1. INTRODUCTION

The seismic performance of buildings can be assessed using a capacity spectrum approach as illustrated in Figure 1. Provided the capacity curve of the structure intersects the seismic demand curve, the structure is deemed not to have collapsed at this return period event. In regions of high seismicity, the maximum displacement demand can be very large and as a result the structural performance is usually controlled by the constant velocity section of the demand curve as illustrated in Figure 1a. Consequently, structures must possess adequate lateral strength and displacement capacity to perform satisfactorily. In such regions, structures are deemed to fail when the lateral strength capacity has reduced to 80% of the design strength.

In regions of lower seismicity, the maximum displacement demands are typically more modest. For example, a PGV of 50-60 mm/sec generated by a small to medium magnitude (M<6) earthquake, would result in a peak displacement demand of less than 50 mm on a rock or stiff soil site. This peak displacement demand could more than double on a softer soil site. Structures may perform satisfactorily provided that the displacement capacity exceeds the displacement demand as shown in Figure 1b. Consequently, buildings that display a significant deterioration in lateral strength with increasing lateral displacement may not necessarily fail in such displacement-controlled conditions associated with the lower seismic regions.

This paper examines the performance of reinforced concrete buildings that have a soft-storey at ground floor level characterised by weak columns and strong beams. The objective of this paper is to develop a model that can predict the maximum displacement capacity of a reinforced concrete column, at which the column can no longer support gravitational loading. Flexure and shear-dominated columns are introduced in Section 2 and 3 respectively, whilst a validation of the analytical model with experimental results is presented in Section 4.

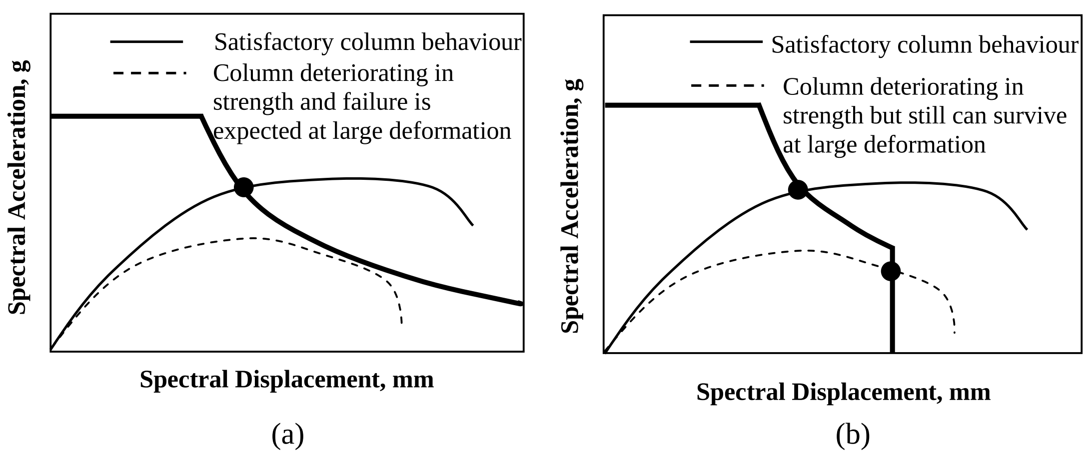


Figure 1 Capacity spectrum diagram of buildings characterised by (a) velocity-controlled and (b) displacement-controlled behaviour

2. DISPLACEMENT CAPACITY OF COLUMN DOMINATED BY FLEXURE

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 following by buckling of longitudinal reinforcement in the compression zone as shown in Figure 2.

