WELLINGTON ELECTRICITY RESILIENCE PROJECT:
A STRUCTURAL AND GEOTECHNICAL APPROACH
FOR THREE REPRESENTATIVE SUBSTATIONS

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SUMMARY

This paper presents the structural and geotechnical aspects from Wellington Electricity’s seismic resilience project, which included upgrades to substation structures across the Wellington Region. The goal of the project was to improve the resiliency of the network when subjected to a major seismic event. Three example strengthening schemes are presented which highlight the collaborative nature of the work between structural and geotechnical disciplines and the Client.

INTRODUCTION

Wellington Electricity (WE*) has been carrying out a programme of seismic assessment and strengthening since 2012 targeting pre-1976 substation buildings. The 2016 Kaikoura earthquake reinforced the importance of this project and a further 91 substation buildings were selected for assessment and strengthening as required. WE* developed a criticality rating system for their electricity network substation buildings which was used to inform the required seismic strengthening. Zone substations, defined as substations which cannot be isolated and by-passed, were rated as the most critical buildings for the network. Non-zone substations were considered less critical as these could be isolated and by-passed if necessary.

AS/NZS 1170.0 requires electrical infrastructure buildings be designed to Importance Level 3 (IL3). WE*, in consultation with Jacobs’ engineers, agreed a strategy of adopting an Importance Level 4 (IL4) for all zone substations and IL4 for non-zone substations where practical and feasible. IL4 designation effectively categorised the zone substations as buildings with ‘special post-disaster function’.

The IL4 designation for all zone substations meant they must be designed for a second serviceability limit state, i.e. SLS2, whereby the building suffers only minor damage and maintains operational continuity after a 1-in-500 year return seismic event. In nominally ductile buildings, the seismic load demand under SLS2 is less critical than that of ULS (which is a 1-in-2500 year return earthquake event for IL4).

For non-zone substation buildings, an IL4 designation was adopted on sites which identified negligible or low risks from geotechnical hazards based on the geotechnical desktop study. Where medium to high rated seismic geotechnical risks were identified, IL3 designation was adopted and WE* accepted that the substation may not be operational after a major earthquake. This was in recognition of the fact that non-zone substations are less critical to the operation of the network.
DESIGN APPROACH

Geotechnical Considerations

The substations are typically located within or near city centres, large retail and residential areas. Wellington Region is formed of steep slopes generated from movement and upthrust of the local faults (i.e. Wellington, Ohariu and Wairarapa faults) (Van Dissen, 2019), alongside deep alluvial basins formed of geologically young and predominantly silty and sandy sediments (Begg & Mazengarb, 1996).

At the outset of the project WE* had an appreciation of the geotechnical hazard in the Wellington Region, the 2016 Kaikoura Earthquake being a timely reminder, and understood that they were likely to have a large proportion of the sites affected by geotechnical hazards. WE* had clauses in their funding model whereby funds had to be used within a two-year period of commencing the resilience project. To support this programme, Jacobs developed a process for early identification and management of geotechnical hazards aimed at providing a balance between seismic performance (IL3 vs IL4), and the cost/programme to deliver a site-specific investigation. The principal characteristics of the process chart are set out below:

- IL4 adopted where geotechnical desk study indicated low risk of geotechnical hazards;
- Site specific ground investigations would be considered when medium or high-risk hazards were identified in the geotechnical desk study;
- Zone substations warranted a comprehensive ground investigation, due to the criticality of the performance to the wider network;
- GNS were consulted where sites were located on or near active faults irrespective of the nature or criticality of the substation.

Jacobs recommended a single geotechnical desk study covering all 91 sites. The findings were presented in a single summary schedule with a covering report setting out the methodology and assessment criteria. The structure and content of the table was developed such that it provided a quick and simplified summary to WE* and the structural engineer, thus facilitating early decision making and advancement of the preliminary seismic assessments.

