Displacement Based Assessment and Improvement of a Typical New Zealand Building by an Average Engineer

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ABSTRACT: The Radiohouse building is a two storey steel frame structure. The analysis and design of the strengthening of the building was completed by an average practicing Engineer using a displacement based method. A push-over process was completed using Microstran, and models of post-yield behaviour were developed. Interventions were designed, and the resultant building capacity determined using a displacement based process. A discussion on the inappropriateness of using an 'adopt a ductility' approach for the building follows.

1 THE BUILDING

1.1 Main Structure

The Radiohouse building is a two storey steel frame structure in the town centre of Masterton. Masterton is a typical small town in New Zealand, located in the highly seismic Wairarapa region.

The Radiohouse building was designed in 1959 by a structural engineer. Its overall plan dimensions are 33.3 metres x 12.8 metres. The eight steel frames form 7 bays of 15'10", or approximately 4.8 metres, with intermediate supporting columns through the centre of the building.

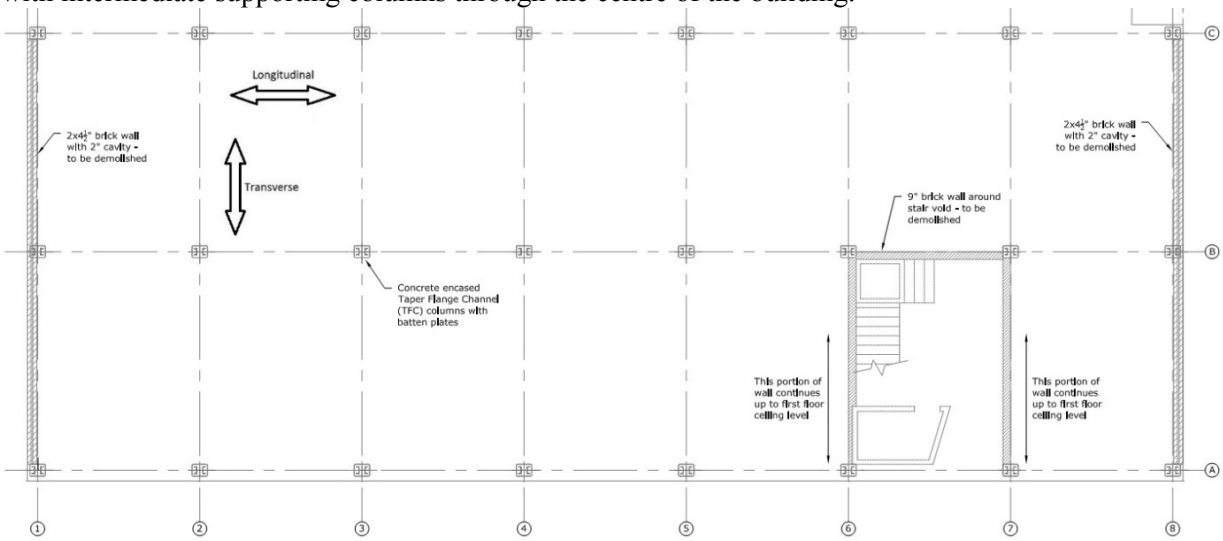


Figure 1 – Existing floor plan

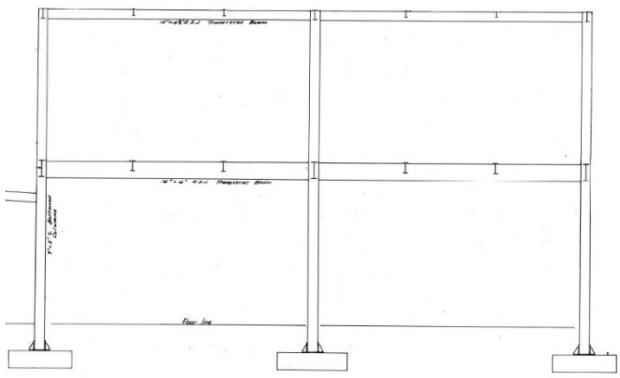


Figure 2 – Typical Frame

The two end walls of the building are formed by two wythes of 4½" brickwork separated by a 2" cavity. The internal wythe is set into the line of the steel frame and is secured at first floor and eaves level by concrete bands. The outer wythe is secured to the internal face by connection to these concrete bands at first floor and eaves level.

The internal stairway is formed with cast in place concrete. This staircase is secured to the main frame with reinforcing bars welded to the main steel beams. There is no separation between the levels. The stair well is surrounded with 9" thick unreinforced masonry walls spanning from the ground floor to the underside of the first floor.

