Should the chimney have rocked?

R.D. Sharpe & K.E Grinlinton

Beca Carter Hollings & Ferner, Wellington, New Zealand.

.

2013 NZSEE
Conference

ABSTRACT: In the late 1970s, NZSEE stalwarts John Hollings and Ivan Skinner collaborated to design a chimney structure at Christchurch Airport that would rock/step in a major earthquake. Moreover, steel energy absorbers were installed which could yield in bending. This was documented in the NZSEE Bulletin Vol.16, No. 2 1983. The recent Darfield earthquake of 4th September was recorded at the Christchurch Aero Club a few hundred metres from the chimney. Inspections of the chimney two days after the earthquake did not reveal any signs of rocking. The authors are undertaking non-linear time-history analyses of the chimney subjected to the local record. They will report on whether the chimney should have rocked or not. This type of analysis is an important contribution to the apparent discrepancy of the observed behaviour of modern buildings in Christchurch and the shaking records so far published.

1 INTRODUCTION

In the late 1970s, John Hollings and Ivan Skinner designed a unique chimney structure at Christchurch Airport that would rock/step in a major earthquake. The chimney was constructed in 1977 and its location is shown in Figure 1.

Figure 1. Chimney and Canterbury Aero Club location where accelerations were recorded (Source: Google Maps)

The chimney is a cruciform shape approximately 35 m high, 7 m wide at ground level, and is constructed of reinforced concrete. Steel plate dampers are connected at the bottom of one leg in each direction and are designed to yield once the chimney begins to rock.

The design philosophy for the chimney was tested during the magnitude 7.1 Darfield earthquake on 4th September 2010. Following the earthquake, inspections of the dampers showed no visible damage or sign of uplift. A photo of the chimney after the earthquake is shown in Figure 2.

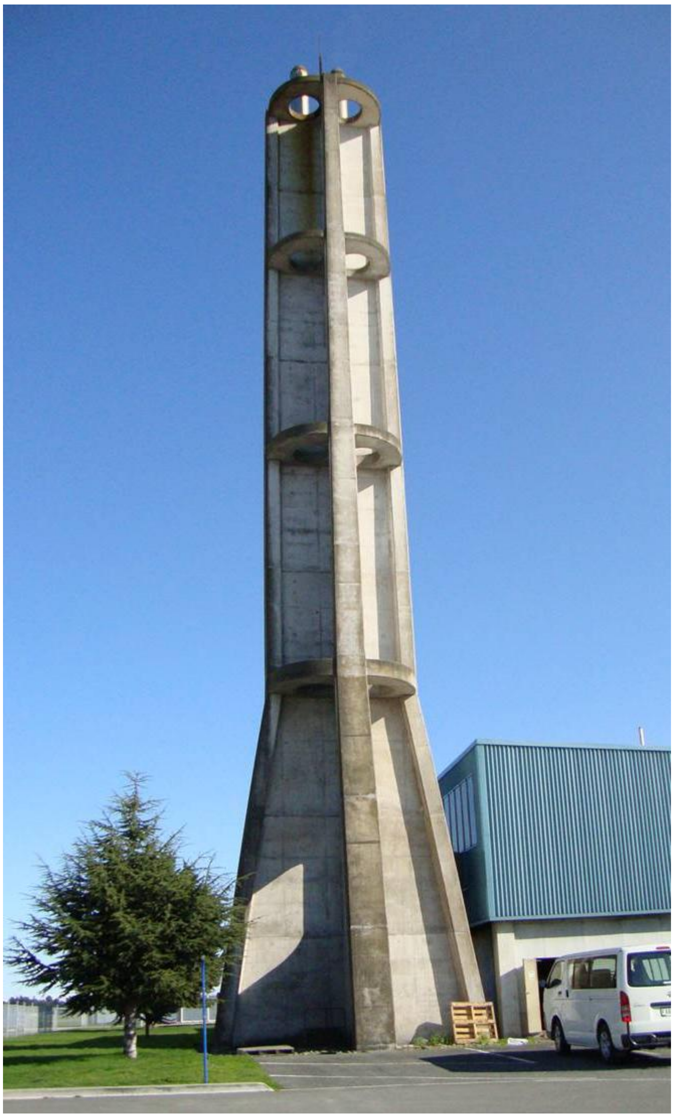

Figure 2. Chimney following 4th September 2010 earthquake

It appears the chimney was designed to rock/step at around 0.15 g and at this stage the steel plate dampers would also come in to effect, bringing the total critical damping to 5 %. The circular diaphragm members at each level were also designed to form plastic hinges, however these also appeared undamaged. No other damage to the chimney was visible.

