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CASE STUDY AND DEVELOPMENT OF SEISMIC RETROFIT SOLUTION FOR A HERITAGE URM BUILDING

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ABSTRACT

Numerous unreinforced masonry (URM) buildings in New Zealand have been declared earthquake prone but have heritage value that restricts changes to their original architecture during seismic retrofit. The earthquake prone building considered in this case study was constructed with cavity walls in the outer periphery, single leaf thick partition walls and two leaves thick interior bearing walls. The materials used in construction were solid clay burnt bricks and a lime mortar, with the masonry laid in a common bond pattern and plastered on the outer and inner faces. The roof consisted of corrugated iron sheets resting over wooden trusses. An initial evaluation of the building structure was performed with procedure suggested by New Zealand Society of Earthquake Engineering (NZSEE) guidelines. The building structure was found earthquake-prone in the transverse direction and earthquake-risk in the longitudinal direction, therefore a detailed evaluation was performed by using a homogenised finite element (HFE) computer model of the as-built structure and also the critically loaded walls were analysed for in-plane strength and out-of-plane stability using NZSEE guidelines. The results showed that the walls were unstable when subjected to out-of-plane loading and it was concluded that the building requires a retrofit. A literature review of existing viable retrofit solutions was performed and on the basis of relative merits and demerits the most suitable seismic retrofit solution (i.e., post-tensioning) was selected. The out-of-plane stability of post-tensioned URM wall was checked with an existing conceptual model and an appropriate seismic retrofit solution to satisfy strength and heritage conservation requirements was recommended.

KEYWORDS: retrofit, unreinforced masonry, strengthening, post-tensioning, earthquake

INTRODUCTION

Unreinforced masonry (URM) was one of the most common construction materials in New Zealand before the 1931 Hawke's Bay Earthquake [1]. The popularity of URM in New Zealand decreased after that earthquake and the use of masonry was subsequently restricted under government regulations, with the current masonry design standard NZS 4230:2004 refering only to reinforced masonry structural elements [2]. The New Zealand URM building stock consists of a significant number of mostly pre-1964 URM structures [1], with many of these buildings contributing to New Zealand's architectural heritage. Unfortunately, these structures also collectively constitute a seismic hazard to New Zealand's citizens. The current New Zealand Building Act 2004 requires earthquake-prone structures to be either demolished or upgraded to ensure their safety under moderate earthquakes [3].

BUILDING DESCRIPTION

Demolition and reconstruction of the case study building named "Avon House" were uneconomical and undesired tasks, due to its historical and cultural importance. Figure 1 shows the pictorial view of the building. This building was constructed by early British migrants in 1880's and is a single storey four bedroom house with British architecture and construction techniques. The subject building is situated on the northern side of Hargreaves Street and western side of Wallace Street in Wellington. A reinforced concrete retaining wall facing Hargreaves Street was constructed after the failure of the old stone masonry gravity retaining wall. The geological formation under the subject building is a moderately to slightly weathered Greywacke rock [4] and the house is a double-gabled building with a covered porch and a footprint area of 142.85 m^2. The main gable runs along Hargreaves Street with the sloping gable end facing Wallace Street. The closest neighbouring building is 5 m to the east.

Figure 1: Avon House street view

The roof of the house is steeply pitched, timber framed and clad with corrugated iron sheets supported on wooden sarkings spaced at 925 mm on centres. The sarkings are further supported by rafters spaced at a centre to centre spacing of 800 mm with runners resting on walls. Exterior walls are typical cavity walls with a single leaf veneer and a double leaf bearing wall having a 50 mm cavity between these two walls, tied with steel bar ties which were deteriorated. These walls consist of handmade brick masonry laid in lime mortar with a cement rendered weather

coat on the exterior surface and set over brick foundations. The interior walls are both single leaf thick and double leaf thick. All the interior walls are plastered with lime-sand mortar and painted with plastic emulsion.
The brick masonry was laid in common bond with every sixth course of masonry as a header course and stretcher courses running between these header courses. The mortar used in the masonry is a soft lime mortar with a sand base, which can be scratched with a finger nail. Metal bar ties were used to hold the veneer walls at the outer periphery, but have corroded and must be replaced. Figure 2 shows the layout plan and facade numbering with all dimensions in millimetres.

