Carterton Events Centre Auditorium Pres-Lam Wall Design and Construction

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ABSTRACT: Driven by sustainability, locally available resources and expertise, and economy, the design of the Carterton Events Centre focused on timber for the majority of the main structural and non-structural components. Combined with a client desire for minimization of earthquake damage, dissipative post-tensioned rocking Laminated Veneer Lumber (LVL) shear walls (Pres-Lam) were considered for the lateral load resisting system. During design development various structural forms were explored and tested through costing to determine an economic design solution meeting the project drivers. Advanced numerical analyses carried out by the University of Canterbury validated the design process assuring confidence with the design of the technology.

1 INTRODUCTION

The Carterton Events Centre is a multi-purpose community facility in the heart of the town. The project was driven by the Carterton District Council's vision to continue to develop a strong and vibrant community with a facility for all local people to use for a multitude of purposes. Opus International Consultants has been assisting the Council achieve this vision from the earliest proposal for this community focal point, some eight years ago.

The Carterton Events Centre combines the strengthened Heritage listed Library dating from 1881, Council and community facilities, the refurbished Youth Centre and the Auditorium. The overall layout is shown in Figure 1. The auditorium features laminated LVL trusses spanning up to 20 metres over the stage. Truss element and midspan splice connections were formed with steel plates and timber rivets to provide manageable (during construction) and unobtrusive (in the finished condition) connections. The structural timber selected was an MSG10 rated LVL product manufactured locally and sourced from local forestry plantations. This paper will concentrate on the design of the Auditorium lateral load resisting system.

Once the design process showed the Pres-Lam LVL (Palermo *et al.* 2006) wall option was the best solution for the Auditorium structure, the walls were designed using displacement based design principles and referring to precast concrete technology (Priestley *et al.* 1999, Nakaki *et al.* 1999). The auditorium features eleven Pres-Lam LVL shear walls, 6.7 metres tall, 2.4 metres long and 180millimetres thick with a central slot for the post-tensioned bars. The walls were constructed by laminating overlapping sheets of 45millimetres thick LVL. Energy dissipation has been provided by embedded mild steel rods in the wall end regions. The two D40 post-tensioning bars for each wall provided self-centering capacity to the system, minimizing post-earthquake residual drifts (Pampanin *et al.* 2003). The University of Canterbury peer reviewed the design of the lateral load resisting system, including non-linear dynamic time history analyses on the Pres-Lam walls.

2 CONCEPTUAL DESIGN OF AUDITORIUM

The auditorium forms part of the Carterton Events Centre complex. The space provided is multi use and fully flexible with the ability to seat up to 300 people. The auditorium was designed as an Importance Level 3 structure however it has the potential to play a significant role in community

recovery after a natural disaster so it is important that both structural and non-structural damage is minimised. The Carterton District Council also wanted to promote the use of sustainable materials in the design. This was due to the council's own timber resource, the local timber manufacturing capability that was available and various potential commercial support opportunities.

The width of the building is 14 metres over the auditorium and 20 metres over the stage area. To create the large open space timber trusses at regular centres are used for the roof system, supported on steel columns or the Pres-Lam walls. The trusses vary in height between 3.2 metres and 4.8 metres. The depth of the trusses allows for high level access walkways and the future provision of a grid and fly gallery over the stage. The trusses are overlain with plywood sheathing to create the structural roof diaphragm.
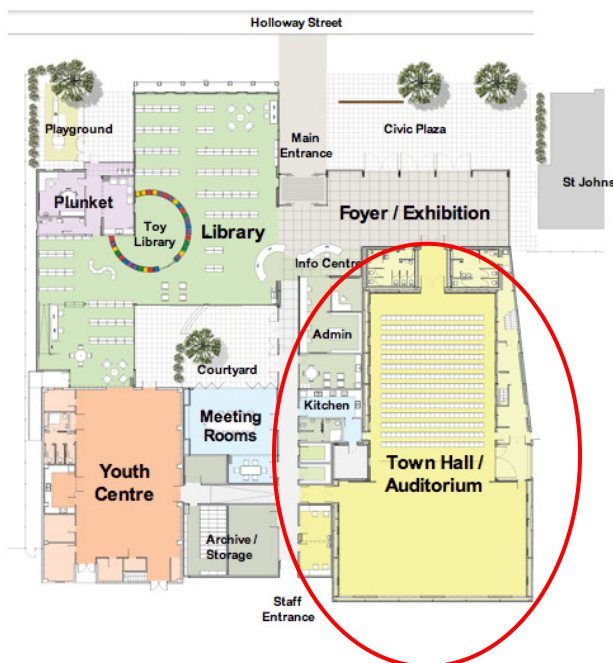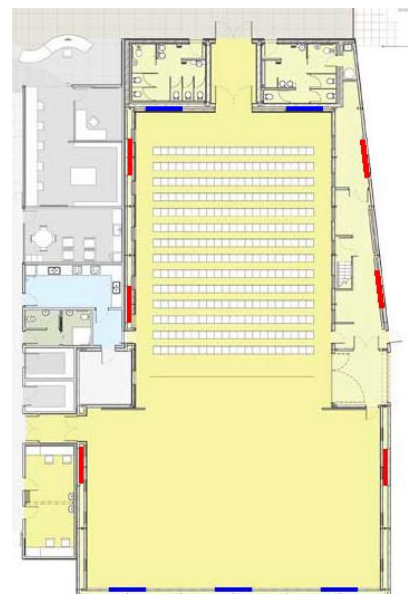

