effects of damping on overturning hazards have also been modeled and expressed in the form of *overturning envelopes* (Dimitrakopoulos and DeJong 2012b). It is noted that the work of Dimitrakopoulos and DeJong (2012a and 2012b) is based on the original work which is related to the rocking response and stability behaviour of columns when subject to cycloidal pulses (Makris and Roussos 2000, Zhang and Makris 2001).

Response spectral acceleration (RSA), or its maximum value (RSA_{max}), which are used for characterising the response potential of short period systems has been found to correlate poorly with the risk of overturning of free-standing objects despite their “short period” behaviour in the initial “at-rest” conditions. In contrast, the RSD_{max} parameter (which is related directly to the peak displacement of the floor) has been found to correlate very well with the risk of overturning. These trends are well illustrated in Figs. 1(a)-(b) in which measured results from shake table experiments are presented, and in Figs. 2(a)-(b) in which results simulated from a validated algorithm developed initially in Lam *et al.* (2003) are presented (Kafle *et al.* 2011a).This important trend is consistent with the findings reported in an independent study by Ali *et al*. (2013).The risk of overturning of a



(a) Correlation with RSA_{max} (b) Correlation with RSD_{max}

Fig. 1 Results from shake table tests (Kafle *et al*. 2011a)

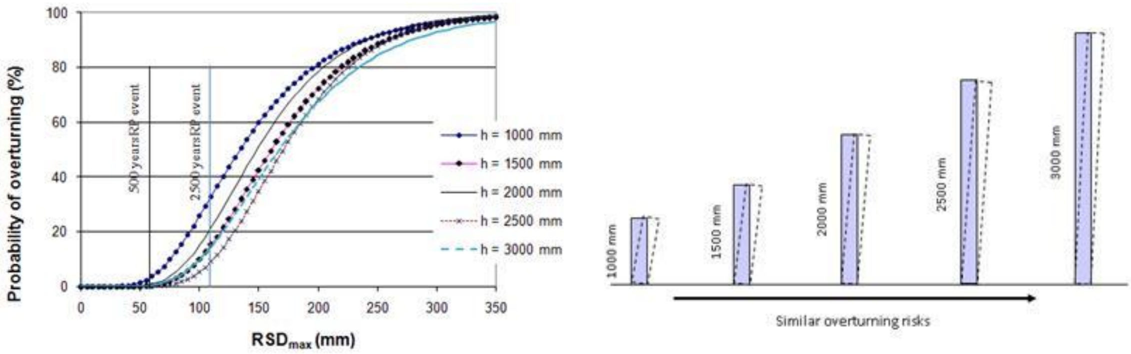


(a) Correlation with RSA_{max} (b) Correlation with RSD_{max}

Fig. 2 Results from validated simulations (Kafle *et al*. 2011a)

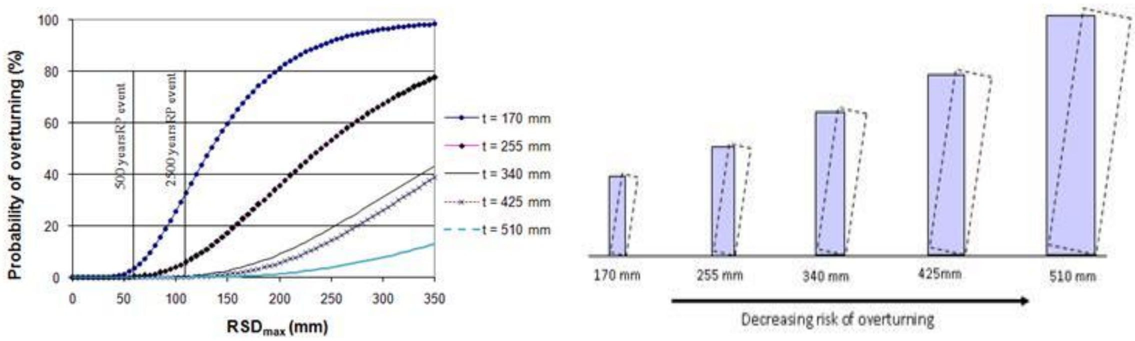
structure was shown not to be correlated with the acceleration component of the applied earthquake ground shaking. These trends have also supported the concept of the rocking spectra which has been introduced by Makris and Konstantinidis (2003), clearly indicating the fundamental differences between the regular pendulum (i.e., regular oscillator) and the inverted pendulum (i.e. rocking object).

Clearly, the risk of overturning of the test objects on the shake table was controlled by the displacement amplitude of the table (and not the value of the peak acceleration). Thus, the RSD_{max} parameter can be used to estimate the amount of maximum displacement that can be imposed on the object in an earthquake. Objects of varying height and aspect ratio were tested repetitively on the shake table, and by simulations, to reveal the underlying trends. The important and somewhat counter intuitive finding was that the risk of overturning did not increase monotonically with increasing height, nor aspect ratio, of the object when their thickness in the direction of ground shaking was kept constant. The risk of overturning was clearly shown to decrease with increasing object dimension at the base when the aspect ratio was kept constant. These trends are well illustrated by the fragility curves of Figs. 3-4 (Kafle *et al*. 2011a). To construct fragility curves for



(a) Fragility curves (Kafle *et al*. 2011a) (b) Schematic illustration

Fig. 3 Trend of overturning risk (height)



