

DIZHUR



DETAILED SEISMIC ASSESSMENT & CONCEPT RETROFIT REPORT

ROTORUA MUSEUM

GOVERNMENT GARDENS ORUAWHATA DRIVE, ROTORUA

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EXECUTIVE SUMMARY

DIZHUR Consulting was commissioned by Rotorua Lakes Council to undertake a detailed seismic assessment and detailed concept retrofit design of the Rotorua Museum, located at Government Gardens, Oruawhata Drive, Rotorua (GMS: S 38°08'07.7" E 176°15'32.9") to seismically upgrade the structure as part of development works to re-open it.

The Rotorua Museum building is considered a landmark structure in Rotorua, and a nationally renowned Historic Category 1 building. The building was built and used as a bath house for over 50 years, as a result of the Government's desire at the time to use the geothermal waters of the region by erecting baths and bath rooms which drew from the surrounding mineral springs and lakes. The earliest part of the building dates from 1908, and features an Elizabethan Revival style of architecture, derived from the original movement in England, wherein it followed typical internal layouts of government sanatoriums during the time period. These included vast corridors to provide access to public and private rooms to their sides.

The on-site investigation, detailed seismic assessment and concept retrofit work of Rotorua Museum was undertaken by DIZHUR Consulting throughout the period from May – December 2021, and the conclusions are summarised below:

Zones 2 to 5: The museum today is comprised of six zones, which are a combination of historical and newer constructions: Zones 2 to 5 comprise the original historic areas. Zone 3 is the Central Atrium, and is flanked by Zone 2 to the south, and Zone 4 to the north. Both these structures are largely original, and are separated from Zone 3 by approx. 10m tall concrete gable walls. The east and west walls of all three zones comprise approx. 1.5m tall concrete nibs topped with timber framed walls infilled with approx. 40-50mm thick pumice concrete panels. Zone 5 lies to the north of Zone 4, separated by another 10m tall concrete wall, and is of largely the same construction as the previously mentioned zones, whilst also containing a number of internal partition walls of similar configuration to the east and west perimeter walls, with lath and plaster internal linings. Zones 2 to 5 all sit on a pumice concrete two-way vaulted floor slab supported on original unreinforced pumice concrete columns, walls and foundations, with localised newer basement construction having been undertaken in various regions over the years. The roof structure across these zones typically consists of heavy ceramic roof tiling supported on timber sarking and trusses. Detailed information regarding Zones 2 to 5 can be found in [§4.2](#) to [§4.5](#).

Zones 1 and 6: Zone 1 lies to the south of Zone 2, and was partially undertaken at around the original time of construction. It was extended in 1983, and again in 2011. This zone is composed of a combination of reinforced concrete masonry walls and steel framing, with a central 'core' of reinforced concrete masonry walls enclosing the lift and staircase structures. It contains a concrete slab on grade, with the use of precast reinforced concrete ribs and timber flooring at upper levels. Zone 6 lies to the north of Zone 5, and was constructed circa. 2007-2008. It was built predominantly using cast in-situ reinforced concrete walls and plywood timber walls, with steel portal framing supporting timber roof rafters and clay roof tiling, and concrete masonry walls at basement level. Detailed information regarding Zones 1 and 6 is provided in [§4.6](#) and [§4.7](#).

Crack investigations and monitoring: An in-depth investigation into the existing cracking observed within the vaulted floor and numerous walls of the structure was undertaken, in an effort to understand the experienced deformations and possible causes, and to develop a possible remediation methodology for the vaulted floor structure. Point cloud analyses of laser scanning data was undertaken to identify the amount of deformation experienced and the associated possible cracking mechanisms, and concluded that large amounts of differential settlement had been experienced by the structure, most notably the relative settlement of the large concrete gable walls with respect to the rest of the structure's foundations. Due to the observations made regarding the extensive cracking of the vaulted floor, a comprehensive crack monitoring study was commenced with the installation of several manual crack monitoring gauges in May 2021. The gauges provided evidence within 5 weeks that the structure was still experiencing movement. Considering the relatively short timeframe in which movement was identified, a more robust crack monitoring system composed of displacement

transducers connected to a fully automated acquisition system was installed in the structure on the 17 December 2021. This system will be utilised to monitor the building throughout its future life stages and to continually inform the detailed design and construction stages of the project. Detailed information regarding cracking investigations and monitoring is provided in §5 and Appendix 3.

Material testing: Testing of the pumice concrete collected from the infill panels and of the unreinforced pumice concrete utilised for the walls and vaulted floor of the structure was undertaken. Samples of timber were also extracted from the perimeter wall framing at several locations for identification of the timber species. Detailed information about material testing is provided in §5.1 and Appendix 2.

Concrete material quality: The concrete was generally found to contain voids and was poorly compacted. Several core samples extracted from walls were found to contain cracks through the cross section of the core sample, which was not discernible from the wall surface, see photographic examples in §6.1.1.

Concrete compressive strength:

A total of 19 core samples were extracted, laboratory test results are as follows (see §6.1.2 and Appendix 2):

- The average **compressive strength of cylindrical concrete core** samples is 9.0 MPa (CoV 41%).
- The average **concrete density** is 1332 kg/m³ (CoV 5%).
- The average **modulus of elasticity** is 6.0 GPa (CoV 37%).

Concrete tensile strength:

A total of 17 core samples were extracted, laboratory test results are as follows (see §6.1.3 and Appendix 2):

- The average **tensile strength of cylindrical concrete core** samples is 0.8 MPa (CoV 42%).
- The average **concrete density** is 1252 kg/m³ (CoV 7.5%).

Concrete (for infill panels) compressive strength:

A total of 18 cube samples were extracted, laboratory test results are as follows (see §6.2 and Appendix 2):

- The average **compressive strength of cubic concrete samples** is 30.7 MPa (CoV 5.5%).
- The average **concrete density** is 1530 kg/m³ (CoV 3%).

Timber species identification:

Timber samples were extracted from several locations of the original timber framed portion of the perimeter walls and the mezzanine floor structure belonging to Zone 3. It was concluded that all samples were found to be Totara.

Detailed Seismic Assessment (DSA): The detailed seismic assessment of the structure was undertaken with multiple approaches to address the newer and the original historic zones. In addition, given the large dimension of the structure and the complexity of its composition, the building was divided into the 6 zones above described, which have been examined individually.

Zones 2 to 5 (original zones) (see §7 to §13):

Analysis: The seismic response of the structure was determined through a series of Nonlinear Static Analyses (pushover analyses NLSPA) as recommended by the most recent international standards for seismic assessment and rehabilitation of existing structures (ASCE SEI 41/17, Eurocode 8, NTC2018 and The Seismic Assessment of Existing Buildings (the Guidelines) (2017)). The building and most of its components are modelled adopting the Finite Element (FE) software SAP2000. Given the complex nature of the structure, its irregularities and of its loading conditions, numerous models were defined so that the nonlinear static pushover analyses were examined both from a global perspective (considering the building in its entirety i.e., a single model comprising Zones 2 to 5), and from a local perspective (studying the response of single structural elements). The building was modelled from the foundation level including also the

basement structure, however in parallel, another model neglecting the basement and floor structure was defined and used for the analyses.