The Canterbury Aero Club is located relatively close to the chimney, and acceleration recordings taken at this site during the Darfield earthquake will be used in the time-history analysis of the chimney.

The aim of this paper is to determine if time-history analyses can produce results consistent with the observations of no rocking/uplift made onsite.

2 MODEL

SAP2000 version 14.2.3 was used for all modelling and analysis. The model was constructed using wall sections for the main cruciform of the chimney and frame sections for the boundary elements.

The wall sections were not meshed as the internal stresses and deformations were not considered important for this study. Mass from these wall sections was therefore lumped at the four corner nodes. The width of the boundary elements are tapered from the bottom to the top so an average section size was assumed over each level. Diaphragm constraints were assigned at each level although the circular diaphragm members themselves were not modelled. No stiffness modifiers were used. Figure 3 shows dimensions and element sizes for the model.

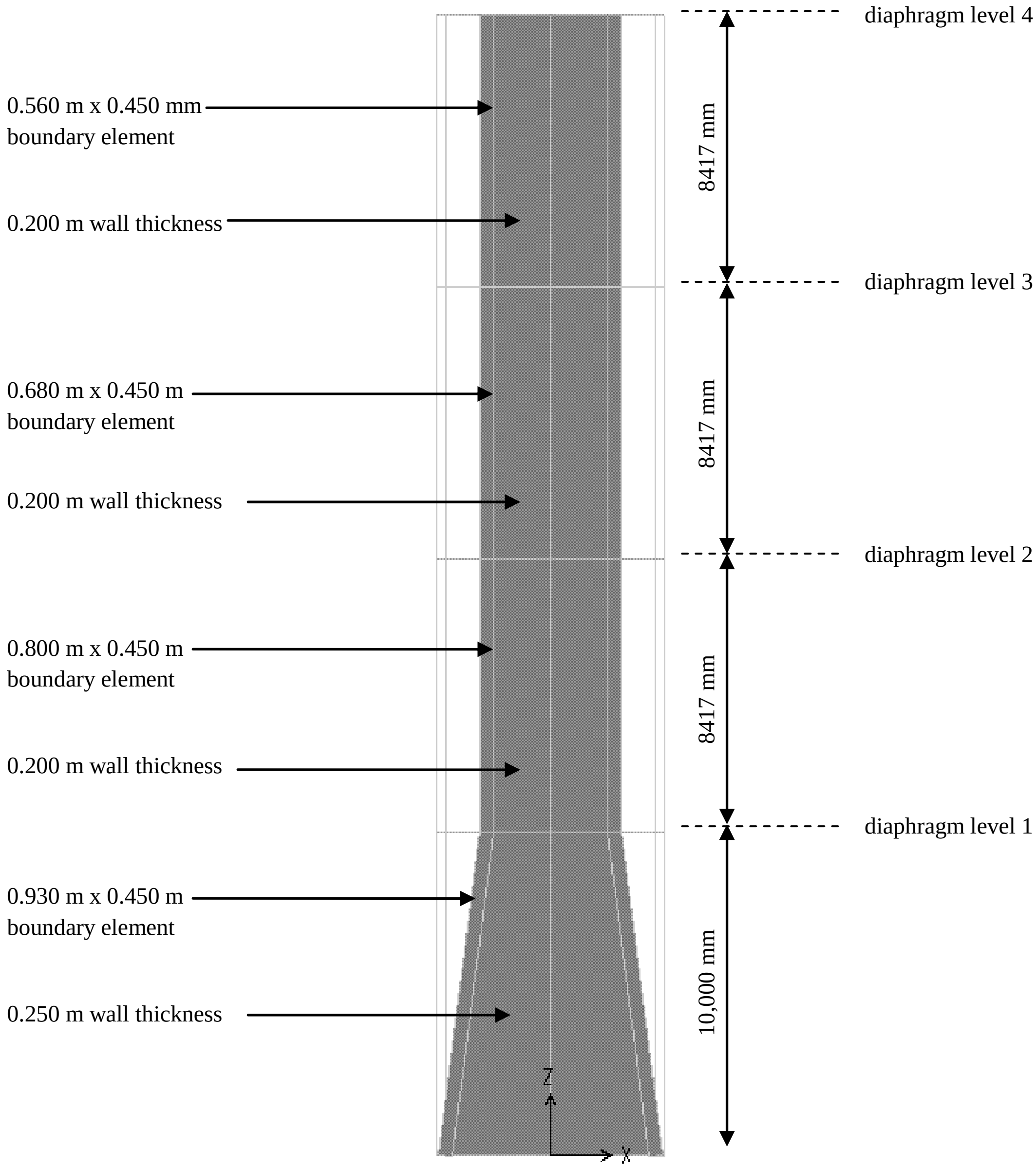

Figure 3. Chimney model elevation and dimensions

