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# Collapse Modelling Analysis of a Precast Soft-Storey Building in Melbourne

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## Abstract

Experimental field testing of a soft storey building in Melbourne has been undertaken by Swinburne University of Technology in collaboration with The University of Melbourne. The upper levels that consisted of precast walls and slabs were demolished to the first floor. The soft storey open ground floor was a precast concrete frame with connections significantly weaker than the members they connected. Four tests were conducted with combination between load directions (strong and weak) and restraints of ground slab (with or without ground slab). The experimental results show that soft storey columns were found to have significant displacement capacity irrespective of strength degradation.

An analytical model has been developed to predict force-displacement relationship of the tested frame. The model includes the influences of: a) connection strength at column ends; b) gravity rocking strength; and c) ground slab restraint. Results from the developed model were found to be in excellent agreement with experimental test results, showing that the top connection in the form of an unbonded high strength steel bars dominated the overall load-deflection behaviour in the strong direction. However, the gravity rocking mechanism dominated the behaviour in the weak direction. The presence of the ground slab provides additional restraint to the column and significant additional lateral strength to the system.

## Keywords
Precast concrete, soft storey structure, force-displacement relationship, earthquake performance, field testing.

## 1. INTRODUCTION

### *1.1. Background*

A Soft storey building (Figure 1) is one that has a discontinuity in the stiffness of the building where one storey is significantly more flexible than adjacent storeys. According to ASCE 7-05 (Reference Section Table 12.6-1), a soft storey has lateral stiffness less than 70% of that of the storey immediately above, or less than 80% of average stiffness of the three stories above. Under substantial ground shaking, soft storey buildings behave like an inverted pendulum with the ductility demand concentrated at the soft storey elements.

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 seismic energy whilst resisting the axial gravity loading. 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 (Wilson & Lam, 2006).

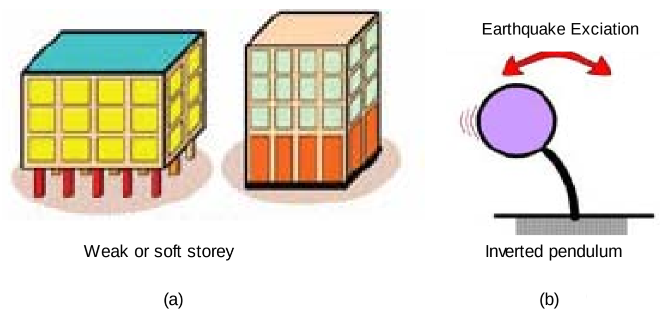


Figure 1 Idealization of Soft Storey Structures

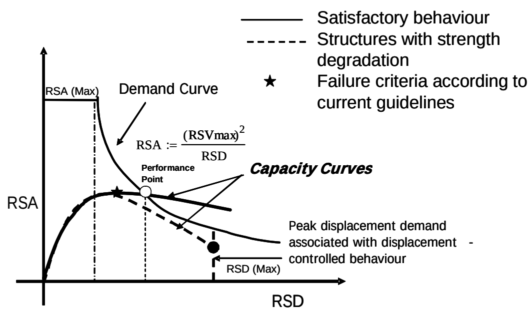


Figure 2 Capacity spectrum method

### *1.2. Scope and Objective*

A unique research project has been conducted by Swinburne University of Technology in collaboration with the University of Melbourne, which involved the experimental field testing of a four-storey soft storey building in Melbourne. The objective of the experimental investigation was to study the load deflection behaviour and collapse modelling of soft storey buildings when subjected to lateral loading.

This paper will provide a brief overview of the experimental field-testing of the soft storey building, including details of the building configuration, experimental test set-up and instrumentation, and test results together with a comparison with theoretical predictions. Due to space constraints, this paper will focus on the results and analysis for the strong direction tests only.

## 2. BUILDING CONFIGURATION AND TEST SET-UP

### *2.1 Building configuration*

The test building in Figure 3 consisted of four levels above the open plan ground storey. The upper levels of its residential building consisted of precast walls and slabs creating a rigid box whilst the ground floor was constructed from reinforced concrete columns and beams founded on individual pad footings. The building is significantly stronger in the short portal direction compared with the long spandrel direction. Observations from the pre-trial test of adjacent buildings indicated that the building to be used for the experimental testing had a precast ground floor storey (Figure 4) with connections significantly weaker than the members they connected. Consequently, the ground floor columns tended to rock when subject to a horizontal load. Several material samples were also collected from the site to investigate the properties of the building elements, and were tested to determine steel and concrete properties.

