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STEEL MOMENT FRAME WITH SUPPLEMENTAL FLUID VISCOUS DAMPERS – 12 MOORHOUSE AVENUE A CASE STUDY

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ABSTRACT

Increasing building damping is an effective method of reducing seismic forces and improving building performance during earthquake events. The use of supplemental fluid viscous dampers in combination with a conventional moment frame structure is one application which provides a significant increase in structural damping, as well as providing low damage design.

This paper provides a case study of a five storey commercial building currently under construction in Christchurch where supplemental fluid viscous dampers are combined with a steel moment frame structure to provide a dual seismic resisting system. The paper outlines the analysis and design procedure, from preliminary through to detailed design, including incorporation of overseas literature and design codes for development of a suitable New Zealand based design procedure for buildings of this form. We will compare the performance of the structure designed using DDBD methods with the response of structure predicted by non-linear time history analysis.

Introduction

A five storey office building at 12 Moorhouse Avenue in Christchurch, that incorporates supplemental fluid viscous damping as part of the seismic resisting system, is nearing completion. This building replaces a two storey industrial building that was previously located on the site that sustained minor damage as part of the Christchurch earthquake sequence.

The design has been based on performance based design principles and utilises a low-damage design philosophy throughout to achieve the stipulated seismic performance criteria. To achieve these criteria, fluid viscous dampers were used in combination with a steel moment frame to reduce the peak response of the frame and the storey shears forces that would have been required for a conventional moment frame or braced frame design. This reduction in response and base shear demand enabled substantial cost savings in the foundation and superstructure to be realised.

Fluid viscous dampers (FVD's)have been used in New Zealand as lock-up devices on bridges and as components in tuned mass damper systems for wind sensitive buildings, but their use as part of the primary seismic resisting system of a building is relative unique here; this case-study being one of the first uses in a seismic application in New Zealand. This may be due to the minimal guidance available in respect to preliminary design and sizing, and optimisation of FVD's in seismic building design. Therefore, the authors have set out the direct displacement based design approach used for this project to assist other design practitioners who may want to apply this technology.

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Building Description

Structure Summary

The building comprises two towers (east tower and west tower), which are separated by a lobby area that provides for access and service corridors between levels. Both towers comprise five storeys (four suspended levels plus ground floor) with an apartment atop the roof of the west tower. The gross floor area of the building is approximately 6,970m^2, with the majority of the building being designed as commercial offices.

The building form consists of steel frame construction to give an open-plan office space, with seismic resistance being provided by steel moment resisting frames combined with the provision of diagonal braces in both principal directions of the frame. Non-linear viscous dampers are incorporated in the bracing system to improve the seismic resilience of the building by providing increased damping, hence reducing both inter-storey drifts, and floor accelerations. This also reduces the overall base shear and axial demand on the foundation system.

Gravity Structure

The suspended floors consist of a 0.9 mm thick ComFlor 60 composite steel/concrete floor system supported on primary and secondary steel beams. The overall slab thickness is 150mm.

The steel frame is set out on a centreline-to-centreline grid spacing of 7.5m by 9.0m. This is to achieve an open floor plate, uninterrupted by a large number of columns, and to be consistent with an efficient pile spacing which allows direct support of the columns. Inter-storey heights are typically 3.6m, with the ground floor being 4.1m.

Due to the potential for liquefaction at the site, and magnitude of the foundation design loads, the foundation system consists of 900mm diameter bored piles with a grillage of ground beams spanning over top. The foundation system was designed using a performance based design approach to give a consistence between building and foundation performance. The design and construction of the foundation system is described in Barounis (2015).

Lateral Load Resisting Structure

The lateral load resisting system is a dual system, incorporating fluid viscous dampers in combination with a steel moment frame. The seismic system is designed such that 70% of the design base shear is resisted by the moment frame and 30% of the design base shear is resisted by the FVD's. This split is denoted by the damping proportion factor, β, which is the proportion of base shear taken by the fluid viscous dampers, i.e. $\beta = 0.3$. This is discussed further in the seismic design procedure.

Additionally, the steel moment frame has been designed with sufficient strength capacity (i.e. ignoring drift limitation provisions of NZS 1170.5) to resist $0.75V_{base}$ of an equivalent moment frame without any supplemental viscous (or hysteretic) damping devices. This is required to satisfy the viscous damped moment frame provisions of ASCE 7-10, ensuring the moment frame has a natural resilience, and sufficient elastic strain energy to provide a re-centering force.

FVD's are provided on bracing lines distributed evenly about the building in both principal directions to provide a lateral load resisting structure with a centre of rigidity close to the centre of mass.

The layout of the FVD's and moment frames is shown diagrammatically in Figure 1.

The roof and floor diaphragms span horizontally to distribute load to lateral load resisting system at each level.

