Seismic and Financial Performance of Fluid Viscous Dampers relative to BRBs: A Case Study

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ABSTRACT: The use of Buckling Restrained Braces (BRBs) as the primary lateral load resisting element in new and retrofitted buildings in New Zealand has become increasingly popular. However, the use of fluid viscous dampers (FVDs) as a supplemental damping system for steel framed buildings has not been widely considered, and their use in New Zealand has been extremely limited. The FVD system can offer substantial performance benefits for negligible additional cost, and in some instances an overall cost reduction in the total building cost.

This paper will briefly outline the performance and cost differences between these two systems for a recently completed building that Opus have designed using FVDs. We will demonstrate the relative performance differences using a non-linear time history analysis, in terms of both drifts and floor accelerations including a relative comparison of the residual drifts and cost benefits that viscous dampers offered over buckling restrained braced frame systems.

1 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.

At the preliminary design stage the use of various bracing systems, such as Buckling Restrained Braces (BRBs) were considered for the lateral load resisting system. However, these were found to provide reduced seismic performance compared to fluid viscous dampers (FVDs) and, surprisingly, had a greater total construction cost than FVDs. Fluid viscous dampers used in combination with a steel moment frame reduced 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 an overall cost saving to be realised.

To understand the differences in performance as well as overall cost, the design of a BRB system for this building has also been completed and non-linear time history analyses (NLTHA) carried out to compare the relative performance of the two systems.

2 BUILDING DESCRIPTION

2.1 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). The gross floor area of the building is approximately 6,970m^2, with the majority of the building being designed as commercial offices.

2.2 Gravity Structure

The suspended floors consist of 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, with 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 for the FVD solution 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 consistency between building and foundation performance. The design and construction of the foundation system is described in Barounis (2015).

2.3 Lateral Load Resisting Structure – Dual Fluid Viscous Damper and Moment Frame System

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.

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.

The FVD system and steel moment frame was designed using direct displacement based designed techniques. The design process used is described in detail in Brown (2015).

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 and the typical FVD frame elevations are shown in Figure 2.

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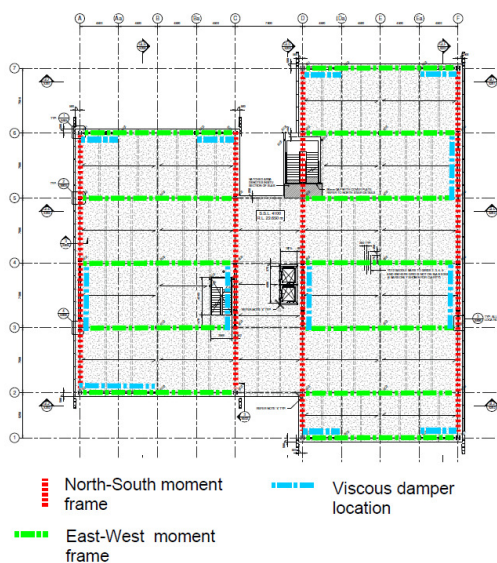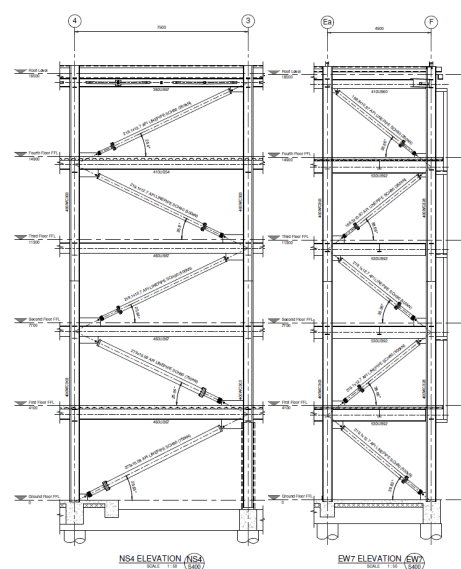

Figure 1: Structural layout

Figure 2: Typical FVD Frame Elevations, North South direction on left, East West direction on right

2.4 Lateral Load Resisting Structure – Buckling Restrained Brace System

For the Buckling Restrained Brace (BRB) system, the base shear is resisted by the braces alone. The steel frame is primarily designed to support gravity loads, and typically has simple pinned type end connections, therefore the frame does not provide any meaningful contribution to the lateral load resisting system.