### *2.2 Test set-up*

Four push-over field tests were undertaken on a ground floor bay consisting of four columns pre-loaded with kentledge. It was decided for safety reasons to demolish the upper levels of the building to first floor level to create the test bay without damaging the portal frames. Four test bays were selected for testing and were separated from each other by saw cutting the floor slab between adjacent bays. A steel frame was constructed at first floor level and positively secured to the slab and beams to provide support for the kentledge and to provide anchorage for the lateral load to be applied to the soft storey bay. Horizontal loads were applied in both the strong and weak directions via steel tension ties and hydraulic jacks secured to a piled reaction located at some distance from the test bay as shown schematically in Figure 5. The four columns in a typical bay would typically support around 200 tonnes of dead load plus a live load from the upper storeys. However, it was not deemed practical to load the frame with the full gravity load and consequently only 50 tonnes of kentledge in the form of precast ‘jersy barriers’ was added to provide a reasonable loading.

It appeared that, the slab on ground provided significant restraint to the columns at ground floor level and consequently two tests were conducted with the ground slab intact and the other two tests with the slab cut away to prevent restraint. The four field tests conducted were:

a. Test 1: Strong direction with ground slab
b. Test 2: Weak direction with ground slab
c. Test 3: Strong direction without ground slab
d. Test 4: Weak direction without ground slab