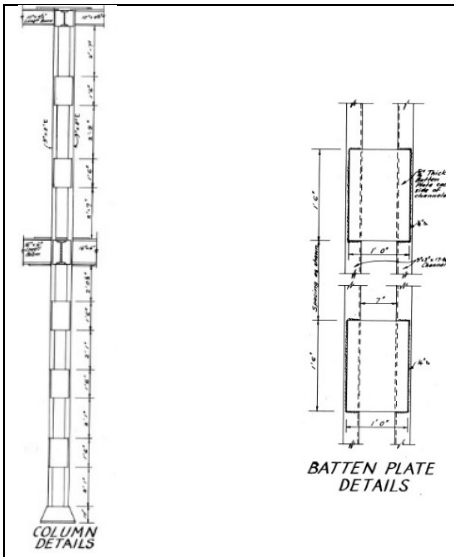


Figure 3a – Column Details

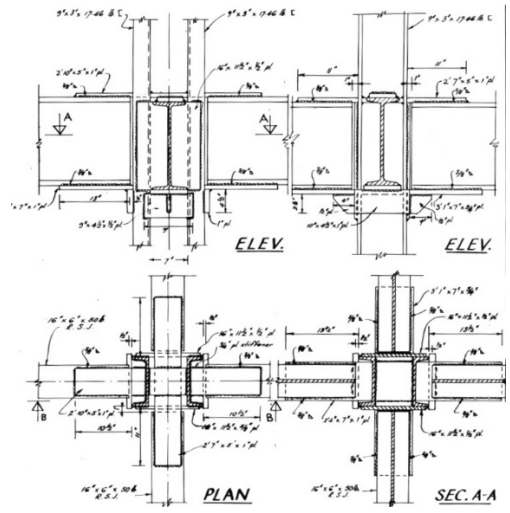


Figure 3b – Beam – Column Joint detail

The steel columns are back-to-back 9" battened taper flange channel (TFC) columns. The primary beams (16'x6"x50 lb RSJs) span in both directions along the main grid lines. The beams and columns are joined together by a complicated arrangement of welded plates.

The suspended first floor is formed with steel tray deck which is welded to supporting RSJs and filled with a light-weight unreinforced pumice concrete topping.

1.2 Lateral Force Resisting System

In the Longitudinal direction, the battened columns and primary RSJ beams attempt to form 'frames'. The orientation of the battened columns meant the TFCs bend in their weak direction, while simultaneously forming a large tension/compression couple.

In the transverse direction, the two TFC column members start acting as 'frames' in conjunction with the primary RSJ beams.

The frames appear to have been primarily designed as a gravity structure. As such, detailing at beam-column joints accommodates little horizontal joint shear. This explains the lack of web doubler plates in the joint region, similar to a lack of joint reinforcing characteristic of a reinforced concrete moment frame of the same period.

2 THE ASSESSMENT PROCESS

2.1 Available Tools

The firm completing this assessment is a typical small consultancy with limited software tools. The analysis package available is Microstran. This appears to be typical of most design offices in regional New Zealand, who would not routinely complete more sophisticated analysis such as Inelastic Time History Methods.

2.2 Document / Structure Review

In depth site inspections were completed to confirm that the structure was constructed in accordance with the available plans.

Due to the atypical system, some research was completed on the behaviour of battened columns under seismic cyclic loads. Limited information on post-yield behaviour was available, however a useful reference was found in documented laboratory testing completed on battened parallel flanged channel columns (Sahoo 2004).

2.3 Review of Existing Structural System

Conventional assessment of existing buildings usually focuses on force based assessments. A force based approach generally requires the designer to judge the available ductility of the building. The configuration of this building made this difficult for the following reasons;

- Longitudinal direction – Modelling the longitudinal system showed that the battened columns would form a large tension / compression couple, combined with weak axis bending of the TFC. The existing 410UB beams provide more capacity than the battened TFC columns.

  Evaluating the available ductility from this non-traditional system was difficult. 'Adopting' a ductility required an assumption that each component forming the column could accommodate the post-elastic mechanism.

- Transverse direction – The performance in the transverse direction is dependent on the capacity of the beam-column joint.

  The beam-column joint required a detailed review in both the longitudinal and transverse direction. In the transverse direction alone, this included the connection between the beam top flange and the TFC, the beam bottom flange and the TFC, the beam web to the TFC and the TFC web panel through the joint.

  Preliminary analysis showed that the central beam-column joint would fail in web panel shear (highlighted in the joint elevation below) at a reasonably low level of applied force. This mechanism could potentially lead to loss of gravity support, hence a ductility of 1 was adopted when analysing the existing structure.

  Allowing for improving the web panel, the joint was reviewed further. One of the joint connections reviewed is illustrated below, namely the top flange of the beam to the TFC column in the transverse direction. Each individual component was checked for its capacity, with the critical element of this specific joint being the weld between the two perpendicular plates.