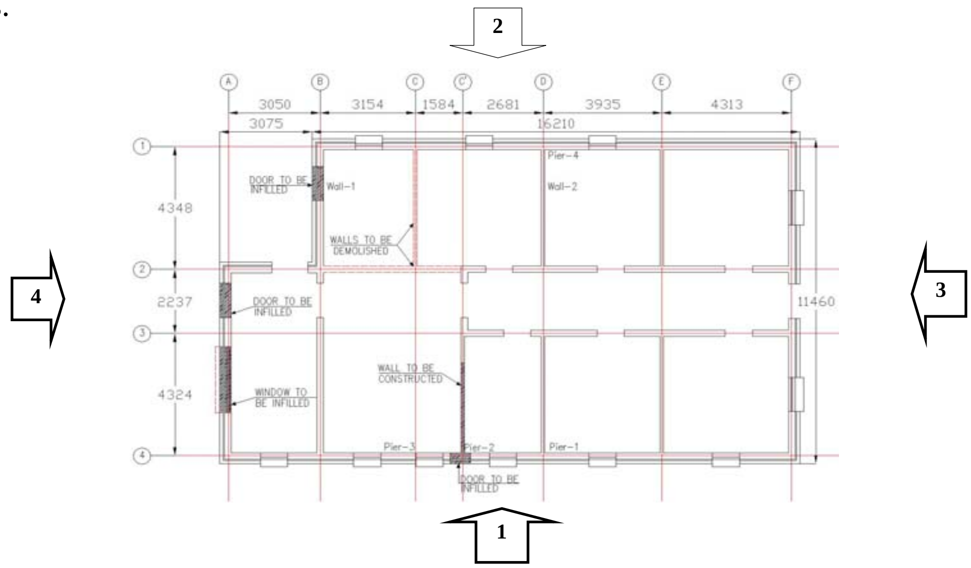
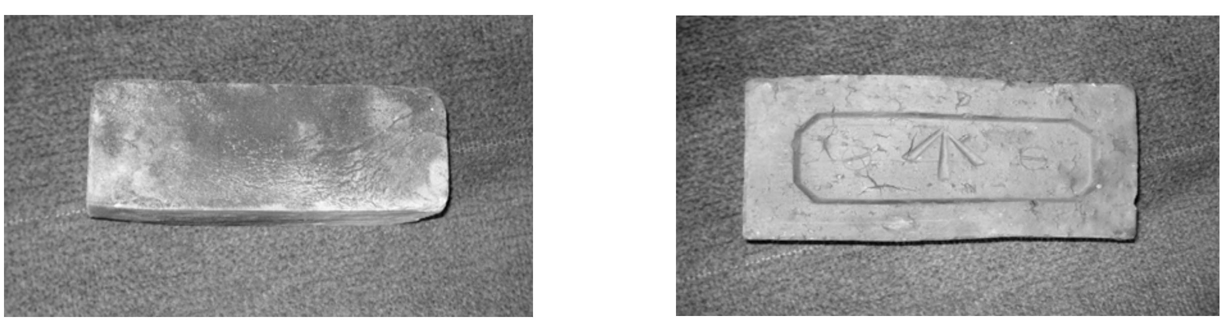


Figure 2: Layout plan of the building structure and facade numbering

Two types of bricks were used in the construction. Figure 3 shows features of the extracted brick samples from this building, suggesting that they were handmade. One of the more obvious signs is the rough surface of the bricks themselves. In the process of making bricks by hand, sand was often placed into the mould to prevent the clay from sticking to the sides. This resulted in the imprint of the sand remaining on the brick surface, giving it a rough face. The second type of bricks is the mould-made bricks manufactured by convicts, which is evident from the Arrow stamp and impressions of tightened bolts that are visible on the bricks. The soft nature of the bricks indicates that they were burnt at a low firing temperature. This is an indicator of early brick production.



(a) (b)

Figure 3: Bricks used in construction: a) handmade bricks; b) mould-made convict brick

