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LOW DAMAGE SEISMIC SEISMIC RETROFIT OF A HERITAGE URM BUILDING IN WELLINGTON, NZ

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Abstract

This paper discusses the unique approach taken in seismic retrofit of an existing four-storey URM building located at 97 The Terrace in Wellington, NZ and constructed in 1906. Listed as a Historic Place Category 2, it is one of the only domestic scale Edwardian townhouses remaining in the commercial zone of The Terrace.

Blue Barn Consulting were engaged to assess the building's seismic performance and provide an effective solution to raise its seismic rating to 100%NBS. We developed a solution which is hidden and preserves the historic nature of the building.

As part of the Blue Barn's proposed low damage system, post tensioned vertical rods were utilised to increase the confinement and compressive forces in the URM walls. This increases their strength and resilience for in-plane and out-of-plane load demands and considerably reduces the expected damage in a design event from a Life Safety perspective, whilst allowing for enough resilience at the MCE level event (Collapse Prevention).

This innovative solution allows for the optimal use of the existing structure. Instead of more elaborate and expensive structural options, installing post tensioned rods in the existing URM walls allows the structure to better utilise these walls to achieve the required global performance and resilience.

In addition to the post tensioning works, lightweight composite concrete diaphragms are being introduced, to improve the diaphragm performance. The existing timber floors are being maintained below the new concrete slab, achieving an exposed timber finish from underneath.

The solutions developed by Blue Barn for this historically significant building at 97 The Terrace has resulted in an effective low damage retrofit option. The use of post tensioning in the URM walls has significantly improved the performance of the building, helping to increase the seismic rating of the building from 45%NBS to 100%NBS (New Building Standards).

Keywords: heritage preservation; unreinforced masonry; seismic retrofit; DIANA FEA

1. Introduction

Blue Barn Consulting were engaged to assess the building's seismic performance and provide an effective solution to raise its seismic rating to the current standards.

The client's brief was to develop a seismic strengthening scheme suitable for this historically significant building to increase its seismic rating to 100% New Building Standard (NBS) in accordance with New Zealand Standards [1, 2, 3, 4]

Blue Barn's aim was to identify the critical structural weaknesses of the existing structure in order to develop a strengthening scheme that added strength and resilience whilst utilising the existing inherent capacity of the building.

2. Building Description

2.1 General

The property is a four-storey plus basement URM building constructed in 1906. Listed as a Historic Place Category 2, it is one of the only domestic scale Edwardian townhouses remaining in the commercial zone of The Terrace. Fig. 1 shows the current day view of the building from The Terrace. A historic elevation of the building viewed from Woodward Street is shown in Fig. 2.

Fig. 1 – 97 The Terrace – View from The Terrace

Fig. 2 – Historic Elevation of the building from Woodward Street

The building was originally used as a mixed-use office and residential building however, it is currently not in use while going through retrofit and refurbishing works. Some alterations have been carried out over time, as listed in the sections that follow.

2.2 Building's superstructure

The building has approximate plan dimensions of 10.5m wide x 21.0m (at ground level) long and is mainly constructed from unreinforced masonry (URM) walls (clay brick) on all elevations, which step in thickness, getting one brick leaf thinner per storey height in certain locations. The basement, ground, second and third floors are constructed from Kauri wood floor joists running in the North-South direction (Across). These joists are supported at their midspan on a steel RSJ beam that is supported by concrete encased cast-iron intermediate columns, at approximately 3.5 – 4.0m centres on the lower three levels and bear on the masonry walls at their ends, with a minimum seating length of 9" (225mm) according to the existing specifications. The steel beams are then covered with concrete and mortar and fully enclosed within the masonry wall. Fig. 3 shows a typical plan and elevation of the building.