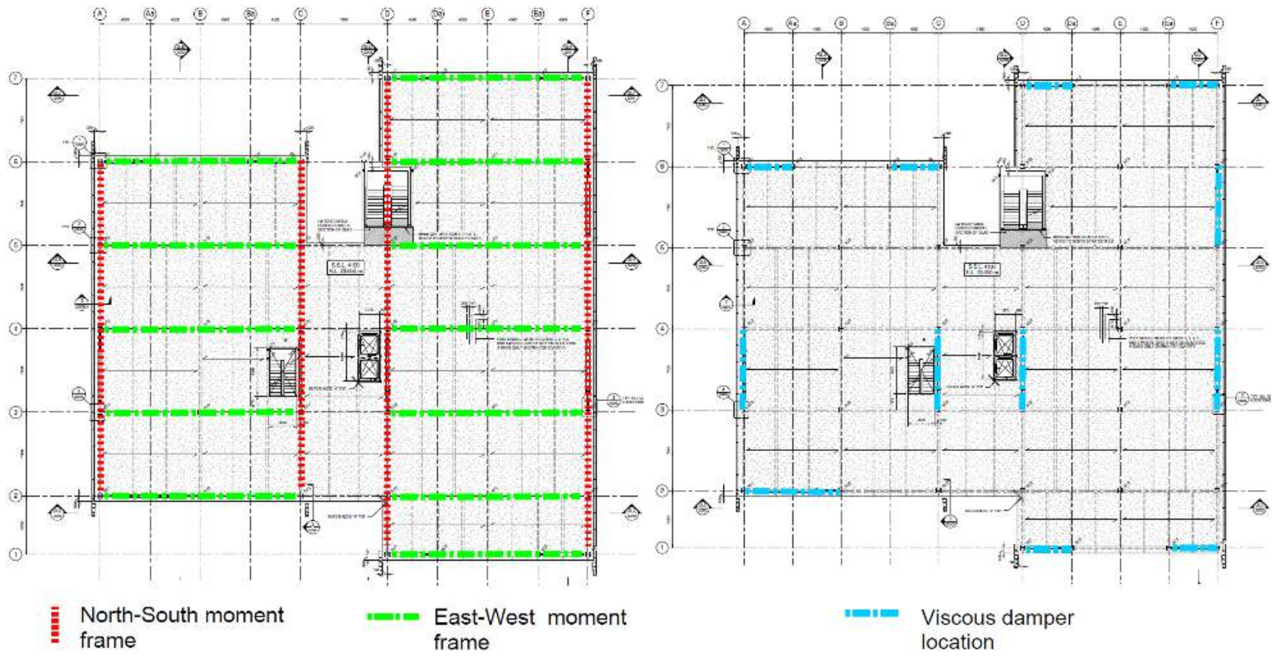

Figure 1: Structural layout

Fluid Viscous Dampers

Fluid viscous dampers typically consist of a cylinder with an internal piston that allows transfer of silicon oil between two chambers through orifices in the piston head. The devices become active during dynamic events when the displacement induced creates a relative velocity between each end of the device, and the energy input is converted to heat.

The force displacement relationship for a fluid viscous damper is primarily a function of the relative velocity between each end of the device. Damper force increases with stroke velocity, with the force-velocity relationship defined by Eq (1).

$$F_{damper} = CV^{\alpha} \tag{1}$$

where:

C = damping constant (KN/(m/s)

V = velocity (m/s)

α = velocity exponent ($0.15 \leq \alpha \leq 1.0$)

The velocity exponent defines the linearity of the damping relationship, with $\alpha = 1.0$ giving a completely linear force-velocity relationship, and the level of non-linearity increasing as the α-value decreases, refer Figure 2.

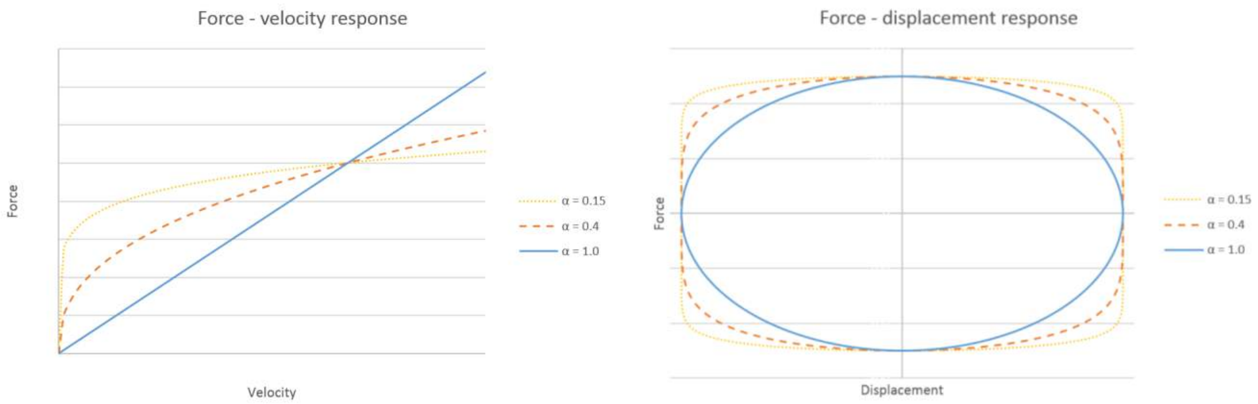

Figure 2: Viscous damper force-velocity and force displacement response

In a typical moment frame building the maximum seismic forces occur at maximum lateral displacement, however the building velocity is typically zero at this point as the displacement direction reverses, therefore the force in the dampers is almost zero. Maximum damper forces will occur at maximum velocity, which typically occurs when the building has around zero lateral displacement (i.e. close to its normal at rest position), and therefore the seismic forces in the frame are typically close to zero. Therefore the peak damper force is usually out of phase with the response of the MRF building

As the seismic induced velocity of building diminishes following an earthquake, the damper force diminishes, therefore provided the building frame has sufficient re-centering ability, the dampers do not restrict the building from re-centering back to its normal at rest position.

Seismic Design

The building has been designed as a moment resisting frame with supplementary fluid viscous dampers. The seismic design of the integrated system is described below. Additionally, the moment frame has been designed to have the capacity to resist at least 75% of the base shear required for standalone MRF system, as required in ASCE 7-10.

The building was designed using the direct displacement-based design (DDBD) approach, as proposed by Priestly et al., (2007), and more formally set out in Sullivan et al., (2012). Additionally, recent publications on the design of viscously damped systems, particularly by DDBD methods have also been relied, in particular, Sullivan and Largo (2012), and Largo et al., (2012). Further relevant publications are cited in the references section within.

Seismic Design Parameters

The elastic site acceleration spectrum is derived in accordance with AS/NZS 1170.5:2004, with site and building parameters of:

Site subsoil class:	D
Proximity to fault:	>100km.
Location:	Christchurch, Z = 0.3
Importance Level:	IL2
S_p factor:	0.7 (except that S_p =1.0 for assessment of the required maximum stroke of the dampers for ULS and MCE analysis).