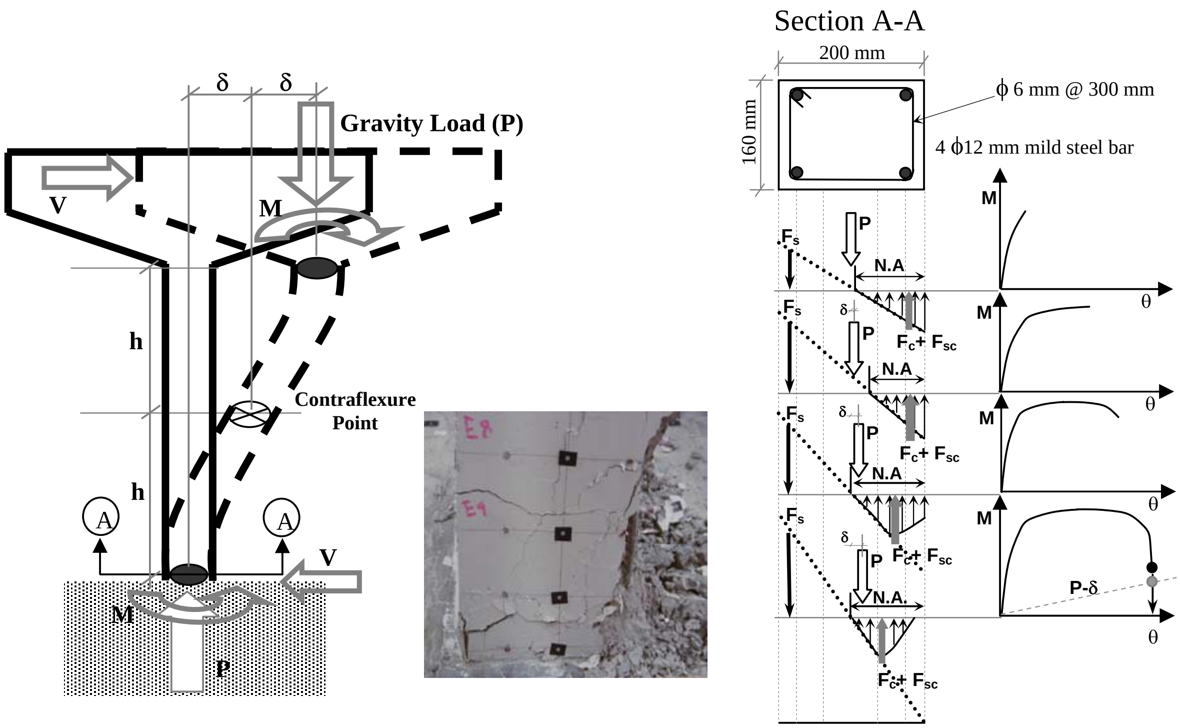
The ultimate displacement at gravity load collapse can be estimated from a deformation model developed by the authors which includes an ultimate compressive strain model suggested by Paulay and Priestley [1992] and a bar buckling model modified from Bae [2005]. The deformation model which takes into account the deflection of the column from flexure, shear and yield penetration has been described in Rodsin [2004]. The 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 2. 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 2004]. When this happens, the part of the concrete section which is within the ultimate strain limit is assumed to contribute to the residual flexural strength as shown by the stress diagrams of Figure 2c 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 moment demand at this location is given by equation (1).

$$M = V \cdot h + P \cdot \delta \qquad (1)$$

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 2d).

3. DISPLACEMENT CAPACITY OF COLUMNS DOMINATED BY SHEAR

A shear failure mechanism is likely to dominate the ultimate behaviour of columns with low shear span-to-depth ratios. Column shear failure occurs when the shear demand exceeds the residual shear strength capacity as shown in Curves 2 and 3 of Figure 3. In contrast, column shear failure is suppressed and the column fails in flexure when the residual shear capacity is higher than the maximum inferred shear force demand as depicted in Curve 1 of Figure 3.



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

Figure 2 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.