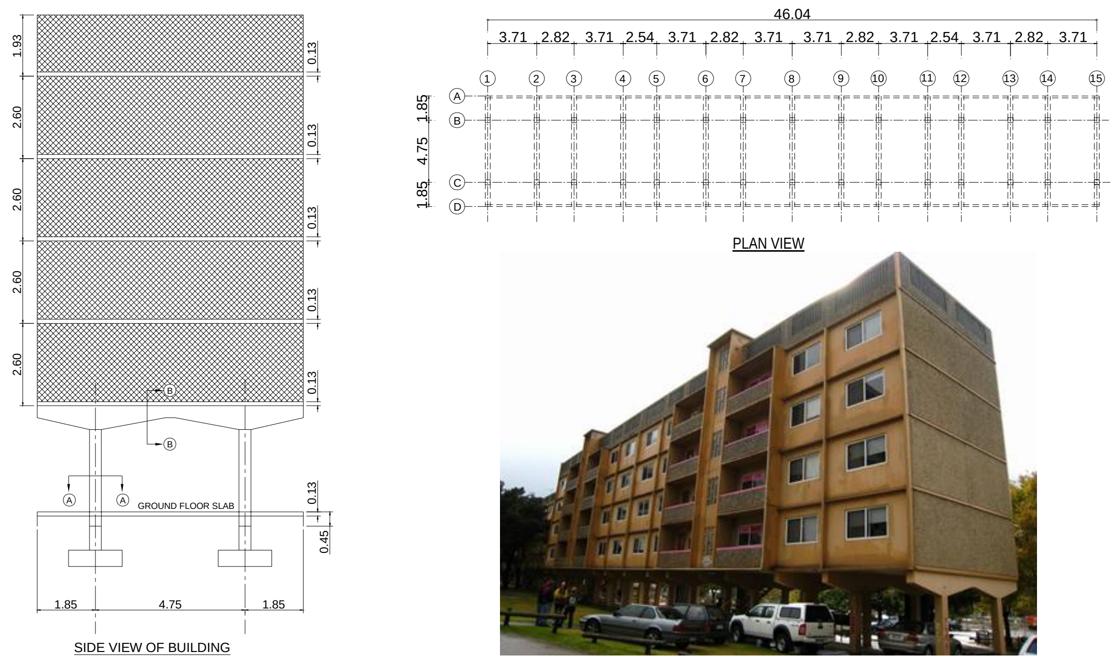



Figure 3 Configuration of the buildings

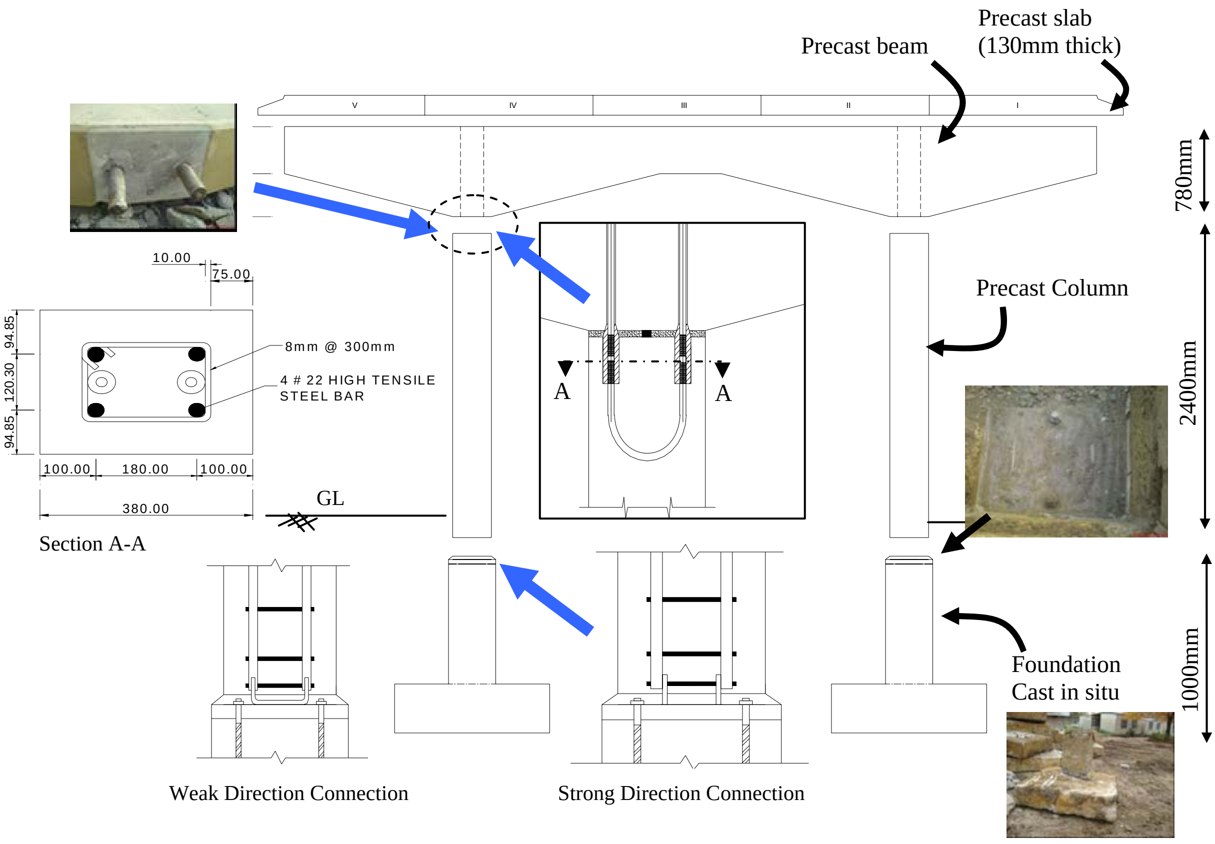



Figure 4 Structural details of a typical frame

## 3. INSTRUMENTATION

Various measurement techniques were utilised to obtain the overall load deflection behaviour of each test specimen as well as curvature of the column and crack width. The applied horizontal loads were measured using load cells, whilst the displacement measurement techniques included global positioning system (GPS), total point station (TPS), laser scanner, photogrammetry, visual measurement using a theodolite and ruler, and LVDT transducers. A degree of redundancy was built into the measuring systems to ensure that if one system failed, results could be obtained from other sources.

## 4. TEST PROCEDURE AND EXPERIMENTAL RESULTS

The test specimens were laterally loaded under 'force' control in increments of 10 KN until the ultimate load was reached. The loading was then applied in 'displacement' control with displacement increments of 25 mm up to around 250 mm in the direction of loading.

