Time history analysis correlation between observed and predicted response of typical industrial buildings with steel portal frame and concrete tilt panel cladding during Christchurch earthquake

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ABSTRACT: For the past five decades, use of steel portals with precast tilt panel cladding has been a common practice for construction of industrial buildings throughout New Zealand. The portals are expected to carry roof loads as well as resisting lateral forces. It is generally thought that panels function as both part of cladding for the building and lateral support resisting seismic forces.

Following the Canterbury earthquakes, cracking of the tilt panels and spalling of concrete around the connections has regularly been observed. In some cases panel cleat failure has resulted in loss of support of the panels, resulting in the panels being thrown out-of-plane. However, there appears to have been no observed tensile failure of the panel reinforcing that has been sufficient to result in the collapse of the cladding panels.

This paper presents a summary of observed and predicted (by Time History Analysis) damage, following the Canterbury earthquakes for this particular building system. The Ground motion recorded from Heathcote Valley Primary School (HVSC) during the February 22nd Christchurch earthquake has been used for the Time History Analysis of a typical portal frame warehouse structure with concrete cladding panels attached.

Results indicate that the ground shaking recorded at Heathcote Valley Primary School during the February 22nd earthquakes was sufficient to yield the reinforcing in precast concrete cladding panels. Panels with rigid attachments (such as return walls) were more likely to have yielded the ductile reinforcing such that a risk of reinforcing fracture would be present. Panels attached to flexible portal frames appear to have been protected from the ground shaking by the long period response of the portals. A significant factor in determining the potential damage to the reinforcing is the strain length. This strain length is difficult to ascertain for lightly reinforced panels and a sensitivity assessment has been used to represent the risk of fracture.

1 INTRODUCTION

After the Christchurch earthquakes Batchelar McDougall Consulting Ltd (BMC) were involved in undertaking building assessments which included typical portal frame structures supporting precast concrete cladding panels.

Many of the cladding panels had cracked and it was typical for the panels to exhibit a yield line failure pattern to varying degrees.

BMC were asked to provide a suitable repair strategy for the damaged panels. Options that existed were:-

- Panel Replacement,
- Epoxy injection of the cracks,
- Provision of secondary support.

In assessing the options, full panel replacement was often impractical, epoxy injection did not attend to the possibility of damage to the reinforcing, therefore secondary support, often in the form of

horizontal girts, was suggested as the most practical repair option.

The potential damage to the reinforcing was however difficult to assess. Initially after the earthquake it was hoped to have the opportunity to be able to extract and test reinforcing at the location of significant cracks in order to determine the extent of strain damage that may have occurred to the reinforcing. This opportunity, however, did not eventuate and we remained uncertain.

It was therefore decided to undertake a non-linear time history analysis of a typical portal frame warehouse structure with concrete cladding panels attached. The intention of the time history analysis was to investigate the plastic strain demand (post elastic stretch) that may have occurred to the reinforcing in the panels under face loading.

2 TYPICAL OBSERVATIONS

Observation showed that cracking to precast panels was widespread and that damage to the connections had also occurred. In a few instances panel connections had failed and panels fell from buildings. However, no visual evidence of reinforcing stretch or fracture was observed.

Photo 1: Upper half of the panel is spanning horizontally resulting (for this case) in widespread vertical cracking.

Photo 2: Example of typical residual crack in a panel.

Photo 3: Instances of panel collapse and stability concerns occurred.

Photo 4: Concrete crack (1.0-1.5mm) ready to receive repair by epoxy injection.

3 THE PORTAL FRAME STRUCTURE

3.1 The Structure

A portal frame warehouse building with concrete cladding panels was sized for analysis.

The structure, which is represented in Figure 1 and Figure 2 below, consists of:-

- 310ub46 portal frames with fixed base connections and spaced at 8.0m centres.
- 150mm thick precast panels attached to the portal at the portal knee and column mid height.

