Refurbishment and Seismic Retrofit of the Aurora Centre using Fluid Viscous Dampers – Case Study, Wellington

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H. S. McKenzie

Holmes Consulting, Wellington.

D. J. Marriott

Holmes Structures, Los Angeles.

J. Houston

Holmes Structures, San Francisco.

M. H. Thomson

Holmes Solutions, Christchurch.

ABSTRACT: The Aurora Centre project in Wellington has resulted in the conversion of this 1960s office building into a modern A-Grade office environment.

Aurora Tower, a 1960s concrete encased steel frame building, is the largest building of the development at 18 storeys. The retrofit of this Tower is a milestone in Structural Engineering in New Zealand as the first building to be strengthened using Fluid Viscous Dampers. Prototype testing of the dampers was carried out for the fabricator, Victor Hydraulics, by Holmes Solutions in their Christchurch testing laboratory in accordance with European Standard EN15129:2009.

In addition the project involved the strengthening and extension of the existing car park structure (Aurora West), a new Retail Extension, and a new build Aurora East building.

The Tower structure was assessed using a non-linear time history analysis (NLTHA) and seismic performance of the Aurora Tower was governed by the concrete encased steel columns which was influenced significantly by seismic displacements in the transverse direction. In order to strengthen the building to an NZSEE A-Grade seismic rating, both seismic drift and column performance issues needed to be addressed. By providing additional damping, along with a more sophisticated column modelling approach (Marriott 2014), a 90%NBS seismic rating was achieved without the need for additional structural intervention.

This paper summarises the design approach adopted for the project to meet the relevant local and international standards, incorporating the benefits of non-linear time history validation and design development of a viscous damped lateral force resisting system.

1 INTRODUCTION

Owned by Kiwi Property, Aurora Tower is 18 storey concrete-encased steel moment frame with fully-welded beam to column connections, built in the 1960s. The design of the building pre-dated modern seismic design provisions, including capacity design principles. An initial seismic evaluation of Aurora Tower performed by Holmes Consulting in 2011 identified that the seismic performance of the building was limited by the performance of the perimeter columns. In particular, the earthquake-induced column axial loads greatly limited the ductility capacity of the steel columns, an issue that was further exacerbated by the deep spandrel beams resulting in a strong beam/weak column condition. Furthermore, for seismic loads beyond load levels associated with acceptable column performance, inter-storey drift demands in the transverse direction started to exceed 2.5%. Under 100%ULS seismic loads the building was not expected to result in collapse, but damage of the perimeter columns was expected to be extensive.

Subsequent seismic evaluations of the building focused effort on two main building characteristics: (i) more accurately modelling (and understanding) the behaviour of the concrete-encased steel columns in order to avoid adopting conservative evaluation techniques that may result in excessive strengthening of the columns (FRP confinement, for example), and (ii) investigating solutions to address excessive drift demands in the transverse direction. Knowing that column performance was going to be dependent on the lateral drift demands, a solution involving viscous dampers made the most engineering sense for the following reasons;

1. Viscous dampers provide additional damping to the system, which reduces lateral displacements and drift demands throughout the height of the structure by dissipating a proportion of the input ground energy.
2. The reduction in drift demands reduces both the earthquake-induced axial loads (beam shears) and plastic rotation demands (column ductility demands).
3. Maximum damper forces are predominantly out of phase with elastic drift-induced column axial loads. This is an attractive property of viscous dampers installed within an existing building. However, damper forces and displacement-induced column axial loads have been shown to be partially additive for taller structures and for nonlinear dampers (velocity exponent <1.0) (Marriott 2017). Thus, the performance of the structure as a whole must be well understood to be able to reliably quantify performance of individual components.

2 CONCRETE ENCASED STEEL FRAME

The Aurora Tower structural system is a concrete encased steel (CES) moment frame. The columns are typically square in plan, although some circular CES columns exist within the lower levels. The size and weight of the steel column sections range from UC203x203x46 up to UC356x406x467. The concrete column encasement around the exterior steel columns is reinforced with (4) ¾” bars longitudinally, and 9.2mm diameter ties at spaced at 50mm (2”) at the top and bottom of the column, increasing to 100mm (4”) within mid-height of the column. The interior columns have similar transverse reinforcement detailing, but with spacing of the ties increasing to 150mm (6”) within mid-height of the columns. The highest transverse reinforcement ratio was approximately 0.3%. An elevation of a typical perimeter column is shown in Figure 1.

