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HARBOUR CITY CENTRE WELLINGTON: SEISMIC STRENGTHENING

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ABSTRACT

The existing Harbour city Centre structure was designed and constructed circa 1929 and consists of concrete encased riveted steel frames (beams and columns). It has a high ground floor space, 6 suspended floors and a concrete roof.

In 1979 concentric chevron bracing was added, with 4 bays of chevrons in each direction (longitudinal and transverse). At elastic stress levels these braces only provided approximately 30% of NZS 1170.5 requirements beyond which brittle compression brace buckling and connection failure would occur.

Their performance has been significantly improved by the installation of axial (in-line), hysteretic dampers which will enable the braces to yield in both compression and tension without the braces buckling.

Scale prototypes and a full mock-up damper was production tested under prolonged cyclic action. The results from the testing were correlated with an analytical model. Mechanical and hysteretic performance was excellent.

Non-linear time history analyses were carried out to verify the design with respect to overall building response, to 100% NBS load levels including near field effects. The analysis/design was Peer Reviewed and given Building Consent.

During 2011 the project was successfully completed in a staged manner with much of the building remaining occupied during the retrofit. The strengthening has assisted the client in securing significant, high-value lease agreements for the building.

Introduction

The Harbour City Centre project initiated from an IEP review of the building's seismic capacity by Wellington City Council as part of their Earthquake Prone Building policy resulting from the 2004 Building Act.

Dunning Thornton carried out a review of WCC IEP, and in examining the building and its history identified the existing bracing retrofit in 1979 as the critical elements requiring assessment and retrofit Arranged in a concentric manner the bracing was prone to brittle, compression failure. Additionally, the arrangement of the end connection where the braces attached to the weak axis of the existing riveted plate columns was identified as having a tension capacity of approximately half of the compression buckling load capacity of the brace. Following an assessment of these weaknesses, Dunning Thornton reported to the owner their concurrence that the building was indeed likely to be Earthquake Prone.

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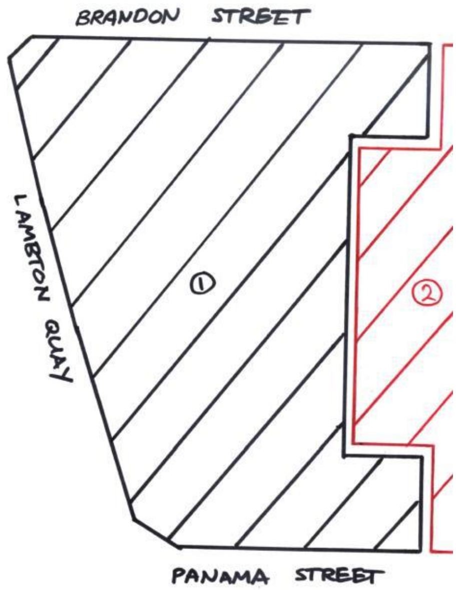
A study of possible strengthening options was commissioned. The building lies on a key part of Wellington retail's high retail value "Golden Mile" and as such, prolonged disruption to the ground floor would have a significant effect on the income stream from the building. The upper floors were of more mixed income quality: the key tenant were about to embark on a refit of their open plan tenancy, but the floors above this comprised many smaller office spaces often still using the original breezeblock partition walls.



The existing braces, retrofitted in 1979, were positioned throughout the floor plate with many located in the existing central light well to minimize intrusion into the floors. Preliminary analysis indicated that if ductility could be introduced into the brace performance, they could meet a significant proportion of 1170.5 demand. While the existing braces compromise the space to some extent, their presence had been accepted and the addition of alternative or further new elements was considered undesirable and potentially more disruptive.

The focus of the subsequent study was therefore how the performance of the existing braces could be improved. Elastic strengthening of the braces by plating up the UC sections would overload the existing end connections, columns and foundations. Options to add energy dissipation to the braces and to minimise the overstrength actions from them was identified as most desirable.

Replacement with new proprietary viscous or buckling restrained braces would provide this performance objective, but would involve significant disruption to the (live) building, and have a significant cost. The strategy of cutting the existing braces and inserting new specifically designed energy absorbing devices was proposed. This paper focusses on the development of these devices, their testing and the analysis of the retrofit building.

The intent of the works was to improve the seismic resistance of Building One such that satisfactory performance occurs during a seismic event with a 1/500 annual probability of exceedance. In other words, the strengthened structure shall comply with current building code seismic requirements.