The typical outcome of a pushover analysis is the capacity curve (or pushover curve), which is a curve representing the building shear capacity versus the displacement of a control point. The safety level of the structure is defined by the % of new building standard (NBS) achieved.

Following NLSPA, nonlinear time history analyses (NLTHA) was performed on the zones comprising the original structure (i.e., single model comprising Zones 2 to 5). The purpose of these analyses were to substantiate the results obtained via NLSPA which was the main assessment approach adopted for the design of the retrofit intervention, as well as to provide supplementary information about the estimated dynamic behaviour of the building when subjected to seismic load.

Retrofit philosophy and strengthening concept: The philosophy adopted for the retrofit scheme (to achieve 80% NBS (IL3)) of the original zones was to utilise as much of the existing structural capacity as possible, with supplementation provided by 'surface applied' materials (typically Cross Laminated Timber (CLT) panels, plywood, Fibre Reinforced Cementitious Matrix (FRCM) coatings, vertical strong-backs) to improve the capacity of individual structural elements. Connections between vertical and horizontal structural elements are then improved so as to enhance the overall global behaviour of the structure. This approach entails the use of materials which are compatible with those of the existing structure. As with the assessment approach, retrofit schemes were developed for each individual zone, with certain retrofit techniques implemented in multiple locations and in multiple zones (within the original zones) where possible to optimise the scheme as well as future project stages. The retrofit scheme intends to maintain the existing level of interdependency of the zones i.e., it has not been designed to increase or decrease the dependency of one zone on another. Specialised retrofit techniques have been utilised in certain areas of heritage significance (such as the east wing of Zone 5) where the use of surface applied materials was not considered appropriate. See [Appendix 4](#) for detailed concept strengthening drawings.

Zone 1 (see §14.1):

The DSA of Zone 1 has been undertaken in accordance with the latest version of the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines). The Guidelines form part of the new framework for managing earthquake-prone buildings that result from the Earthquake-prone Buildings Amendment Act 2016. A staged analysis approach was adopted for the seismic assessment of the building, starting with simpler analysis methods where appropriate and then undertaking an incrementally increasing level of sophistication of analysis to establish the expected seismic performance of the building.

Recommendations for strengthening to Zone 1 are as follows:

- Provide new epoxy anchors to the existing base-plate connections of the columns that the ReidBrace cross braces are connected to, in order to increase the strength of the connections.
- In-fill portions of the existing long & wide floor penetrations along gridlines A & 7 to improve the floor-to-wall connections and enable adequate transfer of forces to the walls.

Zone 6 (see §14.2):

The DSA of Zone 6 has been undertaken in accordance with relevant sections of the latest version of the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines) for calculating member capacities. Reference has been made to NZS 1170 for calculating loadings. The lateral load resisting system of the building has been checked based on Equivalent Static Analysis. The roof bracing checks are based on loads obtained from the Parts and Components provisions of NZS1170.5.

Recommendations for strengthening to Zone 6 are as follows:

- In-fill portions of the existing long & wide floor penetrations along gridline A8 (north side of Zone 6) to improve the floor-to-wall connections and enable adequate transfer of forces to the walls.

Concrete floor retrofit study (see §15):

Based upon the observations regarding the extensive slab cracking of the unreinforced pumice concrete vaulted floor found in Zones 2 to 5, analyses were undertaken to examine the behaviour of the floor under static gravity loading. The analysis was carried out on an isolated module of the floor which was considered representative for the global static behaviour of the structure (i.e., vaulted slab portion supported by four columns). The floor module was modelled adopting the Finite Element (FE) software SAP2000 and the FE software TNO DIANA. Following the analyses of the floor in the as-built condition, two retrofit interventions were proposed. The two interventions differentiate between the part of the floor structure which can be accessed for structural reinforcement work from the top of the floor (Zone 2-3-4 and the West part of Zone 5) and the part of the floor structure where the top surface shall remain untouched for architectural heritage purposes (East part of Zone 5).

The retrofit for Zones 2 to 4 and the west part of Zone 5 comprises a grid of reinforced concrete ribs carved inside the existing slab. An additional FRCM topping connected to both the existing floor and the new ribs is also proposed. The retrofit for the east wing of Zone 5 consists of inserting Rectangular Hollow Section (RHS) steel profiles from basement level onto the soffit of the slab, along the supporting column grid lines. The RHS steel elements are to be bent to adapt to the vaulted floor soffit and will be connected to the existing concrete floor using mechanical fasteners and to the columns via purposely designed steel “collars”.

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1 BACKGROUND AND SCOPE

The on-site investigation, detailed assessment and concept retrofit work of Rotorua Museum undertaken by DIZHUR Consulting throughout the period from May – December 2021 and reported herein was commissioned by Rotorua Lakes Council to investigate the existing structural condition of the building and the requirements for seismic upgrade to achieve 80% NBS (IL3), in order to assist in the redevelopment of the building as a museum.

Table 1-1. Summary of work at the Rotorua Museum building

Work Stream	Item Description
1. Preliminary and general items	Familiarisation with available existing information of the structure and structural layouts from existing drawings and reports. Direct visual assessment of current condition of loadbearing elements
2. Confirmation of structural configurations & materials	Investigation and confirmation of the configuration of structural elements in all zones, including detailed analyses of building deformations in historic areas and resulting damage/deterioration. Concrete elements were scanned using a Surface Penetrating Radar (SPR) to identify the presence and arrangement of reinforcing (if any). Drilling investigations (max 24mm diameter) were performed where required to better understand and interpret the SPR sections. Timber structure (roof, mezzanine floors, timber framed walls) dimensions, configurations and connections were documented. Where accessible, foundation dimensions and configurations were noted and documented on site.
3. Timber species identification	Several timber samples were extracted from the walls at various locations throughout the building to identify the timber species and subsequently investigate the material properties.
4. Concrete material properties	36 concrete core samples were extracted from pumice concrete walls and the floor slab at different locations around the building. Cores of D100 x L200-400 mm were extracted, with the following being investigated: - Compressive strength - Measure density - Assess quality and condition - Measure Modulus of Elasticity Locations of core extractions were finalised onsite with lead conservation architects and Heritage New Zealand.
5. Crack monitoring	Detailed investigation of the structural health of the structure including research into existing documentation on observed slab and wall cracking throughout the building. Point cloud analyses were undertaken to understand building settlements and related deformations. Placement of manual and digital crack monitoring gauges to commence a comprehensive record of structural movement with respect to seasonal and isolated fluctuations in temperature, vibration etc.