Mass for the model was assigned through self-weight of the elements, lumped at nodes. The assembled joint mass as well as the total vertical reactions under dead-weight were compared with a hand calculated mass and were within 2.5 %. The total weight of the chimney is 3040 kN. The mass of the two flues has not been considered in this model.

The steel dampers (one in each x and y directions) were modelled by a non-prismatic steel section as a cantilever from the central set of bolts where yielding is expected to originate. Figure 4 shows the actual damper and the SAP representation modelled.

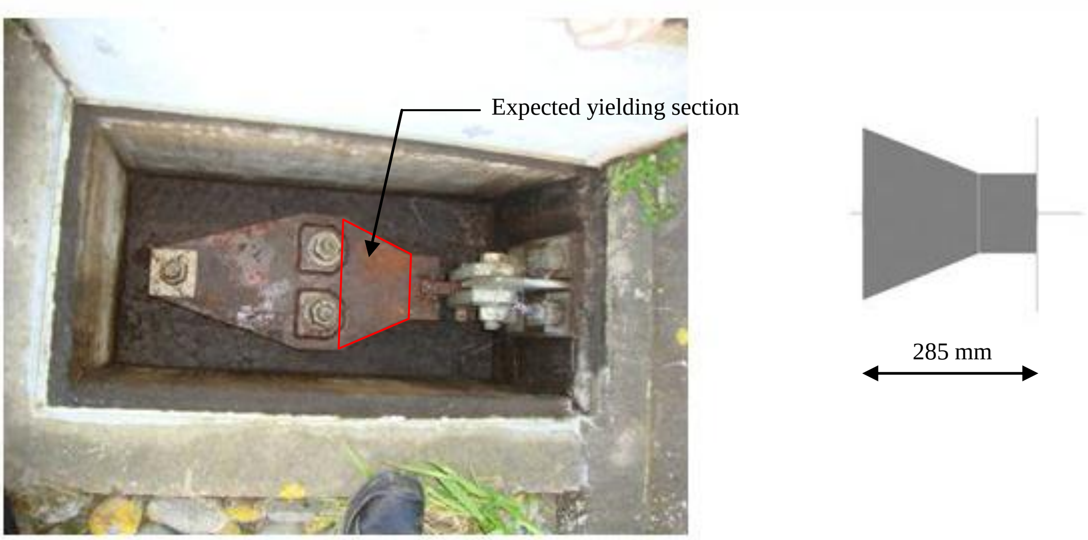

Figure 4. Photo of damper following the 4 September 2010 earthquake & representative SAP element

Each leg of the chimney (boundary element) is supported on a small lead bearing sitting on a concrete foundation pad of dimensions 2.3 m x 2.3 m area by 0.5 m height. As no geotechnical information was available for the site, a soil type of dense granular alluvium was assumed. A single vertical spring was modelled under each leg with a compression stiffness of 160 000 kN/m considered appropriate for this type of soil. The springs were non-linear, specifying for the spring stiffness to only engage in compression and assigning no resistance when uplift occurs.