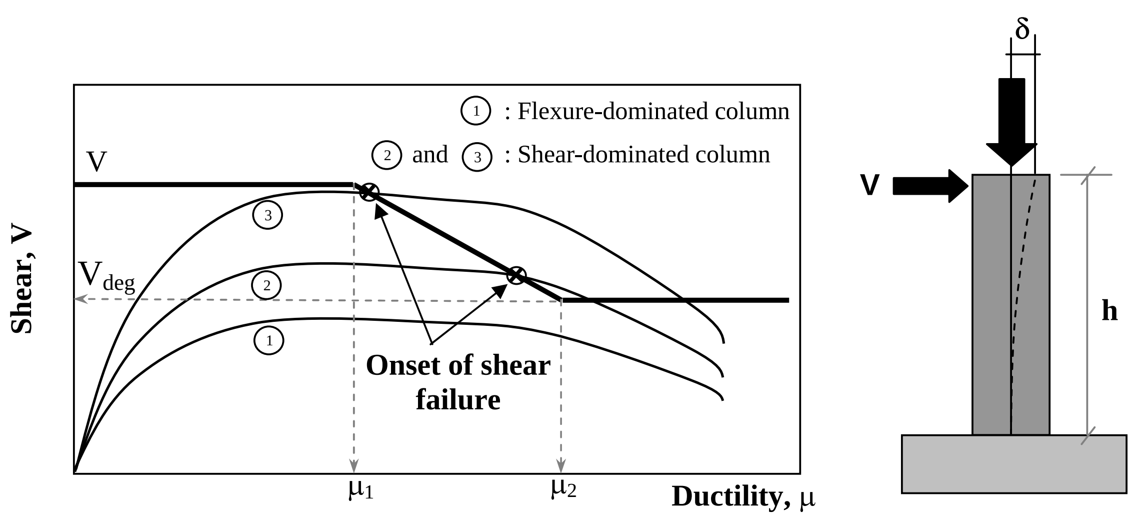


Figure 3 Deformation at onset of shear failure

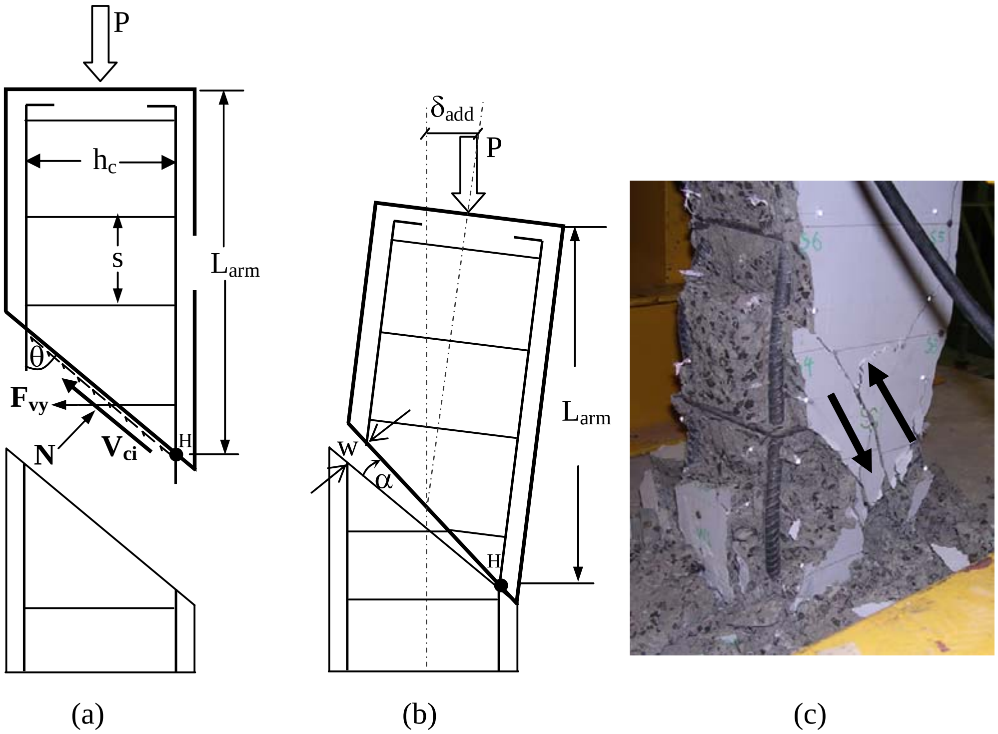


Figure 4 (a) Onset of shear failure, (b) additional displacement due to rotation at critical shear failure plane after onset of shear failure and (c) photo of column shear failure

The onset of shear failure is expected to occur when the degraded shear force capacity of the column intersects with the shear force demand (refer Figure 3). The modified compression field theory (MCFT) first proposed by Vecchio and Collin [1986] can be used to construct the "ductility dependent residual shear strength" relationship shown in Figure 3, whilst the shear force demand relationship (inferred from the force-displacement relationship of the column) can be calculated using the model developed in Section 2. The simplified shear failure mechanism is shown schematically in Figure 4 (the authors have developed a more detailed model that includes the effect of a degrading shear force on the column, but this is beyond the scope of the paper and consequently the model presented assumes that the column shear force V has degraded to a value close to zero).