6. Seismic assessment

Detailed Seismic Assessment of the subject building was undertaken using New Zealand Guidelines and standards as well as the latest available research work on performance of similar structures in earthquakes.

7. Detailed concept retrofit

Detailed concept retrofit designs produced for all zones to achieve 80% NBS (IL3).

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2 BUILDING LOCATION

The subject building is located at Government Gardens Oruawhata Drive, near the Polynesian Spa and the Blue Baths buildings, see [Figure 2-1](#) (GMS: S 38°08'07.7" E 176°15'32.9"). The building lies on slightly raised ground relative to its surroundings, with the ground sloping slightly downward to the Gardens to the north and west of the building and Lake Rotorua to the east and curving around to the south of the building.

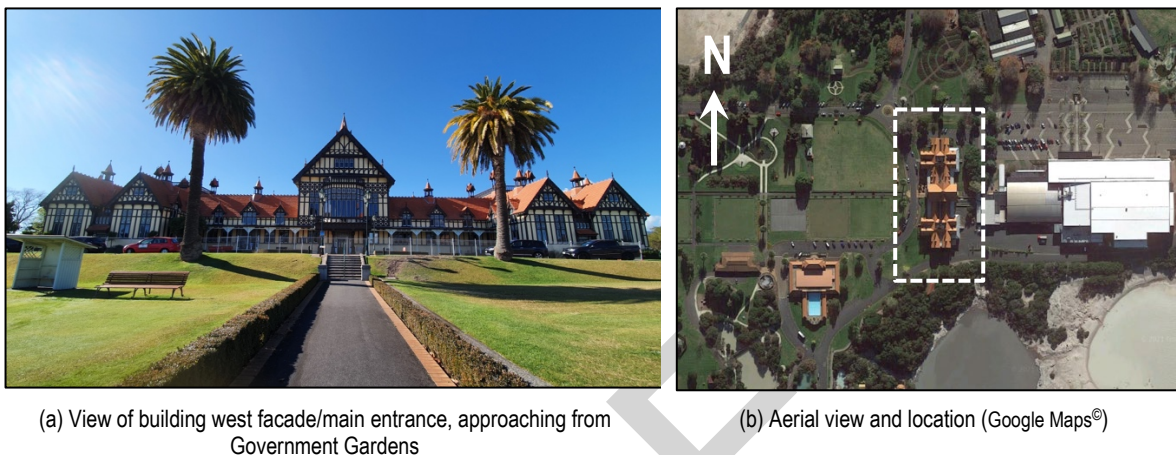


Figure 2-1. Rotorua Museum, Christchurch

3 BUILDING DESCRIPTION & HISTORIC INFORMATION

The following provides a brief background and description of the Rotorua Museum building (former Bath House). Detailed information regarding the building's founding, features and history can be found in the document titled *Rotorua Bath House (former) – Conservation Plan* produced by DPA Architects, dated April 2019.

The Rotorua Museum building is considered a landmark structure in Rotorua, and a nationally renowned Historic Category 1 building. The building was built and used as a bath house for over 50 years, as a result of the Government's desire to use the geothermal waters of the region by erecting baths and bath rooms which drew from the surrounding mineral springs and lakes. The earliest part of the building dates from 1908 and was predominantly designed by government balneologist Dr Arthur Wohlmann, with collaboration by architect Benjamin Stott Corlett. These original portions comprise the Central Atrium containing the main front entrance, two wings on either side of the Atrium, and an additional wing to the northern side of the complex. The original structure cost over £25,000, which was considered a substantial amount at that time. The original construction did not reflect Wohlmann's complete concept which was a symmetrical structure with longer north and south wings.

The baths facilitated medical treatment for several concerns such as rheumatism and psoriasis, in addition to relaxation activities such as massages and mud baths. Because it utilised the surrounding springs, the transport of the acidic water and steam and constant presence of hydrogen sulphide quickly led to maintenance difficulties throughout the following decades. Despite this, the former bath house saw vast numbers of visitors, including wounded soldiers and servicemen during both World Wars.

The building later evolved into a core part of Rotorua's cultural heritage due to its development into a museum circa. 1969, after closing as a bath house in 1966 and narrowly avoiding demolition which presented excessive costs to the Rotorua Borough Council. As an alternative to demolition, the Council had offered to undertake preservation of the building if it was given over to them with a government grant that equalled the cost of demolition. 14 years later the south

wing was further extended, before its final completion in 2011. The north wing had been extended in 2008, resulting in Wohlmann's original concept finally reaching completion after just over 100 years.

The building features an Elizabethan Revival style of architecture, derived from the original movement in England, wherein it followed typical internal layouts of government sanatoriums during the time period. These included vast corridors to provide access to public and private rooms to their sides. The building was built using local materials such as Kauri and Totara timbers, and featured a 'half-timbered' form of construction which was particularly common in New Zealand via the use of timber studs sheathed with weatherboards. This half-timbered technique was adapted to suit the bath house by using timber studs with precast pumice concrete infill panels. In New Zealand, this was the first recorded use of precast panels.

Historical photos of the structure are provided in **Figure 3-1**.



(a) Envisioned Bath House, c. 1903. Painted by Dr. Arthur Wohlmann (Archives New Zealand, Rotorua Museum Te Whare Taonga o Te Arawa)



(b) Construction of the bath house, view from eastern side of Zone 5, c. 1907 (photo provided by DPA Architects)



(c) Construction of bath house foundations showing wall strip footings, columns and column pad footings c. 1907 (photo provided by DPA Architects)



(d) Construction of the bath house c. 1907, view of eastern side of building looking northwest, c. 1907 (photo provided by DPA Architects)



(e) View of the Tudor-style bathhouse and surrounding gardens. Photograph taken by William A Price on 10 Nov 1908 just after the official opening in Aug 1908. (Alexander Turnbull Libraries, ref: 1/2-001503-G)



(f) View of bath house c. early 1930s with tennis courts in front (Alexander Turnbull Libraries, ref: PAColl-5936-25)

Figure 3-1. Historical photos of Rotorua Museum building (former Bath House)

The museum today is comprised of six zones, which are a combination of historical and newer constructions (see [Figure 4-2](#)).

Zone 3 is the Central Atrium, and comprises a large 'foyer' with two sweeping staircases leading up to a mezzanine floor. Zone 3 is flanked by Zone 2 to the south, and Zone 4 to the north. Both these structures are largely original, and are separated from Zone 3 by approx. 10m tall concrete gable walls. The east and west walls of all three zones comprise concrete nibs with timber framing and infill panels above.