Performance Criteria

The following seismic performance criteria were adopted for the design of the integrated system:

SLS:
- Limit damage to non-structural elements
- No damage to structural elements

ULS:
- Damage to non-structural elements
- Repairable damage to structural elements
- Potential residual drift limited

MCE:
- Substantial damage to non-structural elements
- Damage to structural elements, but repairable
- Collapse prevention

Preliminary Design

Because the force response of fluid viscous dampers is out of phase with the response of the moment resisting frame, preliminary design considering both acceleration and velocity response of the system, along with damping contributions of each component would be onerous. Therefore a simplified procedure is required. The design advice contained within the referenced literature gives little guidance on this interim step of the design process.

The approach adopted for this building was to simplify the problem by designing the moment resisting frame members to the ASCE 7-10 requirement to resist 75% of the base shear of an equivalent moment frame, using a system displacement ductility of μ = 3.0. For the preliminary sizing of the dampers, the approach adopted was to select the expected final overall structural damping value and reduce the elastic design spectra for this level of damping (typically in the range of 20 – 40%), then determine the building storey forces using the equivalent static method. The proportion of force carried by the dampers is given based on the selected β-value. For this building this approach gave a good estimate of the size of dampers required for the lower storeys, but overestimated the sizes for the upper floors, even when ignoring the 8% force lumped at roof level that is typically applied when using the equivalent static method.

Seismic Design of Steel Moment Frame

Design Base Shear

The design base shear of the moment frame without dampers was determined in accordance with ASCE 7-10, which requires that the moment frame is designed for a minimum base shear, V_{min}, of the greater than:

$$V_{min} = \frac{V}{B_{V+I}} \tag{2}$$ [5]

$$V_{min} = 0.75V \tag{3}$$ [5]

where:

V = seismic base shear in the direction of interest, determined in accordance with NZS1170.5;

B_{V+I} = numerical coefficient as prescribed in Table 18.6-1 (ASCE 7-10, second printing) for the sum of the effective viscous damping in the fundamental mode of vibration of the structural direction of interest, plus the inherent hysteretic damping of the moment frame of the first mode period.

For this building the $0.75V_{base}$ requirement governed the moment frame design.

Frame Design Approach

The design of the steel moment frame was carried out in accordance with the requirements of NZS 1170.5 and NZS3404 for a Category 2 structural system (μ = 3.0), with amplification on design requirements as given in Hera Report R4-76.

Through this design process the steel frame design was based on strength requirements only, with drift and serviceability requirements neglected, as per the approach of ASCE7-10 for a viscously damped moment frame.

Design of Viscous Dampers

The viscous damper design was carried out using a specific Direct Displacement Based Design (DDBD) approach adopted for this building, which allowed for sizing of the dampers without the need for significant iteration of the DDBD process. This is predominantly because the FVD's do not alter the stiffness of the moment frame, so altering the damper configuration does not affect the static mode shapes derived for the building. Verification of the DDBD was carried out by Non-linear Time History Analysis (NLTHA). The DDBD approach adopted consists of nine main steps, which are shown diagrammatically in Fig. 3. Explanation is provided on the specific design steps used where they vary from the normal DDBD approach.

Step 1 - Define the design displacement profile

The seismic design of the MRF was based on selection of a storey drift limit that is consistent with the performance requirements. For both principal directions the inter-storey drift limit of 2.0% was selected to achieve the performance requirements, including meeting minimum code requirements, limiting damage to structural and non-structural elements, and minimising residual drift of the building.

The target drift value was reduced, based on judgement, by 15% to allow for torsional effects in the 2-D DDBD. This gave a 2-D design displacement of $\Delta_d \approx 110$mm at the effective height. Following initial developed design and damper sizing this value should be reviewed and accounted for in the relative stiffness and locations of the dampers. During detailed design a 3-D NLTHA was undertaken to validate the design, including the effects

[5] ASCE 7-10, second printing

of stiffness eccentricities, and mass eccentricities, including accidental mass eccentricity.

Now, calculate the design displacement using the typical DDBD procedure, but with the displacement at level i given by:

$$\Delta_{i=}\omega_{\theta}\theta_c' \, H_i \left(\frac{p\,H_n - H_i}{p\,H_n - H_1}\right) \quad (4)$$ [6]