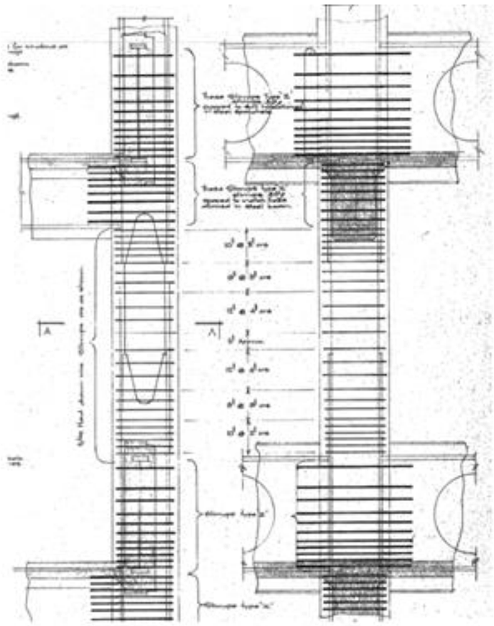


Figure 1: Typical Perimeter Column Elevation

The interior steel beams are rolled structural steel sections, while the perimeter spandrel beams are fabricated steel plate girders. The concrete beam encasement was assumed to prevent local buckling only, allowing the full plastic section capacity to be developed. Any stiffness gains associated with the concrete beam encasement were ignored from the analysis. The weight and mass of any concrete encasement was included via dummy concrete elements as required.

Initial seismic analysis and assessment evaluated the columns using two column elements in parallel, one to model the steel section and the other to model the concrete section. The limitations of this modelling and assessment approach comes down to how axial load was distributed between the two elements and the sensitivity of the element assessment criteria to small changes in axial load. For example, if the axial load within the steel section exceeded an ASCE-41 force controlled limit, this would govern the overall performance of the dual column system, regardless of the demands on the parallel concrete column section.

If CES columns are well confined, the enhanced capacity of the section is associated with the confined concrete encasement providing a compression block internally within the composite section. This composite section has the effect of relieving axial load from the steel section, and also adds an internal moment couple within the composite section. The complete axial-flexure "yield surface" is analogous to an equivalently reinforced concrete column section. While the flexural strength sees only a modest increase, the axial load carrying capacity can increase significantly (due to the compression capacity of the confined encasement). The challenge associated with evaluating the nonlinear cyclic response of existing CES columns is the lack of experimental testing and published literature specifically relating to older (pre-1970s) CES columns. This lack of available experimental data necessitates a somewhat conservative approach from an assessment standpoint, Marriott (2014).

Due to the density and detailing of the transverse reinforcement that was present within the CES columns in Aurora Tower, a single (composite) nonlinear column element was implemented within the nonlinear time history analysis (NLTHA). A single composite column has the advantage of being able to more accurately mobilise the complete section capacity of the composite column, provided appropriate stiffness and strength degradation properties are implemented in accordance with the level of detailing and confinement within the concrete encasement. A single column element can also mitigate force-controlled column behaviour (ASCE41-13) due to the increased compression capacity of the section.

A composite column modelling tool was developed to quantify the nonlinear behaviour of the CES columns (Marriott 2014). The tool was capable of deriving the composite strength and stiffness properties through a moment curvature section analysis of each CES column section. Nonlinear concrete encasement was implanted within the section analysis using material constitutive relationships developed for confined and unconfined concrete.

The section analysis is used to define the composite section's capacity envelope or backbone curve, and identify various material limit states as a function of the imposed curvature demand (concrete spalling, structural steel tension strain exceedance etc). An M-N failure surface can be determined for the various material limit states by monitoring the material strains at each axial load increment. Figure 2 illustrates an example of the component based moment curvature section analysis and M-N interaction surface for each material limit state (further details can be found in Marriott (2014)).

