Seismic Strengthening of a Deep Basement Wall in a High Seismicity Area: A Case Study in Wellington

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ABSTRACT

This paper presents a case study on the design and construction of the seismic strengthening of a 12m deep existing basement wall in Wellington. The site is in challenging ground conditions with liquefiable soils overlying highly fractured rock and a high groundwater level connected to the Wellington harbour. The basement wall comprises soldier piles anchored back with three rows of rock anchors. It also underpins part of the foundation of neighbouring buildings. The strengthening comprises new anchors and waler beams to supplement the existing rock anchors. This strengthening project was undertaken in response to the pressing need for enhanced structural resilience.

The design challenges that had to be overcome included optimal positioning of new anchors, available headroom, presence of live building services, and support for the adjacent building foundations. Consequently, the design team had to constantly identify and innovate alternative load paths to ensure that the continuity of the retaining system could be achieved and that the stiffness of the new anchors is tuned to provide adequate protection to the existing anchors to avoid progressive anchors “unzipping” mechanism.

The project’s construction phase presented challenges, including installing the heavily loaded anchors, constricted workspace within the basement, collapsing anchor bore and the unanticipated disparities between the historical design documents and the actual as-built conditions.

The experience and lessons from this project will offer valuable insights for fellow professionals tasked with strengthening deep basement walls and existing structures.

1 INTRODUCTION

1.1 Project background

The project site is in Wellington CBD at 1 Willis St, opposite the Old Bank Building. On the site of 1 Willis St stands the Aon Centre Tower, the second tallest tower in Wellington, formerly known as the BNZ Centre and State Insurance Building. The 103 m tall tower, once the tallest building in New Zealand, features 27 floors above ground and three basement levels. The Client wanted to understand the seismic performance of the basement wall, particularly the South wall. This paper describes the seismic assessment (in accordance with Part C of the Seismic Assessment of Existing Buildings, MBIE, 2017) and the strengthening design of the South Wall.

1.2 South Wall

The existing South Wall is a tied-back, three-level basement wall parallel to the car park ramp from Victoria Street (east) to Willis Street (west). This retaining wall has a retained height ranging from 11.5 to 12.5 m, with the greater height near Willis St. It features 900 mm diameter bored piles encasing 560 mm rectangular welded steel plate box sections, typically spaced 1.8 m apart and extending up to 15.5 m in length.

Each pile is tied back with three layers of rock anchors, which are 100 mm in diameter, 14 m long, and inclined at 15 degrees from the horizontal, with a fixed length of 9.1 m. BBRV G35 and G50 strand tendons are used for anchors over the Caledonian and National Mutual Building extents, respectively. Generally, there are no waler beams to connect the piles, but in some areas, the basement slabs act as struts to provide a shared load path between the piles. Waler beams are present across the air intake shaft on the Willis Street side. The bored piles also permanently underpin the Caledonian and National Mutual Buildings (Smith et al., 1975).

1.3 Ground conditions

Based on published geology (Begg et al., 2000), the site is covered by Marine Sediments overlying bedrock. The bedrock in the Wellington region comprises folded and faulted indurated sandstone (Greywacke), siltstone and mudstone (argillite). Figure 1 provides an indicative long section illustrating the interpreted soil layers based on the available geotechnical data. Due to the site's proximity to the shoreline, the groundwater conditions were closely associated with tidal levels. Groundwater was taken as the mean sea level for the assessment (i.e. RL +0.2 m).

