Collapse Modeling of Soft-Storey Buildings

Nelson Lam[2], John Wilson[1] and Kittipoom Rodsin[3]

1. Swinburne University of Technology, Hawthorn, Victoria, Australia
2. University of Melbourne, Parkville, Victoria, Australia.
3. King Mogul University, Thailand

Abstract

This paper presents results from an extensive study investigating the seismic performance of 'soft-storey' buildings. Such buildings, rely on the moment and axial action of slender columns to resist horizontal and gravity forces at the ground floor level, and are banned in high seismic regions due to their poor performance in past earthquakes but are a common feature of buildings in low and moderate seismicity regions such as Australia and parts of SE Asia. Analytical and experimental investigations undertaken by the authors on a number of existing buildings employing displacement principles in contrast to the conventional force-based principles indicated that the performance of some of these buildings could be satisfactory for ultimate limit state earthquake scenarios projected for countries like Australia despite their non-compliance with contemporary design standards. A summary of the analytical studies, laboratory testing and full-scale field tests will be presented. This project is aimed at developing a realistic seismic risk model for this class of buildings in order that retrofitting work can be prioritised effectively on existing building stock.

Keywords: Soft storey buildings, earthquake performance, drift capacity

1. Introduction

Studies undertaken by the authors in recent years have indicated that the existing building stock at most risk of damage and collapse from earthquake excitation in low and moderate seismicity regions such as Australia are unreinforced masonry buildings and soft storey structures (those buildings that possess storeys that are significantly weaker or more flexible than adjacent storeys and where deformations and damage tend to be concentrated). Soft storeys commonly occur at the ground floor where the functional requirements dictate a higher ceiling level or a more open configuration (eg. for car parking or retail space) resulting in an inherently weaker and more flexible level. In high seismic regions soft storey structures and unreinforced masonry are banned, yet in regions of lower seismicity such building types and configurations are common and are often occupied by organisations with a post-disaster function or house a significant number of people. This paper will address the performance of soft-storey buildings under earthquake excitations specifically. Research findings presented in this paper are directly relevant to low-moderate seismic regions worldwide such as Thailand, Vietnam, Hong Kong, China and Singapore where similar soft-storey structures of limited ductility are commonly constructed.

Figure1 Photos of soft-storey buildings in Australia

Soft-storey buildings are considered to be particularly vulnerable because the rigid block at the upper levels has limited energy absorption and displacement capacity, thus leaving the columns in the soft-storey to deflect and absorb the inelastic energy. Collapse of the building is imminent when the energy absorption capacity or displacement capacity of the soft-storey columns is exceeded by the energy demand or the displacement demand. This concept is best illustrated using the 'Capacity Spectrum Method' shown in Figure 2 where the seismic demand is represented in the form of an acceleration-displacement response spectrum (ADRS diagram) and the structural capacity is estimated from a non-linear push-over analysis expressed in an acceleration-displacement relationship (as illustrated in Wilson & Lam, 2006).

The structure is considered to survive the design earthquake if the capacity curve intersects the demand curve and collapse if the curves do not intersect. In regions of high seismicity, the maximum displacement demand could exceed 200-300mm which translates to a drift in the order of 5-10% in a soft storey structure, which is significantly greater than the drift capacity of such structures even if the columns have been detailed for ductility. This is the reason soft-storey structures have behaved poorly and collapsed in larger earthquake events around the world. In high seismic regions, buildings are configured and detailed so that in an extreme event a rational yielding mechanism develops to dissipate the energy throughout the structure and increase the displacement capacity of the building. Ductile detailing in reinforced concrete columns includes closely spaced closed stirrups to confine the concrete, prevent longitudinal steel buckling and to increase the shear capacity of columns (Mander, 1988; Park, 1997; Paulay & Priestley, 1991; Watson *et al*, 1994; Priestley & Park, 1987; Bae et al, 2005, Priestley, 1994; Bayrak & Sheikh, 2001; Berry & Eberhard, 2005; Pujol et al, 2000; Saatcioglu & Ozcebe, 1992). The emphasis is on the prevention of brittle failure modes and the encouragement of ductile mechanisms through the formation of plastic hinges that can rotate without strength degradation to create the rational yielding mechanism.