A comprehensive set of results were obtained from the experimental testing and a sample load displacement curve for strong direction tests are shown in Table 1 and Figure 6. The displacement shown corresponds to the lateral displacement at the slab level and the load is the total lateral force imposed on the structure. In the strong direction, the majority of the deformations were concentrated at the column end connections, with gaps opening at the foundation and header beam interfaces. This was a clear indication that the columns were significantly stronger than the connections. It can be shown analytically that the load-deflection behaviour of the strong direction was mostly affected by the connection strength at the top of the column.

Table 1 Summary of maximum load, displacement and drift of all tests

| Orientation | Test | Maximum Load | Maximum Lateral Displacement | Maximum Drift |
|---|---|---|---|---|
| Strong Direction | Test 1 | 310 kN | 200 mm | 5.9 % |
| | Test 3 | 250 kN | 255 mm | 7.5 % |

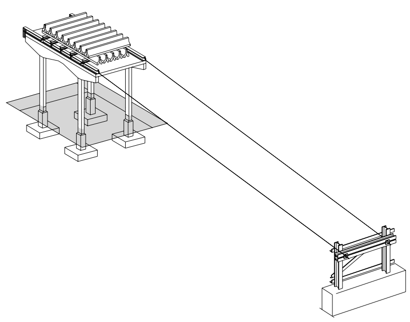


Figure 5 Test set-up configurations

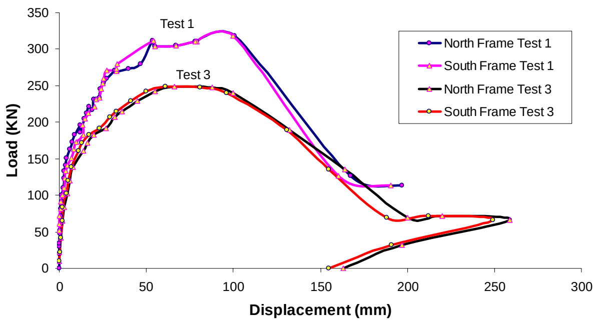



Figure 6 Experimental load-deflection results

The soft storey column was found to have significant displacement capacity irrespective of strength degradation. An important outcome of this work is that the columns maintained their gravity load carrying capacity at a lateral displacement of about 260mm or a drift capacity of about 8% under these quasi-static conditions. Interestingly, the weak column/foundation and column/beam precast connections allowed the columns to rock about their ends, greatly enhancing the displacement capacity of the soft storey system compared with rigid end column connections more typical of in-situ construction. The ground slab provided significant restraint to the frame about 25 percent in the strong direction.

## 5. THEORETICAL PREDICTION ANALYSIS

This section develops a theoretical model to predict the load–deflection behaviour of the soft storey test bays which is then compared with the experimental results. The ground floor framing had been designed as a precast system with the connections significantly weaker than the members. Consequently, the horizontal capacity could be calculated from both the connection capacity of the top and base of the column ($F_{TC}$ and $F_{BC}$) combined with the gravity load rocking mechanism ($F_R$) and the additional restraint provided by the slab on ground ($F_{GS}$) as described in Table 2 and expressed in the following equation:

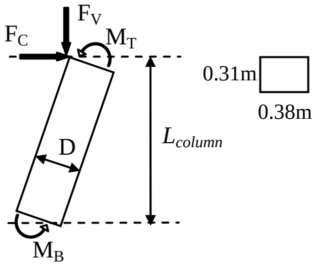
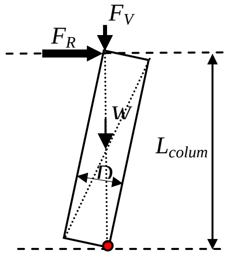
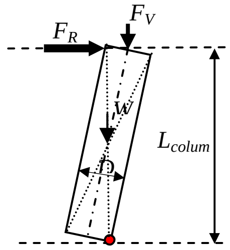
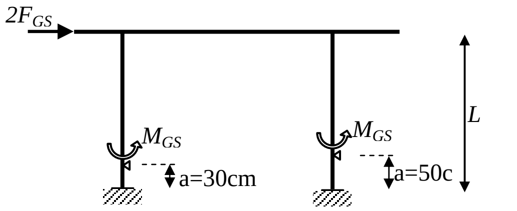
$$F_H = F_C + F_R + F_{GS} = F_{TC} + F_{BC} + F_R + F_{GS} \quad (1)$$

Based on an ultimate strength analysis, the moment strength of the base connection provides a negligible contribution to the overall system capacity and can be ignored. However, the shear strength of the base connection influences the overall system behaviour for the tests that involve column interaction with the ground slab.