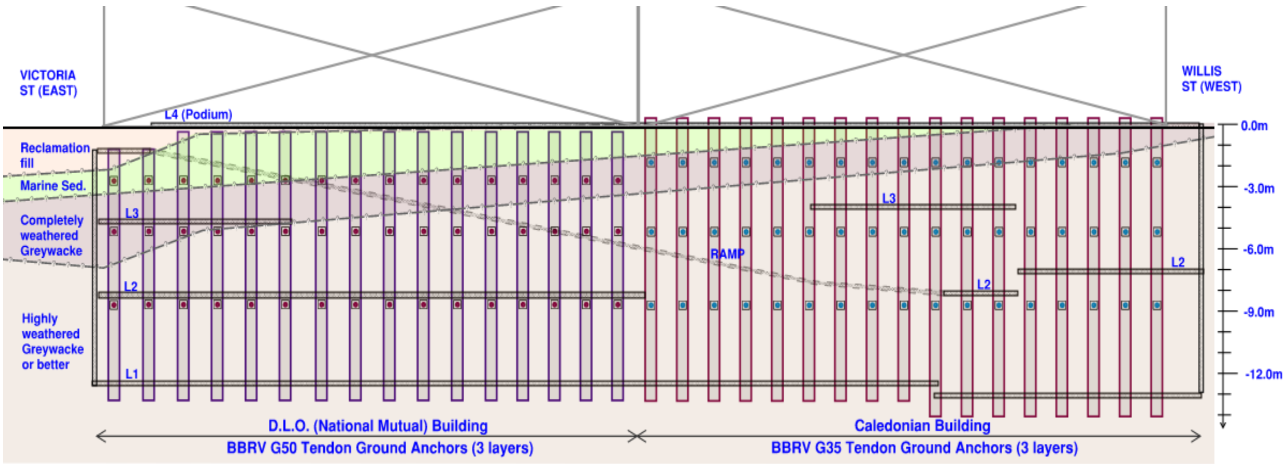

Figure 1: Indicative elevation view of the South Wall

2 ASSESSMENT OF THE EXISTING WALL

2.1 Seismicity

The seismically active Wellington Fault is approximately 2 km northwest of the site. The design earthquake shakings are presented in below. The derived PGA was used to determine the site's liquefaction potential and calculate seismic earth pressures.

Table 1: Ground seismic hazard

	Return Period	PGA, g	M_{eff}
Serviceability limit state (SLS)	25 years	0.11	6.2
Ultimate limit state (ULS)	500 years	0.45	7.1

2.2 Geotechnical issues

The geotechnical issues that could impact the south wall were identified as follows:

1. Large seismic earth loads;
2. Liquefaction of the Marine Sediments (loss of strength and stiffness) causing increased lateral earth pressures. It was assessed that this layer is susceptible and likely to liquefy at a 55% ULS level or higher earthquake shaking.

2.3 Seismic loading on the wall

Figure 2 illustrates the seismic loading imposed on the South Wall. These are briefly described below.

1. Seismic soil and groundwater loading calculated in accordance with NZGS Module 6 and Wood and Elms (1990). A 'stiff wall' scenario was considered.
2. Inertia from the wall self-weight.
3. Podium loading. The seismic gap between the podium slab and the main block causes lateral forces that make the wall above ground level act like a cantilever beam, transferring the load as an equivalent moment and shear to the top of the posts.
4. Loads from the adjacent building that the South Wall underpins. This comprises the direct base shear resulting from the underpinned part of the adjacent buildings and the surcharge loading from the floor slab and shallow foundations within the zone of influence behind the wall.

2.4 Resistances

Figure 2 illustrates the resistances available to the South Wall. These are briefly described below.

1. Passive resistance of the embedded portion of the wall.
2. Rock anchors – see Section 2.4.1 below.
3. Strut restraints – see Section 2.4.2 below.

2.4.1 Rock anchors

The rock anchors were grouted into a 100 mm diameter bore. The probable geotechnical anchor capacity was assessed as 850 kN. This assumed an average grout-to-country bond of 300 kPa, which accounts for the progressive debonding of long rock anchors. The properties of the rock anchors are summarised in Table 2 below.