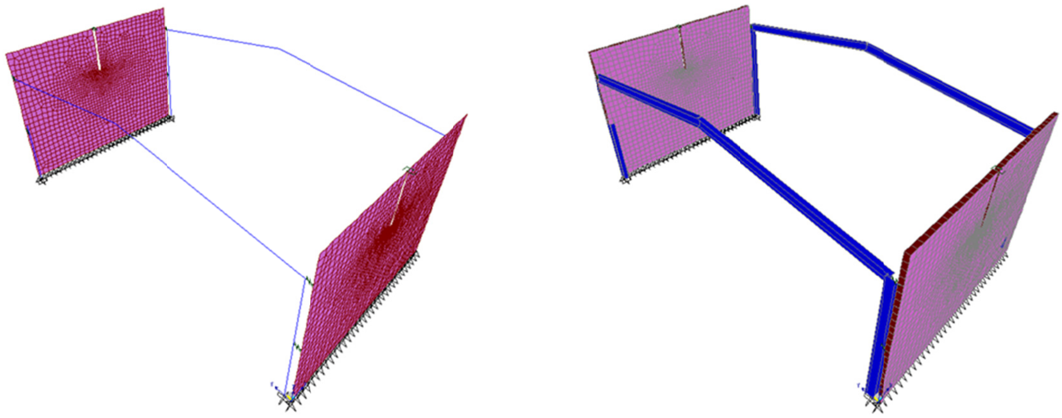

Figure 1. (a) Building model, (b) extruded view

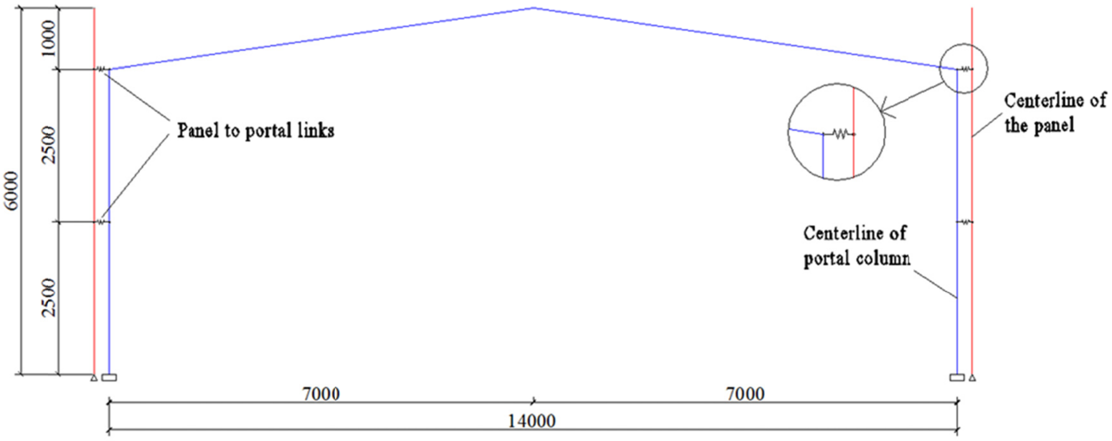

Figure 2. Structural section

The analysis of the structure was undertaken using SAP2000 V14.1.0.

To make allowance for the structure being continuous along its length the section properties of the portal frames were reduced by half.

The periods of vibration for both the portals frames with panels attached and precast panels only were calculated and are presented in Table 1.

Table 1. Calculated Periods

Full system. Portals with precast panels.	1.00 s
The period of the precast panels (excluding the action of the portal frames).	0.34 s

3.2 Panel material properties

The selected material properties are presented in Table 2.

Table 2. Material Properties

Concrete compressive strength, f'c.	30 MPa
Steel yield strength (increased by 1.1 for the effective yield strength).	550 MPa

For the purpose of this paper the reinforcing has been assumed to be seismic reinforcement to the requirements of AS/NZS 4671 Steel Reinforcing Materials and therefore able to achieve 10% uniform elongation. This is not consistent with the mesh used in the older buildings but has been selected for the analysis to both avoid including the ductility implications of using hard drawn wire mesh and to provide an indication of how current materials would be expected to perform.