The strength of the top connection between the precast concrete column and beam members is provided by precast concrete ductile connection via the concrete in compression and the unbonded steel tendon in tension (and fully bonded steel reinforcement, if present at the connection interface). The characteristics of precast concrete ductile connections have been conducted over the last two decades. Several approaches for predicting the behaviour of this mechanism consist of trilinear idealisation of moment-rotation behaviour (Priestley and Tao, 1993), calibration of the hysteresis parameter of hybrid connections (Cheok *et al*., 1998) and finite element modelling (El-Sheikh et al., 1999). The top connection analysis (Figure 7) in this paper was developed based on the moment-rotation principle proposed by Pampanin *et.al* (2001).

The presence of the unbonded tendon prevents the use of a closed form solution to estimate the depth of the neutral axis because of strain incompatibility between the steel and concrete at the connection interface. Consequently, the neutral axis position was estimated from a trial and error procedure that solved the equilibrium equations at the connection interface by estimating the tendon tensile strains and forces from a global displacement and joint rotation consideration. The compression concrete force is determined using moment-curvature analysis (Park and Paulay, 1974), whilst the forces in the mild steel and in HS steel bars are calculated as a function of steel strain from MS and HS stress-strain relationship respectively. In the case of this test building, there were no bonded steel reinforcement bars at the connection interface which simplifies the model as shown in Figure 8.

Table 2 General Principles of Lateral Strength Prediction Analysis for Strong Direction Test

| | | | |
|---|---|---|---|
| Total Capacity per column | $F_{H-STRONG} = F_C + F_R + F_{GS}$ | | |
| Connection Strength |   | $F_C = \dfrac{M_C}{L_{column}} = \dfrac{M_T + M_B}{L_{column}}$ | |
| Gravity Rocking Mechanism |  Upper Bound Case |  Lower Bound Case | $F_R = \dfrac{F_v D + W(D/2)}{L_{column}}$ The actual location of vertical load lies between lower and upper bound depends on an actual neutral axis position. |
| Ground Slab Effect |  | | The ground slab functioned as a reaction support to column; hence, the increase of horizontal load can be determined by calculating its cross section moment capacity. |