Figure 1. Ground floor plan

Figure 2. Auditorium plan and walls location

For the lateral load resisting system below the roof, several options were considered initially at the conceptual stages. These included elastic and ductile precast concrete panel shear walls, ductile plywood lined timber shear walls, and elastic and ductile steel braced frame with timber walls. The ductile options were all based on a ductility of $\mu=3$.

2.1 Elastic and Ductile Precast Concrete Panel Shear Walls

The first of the options for perimeter walls consisted of 200 millimetres thick precast concrete panels, approximately 2.25 metres wide and 9.0 metres high, spanning vertically between the ground and the roof diaphragm, resisting lateral loads in in-plane shear. Due to the significant weight of the panels, the roof diaphragm demands were high and supplemental steel angle bracing was required. The concrete panels provided sufficient acoustic mass to allow the auditorium walls to be single skin.

2.2 Ductile Plywood Lined Timber Shear Walls

The perimeter walls for this option consist of two layers of timber framing, separated for acoustic reasons. The walls spanned horizontally between steel columns and used a layer of 25mm thick F11 plywood to resist lateral loads in in-plane shear. The outer wall layer consisted of 140x45 framing, supporting the external lightweight cladding. The inner layer consisted of 140x45 framing, supporting the ply, insulation and internal gib lining on a resilient clip system. The inner shear wall layer panels were connected through the steel column web to adjacent panels to provide the lengths of wall required. For acoustic reasons, the total wall thickness required was approximately 400mm. Due to the lighter weight of the timber framing, the roof diaphragm required only supplemental steel rod

bracing.

2.3 Elastic and ductile steel braced frame with timber walls

This option featured timber walls that support the external cladding and internal linings as described above, however the plywood shear walls were removed and replaced with eight bays in each direction of vertical steel bracing between the columns, located between the timber wall layers. The roof diaphragm details are the same as for the ductile timber wall option above.

For the elastic design philosophy, internal wall lining damage was expected to be cracking at sheet edges and minor crushing at the base, requiring only minor repair after an earthquake. Damage for the ductile design philosophy was expected to be similar but significantly more extensive.

2.4 Meeting Client Drivers

It became obvious very quickly that a ductile design philosophy would give rise to significant damage to both the lateral loading resisting system and non-structural elements within the facility. The client drivers of minimising damage and down time after a significant earthquake meant that an appropriate design philosophy would be necessary. An innovative structural system was also going to be required to meet all of the client drivers of sustainability, damage minimisation and maximising timber use if timber was going to be utilised for more than just non-structural elements and linings.

3 DETAILED DESIGN OF AUDITORIUM

Following conceptual design of the auditorium lateral load resisting system, the options were costed and presented to the client. A Pres-Lam solution was not provided initially in conceptual design phase, however it was put forward as an alternative solution while the concepts were being considered. The Pres-Lam wall solution was cost neutral with the cheapest alternative solution of the ply lined ductile timber walls that did not meet the damage minimisation requirements. Having met all the client drivers, the damage limiting Pres-Lam system was the obvious choice to be taken into detailed design.