BRB's are provided on bracing lines to match the FVD layout shown in Figure 1.

Table 1: Comparison of typical brace forces - East West direction

Level	FVD		BRB	
	ULS axial force (kN)	Overstrength force, P_{os} (kN)	ULS axial force (kN)	Overstrength force, P_{os} (kN)
Fourth Floor	250	338	341	465
Third Floor	250	338	550	804
Second Floor	500	676	838	1206
First Floor	500	676	1022	1565
Ground	500	676	1153	1917
Foundation force (vertical component)	**-**	**1723**	**-**	**3814**

3 SEISMIC SYSTEM

3.1 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).

$$F_{damper} = CV^{\alpha} \qquad \text{where:} \qquad (1)$$

$$C = \text{damping constant (KN/(m/s))}$$
$$V = \text{velocity (m/s)}$$
$$\alpha = \text{velocity exponent } (0.15 \leq \alpha \leq 1.0)$$

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 3.
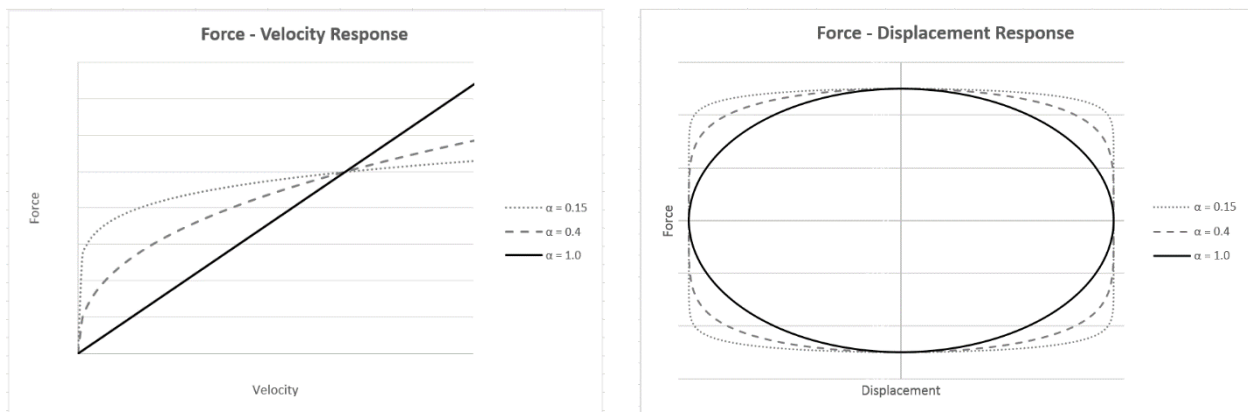
Figure 3: Viscous damper force-velocity and force displacement response

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.

For this design, viscous dampers supplied by Taylor Devices with $\alpha = 0.4$ were used, giving an equivalent viscous damping of approximately 34% of critical damping for the combined system. The damper capacities are shown in Table 1.

3.2 Buckling Restrained Braces

Buckling restrained braces typically consist of a yielding steel core member that is encased in concrete and an outer steel casing. The steel core provides resistance to tension and compression loads and is prevented from buckling under compression loading by the concrete and steel casing. De-bonding of the steel core from the concrete encasement is required to allow axial displacement as the core resists load and yields.

For the BRB's and their connecting elements, the overstrength force that can be developed will depend on the expected maximum strain demand on the steel core. The brace overstrength force can be up to or greater than two times the brace design capacity.