(a) Fragility curves (Kafle *et al*. 2011a) (b) Schematic illustration

Fig. 4 Trend of overturning risk (size)

overturning, a large number of accelerograms were used to represent a range of earthquake scenarios featuring a gradual change in the intensity of ground shaking. The accelerograms ensemble used for input into the analysis consisted of thirty scenarios of magnitude 7 events with distances varying between 15 km and 175 km (details of these accelerations were presented in Kafle 2011). Out of those earthquake scenarios considered, those with shorter epicentral distances and hence higher intensity of ground shaking can be described as being representative of conditions in regions of high seismicity. These earthquake scenarios would generally result in high maximum displacement demand (RSD_{max}) causing overturning, and hence limiting the application of the proposed methodology to regions of low and moderate seismicity. Only a small proportion of such earthquake scenarios have been included in the ensemble.

An expression (Eq. (1)) showing the threshold peak displacement demand (RSD_{max}) value of the ground shaking to impose a 5% chance of overturning was derived following an extensive parametric study carried out by the authors (Kafle *et al*. 2011a). It is noted that Eq. (1) is based on experimentations on objects with aspect ratio equal to 5.9 and for the considered earthquake scenarios of M7 events on class D sites. The expression is presented herein to emphasize the significance of the size effects, and is not to be treated as being generally representative of all seismic conditions. The strong dependence of the capacity of the object to resist overturning on its dimension at the base in the direction of ground shaking is evident.

The set of linear relationships of Eqs. (2a) – (2c) as proposed in an earlier study (Al Abadi *et al*. 2006) represents an attempt to develop some simple *rule-of-the-thumb* method for addressing a very complex phenomenon. The computed risks of overturning do not always correlate well with the simplified relationships as much depends on the aspect ratio of the object and the nature of the excitations that have been applied. It is shown that objects classified as "low risk" as per equation (2c) have risks of overturning much lower than 5% in certain earthquake scenarios. Thus, the conservatism of the simplified relationships proposed in Al Abadi *et al*. (2006) has been well demonstrated.

$$RSD_{\max} = 35.8 \times \exp^{0.004t} \qquad for\ 5\%\ probability\ of\ overturning \tag{1}$$

where t is the thickness of object in direction of ground shaking.

$$RSD_{\max} \geq \frac{2}{3}t \qquad [high\ risk] \tag{2a}$$

$$\frac{2}{3}t > RSD_{\max} > \frac{1}{3}t \qquad [moderate\ risk] \tag{2b}$$

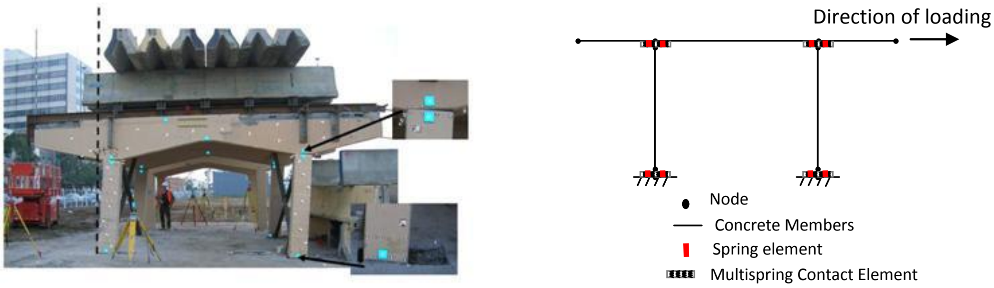
$$RSD_{\max} \leq \frac{1}{3}t \qquad [low\ risk] \tag{2c}$$

The findings as described for rigid free-standing objects have profound implications for the design and assessment of structures for countering overturning but would require further investigations to ensure that this modelling technique could be adapted for the assessment of gravity load carrying structural elements such as precast reinforced concrete columns (that are nominally connected to the adjoining elements to facilitate rocking).

3. Risk of overturning of precast reinforced concrete column

The analysis of rocking behaviour as applied to free-standing rigid objects on the shaking table has been extended to precast RC columns which are in support of the soft-storey of the building on the ground floor. This type of building is known to be highly vulnerable to collapse when subject to strong ground shaking. However, it is important to note that the displacement capacity of the building with a soft-storey can be enhanced by the rocking of columns provided that undesirable (premature) failure mechanisms have been prevented from occurring. Field tests conducted on a four-storey soft-storey building supported by precast RC columns in Melbourne, Australia revealed that precast RC columns could be designed to maintain their gravity load carrying capacity up to a drift limit of about 8% under quasi-static conditions.

Evidences from field testing (Fig. 5(a)) can be used to substantiate the claim that for certain precast RC columns their force-displacement relationship resembles that of a free-standing object experiencing rocking behaviour although there are minor differences with details. For the precast



(a) Test set up (b) Ruaumoko 2D model

Fig. 5 Test set up and Ruaumoko 2D model of precast soft-storey structure

RC columns that have been tested rocking was observed to occur without causing any noticeable damage to their ends as shown in Fig. 5(a). It is cautioned that such behaviour cannot always be assumed for all RC elements. Results presented in this paper only provide the evidences that rocking of the columns can be achieved with certain designs. The important requirement is that the nominal precast connections at both ends of the column allow rocking to take place and hence facilitates a generous displacement capacity without compromising structural integrity. Full details of the test set-up and field recorded results have been reported in Wibowo *et al*. (2010).