The Pres-Lam walls had the advantage of providing reduced deflections and therefore minimising damage and down time after a seismic event. The design is also likely to have minimal costs to repair after a seismic event and the self-centering mechanism that allows the wall to return to near vertical after a seismic event. The Pres-Lam walls also had potential Health and Safety benefits during construction.

The Carterton Events Centre is the first use of Pres-Lam technology in the North Island, echoing the local community drivers of sustainability and innovation. Similar technology has been used previously, however this project has extended the previous scope of this structural system beyond the low to medium rise commercial type building form.

3.1 Design of Pres-Lam Walls

The structural system of the Pres-Lam walls consists of three items. The post-tensioning through the centre of the wall that provides re-centering, mild steel bars at the wall ends to provide energy dissipation (together providing resistance to overturning) and the LVL shell that provides in plane shear resistance and houses the steel members.

There are six walls in the longitudinal direction (N-S) and five walls in the transverse direction (E-W). The placement of the walls is considerate of architectural requirements and fit around openings and services that are required. The distribution of the walls in the auditorium is such that effects of torsion are minimal. The walls in the transverse direction take a higher amount of load than the longitudinal direction (due to there being fewer walls in this direction) however the footprint of the walls in each direction is the same.

The outline dimensions of the Pres-Lam walls are 6.7m high, 2.4m wide and 180mm thick. The width

and length were designed to maximise the utilisation of the billets of LVL timber available from the local Juken New Zealand Ltd Wairarapa Mill only 13 kilometres from the site. The billets produced were 3 metres long, 1.2 metres wide and 45 millimetres thick. The billets were then glue laminated together to form the wall panels. Layering of the walls was arranged so that the joins of the individual plies did not coincide with the layer adjacent and create a weakness in the panel. With the use of multiple layers of timber a central slot for the post-tensioning could be easily achieved. The slot is the width of two plies (90 millimetres) providing sufficient room to allow the post-tensioned Macalloy bars to be coupled in place easily. The LVL used was rated at MSG10, showing the Pres-Lam system is not the preserve of high strength engineered timber. Wall geometry was designed such that the compression zone at the ends of the wall panels was within the 180mm thick wall end flanges. The wall arrangement is shown in Figure 2.

Utilising displacement based design philosophy, slightly modified for timber, the drift level of the walls was set to 1% of the height of the wall. This is a total displacement of 67 millimetres at the top of the walls. This drift level was found to provide the optimum solution, minimising deflection yet also limiting the amount of stress induced on the Macalloy post tensioned bar and mild steel dissipator elements. With higher drift levels the structure will have a higher fundamental period, which in turn will reduce the base shear. However with higher drift levels there will be greater elongation and hence strain in the steel elements.

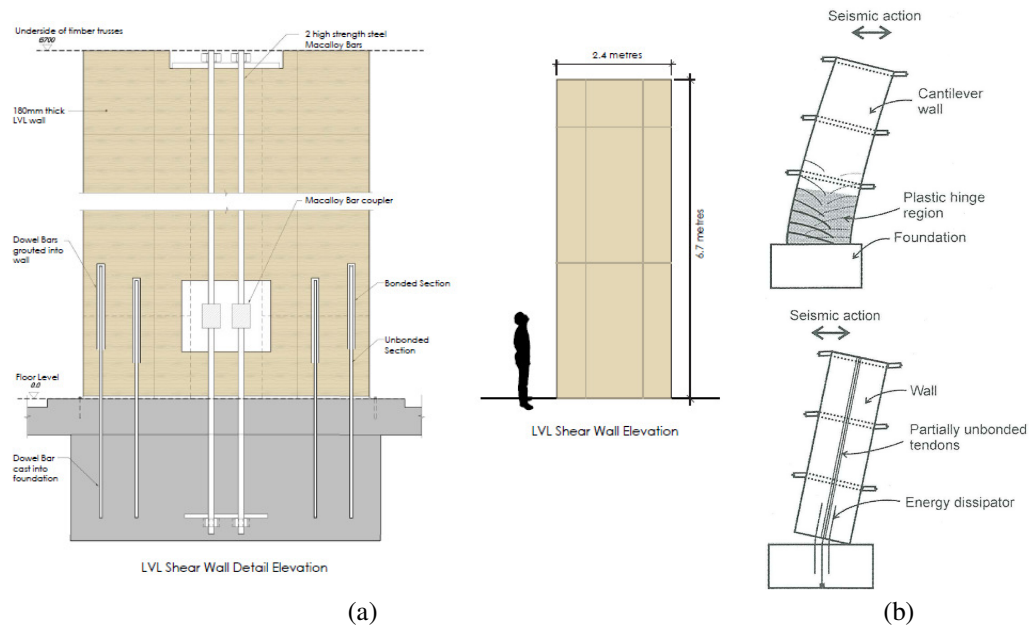(a)   (b)