The BRB system was designed using the direct displacement based design procedure, with the same target displacement at the effective height as the FVD system. This gave an equivalent viscous damping of 16% and a displacement ductility of $\mu \approx 5.5$ for the design (ULS) event.

100% of the lateral force demand is taken by the BRB systems, as simple connections are used in the structural frame, which offer no moment resistance.

3.3 Differences in Design between Systems

The differences in the design of structural elements between the two systems is outlined in Table 2

below. These differences arise from the use of a moment frame and lower overstrength values used for the FVD system, compared to pinned frame and higher overstrength values for the BRB system.

Table 2. Design Differences between FVD and BRB structural systems

Item	FVD system	BRB system	Comment
Structural Frame	Beam-to-column fully welded moment connections	Beam-to-column simple shear connections	
Beams - gravity	510UB98 No pre-camber	510UB98 Pre-cambered	
Columns	400WC303	400WC270	39 No. each
Collector Beams/Diaphragm and Gussets	-	-	Not explicitly checked but greater demand for BRB system
Piles	39 No. 900mm diameter bored concrete pile	39 No. 1200mm diameter 8 No. 900mm diameter bored concrete pile	Additional tension anchorage required for BRB not included
Design Process	Non-linear time history analysis required	Linear response spectrum analysis sufficient	

4 ANALYSIS

For the purposes of the comparative study the building was modelled in 2D using ETABs 2015. Non-linear time history analyses were performed on each building with non-linear elements used for the BRB's, viscous dampers, and moment frame connections.

Each of the models were analysed using a total of 14 ground motion pairs selected based on the seismic sources that contribute to the hazard for Christchurch. However, results from only eight records are presented within for clarity and brevity purposes. The records used are shown in Table 3 and they are grouped in two sets.

Table 3. 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	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

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. In most cases these records represent the MCE (2500 year return period) or greater event.

Set 2 – 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) response spectra in accordance with NZS 1170.5.

Non-linear time history analyses (NLTHA) were carried out on two-dimensional (2D) models for the purpose of comparison of building performance between the FVD and BRB systems.
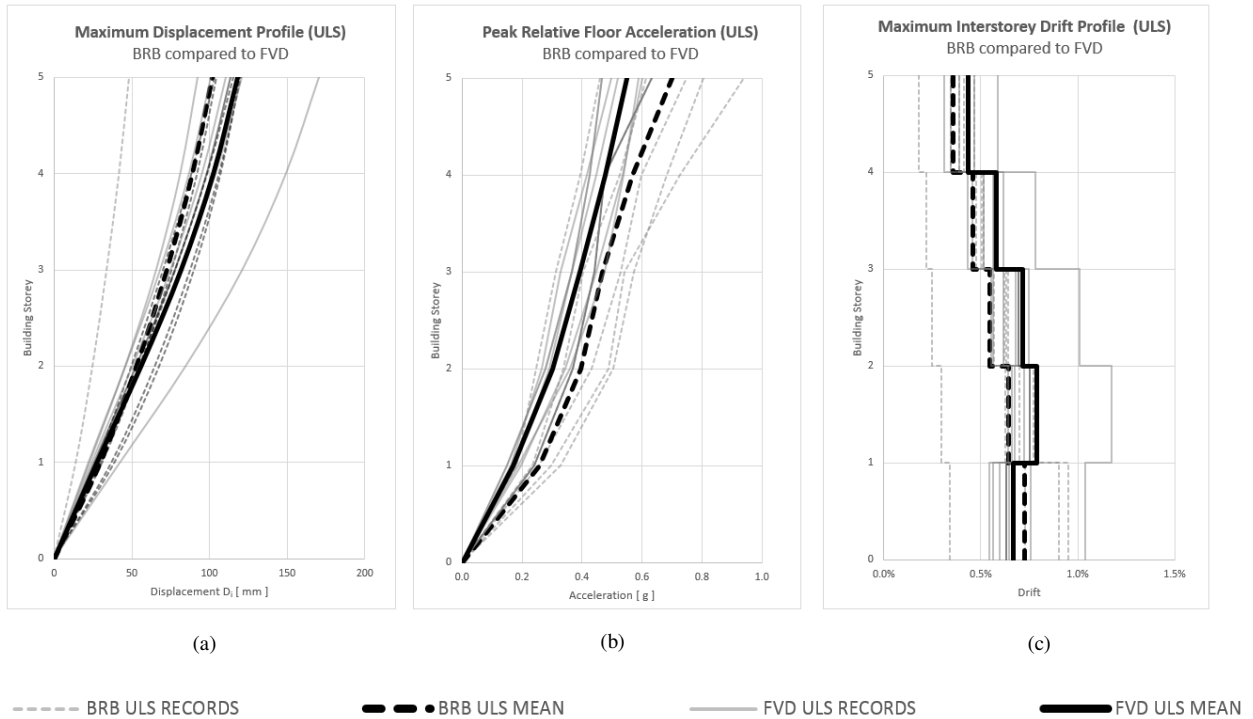
4.1 Drift and Acceleration Comparison