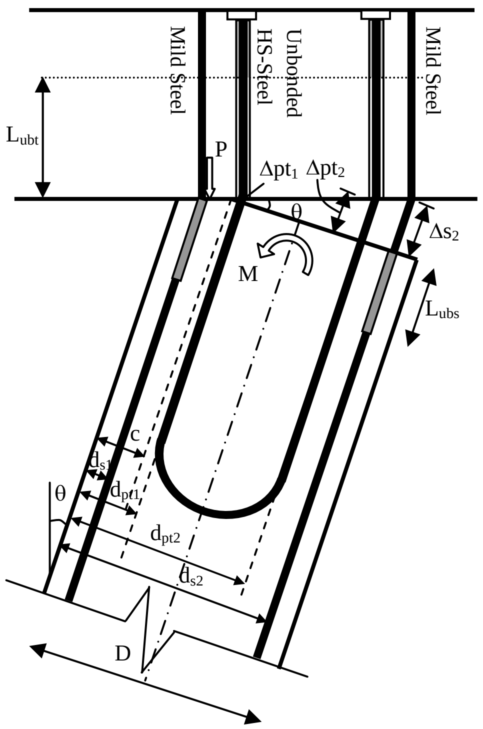



Figure 7 Gap opening mechanism for common hybrid precast connection

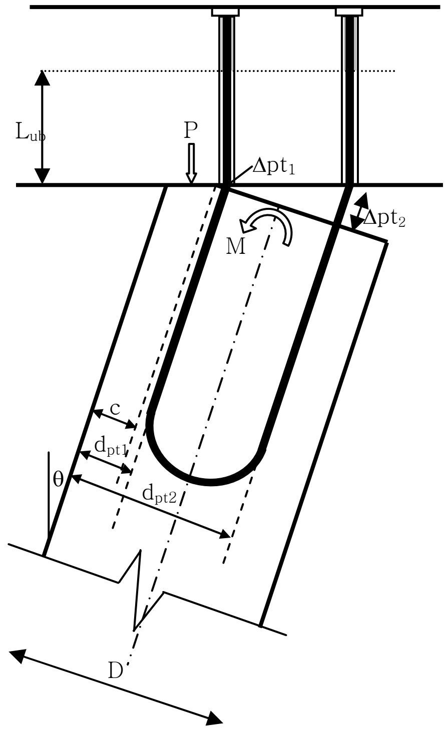



Figure 8 Gap opening mechanism for the experimental field test case

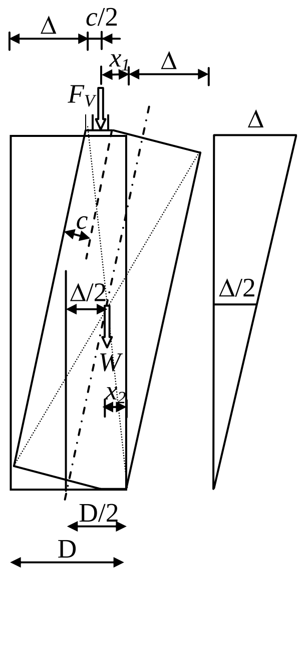


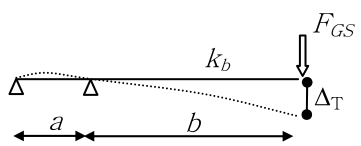
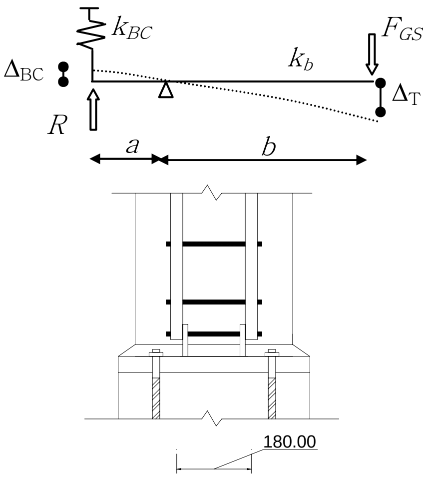

Figure 9 Gravity rocking mechanism

The simple horizontal load prediction for the gravity rocking mechanism can be obtained by using an upper and lower bound estimate, based on the possible location range of the neutral axis (the neutral axis was assumed at the centre and edge of the column for the lower and upper bounds respectively). In this section, the actual location of the neutral axis provided by top connection analysis has been used to calculate the horizontal capacity of the rocking mechanism (Figure 9) and expressed as:

$$F_R = \frac{F_V\left(D - \Delta - \frac{c}{2}\right) + W\left(\frac{D-\Delta}{2}\right)}{L_{column}} \qquad (2)$$

The ground slab provides a restraint to the lateral movement of the column as described in Table 3. This additional restraint modifies the global behaviour of the structure from a pure rocking mechanism into a partial cantilever column mechanism. As a result, a bending moment develops in the column at the level of the ground slab.

Table 3 Analysis of Ground Slab Restraint

| | |
|---|---|
|  | $k_b = \dfrac{3E_C I_C}{b^2(a+b)}$ (3)<br><br>$F_{GS} = k_b.\Delta_T = \dfrac{3E_C I_C}{b^2(a+b)}\Delta_T$ (4) |
|  | Strong direction<br>$k_{BC} = \dfrac{3E_s I_s}{(L_s/2)^3}$ (5)<br><br>$R = \dfrac{b}{a}F_{GS} = \dfrac{3E_C I_C}{ab(a+b)}\Delta_T$ (6)<br><br>While : $R = 2\,k_{BC}\,\Delta_{BC}$ (7)<br><br>$\Delta_{BC} = \dfrac{R}{2k_{BC}} = \dfrac{3E_C I_C}{2ab(a+b)\dfrac{3E_s I_s}{(L_s/2)^3}}\Delta_T$<br>...................... (8) |