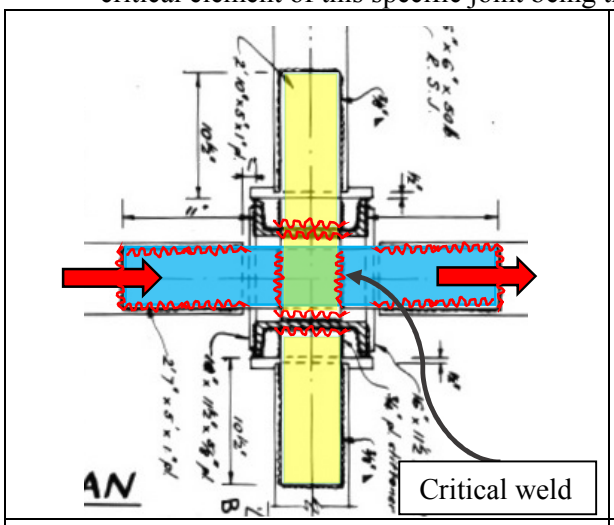


Figure 4a – Schematic of top of beam/column joint in transverse direction

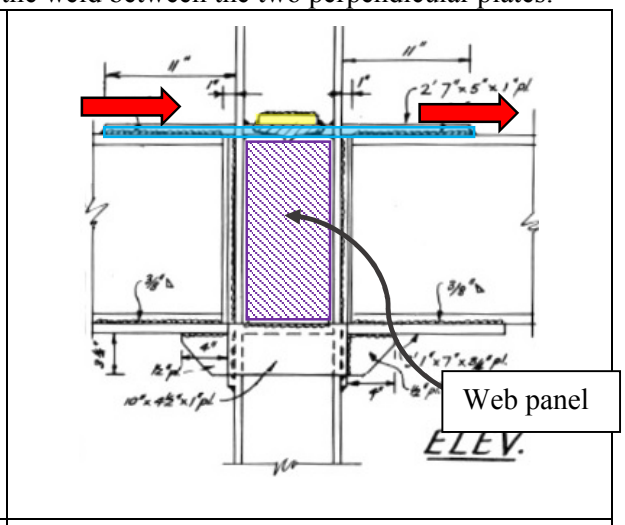


Figure 4b – Elevation of beam/column joint in transverse direction

Initial attempts to review the building using a force based approach kept circling back to the question of 'what ductility is appropriate?'. Seeking further guidance, a question was asked which helped focus the assessment, which was

'If you push it, what will break first?'

2.4 Capacity of Sub-Assemblages

To answer the above question and progress the analysis, the structure was reduced to its components, and the capacity of each component was calculated. Some of the components analysed were;

Longitudinal

- Weld securing the batten plates to the TFC columns
- Batten plate in shear and bending
- TFC in weak axis bending combined with axial loads
- RSJ primary beam bending capacity
- Base fixity at foundations
- Beam/column joint capacity

Transverse

- TFC strong axis bending combined with axial loads
- RSJ primary beam bending capacity
- Weld length and size at beam-column joint
- Plate length and size at beam-column joint
- Web shear capacity at beam-column joint

The building was then modelled in Microstran. An initial model was completed with the battened TFC columns modelled as one single element. The model was then developed to include individual elements forming the columns. Modelling the battened column elements required a judgement as to the appropriate configuration.

Modelling was initially completed in three-dimensions, however this was further simplified to two-dimensions for the analysis. Calibrating and reviewing the appropriateness of the model at the initial stages was important.

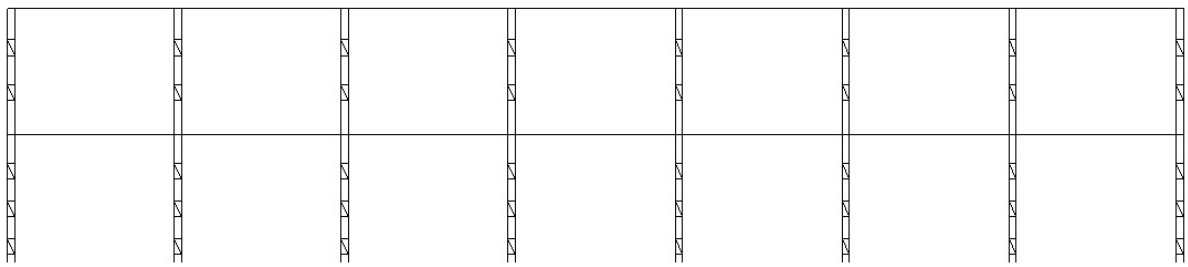


Figure 5 – Longitudinal Model

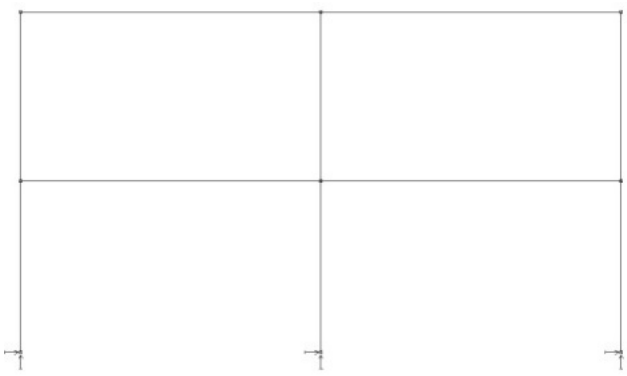


Figure 6 – Transverse Model

2.5 Capacity of System (completing a push-over analysis)

In each orthogonal direction, a push-over analysis was completed. The basic steps to this were as follows;