A large proportion of the substations were built in the 1950’s and 1960’s when there was only fairly limited consideration of geotechnical hazards. The desk study identified that liquefaction was the predominant geotechnical hazard, along with slope stability, depending on the position of the building within the surrounding landscape. A small number of sites were subject to fault rupture (abrupt surface movement). An example of this is the active Ohariu fault (GNS Science, 2015), where the published fault rupture zone extended below or very near to three substations in the Porirua Basin. Figure 1 illustrates this.

Surprisingly, there were few sites where the site subsoil classification (Class C/D boundary) was not clear, and no ground investigations were scoped as part of this project to determine the Class C/D boundary. This is a reflection of both the publicly available geotechnical data (NZGD, 2019) and published papers which cover this topic, i.e. (Boon, et al., 2011) and (Semmens, 2010). It also echoes a commitment from WE* and Jacobs to take an appropriately conservative approach to this parameter, in lieu of spending a considerable sum on deep boreholes and seismic testing. In other words, for any single site, the cost of the investigations would in some cases be more than the cost of the strengthening scheme itself, therefore, warranting a conservative approach. The geotechnical assessments were guided by the C4 section of the Seismic Assessment Guidelines (NZSEE, 2017).
Structural Considerations

Electrical substation buildings in Wellington vary significantly in nature, with almost every substation building unique in some way, presenting a key challenge to the project. The building stock varied in age from buildings built in the 1920’s to the 1980’s. To improve seismic resilience the substation buildings were strengthened to either 100%NBS (IL3) or alternatively 67%NBS (IL4), as recommended in the Seismic Assessment Guidelines (MBIE, EQC, NZSEE, SESOC, and NZGS, 2017).

For both the assessment and the retrofit of the buildings, the design seismic demand was based on a 50-year design working life, in accordance with requirements of the New Zealand Building Act. However, given the age of many of the buildings, the strengthened structures will likely have less than 50-year design working lives going forward.

Most of the buildings were either brick or blockwork masonry construction, with a small percentage comprising singly-reinforced concrete structures. The brick buildings were predominantly unreinforced. The level of reinforcing in the blockwork masonry buildings varied from unreinforced masonry to heavily reinforced masonry. Most buildings had a relatively small footprint of approximately 6m by 4m, however, the zone substations had larger footprints and, in many cases, significant associated transformer yard structures. Given that most structures were either brick or blockwork, an assumed structural ductility of 1 or 1.25 was typically used to derive the design seismic forces.

The geotechnical input into the Detailed Seismic Assessment (DSA) and the structural strengthening design included seismic sub-soil type classification and identification of relevant hazards and risks to the structure identified in the desk study. The structural engineer would then discuss the risks and hazards with the geotechnical team and plot a way forward for the structural design.

To capture the best possible knowledge of the likely performance of each structure, including likely deformation considerations, a SAP2000 model was produced for each building. The models identified likely deformations and allowed the team to assess deformation compatibility issues.

The most common structural seismic performance issue for many of the buildings related to the out of plane capacity of the walls of the building; mainly due to the lack of an effective diaphragm at the ceiling level of these building. The second most common issue was related to the in-plane capacity of walls under seismic lateral load.
For the small 6m by 4m simple rectangular substation buildings, where enough reinforcing was provided in the blockwork, steel collars around the tops of the building walls were enough to markedly increase the structural seismic performance of the building and thereby the %NBS score for the building. This strengthening strategy mitigated the main structural issue with many of the buildings that did not have existing ceiling diaphragms. The advantage of using external steel collars was to reduce the time spent inside the substation, thereby reducing the exposure risks associated with strengthening work within a live substation.

For larger buildings, options for strengthening walls to ensure in plane seismic performance included the use of shotcrete and in some cases steel cross braces. To mitigate the out of plane failure mode, either shotcrete or steel whalers were used to reinforce the existing walls. In some cases, it became impractical to use an external steel collar due to the large spans between walls that required very large steel sections, therefore, steel cross braced ceiling level diaphragms were used.