Where :

| | |
|---|---|
| $k_b$ | = stiffness of column for hinge-hinge support |
| $I_C$ | = Moment inertia of column cross section |
| $a$ | = distance between column base and ground slab |
| $b$ | = distance between ground slab and top of column |
| $F_{GS}$ | = horizontal load on top of column from ground slab resistance |
| $\Delta_T$ | = global displacement of structure on top of column |
| $k_{BC}$ | = lateral stiffness of mild steel U-bar at column base |
| $A_s$ | = cross section area of mild steel U-bar at column base |
| $I_s$ | = moment of inertia of mild steel U-bar cross section at column base |
| $L_s$ | = length of mild steel U-bar at column base |
| $R$ | = horizontal reaction at column base |
| $\Delta_{BC}$ | = lateral displacement of column base |

## 6. ANALYSIS RESULTS

The summary of each strength component are shown in Table 4, whilst the comparisons between predicted and actual results are shown in table 5. The load-deflection curve for the strong direction test with the ground slab (Test 1) is shown in Figure 10, whilst the contributions from the components are shown in Figure 11. Similarly, the results without the ground slab (Test 3) are shown in Figures 12 and 13. The maximum lateral resistance from the top connection (164 kN) is markedly higher than that from the rocking mechanism (92 kN), whilst the base column connection provides negligible strength for both tests (ie. around 0.1kN). Significantly, the ground slab in Test 1 provides an additional 20% strength capacity, compared with the strength achieved in Test 3 without the ground slab.

The complete force-displacement relationship for the test bay can be idealised into the following four stages as illustrated in Figure 14.

(i) The resisting force increases steeply as a combination of the increase in both the connection and rocking mechanism strength until the rocking mechanism reaches peak strength at about 10mm displacement.

(ii) The resisting force increases more gradually as the rocking strength component decreases.

(iii) The resisting force plateaus as the unbonded high-strength steel bars yield and the concrete stress reaches ultimate strength.

(iv) The resisting force then decreases significantly, as the connection loses strength, high strength steel bars fracture, compression mild steel bars yield, concrete cover commences to spall and the rocking mechanism dominates.

Table 4 Horizontal capacity estimate for strong direction

| Strength Components | Predicted Capacity (kN) | |
|---|---|---|
| | (i) | (ii) |
| $F_C$ | 164 | 160 |
| $F_R$ | 92 | 82 |
| $F_{GS}$ | 69 | 65 |
| $F_H$ (without ground slab) | - | 242 |
| $F_H$ (with ground slab) | - | 307 |

Note:
i) Maximum strength of each component
ii) Strength of each component at maximum lateral capacity

Table 5 Comparison between predicted and actual results

| Orientation | Test | Predicted Capacity (KN) | Actual Maximum Load (KN) |
|---|---|---|---|
| Strong Direction | Test 1 (with ground slab) | 307 | 310 |
| | Test 3 (without ground slab) | 242 | 248 |

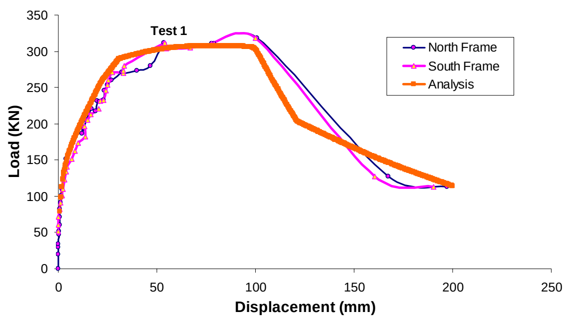



Figure 10 Comparison between experimental and analysis result for Test 1 (without ground slab)