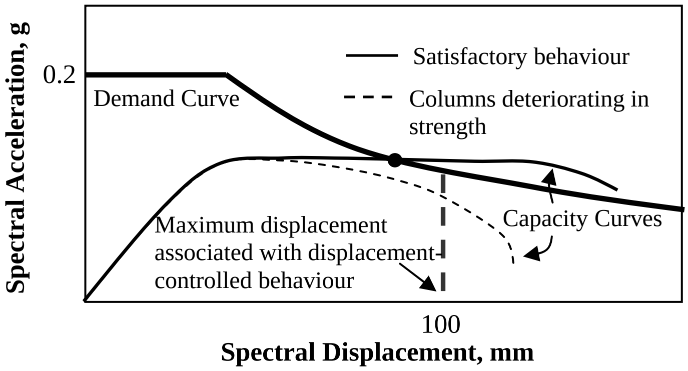


Figure 2 Schematic view of acceleration-displacement response spectrum diagram

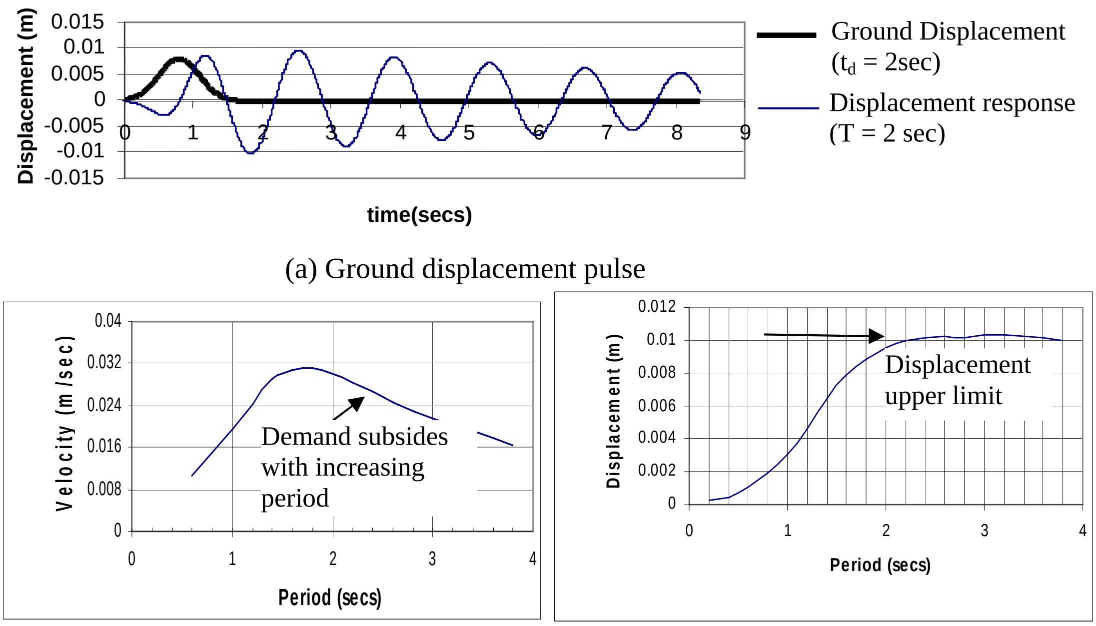
Current detailing practice in the regions of lower seismicity typically allow widely spaced stirrups (typical stirrup spacing in the order of the minimum column dimension) resulting in concrete that is not effectively confined from crushing and spalling, longitudinal steel that is not prevented from buckling and columns that are weaker in shear. Design guidelines that have been developed in regions of high seismicity (ATC40, FEMA273) recommend a very low drift capacity for columns that have such a low level of detailing. The application of such standards in the context of low-moderate seismicity regions results in most soft-storey structures being deemed to fail when subject to the earthquake event consistent with a return period in the order of 500 – 2500 years.

2. Displacement Controlled Behaviour

2.1 General

The current force-based design guidelines are founded on the concept of trading strength for ductility to ensure the structure has sufficient energy absorbing capacity. The developing displacement-based (DB) design methodologies may also be calibrated to fulfill this objective more elegantly (eg. Chopra & Goel, 1999; Davidson et al, 1999; Fajfa & Gaspersic, 1996; Goel et al, 2000; Miranda & Ruiz-Garcia, 2002; Priestley, 2000; Priestley & Kowalsky, 2000; Wilson & Lam, 2006). In each load-cycle, the amount of energy absorbed is equal to the integral product of the resisting force (strength) and deformation ("ductility"). This approach assumes that the imposed kinetic energy does not subside during the displacement response of the building which is not unreasonable in regions of high seismicity where the earthquake magnitudes are larger and the duration of ground shaking longer. The limitation of this approach in lower seismic regions is examined herein with the idealized pulses shown in Figure 3.