Description of Existing Structure

The existing structure (Building One) was designed and constructed circa 1929. It has a high ground floor space, 6 suspended floors and a concrete roof. It abuts an older, timber floored building constructed around 1900 that is 3 stories high (Building Two). The two buildings are currently tied together but Building Two is scheduled for demolition and redevelopment.

The structure of the 1929 building consists of concrete encased riveted steel frames (beams and columns). The floors are reinforced concrete supported on a grid of secondary and primary concrete-encased steel beams.

On three sides (north, south and west) there are concrete facade frames: the beams are again concrete-encased steel but the columns are further stiffened by the extension of the encasement to form wing walls. At the east side are two stair/lift towers. These are essentially steel-framed with concrete infilled panels. The remainder of the structure is an open framework.

The columns are all founded on concrete pads/piers sitting on rock, 1-3m below Lambton Quay.

In 1979 concentric chevron bracing was added, with 4 bays of chevrons in each direction (longitudinal and transverse). At elastic stress levels these braces only provide approximately 30% of NZS 1170.5 requirements. Beyond the elastic limit compression brace buckling and connection failure would occur. More fundamentally, a critical weakness in one of the chevron connection types was also identified; this would likely lead to premature failure even before the brace capacity was fully developed.

Proposed Strengthening

Working within their elastic limit, the braces provide approximately 30% of NZS1170.5 requirements in compression. During 20011 their performance was improved by the installation of axial (in-line), hysteretic dampers which will enable the braces to yield in both compression and tension without the braces buckling.

The resulting dampers utilise steel pins fitted through the cut braces, put into “two-point” bending by the addition of flange plates across the cut. This would retain the high initial stiffness of the dampers, but allow significant post-yield capacity, hence effective ductility, through curvature of the pins over a long length.

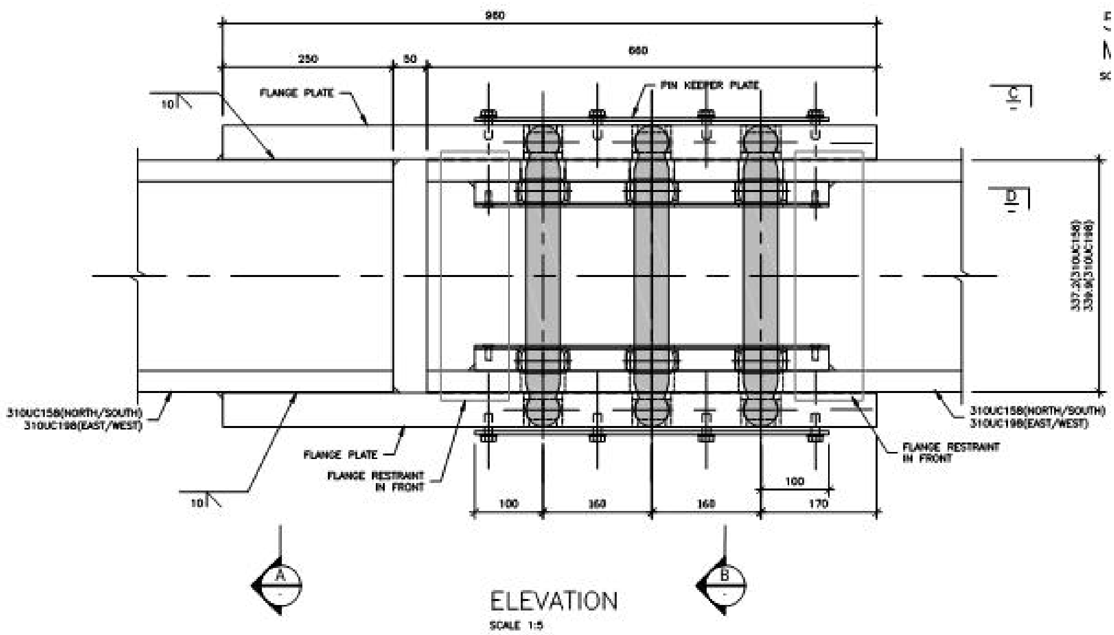


Figure 1. Elevation of a typical damper.

Figure 2. Yielded pin after testing.