3.3 **Panel ultimate strength**

A reinforcing content considered to be consistent with the observed reinforcing contents was selected. This was 663 mesh (As = 205mm^2/m width). It should be noted that this is area of steel is considerably less than the current requirements of NZS 3101 Concrete Structures Standard.

Table 3. Flexural Strength

M_{ult}	8.2(kN/m/m width)

3.4 **Panel cracking moment**

The cracking moment of the 150mm panel has been calculated and presented in Table 4 below. The calculation is based on equation 1 below from clause 6.8.3 of NZS3101 Concrete Structures Standard

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{1}$$

A range for the modulus of rupture (f_r) has been selected. The average value uses 0.6 $\sqrt{f'_c}$ while the upper bound has been calculated using $\sqrt{f'_c}$.

Table 4. Panel Cracking Moment

	f_r (MPa)	M_{cr} (kN/m/m width)
Average value.	3.29	12.33
Upper bound.	5.48	20.54

3.5 **Panel strength vs. cracking moment**

Due to the lightly reinforced nature of the panels the tensile strength of the concrete provides a bending strength that is 1.5 – 2.5 above the strength offered by the reinforced section. This would encourage the crack to be localised as a single crack. This would potentially limit the effective plastic hinge length and therefore focus the steel strain over a short length.

Therefore, the strain penetration is likely to be dominated by the debonded length of the reinforcing rather than by the distribution of flexural cracking.

4 **TIME HISTORY ANALYSIS**

4.1 **Description of selected ground motion (HVSC S26W)**

The ground motion recorded from Heathcote Valley Primary School (HVSC) during 22nd February 2011 Christchurch earthquake (GeoNet Strong Motion Database. 2011. GeoNet.) has been used for the nonlinear dynamic (time-history) analysis. This ground motion was selected as several portal frame warehouses in close proximity (Port Hills Road) had suffered significant panel damage. It was also a ground motion that was considered likely to significantly ‘test’ the panels.

From the two recorded orthogonal components, the component with the higher acceleration level HVSC S26W has been selected. It is assumed that the ground motion acts perpendicular on the building in the transverse direction to produce the most severe case. Figure 3**Error! Reference source not found.** shows a plot of ground motion from HVSC S26W.

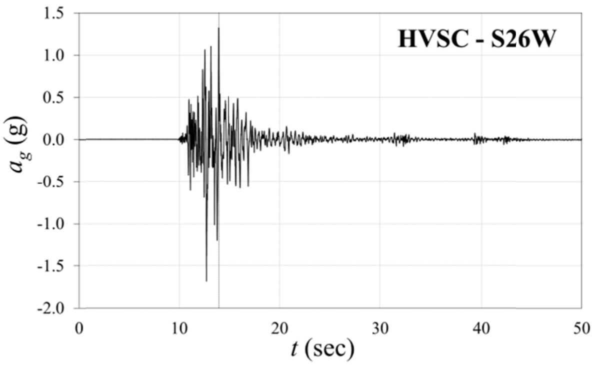

Figure 3. Ground motion record from HVSC S26W. The time of maximum response at t = 14.0s is shown dotted.

4.2 Modelling of portal frames

The portal frames were sized to be flexible and therefore achieve the maximum drift of 2.5% when designed to NZS 1170.5 Earthquake Actions. As the frames were expected to remain elastic during the analysis no non-linear hinges were added.

4.3 Modelling of panels

The panels are an elastic element and a mesh layout, as shown above in Figure 4, was generated by SAP. No allowance within the mesh has been made for post elastic non-linear response.

A nonlinear rotational spring at the top edge of the panel, were the yield line crack is expected to initiate, has been added and is described in section 4.5 below.