The velocity developed in an elastic single-degree-of-freedom system would increase with increasing natural period (T) until T approaches the pulse duration (t_d) when maximum velocity is developed. Importantly, as T continues to increase, the velocity demand subsides while the displacement levels-off to a value constrained by the peak ground displacement (PGD). It is hypothesized that this phenomenon of displacement-controlled behaviour can be extended to inelastically responding systems in which case T/2 corresponds to the time taken by the structure to load-and-unload (eg. Priestley, 1995).



(a) Ground displacement pulse

(b) Velocity response spectrum (c) Displacement response spectrum

Figure 3 Displacement and velocity response spectra from an idealized pulse

The single-pulse scenario, despite its simplicity (which is convenient for illustration), has been used in formal evaluations to quantify the seismic demand of the more complex pulse trains in small and moderate magnitude earthquakes on rock sites in intraplate regions (Lam & Chandler, 2005). However, on some soft soil sites, the displacement demand of periodic pulses on the structure can be many times higher than the PGD when conditions pertaining to soil resonance behaviour are developed (refer Figure 4). Even then, the peak displacement demand on the structure is well constrained around a definitive upper limit. Extensive research undertaken by the authors (eg. Lam et al, 2000a-c, 2001, 2003; Lam & Wilson, 2004; Wilson & Lam, 2003 & 2006; Lam & Chandler, 2004) has culminated in the drafting of the new Standard for earthquake actions for Australia incorporating this important upper displacement demand limit (AS/NZS 1170.4-2007). A 500 year return period hazard factor of Z=0.08g, which corresponds to a notional peak ground velocity of PGV=60 mm/sec, is consistent with an upper displacement limit (RSD_{max}) of between 30 mm and 90 mm depending on the soil conditions. These predictions, associated with displacement-controlled behaviour, were based on the assumption that the earthquake magnitude would not exceed an upper limit of around M=7 in view of the size of active faults that have been identified within most intraplate regions.

This new displacement-controlled design phenomenon (not to be confused with the displacement-based design methods) is particularly relevant to low-moderate seismic regions where the size of active faults are more modest (although there are exceptions; eg. Memphis in Eastern US). In theory, similar displacement constraints could be identified for high seismic regions but the associated larger displacement demand values would not be tolerated by most structures and hence is of limited practical interest.

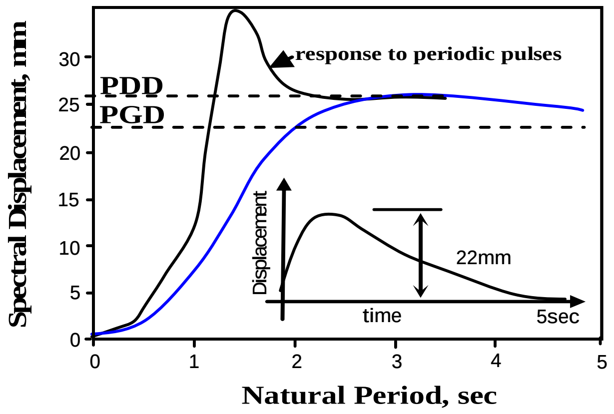


Figure 4 Displacement response spectrum for single and periodic pulses

2.2 Torsionally-unbalanced behaviour

It is commonly the case in soft-storey buildings that the centre of strength and centre of mass in plan are significantly eccentric. According to current concepts (which are supported by field experiences in major earthquakes), the building is expected to translate and rotate in plan, amplifying the drift demands in the columns which are more distant from the centre of strength (eg. Chopra & Goel, 1991; Tso & Wong, 1995; Rutenberg & Pekau, 1989). However, displacement-controlled behaviour could also mean that the maximum displacement demand on the structure is insensitive to changes in mass (hence natural period) as the maximum displacement demand limit is reached. Consequently, different parts of the building have the tendency to displace by similar amounts, even if the distribution of the tributary masses and/or lateral resistant elements are non-uniform. This leads to another important phenomenon which has been demonstrated by the authors (Lumantarna *et al*, 2007) hypothesis that the amplification of displacement demand in a torsionally irregular building is also limited by displacement-controlled behaviour.