The design challenges were around the connection points of the pins; how to allow the high rotations around the bearing pints without inducing double-curvature to the pin or unwanted additional friction forces. The final design employs a combination of brass bearing seatings, and proprietary SKF shaft bearings.

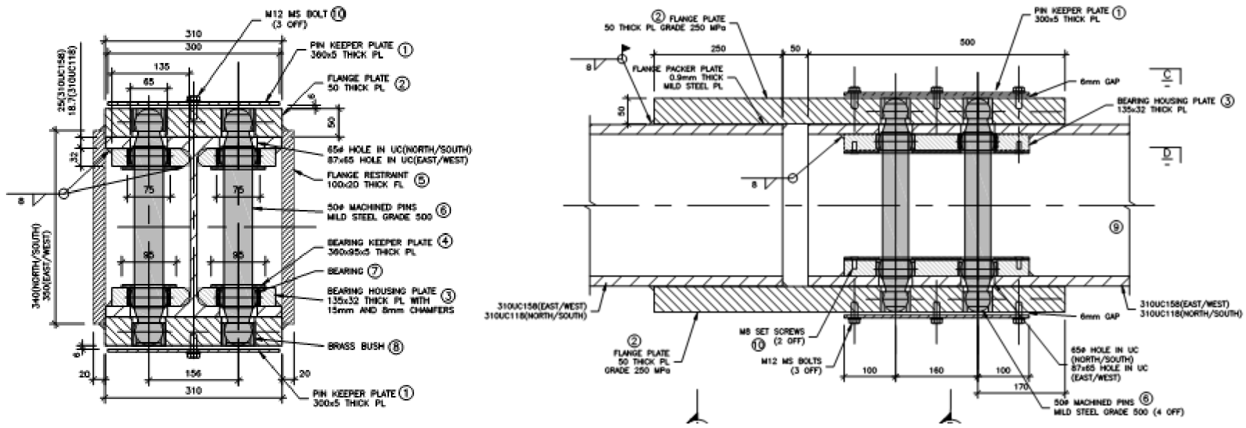


Figure 3. Section through pin showing bearing arrangement.

Other strengthening elements for the building included:

- The addition of an additional bay of conventional k-bracing to the north side to balance horizontal irregularity (resisting torsion).
- The addition of vertical anchors to resist uplift and to increase the compression capacity of the foundations.
- Re-jacketing (encasing) of some columns to improve axial capacity and to provide additional resistance to soft-storey inclinations in the high ground floor space (and associated bracing of the mezzanine floor).
- Strengthening of selected brace connections to the existing beams and columns (in doing so eliminating the critical weaknesses).
- Conventional compression strengthening of some of the upper floor braces.

Blockwalls are provided between the mezzanine floor and the ground floor. These walls provide acoustic separation and, in some locations, support gravity loads from the mezzanine floor. To mitigate “short column” effects the walls have been de-stiffened in-plane as much as practicable by the introduction of vertical joints/sawcuts.

Interaction with Building Two

The structural work detailed will improve the performance of Building One, though the facades are built together with those from Building Two. The full performance benefit from the upgrade will not take effect until Building One is seismically isolated from Building Two, or Building Two is removed. In the interim Building One will not meet 100% NBS however it would still remove the building from earthquake prone status and achieve an estimated 80%NBS.

Future Additions

An allowance has been made for a possible future high-rise structure on the site of Building Two that would partially bear on the existing Building One. The future structure was assumed to extend nine storeys above the existing structure. Lightweight (steel framed) construction was assumed and the additional vertical loads to be imposed on the existing structure would be of the order 6000kN.

One line of columns was strengthened to increase their gravity load capacity for this demand. This allowance was solely for gravity loads and does not account for the additional P-delta seismic demands that are inherent with increases in vertical load. Analysis of the combined structures would be required should any redevelopment occur, and the combined structure would need to perform to full new building standard.

Testing

The dampers went through a two-stage testing process. Dunning Thornton particularly thank Chris Gannon from Robinson Seismic for assistance with organizing the testing and with assistance in brainstorming ideas of how to achieve robust detailing around the pin connections.

The first stage involved prototype testing of a single dowel and led to development of the seating/bearing details at the end of the dowel. The successful hysteretic performance of the dowel was confirmed.