INITIAL EVALUATION PROCEDURE (IEP) AS PER NZSEE 2006 GUIDELINES

The New Zealand Society for Earthquake Engineering (NZSEE) initial evaluation procedure (IEP) was used to assess the percentage compliance of Avon House with the current building standard, which is referred to as the percentage of New Building Standard (%NBS) [5]. The IEP began with the collection of general information about the building structure. The underlying geotechnical stratum for Avon house was slightly to moderately weathered sandstone rock and can be categorised as A or B rock. The building is from a cluster of structures that were built before 1935. Using this information and a calculated period of 0.12 sec for this URM building structure, a nominal %NBS is calculated. The case study building is within a 2 km radius of the Wellington fault line and is a single storey high; therefore, a near source factor (A) of 1 was used for the IEP. The current seismic zonation of New Zealand [2] suggests a seismic hazard factor (Z) of 0.4, and consequently a factor B (B= 1/Z) of 2.5 was used. Building importance level is accounted for by the introduction of factor C in the IEP which in is taken as 1 (for general occupancy building), whereas factors D and E represent the effects of ductility and structural performance in the IEP. To account for possible critical structural weaknesses, the performance achievement ratio is multiplied by the baseline %NBS to get the actual %NBS. In the last step the building structure was given a grading from the pre-set standard, based on its %NBS score. The IEP results were given in Table 1.

Table 1: Evaluation based on percentage New Building Standard (%NBS) [5]

Direction	*Baseline %NBS = $(\%NBS)_{nom}$xAxBxCxDxE*							*Performance achievement ratio (PAR) = Ax Bx Cx Dx Ex F*							*% NBS*
	Nom. %NBS	*A*	*B*	*C*	*D*	*E*	*Baseline % NBS*	*A*	*B*	*C*	*D*	*E*	*F*	*PAR*	
Longitudinal	*4.8*	*1*	*2.5*	*1*	*1.29*	*1.18*	*18.3*	*1*	*1*	*1*	*1*	*1*	*2.5*	*1.22*	***46***
Transverse	*4.8*	*1*	*2.5*	*1*	*1.29*	*1.18*	*18.3*	*0.7*	*1*	*1*	*1*	*1*	*2.0*	*1.22*	***26***
Result	*Building is **potentially earthquake prone** in the transverse direction and falls in the seismic grade **D** (As %NBS<33%)*														

As the IEP score of this building in the transverse direction was less than 33% NBS and less than 66% NBS in the longitudinal direction, the NZSEE guidelines suggest that the building is potentially earthquake-prone in the transverse direction and earthquake-risk in the longitudinal direction.

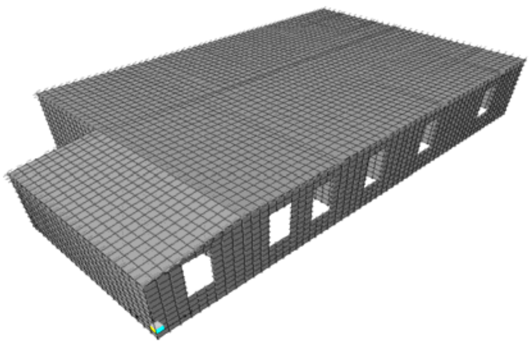
DETAILED EVALUATION

Several modelling techniques including macro-modelling, micro-modelling and homogenised finite element modelling have previously been used for the analysis of URM structures. The homogenised finite element modelling technique appears to be effective for continuum models in which structural elements are represented in detail and local failure can be clearly captured. The seismic behaviour of Avon House was determined using a finite element model with homogenised masonry and timber properties as recommended by NZSEE guidelines for New Zealand URM building stock. The values of material constants used in the homogenised model are given in Table 2.

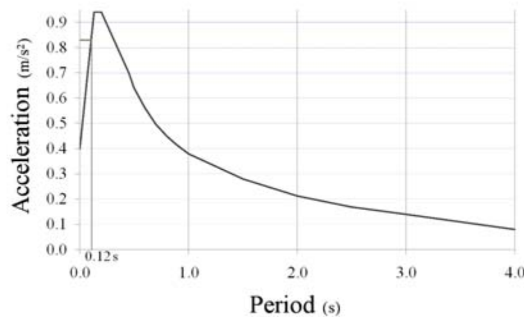
Table 2: Material properties used for modelling

Material	Density (kg/m^3)	E (GPa)	Poisson's Ratio
Masonry	1835	5	0.2
Timber	545	12	0.2

The building foundations were modelled using hinged supports because the building has URM strip foundations resting directly onto moderately weathered rock. A seismic analysis was performed with 5% damped site elastic response spectra drawn as per NZS 1170.5. Figure 5 shows the finite element model and elastic response spectra.