Figure 3. (a) Pres-Lam Wall General Arrangement (b) comparative response of a monolithic system and a dissipative rocking post-tensioned solution (NZCS 2010)

The Pres-Lam system relies upon a controlled rocking mechanism. This dissipative rocking mechanism results in a very efficient flag shape hysteresis that is given by the two steel components. The axial load given by the self-weight of the wall and supported roof, and the tension force of the unbonded Macalloy bars provide self-centering, which aid the return of the wall to the original configuration after an earthquake. The mild steel bars that are embedded into the timber wall provide energy dissipation to the system. The combination of these two elements to the overall moment capacity of the wall gives the flag-shaped hysteresis loop on which the system is based. The target re-centering ratio that was used for the design was 1.5 or 60:40 (self-centering: energy dissipation). This does not produce the perfect flag shape hysteresis (50:50) , however having this ratio ensures the self-centering moment can overcome the dissipative elements and return the wall to near vertical after an earthquake. Similar guidance in provided in Appendix B of NZS 3101 (Standards New Zealand 2006).

There are some constraints that had to be considered and satisfied to ensure the system worked in the intended manner.

The Macalloy bar size, and hence coupling detail, was selected so that the required slot width in the wall coordinated with the ply thickness and did not make the wall unnecessarily thick. The bar coupler should fit easily within the wall slot to allow for connection and construction tolerance. The Macalloy bars are also stressed initially to provide the wall self-centering force. However the initial stressing of the bar had to be limited such that additional stress induced once the wall started rocking was limited to 0.8 the yield strength.

The mild steel bars providing energy dissipation were to fit within the flanges at the ends of the timber walls. This is to take advantage of the distance from the compression centroid of the timber. The bars had to be spaced so that the pull out cone of the epoxied bars in the timber could be developed without coinciding with the pull out cone of adjacent bar. This was essential, as to activate the unbonded section (whose tension and compression yielding provides the energy dissipation) of the mild steel bar there had to be sufficient fixity at both ends of the bar. The unbonded length of the bars differed with distance from the centroid so that the two bars would yield at the same time.

The balance between the two steel elements to create the correct recentering ratio required an iterative process. The size of the mild steel bars and the position of the Macalloy bars are fixed. This meant that the stresses could not be changed so the other variables had to be changed. These included the distance of the mild steel bars to the compression centroid and the initial stressing in the Macalloy bars. Changing these elements iteratively would change the moment capacity of the two elements and in turn the ratio would change until the desired ratio was achieved.

Direct compression stress in the timber beneath the Macalloy bar end anchorages were kept intentionally low to reduce long term creep effects and issues with total allowable stresses. A layer of plywood was also introduced on the vertical central slot inner face beneath the end anchorage to provide additional cross laminated layers and reduce unseen buckling effects of the uni-directional LVL in this region.

3.2 Design of Foundations

The foundations beneath the walls were large concrete pads that had to cope with the large concentrated loads from the Pres-Lam walls. The seismic loads on the auditorium are resisted by the six walls in the longitudinal direction and the five walls in the transverse direction. Due to the nature of the cantilever rocking walls, large overturning moments will be applied to the foundations as well as direct base shear. The foundations beneath the Pres-Lam walls are 2.6m long, 1.6m wide and 0.9m deep. The Macalloy bars and mild steel bars are embedded the majority of the foundation depth. The Macalloy bars have also an additional steel anchor plate to help distribute the loads from the initial pre-stress and additional load once the walls move in a seismic event.

The shear transfer to the foundations was achieved by shear keys at both ends of the wall. Although there is sufficient strength within the mild steel bars acting as shear dowels, it was decided that shear keys would be put into place to remove any combined action effects on the yielding elements and provide some redundancy in the design. The shear keys are made from bent steel plates that are fixed to the foundations. The steel plates are bent away from the wall so that they allow the wall to rock without imposing additional concentrated loads on the more highly stressed compression zone and the end of the wall.