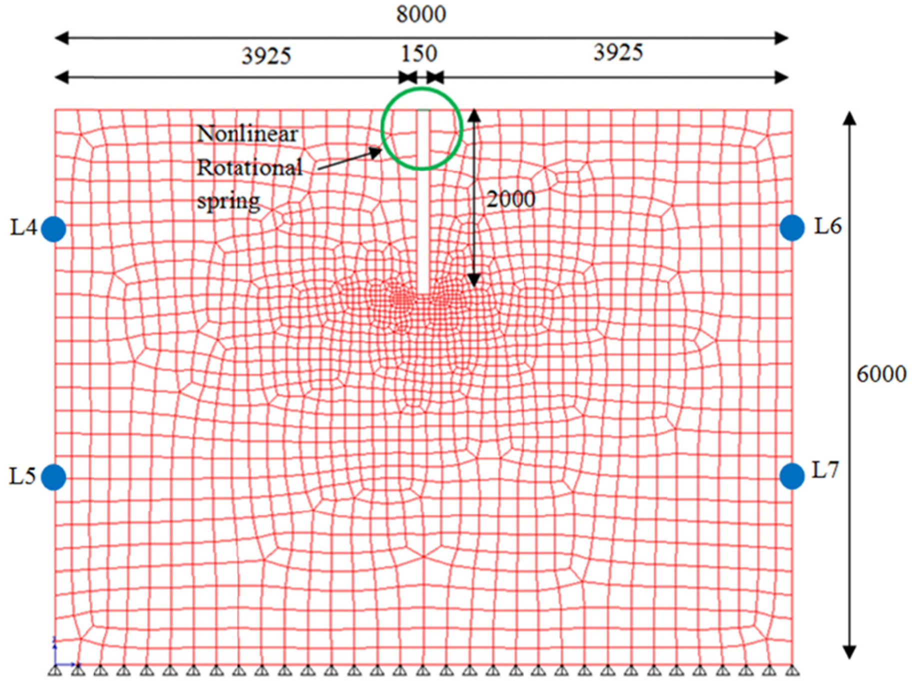

Figure 4. Non-linear shell element in SAP2000 for out-of- plane action

4.4 Description of link element connection panel to portal (L4-7)

The link connection between the panels and the portal frames is a linear element which is free and flexible to rotate and translate in each direction. The axial deformation of the link was neglected by inputting a large value for the axial stiffness. There are four of these links connecting each panel to the portals (L4-7). The locations of the links are shown in Figure 2 and Figure 4 above.

4.5 Description of nonlinear link element (rotational spring)

The post elastic response of the panels is modelled with a nonlinear link element which is a two node

nonlinear rotational spring located at top edge of the panel. As noted above, it is the only element in the panel that has nonlinear behaviour.

As shown in Figure 4 above, the link is located at the top end of the panel in the region where the highest stresses and bending moments during out-of-plane action can be expected. The link has a length of 150mm and an equivalent moment-rotation plot of a 2m height strip of the panel in out-of-plane loading.

The nonlinear spring has a Takeda hysteretic model (Takeda et al. 1970) which is commonly used to model plastic hinges in reinforced concrete beams. The model takes into account the strength degradation as shown in Figure 5a below.