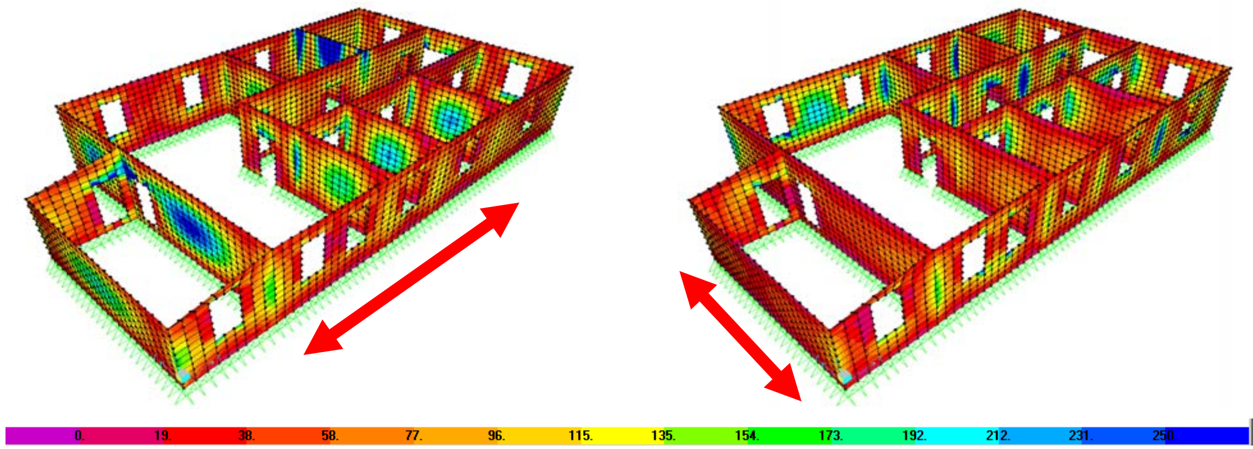
(a)



(b)

Figure 5: Computer model: a) homogenised finite element model; b) site elastic response spectra

The flexible diaphragm was modelled with homogenised timber properties and applying linear constraints were applied at wall to diaphragm joints restricting any moment transfer at the connection. However, finite element modelling of URM buildings with flexible diaphragm is computationally expensive and laborious, because a large number of modes are required to satisfy the code requirement of 90% modal mass participation in both orthogonal directions [5]. The building model was analysed for 300 modes and only those modes which had a high mass participation from walls were considered. As mentioned previously, the calculated first mode period of the building was 0.12 seconds and a corresponding lateral acceleration of 0.83 m/s^2 was obtained from Figure 5(b). The wall stress trajectories suggested that the maximum in-plane shear stresses were developed in the piers of Face 1. The stress trajectories shown in Figure 6 indicated a peak in-plane stress in piers 1, 2, 3 and 4 (Fig. 2) of 102 kPa, 101 kPa, 99 kPa and 89 kPa respectively.



(a) **(b)**

Figure 6: Finite element analysis: a) stress trajectories for lateral loading in longitudinal direction of the building (kPa); b) stress trajectories for lateral loading in transverse direction of the building (kPa)

The actual shearing force was calculated from the model results and compared with the in-plane strength calculated using the NZSEE guidelines suggested Equations 1-4 [6].

$$\tau_s = \frac{1.5C + \mu p}{1 + 3C\alpha_v / p} \tag{1}$$

$$\tau_j = \frac{C + \mu p}{1 + \alpha_v} \tag{2}$$

$$\tau_b = \frac{f_{bt}}{2.3(1+\alpha_v)}\sqrt{1 + \frac{p}{f_{bt}}} \tag{3}$$

Where τ_s, τ_j and τ_b, represents the shear strength of the wall pier corresponding to bed joint sliding, mortar failure in joints and diagonal shear cracking respectively. The wall piers were analysed using a calculated overburden axial stress p of 50 kPa and a shear ratio $\alpha_v = H/D$ calculated from pier geometry. The values of the parameters for lime based firm mortar and stiff common bricks were taken as: cohesion of bed joint $C = 0.2$ MPa, friction coefficient $\mu = 0.6$, compressive strength of mortar $f_{mc} = 4$ MPa, compressive strength of bricks $f_{bc} = 15$ MPa, direct tensile strength of masonry $f_{bt} = 1.5$ MPa. The least of these stresses were multiplied with its shearing cross sectional area to get the shear capacity of the wall pier, using Equation 4 [6].

$$V_u = \tau_u \times Dt \tag{4}$$

The in-plane strength was calculated for critical wall piers using values of constants taken from the NZSEE recommendations, and the shearing lateral load acting at top of the pier, V_d, was calculated from the finite element model. Table 3 gives the in-plane analysis results for the critical wall piers 1-4 shown in Figure 2 and Figure 6.