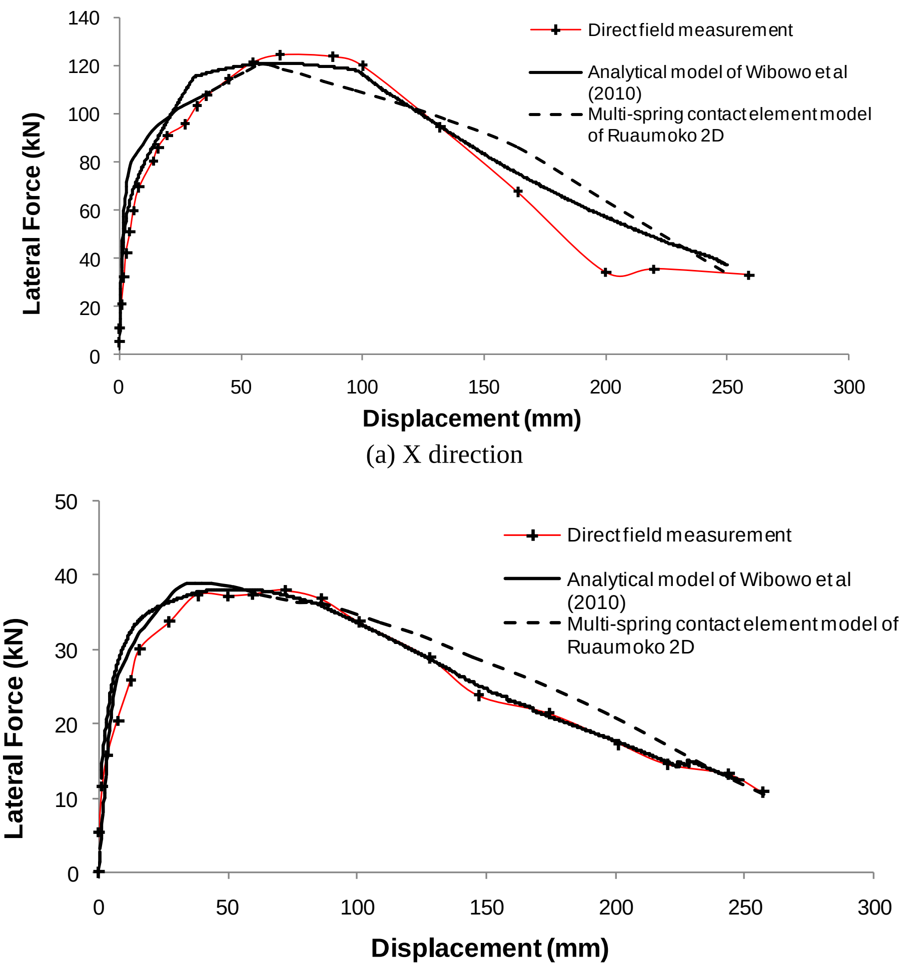
The concept of generalising both free-standing objects and top loaded precast RC elements by equivalent single-degree-of-freedom (SDOF) systems was first introduced in Doherty *et al*. (2002) which was concerned with the out-of-plane behaviour of unreinforced masonry walls. Because of the differences in geometry between the two types of elements there is a 50% increase in the displacement capacity from a free-standing object to a top-loaded RC element. With a free-standing object the displacement capacity is exhausted when the centre of gravity of the object is displaced to a position which is vertically above the pivotal position (i.e., centre of gravity is displaced by the amount $\boldsymbol{t}/2$ where $\boldsymbol{t}$ is the thickness of the object). As this happens the centre of inertia of the element (at 2/3 up the height) is displaced by two-thirds of its thickness: $2/3\boldsymbol{t}$. With a top loaded RC element the displacement capacity is exhausted when the point of load application from the superstructure is displaced to a position which is vertically above the pivotal position. This involves a displacement amount which is simply equal to thickness, $\boldsymbol{t}$. The difference between "2/3 $\boldsymbol{t}$" (object) and "$\boldsymbol{t}$" (column) explains the 50% increase in displacement capacity for a given thickness. It is noted that the reasoning presented above would only be valid for lightly loaded columns in which case the superstructure is assumed to transfer to the column through a point. In reality, the finite width of the compression block will need to be taken into account and this is particularly important for modelling the displacement capacity of more heavily loaded columns.