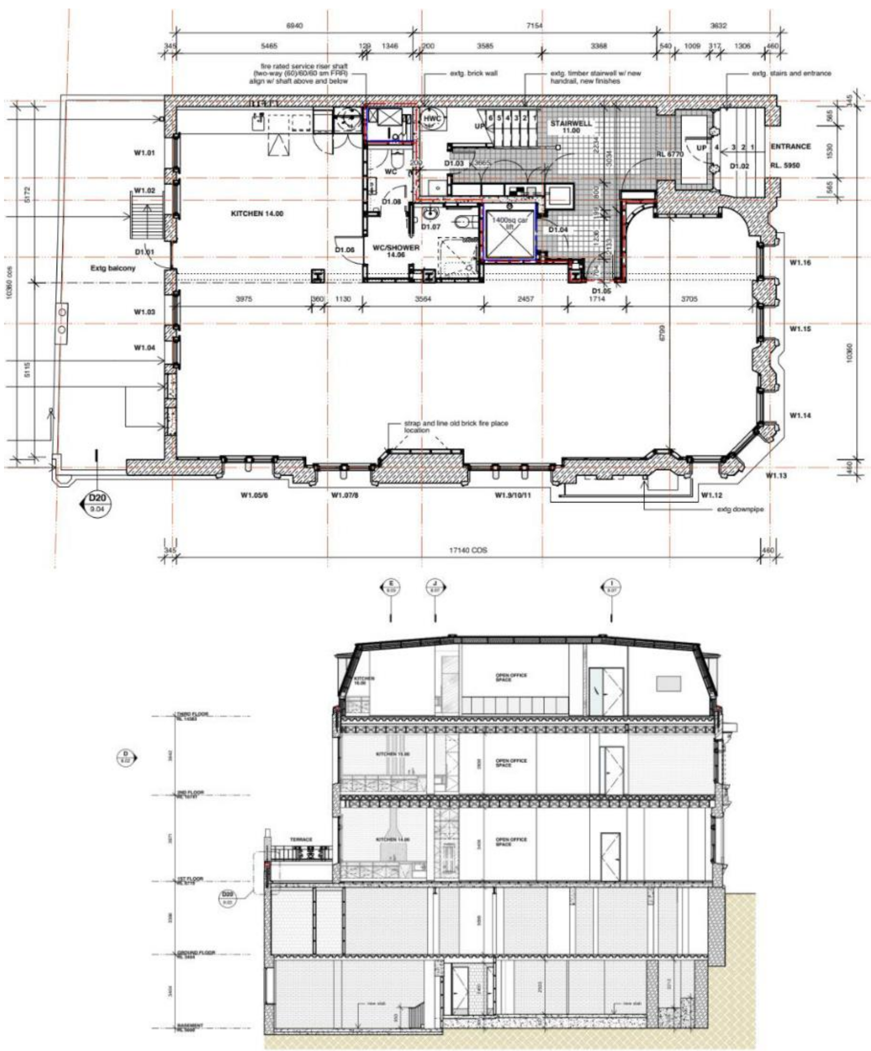

Fig. 3 – Typical plan and cross section of the building

Major alterations to the roof took place in 1985 and the roof was converted into a full floor within the roof space with the construction of it being steel framing supporting light weight cladding. During these alteration works, some masonry walls were removed and replaced with reinforced concrete arches on the ground floor. Some steel retrofit work including a portal frame and a steel braced frame were introduced in the ground floor area. A reinforced concrete "L" shape wall was introduced on levels one and two to enclose the staircase but doesn't continue down to ground and basement floors.

On the basement and ground floor levels of the building, the wall thicknesses were measured to verify the dimensions of the wall thicknesses shown on the historical drawings. The wall was measured to be 450mm thick which was approximately 4 wythes (or 18") in the basement and approximately 250mm for an internal plaster covered wall on the ground floor which is approximately 2 wythes.

2.3 Building's foundations and soil conditions

According to schematic historical details and written specifications available, the foundations are minimum twice as wide as the wall they are supporting (i.e. half a wall thickness wider on each side). The foundation depths were specified to be no less than the depth of any of the footings that were constructed, approximately 350-550mm below natural ground level, all bearing on flat ground of similar quality.