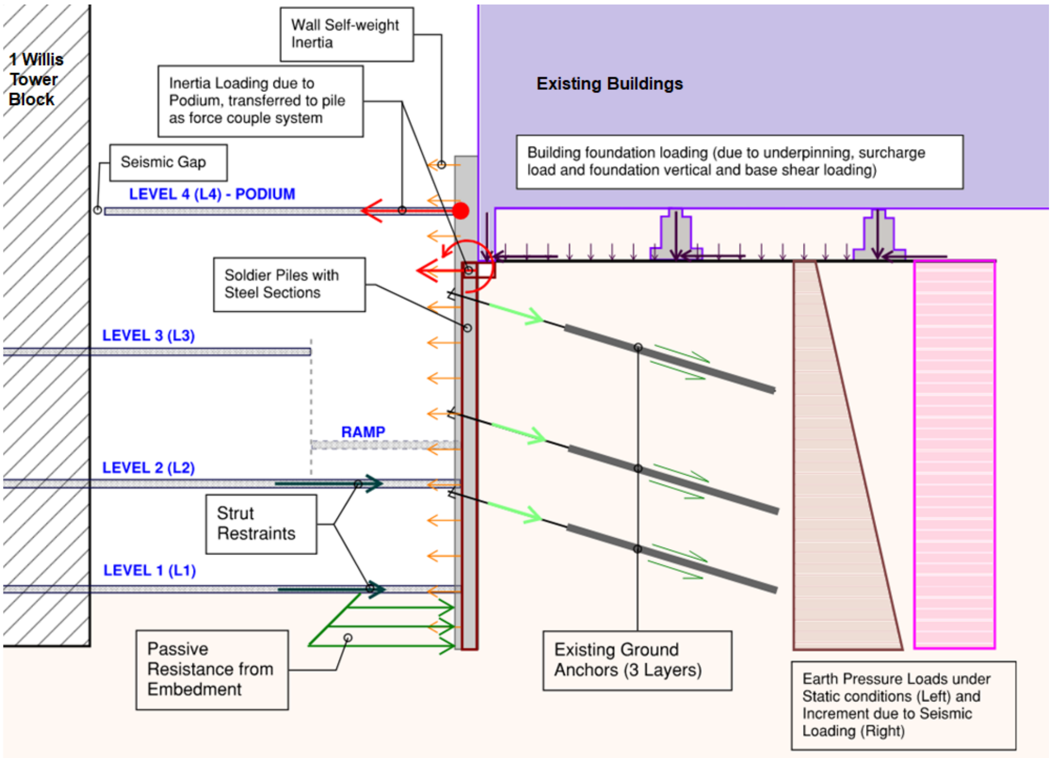

Figure 2: Illustration of the different loading and available supports on the South Wall

The structural engineer advised that the maximum capacity of the tendons of these existing rock anchors was to be limited to 90% of their ultimate tensile strength (UTS). Therefore, the structural limiting capacity of the existing rock anchors was assessed as 462 kN and 694 kN for the BBRV G35 and G50 tendon anchors, respectively. This meant that the tensile capacity of the tendons limits the capacity of the existing ground anchors.

Table 2: Properties of existing anchors

	Strands	**Free length [m]**	**Bond length [m]**	**Stiffness [MN/m]**	**Prestress force [kN]**	**UTS [kN]**	**0.9*UTS [kN]**
G35	8 × 7 mm	4.9	9.1	12.6	360	514	462
G50	12 × 7 mm	4.9	9.1	19.0	540	771	694

2.4.2 Strut restraints

In addition to the rock anchors, the South Wall is restrained by some strutting action from the basement slabs. As shown in Figure 2, only Level 1 (bottom basement floor) and Level 2 floor slabs are considered to offer lateral restraint to the South Wall. Level 3 floor slab does not extend to the wall for the most part due to the carpark ramp. Level 4 (ground) floor slab has a seismic gap, resulting in loading rather than restraining the wall.

2.5 Soil-structure interaction assessment

Due to the complex support conditions (anchor/struts) and/or pile spacing and/or retained heights, six representative cross sections were developed for the soil-structure-interaction (SSI) analysis using WALLAP (Geosolve) retaining wall design software. To understand the locked-in stresses in the wall and anchors due to the anchor prestress and underpinning loading on them, the modelling sequence started from the construction of the South Wall (with assumptions on the construction, excavation and support sequence) to present-day condition. Seismic loadings are then applied to understand the seismic performance of the existing system.

2.5.1 Sensitivity analyses

Various parameters that affect the analysis were identified during the assessment. A series of sensitivity analyses were then conducted to check the parameters' effect to allow evaluation and judgement of the final parameters to be adopted for the assessment and strengthening design. These are summarised in Table 3 below.