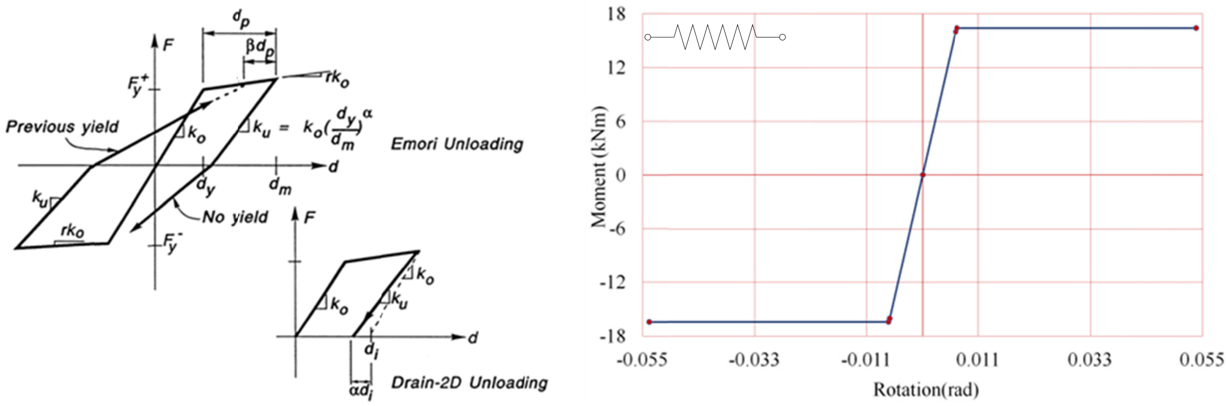

Figure 5. (a) Typical Takeda hysteresis after Carr (2005), (b) Moment-Rotation plot for the link element

The limiting steel strain has been selected as ε_{su} = 0.025. As the panel is lightly reinforced, concrete crushing does not occur and the steel strain limits the available curvature ductility.

5 OBSERVATIONS OF TIME HISTORY OUTPUTS

5.1 Panels attached to flexible portals

5.1.1 *Deflection Profile (Flexible Portals)*

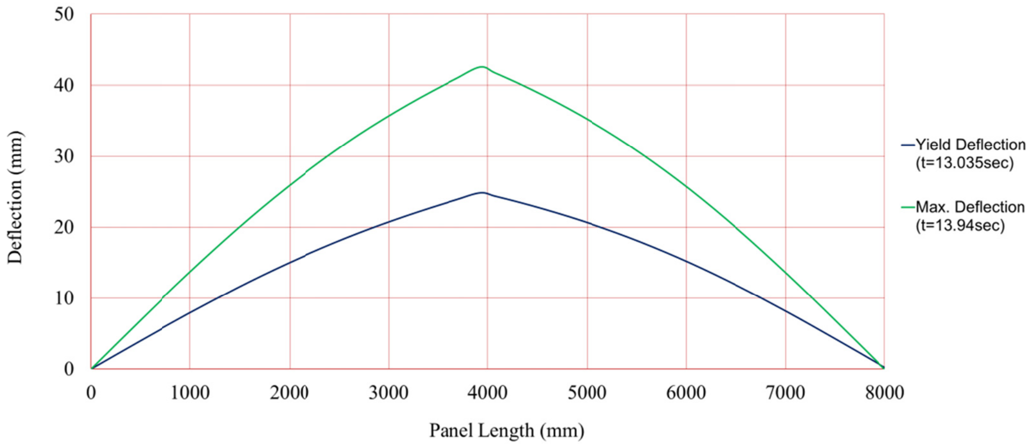

Figure 6. Yield and maximum deflection profiles for panels attached to flexible portals

Both profiles in Figure 6 show a hinge at the midspan of the panel. This would be expected for deflections beyond yield, with the rotation in the deflection profile being indicative of a plastic hinge. The hinge observed at yield is expected to be caused by the discontinuity between the stiffness of the cracked panel (being a function of the yield of the reinforcing steel) and the stiffness of the uncracked panel (being a function of the tensile strength of the concrete). The same yield profile is observed in Figure 8 for the rigidly attached panels.

5.1.2 *Plastic Hinge Rotations (Flexible Portals)*

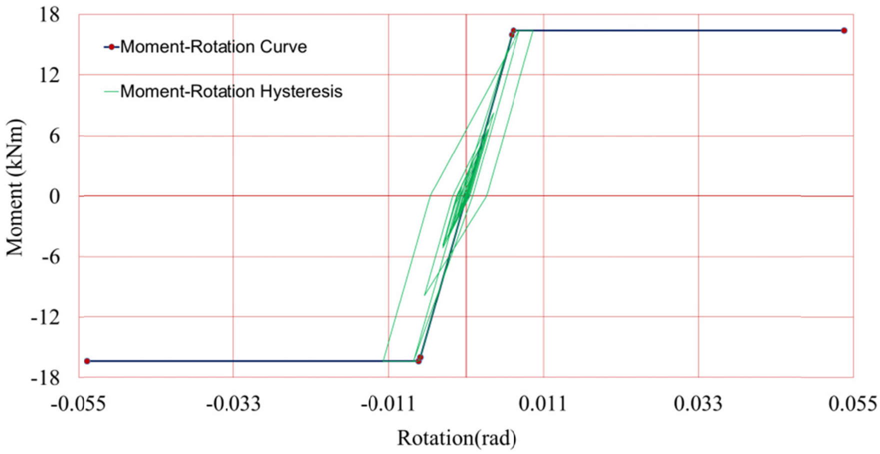

Figure 7. Moment – Rotation Hysteresis response of panels attached to flexible portals