A recent geotechnical investigation and report by Engeo Geotechnical Engineers was undertaken in February 2018 at 97 The Terrace, Wellington. Based on this report, an ultimate bearing capacity of 1000kPa is proposed for shallow strip footings near the surface. The borehole logs in the location of the three tests indicate that the ultimate geotechnical bearing capacity can be achieved between 500 – 1200mm below natural ground level. There is no evidence of ground settlement, differential settlement or any indicators of ground related issues on site under the existing static loads. No risk of liquefaction or lateral spreading is envisaged for this site.

3. Assessment Methodology

A detailed modelling and analysis of the building was carried out using DIANA Finite Element software. This assessment included the non-linear behaviour of the unreinforced masonry elements under various earthquake scenarios (ground motions), with different intensities, that provided a realistic representation of the building behaviour in real earthquake events. In this assessment a performance-based approach was adopted based on the available explicit URM model within DIANA software. The performance of the building has been evaluated in accordance with the requirements of the NZSEE Guidelines [2, 3, 4] utilising mainly a displacement-based assessment as well as the ASCE 41-17 [5] performance criteria in respect to floor accelerations for Life Safety performance levels. In addition, force-based assessment was also considered for some of the building elements such as the pounding effect from the neighbouring building.

Areas of localised yielding or failure were identified in the model and closely analysed in conjunction with displacement and drift information, to understand whether they will result in a major failure in the building or if they are limited to localised areas only. The main mode of failure was the out-of-plane failure of the masonry walls.

4. Analysis

Non-linear, time history analysis has been utilised in this assessment.

4.1 Modelling

The building geometry was modelled using the information available on the property file drawings, which was validated during our site inspection. The material properties were determined based on the recommendations of the NZSEE Guidelines [2, 3, 4] and available academic literature [6, 7, 8]. These properties are summarised in Table 1. A 100mm thick shell element is considered for timber diaphragms on ground floor and levels 2 and 3. The material properties of this element are adjusted to provide the equivalent properties of the floor arrangement in place. An in-plane shear stiffness of 215 kN/m is assumed in accordance with Table C8.8 of the NZSEE Guideline [4] for timber floors in good condition with continuous joists without reliable mechanical anchorage. Fig. 4 shows a 3D representation of the model.