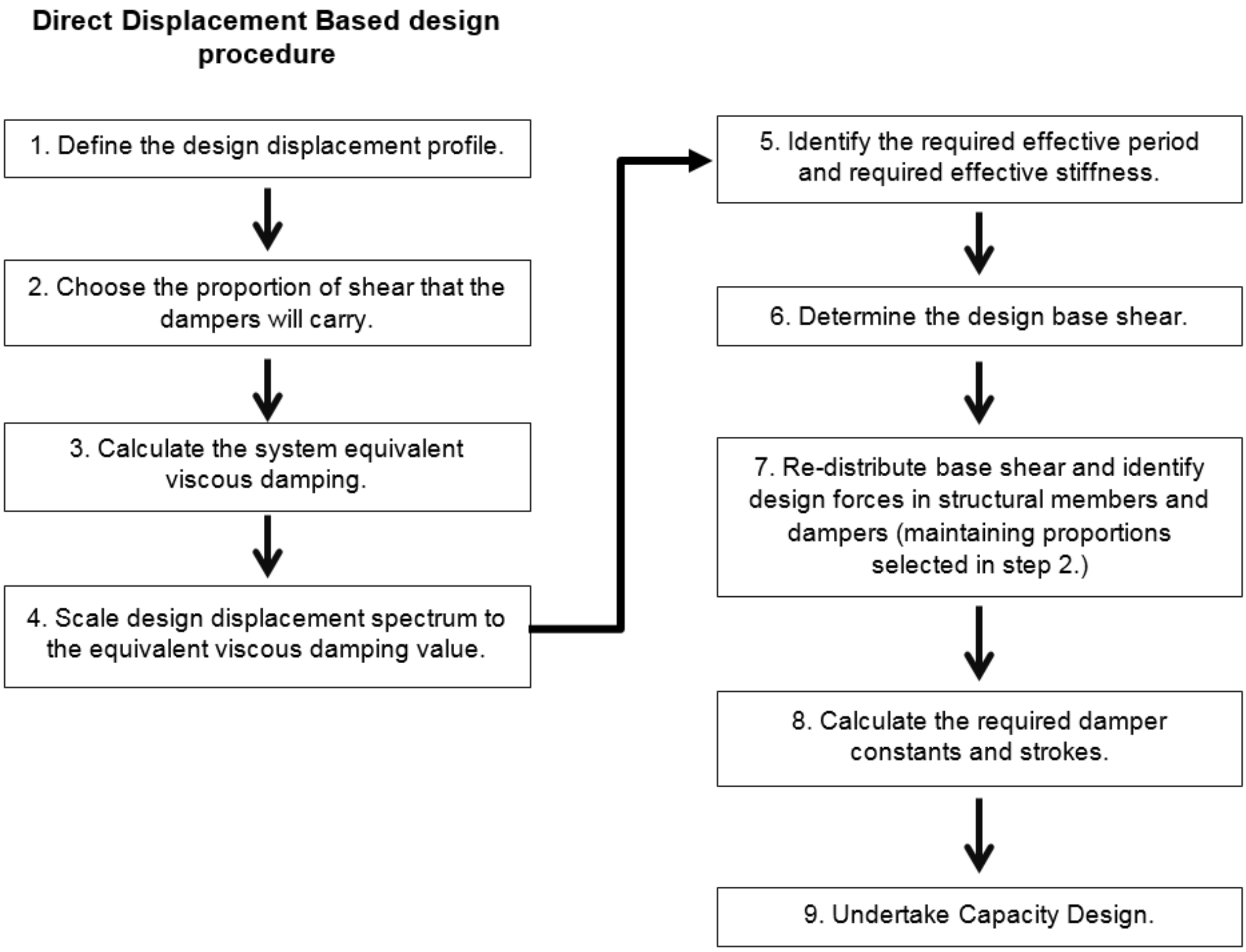

Figure 3: Adopted direct displacement based design procedure (after Sullivan, Largo 2012)

The reduction factor ω_θ accounts for higher mode effects on storey drifts and can be calculated from Sullivan, Priestley (2012), but for 6 storeys or less the value is equal to 1.

Generally, the constant p = 4 for frames (Priestley, 2007), but Sullivan, Largo (2012) suggests an alternative value is required for viscously damped frames to achieve the correct displacement profile. Sullivan, Largo (2012) suggest an initial value of p=2 as a trial value and the value should be optimised against the designed displacement profile. The actual displacement profile can be verified against the analysis model in Step 8, and the design adjusted as required.

The effective mass and effective height can be calculated using conventional DDBD procedures.

Step 2 - Choose the storey shear proportion for the dampers

The proportion of the design storey shear that each damper will resist, β_i, can be selected. This design used a constant value of β_i up the building, so β will be given by:

[6] Sullivan, Largo, 2012

$$\beta = \frac{V_{dam}}{V_{stru}} \tag{5}$$

This β value can be altered if a non-constant split of storey shear between the dampers and frame is required up the building. Optimising the seismic system will be an iterative process, balancing the relative benefit provided by increasing the proportion of design storey shear taken by the dampers, and hence the viscous damping provided, relative to the higher cost of larger damping units and associated larger capacity design actions.

β is set as the value between 0 (no contribution to resistance provided by dampers) to 0.6 (60% of system base shear resisted by dampers). Suggested values for regular steel frame buildings are typically in the 0.25 to 0.40 range.

The initial value selected for the design was β = 0.4.

Step 3 – Calculate the equivalent viscous damping

The equivalent viscous damping of the SDOF system was determined by calculation as follows.

$$\xi_{eq} = \xi_{eq,st} + \frac{\beta}{1+\alpha}$$ (6)[7]

Where $\xi_{eq,st}$ is equivalent viscous damping offered by the structure (excluding dampers) for the design displacement of the system:

$$\xi_{eq,st} = 0.05 + 0.577\left(\frac{\mu-1}{\mu\pi}\right)$$ (7)[7]

where μ is the ductility demand at the design displacement of the frame. The yield displacement for both primary directions was verified against a bi-linear force-displacement response for the lateral system obtained by pushover analysis of the initial design.

The equivalent viscous damping of dampers is given by:

$$\frac{\beta}{1+\alpha}$$ (8)[7]

where β is the ratio of the shear resistance offered by the dampers to the base shear, as selected in step 2, above; and α = viscous damper velocity power constant, with typical values of α falling within the range of ($0.15 \le \alpha \le 1.0$). A value of $\alpha = 1.0$ gives a linear viscous damper response, whereas values of $\alpha \le 1.0$ gives a non-linear viscous damper response, refer Fig. 2.

For this building a value of α = 0.4 was adopted to try to maximise the non-linear viscous damping response. This will reduce the base shear and capacity design actions compared to linear viscous dampers, and will maximise the damping forces that can be obtained at the peak displacement response of the structure. The steel frame has an approximate yield drift of θ_y= 1.4%, so limited hysteretic damping was provided the structure at the peak displacement. Therefore a lower value of α will help to maximise the damper equivalent viscous damping at the peak displacement in order to maximise the benefit of the dampers.

The α value is dependent on what can be supplied by the various damper manufacturers.

Step 4 – Scale the displacement spectra

Modify the elastic displacement spectrum (obtained by conversion of elastic acceleration spectrum of 1170.5), to obtain the displacement spectra for the equivalent viscous damping value calculated above, allowing for a highly damped structure, responding principally elastically. The following response spectrum modification factor is suggested for highly damped structures (Sullivan, Priestley, 2012):

$$R_{\xi} = \left(\frac{0.10}{(0.05+\xi)}\right)^{0.5}$$ (9)[7]

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[7] Sullivan, Priestley, 2012

This will give a more conservative spectrum than modification factor of $R_\xi = \left(\frac{0.10}{(0.02+\xi)}\right)^{0.5}$ that is typically used for structures responding inelastically (NZSEE Red Book, 2006). For the initial design, only far-field effects were considered, i.e. the power factor in the modification factor of 0.5 is used, rather than a value of 0.25 that is suggested for near field sites, where forward directivity is possible. Near-field effects have been shown to be relevant for sites in Christchurch, but experience suggests that far-field effects will govern the design. Therefore, a displacement spectrum modified for far-field effects only was used to develop the initial design, and the performance of the design was checked by NLTHA, which incorporated a number of near-field earthquake records.