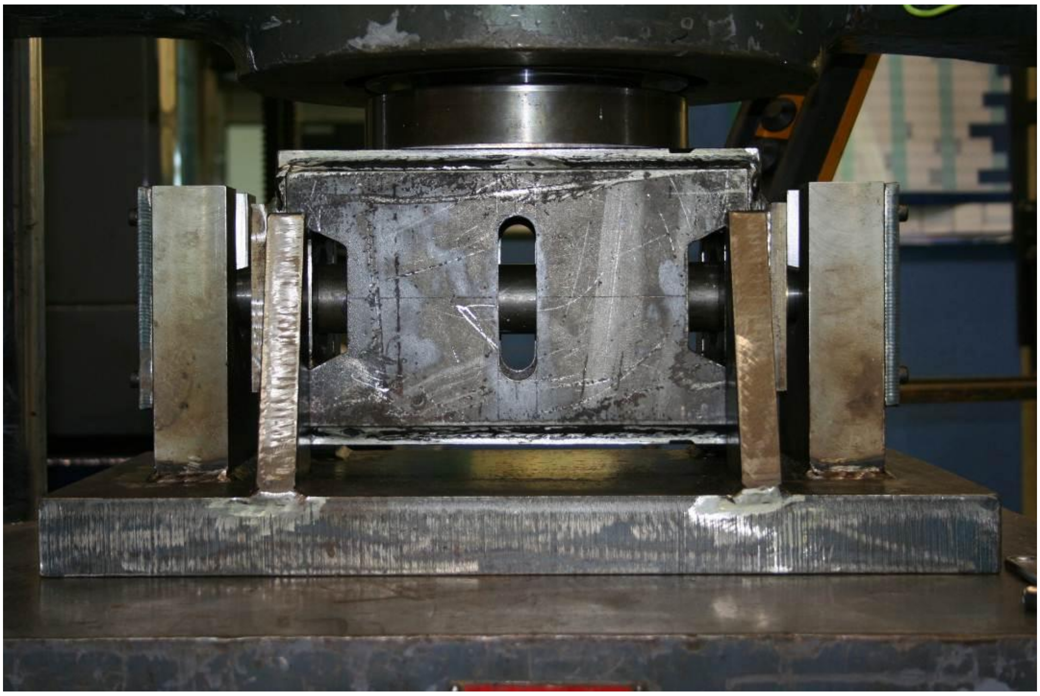


Figure 4.   Prototype damper setup.

In the second stage a full mock-up damper/brace arrangement was production tested under prolonged cyclic action. The results from the testing were correlated with an analytical model. Mechanical and hysteretic performance was very satisfactory.

As all properties of the brace system were theoretically derivable from “code” physics, the testing was determined to be “production testing” in accordance with AS/NZS 1170.0 provided that the results correlated with the theoretical predictions. A single test would therefore be sufficient as the design was not dependent on achieving a 95% certainty from the “scatter” of results that would arise from a system that was not theoretically predictable.

During developed design the damper was tested beyond the limit of its flexural (hysteretic) displacement to determine its ultimate capacity. The capacity of the damper was found to exceed the rated capacity of the testing equipment. Hence, the ultimate failure capacities of the dampers are unknown. The ultimate demand on the dampers in this application is within the tested load.

Typical results from testing are shown in the graph below.

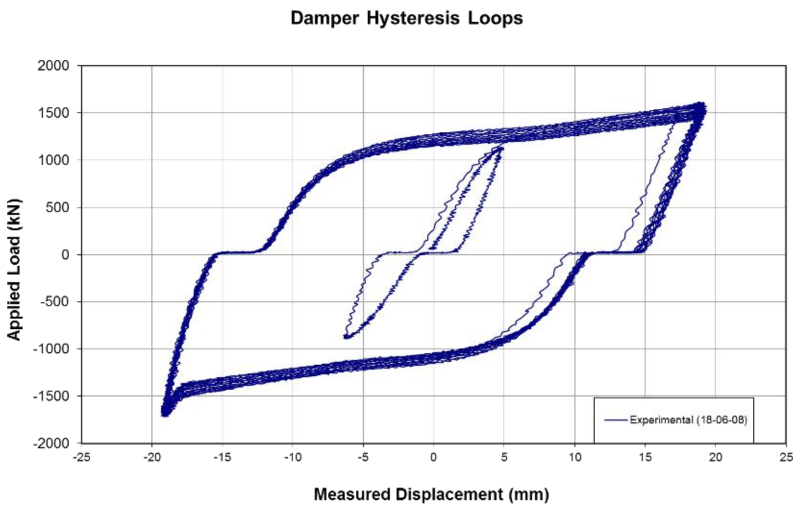


Figure 5.   Production testing results.