3.1 Numerical modelling

Numerical modelling of the precast RC columns in support of the soft-storey of the building was undertaken in two stages. In the first stage of the model development a frame model was developed in program Ruaumoko (Carr 2008) to simulate the response behaviour of the laterally loaded frame. The 2D model was designed to provide force-displacement behaviour which is consistent with measurements from full scale tests conducted in the field in quasi-static conditions. The numerical frame model featured the use of multi-spring contact elements that were placed at the interface between the beam and column, and between the column and the foundation in order to emulate high compressive stresses that can be developed at the edges of the column when undergoing large displacement. The properties of the spring elements were chosen in consideration of the stress-strain behaviour of the concrete material (Kafle 2011).This type of elements involves the use of up to 10 contact springs which are compression-only springs. High bearing (compressive) stresses developed in the concrete associated with a significant shift in the position of the neutral axis in the vicinity of the beam-column interface can be modelled by this type of elements (Spieth *et al*. 2004, Palermo *et al*. 2005). Essentially, contact elements that have been placed at both ends of the column function like "hinges" in order that large rotation of the column is possible without the formation of plastic hinges. The beam and columns forming the portal frame were modelled as linear elastic Giberson beam elements. High tensile unbonded steel bars connecting the beams and columns were modelled as spring elements instead. The force-displacement behaviour of the spring element has been calibrated to match with measurements obtained from laboratory testings of high tensile unbonded steel bars (Wibowo *et al*. 2008, Kafle 2011). A typical set up of the Ruaumoko 2D model for computer analyses is shown in Fig. 5. Pushover analyses were conducted using the program to obtain the force-displacement relationship in the X direction as shown in Fig. 6(a) and in the Y direction as shown in Fig. 6(b). The effects of overburden from the superstructure have been included in the analysis in the form of axial forces applied to the top of the columns in the soft storey. The axial load ratio was kept constant at 4% which is consistent with conditions employed in the field tests. The displacement capacity of the column has been found to be enhanced by an increase in the overburden of the superstructure as demonstrated in Makris and Vassiliou (2013) in which the dynamic rocking behaviour of free standing columns capped by a rigid beam was studied. The displacement capacity was defined at the point where the maximum strength was reduced by 70% consistent with field test results. The simulated load-displacement relationships were generally consistent with field measured values and with predictions by the analytical model of Wibowo *et al*. (2010). Essentially, frame models developed by program Ruaumoko involving the use of multi-spring contact elements have been validated to provide realistic simulations of displacement demand behaviour in order that field test results can be augmented with simulated results for frame dimensions that have not been covered so far in any physical experiments.

In the second stage of the model development a single-degree-of-freedom (SDOF) lumped mass system was incorporated with pre-defined hysteresis behaviour as shown in Fig. 7. Observations of the tested specimens revealed only minor damage to the concrete in comparison with the much more significant damage to lightly reinforced cast-in-situ RC column specimens of similar dimensions in tests conducted by the authors in an earlier investigation (Rodsin 2007, Wilson *et al*. 2009). A conservative approach of assuming similar unloading and re-loading behaviour between the two column types was adopted. A hysteresis model of the Modified Takeda form (Fig. 7(b)) was employed to represent the hysteretic behaviour of the column. The model has been calibrated to match hysteretic loops which were recorded from laboratory cyclic tests to define the loading and unloading parameters (α and β, respectively). Strength degradation behaviour (as defined by parameter r in the model) was assumed to be based on forced-displacement behaviour observed from the monotonic field tests. An equivalent viscous damping of 5% has been assumed in the analysis, which is considered to be conservative for slender elements with aspect ratio of 6.8 (Makris and Konstantinidis 2003, Al Albadi *et al*. 2006). Experimental studies conducted by Cheng (2007) on rocking bridge column also recommended a radiation damping value of up to 5% to account for energy losses generated by impact actions. The weight of the superstructure has been included in the SDOF model as a lumped mass at the top.

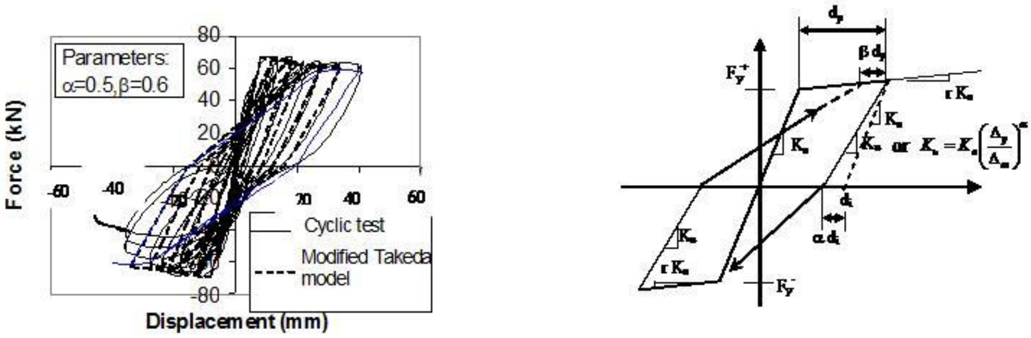
The amount of displacement demand imposed on such structural systems has been estimated from non-linear time-history analyses (THA). The objective of the analyses was to examine the sensitivity of the probability of overturning of the precast RC columns to changes in the values of parameters affecting strength degradation behaviour. Fragility curves based on field measured force-displacement relationships are compared with fragility curves that were simulated numerically using the multi-spring contact elements model (Fig. 8). Accelerograms employed for THA for developing the fragility curves were from the same ensemble employed in the previous



(a) X direction

(b) Y direction

Fig. 6 Comparison of force-displacement behaviour (with ground floor slab isolated from column)



(a) (b)

Fig. 7 (a) Recorded and calibrated hysteresis curves (Rodsin 2007) (b) Modified Takeda model (Carr 2008)