Safety in Design

The seismic strengthening was carried out in live substations with the associated risks of electrocution and of working in a confined space.

To minimise the risks associated with working above live electrical equipment, every effort was made to seek strengthening solutions that reduced the need to install ceiling diaphragm systems. In some cases, this was unavoidable due to practicality and cost considerations.

Construction details had to be thoroughly thought out to ensure efficient constructability and avoid prolonged exposure of workers to these risks. Only contractors experienced in this type of work were used and were often asked to participate in design meetings to help ensure the safest practicable strengthening solution.

SAMPLE SUBSTATIONS STRENGTHENING SCHEMES

SUBSTATION S2058 HERETAUNGA, UPPER HUTT

The substation is located at 180 Fergusson Drive, Heretaunga, Upper Hutt. The building was constructed in 1987 and is a single story and regular in shape. It measures approximately 7.4m by 3.6m, and approx. 3m in height. It consists of reinforced masonry walls acting in each direction. The roof does not act as a rigid diaphragm. The building is founded on shallow footings with a concrete slab on grade (Refer to Figure 2).

The building rating was found to be 15%NBS (IL4), governed by the out-of-plane shear capacity of the masonry walls. The building was assessed using the current earthquake loading standards NZS 1170.5:2004 with Importance Level 4 and a design working life of 50 year. The strengthening works have been designed to 67%NBS (IL4), for the ultimate limit state (ULS), and full serviceability limit state (SLS1 and SLS2) requirements.

Geotechnical Considerations

The geotechnical desktop study identified the site as low risk of liquefaction. Jacobs' had previously undertaken extensive investigations in the immediate vicinity where liquefication was discounted as a geotechnical hazard. No geotechnical investigations were considered necessary for this substation.
Structural Strengthening Required

In the strengthening design, the existing elements have been assessed using their probable capacity and the new elements added have been designed using yield capacity. The proposed strengthening for the walls is presented in Table 1, Figure 3 and Figure 4.

Table 1. S2058 Strengthening design summary

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Element strengthened</th>
<th>Strengthening</th>
</tr>
</thead>
<tbody>
<tr>
<td>All external walls</td>
<td>In-plane</td>
<td>Walls</td>
<td>No strengthening required.</td>
</tr>
<tr>
<td></td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>The top support at the bond beam level was strengthened by adding a PFC.</td>
</tr>
</tbody>
</table>

Figure 3. S2058 Proposed strengthening plan at near top bond beam level

Figure 4. S2058 Proposed strengthening elevations
SUBSTATION S1009, SILVERSTREAM, UPPER HUTT

The substation is located at 20 Sutherland Avenue, Silverstream, Upper Hutt City. The building, which is single story and regular in shape, was constructed in 1988. It measures approximately 14m by 5m, and approx. 3.2m in height. The building structure comprises masonry walls with partially-filled lightly reinforced concrete blockwork supported on concrete strip footings and a concrete slab on grade. The light roof is made up of ‘brownbuilt’ decking and is assumed to have 10mm construction ply supported by 4¾” x 1¾” RSJ purlins. The purlins are supported by four 7” x 4” portal frames. No rigid diaphragm action was identified in the roof structure to transfer the masonry block wall load to the perpendicular walls. Additionally, internal walls are present which are considered non-structural in nature but are integral for the substations since they function as blast protection. Consequently, these internal walls are also strengthened to the same criteria. The switch room and toilet are located externally, and were constructed at a later date, presumably around 1972 (Refer to Figure 5 below).

![Figure 5. 20 Sutherland Avenue substation](image)

The building rating was 15%NBS(IL4) governed by the out of plane flexural and shear capacity of the block walls. This rating is attributed to the block walls having less than minimum reinforcement and thus are considered unreinforced for design purposes. The building was assessed using the current earthquake loading standards NZS 1170.5:2004, with Importance Level 4 and a design life of 50 years. The strengthening works have been designed to 67%NBS (IL4), for the ultimate limit state (ULS), and full serviceability limit state (SLS1 and SLS2) requirements.