3. Experimental Investigations

Cyclic testing of half-scale reinforced concrete column specimens has been carried out by the authors to investigate the drift capacity of columns with wide stirrup spacing and relatively low aspect ratios as outlined in Table 1. These specimens were designed to be representative of columns from a range of soft-storey buildings that were identified from a reconnaissance field study. Specimens were subjected to quasi-static loading history as suggested by Priestley and Park (1987). The loading history was modified to suit the non-ductile behaviour of the column. A column specimen was initially subjected to one cycle of lateral loading ± 0.75 times the ultimate strength at the critical section (F_u). The yield displacement (Δ_{yu}) was then found by extrapolating a straight line from the origin through the force-displacement point at 0.75 F_u to the theoretical flexural strength F_u. The average of the two values calculated for the two cyclic reactions was adopted. Subsequent loading considered of displacement-controlled testing to ductility ratios (μ) of ± 1, ± 2, ± 3, ± 4, ± 5, and ± 6. Two cycles of loading were used with each ductility ratio to ensure that the hysteretic behaviour could be maintained. Digital photos of the columns were taken when the peak displacement had been reached in each load cycle. The test was terminated only when the column had lost its axial load carrying capacity.

Two drift limits were of particular interest in those tests: (i) the drift at maximum horizontal strength of the column (refer “hump” on Figure 5) and (ii) collapse drift limit which the gravity load carrying capacity of the column could not be sustained. The 1.0% - 1.3% drift measured at maximum strength (as shown in Table 1 for specimens S1 and S2) can be specified from theory quite accurately by considering the different mechanisms of (i) flexure, (ii) shear and (iii) yield penetration (refer Section 4).

Table 1 Concrete column specimens

Specimens	Dimensions (mm)	Aspect Ratios	A_s/bd (in %)	Stirrups	f_c' MPa	$P/(A_g f_c')$	Drift at peak strength	Drift at collapse
1	200 x 160 x 750	3.75	1.4	R6 @ 150mm	45	0.2	1.3%	2.7%
2	200 x 160 x 550	2.75	1.4	R6 @ 150mm	45	0.2	1.0%	3.6%

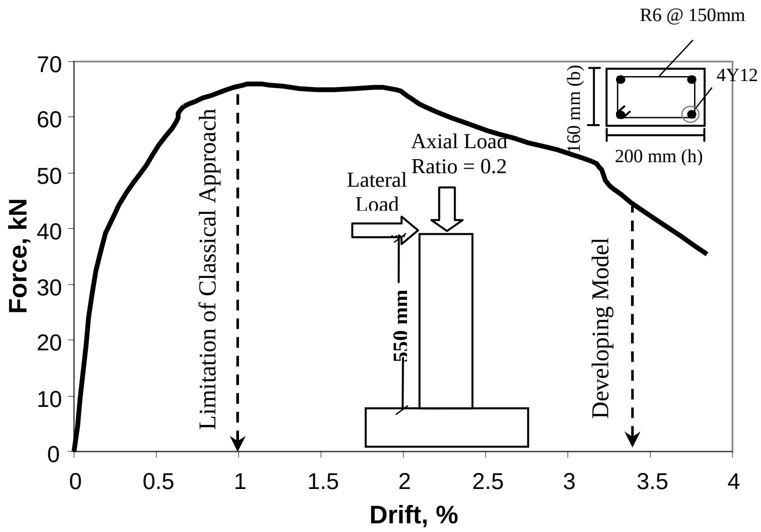


Figure 5 F-Δ relationship for Specimen S2

Deformations constituted by each of these mechanisms were identified in the experiment using Digital Close Range photogrammetry technique, which is commonly known as the Vision Methology System (VMS). The collapse drift limit is importantly different to the commonly used ultimate drift limit, which is associated with the drift at the threshold of significant strength degradation typically assumed to be 20% from the maximum strength. In contrast, the collapse drift is defined as the drift at the threshold of gravity collapse (ie. the gravity carrying capacity of the column is compromised).

With the flexure-dominated column (specimen S1), the gravitational load carrying was lost following the buckling of the compression reinforcement. With column specimen S2, high shear forces have resulted in the formation of shear cracks distributed along the length of the specimen. The specimen was capable of carrying full axial load whilst its lateral strength deteriorated.