It was observed by the authors from recent experimental investigations that if slippage does not occur along the shear failure surface, then the column may rotate by a small angle α about the compression edge causing the crack to open slightly as shown in Figure 4b. The additional column deformation (δ_{add}) associated with this crack opening can be found by substituting the maximum crack width (w) as calculated from equation (6) into equations (2a) and (2b).

$$\alpha = \frac{w \sin\theta}{h_c} \tag{2a}$$

$$\delta_{add} = \alpha \cdot L_{arm} = w \frac{\sin\theta}{h_c} L_{arm} \tag{2b}$$

where α is the angle of crack opening, θ is the angle defining the orientation of the crack, h_c is the width of the concrete core and L_{arm} is the distance between the tip of the column and the rotation point (H).

It is noted that although the angle of crack opening (α) is generally very small, the associated increase in the displacement capacity of the column can be significant due to the column geometry. The calculated column deflection at the point of gravity load instability (collapse) may include the additional displacement (δ_{add}) associated with the crack opening. The remainder of this section describes in detail the proposed methodology for calculating the maximum crack width (w).

Gravity collapse or slippage is deemed to occur when the gravity load component resolved in the direction of slip along the shear failure surface ($P\cos\theta$) approaches the resistance to slip from the contribution of both the stirrups and the aggregate interlock as described in equation (3a) (It is conservatively assumed that the longitudinal steel reinforcement has buckled and therefore does not contribute to an increase in the normal force N on the shear failure surface). In equation (3a), P is the gravity load, F_{vy} is the stirrup yield strength, S is the stirrup spacing and V_{ci} is the shear resistance attributed to aggregate interlock. Angle θ can be estimated using MCFT or alternatively, a conservative value of 30^o may be assumed.

$$P\cos\theta = F_{vy}\frac{h_c}{S}\cos\theta + V_{ci} \tag{3a}$$

$$\therefore \quad V_{ci} = P\cos\theta - F_{vy}\frac{h_c}{S}\cos\theta \tag{3b}$$

The normal stress on the shear failure surface (σ) can be calculated by resolving forces in the direction normal to the shear failure surface as shown by equation (4).

$$\sigma = \frac{N\sin\theta}{h_c b} \quad \text{and} \quad N = P\sin\theta + \frac{F_{vy}h_c}{S\tan\theta}\cos\theta \tag{4}$$

where N is the force applied normal to the shear failure surface.

The shear resistance (V_{ci}) required at the failure surface of the column to support a given gravitational load P can be calculated from equation (3b) and substituted into equation (5) to solve for v_{cimax}, the maximum shear stress parameter.

$$V_{ci} = v_{ci\max}\left(0.18 + 1.64\frac{\sigma}{v_{ci\max}} - 0.82\left(\frac{\sigma}{v_{ci\max}}\right)^2\right)\cdot b\frac{h_c}{\sin\theta} \qquad (5)$$

where b is width of the column section.

The maximum crack width (w) which corresponds to v_{cimax} can be obtained from equation (6) based on the model developed by Walraven [1981].

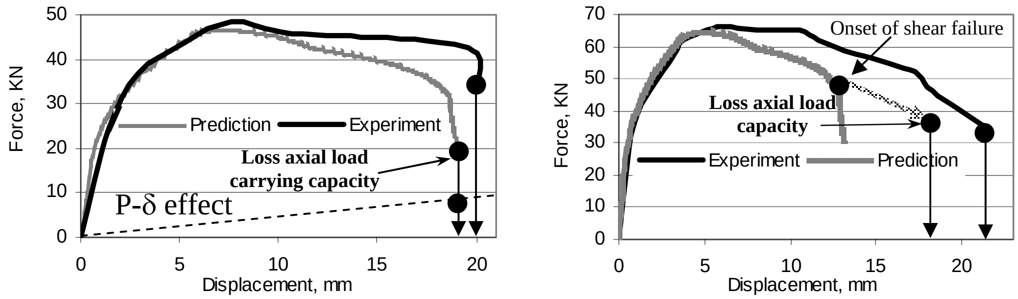
$$w = \frac{(\sqrt{f_c'} - 0.36v_{ci\max})\cdot(C+16)}{12v_{ci\max}} \qquad (6)$$

where f_c' is the compressive strength of concrete and C is the maximum aggregate size (mm). This value of w can then be used to solve for δ_{add} using equations (2a) and (2b).