**Geotechnical Considerations**

The geotechnical desktop study identified the site as medium risk of liquefaction, on the basis of the underlying alluvial deposits. The superficial deposits in the Hutt Valley are known to be highly heterogenous, therefore, liquefaction could not be discounted.

**Ground Investigations**

The ground investigation scope comprised two 10m deep machine boreholes and two 15m deep Cone Penetration Tests (CPTs). The CPTs were provisional items, to be completed following the findings of the machine boreholes. The purpose of this approach was to understand the fines content, gravel/cobble contents and relative densities to assess if the site could be penetrated by a CPT. The ground investigations encountered dense to very dense sandy gravels with occasional cobbles throughout, which were considered unsuitable for CPTs. Liquefaction was discounted due to the presence of the dense to very dense gravels.
Structural Strengthening Required

The DSA identified that the external block walls require strengthening. The scheme selected for this purpose is external application of shotcrete with 1 layer of reinforcement (vertical and horizontal bars). Existing connections between walls and steel portal frames also required improvement. In order to redistribute out-of-plane load from the longitudinal walls to the side walls, a roof bracing system is provided.

Internal walls were strengthened by vertical and horizontal PFCs; vertical PFCs are connected to existing purlins via a horizontal PFC at roof level. Existing connections between internal and external walls were considered poor quality, and thus were not included in the analysis (except for part height wall). To improve the out-of-plane performance of the wall additional supports were provided at corners with angle struts and a horizontal PFC. Lateral deflection of the roof is controlled through cleats connecting the existing purlins and the new horizontal PFC.

In the strengthening design, the existing elements have been assessed using the probable capacity and new elements added have been assessed using yield capacity. The proposed strengthening for this is presented in Table 2, Figure 6 and Figure 7.

Table 2. S1009 Strengthening design summary

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Element strengthened</th>
<th>Strengthening</th>
</tr>
</thead>
<tbody>
<tr>
<td>All external walls</td>
<td>In-plane</td>
<td>Walls</td>
<td>110mm thick shotcrete with HD12-200 vertical and horizontal reinforcement (with extra bars at specified locations). Foundation extension is also required at specified locations for starter bars.</td>
</tr>
<tr>
<td>All external walls</td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>110mm thick shotcrete with HD12-200 vertical and horizontal reinforcement (with extra bars at specified locations). Foundation extension is also required at specified locations for starter bars.</td>
</tr>
<tr>
<td>Fire partition</td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>2 vertical 150PFC (1 at each side), both connected at top to existing purlins through horizontal 150PFC; 2 horizontal 150PFC (one at top, another below gap for busbar).</td>
</tr>
<tr>
<td>Part height partition</td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>1 horizontal 150PFC at top right corner propped to adjacent wall (extended wing wall).</td>
</tr>
<tr>
<td>Extended wing wall</td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>2 vertical 150PFC (1 at each side), both connected at top to existing purlins through horizontal 150PFC; 3 horizontal 150PFC (one at top, one at h=2.1m, another at h=0.9m).</td>
</tr>
<tr>
<td>Bond beam levels of walls</td>
<td>Out-of-plane</td>
<td>Steel to wall connection</td>
<td>12mm thick steel flat plate cleats connecting each side of steel column flange to shotcrete by M16 through-bolts.</td>
</tr>
<tr>
<td>At roof level</td>
<td>Both</td>
<td>Portal frame</td>
<td>2-100x100x6 SHS cross bracing at each bay.</td>
</tr>
<tr>
<td>At roof level</td>
<td>Out-of-plane</td>
<td>Internal walls</td>
<td>250PFC at internal wall locations connected to SHS eaves struts.</td>
</tr>
<tr>
<td>At top bond beams</td>
<td>-</td>
<td>-</td>
<td>Seat angles with 4-M16 through-bolts connecting SHS eaves struts to external walls.</td>
</tr>
<tr>
<td>Top of side walls</td>
<td>Out-of-plane</td>
<td>Steel portal frame</td>
<td>Steel angle with 5-M20 bolts each corner, connecting end portal frames to side walls.</td>
</tr>
<tr>
<td>Switch room</td>
<td>Out-of-plane</td>
<td>Switch room walls</td>
<td>Angle cleats with 2-M16 bolts each leg at 3 bond beam levels connecting to main structure walls.</td>
</tr>
<tr>
<td>Toilet Room</td>
<td>-</td>
<td>-</td>
<td>To be demolished</td>
</tr>
</tbody>
</table>
SUBSTATION S1018 TITAIH BAY, PORIRUA