Material properties assumed were:

- Mild steel with a yield stress of 275 MPa
- Concrete strength of 45 MPa (1.5 x 30 MPa design value)
- Concrete density 23.5 kN/m^3

3 INPUTS

The earthquake records used for the time history analyses were obtained from the Institute of Geological and Nuclear Science (GNS Science). These were recorded at the Canterbury Aero Club, located 1.7 km west of the chimney. Two corrected horizontal records orthogonal to each other were used (Figure 5) as well as a corrected vertical record. The records lasted 80 s and were recorded in time steps of 0.02 s.

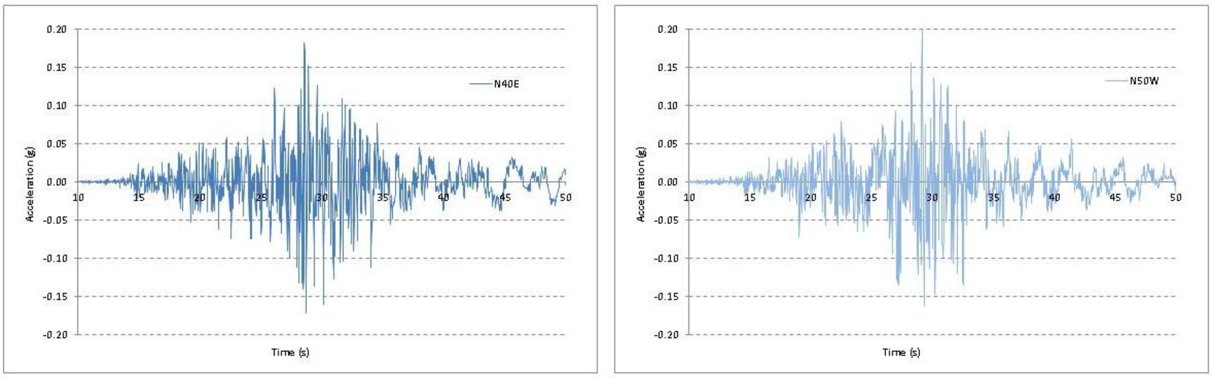

Figure 5. Input accelerations recorded at Canterbury Aero Club. Direction N40E (left) and N50W (right)

4 ANALYSIS

A modal analysis with linear springs was run first to determine natural periods and as a general check of the model. The results were consistent with those assumed in the design calculations.

Originally it was decided to run the time history analysis continuing from the dead load case, however this created a problem where the cantilever end of the dampers connected to the soil springs was deflecting as these compressed, inducing a moment in to the steel member. In reality this would not have occurred as the dampers would have been installed once this settlement had taken place.

The solution to this was to run the dead load case first with no dampers present in the model. This allowed all the weight of the chimney to be supported evenly on the four bearing pads and provided a settlement value which can also be calculated by hand. This settlement allowed the soil spring force-displacement input to be adjusted accordingly so that any upwards movement over this settlement distance was able to be accounted for as unloading of the soil before this is overridden with zero stiffness as rocking begins. A typical example of this is shown in Figure 6. A plot of the force – displacement graph from the analysis output can confirm that the spring is behaving as intended.