Table 3: Summary of sensitivity analyses

Parameter	Description, range considered
K_o (Greywacke)	Range explored: 0.75 (lower bound), 1.0 (Reference), 1.5 (upper bound) The effect of varying the earth pressure at rest (K_o) coefficient of the Greywacke rock on the results was explored. Potential variability can emanate from depth, overburden pressure, stress history and the rock's geological properties. Displacements, member forces and anchor/strut forces increase with increasing K_o value, but only marginally. Considered as not being critical, and reference value adopted.
Podium mass	Range explored: earthquake acceleration at C(0) or PGA versus the structure's spectral acceleration, C(t). The obvious consequence is that the podium loading with C(t) resulted in higher loading on the wall and, therefore, on the resulting anchor forces (+15%). This was adopted for further assessments.
Column *EI* value	For the long-term conditions, the wall flexural stiffness (*EI*) is considered as either 50% of the upper bound *EI* (steel + concrete), the cracked *EI*, or the *EI* of the steel only (assuming long-term concrete cover spalling and no concrete inside the steel section). Some nuanced differences in the results. The stiffer the wall the less the deformations and the higher the member forces. However, the stiffer the wall, the lesser the anchor/strut forces. As a result, it was decided to adopt *EI* of the steel section only.
Sea level rise	The assessment considered the effect of sea level rise by increasing the design water level by half the expected sea level rise over the design life (an additional 0.2 m). No appreciable difference in the anchor forces, and only +1-2% changes were noted. This effect was not considered further.

2.5.2 Assessment results

The assessment of the existing wall had the following main findings:

1. Significant lateral loads are imposed on the south wall from external sources (e.g. podium, neighbouring building base shear), leading to an overstress of the top row of anchors. The lack of strutting support near the top portion of the wall also exacerbates this. The chart in Figure 3 below shows the anchor load demand of the three layers of existing rock anchors over the stages of analysis. The anchors' breaking capacity is also shown for comparison.
2. At the ULS seismic event, the instability mechanism starts with the first row of anchors being overstressed and possibly failing. This is followed by overloading the lower-level anchors and perhaps leading to the 'unzipping' effects. On the western half of the wall, where the weaker G35 ground anchors are used, most of the existing rock anchors on the second row are also overstressed during a ULS seismic event. This is worsened by the carpark ramp interrupting the L2 and L3 floors from strutting the south wall over the Western half and the foundation loads imposed by the Caledonian Building being higher than those on the Eastern side.

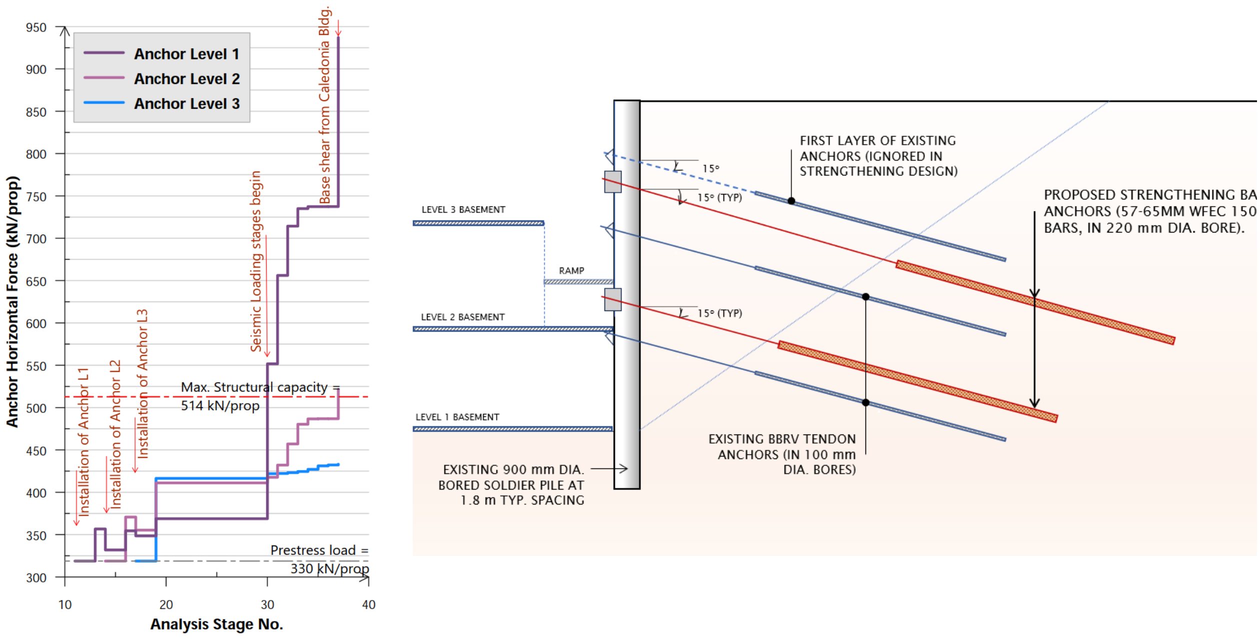

Figure 3: Anchor demand evolution for the three layers of G35 ground anchors (right), and schematic diagram showing the relative positions of the existing and proposed strengthening works (left)