In both column specimens, yield deformation occurred at approximately 0.5% drift when existing cracks increased in size and new inclined cracks developed. Shear cracks gradually developed and widened with subsequent cycles of loading. Deterioration of the column lateral strength was observed under cyclic loading. Lateral strength of column specimens dropped by 20% at a drift limit of approximately 2.7%. Bar buckling in column specimen S1 (due to spalling of concrete cover at the critical section) resulted in the sudden loss of the column gravitational load carrying capacity. It is concluded that column S1 failed in flexural compression given that the column failure was initiated by the buckling of the compression reinforcement. In contrast, column S2 was able to sustain axial loading following a 20% reduction in its lateral load resisting capacity. Such desirable behaviour is associated with the uniform distribution of shear cracks (as opposed to the localised spalling of concrete) in the column. Due to the influence of shear, the column is able to undergo further displacement before the compression strains reach a value sufficient to cause spalling of the cover concrete. Finally, at a drift of 3.6%, shear failure associated with the opening of diagonal cracks and buckling of longitudinal bars could be observed just before the column lost its axial load carrying capacity (refer Figure 5). Column S2 eventually failed in compression flexure shear.

4. Theoretical Model for Column Collapse

Columns that possess high shear span-to-depth ratio (for example greater than 3.5) are typically characterised by a flexural failure mechanism. The displacement limit for gravity load collapse is defined as the point where the moment of resistance of the section has reduced to a value equal to the moment generated from the "P-δ" effect. The significant reduction in moment of resistance of a section is due to crushing (spalling) of concrete and followed by the buckling of longitudinal reinforcement in the compression zone as shown in Figure 6.

The ultimate displacement at gravity load collapse can be estimated from a deformation model developed by the authors which includes an ultimate compressive strain model suggested by Paulay and Priestley (1992) and a bar buckling model modified from Bae (2005). The deformation model which takes into account the deflection of the column from flexure, shear and yield penetration has been described in Rodsin (2004). A unique feature of the proposed model is that the crushing of concrete and buckling of the longitudinal reinforcement on the compression side of the critical section is permitted even when the concrete has not been well confined by stirrups as shown in Figure 6. Crushing (spalling) of the unconfined concrete is assumed to occur at an ultimate strain limit of 0.006 based on results of tests conducted by the authors (Rodsin *et al*, 2004). When this happens, the concrete section within the ultimate strain limit is assumed to contribute to the residual flexural strength (as shown by the stress diagrams of Figure 6c) whilst the concrete beyond the ultimate strain is ignored. The residual strength of the column is presented in the form of a moment capacity versus rotation (M-θ) relationship for the plastic hinge located at the base of the column. The rotation is calculated from the product of the curvature from the strain diagram and the plastic hinge length assumed equal to the column stirrup spacing. The moment demand at this location is given by equation (1).

$$M = V \cdot h + P \cdot \delta \tag{1}$$

It is shown from the M-θ relationship that at the point of collapse, the moment generated by the P-δ term is equal to the residual moment capacity of the column section (refer Figure 6d).

The aforementioned deformation model has been used to construct envelope curves for hysteretic force-deformation behaviour of the columns (refer Figure 5). The model is shown to provide a good prediction for the ascending curve for column S1 but some conservatism between experimental results and theoretical predictions are evident for column S2. This is explained by the additional contributions to the rotation of the column by shear cracking. The predicted drift limits using the developed model are significantly greater than those recommended in the well publicised design guidelines of ATC40 & FEMA273 as shown in Table 2. This conservatism of the guidelines is particularly evident in the shear dominant (S2) specimen. Interestingly, ATC40 & FEMA273 stipulate zero post-yield drift capacity for shear critical columns (with widely spaced stirrups).