Zone 5 lies to the north of Zone 4, separated by another 10m tall concrete wall, and is of largely the same construction as the previously mentioned zones. Zones 2 to 5 all sit on a pumice concrete two-way vaulted floor slab supported on original concrete columns, walls and foundations, with localised newer basement construction having been undertaken in various parts over the years.

Zone 1 lies to the south of Zone 2, and was partially undertaken at around the original time of construction. It was extended in 1983, and again in 2011. This zone is composed of a combination of reinforced concrete masonry walls and steel framing, with a central 'core' of reinforced concrete masonry walls enclosing the lift and staircase structures. It contains a concrete slab on grade, with the use of precast reinforced concrete ribs and timber flooring at upper levels.

Zone 6 lies to the north of Zone 5, and was constructed in 2008. It was built predominantly using cast in-situ reinforced concrete walls and plywood timber walls, with steel portal framing supporting timber roof rafters and clay roof tiling, and concrete masonry walls at basement level.

The Rotorua Museum is of national and international importance for its contribution to the development of international tourism in the country, in being one of the first government projects to stimulate international interest in its spas. It is commonly referred to as one of New Zealand's finest examples of Elizabethan Revival architecture. It is technologically significant for its innovative use of timber framing with lightweight precast concrete panels, and it also enables greater understanding of New Zealand's early approaches to medical science and technology. It forms an integral historical part of Government Gardens, which otherwise includes geothermal features, archaeological deposits and associated structures. The building is listed as a Historic Place Category 1 structure, entered in April 1985 at #141.

Exterior photos of the subject structure are provided in [Figure 3-2](#). Building plans for the proposed redevelopment and a selection of interior photos are provided in [Figure 3-3](#) and [Figure 3-4](#). Note that any room identification numbers are with respect to those established by DPA Architects (refer [Appendix 1](#) for room identification and gridline system).



(a) View of building main entrance from Government Gardens (west elevation)



(b) View of Central Atrium (Zone 3) main entrance and bay window (west elevation)



(c) Architectural details and turrets atop Zone 3 roof



(d) View of southeast corner of building



(e) View looking north of east elevation (Zone 3 onwards)

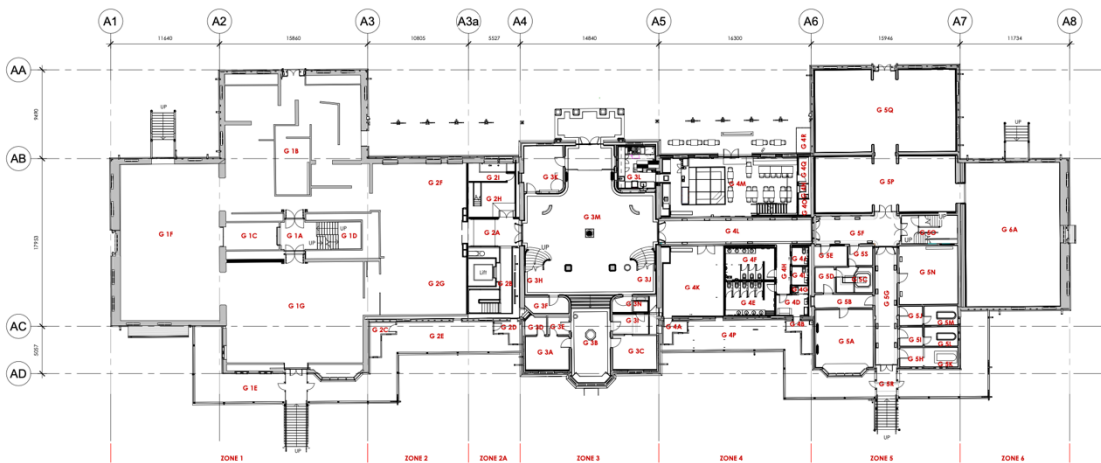


(f) View of roof walkway between Zone 4 roofs (looking south to Zone 3)

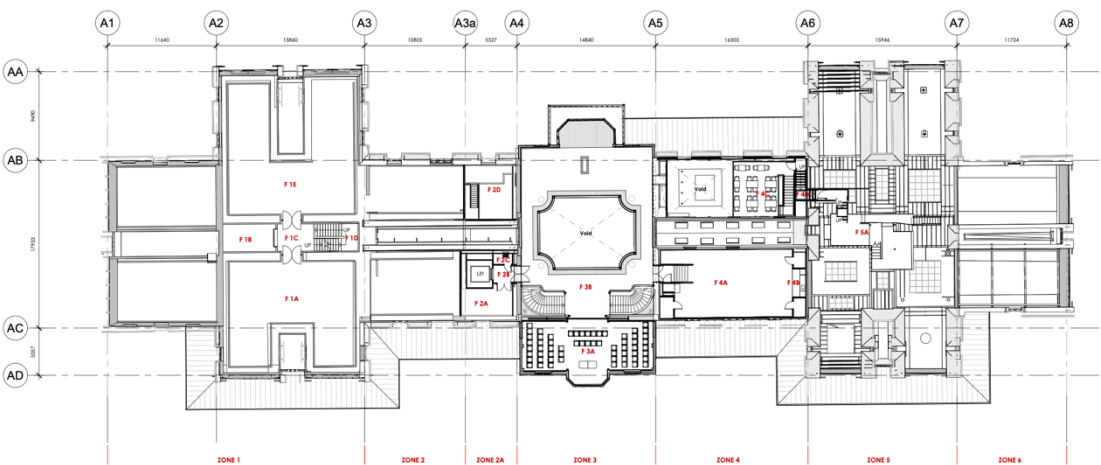
Figure 3-2: Exterior views of Rotorua Museum



(a) Proposed basement floor plan for the redevelopment



(b) Proposed ground floor plan



(c) Proposed first floor plan

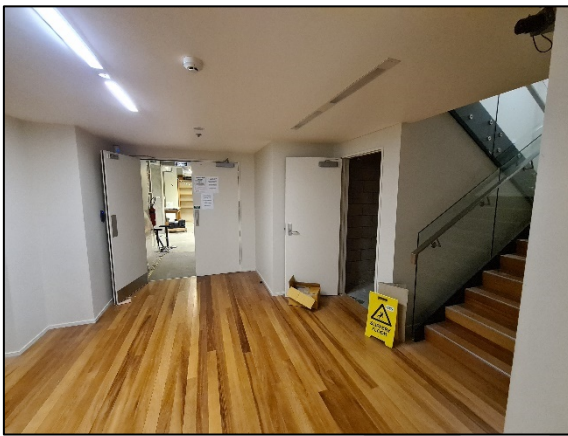
Figure 3-3. Proposed floor plans for the Rotorua Museum Development (as of Dec 2021)