Step 5 – Calculate the effective period and effective stiffness

The effective period, T_e, for the SDOF system at the design displacement was determined from the displacement spectrum and then the effective stiffness of the system K_e was calculated in the normal way.

Step 5 – Calculate the design base shear

The design base shear, including P-Δ effects, can be calculated as:

$$V_b = k_e \Delta_d + V_{p-\Delta} \qquad (10)^8$$

For P-Δ effects, check that:

$$\frac{m_e g}{k_e H_e} < 0.05 \qquad (11)^8$$

and if so, ignore as insignificant, otherwise:

$$V_{p-\Delta} = C\frac{\sum_{i=1}^{n} P_i \Delta_i}{H_e} \qquad (12)^8$$

Step 7 – Calculate the design member forces and design damper forces.

The base shear force was distributed up the building with higher mode effects accounted for by lumping 10% of the design base shear at the roof.

The proportion of the storey shear assigned to the dampers *β,* was determined in step 2, so the damper force is given by $F_{i'd}$ for each level.

$$F_{i,d} = \beta V_i$$

$$= \beta \sum_{j=i}^{n} F_j \qquad \text{i.e. } \Sigma F_j = \text{design storey shear on } i^{th} \text{ level}$$

Step 8 – Calculate the required design damper constants and stroke.

The peak damping force of non-linear dampers is given by the equation:

$$F_d = C\left(\frac{2\pi\Delta_d}{T}\right)^{\alpha} \qquad (13)$$

Where *C* is the viscous damper constant and *α* is the viscous damper power constant, refer Fig. 2.

Therefore the design viscous damper constant can be determined for the dampers located at each level by:

$$C_{d,i} = F_{i,d}\left(\frac{T_e}{2\pi\delta_{di}}\right)^{\propto} \qquad (14)^8$$

Where δ_{di} = displacement damper demand at level *i.* Note that this is not the storey displacement demand, but the damper displacement demand, which can be easily calculated based on the configuration of the damper. Rigid body rotation should be considered when calculating the displacement demand of the damper

[8] Sullivan, Priestley, 2012

as this component of the storey drift will not contribute to the damper stroke, refer Fig. 4. Allowance for this will reduce damper stroke, and hence viscous damping provided at each level, and further guidance on how to allow for this is given in Maley (2010).

Once this system is analysed, the displacement profile of the integrated system, and effective total damping of the system can be compared to the design values and adjustments made, if necessary, then step 1 to 8 repeated until the analysed values match design values.

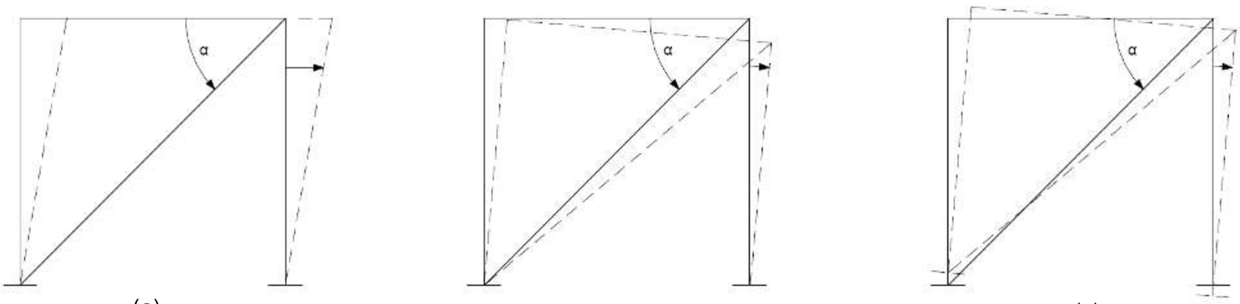

Figure 4: Viscous damper displacement components: (a) brace elongation, (b) column axial deformation at level under consideration, (c) rigid body rotation from axial deformation (after Maley 2010)

Step 9 – Undertake capacity design.

Having determined the storey shear demand on the frame, the frame can be designed to provide the stiffness to achieve the displacement profile defined in step 1 above (or the displacement profile can be modified and the DDBD process can be refined through iteration).

Having determined the required strengths at potential plastic hinge locations, capacity design for the structure can be undertaken using normal capacity design procedures. For the dampers and their connecting elements, the overstrength force that can be developed will depend on the maximum velocity that can be developed. FEMA 356 suggests the maximum velocities will be between 1.3 to 2.0 times the velocity associated with the first mode response to the maximum considered earthquake. For this project the over-velocity factor was applied as set out in EN15129 is given as:

$$\gamma_v = (1 + t_d) \cdot (1.5)^{\alpha} \qquad (15)^9$$

Where t_d is the design reaction tolerance given by the manufacturer. The typical value of the reaction tolerance is t_d = 0.15, which gives a maximum velocity consistent with the range given by FEMA 356. This value has been adopted for the preliminary design.

Analysis

Non-Linear Static Analysis

As noted above the design was completed using simple static analyses based on DDBD principles, and non-linear methods were used to verify the design and assumptions made about the expected response of the building, including torsional response and equivalent viscous damping.

Initially, a non-linear static pushover analysis was performed using 2D models of the steel MRF for both principal directions of the building. This was a carried out in ETABs using elastic frame elements with lumped plasticity to represent hinges in the beams and at the column base.

From the pushover model a force-displacement curve was obtained for both orthogonal directions of the building at the effective height. This was used to develop a bi-linear force displacement response for the frames by fitting elastic and post-elastic lines to the force displacement curve, ensuring that the area under the bi-linear approximation equals the area under the force-displacement curve. Refer Fig 5.