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Structural Modelling

The structure was recreated as a three-dimensional model in ETABS for the purpose of undertaking non-linear time history analyses and verifying the design. Non-linear elements are confined to the strengthened "brace" elements and a rocking wall each side of the east-side stairwells.

In order to allow for the effect of P-Δ, the approximate average horizontal centre of mass was determined and a column member was created in this location. The column member was pinned at the top and bottom of each storey so that it did not provide any lateral resistance, and had a very high axial stiffness so that its natural period was less than that of the lateral modes used in the analysis. A vertical point load representing the weight of each floor was applied at the relevant height.

The rotational inertia was calculated manually and input for each floor, and extra mass was included at first floor level as an allowance for the mezzanine. The horizontal masses assigned to each (rigid) diaphragm were sourced (by ETABS) from the vertical loads and mass moments of inertia applied to the "column" in the model. The structure's effective vertical centre of mass was found to be close to level four.

Some members of the existing structure were de-stiffened from their gross concrete stiffness such that their contribution to resisting base shear was limited to their capacity. The additional ductility demand on these elements was also considered.

Actions were checked on the existing plated steel girders that contribute to frame action. The contribution of the girders was limited to the nominal capacity of the typical riveted beam-column connection. Flexural yielding of the steel angles fixed to the flanges was found to be the governing mechanism (note that this is a ductile yield mechanism).

The concrete façade, which has a significant effect on the shape of the first mode, was also de-stiffened in the model. As the short beam members will be subject to high shear demands, any concrete encasement would crack. Accordingly the beam elements were assigned the stiffness of the bare steel beam section only. The stiffness of the columns in the façade was reduced from that of the gross concrete section to reflect the reinforcing in the wing walls.

To re-create the observed behaviour of the dampers, three non-linear link elements were used in parallel. The link elements were as follows:

- A "plastic" link – a bi-linear hysteresis with properties such that the initial stiffness, yield force and post-yield stiffness agreed with tested and theoretical results.
- A "hook" link – a link element with no stiffness except for when its positive (tensile) displacement exceeds a specified value. The value used was the displacement at which the damper ran out of free travel and began to stiffen. The stiffness of this link was such that, when combined in parallel with the "plastic" link, it agreed with experimental results
- A "gap" link – as for the "hook" link, except using negative (compressive) displacement.

To verify that this combination of links produced the required behaviour, a separate model was created with a single pair of braces and a mass. The output from this model suggested that the combination of links does not create numerical instabilities and generates the appropriate hysteretic behaviour.

The eastern side of the building contains a stair well and elevator shaft at each end, and reinforced concrete walls (constructed in relation to damage following 1942 earthquake) are positioned in these areas. The capacities of these walls were assessed (refer section M) and it was determined that some would rock. Some walls were not included in the model because there was insufficient diaphragm capacity to transfer lateral demands. The historic repairs had only been undertaken at the first two levels, hence only these levels were modelled. Modelling of these walls was considered necessary because their corner locations mean they are likely to have an influence on the structure's torsional behaviour. The historic strengthening also provides part of the resistance to the ground floor's inclination to otherwise form a soft storey.

These walls were modelled as rocking elements with vertical loads on top of the walls to represent the gravity load borne by the walls. The gravity load is eccentric and the distribution of the modelled vertical load reflects this. To create rocking behaviour, "gap" or compression-only springs were provided as vertical restraints.

The stiffness of these springs was approximated to the foundation conditions. Similar to the braces, a separate model was created to verify that the behaviour of walls matched what was intended. Dynamic (vertical "pounding") effects on the column bases are not considered critical because of the low stresses in the wall and the small contribution of the walls to the total base shear. The vertical loads on the walls did generate some vertical modes but inspection of the participating masses shows these walls to have negligible influence on the structure's response.

Seismic Demands

Earthquake records were, in general, scaled using the procedure prescribed by NZS 1170.5. However, in a departure from 1170, seven time-history ground motion records were used and the records were scaled so that their average spectra matched the design spectra found using NZS 1170.5. The period range of interest was taken around the effective period. Design demands were determined from the average maximum of records.