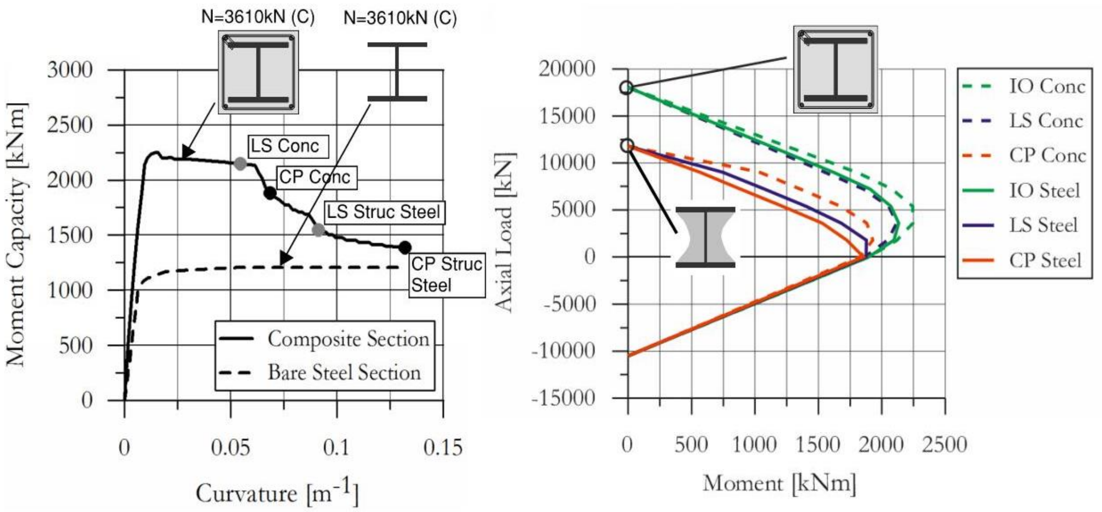


Figure 2 Left: section analysis of CES column, N = 0.2Pcr = 3610kN, Right: Moment-Axial limit-state surface as a function of material strain limit, Marriott (2014).

The existing perimeter CES columns contain cover plates above and below the column mid-section as illustrated in Figure 1. The cover plates add strength and stiffness which in turn decreases the CES column ductility. For this reason each column containing cover plates was separated into three lengths of cross-sections over the column height.

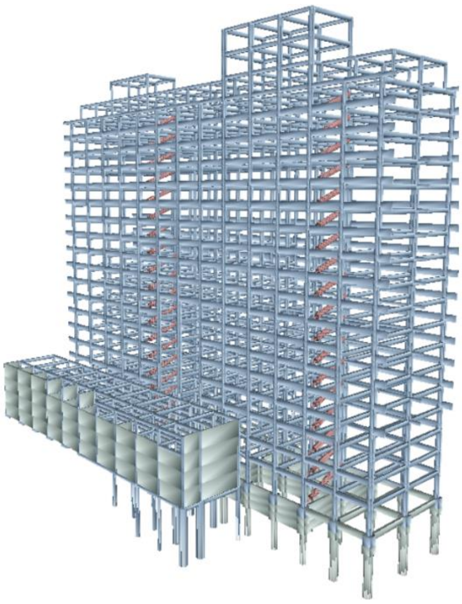
The existing building was re-evaluated using NLTHA after having updated the CES column strength and stiffness properties based upon the composite column tool and accounting for the cover plate detailing at the perimeter column lines.

Implementation of the single composite flexural column elements based upon moment curvature proved to be worthwhile. All columns that were previously flagged as force-controlled in the initial building evaluation now exhibited column performance below Immediate Occupancy acceptance criteria at both 70% and 100% of code loading. Maximum plastic rotations and axial load ratios exhibited during the refined existing building evaluation were 1.5% and 0.47 respectively. The improved ductility and flexural capacities attributed to the composite column tool eliminated the need for costly strengthening of the tower columns.

3 IMPLEMENTATION OF VISCOUS DAMPERS

Even with the more accurately modelled CES columns, the global building response continued to exhibit interstory drifts above 2.5%. In fact, the building generally exhibited a softer response in comparison to the initial building evaluations that relied upon two elements in parallel to capture the CES column behaviour. Interstory drifts in both the longitudinal and transverse direction increased relatively with transverse interstory drifts exceeding 2.5% at both 70% and 100% code loading.

To mitigate the higher than code permissible interstory drift limits, non-linear fluid viscous damping devices (VDD) were quickly deemed the ideal solution. Non-linear fluid viscous dampers provide a means of reducing global drifts through the absorption of kinetic energy. Additionally, the peak demands imposed by the dampers generally do not coincide with the peak demands caused by earthquake actions reducing any additional demands imposed upon the structure. The damper layout chosen consists of two bays of dampers per storey over the Tower height. Figure 3 illustrates the locations of the damping devices in red. Damper locations were chosen to coincide with existing service cores and were therefore readily incorporated into the buildings space planning.



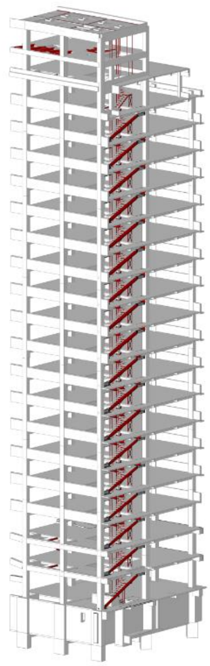


Figure 3: Location of Damping Devices in the Tower

The VDDs were modelled as visco-elastic damper (dashpot) elements configured in a brace configuration in the steel moment frame bays. The elastic stiffness of the damper casing hardware, extender, and gussets are explicitly modelled by inclusion of spring elements in series with a dashpot element. This stiffness equivalency modelling is illustrated in Figure 4.