Table 3: In-plane Analysis Results

Results / Pier No.	Shearing force V_d (kN)	Pier geometry				In-plane shear strength V_u			*Check* $V_d<V_u$
		H (mm)	*D* (mm)	*t* (mm)	α_v $=H/D$	V_s (kN)	V_j (kN)	V_b (kN)	
Pier -1	9.4	1827	2302	220	0.8	26.9	96.6	278.6	OK
Pier -2	6.2	1827	1746	220	1.0	9.4	43.2	124.4	OK
Pier -3	3.6	1827	904	220	2.0	2.6	15.1	43.6	Fails*
Pier -4	8.5	1827	3053	220	0.6	26.9	96.6	278.6	OK

*Pier 3 is predicted to fail in bed joint sliding.

Walls 1 and 2 shown in Figure 2 were checked for out-of-plane stability using the step-by-step procedure, assuming that the walls are regular and that a single horizontal crack will be developed at mid-height [7]. Table 4 shows the out-of-plane analysis results for these walls.

Table 4: Out-of-plane Analysis Results

Result / Wall No.	Axial Stress (kPa)	Soil type	Seismic Hazard Factor	Wall geometry			%NBS	Wall Classification
				H (mm)	t (mm)	H/t		
Wall -1	50	A or B Rock	0.4	3350	220	15.2	52	Low Hazard**
Wall -2	50	A or B Rock	0.4	3350	110	30.4	24	Moderate Hazard**

** categorized as per NZSEE guidelines [5]

It was concluded that Walls 1 and 2 had insufficient capacity when subjected to face loading.

DISCUSSION AND SELECTION OF RETROFIT SOLUTION

The most common deficiencies in URM buildings are unbraced parapets and weak wall-diaphragm connections [8-10], and most of the URM structural damage reported to occur during earthquakes is caused by these deficiencies [11]. Given that Avon House is positioned on strong ground strata, it is assumed that there are no deficient foundations which can pose seismic hazard and therefore the foundations were not considered for any seismic retrofit. It has been shown by the detailed evaluation of the building structure that the URM walls were deficient for both in-plane and face loading but that out-of-plane stability was critical. The reversibility and architectural preservation requirements excluded the surface applied retrofit techniques, hence fibre reinforced polymers and other surface treatments were not considered further. Table 5 shows a literature review summary of available seismic retrofit solutions and discussion on their relative merits and demerits. It was concluded from literature review that the most viable option for such an architecturally sensitive building is post-tensioning to enhance the strength capacity of URM walls. The use of post-tensioned masonry was introduced in 1950's as the modern engineered construction and later used effectively for retrofit of URM walls [12-15]. It is believed that the post-tensioning is most beneficial for walls with very small overburden stresses and large transverse loads [16].

Table 5: Literature review summary

	Strength increase	Disadvantages	Advantages	Reference
Centre Core Steel Inserts	In-plane strength by a factor of 2-3 and an increase in ductility also out-of-plane strength increased	- Anchorage problem, high cost - Coring inaccuracy for buildings taller than 4 storey high	- No architectural impact and quick	[11, 17, 18]
Post-tensioning	In-plane strength by a factor of 5-6 and reasonable increase in out-of-plane strength	- Relaxation losses and anchorage problems, - Skilled personnel required - Rapid strength degradation	- No architectural impact, - Reversible, ideal for heritage structures - Effectively increase capacity of walls with low axial stresses and high lateral loads	[14, 16]

RETROFIT DESIGN

The post-tensioned walls were analyzed for face loading using a conceptual model based on benchmark events [16]. Equations 5, 6 and 7 represent this conceptual model, which calculates the out-of-plane flexural capacity of masonry walls. The notations M_c, M_h and M_n represents the moments developed at benchmark events.

$$M_c = \frac{I_n}{c}\left[f_r + \left(\frac{P_v + P_{sw} + A_{ps} f_{se}}{A_n} \right) \right] \tag{5}$$

$$M_h = \left(P_v + P_{sw} + A_{ps} f_{se} \right) \left[d_{eff} - \frac{2\left(P_v + P_{sw} + A_{ps} f_{ps} \right)}{3\left(\lambda_h f'_m b \right)} \right] \tag{6}$$

$$M_n = \left(P_v + P_{sw} + A_{ps} f_{se} \right) \left[d_{eff} - \frac{\left(P_v + P_{sw} + A_{ps} f_{ps} \right)}{2\left(\lambda_n f_{bc} b \right)} \right] \tag{7}$$