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[9] EN15129, 2009

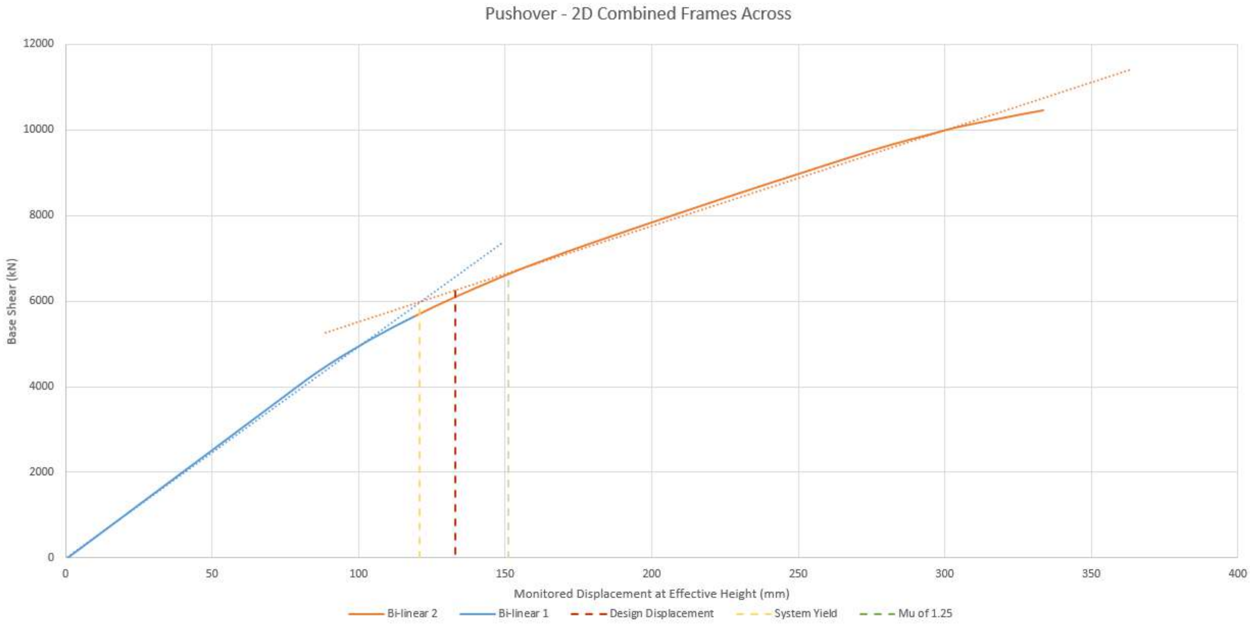

Figure 5: Force displacement curve of transverse frames only

This bi-linear curve was then used to set the design displacement (for the effective height at the ULS), considering that the displacement ductility demand on the frame was limited to minimise the plastic strains in the first formed hinges in the frame to a level that was consistent with the design philosophy. The bi-linear curve was also used to ensure that there was sufficient elastic strain energy in the frame to provide sufficient restoring force to prevent ratcheting and reduce residual drifts at MCE. Through this process a design displacement that gave a displacement ductility for the frame alone of approximately 1.25 was selected.

Earthquake Record Selection and Scaling

A series of 14 ground motion pairs were used for the NLTHA to provide a thorough assessment of the seismic performance of the building and allow for consideration of the expected building response. This also allowed the application of the ASCE 7-10 approach of averaging of the results of at least seven records rather than considering the maximum of three records, as per NZS1170.5. However, we note that the displacement critical elements, such as the maximum damper stroke, were not taken as the average, but rather the maximum of the records considered.

The records used are shown in Table 1 and they are grouped in four sets.

Table 1. Selected Earthquake Records

Set	Event	Station	Year	Mag	PGa (g)	PGV (cm/s)
1	CHC 22/02/2011	CBGS	2011	6.2	0.529	33.4
	CHC 22/02/2011	CCCC	2011	6.2	0.483	18.6
	CHC 22/02/2011	CHHC	2011	6.2	0.336	14.3
	CHC 22/02/2011	REHS	2011	6.2	0.713	23.9
	CHC 22/02/2011	D06C	2011	6.2	0.229	53.3
2	Northridge	Sylmar Converter	1994	6.7	0.795	130.3
	Chi-Chi, Taiwan	CHY101	1999	7.6	0.452	108.5
	Turkey	DUZCE	1999	7.1	0.519	79.5
3	Northridge	Camarillo	1994	6.7	0.117	13.1
	San Fernando	Glendale	1971	6.6		
	Chi-Chi, Taiwan	CHY015	1999	7.6	0.176	26.8

4	Loma Prieta	Hollister – South & Pine	1989	6.9	0.301	56.6
	Landers	Amboy	1992	7.3	0.147	19.7
	Chi-Chi, Taiwan	TCU065	1999	7.6	0.823	127.8

Set 1 – Five un-scaled records from the CBD for the Christchurch earthquake, selected to understand how the building could be expected to perform relative to the observed performance of buildings in Christchurch. Considering the high spectral acceleration and displacement demand of these records, well in excess of the 500 year design level, and in most cases also of the 2500 year design level, it could have been anticipated that these un-scaled records would result in a drift demand well above the MCE design drifts of approximately 3.0% and 3.6% for 2D and 3D analysis models respectively (2D drift is reduced to allow for torsional effects). However, the predicted drift of the 2D and 3D model was still within this limit, as shown in Figure 6.