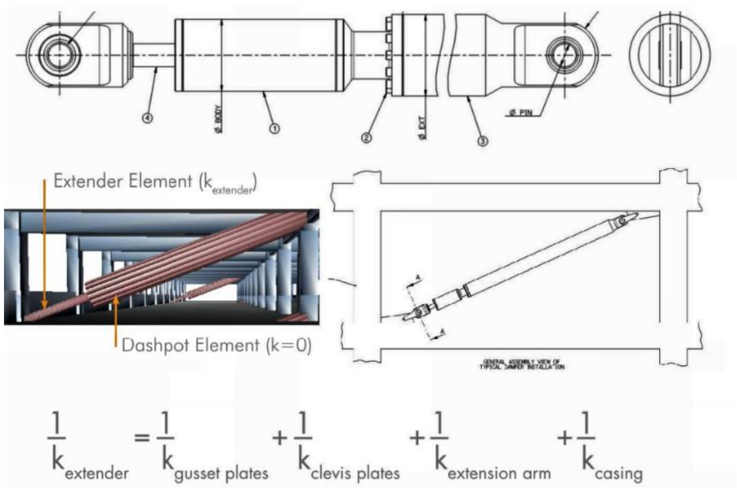


Figure 4: Stiffness Equivalency Modelling

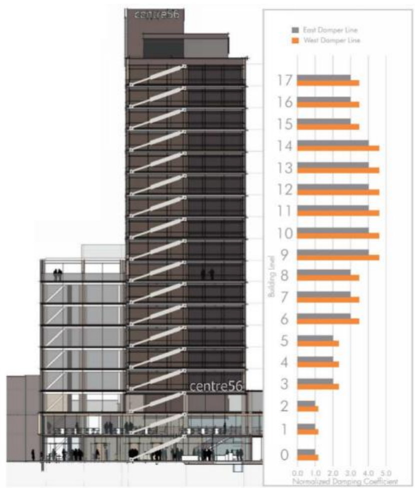


Figure 5: Elevation of Damping Coefficients over the Tower Height

The optimal damper configuration was determined to be a damper coefficient distribution in which the greatest damping coefficients are approximately at two-thirds the building height. Additionally, due to the slightly torsional behaviour of the tower, it was determined that the east line of dampers can have approximately a 15% smaller distribution of damping coefficient in comparison to the west damper line. This was based upon observing that the velocities of the east damper line were consistently less than the west line. All dampers were based on a velocity coefficient (i.e. alpha factor) of 0.6. Note that an alpha of 1.0 would be representative of a linear viscous damper. An elevation of the normalised damper coefficient distribution over the height of the tower is illustrated in Figure 5. The non-linear viscous damping coefficient ranged from 1000kNm/s at the lower levels to 4600kNm/s at two-thirds the tower height.

With the tuned VDD solution, the tower demonstrated 13% critical damping, more than twice the 5% critical damping associated with the undamped existing structure. This increase in damping translated to improved global responses. At 90% of code loading interstory drifts were limited to 2.13% and 2.45% in the longitudinal and transverse directions respectively, achieving the project performance goal of 90%NBS seismic rating.

4 DAMPER TESTING

Testing of the viscous dampers was carried out at Holmes Solutions test laboratory in Hornby, Christchurch, on behalf of the damper fabricator, Victor Hydraulics. The principal test apparatus was HTM-2 (see Figure 6). This multipurpose test machine has an axial static force capacity of 4500kN, peak velocity of 1.0m/s and a stroke of 1100mm.

Figure 6: Viscous Damper VD-8 under test in HTM-2

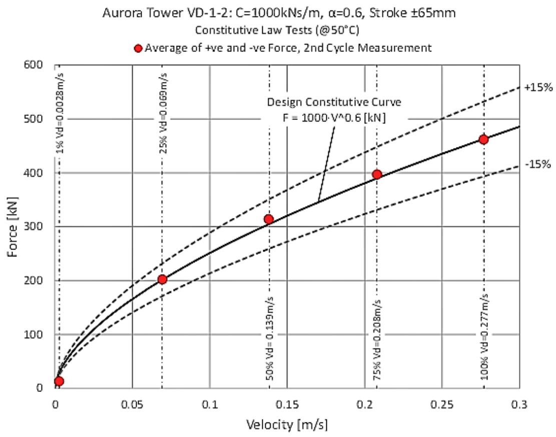


Figure 7: Constitutive Law Test Result - VD-1, 50°C

Testing of the dampers was undertaken in accordance with EN15129:2009. This standard prescribes a number of tests to assess damper performance and durability. Prototype testing covers the entire range of tests as prescribed in the standard. Production testing (1 in every 20 dampers per batch) covers a reduced suite of tests intended to ensure consistency of manufacture. Tests relating to performance include;