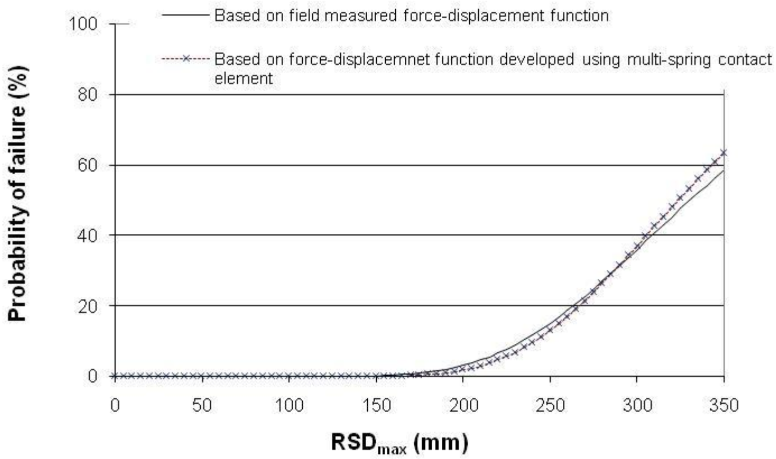


Fig. 8 Fragility curves based on force-displacement relationships obtained from field test and from program Ruaumoko in X direction

study by Kafle *et al*. (2011b) as described in Section 2. It is shown that the predicted peak displacement demand (RSD_{max}) values corresponding to a 5 % probability of overturning were generally insensitive to details of the assumed hysteretic properties of the column. Fig. 8 yet again confirms that the column model developed using multi-spring contact elements in program Ruaumoko provided realistic results.

3.2 Parametric study

Once the column model (involving the use of multi-spring contact elements in program Ruaumoko) has been verified, further analytical simulations of the force-displacement relationship were undertaken for columns with varying dimensions. The parametric study was aimed at modelling the influence of the *size effects* and other design variables. Table 1 presents details of columns involved in the study. The column tested in the field (310 mm × 380 mm × 2600 mm) was used as reference and with the dimensions designated herein as C380. Three other column dimensions have also been considered. Two of the columns (C475, C570) had dimensions that were 25 % and 50 % larger than the reference model (C380) and the third (C285) had dimensions 25 % smaller than the reference model. In all these cases the aspect ratio was kept constant at 6.8 (as for the reference model). The axial load ratio was kept constant at 4% consistent with results from the field tests. For each of the simulated column specimen the force-displacement relationship was developed using program Ruaumoko. The displacement capacities of the column as obtained from analyses are listed in Table 1. The force-displacement relationships that had been simulated were then employed for non-linear THA. Results from these analyses were used for constructing representative fragility curves.

The sensitivity of the probability of failure of the columns to variations in their hysteretic behaviour has also been evaluated by THA in which the *Modified Takeda Model* with β values of 0.2 and 0.6 and constant α value of 0.5 were used. The alternative *Origin Centered Model* has also been used (Kafle 2011). It is revealed from the developed fragility curves that the probability of collapse based on analyses employing the *Modified Takeda Model* and the *Origin Centered Model* were generally consistent meaning that results were not sensitive to variations in the assumed hysteresis relationships. The interesting observation from the developed fragility curve is that the probability of failure of the precast soft-storey column decreases with increasing size of the column; once again illustrating the "size effect" phenomenon (Fig. 9).

Maximum response spectral displacement (RSD_{max}) values associated with 5% probability of failure have been obtained from the developed fragility curves for all the considered column models. Correlation of the RSD_{max} values with the (depth) dimension of the column (i.e. dimension in the direction of excitation) is shown in Fig. 10 for earthquake scenarios of M7 on class D site. RSD_{max} values required to cause collapse of each of the columns for 5% probability of failure were correlated against the base dimension of the column (in the direction of excitation). The curve representing 5% probability of overturning of precast RC columns is presented in Fig. 11 along with another curve representing the rigid blocks. Spaces separated by straight lines and annotated with the "high risk", "moderate risk" and "low risk" tags in the figure are based on Equation (2) as per recommendations by Al Abadi *et al*. (2006). It is shown in Fig. 11 that the curve characterizing the risk of failure of the considered precast RC columns and the rigid body objects have elements of similarities in trends although the exact level of risks can be very different. In summary, a precast RC column (in which rocking behaviour can be facilitated) may be considered to be subject to a probability of overturning not exceeding 5% if the value of the peak displacement demand of the earthquake (RSD_{max}) is exceeded by one-third of the dimension of the column at the base in the direction of ground shaking.

Table 1 Basic dimensions of columns considered in the analysis

Column designation	Cross section (b × d) (mm × mm)	Height, h (mm)	Aspect ratio (h/d)	Displacement capacity (mm)
C285	230 × 285	1950	6.8	185
C380	310 × 380	2600	6.8	250
C475	390 × 475	3250	6.8	310
C570	465× 570	3900	6.8	400

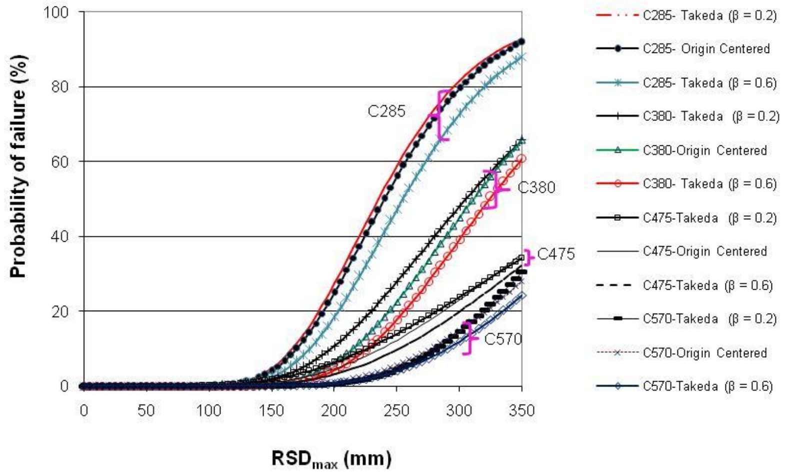


Fig. 9 Comparison of fragility curves for different column dimensions with same aspect ratio (M7 earthquake scenario on class D site)