4 PEER REVIEW OF AUDITORIUM WALLS

The Peer Review by the University of Canterbury gave the assurance to the Client and consultants that the analysis procedures provided a solution that met the design intent and Building Act requirements. As the design was an alternative solution, it was important for all parties that this was completed. The peer review provided useful feedback on elements of the system and ways the design could be improved. As a result of the Peer Review the following elements were modified:

- The unbonded lengths of the mild steel bars were limited due to concerns over the bars

buckling under the compressive loads.

- The top steel anchor plate transferring loads from the Macalloy bars to the timber wall was made larger. This was done to reduce compressive stress in the timber, particularly if the moisture content of the timber increased during construction and remained high when the bars were stressed.

- Two smaller Macalloy bars were used instead of one large bar. This provided a better distribution of load and provided for short term bar availability.

- Strain penetration effects were included in the design of the mild steel bars, taking into account the bond slip of the longitudinal bars due to the rotation of the wall at the foundation interface. As a result there is an additional unbonded length of bar that needs to be accounted for and in turn will affect the strain in the mild steel bars..

The Peer Review was performed by the Civil Engineering Department of the University of Canterbury and included the complete lateral load resisting system of the auditorium.

5 CONSTRUCTION OF AUDITORIUM

The construction of the auditorium presented the Contractor with no new construction methods, only new combinations and uses of materials.

The large excavation and the formation of the amount of steel within the foundation pads quickly demonstrated the large scale of the construction. At this stage the mild steel bars and the base plate and lower section of the Macalloy bars were put into place and cast within the foundation, projecting upwards for the LVL wall panels to be lowered into place. The Macalloy bar splice was located just above floor level for ease of access (refer Figures 4 and 5).

The LVL wall panels and auditorium roof trusses were fabricated offsite by McIntosh Timber Laminates. Once delivered, these elements were easily handled and stacked in a small area, ready for erection. The relatively lightweight elements were craned into place and positioned on the supporting structure (refer Figures 6, 7 and 8). The walls were propped into place until the mild steel bars were epoxy grouted into place and the Macalloy bars tensioned (refer Figures 9 and 10).

Figures 4 and 5. Wall Foundations, MacAlloy Starters and Dissipator bars

Figures 6, 7 and 8. Wall Panel Erection

Figures 9 and 10. Macalloy Bar Connection, Construction Progress.

6 NUMERICAL ANALYSES

This section illustrates the non linear static (pushover) and dynamic numerical analyses adopted as validation of the design methodology of the system that were carried out. The objective was the assessment of the seismic performance of the LVL Pres-Lam wall system and its connection to roof diaphragm. A lumped-plasticity model has been adopted through Ruaumoko 2D (Carr 2004).

Figure 11 shows the layout of the wall and reports general assumptions made in the development of the model. Wall reinforcements, either post-tensioned or mild steel bars, were modelled through longitudinal spring elements and the contact through a multi-spring element, which also enabled the evaluation of the neutral axis depth and gap opening since the springs are unilateral.
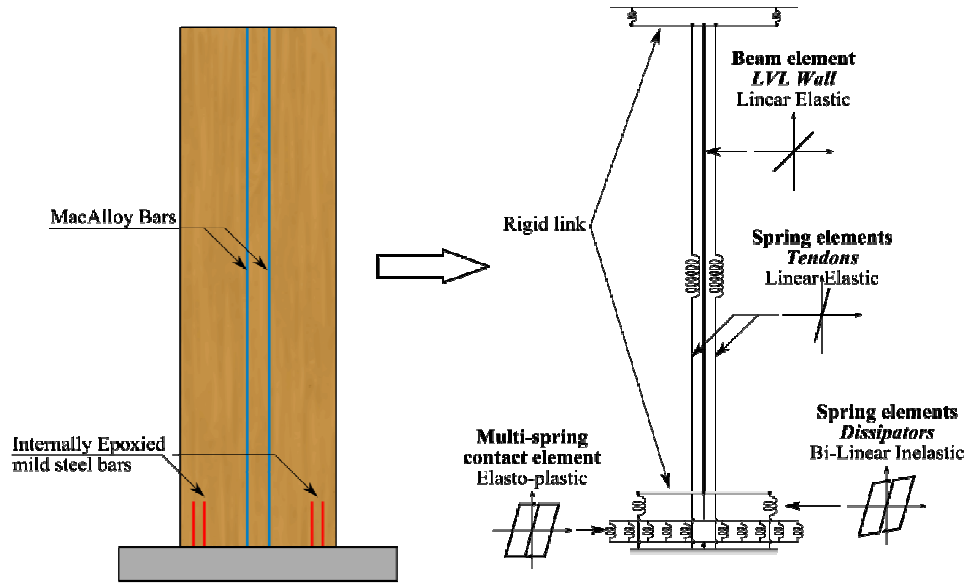
Figure 11. Post-tensioned wall and lumped-plasticity model