(a) Zone 1 basement looking southeast



(b) View of basement looking west



(c) Zone 1 basement looking west, adjacent to lift and stairwell



(d) View of basement looking north, adjacent to stairwell



(e) Basement going through doorway in (d), looking north



(f) Basement going through doorway in (f), looking northwest



(g) Ground floor, looking northeast



(h) Ground floor, looking southwest

Figure 3-4: Interior views of Zone 1 (South Wing)



(a) View of Zone 2 basement looking north (image taken from Jetstream 2021 file)



(b) View of basement looking southeast (image taken from Jetstream 2021 file)



(c) View of ground floor looking north



(d) Ground floor looking south (east side)



(e) Ground floor looking east



(f) Ground floor looking south (west side)

Figure 3-5. Interior views of Zone 2 (Intermediate South Wing) (Tarawera Gallery)



(a) Zone 3 basement looking west



(b) Basement looking west showing vaulted floor slab lowering in height towards west



(c) Zone 3 basement walkway (north side, looking east)



(d) Basement – looking north, historic circular fountain foundation structure on LHS



(e) Ground floor looking east



(f) Ground floor looking northeast



(g) Ground floor looking west



(h) West (front) wall and mezzanine floor



(i) Mezzanine level looking west



(j) Mezzanine level looking northeast



(k) Mezzanine level – room in northeast corner
(room in southeast corner similar)



(l) Ceiling and roof trusses (looking north)

Figure 3-6. Interior views of Zone 3 (Central Atrium)



(a) Zone 4 basement corridor, looking north



(b) Basement corridor looking south



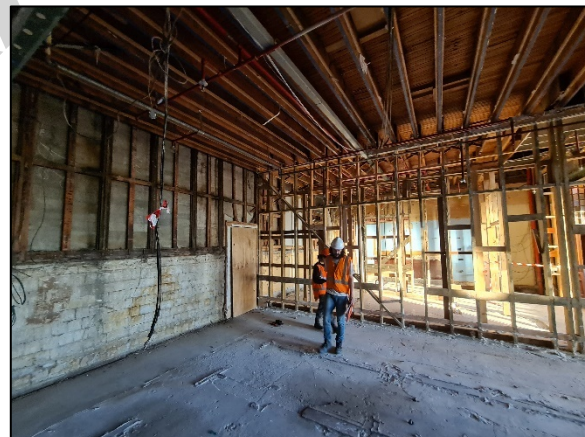
(c) Zone 4 ground floor looking north (west side)



(d) Ground floor looking south (west side)



(e) Ground floor looking to west façade wall



(f) Ground floor looking southeast (east side below mezzanine floor)



(g) Views looking north and south, respectively, of mezzanine floor on east side of Zone 4

Figure 3-7. Interior views of Zone 4 (Intermediate North Wing)



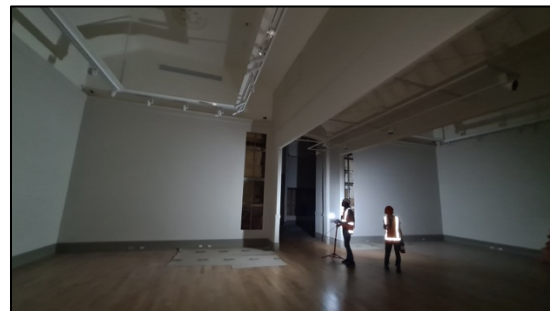
(a) Zone 5 basement looking northeast (staircase to GF in LHS background)



(b) View of basement from west side, looking east



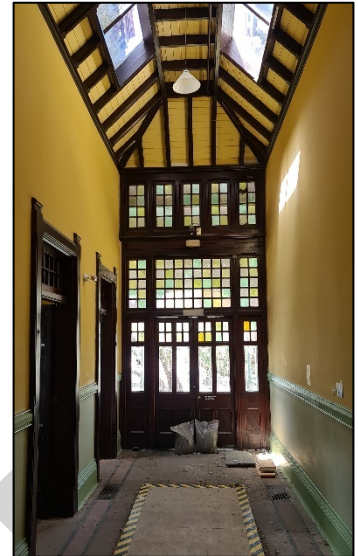
(c) View of gallery at ground floor, west side (looking northwest)



(d) View of gallery at west side, looking east



(e) Ground floor east side corridor running E-W, looking west



(f) East side corridor, looking east to entryway



(g) Cooling room (room G5A), south east corner of Zone 5, looking east



(h) Cooling room, looking northwest towards entryway



(i) Historic bath room (room G5N), east side of Zone 5 against north wall



(j) View of 1 of 3 remaining bath rooms at east end of Zone 5, looking north

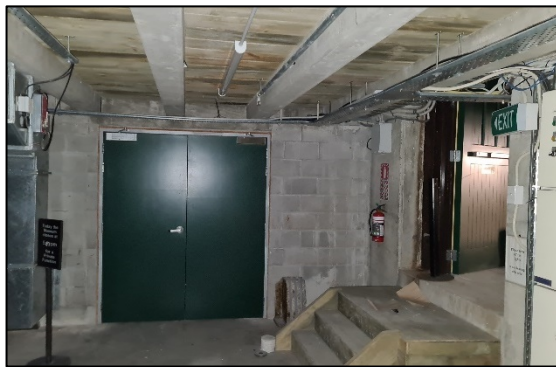


(k) Easternmost bath room, looking north



(l) View of east perimeter wall from inside bath room in (k)

Figure 3-8. Interior views of Zone 5 (North Wing)



(a) Zone 6 basement view (looking east), entryway from Zone 5 to the right



(b) View of basement through green doors in (a)



(c) View of ground floor, looking northeast



(d) View of ground floor, looking north

Figure 3-9. Interior views of Zone 6 (North Extension)

4 ON-SITE OBSERVATIONS

The following sections describe the results from various site investigations of the structure undertaken by DIZHUR Consulting. These investigations were undertaken via a combination of direct visual assessment, Surface Penetrating Radar (SPR) scanning, localised drilling and/or removal of material throughout the building to understand the construction and the various alterations undertaken over the years. Note that the sections are in order of assessment, and not from south to north or vice versa. Also note that the observations intend to provide a detailed overview of each zone in its current state, and do not intend to cover the vast extent of alterations that have taken place within the building over the course of its life to date.

4.1 Site assessment reference plan

The plan below serves as a general reference for zones with respect to the findings of the site investigation described in the following sections. The grid system and specific room identifications (where mentioned) have been adopted from the identification system established by DPA Architects (refer [Appendix 1](#) for room identification and gridline system).