1. Calculate the seismic weight and lateral force distribution, and model the system in Microstran.
2. In Microstran, set up a load case for the permanent loads (G) and a 1g lateral force (E). Also create a combination case for G+E. Apply a percentage of the 1g load for the push-over. Vary this percentage to suit the analysis.
3. Apply the lateral load in increments. Start with a force low enough to ensure none of the sub-assemblages have reached yield. Review the analysis results. Does the displacement pattern make sense? Are the forces approximately what you would expect?
4. Increase the force in small increments. After each analysis, stop and review the results. Note down the displacements. Continue checking that no failure has occurred.
5. Continue increasing the applied force until the first sub-assemblage 'yields'. In the longitudinal direction, this was yielding of the TFC in the weak direction as the combined axial force and out-of-plane bending moment increased. In the transverse direction, this was web shear failure of the central beam-column joint. Note the displacement at this point.
6. At the point of yield, the 'failure' needs to be modelled. In the transverse direction, the analysis was halted at this point, as the failure mechanism was considered to have been reached. In the longitudinal direction, post-yield displacements were needed. In Microstran, an additional load case was created titled 'yield forces'. At the point of yield, a 'pin' was introduced into the member, modelling the yield and ensuring the member could not accept more moment. In the 'yield forces' load case, a moment is then applied equal to the moment that was present in the member at yield. Where this occurs in the length of a member, provide equal and opposite moments either side of the pin of equivalent magnitude to the yielding moment. Note that this assumes that flexural capacity of the element is maintained without strength degradation.
7. Increase the seismic forces and continue with the push-over. Note that the deflections will now be non-linear. Slightly increase the forces, run the analysis and note the displacements at each analysis. At any point of yield, continue amending the model to accommodate the yield.
8. Continue as above until a collapse mechanism is determined to have formed. Check that rotation demand is met as required at remaining joints. In the longitudinal direction, the final displacement of the system occurs at the culmination of many hinges forming in the TFC columns.
9. Using the above information, plot the force vs displacement. Note the equivalent linear ductility can be determined from this plot, if desired. In the graphs below, Force has been plotted as a ratio of the total weight, ie G. Displacement is the displacement of the effective mass.
10. Using the provided equations, determine the displacement spectra for the site and calculate the displacement demand. This requires an estimate of the equivalent viscous damping.
11. Determine %NBS

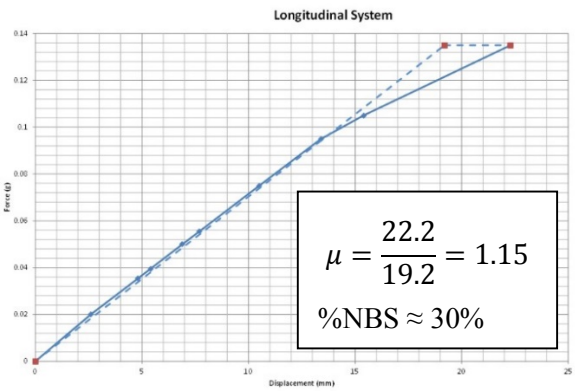


Figure 7 – Longitudinal Force vs Displacement

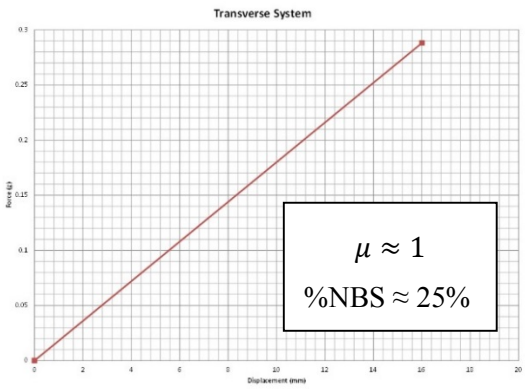


Figure 8 – Transverse Force vs Displacement

2.6 **Design of the Interventions**

Once the above assessment of existing is complete, the interventions can then be designed. For each intervention, repeat the above steps and determine the force vs displacement. Choosing the type of intervention, and then calibrating the response of the interventions is an important and sometimes time consuming step in the design process. The interventions selected for the Radiohouse building were as follows;

Longitudinal

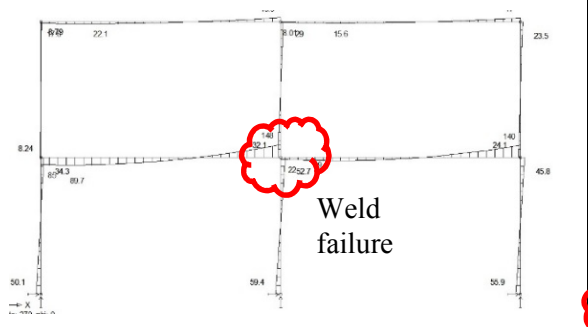
- Three new eccentrically braced 'K' frames with new ground beams. These were chosen to introduce ductility into the building. K-frames are initially very stiff, and their yield mechanism can be calibrated to protect the existing structural system. The new K-frames have been designed to yield at reasonably low displacement. The existing frames are still elastic at this point, with a first yield not expected until displacement reaches approximately three times the displacement when the K-frame yielded. The final design was achieved through an iterative process, calibrating the stiffness of the K-frames.

Transverse

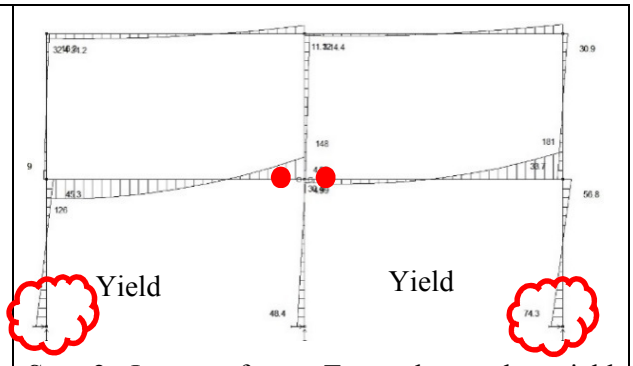
- Remediate the central and outside beam column joints by installing new web-doubler plates to the TFC. This changes the joint failure from web-panel shear, to forcing a failure in the connection between the two perpendicular plates joining the top flange of the RSJ beams to the TFCs. While this in itself is not a ductile yield, it allows the central TFCs to act as a propped cantilever from the ground floor, supporting the first floor and roof. An additional check was also completed to ensure gravity support for the first floor was maintained after this connection fails.
- At six of the eight 'frames', a single new 530UB82 column, with a moment joint to the existing 410UB primary beams. This turns the frames into a four-leg system, with the column size attracting a reasonable proportion of the load.
- The architect specified five internal inter-tenancy walls. These walls were designed and detailed as ductile plywood shear walls.