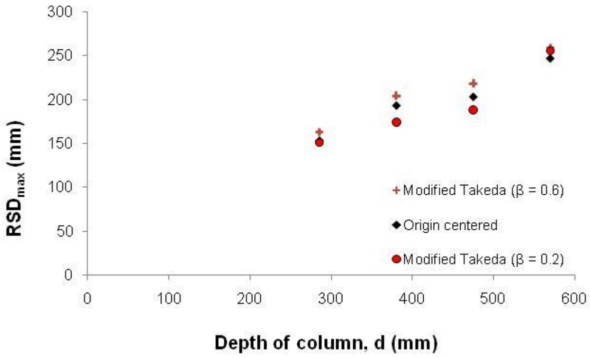


Fig. 10 Correlation of RSD$_{max}$ with column depth for 5 % probability of failure

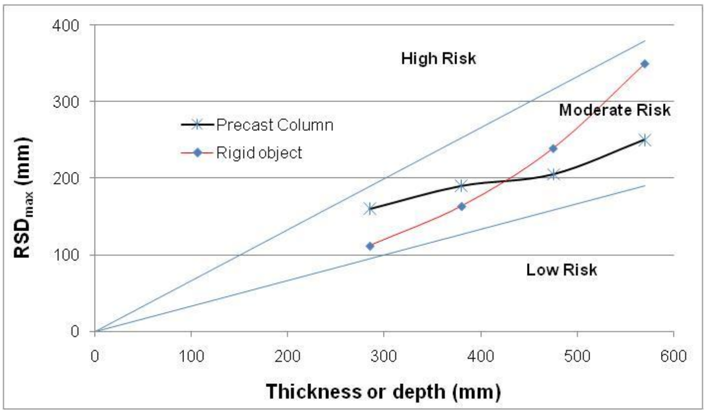


Fig. 11 Risk of failure

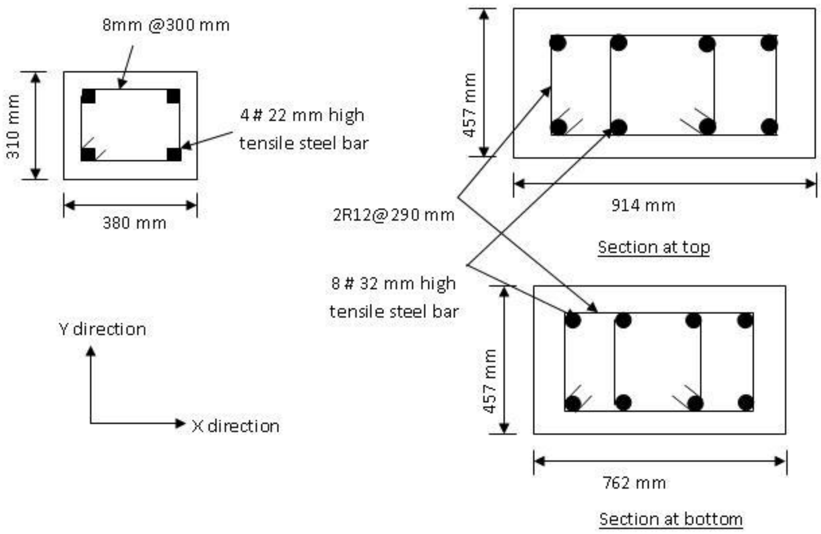
4. Case study

Results of the parametric study revealed an interesting phenomenon concerning the effects of column size and the vulnerability of the soft-storey to collapse. A 4 storey building (similar to the one that was field tested) and a 20 storey building each featuring a soft-storey at ground floor level were used as case studies (refer Fig. 12). The conventional *force-based* procedure as stipulated by the current code of practice (AS1170.4 2007) and the more realistic *displacement-based* procedure which involved non-linear THA of SDOF systems were both employed to assess the ultimate seismic resistant capacity of the buildings.

(a) 4 storey building

(b) 20 storey building

Fig. 12 Photo of case study precast soft-storey buildings



(a) 4 storey (b) 20 storey

Fig. 13 Typical cross section details of columns

Table 2 Basic properties of columns of 4 and 20 storey buildings in Melbourne

Building	Column Width (mm)	Column Depth (mm)	Clear storey Height (mm)	Axial Load Ratio	Transverse reinforcementratio (%)	Longitudinal reinforcement ratio (%)
4 storey	310	380	2600	0.16	0.30	1.05
20 storey	457	762[1] 914[2]	3500	0.27[1] 0.23[2]	0.44[1] 0.41[2]	2.00[1] 1.65[2]

[1] Measured at column base (taper column)
[2] Measured at top of column (taper column)