Figure 4-1. 3D model of the entire complex, Zone 1 at south end and Zone 6 at north end of building

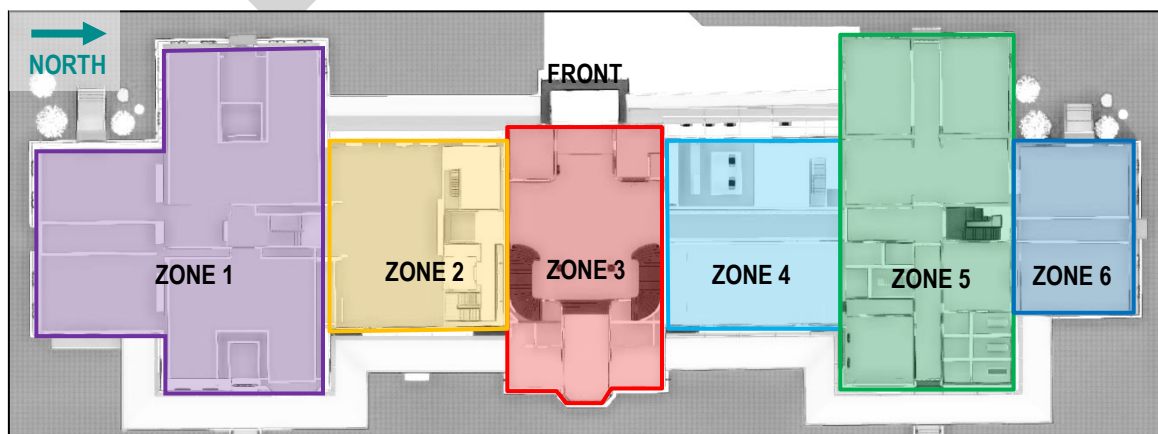


Figure 4-2: General building plan with identification and labelling of the various zones of the structure

4.2 Zone 3

4.2.1 Walls

- The east and west perimeter walls of Zone 3 (see [Figure 4-3a, b, f, p](#)) are composed of approx. 1.5m tall, 240mm thick concrete nibs at the base, and largely fenestrated timber framed walls atop the nibs with approx. 40-50mm thick pumice concrete infill panels within the framing.
 - The infill panels contain a layer of 'chicken wire' reinforcing (see [Figure 4-3e](#)), which has appeared to largely hold the infill panels together despite cracking/localised deterioration of the panels. The panels are secured within the framing via rebates in the horizontal and vertical timber members, and timber beading around the perimeter of each panel (see [Figure 4-3n, o](#)).
 - The timber framing is constructed such that the horizontal members are continuous, and the vertical members are interrupted (see [Figure 4-3m](#)). Horizontal and vertical primary members are connected via mortise-tenon joints and wooden pegs, see [Figure 4-3j, k](#). Secondary vertical and horizontal members are connected to the primary members via simple nailed connections. For Zone 3 only, the east and west walls contained diagonal members which were connected to the primary members via simple nailed (i.e., compression only) connections, see [Figure 4-3c, d, q](#). See [Figure 4-3i, n, r](#) for typical measurements and details of the timber framing members.
- The west wall (front wall) of the structure was observed to be a jetty structure, wherein the wall juts out above the main bay window, see [Figure 4-3f-h](#). Note that the jettying structure is local only to the west wall of Zone 3. Based on visual observations and existing documentation, this wall has leaned inwards at the top, resulting in the placement of diagonal steel sections just above the top of the bay window (undertaken c. 1996) and anchored from the west wall back to the north and south walls to brace the wall against further inward movement, see [Figure 4-3b, c](#).
- The north and south walls of Zone 3 (walls A5 and A4 respectively) are shared with Zones 4 and 2 respectively and are approx. 500mm thick (at ground floor level) unreinforced pumice concrete gable walls that extend to a height of approx. 10m. Above this height, the walls extend further up as timber framed walls. The timber framed walls are of the same composition as that found on the east and west elevations, with pumice concrete infill panels, see [Figure 4-3s, t, v](#).
 - At the foundation level, the concrete gable walls are approx. 550mm thick. SPR scanning and visual observations in localised areas also indicated that (as per historical documentation), the gable walls contain railway lines running horizontally at the basement as a form of reinforcement, spaced at approx. 500mm.
 - At mezzanine level, the walls reduce to approx. 430mm thickness. At a point above the mezzanine level (approx. 2-3m above), the walls step in and reduce to approx. 300-330mm thickness, see [Figure 4-3v](#).
 - No significant mechanical connection was observed between the tops of the concrete gable walls and the timber framed walls atop them, see [Figure 4-3u](#).
- Partition walls at the ground floor level are approx. 150mm thick unreinforced pumice concrete walls which extend to the soffit of the mezzanine floor. The walls comprising the rooms at the west side of the building and supporting the mezzanine floor were approx. 180mm thick, see [Figure 4-3a](#).
- Partition walls at the rear of Zone 3 at mezzanine level were found to be newer, timber framed walls, see [Figure 4-3w](#).
- At the northeast corner at ground floor (room G3N), a steel beam was found to be supporting a portion of concrete wall above, which had otherwise been removed.



(a) General view of west (front) wall of Zone 3



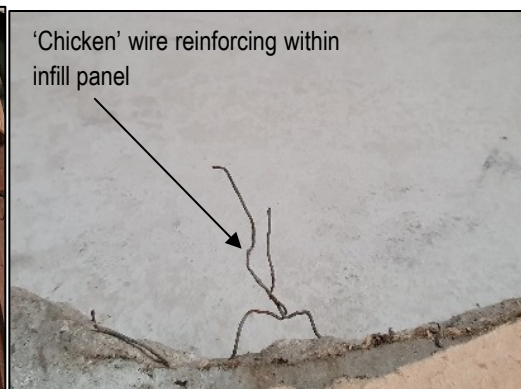
(b) View of west (front) wall at mezzanine level showing wall framing



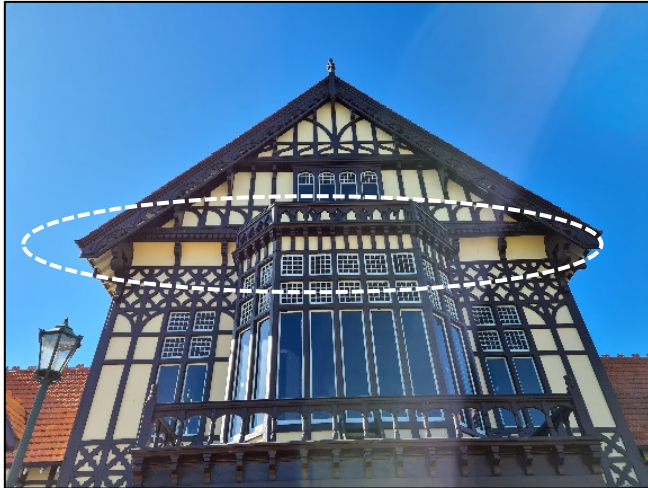
(c) Diagonal steel elements bracing west wall + curved panels indicating jetty wall structure – see also (e) and (f)