Inspection of the hysteresis loops shows that there were 4 cycles that were expected to exceed the yield rotation capacity of the panel.

The maximum hinge rotations observed were +0.00952 and -0.01173.

5.2 Panels attached to Rigid Elements

5.2.1 *Deflection Profile (Rigid Attachment)*

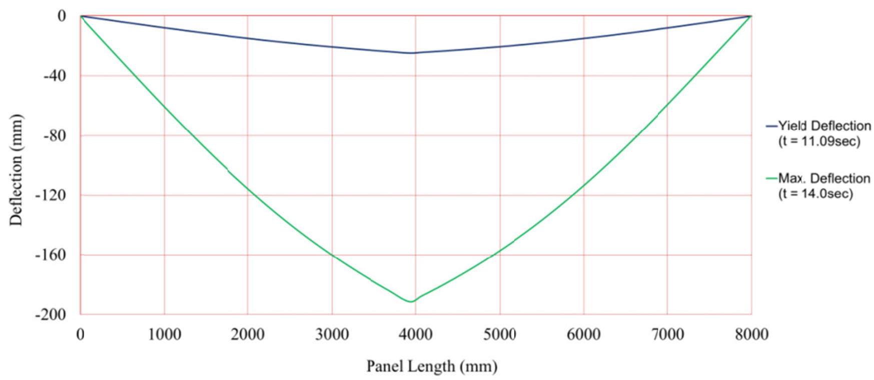

Figure 8. Yield and maximum deflection profiles for panels with rigid attachment.

The deflection profile of the rigidly attached panels indicates a high ductility demand on the panels.

5.2.2 *Plastic Hinge Rotations (Rigid Attachment)*

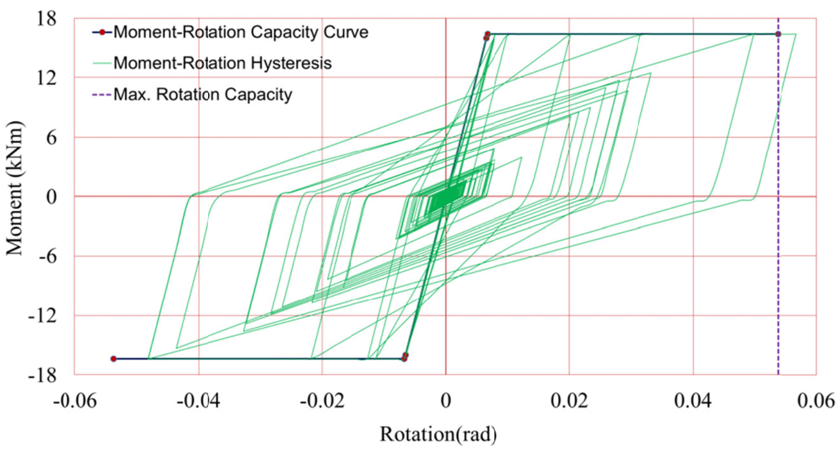

Figure 9. Moment – Rotation Hysteresis response of panels with rigid attachment

Inspection of the hysteresis loops shows that there were 12 cycles that were expected to exceed the yield rotation capacity of the panel.

The maximum hinge rotations observed were 0.05654 and -0.0482.

6 DISCUSSION AND COMMENTS

6.1 Flexible and Rigid Panel Support

It is apparent that the panels attached to flexible portal frames have been subjected to substantially smaller ductility demands that the same panels attached to rigid elements (such as return walls). It is expected that this behaviour is caused by the flexible, longer period, portals protecting the cladding panels from the ground shaking.