Figure 4: Ultimate Limit State (Design Case) results of 2D NLTHA comparing (a) maximum displacement profile, (b) relative floor acceleration peak and (c) maximum inter-storey drift

Figure 5: Christchurch Earthquake CBD Maximum Considered Event (Design Case) results of 2D NLTHA comparing (a) maximum displacement profile, (b) relative floor acceleration peak and (c) maximum inter-storey drift
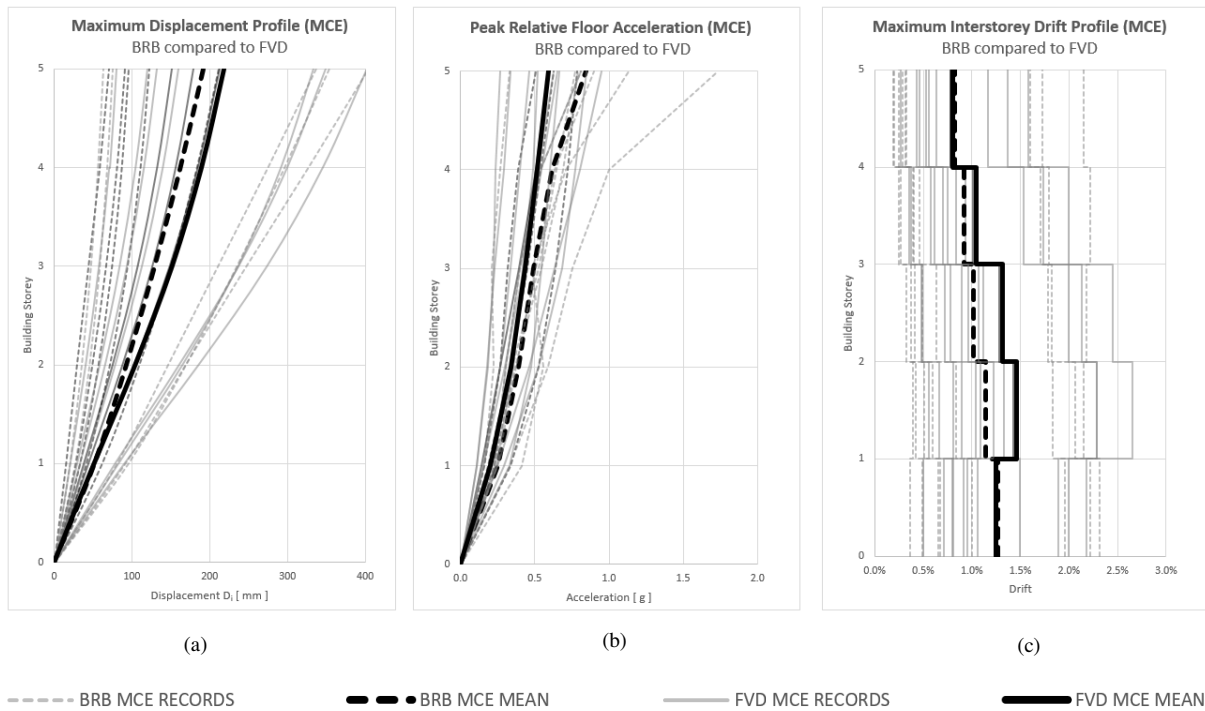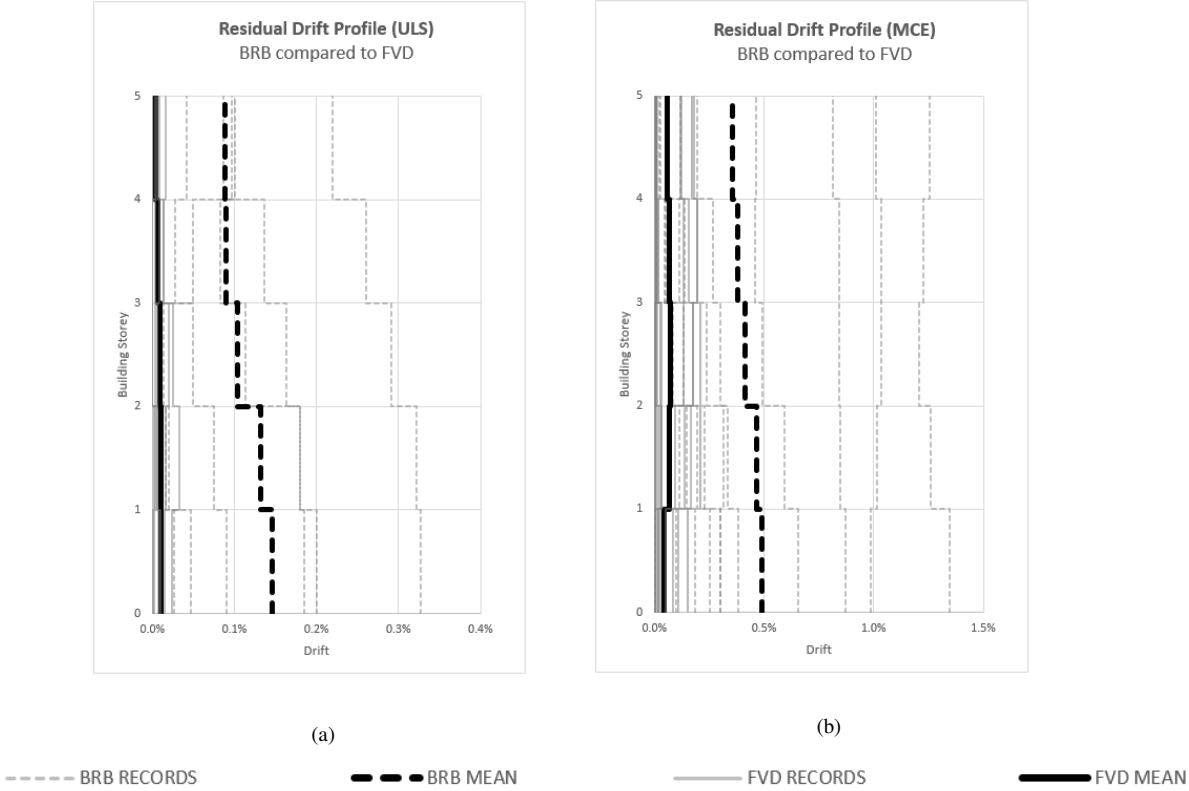

(a) (b)

Figure 6: Maximum Residual drift profiles of each storey for BRB and FVD systems (a) ULS case and (b) MCE case

The primary results from the non-linear analyses are shown in Figs 4 and 5. It can be seen that the maximum displacement at ULS for both FVD and BRB systems is similar, as would be anticipated given that both systems have been designed by DDBD methods to the same target drift.