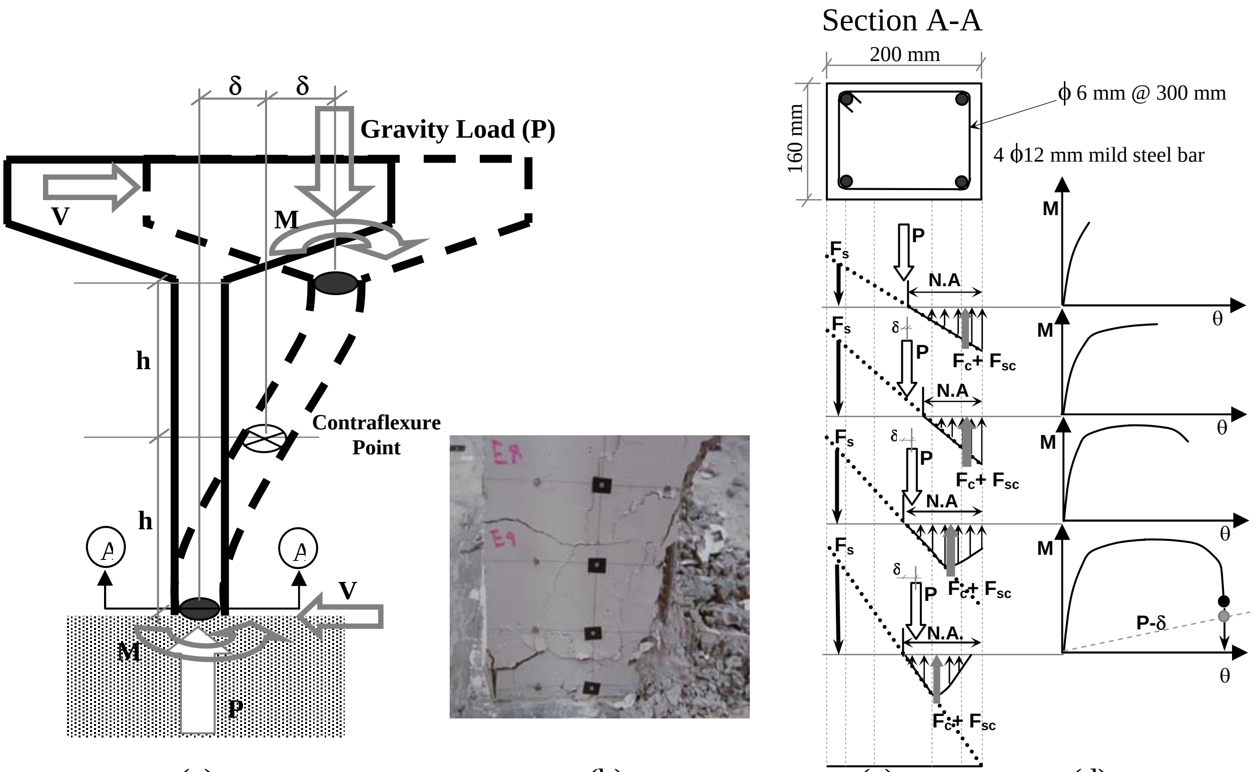
Table 2 Drift limit of columns

Specimen	Experimental Results	Model Predictions	ATC40 & FEMA273
S1	2.7 %	2.1 %	1.5 % (1.0 % + 0.5 %) *
S2	3.6 %	2.7 %	1.0 % (0.5 % + 0.5 %) *

*The drift limits shown were obtained as the sum of the post-yield drift (values read off directly from ATC40 and FEMA273) and the yield drift estimated by the authors based on a yield strength of 400 MPa for the longitudinal reinforcements.

5. Closing Remarks

Results from experimental investigation into the cyclic behaviour of half-scaled reinforced concrete columns have been reported. For both columns, deformation at maximum resistance (hump) and 20% loss of lateral strength was found to be 1% and 2.7% drift respectively. Compression bar buckling has resulted in the axial failure of column specimen S1 at a drift limit of 2.7%. In contrast, column specimen S2 was able to maintain full axial load up to a drift limit of 3.6% (at which the column failed by flexural shear). The tests clearly demonstrated that existing design guidelines such as ATC40 and FEMA273 provide very conservative ultimate drift limits for columns with a low level of detailing. The test revealed that additional rotation at shear cracks increased deformation capacity of the columns. Importantly, the ability of columns to sustain axial load at large deformations was improved due to this effect. The deformation model provided satisfactory predictions for the ascending envelope and was conservative for the post-peak deformation. An analytical model that can accurately and reliably estimate the drift limit of a column at the threshold of loss in axial load carrying capacity has yet to be developed. Further column tests with different detailing and shear span-to-depth ratios will be undertaken to validate the model. Subsequently, the model will be part of the displacement-based methodology that can be used in predicting the performance of soft-storey buildings in regions of low to moderate seismicity. In addition, a series of full scale field tests are planned to measure the force and displacement capacity of four soft storey buildings that are in the process of demolition.



(a) (b) (c) (d)

Figure 6 Flexural failure mechanism of a column ; (a) a column supporting a soft-storey building subject to lateral force, (b) photo of a column failed in flexure (c) stress diagram at the critical section of a column subject to increasing lateral deformation, (d) moment – rotation (M-θ) relationship at the critical section of a column.

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