It is also possible that this behaviour could be peculiar to the near fault effects (high frequency vibrations) associated with the selected ground shaking records. Longer period ground shaking may place a higher demand on the panels. In time we anticipate that we will test the panels with longer period shaking which would be associated with earthquakes located at some distance from the effected building.

6.2 Reinforcing Strain Demand

The strain demand on the reinforcing, as a result of the cyclic loading, could be a cumulative extension in tension due to the lack of compression loading on the reinforcing. It is therefore considered appropriate to consider the cumulative strains developed during both positive and negative rotation.

The results, for the rigidly attached panel, have been plotted in Figure 10 below as the strain path. This shows the total strain of 0.04882 (4.9%). The 10% uniform elongation required in AS/NZS 4671(2001) is also plotted as a reference.

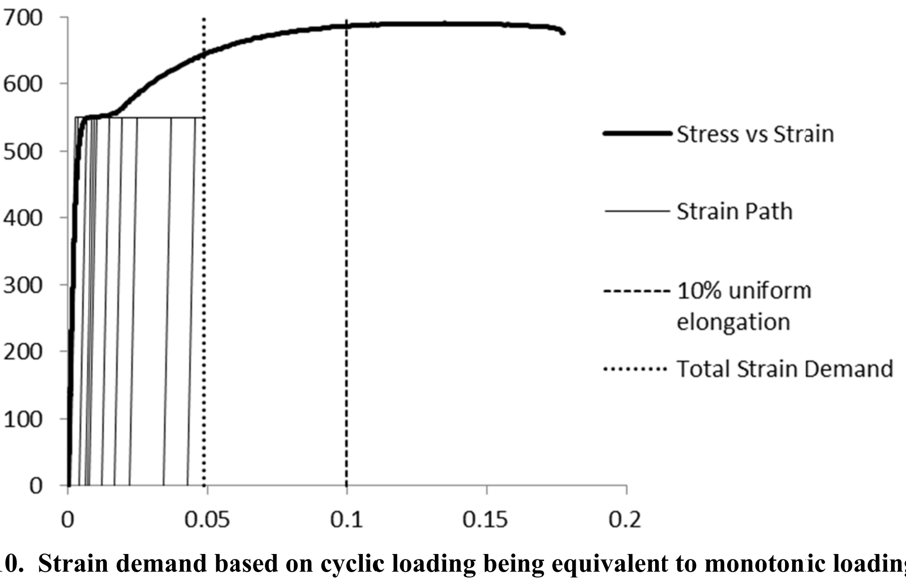

Figure 10. Strain demand based on cyclic loading being equivalent to monotonic loading, i.e. no compressive strains

6.3 Crack size

The maximum crack size is considered to have occurred during the final cycle of loading and is directly proportional to the strain.

The total strain at the end of the final plastic cycle is calculated to be equal to 0.04882. The strain length is the effective plastic hinge length (150mm). Therefore the plastic stretch in the bar is 0.04882 times 150 which is equal to 7.32mm. At the concrete surface the associated maximum crack size is therefore in the order of 14mm.

However, the residual crack size is likely to be considerably less than this due to the panel tending to return to its straight position (perhaps due in part to in-plane loading). From this analysis the plastic

extension of the reinforcing would be expected to visible as a full depth crack in the order of 7mm.

Flexural cracks of this size have not been observed in the field. Typical flexural cracks in the panels appear to have been limited to a maximum of 1-2mm. This may, in part, be due to the link element being modelled at the top edge of the panel which would reflect the maximum crack size at the top of the panel and may also be overestimating the displacements. Consideration would be given to the appropriate location of this link in further modelling.

6.4 Strain Penetration

A critical factor in the calculation of the strain demand on the reinforcing is the effective plastic hinge length. As noted previously, the wall sections tend to be lightly reinforced with the cracking moment significantly exceeding the moment capacity of the reinforced section. Therefore, strain penetration through distribution of flexural cracking may not occur. The primary remaining mechanism for the strain penetration is debonding of the reinforcing bar from the concrete. The extent of debonding would depend on whether the bars are deformed or round, and is considered to be difficult to calculate.