Figure 12 gives details of the truss-to-wall system and its connection detailing.
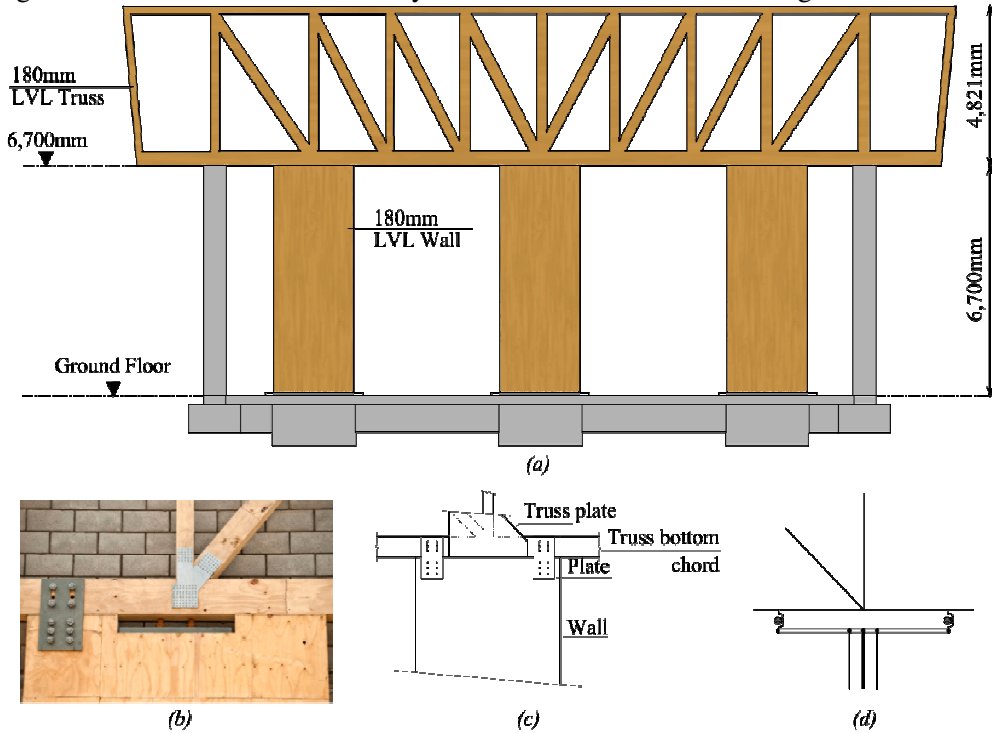

Figure 12. (a) Side view of truss-wall system; (b) Transverse wall connection during construction; (c) Sketch and (d) Model of the truss-to-wall connection

Wall design values of 1.0% target drift and a total seismic weight of 1,500kN were considered; adopting a Displacement-Based Design (DBD) approach and assuming a design equivalent viscous damping of 12% lead to an equivalent period of 0.57s and effective stiffness of 18,100kN/m. The total base shear force to be resisted by wall systems in each direction was 1,215kN, distributed over six longitudinal and five transverse walls; consequently, design base shear values of 203kN and 243kN were needed for longitudinal and transverse wall respectively.