Table 3 Calculation details for *force-based* method

Building	Mass (tonnes)	Direction	T (s)	RSA (m/s/s)	Base Shear (%W)* Capacity	Demand	Capacity/Demand Ratio
4 storey	61.8	X	0.39	3.90	13.1	15.3	0.86
		Y	0.63	2.43	7.4	9.5	0.78
20 storey	330	X	0.43	3.53	12.0	13.8	0.87
		Y	0.91	1.67	6.2	6.6	0.92

*W is weight on top of the column

The building information and detailing were collated by Rodsin (2007) and predominantly based on information provided by the *Department of Human Services*, Victoria, Australia and sourced from reference works relating to building materials used at the time of construction. Values of the modelling parameters assumed in the analyses for the two buildings are as listed in Table 2 with typical details of the cross-section shown in Fig. 13. The force-displacement relationships of the columns for both the 4 and 20 storey buildings in both X and Y directions were analysed using program Ruaumoko for both *force-based* and *displacement-based* methods of seismic assessments.

4.1 Force-based method

The moment capacity of the columns was first determined in accordance with rectangular stress block principles as stipulated in AS 3600 (2009) whilst the seismic demand imposed on the buildings was calculated using the design acceleration response spectrum as per AS 1170.4 (2007) for site class D. The seismicity assumed for a 500 years return period event corresponds to a peak ground velocity of 60 mm/s or a notional peak ground acceleration of 0.08g on rock. A ductility factor μ = 2 and overstrength factor 1/Sp = 1.3 were assumed in the analyses, effectively reducing the elastic seismic forces by a factor of 2.6 (AS1170.4 2007; AEES 2007). Significantly larger values of ductility factors are recommended by other international building codes for higher seismic regions (e.g. IBC 2006, Eurocode 8 2003) which may alter the force demand behaviour on columns in those regions. The *Capacity/Demand (C/D) ratios* were back calculated for each case by comparing the maximum calculated seismically induced bending moment demand on the columns with the respective calculated nominal bending moment capacity (refer Table 3). Values of C/D ratios were found to be less than unity for both cases meaning that the 4 storey and 20 storey buildings both failed to fulfill conventional *force-based* seismic design provisions for a return period of 500 years. In other words, both buildings could be deemed seismically unsafe, warranting an increase in their lateral strength capacity.

4.2 Displacement-based method

The *displacement-based* method of assessment was then applied to the two buildings. Non-linear THA were undertaken using program Ruaumoko (Carr 2008). The accelerograms ensemble used for time-history analyses were essentially based on earthquake scenarios (of M7 events) on class D sites as described in Sections 2 and 3.The maximum displacement demand at the top of the columns as obtained from the analyses was compared with the displacement capacity of the column to ascertain the risk of collapse. Fragility curves for estimating the probability of failure of the precast column were then constructed based on results from the non-linear THA. A typical fragility curve is shown in Fig. 14 for both buildings in the X direction. Values of RSD_{max} associated with the limit state of collapse for 5 % probability of failure is compared with the value of the displacement demand as read off from the displacement response spectrum for 500 years return period on class D site consistent with AS 1170.4 (2007). The displacement demand for both buildings was estimated to be 60 mm irrespective of their natural period of vibration. Alternatively, the value of RSD_{max} recommended by the authors for a range of magnitude-distance combinations (Lumantarna *et al*. 2012) could also be used for estimating the displacement demand. Earlier studies by the authors (Kafle *et al*. 2011a) demonstrated that the risks of overturning of objects in an earthquake can be estimated by comparing the RSD_{max} value of the ground motion with the object's base dimensions. A similar phenomenon has been observed with the seismic performance behaviour of unreinforced masonry walls in ultimate conditions (Doherty *et al*. 2002, Lam *et al*. 2003). High C/D ratios as listed in Table 4 indicate very low risk of collapse of both buildings.

The risk of failure of the 20 storey building in both directions have been found to be much less than the 4 storey building in view of their respective C/D ratios as summarised in Table 4. The *displacement-based* method of assessment revealed that the precast concrete columns supporting the 20 storey buildings were by far more robust than that of the 4 storey buildings. Importantly, both buildings could be deemed safe. The *size effect*s phenomenon is once again well demonstrated.

It is shown in Table 5 that the 4 storey soft-storey building could be deemed safe from overturning in view of results derived from the *displacement-based* method and more so for the 20 storey building which was supported by much larger size columns. In summary, the *displacement-based* approach clearly reveals the important influence of the column size. Significantly, this phenomenon has not been well captured by the conventional *force-based* calculation method which was found to provide misleading predictions for the vulnerability level of the two buildings.