2.6.1 *Transverse Direction – Push-over Process*

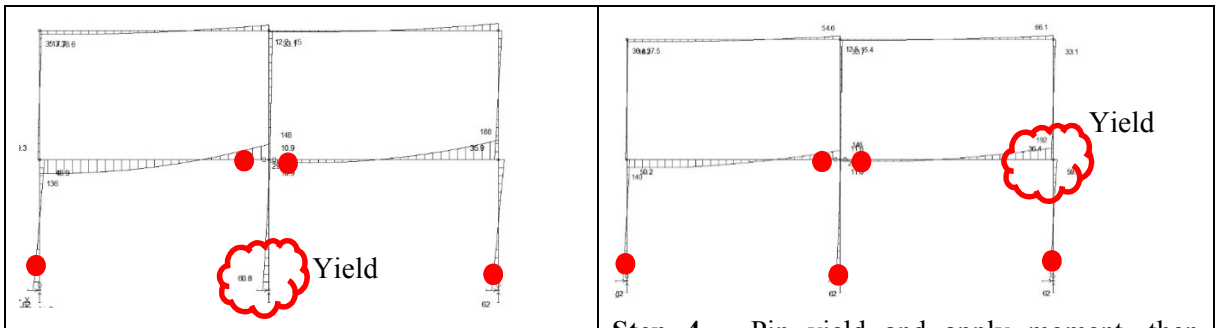
For each of the above lateral load resisting systems, a push-over analysis was completed. In the transverse direction, this meant there were three models to consider, shown below.



Step 1 – Apply loads until yield - central joints connection fails – TFC acts as propped cantilever from ground floor. Introduce pins at this location. Δ = 27mm, V=0.065g



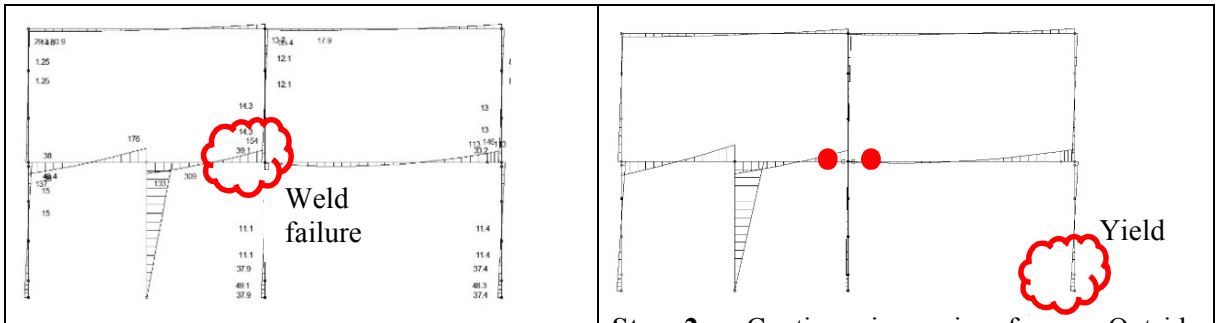
Step 2 –Increase force. Two columns then yield as moment re-distributes. Δ = 46mm, V=0.065g



Step 3 – Pin yield and apply moment, then continue increasing force. Central column yields at base. Δ = 47mm,V=0.066g

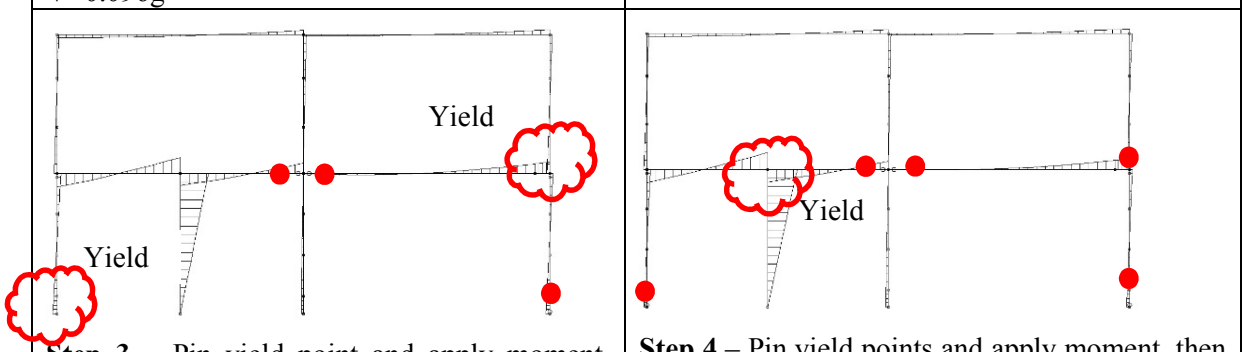
Step 4 – Pin yield and apply moment, then continue increasing force. Top of central column yields. Δ = 50mm, V=0.067g