3 STRENGTHENING DESIGN

3.1 Design scheme

As a result of the assessment findings, the client required the South Wall to be strengthened. Through collaborative optioneering with the structural engineer, the client and the strengthening contractor, the preferred strengthening solution was to add high-strength bar anchors at critical locations and layers. Some features of the scheme include:

1. The existing anchors in the top row were considered unreliable as they are heavily loaded from external loads. Accordingly, they were discounted from the strengthening scheme.
2. The new anchor locations were optimally selected to prevent the piles from being overloaded and to offer 'protection' or added resilience to the existing rock anchors.
3. The new anchors' stiffnesses were tuned by applying appropriate prestress load to protect the existing anchors from overloading.
4. The bar anchors proposed were 57mm and 65 mm diameter Williams 1050 MPa threaded bars. The chosen bar size also provides extra tensile capacity for added resilience at the appropriate cost.
5. Each additional anchor was to be placed between the posts and connected to them via a waler beam spanning between the two posts. Compared to the original design, this achieved continuity of the wall with alternative load paths if required, leading to a more resilient and better-performing system.

3.2 Challenges

Some challenges faced during the strengthening design of the additional rock anchors are discussed below.

3.2.1 Positioning of anchors

Positioning the new anchors presented significant challenges due to space constraints and the accessibility of the proposed points. The building was also to remain tenanted during construction. Additionally, the ongoing strengthening of the basement floors meant that certain areas would become unavailable as construction progressed. The South Wall also housed live services such as ventilation ducts, electricity supply, and firefighting hydrants, necessitating careful placement of the new anchors to avoid these zones and amenities. Constructability was another crucial factor, requiring early engagement with a specialist contractor to ensure anchor positions were accessible and practicable to construct.

Figure 4 shows a 3D model of the proposed rock anchors with their tolerances (grey cones), adjacent building foundations and other potential obstructions. Each anchor position was checked and scrutinised thoroughly before finalising the design.

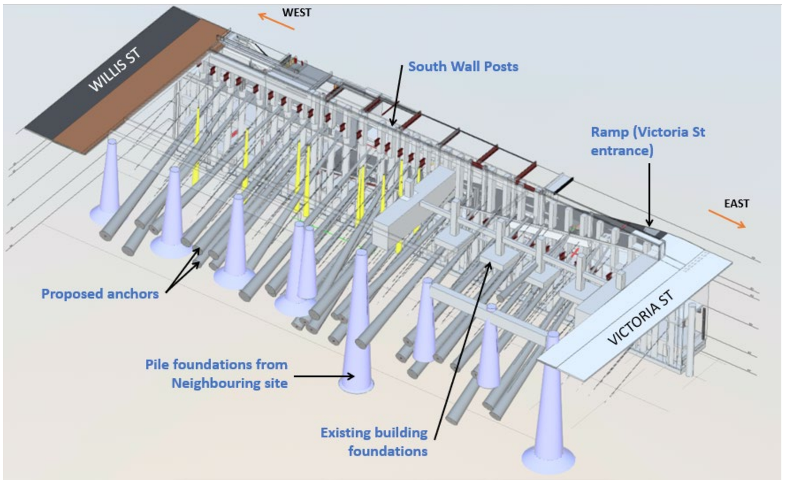

Figure 4: 3D model of the proposed anchors and other potential obstructions

The final strengthening design scheme included 34 nos. new anchors, typically at an inclination of 15-deg from the horizontal. Steel struts propped to basement floors were used to support the wall in areas where new anchors were not feasible due to space and access constraints.

3.2.2 Anchor proving tests

During the developed and detailed strengthening design, two investigative (sacrificial) test anchors were undertaken to inform the grout-to-country bond capacity. Two different bore diameters were tested, with the results as summarised in Table 4 below. The sacrificial anchors were constructed with a 3 m bond length into highly weathered Greywacke or better and were tested to failure. The low residual bond stress (typically 25% to 30% of the peak) is reflective of the brittle nature of rock anchors in Greywacke in the Wellington region.