Table 4 Calculation details of displacement based method

Building	Direction	Displacement (mm)		Capacity/Demand ratio
		Capacity	Demand	
4 storey	X	228	60	3.8
	Y	229	60	3.8
20 storey	X	432	60	7.2
	Y	430	60	7.2

Table 5 Comparison of force based and displacement based method

Building	Direction	Column dimension in X direction (mm)	Column dimension in Y direction (mm)	Capacity/Demand ratio	
				Force based approach	Displacement based approach
4 storey	X	380	310	0.86	3.8
	Y			0.78	3.8
20 storey	X	762[1]	457	0.87	7.2
	Y	914[2]		0.92	7.2

[1] *Measured at column base (tapered column)*
[2] *Measured at top of column (tapered column*

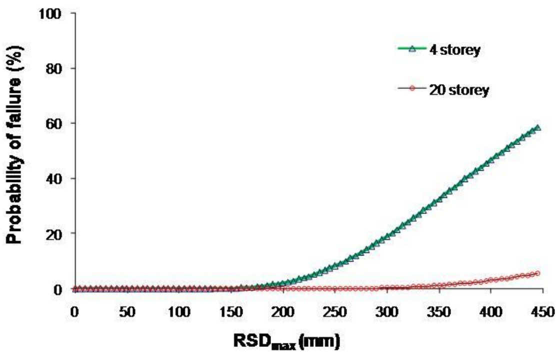
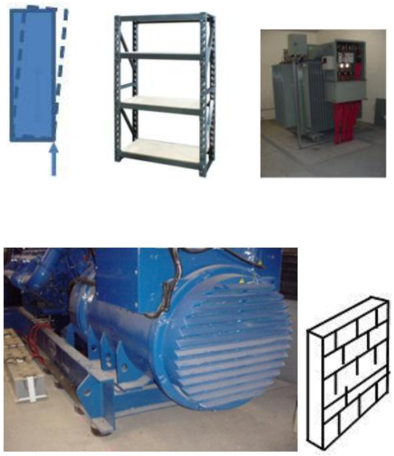


Fig. 14 Fragility curves for collapse of case study soft-storey precast columns

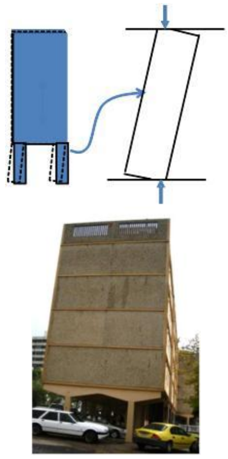
5. Structural engineering applications

The overturning assessment procedure introduced in Section 2 can be applied to assess the risk of free-standing objects, furniture items, mechanical/electrical equipment as well as unreinforced block walls and brick walls from overturning (Fig. 15(a)). In cases where there are holding down devices to secure the item in place, the procedure may still be applied to cater for the extreme scenario where the holding down devices fail to resist the overturning actions. Thus, the methodology presented in this paper provides estimates of the last line of defense against overturning.

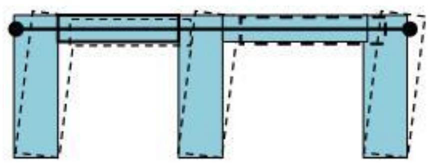
The same procedure has been adapted for assessing the risk of collapse of precast concrete columns which provide support of buildings featuring a soft storey as explained in Section 3 (Fig. 15(b)). Given that the column is carrying most of the load from above and not its own weight the boundary conditions are different to that of a free-standing object. Thus, Eq. (2c) would need to be modified to take into account some 50% increase in the displacement capacity value. It can be shown that the column is deemed to be subject to a low risk of overturning if the peak displacement demand value is equal to, or exceeded by, half the effective thickness of the column in the direction of ground shaking. The effective thickness of the precast concrete column is the



(a) Free standing furniture, equipment, block/ brick walls



(b) Precast concrete columns



(c) Unbonded post-tensioned precast concrete frames

(d) Retaining/ abutment walls

Fig. 15 Applications of the overturning assessment methodology

relative offset of the centroid of the reaction forces developed at both ends of the column and is a fraction of its physical thickness (Wibowo *et al*. 2011).

The displacement demand model introduced in this paper can also be used to assess damage inflicted by pounding of the base of the rocking structure or equipment against the ground (Al Abadi *et al*. 2013).

The assessment procedure could be adapted to assess the displacement capacity of innovative systems featuring precast concrete frames with connections that are joined by unbonded post-tensioned cables to mitigate the seismic risk (Fig. 15(c)). There is also potential to adapt the methodology to assess retaining walls and bridge abutments. However further research is required to study the influence of the interaction between a retaining wall experiencing overturning and the materials used for the backfill (Fig. 15(d)).

6. Conclusions

The displacement behaviour of non-deformable free-standing objects experiencing rocking actions, and their risk of overturning, have been found to correlate well with the value of the maximum response spectral displacement (RSD_{max}) of the applied base excitations. Expressions have been derived to determine the level of risk of overturning as a function of RSD_{max} and the thickness of the object in the direction of earthquake ground shaking.

The displacement behaviour of innovative systems featuring precast concrete frames forming the soft-storey has also been analysed and their risk of collapse have been assessed. The interesting observation from the parametric studies and the developed fragility curve is that the probability of failure of the precast soft-storey column decreases with increasing size of the column importantly illustrating the "size effect" phenomenon. The *displacement-based* approach clearly revealed the important size effects and their implication to the vulnerability of failure over conventional *force-based* calculation method, as presented in case studies conducted on precast RC columns in support of multi-storey buildings.

The procedure introduced in this paper has been applied for the seismic assessment of free-standing objects, furniture items, mechanical/electrical equipment as well as unreinforced block walls and brick walls. The same procedure has also been adapted for assessing the risk of collapse of precast concrete columns at the base of soft-storey buildings.

The procedure can potentially be used for the displacement capacity assessment of innovative systems featuring precast concrete frames with connections that are joined by unbonded post-tensioned cables, and for assessing retaining walls and bridge abutments.

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