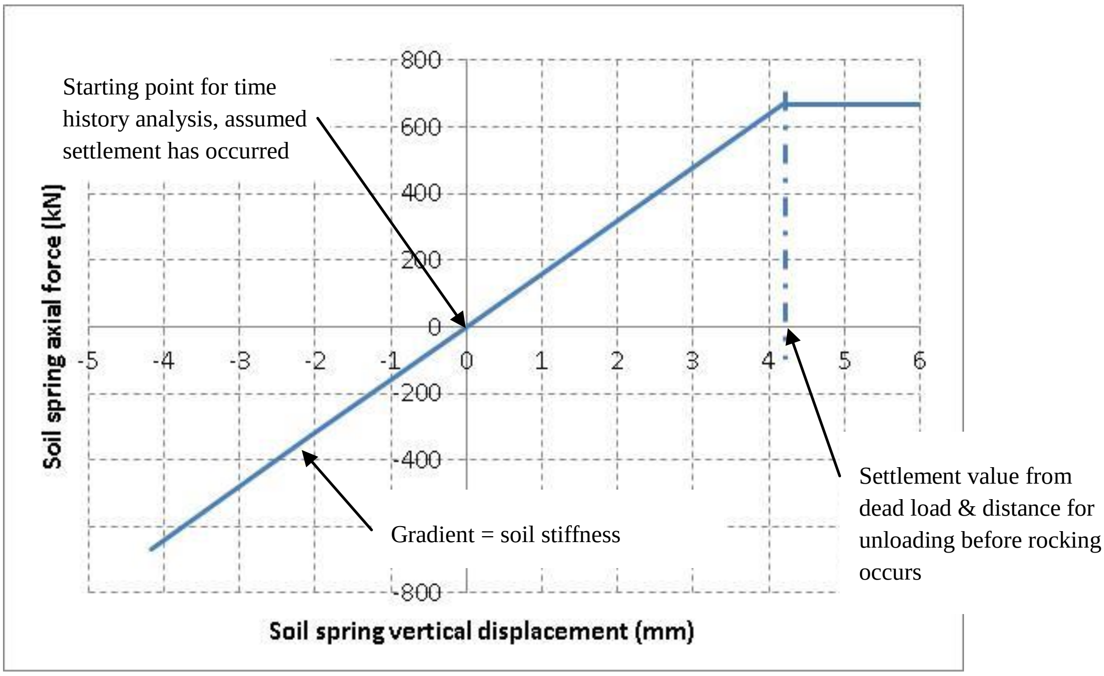

Figure 6. Typical force – displacement graph for soil spring, dependent on stiffness

A direct integration non-linear load case was run for the main analysis. The three input time history records from GNS were scaled for units and correctly orientated to the global axes. From the requirements in NZS1170.5 the time step was specified as 0.005 s. Proportional damping was used of 2%, specified at the first and third modes previously calculated with modal analysis. This value is low as the steel dampers are designed to provide the majority of the design 5 % critical damping once they yield, designed to correspond with uplift of the structure.

Using Equation (1) and a yield stress of 275 MPa, the moment where yield would first occur in the steel plate dampers closest to the central bolts was able to be calculated.

$$\sigma = \frac{Mc}{I} \tag{1}$$

where σ = yield stress; M = maximum moment; c = distance from neutral axis to extreme fibre; and I = moment of inertia.

A hinge was added to the two steel member damping elements at the fixed end. This hinge specified the moment – curvature relationship and is needed to include the plastic behaviour and hence additional damping of the member in the analysis. It is possible in some analyses cases, dependant on the distance to unload from the soil stiffness, that the dampers will yield before uplift occurs. This is considered consistent with the design philosophy where the dampers will act to stop or limit uplift.

5 SENSITIVITY

As the uplift value is critical to the moment and potential yielding of the steel dampers, sensitivity analysis was undertaken with varying soil spring stiffness, as this value had a large influence. The spring stiffness was halved (80 000 kN/m) and doubled (320 000 kN/m) and the analysis re-run.

Analyses were also completed for the different soil stiffness's excluding the steel dampers from the model. These were intended to give an indication of how effective the dampers are in reducing uplift values.

Additional analyses with the original 160 000 kN/m soil stiffness input were also undertaken to see the effect of inherent damping, scaling of the earthquake records, and the effect of stiffness modifiers or cracking to the concrete.

As there was some concern over the behaviour of the hinges, a couple of other hinge options were explored to determine the range of results possible. One option was applying a hinge with different yielding values (dependant on cross section properties) at 10 points along the length of the steel damper member (as opposed to one at the fixed end). Another alternative option was to replace the steel member with a link element with equivalent force – displacement properties. These options will give a good indication of whether it is appropriate to use one hinge only.

6 RESULTS

Table 1 shows the results achieved when the chimney is analysed without dampers.

Table 1. Summary of uplift values without effect of dampers.

Soil Spring Stiffness (kN/m)	Dead load displacement (mm)	X Axis Uplift* (mm)	Y Axis Uplift* (mm)	Yielding of steel plates?
80,000	-8.4	-0.2	2.6	N/A
160,000	-4.2	2.4	1.0	N/A
320,000	-2.1	4.9	6.8	N/A

*Uplift defined as additional distance moved vertically after unloading of settlement displacement

These results indicate that in all cases uplift/rocking would occur, except for in one direction with the softer soil.