4. MODEL VALIDATION

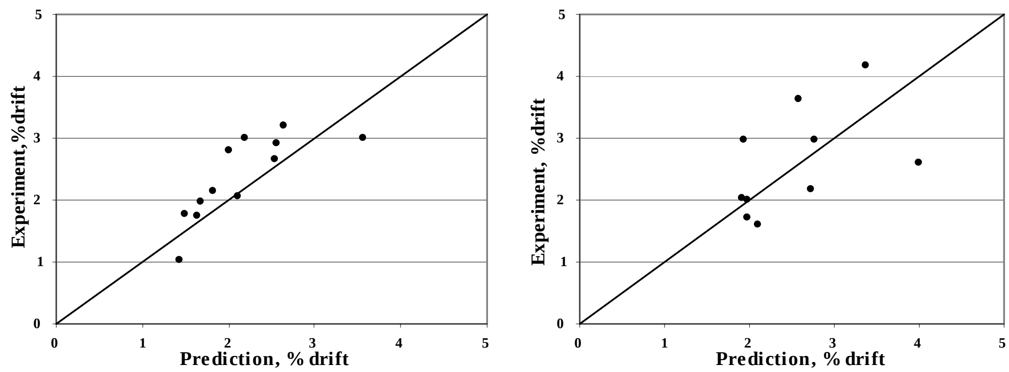
A model has been developed by the authors for determining the force-displacement behaviour of the columns that are either dominated by flexure or by shear. Separate models have also been developed for predicting the ultimate displacement at which gravity load collapse occurs (refer Sections 2 and 3). The force-displacement relationships calculated in accordance with the models developed are shown in Figures 5a-5b along with relationships obtained experimentally from a quasi-static experiment undertaken by the authors. The comparisons show that the displacement limits predicted by the models are generally consistent with the experimental results. For a column failing in shear, the additional displacement capacity attributed to crack opening (ie. rotation about the edge of the failure surface) is represented by the right-hand side of the "prediction" relationship shown in Figure 5b. The proposed deformation model is conservative since only rotation at the failure surface has been considered. In reality, the opening of other shear cracks contributing to additional rotations of the column was observed from the experiment.

The proposed models have been further evaluated using a more extensive database of experimental results reported in the literature. Results of the evaluation are presented in Figure 6 which correlates the predicted displacements at failure with experimental observations. The statistical parameter characterising the quality of the correlation is presented in Table 1 which indicates that the proposed model is an improvement to the three other predictive models commonly referenced in the literature.



(a) Flexure-dominated Column (b) Shear-dominated column

Figure 5 Force-displacement relationships of flexure and shear-dominated column



(a) (b)

Figure 6 Comparison between predicted and experimental observations ultimate deformation for (a) flexural failure and (b) shear failure column

Table 1 Comparison of experimental versus predicted ultimate column displacement using 4 different models

	Flexure-Dominated Column		**Shear-Dominated Column**	
Model	Exp./Predt. (Average)	Exp./Predt. (SD)	Exp./Predt. (Average)	Exp./Predt. (SD)
Proposed Model	1.12	0.17	1.06	0.27
FEMA-273, 1997	1.94	1.08	1.64	1.01
Panagiotakos et al, 2001	1.16	0.56	1.07	0.40
J.P. Moehle et al, 2002	N.A.	N.A.	1.67	1.17

5. CONCLUSIONS

The concept of displacement-controlled behaviour has been introduced in this paper whereby the ultimate drift limit is based on the condition when gravity loading can no longer be supported by the damaged column. An analytical model for predicting the ultimate deflection of the column dominated either by flexure or by shear has been developed. The predictions obtained from the model show good agreement with experimental results obtained from tests performed by the authors and that collated from the international literature. The development of the gravity-collapse model for estimating the displacement capacity of soft-storey columns forms an important part of the displacement-based methodology for assessing the seismic performance of building structures.

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