Quasi-static pushover and push-pull analyses were performed. The former provides information about the capacity of the wall, pushing the system to a top drift of 1.5% (0.5% more than the design drift). The latter has been carried out imposing a cyclic schedule at ±1% drift.
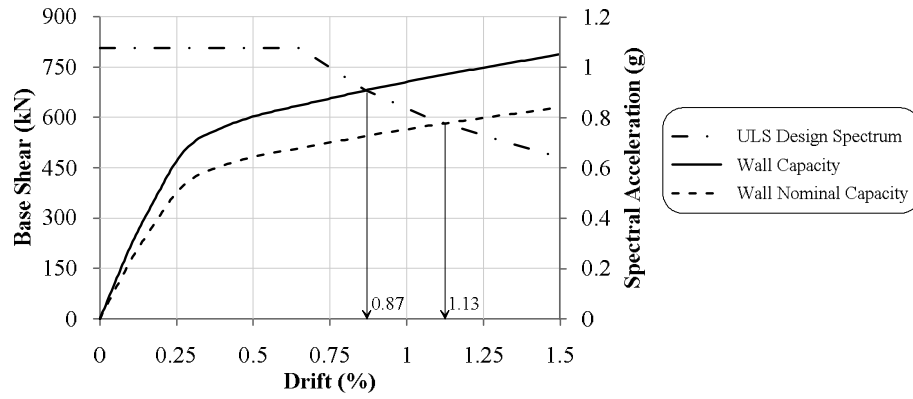
Figure 13. Seismic Performance Assessment

The Capacitive Spectrum Method (CSM) has been used to make an explicit comparison between the structural capacity and the earthquake demand. This method estimates the deformation of inelastic Single Degree of Freedom systems consistent with the selected inelastic design spectrum (Chopra and Goel 1999). The intersection between the capacity curve and the demand spectra is the required "performance" point, which should match the design drift value.

Capacity spectrum method results in Figure 13 show 0.87% drift level corresponding to the wall capacity and 1.13% to the nominal capacity. Those values are successively compared to the results of dynamic analysis presented below.
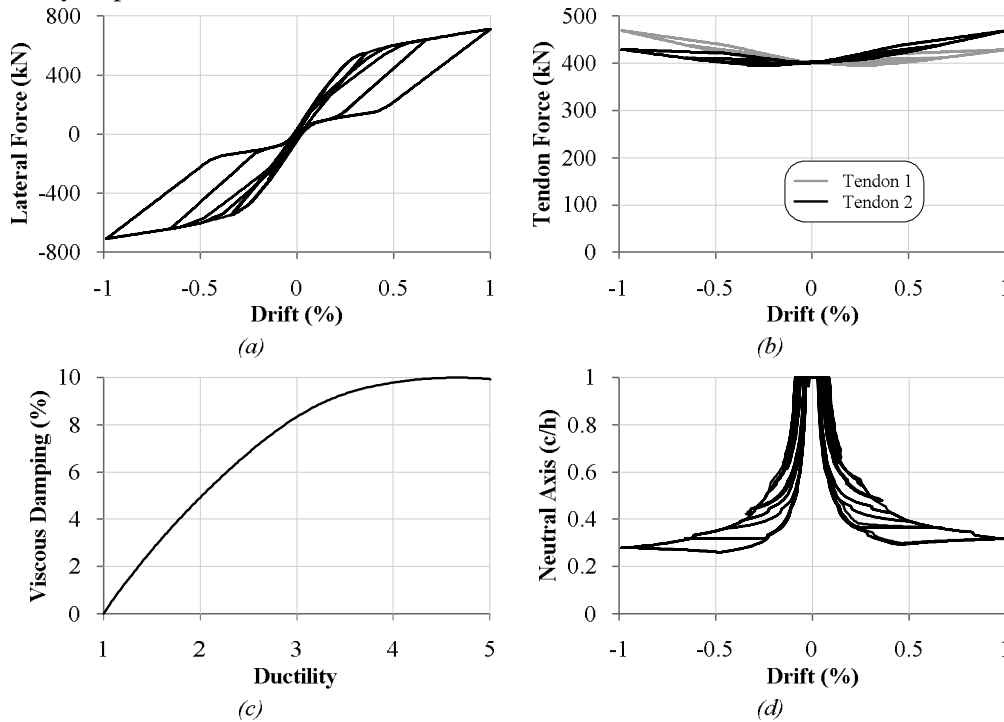

Figure 14: quasi-static analysis (a) Lateral force, (b) Tendon forces, (c) Viscous Damping and (d) neutral axis vs. drift

Key parameters such as lateral force vs. drift, tendon forces vs. drift, area-based viscous damping and dimensionless neutral axis versus drift are shown in Figure 14. Figure 14a confirms that the system has a proper flag-shape hysteresis as assumed during the design ($\lambda = 1.5 = M_{pt} / M_s$). Elongation of the tendons seems to be negligible (Figure 14b) while the equivalent viscous damping has values quite consistent with the design phase. The neutral axis stabilizes at 0.3h for 1% drift level (Figure 14d).