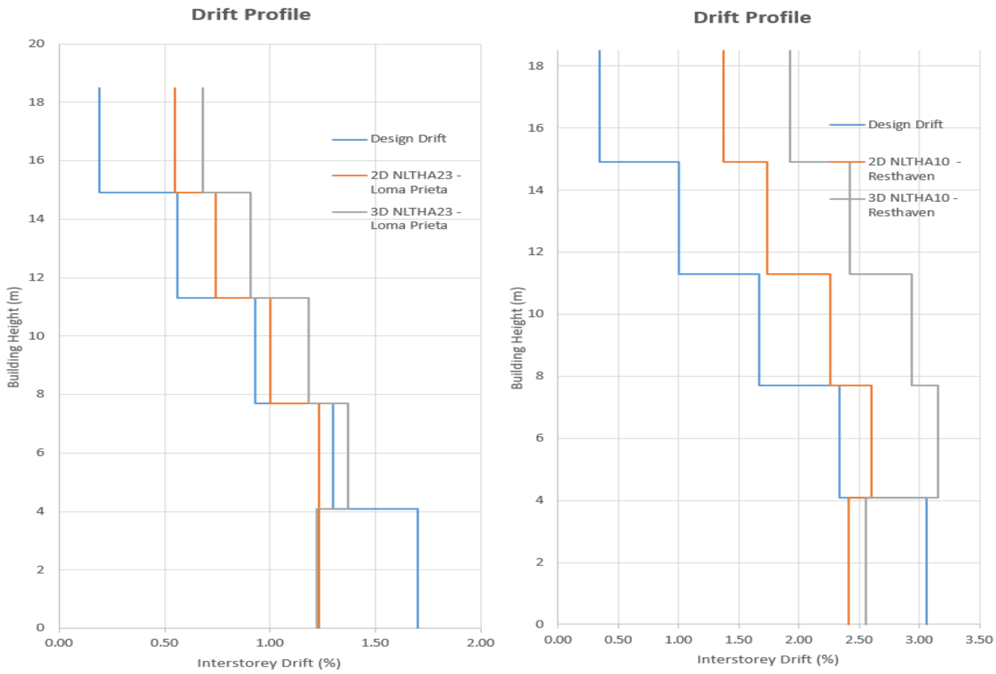

Figure 6: Inter-storey drift profile and ULS and MCE, respectively

Set 2 – Three records featuring strong near-source effects (fault to site distance < 30km) from different shallow crustal events worldwide. All records within this set are scaled to the ULS (500 years return period) response spectra in accordance with NZS1170.5.

Set 3 – Three records featuring far-field properties (fault to site distance >30km), significant magnitude (M>6.5) and significant duration. All records within this set are scaled to the ULS (500 years return period) and MCE (2500 years return period) response spectra in accordance with NZS1170.5.

Set 4 – Three records featuring far-field properties (fault to site distance >30km), significant magnitude (M>6.5) and significant duration. The disaggregated seismic hazard model for Christchurch shows that for major events with larger magnitudes will dominate long period vibration structures i.e. T_1>1s - refer Tarbali (2014). Therefore, records have been selected in this set that represent rupture of the Alpine, Hope and Porters Pass fault sources as per Tarbali (2014), with weighting given to those records with higher peak ground velocity (PGV). All records within this set are scaled to the ULS (500 years return period) and MCE (2500 years return period) response spectra in accordance with NZS1170.5.

Records sets three and four were used for analysis at both ULS and MCE, while record sets one and two were used for analysis at MCE and ULS respectively.

Non-Linear Time history Analyses

Non-linear time history analyses (NLTHA) were first carried out on two-dimensional (2D) models initially to ensure that modelling results were compatible with expectations from the DDBD, particularly with respect to damping contributions from the FVD system. This was done by calculating energy components for the peak (in terms of drift demand at the effective height) ULS cycle based on initial and final conditions of the system and force, velocity, and displacement outputs during the peak cycle.

This analysis confirmed that the viscous damping provided by the FVD was approximately 32% of critical damping, which was consistent with the value used in the DDBD. The hysteretic damping provided by the frame was simply taken from the force-displacement profile of the frame without the dampers. The combined systems gave an equivalent viscous damping of approximately 36%, which aligned with that used in the DDBD.

Additionally there was relatively good alignment between the storey drifts used in the DDBD and 2D NLTHA, as shown in Fig 5. The DDBD procedure tended to overstate the storey drifts at the lower (critical) levels and understate the drift at the up levels relative to the NLTHA.

The design was finally verified on three-dimensional models for the whole set of acceleration records, to consider the effects of bi-directional excitation, stiffness eccentricity and accidental eccentricity. The response is evaluated in terms of structural deformations, as storey drift up the building compared to the DDBD drift profile in Fig. 5 for both ULS and MCE limit states. The corner displacements from the 3D NLTHA are shown for an average ULS and peak MCE record relative to the DDBD design profile in Fig 7. The relative maximum floor accelerations for an average ULS and peak MCE record are also shown in Fig 7.