The substation is located at Mana Avenue, Titahi Bay, Porirua City. The existing drawings show that the building was constructed circa 1969. This rectangular, single storey structure is approximately 15m by 5m on plan, and 4.5 m in height. All walls are reinforced concrete with an intermediate column for East and West walls. The roof is a concrete slab supported on an L-shape beam with no proper connection for lateral load transfer and inadequate seating width. The roof concrete is not acting as a diaphragm or providing lateral support for the walls. The building foundation comprise a concrete slab on grade and strip foundations supporting the walls (Refer to Figure 8).

Figure 6. S1009 Proposed strengthening plan at roof level

Figure 7. S1009 Proposed strengthening elevation

Figure 8. Mana Avenue, Titahi Bay, Porirua City Substation
The building rating was found to be 15%NBS (IL3). The building was assessed using the current earthquake loading standards NZS 1170.5:2004, with Importance Level 3 and a design working life of 50 year. The strengthening works have been designed to 100%NBS (IL3) for both the ultimate limit state (ULS) and serviceability limit state (SLS1) requirements.

Geotechnical Considerations

The geotechnical desktop study identified the site as medium risk of liquefaction. The site lies at the boundary of three geological units: Holocene Alluvium, Pleistocene to Holocene Alluvium and Holocene shoreline deposits.

Ground Investigations

The ground investigation scope comprised two 10m deep machine boreholes and two 15m deep CPTs. As with S1009, the boreholes were carried out in advance of the CPTs in order to assess the suitability to advance a CPT. The findings of the ground investigations showed a significant difference in the deposits at either end of the structure. The northern end indicated loose to medium dense sands with lenses or organic material (inferred to be Holocene Alluvium); the southern end indicated medium dense to very dense silty sand with occasional lenses of peat (inferred to be late Pleistocene Alluvium). The CPTs were terminated at approximately 6.0m and 4.5m due to reaching the weight capacity of the rig.

Liquefaction Analysis

The susceptibility to liquefaction was assessed using the simplified procedure (Boulanger & Idriss, 2014) for both the CPT and standard penetration test (SPT) data. The findings between the two methods were consistent, indicating that liquefaction was not occurring at the northern end of the substation. However, liquefaction was occurring to the south under SLS2 and ULS shaking. This was interpreted to be as a result of the differing age and composition of the geological units.

Predicted free field and building settlements

The total free field settlements were calculated using both the CPT and SPT data. As SPTs are typically less reliable, two different methods (Tokimatsu & Seed, 1987) and (Ishihara & Yoshimine, 1992), were used to evaluate the approximate total liquefaction induced free field settlement. Where CPT data was available in the upper soils, settlements were evaluated using the simplified method (Boulanger & Idriss, 2014). The results are presented in Table 3, which show the calculated total free field settlement. Building settlements were estimated as a combined result of volumetric strain, shear induced failure and sediment ejecta (Bray & Macedo, 2017). Due to the variability in ground conditions, the liquefaction potential across the substation also varied. Therefore, the input to the structural design was given in terms of a differential settlement. The crust of non-liquefiable deposits was considered when providing settlements to the structural team. However, it was determined that little reliance could be placed on this material to support the structure’s foundations. This is as a result of the nature and estimated relative density of this material.