While conducting the time-history analyses, it was observed that, during oscillations that corresponded to velocity pulses of forward-directivity/near fault events (typically the Lucerne record), the dampers exceeded their design displacement and their stiffness increased accordingly. The effective period of the structure therefore shortens during this peak cycle. Comparing the NZS 1170.5 spectra and the scaled spectra at this effective period, the acceleration of the NZS 1170.5 spectra is noticeably higher. To account for this overestimation of demand, the magnitude of the scaling was reduced so that the average of the records' spectra coincided with the NZS 1170.5 design acceleration at the critical effective period (i.e. when the structural demand is the greatest).

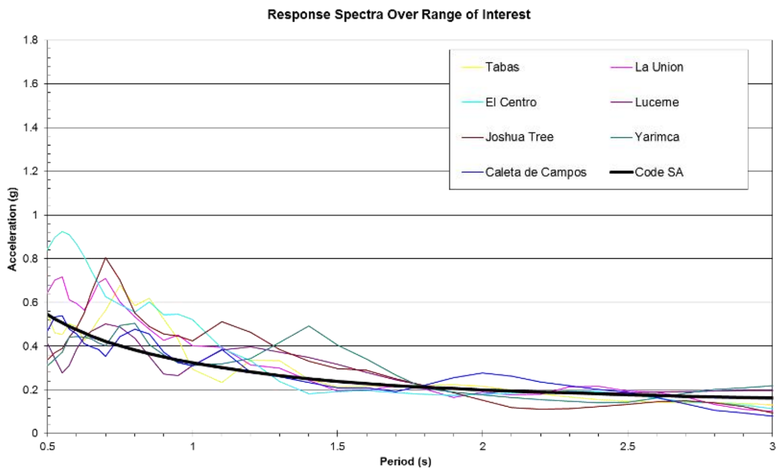

Figure 6. Spectral trace at the point of interest.

Acceptability Criteria

An unacceptable response is considered to be failure of members or connections which results in either a loss of gravity load bearing capacity or a significant, irrecoverable reduction in lateral stiffness. Specifically, these were considered to be:

- Buckling of columns.
- Shear failure or confinement failure (i.e. "bursting") of reinforced concrete column encasements.
- Foundation failure or uplift below braced frames.
- Failure of a brace connection.
- Fracture of a damping device (ultimate capacity taken as the maximum tested load).

Buckling of a brace is assessed on the circumstances of the buckling. If, for example, a brace buckles multiple times during a seismic event, this is considered unacceptable. However, if the brace only buckles under a single extreme cycle due to, for example, a single near-field velocity pulse, this was considered acceptable provided that:

- The girder that the brace connects into can form a uni-directional plastic hinge at mid-span, and the connections along the relevant load paths have been detailed to resist the over-strength actions (i.e. collapse of the floor is prevented).

- The brace buckles very close to the peak of the cycle and the reduction in stiffness is not significant enough to allow a soft-storey collapse to occur due to P-Δ effects (i.e. one brace buckling at the edge of the structure, not all braces on the floor buckling).
- Inter-storey drift of the floor restrained by the brace is considerably smaller than 2.5% (i.e. there is ample opportunity for the ground motion to reverse and “catch” the building).

Results and Design Checks

Axial capacities of columns subject to brace loads were checked. Peak brace loads were found by summing the plastic link and the largest absolute value of the hook or gap links. These were checked against the axial capacity of the critical braces and the maximum tested capacity of the dampers. Upper level (elastic) braces were found to buckle prematurely and accordingly conventional strengthening is to be provided in these locations.

Maximum storey displacements were within the drift limits specified by NZS1170.5. Due to there being no adjacent structures, there is no issue with pounding (Refer to introduction regarding separation of Building Two).

Notably, under records with near-field characteristics the dampers exceeded their intended maximum displacement (i.e. “bottomed out”). The tangent stiffness and secant stiffness of the dampers increased, and effective damping therefore decreased accordingly.

Inspection of the backbone of a base shear vs. centre of mass displacement plot shows an idealised ductility of 1.35. Using this ductility and the effective period at the peak displacement to derive the equivalent static base shear results in a value similar to that observed in the time-history output.