The symbols used in Equations 5-7 are: M_c = applied moment at crack penetration; M_h = applied moment at hinge formation; M_n = moment capacity at nominal strength; I_n = net moment of inertia of the masonry; c = distance of extreme compression fibre to neutral axis; f_r = modulus of rupture; P_v = overburden vertical load producing axial compression on the masonry; P_{sw} = axial load due to self weight; A_{ps} = area of pre-stressing steel; f_{ps} = tensile stress in pre-stressing tendons at nominal strength; f_{se} = effective stress in pre-stressing tendons after all losses; A_n = net cross sectional area of the masonry; d_{eff} = distance of extreme compression fibre to centroid of tension reinforcement; f'_m = specified compressive strength of masonry; b = width of cross section; λ_h = parameter representing the fraction of maximum compressive stress at nominal strength; λ_n = parameter representing the fraction of maximum compressive stress at hinge formation.

Bending moment developed for applied lateral loading M_d was calculated for a simply supported unit width strip of URM wall and to meet this demand a post-tensioning scheme was designed. The variables for design were the tendon diameter and initial pre-stressing force. By using the values of the required pre-stress force and maximum permissible tendon stress, tendon diameter was calculated and 12 mm Grade 500E bars were used for post-tensioning from available sizes of 12 mm, 16 mm, 20 mm, 25 mm and 32 mm. An optimal pre-stress force of 56.5 kN was applied to the mechanically restrained threaded steel bars and a calculated spacing of 900 mm on centres was used to create an effective pre-stress (i.e., magnitude of the stress acting on net cross sectional area of masonry wall) of 0.29 MPa to URM walls. All pre-stress losses due to jack tension, friction between tendon and duct, elastic shortening of wall and long term effects of shrinkage and creep were considered in design by using an effectiveness ratio (i.e., the ratio of effective pre-stress and applied initial pre-stress) of 0.85. Table 6 shows the input data and results for out-of-plane analysis of post-tensioned wall.

Table 6: Input data and results of analysis based on benchmark events

	Input data								Results	
	b *(mm)*	λ_n	A_{ps} *(mm^2)*	P_v *(kN)*	P_{sw} *(kN)*	f_{ps} *(MPa)*	f_{se} *(MPa)*	f_{bc} *(MPa)*	M_d *(kN-m)*	M_n *(kN-m)*
Wall-2 Post-tensioned	1000	0.85$^+$	113	0.55	6.63	398^{++}	412^{+}*	15	5.43	6.91
Result	As M_n>M_d , Wall-2 after post-tensioning is safe.									

$^+$ the equivalent rectangular stress block parameter which was taken from another research work [19].
$^{++}$the permissible tendon stress at nominal strength calculated as the minimum of 0.74f_{pu} and 0.82f_{py} [20].
$^+$*the effective stress in tendons after losses

Because the nominal out-of-plane strength of the post-tensioned wall was greater than the moment developed at peak loads, it was concluded that the post-tensioned wall is safe.

CONCLUSIONS AND RECOMMENDATIONS

Initial evaluation suggested that the building was potentially earthquake prone in transverse direction and requires a detailed evaluation, whilst the results of detailed evaluation showed that URM walls were deficient in both the in-plane strength and the out-of-plane stability, however out-of-plane stability of URM walls was more critical. As the building structure was architecturally sensitive, therefore application of low level post-tensioning seismic retrofit solution was selected and a post-tensioning scheme was designed using an existing bench mark based analysis approach. It was concluded from analysis results that building is safe after retrofit application and meets 100% NBS requirements of current legislation

As discussed before in building introduction, the veneer ties were deteriorated due to corrosion and should be replaced with high strength self drilling helically deformed stainless steel tie rods at a NZSEE recommended minimum spacing of 600 mm vertically and 900 mm horizontally [5].

Self drilling stainless steel ties were used because they have the least architectural impact and are not susceptible to corrosion.

Post-tensioned tendons were used to increase the in-plane and out-of-plane capacities of URM walls. A retrofit application, as determined by the design in previous section, was proposed with a tendon spacing of 900 mm and a minimum edge distance of 220 mm. All tendons used were Grade 500E threaded steel bars with nominal yield and tensile strengths of 500 MPa and 680 MPa respectively. The required pre-stress was created by applying an initial pre-stressing force of 56.5 kN to the mechanically restrained 12 mm threaded steel bars.

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