Figure 9 – Transverse Direction Push-Over – with new web doubler plates only



Step 1 – Apply loads until yield - central joints connection fails. Introduce pins at this location, apply yield moments as a force. Δ = 26mm, V=0.096g

Step 2 – Continue increasing force. Outside compression columns yields at base. Δ = 32mm, V=0.097g



Step 3 – Pin yield point and apply moment, continue increasing force. Yield of two columns. Δ = 33mm, V=0.10g

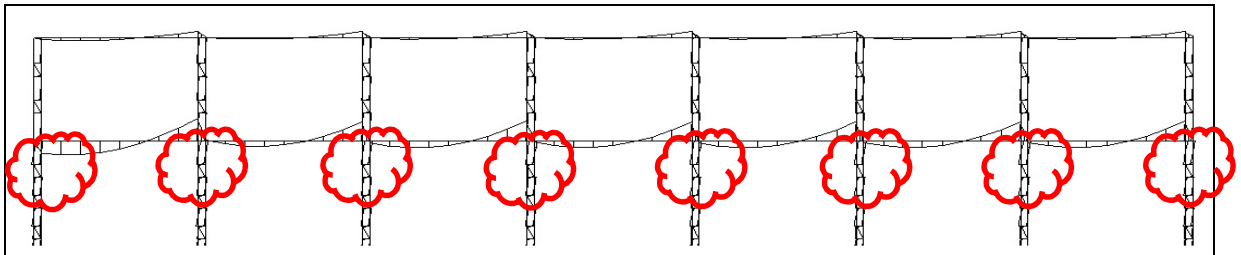
Step 4 – Pin yield points and apply moment, then continue increasing force. Top of new 530UB82 yields. Δ = 52mm, V=0.123g

Figure 10 – Transverse Direction Push-Over – with new 530UB82

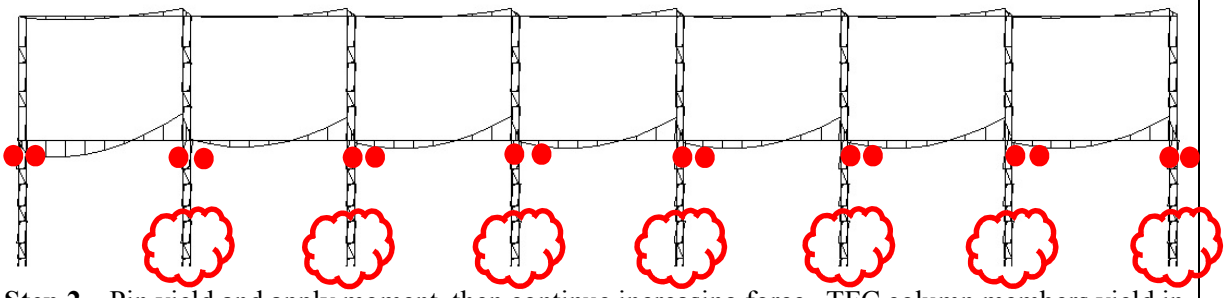
Force vs displacement information for the new plywood shear walls was obtained using equations from NZS3603 clause 5.2.5. The displacement at points of average and maximum nail slip were back calculated using the forces noted in NZS3606 clause 4.2.2.3.

2.6.2 *Longitudinal Direction – Push-over Process*

A Microstran model was prepared with an equivalent static load distribution. A load case of G+E was introduced, with the seismic coefficient being incrementally increased.



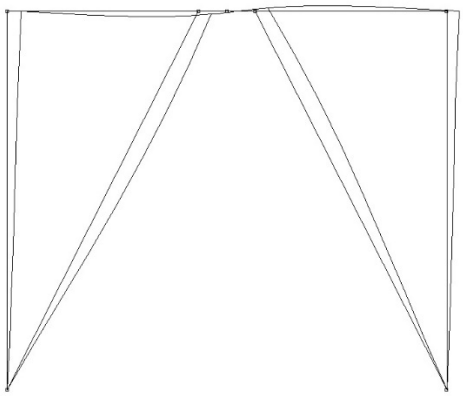
Step 1 – Apply load until yield – TFC columns yield with combined out-of-plane moment plus axial compression. Pin yield point and apply moment, continue increasing force. Δ=13mm, V=0.095g



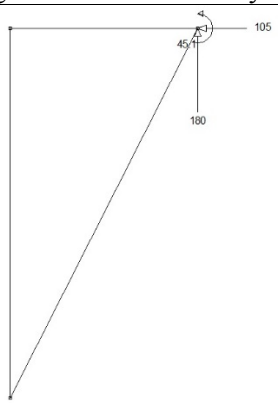
Step 2 – Pin yield and apply moment, then continue increasing force. TFC column members yield in multiple locations. Δ = 22mm, V=0.135g

Figure 11 - Longitudinal Direction Push-Over – Existing Frames

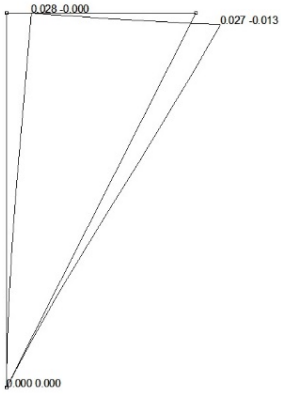
The above steps were then repeated modelling the new eccentrically braced K-frames.