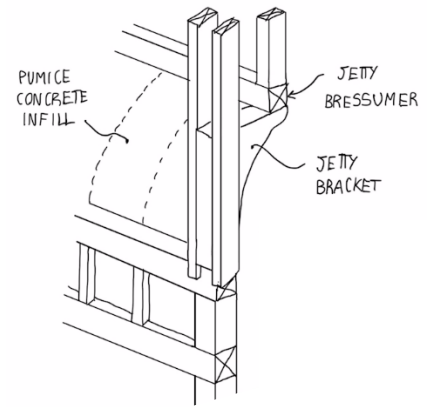
(d) Diagonal bracing elements (nailed connections only) along west wall, see also (c)



(e) 'Chicken' wire reinforcing within infill panels (note photo taken in Zone 4)



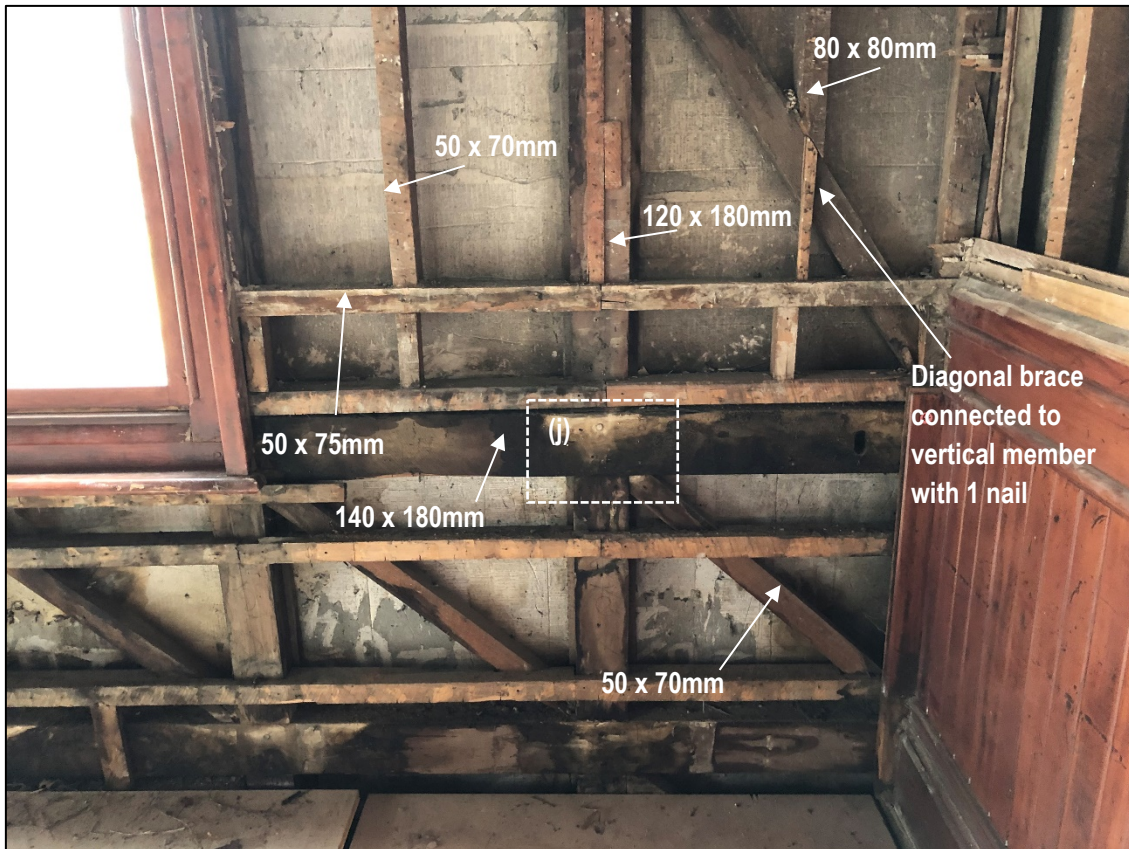
(f) Exterior view of west (front) wall to show jetty wall structure, see also (f)



(g) General schematic to illustrate jetty wall structure



(h) View from within roof space looking down west wall showing curved panels. Approx 24mm diameter metal rods with turnbuckles supporting abutting wall



(i) Closeup of west (front) wall framing with dimensions of various timber elements. Note: for indicative purposes only, dimensions of timber members vary within and between zones



(j) Closeup of mortise-tenon joint connecting primary horizontal and vertical members. Two wooden pegs also used to connect members



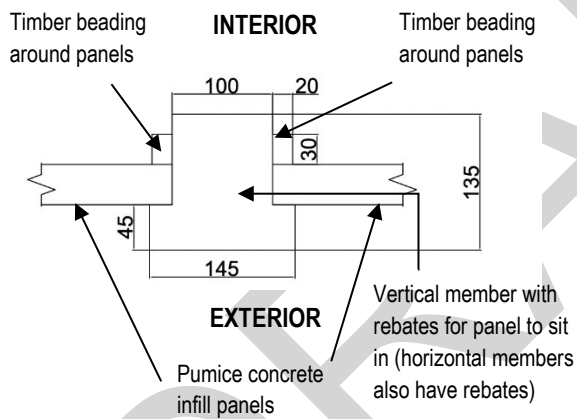
(k) Dimensions of wooden peg in mortise-tenon joint – approx. 9mm diameter, 65mm length



(l) Secondary horizontal members with slots for vertical/diagonal members to pass through



(m) Wall framing showing primary horizontal members as continuous



(n) Cross section schematic through vertical timber member. Note: Measurements done at Zone 4 wall – schematic is for understanding of arrangement only as dimensions vary within and between zones



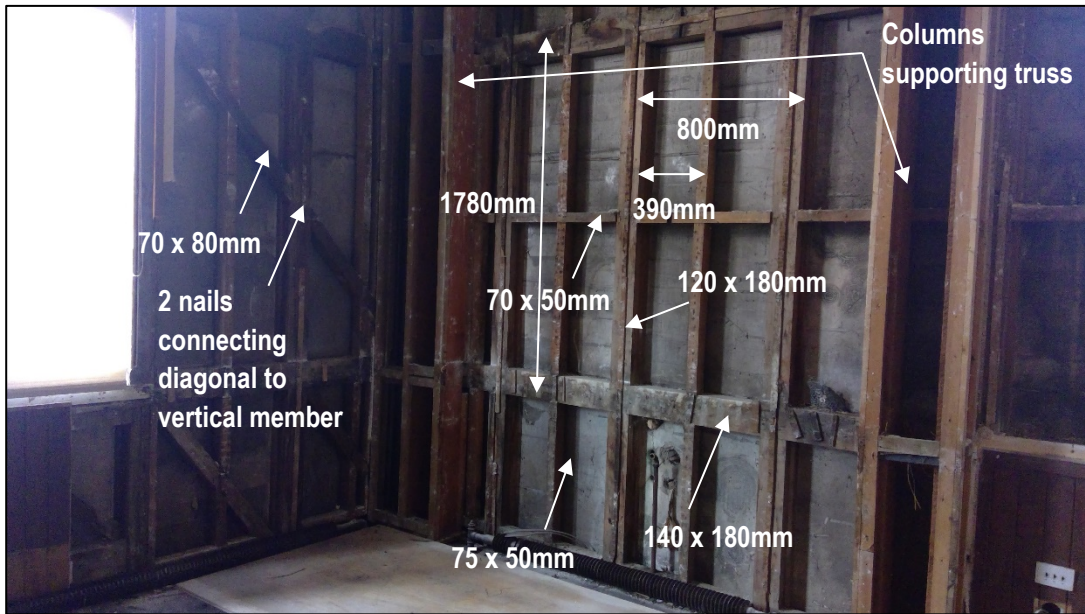
(o) Timber beading around infill panel interior