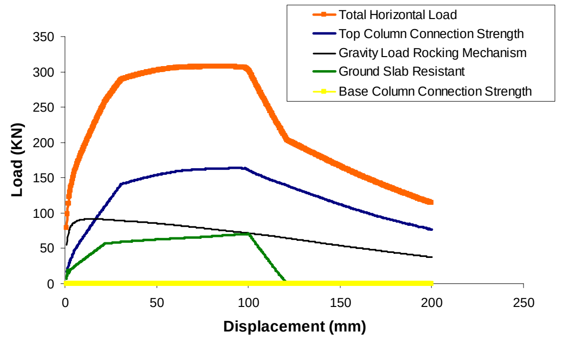



Figure 11 Component of lateral strength resulted from the analysis for Test 1

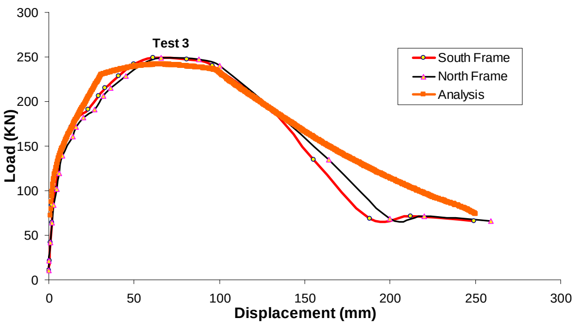



Figure 12 Comparison between experimental and analysis result for Test 3 (without ground slab)

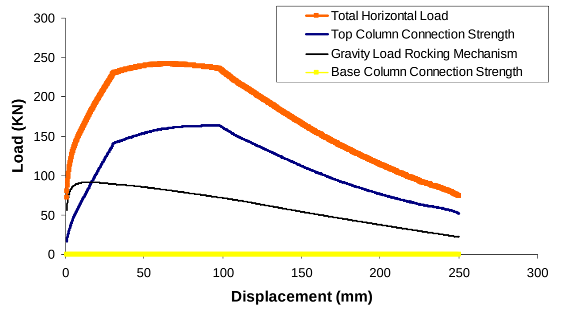



Figure 13 Component of lateral strength resulted from the analysis for Test 3

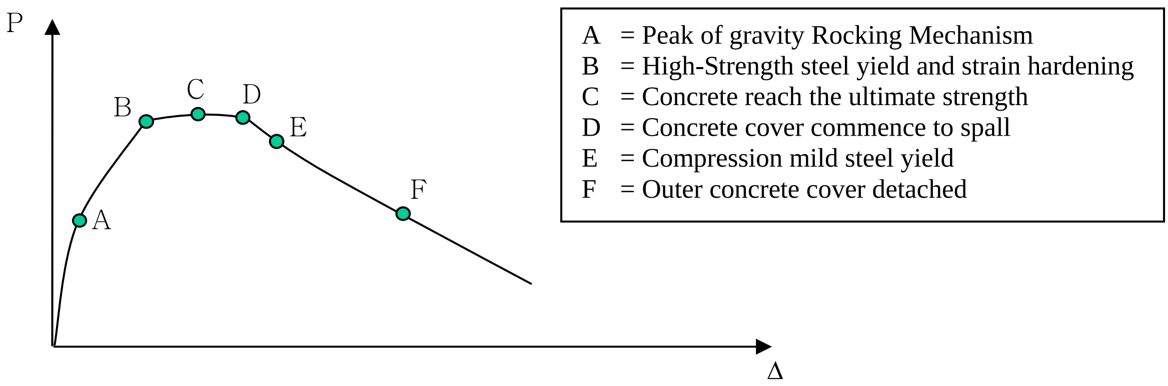



Figure 14 Typical lateral load-displacement relationship resulted from analysis

## 7. CLOSING REMARKS

A collaborative research project involving experimental field testing of a four-storey high soft storey building in Melbourne has been undertaken by the Office of Housing, Swinburne University of Technology and the University of Melbourne. Four test bays were tested in the strong and weak directions to obtain actual push-over force-displacement curves. The preliminary results showed that the soft storey columns could sustain large drifts in the order of 6-8% whilst maintaining the gravity axial loads despite the reduced lateral strength capacity due to P-delta effects. The horizontal strength and drift capacity predicted by a rocking model was in excellent agreement with the lateral capacity obtained from the experimental tests.

The large drift capacity of the precast soft storey structure was attributed to the weak connections which allowed the columns to rock at each end. Interestingly, the lateral strength capacity would have increased significantly if the column end connections were as strong as the members, but the drift capacity would have reduced substantially since the rocking mechanism would have been prevented forcing the columns to deform inelastically in shear and flexure. Hence, the precast soft storey construction resulted in a weaker structure with far greater drift capacity compared with a more traditional insitu reinforced concrete structure.

Another important result from the experimental testing was the influence of the ground floor slab in providing restraint to the base of the columns and increasing the lateral capacity, particularly in the weaker spandrel direction.

## 8. ACKNOWLEDGEMENT

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