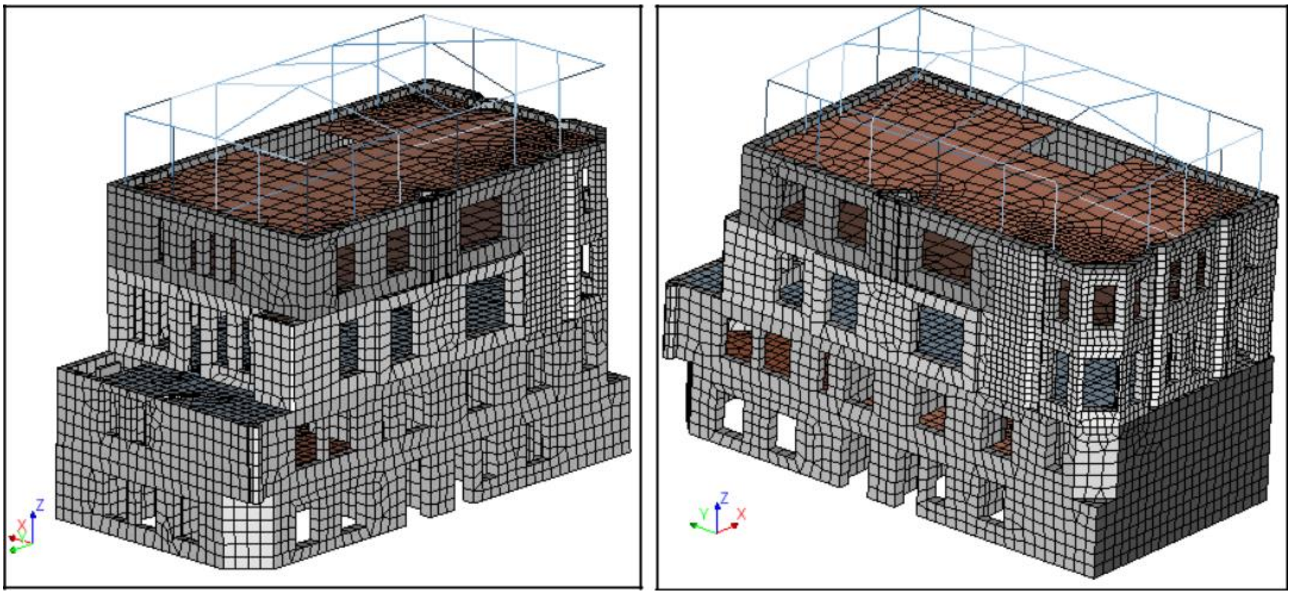

Fig. 4 – 3D views of the finite element model; Left: View from Alley along the eastern side, Right: View from corner of The Terrace and Woodward Street

4.2 Material properties

Our assessment of the in-situ brick and mortar elements show that they are of a medium to hard nature and in good condition. On that basis and with reference to material property tables provided in NZSEE Guideline Section C8.2 [4], we determined the material properties to be used in our modelling. Table 1 summarises the material parameters used in the modelling.

Table 1 – Masonry material properties used in modelling

Parameter	Symbol	Value
Probable Masonry Compressive Strength	f'_m	12.6 MPa
Probable Masonry Tensile Strength	f_t	0.3 MPa
Masonry Modulus of Elasticity	$E_m = 300 f'_m$	3.78 GPa
Shear Modulus of Masonry	$G_m = 0.4E_m$	1.5 GPa
Unit Weight of Masonry		18 kN/m^3
Poisson's Ratio		0.2
Tensile Fracture Energy (Area under the curve in Fig. 3-2 on tension side)		0.01 N/mm
Compressive Fracture Energy (Area under the curve in Fig. 3-2 on compression side)		6 N/mm

4.3 Crack model

DIANA has a built-in non-linear rotational crack model that has been used in our modelling. Fig. 5 below shows the model used for the masonry in the computational model with parameters defined in Table 1. This crack model has been verified by a research programme at Delft University [9].

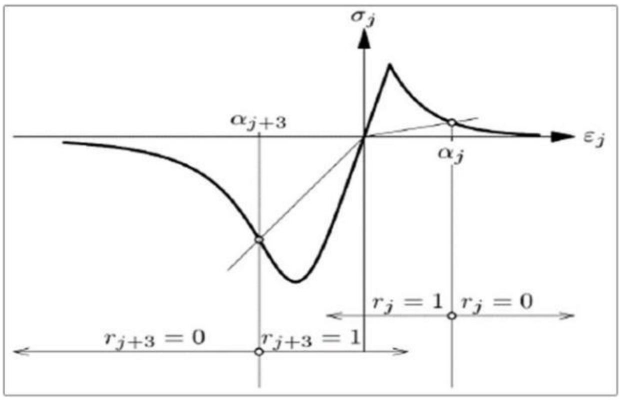

Fig. 5 – Rotational Crack model used in the computational model [9]

The loading–unloading–reloading condition is monitored with the additional unloading constraints which are determined for both tension and compression to model the stiffness degradation in tension and compression separately.

4.4 Time history records

The non-linear time history analysis has been carried out in accordance with recommendations of NZS1170.5 [1] based on three different earthquake scenarios (earthquake records) that are scaled as per requirements outlined in this standard. Engineering Geology carried out a site-specific hazard assessment to supply us with three suitable earthquakes with at least one of them with forward directivity properties. Table 2 shows the three earthquake records and associated scaling factors used for this assessment. These records were scaled to different levels of intensities to finally determine the minimum rating of the building, based on the acceptable performance of the building. The principal component of the earthquake was applied in both principal directions being North-South (N-S) and East-West (E-W) termed Across and Along in this paper respectively.