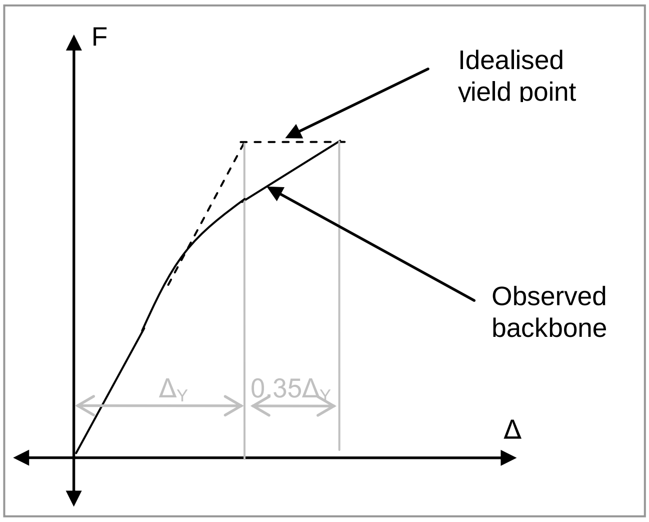

Figure 7. Backbone curve.

The near-field characteristics tended to induce greater displacements and, in combination with a higher effective stiffness, this reduces the hysteretic damping considerably. If the intensity of the near-field events is reduced by 10-20%, the dampers are far more effective. The table below presents the base shears (in kN) as reported by ETABS and as calculated using a displacement-based design (DBD) approach. The noticeable discrepancy in values for near-fault events occurs because one record causes a large, one-off spike (~30,000 kN) in base shear. This record possesses forward-directivity characteristics.

Excitation Characteristics	DBD Calculation	ETABS Output
Near-fault	20,890	23,130
Far-field	15,660	16,800

Torsional Response

To assess the torsional response, the structure’s centre of mass was shifted to induce a torsional response. The mass was offset by 10% of the structure’s width towards the north-western corner and also by the same distance towards the south-eastern corner. The time-history analyses were then re-run (without adjusting record scaling). The output from this can be seen on page EP80 of section I and is reproduced below. The displacements shown are the average of the peaks from seven earthquake records. For a north-south excitation the structure displays considerable torsional robustness. However, under east-west excitation the structure's behaviour becomes more torsional.

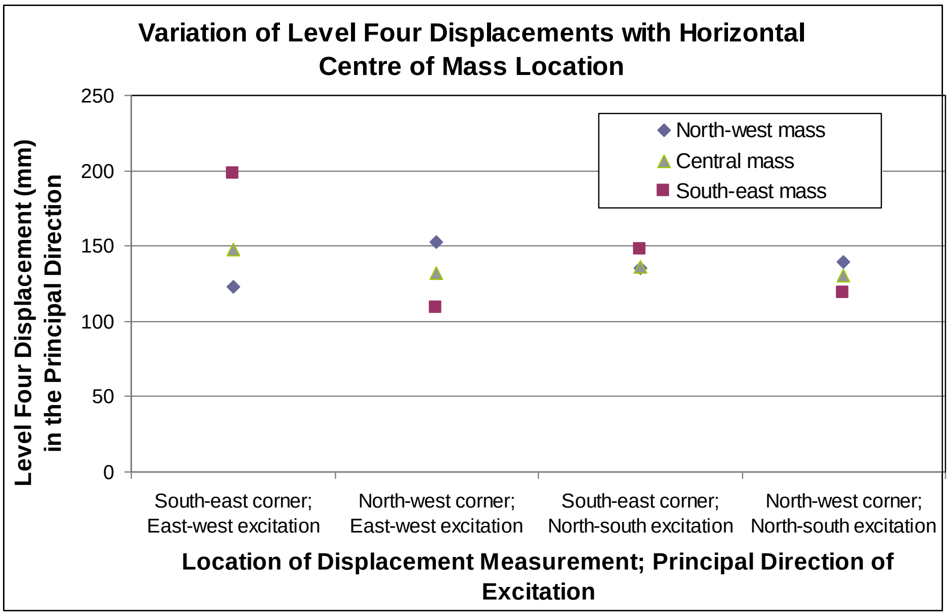

Figure 8. Outcome of torsional response studies.

It should be noted, due to the nature of the structure, that greater than 75% of the mass is dead load which has been explicitly located at their respective centres of mass. Therefore actual eccentricity is likely to be considerably less than 10%.