Therefore, the following plot in Figure 11 has been calculated to represent the sensitivity of the strain demand relative to the depth of strain penetration.

Figure 11 shows that if 10% strain is considered to be the point of reinforcing fracture then the critical effective plastic hinge lengths are 75mm for the rigidly attached panels and 15mm and for the panels attached to flexible portal frames.

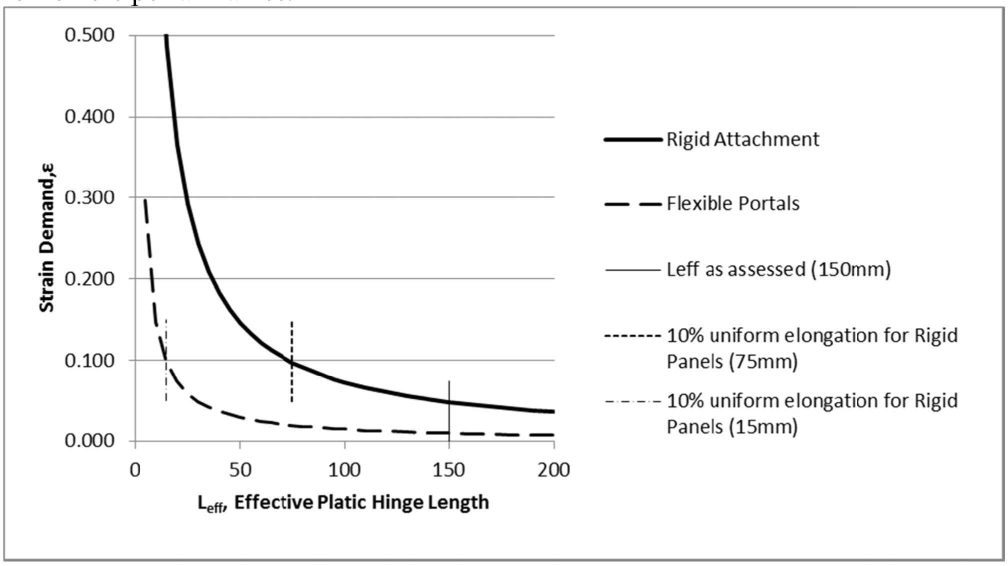

Figure 11. Strain Demand vs. Effective Plastic Hinge Length, L_{eff}

7 CONCLUSIONS

With consideration to out of plane actions, the earthquake of 22nd February 2011 developed sufficient ground shaking to yield the reinforcement within precast concrete panels.

The risk of post elastic strain damage to the reinforcing appears to be significantly affected by the flexibility of the supporting structure. For panels that are attached to flexible steel portal frame structures, this paper indicates that the flexible portal frames protect the panels from significant plastic demand, while panels that are rigidly supported, by other panels for instance, exhibit a much greater risk of being exposed to significant plastic strains.

A critical consideration in the strain calculations is the effective length of the plastic hinge, or the strain penetration along the reinforcing. Rotational hinge calculations in this paper are based on an effective hinge length of 150mm. There is a substantial risk that the effective hinge length could be much less than this and also the accurate calculation of an effective hinge length is difficult (especially for lightly reinforced sections). A sensitivity study was therefore carried out. The results indicate that for a hinge length of 75mm the panels rigidly supported would be at risk of reinforcing failure and for

panels connected to a flexible frame the critical hinge length would be much less at 15mm.

When reflecting on the results of this time history analysis and the correlation that may exist with actual observations it is notable that inspections:-

- Did not observe any significant post elastic behaviour of the steel portal frames (except for some damage to the concrete encasing often provided for fire rating purposes).
- Did not observe any fracture of panel reinforcing.
- Did observe significant flexural cracking of panels supported both by flexible frames and rigid return walls.
- Did observe that damage to panel connections was often more severe at the junctions with rigid return walls.

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