The inter-storey drift profile, likewise, is similar at both ULS and MCE limit states. The most significant difference between the two systems is peak zero period floor accelerations, particularly for the upper floors, where the ratio of the BRB to FVD floor accelerations is 1.5 (0.94g/0.63g) and 1.8 (1.73g/0.95g), at ULS and MCE limit states respectively.

The other significant difference between the two systems is the residual displacements, particularly for the MCE event. Figure 6 shows that the average of the residual drift at both ULS and MCE is approximately an order of magnitude greater for the BRB system than for the FVD system. Figure 6 also shows that for the design level event (ULS) the residual drift in the FVD system is negligible (approximately 3mm for the peak displacement at the top floor). Whereas for the BRB system, the residual drift is significant, up to a peak value of 54mm at the top floor.

4.2 Relative Performance

The performance of the system can be evaluated by considering damage to the structural system that is difficult to repair after a severe earthquake, damage to the non-structural components, and damage to the building contents. Residual drift, inter-storey drift, and floor acceleration are important variables that can be used to broadly evaluate the relative performance of the structural faming system.

Given that the inter-storey drift is similar between the two systems, it can be expected that the damage to the structural frame facade, partitions etc will be similar between the two systems. The floor accelerations are greater for the BRB system, so it can be could be expected that building content, secondary structure such as stairs, and fit out items such as lifts, that are acceleration sensitive, will sustain greater damage than for the FVD system.

The most significant difference between the performance of the two systems is the residual drift. It was anticipated that this would be more significant for the BRB system at the MCE event, but, surprisingly, the residual drift at the design level (ULS) event indicates that significant structural repair would be required to rectify the residual drift as it exceeds the allowable verticality tolerance for steel construction.

5 COST

The cost difference between the two systems has been estimated based on the differences in the structural system noted above. The differences, which are itemised in Table 4, equate to a premium for the BRB system of approximately $380,000 (NZD), or 3% of the total construction cost (and a much greater percentage of the structural cost).

The piles beneath external bracing lines for the BRB system need to have significant tension capacity. This has not been allowed for in the costings below, and this would increase the cost of the BRB system further. The estimated cost to achieve sufficient tension capacity from the piles is $352,000, which gives an overall cost saving of $733,000 (NZD) for the FVD system.

Another significant advantage of this form of the FVD 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 provide 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. The costs associated with P&G savings such as this are not accounted for, but would increase the relative cost saving for the FVD system.

Table 4. Itemised Cost for FVD Compared to BRB Systems

Component	FVD		BRB	
	Details	Cost	Details	Cost
Piles	39 no. 900 diameter	$731,250	31 no. 1200 diameter, 8 no. 900 diameter	$1,390,000
Columns	39 no. 400WC303	$1,237,358	39 no. 400WC270	$1,102,596
Seismic System	55 dampers	$434,844	55 BRB's	$291,879
TOTAL	-	**$2,403,452**	-	**$2,784,476**
Cost Difference	-	**($381,024)**	-	-

6 CONCLUSIONS

In terms of drift damage both systems are expected to produce a similar level of structural and non-structural damage. However, damage to acceleration sensitive items is expected to be greater for the BRB system, particularly in the upper levels. Additionally the residual drift is a significant performance issue for the BRB frame, even at ULS.

Based on this it can be concluded that the FVD system offers better seismic performance than the BRB system for less overall cost.

The conclusions drawn from this comparative study should not be universally applied, as building form and foundation conditions will have a significant impact on relative cost and performance between the

systems studied within. However, the comparative study does demonstrate that it is possible to achieve better seismic performance for less cost by early consideration of all options available for the lateral resisting system.

7 REFERENCES

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