Dynamic non-linear time history analyses are briefly herein presented. The walls have been subjected to a suite of ground motions (Pampanin 2001) scaled individually according to NZS 1170.5 (Standards New Zealand 2004) elastic spectra; importance level 3, annual probability of exceedance of 1/1000,

soil class C, hazard factor of 0.42, return period factor of 1.3 and near fault factor of 1.0 are assumed. As shown in Table 6.1, the maximum drift value of 0.98% is obtained under ground motion EQ 13. This maximum drift value reached is consistent with Capacity Spectrum Method shown in Figure 13, and it is below the design value of 1.0%. Thus, the results confirm the consistency of the design methodology with the numerical analyses.

Table 6.1: maximum and residual displacements

	Eartquake Record	Year	Mw	Station	Soil Type	Scaling Factor	Scaled PGA (g)	Max Drift (%)	Res. Drift (%)
EQ1	Superstition Hills	1987	6.7	Brawley	D	6.1	0.626	0.59	0.046
EQ2	Superstition Hills	1987	6.7	El Centro Imp. Co. Cent	D	2.8	0.711	0.52	0.001
EQ3	Superstition Hills	1987	6.7	Plaster City	D	3.4	0.634	0.85	0.001
EQ4	Northridge	1994	6.7	Beverly Hills 14145 Mulhol	C	1.6	0.673	0.60	0.005
EQ5	Northridge	1994	6.7	Canoga Park – Topanga Clan	D	1.6	0.568	0.56	0.094
EQ6	Northridge	1994	6.7	Glendale – Las Palmas	D	1.9	0.696	0.40	0.060
EQ7	Northridge	1994	6.7	LA – Hollywood Stor FF	D	2.7	0.615	0.63	0.065
EQ8	Northridge	1994	6.7	LA – N Faring Rd	D	2.3	0.631	0.51	0.022
EQ9	Northridge	1994	6.7	N Hollywood – Coldwater Can	C	2.4	0.641	0.74	0.023
EQ10	Northridge	1994	6.7	Sunland – Mt Gleason Ave	C	4.4	0.69	0.86	0.321
EQ11	Loma Prieta	1989	6.9	Capitola	D	1.2	0.608	0.72	0.056
EQ12	Loma Prieta	1989	6.9	Gilroy Array #3	D	1.1	0.622	0.73	0.269
EQ13	Loma Prieta	1989	6.9	Gilroy Array #4	D	1.4	0.604	0.98	0.224
EQ14	Loma Prieta	1989	6.9	Gilroy Array #7	D	2.7	0.618	0.68	0.008
EQ15	Loma Prieta	1989	6.9	Hollister Diff. Army	D	1.9	0. 528	0.66	0.019
EQ16	Loma Prieta	1989	6.9	USGS Anderson Dam	D	2.7	0.651	0.78	0.096
EQ17	Cape Mendocino	1992	7.1	Fortuna Fortuna Blvd	C	4.9	0.569	0.54	0.142
EQ18	Cape Mendocino	1992	7.1	Rio Dell Overpass – FF	C	1.4	0.55	0.56	0.031
EQ19	Landers	1992	7.3	Desert Hot Springs	C	2.4	0.582	0.46	0.143
EQ20	Landers	1992	7.3	Yemo Fire Station	D	2.2	0.451	0.62	0.011

7 CONCLUSION

The combination of the restoration of the Heritage Library, provision of community services (Plunket, Toy Library, Youth Centre, Meeting Rooms) areas, and the community focused auditorium has been a uniting process for the Carterton community and an incredibly rewarding process for all involved.

The construction of the auditorium, featuring the innovative Pres-Lam system, has created a lot of interest locally and nationally, particularly in the aftermath of the Canterbury earthquakes and the increasing public awareness of damage limiting construction philosophies. The significant use of timber has been a feature in its own right and been followed closely by the local building trades.

The innovative Pres-Lam system has met all of the client drivers of damage minimisation, sustainable design and maximising timber use. More importantly the joint collaboration of the University of Canterbury and Opus International Consultants made this advanced technology an economic construction option.

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