Step 1 – Apply incremental loads until active link yields. Δ = 5mm, Force=0.0845g



Step 2 – Model post-yield behaviour. We applied the yield forces in the link as loads, and amended the model as shown.



Step 3 – Using NZS3404, calculate deflections at link maximum rotation. Using the model, determine force at this displacement. Δ=27mm, V=0.0927g

Figure 12 - Longitudinal Direction Push-Over – K-Frames

2.7 Determine the System Capacity (ie %NBS once 'strengthened')

To determine what the %NBS is once the interventions are completed, we need to plot a 'System' force vs displacement. To complete this, we need to sum the force required for each intervention to move a certain displacement. This is relatively simple, however it does mean each intervention must have a matching displacement.

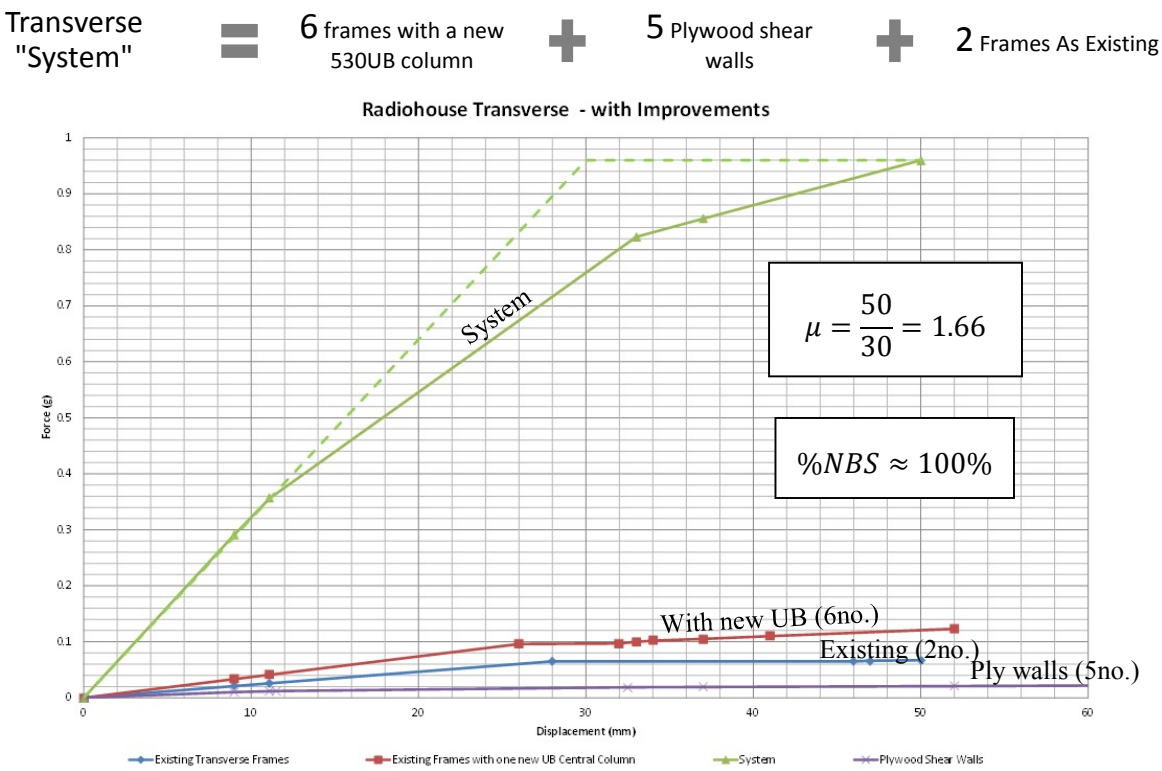


Figure 13 - Transverse Direction Force vs Displacement

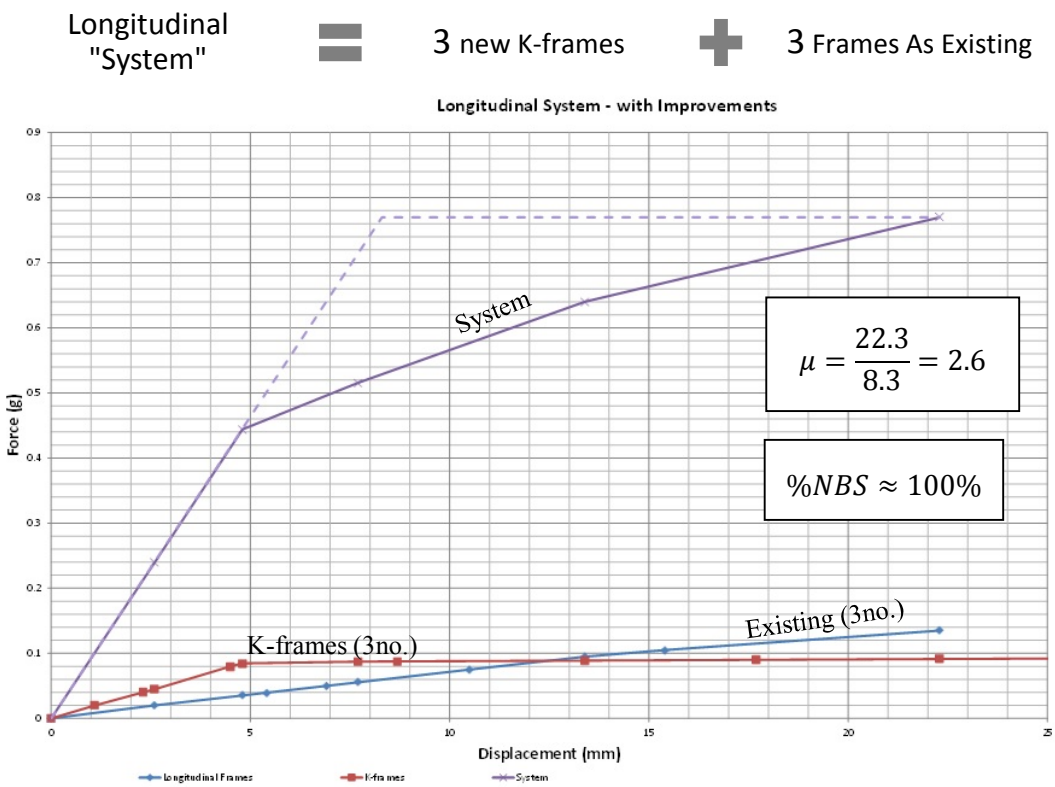


Figure 14 – Longitudinal Direction Force vs Displacement

Using the equations provided in the NZSEE Recommendations, determine the displacement spectra for the site and calculate the displacement demand. From this determine the %NBS.

The above requires the adoption of a value for Equivalent Viscous Damping, and a Structural

Performance Factor. The Structural performance factor was calculated using Clause 4.4.2 in NZS1170.5. The actual system ductility was used when determining this factor. However a conservative approach was adopted by the author to limit Sp to no less than 0.85. Equivalent viscous damping of 8% was adopted for the system. However a sensitivity analysis was also completed to review the system results at a damping value of 5%.

3 DISCUSSION

3.1 Complications / Barriers to use

Summing the forces at a displacement requires that the force at a common displacement is known for all interventions. This is relatively easy to interpolate if in the linear range, but can require some trial and error in Microstran if in the non-linear.

Most practicing engineers are not used to displacement based procedures, having more experience and familiarity with the force based equations. This is simplified by a force based spectra being provided in the code. Determining the displacement spectra and using the displacement based equations requires practice and developing an understanding of a new process.

3.2 Benefits

The displacement process gives a much more accurate feel for the response of a building. Any assessment process requires particular attention to the details of the structure. The displacement process gives the actual ductility available, as opposed to using an 'adopt a ductility' approach.

All elements of a structure, from primary beams down to the smallest beam/column joint must be reviewed for their failure mechanism.

The displacement approach gives an exact force that the intervention has to be designed for. This is critical to make sure that a design is economical. Design can be completed knowing the forces that the intervention must accommodate, which means efficient sizes of foundations and connections is possible. The displacement process maximises the existing structure load resistance.