Table 2 – Details of time history records used for this assessment

Record	Scale factor	Subsoil Class	Distance to Fault (km)	Principal Component	Forward Directivity
Duzce, Turkey, 1999	1.59	B	8	E	No
Darfield, New Zealand, 2010	1.27	B	24	S80W	Yes
Michoacan, Mexico, 1985	2.02	A	13	N00W	No

5. Assessment results

The results of the time-history analysis show that the Darfield earthquake with the main component applied in a N-S direction causes the worst actions and displacements in the building.

The DSA (Detailed Seismic Assessment) determined that the building, in its current state, achieves a rating of a minimum of 45% NBS (New Building Standard).

The main areas showing signs of crack initiation under seismic shaking were the eastern and western end walls at the second floor and some minor crack initiation in the northern and southern walls. Fig. 6 below shows the distribution of the maximum tensile stresses in the building. Our Engineering interpretation indicates out of plane failure of the URM walls as the main mode of failure with some localised areas of shear failure of slender/weak URM piers. Fig. 6.1 at the next page shows the maximum deflected shape of the building at 45%NBS.

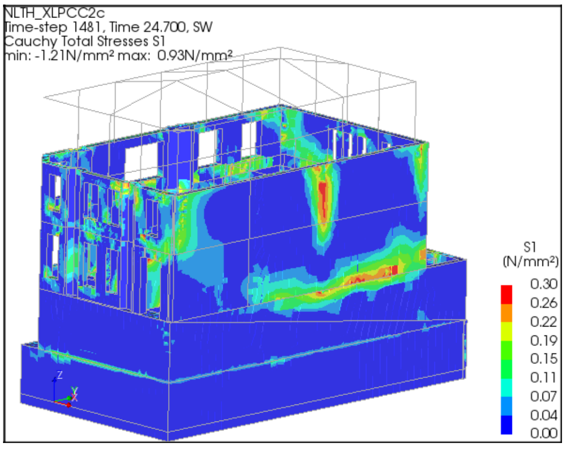

Fig. 6 - View of maximum tensile stresses in the building due to worst time history analysis case (Darfield scaled to 45%NBS, with primary component in the N-S direction)

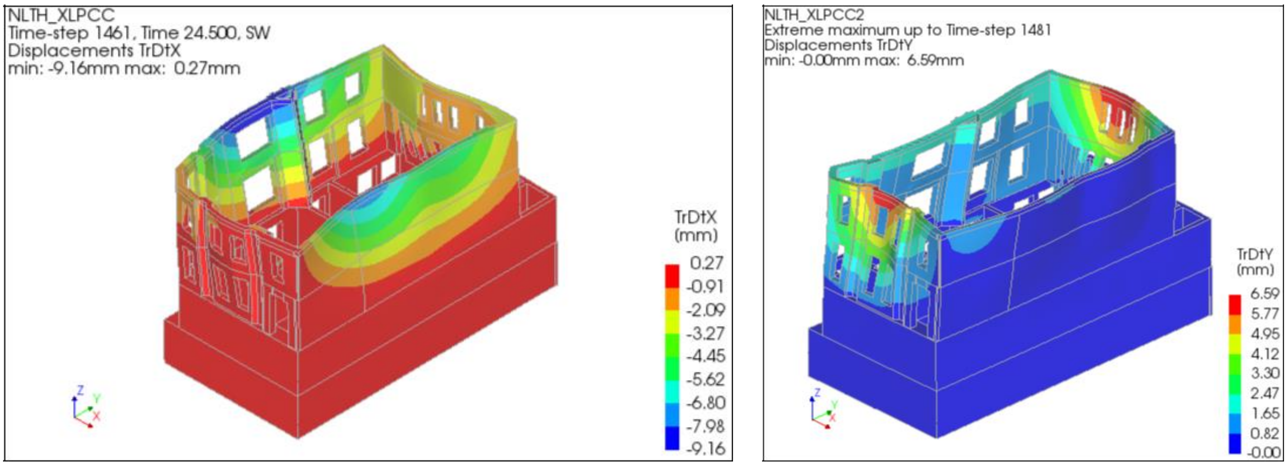

Fig 6.1 – Maximum Deflected Shape of the building due to 45%NBS SCALED Darfield Earthquake, with Principal Component in “X” direction