- Constitutive Law Test - 3 sinusoidal cycles at design displacement with peak velocities of 1%, 25%, 50%, 75%, and 100% of design velocity, and at maximum and minimum design temperature, in this case 50°C and 0°C respectively. For each of the ten tests the damper is carefully thermally stabilised at the target test temperature.
- Damping Efficiency Test - five room temperature (23°C ±5°C) sinusoidal cycles at design displacement and building fundamental frequency.
- Low Velocity Test - one constant velocity cycle at 0.075mm/s and design thermal displacement (±10mm).

Damper performance is assessed against the design specification; in particular the damper response during Constitutive Law and Damping Efficiency tests must always lie within the specified tolerance band, in this case ±15% of the nominal design constitutive law response curve. Tests relating to durability include;

- Seal Wear Test - 10,000 cycles at design thermal displacement.
- Wind Load Test - 200 sinusoidal cycles at ULS wind displacement and building fundamental frequency.
- Pressure Test - Static pressure equivalent to 125% of the pressure which would be produced by the damper at the upper bound design force for 120 seconds duration.

The general assessment criteria after these tests is that the damper should not show any signs of wear, leakage, deformation, or loss of performance. A total of five prototype and three production tests were completed for this project. A typical constitutive law test result plot is shown in Figure 7.

5 INSTALLATION

The VDDs were connected into the tower structure using steel fabricated tee brackets and a combination of high tensile stress bar and high strength bolts. Brackets were complimented with steel bearing plates to the opposite side of the beam from the bracket. Holes were cored through the existing beam and column flanges for the full depth of the sections. The bolts were fully grouted to provide positive bearing throughout their length. The VDD could then be lifted into place and fixed with pins.

Figure 8: Installation of Damper Connection Brackets

Figure 9: Viscous Dampers within Office Fitout

Due to the flat profile of the VDD, the geometry of the brackets were long and flat also. This resulted in a typical bracket that was connected to the tower beams with multiple bolts and a strut-and-tie load path developed with the bolts and steel plates. To minimise imposed deformations to the brackets, the brackets were also designed to strengthen the beams at the column face to ensure any beam hinging which could potentially develop was relocated away from the brackets. This was achieved by utilising the flange plates of the bracket to provide additional section to the beams and the bolts through the beams to provide the longitudinal shear path.

Given the multiple functions of the bracket arrangements, the stress distribution of the brackets and bolts was complex. In order to provide certainty in the flow of forces, finite element assessment was undertaken with Strand7 software.

Due to the reliance of the CES lateral system on adequate confinement, bolting of the brackets into the structure was selected over an initially proposed welding option. In addition to the bolts adding to the confinement, the bolts could be installed with minimal disturbance to the existing stirrups.

6 CONCLUSION

The Aurora Centre development project has led to the successful refurbishment of this large 1960s building which was otherwise nearing the end of its original design life. The seismic assessment and strengthening methodology used an advanced analytical approach, combined with an innovative strengthening methodology to achieve an efficient and effective outcome.

The use of a viscous damping solution provided the ideal seismic strengthening intervention for this building, as it effectively reduced building seismic drifts without adversely impacting the existing structural frame system. Not only have the viscous dampers been shown to be the ideal structural solution for this building, they have been effectively integrated into the architectural fitout and provide a visual cue to the strengthening works that have been undertaken, serving as a valuable reference point for building users in terms of the seismic resilience of their workplace.

The viscous dampers have proven to be a very cost effective seismic strengthening solution for this building, and together with the wider project redevelopment have brought a new life (and lifespan) into this previously austere 1960s tower.

7 ACKNOWLEDGEMENTS

The team at Holmes Consulting would like to acknowledge the client, Kiwi Property, for their interest in adopting innovative technologies into this important redevelopment project. Acknowledgement of the wider consultancy and contractor team is also warranted and without which this project would not have been a success. Key parties included;

Client: Kiwi Property

Architect: Warren and Mahoney

Project Manager: RCP

Contractor: LT McGuinness

Viscous Damper Fabricator/Sub-contractor: Victor Hydraulics

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