3.3 Secondary Elements

By focusing on displacement, the displacement demands placed on other structural elements becomes obvious. For this building, the two end walls of the building are brickwork, secured to the primary steel beams. In addition, the internal stairway is formed with cast in place concrete, and the stair well is surrounded with 9" thick unreinforced masonry walls.

If these stiffer elements remain connected to the main frames, they will attract loads. These elements need to be altered to accommodate the drift demands, either by being separated from the main frames and independently secured, or removed.

The final design involved removal of the brickwork, with a light timber framed replacement, and seismic separation of the stairs.

3.4 Lower Bound Checks

The modelling of such a structure requires assumptions and judgement. A lower bound check on the model was completed in each direction. This lower bound check reviewed the %NBS achieved assuming no ductility was available in the existing structural systems, ie that the 'System' reaches capacity as soon as the existing structure has a yield point. This check confirmed that with the interventions proposed, a compliance level of above 80%NBS (New Building Standard) was achieved in both directions. This means that there is no strong reliance on post-yield mechanisms of the existing structural system.

Mindful of the uncertainties in modelling such a structure, the author also adopted conservative values for structural performance factors and damping.

No allowance was made for stiffening provided by concrete encasement to the ground floor columns.

4 SUMMARY

The displacement based assessment and design of interventions requires a different approach than the usual force based methods. For the design of new structures, "adopting a ductility (μ)" is an acceptable premise. The designer adopts a μ, then details the structure in accordance with the code to ensure that the chosen ductility can, in theory, be achieved.

For assessing existing structures however, using the displacement based approach provides the available ductility. This removes any assumption of ductility, which may or may not be available.

The displacement process allows the designer to maximise the use of existing structural systems, and design interventions for a measurable demand.

For the displacement process to be adopted by more practicing engineers, it would be beneficial to have available some fully worked examples for typical low-rise New Zealand buildings.

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ACKNOLWLEDGEMENTS:

The author wishes to thank the Land Trust Masterton for allowing the technical content to be published.

Many thanks to Gareth Alley from Dunning Thornton Consultants and Stefan Lanser from LGE Consulting for reviewing and providing comment on the paper.