It can be determined that the vertical displacement required for the steel plate to yield equates to 2.4 mm. For the original base spring stiffness case (160,000kN/m), this indicates that even before the chimney has fully unloaded the soil, the dampers should yield.

Table 2 shows the settlement under dead load as well as the maximum uplift values that occurred at the legs adjacent to each of the dampers when the steel members were included in the model with a moment limiting hinge. The result below for 160 000 kN/m soil stiffness is referred to as the 'base case' in this report for which the sensitivity results can be compared.

Table 2. Summary of uplift values with dampers.

Soil Spring Stiffness (kN/m)	Dead load displacement (mm)	X Axis Uplift* (mm)	Y Axis Uplift* (mm)	Yielding of steel plates?

80,000	-8.4	0.2	3.3	Yes
160,000	-4.2	1.3	0.6	Yes
320,000	-2.1	4.8	6.9	Yes

*Uplift defined as additional distance moved vertically after unloading of settlement displacement

These results provide interesting comparisons with the results excluding dampers and it appears these are very dependent on the soil stiffness. All of the analyses indicated that the dampers should yield and therefore should act to reduce uplift. For the base case the addition of the dampers does decrease the uplift However for the softer soil, the addition of dampers actually increases uplift and the uplift is similar for the stiffer soil. This did not match expectations that the corresponding models with dampers would produce smaller uplifts.

Figure 7 below shows the uplift occurring in the one leg for the base case. For this case the leg only lifts off once.