Table 4: Summary of investigative tests carried out at the site

Test no.	Bore Diameter [mm]	Bond length [m]	Peak stress [kPa]	Residual stress [kPa]	Remarks
1	220	3.0	723	198	-
2	200	3.0	557	140	Softer drilling reported

Based on these tests, a peak bond stress of 600 kPa and residual bond stress of 175 kPa were adopted for the new anchors, offering an ultimate geotechnical tension capacity of over 2.0 MN per anchor. The peak bond stress was used over 3 m of the permanent anchors, while the residual bond stress was used for the remainder of the bond length. This is to simulate the progressive debonding of long bond length and brittle failure nature of rock bond in Greywacke. Because of the testing undertaken and experience with anchors bonded into Greywacke, a strength reduction factor of 0.70 for a tension-only anchor can be justified.

3.2.3 Waler beams

As noted above, the spacing of the piles and general geometry varied along the wall, making a simple standard waler design impossible. Three main types of walers were used, but each required bespoke connection details to suit site conditions. Most of the walers were steel sections, and breaking out the piles to expose the steel section to connect to it required. On a few occasions, cast in-situ reinforced concrete walers were used.

4 STRENGTHENING CONSTRUCTION

The new rock anchors were installed progressively. The installation was closely inspected, and a robust quality control regime was followed. All anchors were acceptance tested and stressed to 120% of their working load for two cycles before finally being locked off at the prescribed prestressing load. Figure 5 below is a photo of the acceptance testing of one of the anchors and the Waler beams. The subsections below describe challenges encountered during the project's construction phase.

Figure 5: Construction photos showing a new ground anchor's acceptance test in progress

4.1 Design challenges and changes

Some issues uncovered during the early stages of construction affected the proposed design, particularly the anchors' placement and prestress loads. The main design challenges came from the condition of the existing anchors. Upon removing the skin wall, which was acting as a façade over the South Wall, it was possible to inspect the existing anchors. As the first row of rock anchors was discounted from the strengthening design, their condition was of lesser concern. However, some of the second row of anchors were either non-existent or some anchor strands were fully corroded within the free length. As a result, the design had to be updated to reflect the missing or corroded existing rock anchors. The consequence was that some new anchors were required, and other planned new anchors had to be upsized or relocated to be more effective.

On one occasion, the drilling for a new anchor partially cut through an existing anchor, which was discovered in the CCTV inspection after the drilling was completed. The design was then checked and amended to consider the effect of the severed existing anchor.

4.2 Other construction challenges

Figure 4 demonstrated that the anchor positions were decided based on the checks against potential obstructions. However, much of the information is based on old records from the older buildings, which was not always accurate. Therefore, caution during drilling was imperative to avoid drilling through the existing foundations. On a couple of occasions, the anchor appeared to have hit an obstruction at depth, and drilling immediately ceased. The position of the anchors was shifted (usually by adjusting the inclination), and drilling resumed. If the anchor approached its full depth, the remediation used a slightly reduced free length to achieve the full design bond length.

The presence of soil layers for the top row of new anchors also posed a construction challenge regarding groundwater inflow and soil collapse, which needed to be cased temporarily. Hole stability was an issue due to the collapse of highly fractured and jointed Greywacke at the crown of the bore. Flushing the bore proved difficult due to the diameter and depth of the bores. This required vacuum excavation to clean out the hole of debris, followed by a CCTV inspection of the bore.

Other challenges during construction included working platforms for the drilling rig and crew to install some of the anchors due to access constraints, spoil management and noise and dust control. All these led to a slower production rate and programme pressure.

5 CONCLUSION

Strengthening existing buildings brings about extreme challenges compared to new builds, typically because of access constraints and various unknowns or surprises. Strengthening a tenanted building in a high seismic area like Wellington doubles the challenge. In addition to technical knowledge, the collaboration of all parties is imperative for a successful project. The 1 Willis project pulled together a team that had worked collaboratively for many years. The team successfully navigated challenging ground conditions and design constraints through innovative problem-solving and collaborative teamwork. The lessons from this case study provide valuable insights for engineers and professionals facing similar challenges in strengthening existing structures. By sharing these experiences, the authors consider this project useful for improving methodologies and approaches in future strengthening endeavours.

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