Brace Design

To allow the structure to withstand the demands of a 1/500 probability of exceedance event, strengthening of braces, brace connections and foundations has been detailed. Demands were determined from the non-linear time history output, and the mean load in the element under consideration was found for the seven records. This average inherently includes some over-strength actions as the link elements (representing the dampers) exhibit "hardening" so it is therefore acceptable to apply a strength reduction factor of unity, also the steel frame has some natural nominal ductility. However, when designing ground anchors, as they are non-ductile, a strength reduction factor of 0.8 was applied.

Steel plating was added to the upper level braces (without dampers) to increase their compression capacity. Although the compression capacity was increased, the expected failure mechanism is still buckling (i.e. the plating is not significant enough for yielding of the non-plated cross-sections to govern). Accordingly, the capacity of these braces' connections needed to be evaluated. Generally, the existing bolted connections had sufficient capacity to transfer the brace loads. However, the welded connections of the north-south braces (a critical structural weakness) were determined to be ineffective and are to be strengthened by adding steel gusset plates above and below the braces.

The lower level braces (with dampers) do not require plating. To improve robustness, the braces' connections were strengthened such that if the compression brace was to buckle, the beam at the apex of the braces can yield. The connection strengthening was designed to develop the over-strength capacity of this beam.

Installation

The methodology for step-by step installation of the dampers to the braces was developed by Dunning Thornton. The structural steel subcontractor (MJH Ltd) developed this further to include their new clip-on, computer-controlled cutting torch, which was faster and more accurate than cutting the holes by hand. The simple methodology meant that installation of the dampers into the existing braces got quicker as the project progressed. The methodology can be summarised as follows:

- Cut out the flanges of the UC brace, leaving the web in place to keep the brace in alignment during the installation.
- Form the holes through the UC flanges for the pins using the cutting torch.
- Fit up the pins, bushes, bearings and plates.
- Complete the site welds and cut out the remaining section (web) of the UC brace.

As the on-site work was deliberately kept simple and minimal, the installation time for the dampers was shorter than anticipated. This was beneficial to the contractor, considering they were managing multiple work faces within a live office building.

The strengthening was completed in a staged manner during which much of the remainder of the building remained live. In this was loss of rental income for the owner was minimised, and decanting of occupants could occur so that desirable tenants would not seek alternative leases elsewhere.

Figure 9. Insitu completed damper.

Conclusion

Rigorous analysis was carried out using non-linear time history techniques and good correlation was achieved with both the force-based and the displacement-based approximate analyses. This encourages the belief that the model is a fair representation of the structure (provided that the fundamental assumptions common to all three methods are correct).

The completed structure would be expected to perform in excess of code requirements under far-field events (such as a rupture of the Wairarapa Fault). The axial dampers limit the demand on the upper levels and provide reasonable damping.

Similarly, the completed structure would be expected to perform well under a full-code near-fault event (such as a rupture of the Wellington Fault). Again, the axial dampers – although not as effectively – limit the demand on the upper levels and provide damping. Under an event with a 1/500 probability of exceedance, the dampers would likely exceed their “damping” design displacement (i.e. they bottom out) but buckling of the braces would not be expected to occur. Therefore, the structure has sufficient capacity to perform in a predictable, dependable and repairable manner under an event of this magnitude.

When subject to a near-fault event which contains a forward-directivity characteristic (such as a rupture that initiates in the Hutt Valley and propagates towards Wellington), the structure would not perform in an entirely optimal manner. The “pulse” which characterises this type of event causes a single significant excursion outside of the structure's design displacement. This one-off excursion is likely to cause some lower level braces to buckle in compression. However, the strengthening details provided have been designed for a secondary mechanism (pull-down of the steel girder above) to develop during the single cycle of the pulse. This mechanism ensures that lateral resistance is not lost entirely and hence avoids collapse in this low-probability event. It is of note that a forward-directivity pulse of 70% of code does not cause brace buckling. It should also be noted that NZS 1170.5 states "the inclusion of the forward-directivity component will result in actions more consistent with much lower annual probabilities of exceedance" (refer clause C7.5 of the commentary.

The seismic retrofit was completed successfully within an occupied building. Key tenants were retained, including a major anchor tenant whose nature of business set a high priority on the seismic capacity of their building. With the subsequent Canterbury series of earthquakes, Wellington lessees have become very sensitive to buildings', seismic performance. By embarking on this bold and innovative retrofit project, the owners have secured high-quality, desirable tenants in a key CBD location.

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