Estimates of differential settlements were provided to advance the structural strengthening design, which are summarised in Table 4. These were provided as differential settlements due to the presence of liquefaction on one side of the structure and not the other.
Table 3. S1018 Predicted total settlements

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Settlement CPT (mm)</td>
<td>Total SPT settlement below 6 m (mm)</td>
</tr>
<tr>
<td>SLS2</td>
<td>30</td>
<td>300</td>
</tr>
<tr>
<td>ULS</td>
<td>40</td>
<td>300</td>
</tr>
</tbody>
</table>

Table 4. S1018 Predicted differential settlements

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Settlement at southern edge (mm)</th>
<th>Settlement at northern edge (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td>&lt;10</td>
<td>100</td>
</tr>
<tr>
<td>SLS2</td>
<td>&lt;10</td>
<td>80</td>
</tr>
<tr>
<td>SLS1</td>
<td>&lt;10</td>
<td>&lt;10</td>
</tr>
</tbody>
</table>

Structural strengthening Required

Roof bracing to support and confine the walls was provided as a strengthening solution. Screw piles are proposed as a strengthening solution to control the overturning of the building. These are founded below the liquefiable Holocene soils.

Initially the piles were proposed to be located outside the building, walls and columns with foundations extended to transfer load on the pile. The strengthening scheme was still, however, under development at the time of preparing this paper. A more substantial foundation strengthening system is being considered, comprising screw piles positioned across the entire footprint of the building, supplemented with in-situ ground beams with the aim of mitigating the liquefaction effects and increase resistance overturning. This solution would achieve 100%NBS (IL3).

The proposed strengthening is presented in Table 5 and Figure 9 to Figure 11.

Table 5. S1018 Strengthening design summary

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Element strengthened</th>
<th>Strengthening</th>
</tr>
</thead>
<tbody>
<tr>
<td>External/internal walls</td>
<td>In-plane</td>
<td>Walls</td>
<td>No strengthening required.</td>
</tr>
<tr>
<td></td>
<td>Out-of-plane</td>
<td>Walls</td>
<td>Providing roof bracing with strut between East and West wall. Fixed UB as columns at west wall between intermediates columns.</td>
</tr>
<tr>
<td>Connections</td>
<td>Out-of-plane</td>
<td>Foundation</td>
<td>Fixing piles on the extended walls foundation of East and West walls as shown in the drawing.</td>
</tr>
</tbody>
</table>

Figure 9. S1018 Proposed strengthening plan at roof level
DISCUSSION

Management of seismic strengthening projects for large stock of industrial buildings requires careful planning and good communication between the various stakeholders. The geotechnical and structural engineers must work closely to understand how the critical interactions between geotechnical site conditions and structures will impact the building’s seismic performance. In places with high seismic risk, geotechnical considerations have the potential to govern the %NBS score and require significant and costly solutions. Attention must be given to the principal risks associated with geotechnical step change phenomena including: liquefaction, slope stability, lateral spreading and fault rupture. Engineers must also work closely with clients to ensure that the seismic performance of the strengthened buildings meet their expectations.

An understanding of the building importance levels designated in the AS/NZS 1170.0 loading standard is required for all stakeholders in the seismic strengthening process. The current New Zealand Building Act and AS/NZS 1170.0 require buildings related to electrical infrastructure to be design to IL3 unless the building is designated as having a ‘special post-disaster function’. Achieving the post-disaster SLS2 performance requirements, for existing building infrastructure, is challenging for buildings of the typical form and age of WE**’s substations.

Going forward consideration should be given in the seismic guidance documents for design performance criteria specifically for building associated with critical infrastructure. This allows
for strengthening of existing building to between the SLS2 performance of IL4 and SLS1 of IL3 to reduce the potential burden on utilities of costly replacement of buildings.

REFERENCES


NZSEE, 2017. The Seismic Assessment of Existing Buildings; Part C - Detailed Seismic Assessment; C4 - Geotechnical Considerations, s.l.: Ministry of Business, Employment and Information; Earthquake Commission.