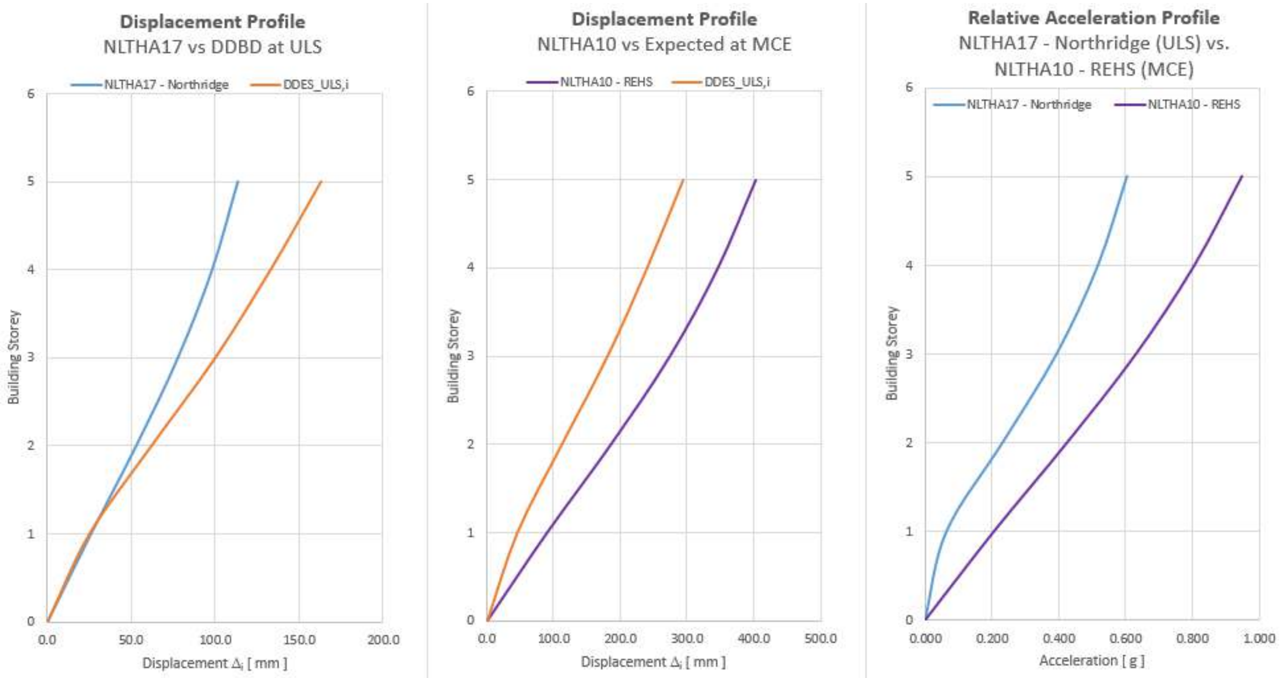

Figure 7: Storey displacements for ULS and MCE records, and
Inter-storey drift profile

When considering bi-directional excitation, the stiffness eccentricity and accidental eccentricity, the 3D NLTH analysis showed an increase in the drift demand from the results obtained with 2D planar models, as was expected. This increase was approximately 15% at ULS, as was anticipate, but increased to 25% as that largest storey increase at MCE, which was larger than anticipated and allowed for in the 2D DDBD approach.

This increase in drift by such an amount was unexpected, as the building is relatively regular with only a small difference between the centres of mass and of stiffness. This is likely to be due to the viscous dampers, providing little stiffness at the peak drift (as velocity approaches zero) even when positioned around the perimeter of the building, and reduced stiffness in the perimeter moment frames as yielding becomes more pronounced at for the MCE event.

The maximum inter-storey drift obtained by 3D NLTHA at ULS is 1.35% which is less than the target drift of 2.0% and well inside the code limit of 2.5%.

The peak floor accelerations have been evaluated to estimate the magnitude of the diaphragm forces and hence the strength required at connections of the latter with the lateral resisting elements (frames and dampers). The peak floor velocity has also been evaluated to ensure that, at MCE, the peak damper velocity is within the maximum manufacturing limit specified of 300mm/s.

Detailed Design and Construction

Detailed Design

The FVD's are typically delivered as a proprietary unit, and the design for the connecting steel brace and gusset connection is the designer's responsibility. The brace design was completed in accordance with NZS3404, using element category and designing for the over velocity actions from the damper. Likewise, the gusset was design using a capacity design approach, but using the uniform force method of AISC 2012.

The FVD's require a pinned connection at (at least) one end to prevent damage to the damper unit. For this project damper units were supplied by Taylor Devices and incorporated a spherical bearing in the clevis plate to allow the building to drift in the direction perpendicular to the primary axis of the brace without causing damage to the damper piston and seals. For this project both ends of the brace were pinned due to aesthetic reasons, but is common practice to weld or bolt the connection at the 'dead' end of the brace in a similar manner to connections for a BRB.

The use of gusset plates with a ductile moment frame design did require consideration that effective rigid zone that the gusset creates beyond the column face, effectively pushing the critical region of the plastic hinge to the end of the gusset plate.

One significant advantage of this form of braced lateral system is that it allows the construction time on site to be significantly reduced. Firstly the use of a moment frame that is designed for 75% of the base shear allows the steelwork to be erected without any construction bracing to temporary lateral support. Secondly, the dampers are a long lead item (approximately 26 weeks for this building), so the majority of the building could be completed without dampers in place, and these were installed at a later date.

To maximise the speed of erection on site the frame was designed as a "column tree", with beam moment joints welded in the shop, and simple bolted beam splice connections used to erect the frame on site. Columns were delivered to site and stood up as a single unit, to allow the maximum amount of work in the shop and do away with time consuming column splice joints on site. Figure 8 and 9 show the columns being erected and the completed frame with the dampers in place.

Figure 8: Erection of columns

Figure 9: Frame with the viscous dampers in place

Conclusion

The use of fluid viscous dampers in combination with steel moment frames provide substantial benefits in terms of building performance. Whilst this technology has seen little application in New Zealand it has wide acceptance as part of a seismic system in the US, Asia and parts of Europe. With a move to greater focus on building performance and low damage design in New Zealand, the use of FVD's offers a solution that is cost effective and merits greater consideration by designers, both in new builds and seismic retrofit projects.

The use of FVD's on this case study project reduced the seismic demand on the steel moment frame, by providing high levels of equivalent viscous damping of the system, providing substantial benefits in both the size and cost of the steel elements of the superstructure and of the foundation system.

FVD's also provide a substantial reduction in floor accelerations and drift over an un-damped frame or braced frame systems, and in that sense can be considered a true low-damage solution that improves overall building performance. Only base isolation would be likely to provide a better level of seismic performance.

However, practitioners should be aware that viscously damped solutions are an Alternative Solution. Additionally, due to overall system having elements with force relationships that are a direct function of both velocity and acceleration, the design solutions are slightly more complex than conventional streel frame designs that are solely force based.

This paper summarises a practical design approach that can be used to quickly arrive at preliminary sizing of members and dampers in viscously damped steel structures. Viscous dampers are able to be modelled directly in combination the non-linear response of the steel frame by direct integration non-linear time history via commonly used analysis packages, such as ETABs (Ultimate) and SAP2000 (Advanced). This allows verification of the design to be undertaken by many practitioners, albeit with the usual caveats around the use of "black boxes".

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