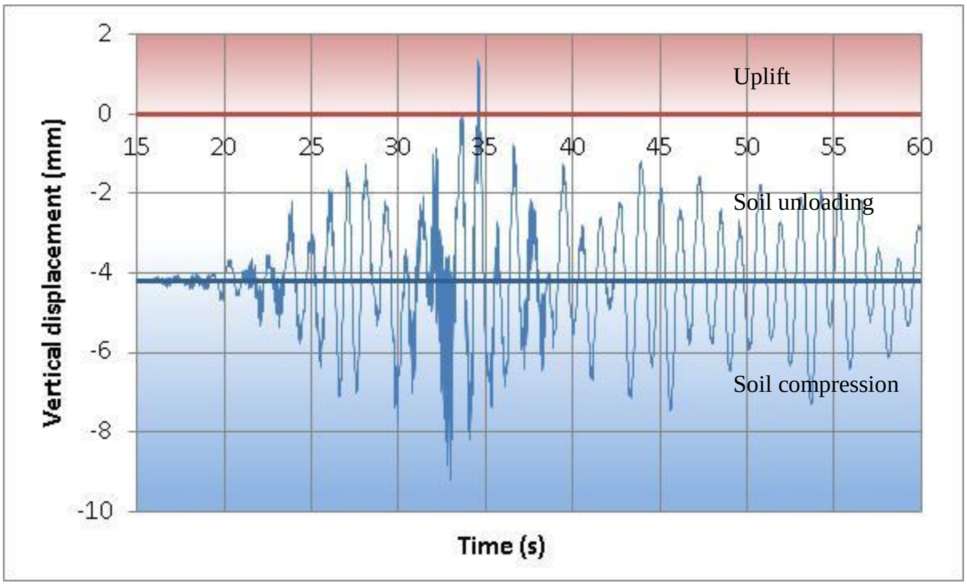

Figure 7. Vertical displacement of one leg for the base case analysis

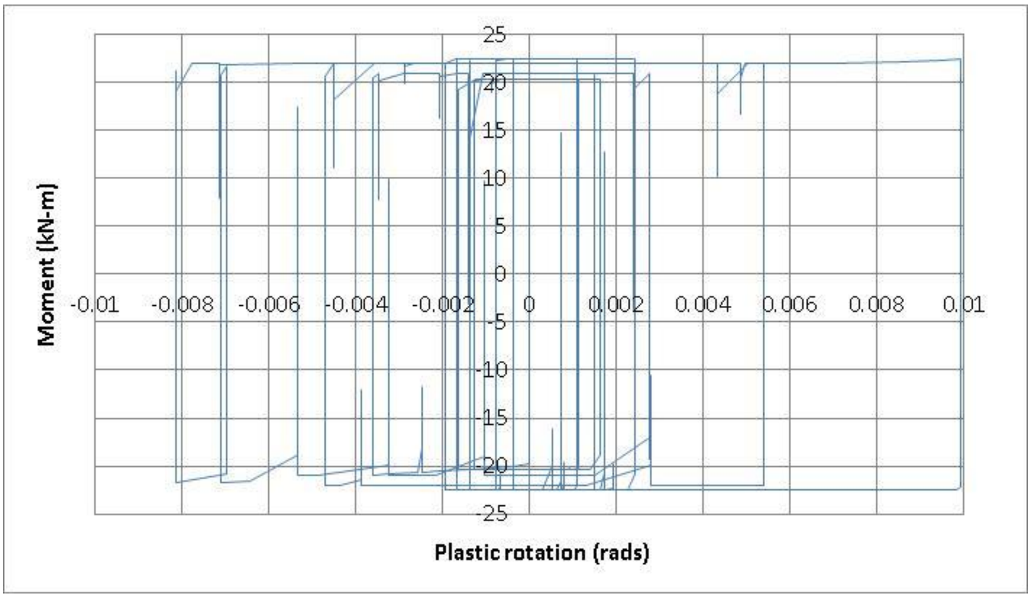

Figure 8. Y direction hinge Moment – Rotation output of steel member from base case analysis

Figure 8 shows the output hinge results for the base case analysis for the hinge located at the fixed end of the steel damper member in the y-direction. This clearly illustrates yielding in the member. For sensitivity, other hinge options were investigated. These produced very similar uplift results to modelling only the one hinge, thus this was considered appropriate and not a critical parameter.

Additional sensitivity analyses run produced interesting results.

Table 3. Summary of uplift values for sensitivity analysis.

Variation	Dead load displacement (mm)	X Axis Uplift* (mm)	Y Axis Uplift* (mm)
no variation (base case result from Table 2)	-4.2	1.3	0.6
5 % structure damping	-4.2	-0.2	-0.4
0.5 % structure damping	-4.2	1.1	1.5
half input acceleration records	-4.2	-1.8	-1.8
0.4 wall stiffness modifier	-4.2	0.9	0.5

*Uplift defined as additional distance moved vertically after unloading of settlement displacement

The results show damping has a large effect on uplift values and is a very important input parameter. Increasing the structure damping to 5 % produced a result indicating no uplift would occur. However this value still indicates that the steel dampers should have yielded. Decreasing the damping to 0.5 % increased uplift in one direction but not in the other, which was surprising.

Scaling the earthquake records by half greatly reduced upwards vertical movement and indicated uplift and yielding of the dampers would not occur. These results were predictable.

Applying wall stiffness modifiers to the structure to account for cracking and reduced effective member area slightly reduced the uplift. This difference was insignificant and indicates this value does not have a large impact on the analysis.

7 CONCLUSION

The analysis results show that the chimney structure should have uplifted in the Darfield earthquake and that the steel dampers should have yielded. This is inconsistent with observations made at the site in the days following the earthquake.

Many assumptions were made to run the analysis and this outcome indicates that these were not accurate enough to produce a realistic model. Two of the major assumptions made that had a large impact on analysis results were the soil spring stiffness and damping value. Considering damping as the only variable, the results from this paper indicate that damping assumed in the analysis has been underestimated.

If the analysis was being used as a design tool, soil investigations would be conducted at the site and a more accurate stiffness value deduced. For this paper this was not possible, and the assumed value adds a lot of uncertainty to the results. However, it should be noted that uplift still occurred for all three soil spring stiffness'.

The difference between onsite observations and the analysis results could also imply that the chimney was not subjected to the level of accelerations recorded at the Aero Club.

8 REFERENCE

Sharpe, R.D. & Skinner, R.I. 1983. The Seismic Design of an Industrial Chimney with Rocking Base, *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol 16(2) 98-106.