5.1 Global performance of the building

The performance of the 97 The Terrace Building, based on our calculations obtained from DIANA analysis at 45%NBS seismic loads, sits between the “*Immediate Occupancy*” state and the “*Life Safety*” state as shown in Fig. 7. The blue line on the graph indicates the approximate location of the Building on this curve at 45%NBS loads. The global performance of the building was evaluated in accordance to the NZSEE Guidelines [2, 3, 4] and the ASCE 41-17 [5].

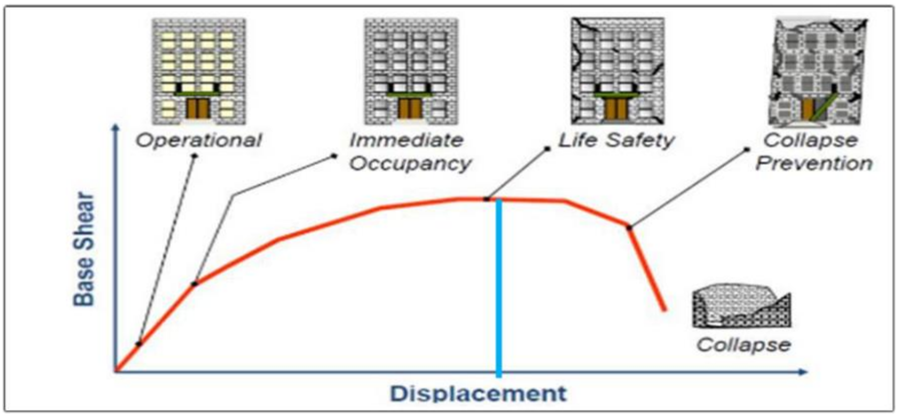

Fig. 7 – Force deformation diagram depicting different stages of building damage

6. Strengthening scheme

Blue Barn developed a low damage system utilising post tensioned vertical rods to increase the confinement and compressive forces in the URM walls. This increases their strength and resilience for in-plane and out-of-plane directions and considerably reduces the expected damage in a ULS design event, whilst allowing for enough resilience at the CLS event. Our analysis shows the out of plane failure mode is largely eliminated by introduction of the vertical rods and the in plane shear failure mode changes to rocking of the piers that is a more favourable failure mode.

32mm diameter high tensile strength Freyssinet post-tensioned vertical rods are proposed to be installed around the perimeter of the building, extending to below the ground floor. An 80mm diameter duct is first drilled, before the 32mm diameter high strength Freyssibar is placed, tensioned and then grouted. The bars are tensioned to a maximum of 250kN, this results in a maximum compressive stress ratio of approximately 10% compared to the masonry's compressive strength.

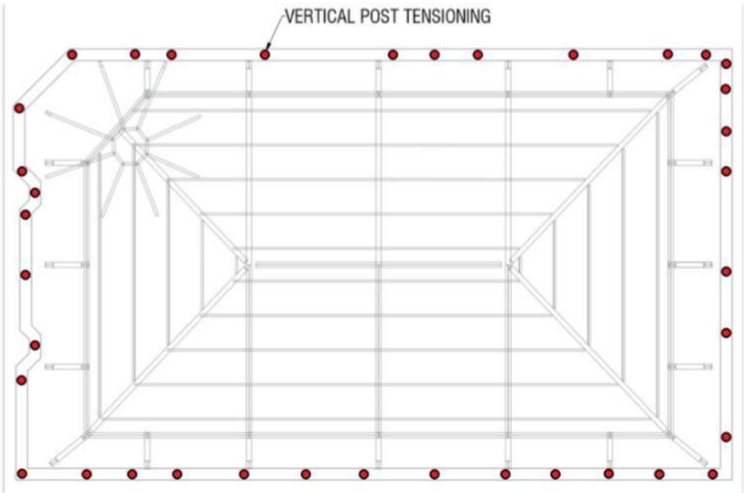

Fig. 8 – Plan view of the vertical post-tensioning rods around the perimeter of the building

The post tensioned bars are anchored at the top of the structure, on top of the existing reinforced concrete bond beam and restrained at the bottom using a bonded length and confinement detail prior to post tensioning. Fig. 8 shows the plan view of the proposed post-tensioning rods around the perimeter of the building.

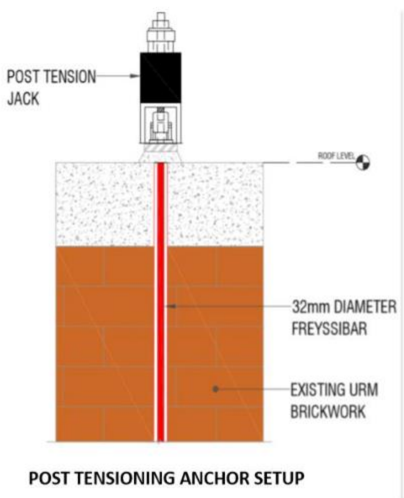

Fig. 9 – Left: 3D view of the post tensioned vertical rods; and Right: Anchor setup detail

This innovative solution allows for the optimal use of the existing structure. Instead of more elaborate and expensive structural options, installing post tensioned rods in the existing URM walls allows the structure to better utilise these walls to achieve the required global performance and resilience. A 3D rendered view of the vertical tendons as well as their proposed head assembly is shown in Fig. 9.

In addition to the post tensioning works, lightweight composite concrete diaphragms are being introduced, to improve the diaphragm performance. The existing timber floors are being maintained below the new concrete slab, achieving an exposed timber finish below floors. The proposed floor detail is shown in Fig. 10.

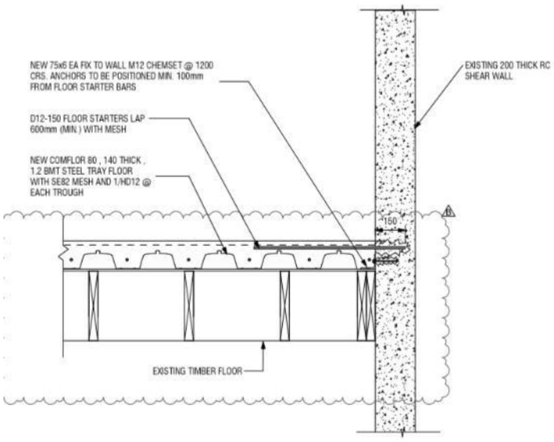

Fig. 10 – Detail of the proposed light-weight composite floor above the existing timber floor

7. Conclusions

The solutions developed by Blue Barn for this historically significant building at 97 The Terrace has resulted in an effective low damage retrofit option. The use of post tensioning in the URM walls has significantly improved the performance of the building, helping to increase the seismic rating of the building from 45%NBS to 100%NBS.

The strengthened building will then meet the requirements of the current New Zealand Building Code to achieve life-safety and safe evacuation requirements at the Ultimate Limit State (ULS) seismic loading, with sufficient resilience for maximum considered earthquake loading (MCE).

This innovative solution allows for the optimal use of the existing structure. Instead of more elaborate and expensive structural options, installing post tensioned rods in the existing URM walls allows the structure to better utilise these walls to achieve the required global performance and resilience.

We believe that the proposed solution will assist the structural Engineers preventing the Heritage status of the existing historic buildings.

8. Acknowledgement

We would like to express our appreciation to Professor Jason Ingham and the University of Auckland for the Peer Review of our design and the very useful comments.

9. References

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