(p) View of Zone 3 east (rear) wall at ground floor



(q) East wall framing showing diagonal members (mezzanine level)



(r) Closeup of east (rear) wall framing with dimensions of various timber elements. Note: for indicative purposes only, dimensions of timber members vary within and between zones



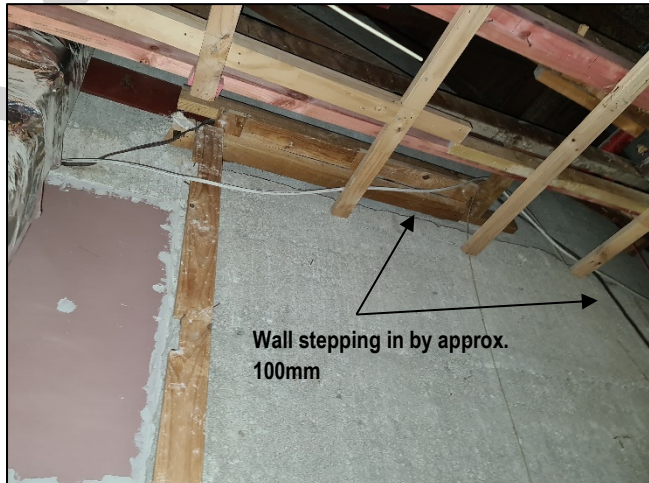
(s) View of north (A5) wall of Zone 3 showing concrete gable wall with timber frame and concrete infill panels above (south (A4) wall lined at time of visit but is the same build up)



(t) Closeup of north (A5) wall framing with observations/dimensions of various timber elements. Note: for indicative purposes only, dimensions of timber members vary within and between zones



(u) No visible mechanical connection between diagonal timber baseplate of timber framed wall and the concrete gable wall



(v) Zone 3 wall (photographed from south side of wall i.e. Zone 2), showing wall stepping in at height to approx. 330mm total width



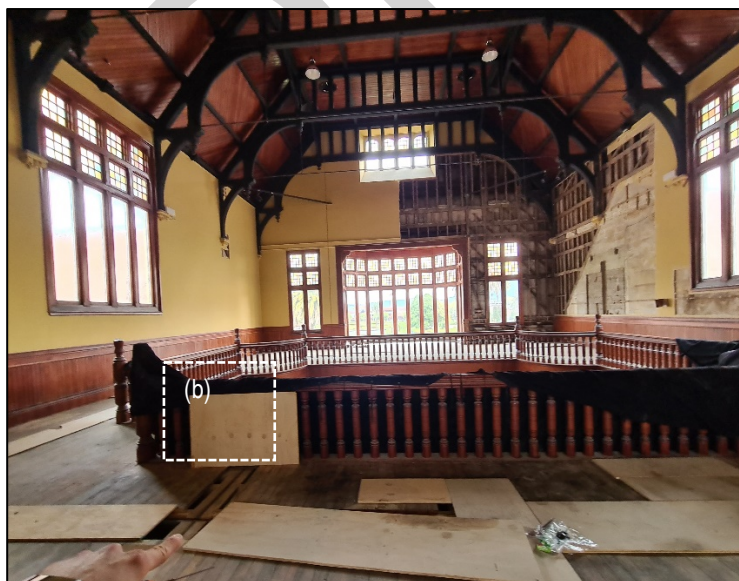
(w) Newer timber framed partition walls at rear rooms (east perimeter wall is to the left of image)

Figure 4-3 Observations for walls of Zone 3 structure

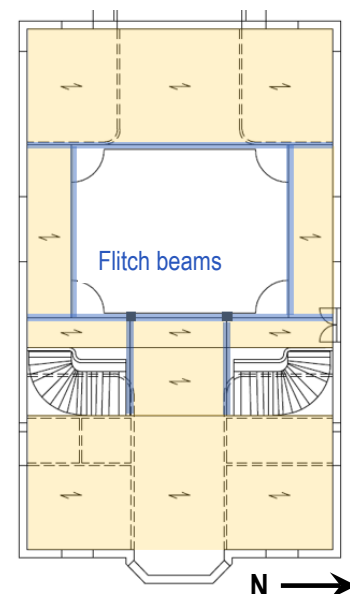
4.2.2 Floors and foundations

Mezzanine floor

- The mezzanine floor (see [Figure 4-4a](#)) is composed of approx. 35mm thick tongue-and-groove timber boards (see [Figure 4-4h](#)) supported on 45mm x 350mm timber floor joists, see [Figure 4-4e](#). The joists span north-to-south and are pocketed into the concrete gable walls – no significant mechanical connection between the joists and walls was identified, see [Figure 4-4g](#).
- Several primary beams are located around the perimeter of the central void of the mezzanine floor; these were found to be approx. 220mm wide x 320mm deep flitch beams, composed of two timber joists with two steel plates in between them, and bolts to connect the elements together, see [Figure 4-4b-d](#).
- Several newer metal rods were observed within the mezzanine floor structure, which appear to be assisting the floor in some areas from sagging, see [Figure 4-4f](#).



(a) General view of mezzanine floor (facing west) showing large void in centre, see (b) for schematic describing mezzanine floor structure



(b) Locations of flitch beams (blue lines). Dashed lines indicate walls beneath mezzanine floor. Joists run North-South



(c) Flitch beam approx. 220mm wide – 2no. 90mm wide joists, 2no. 8-10mm thick plates



(d) Flitch beams approx. 300mm deep, bolts to connect timber members and steel plates together



(e) Approx. 350mm deep timber joists



(f) Diagonal and orthogonal blocking at base of floor structure near void corners



(g) Northwest corner of mezzanine floor showing joists pocketed into concrete gable wall – no plates identified



(h) Approx. 35mm thick tongue and groove floorboards

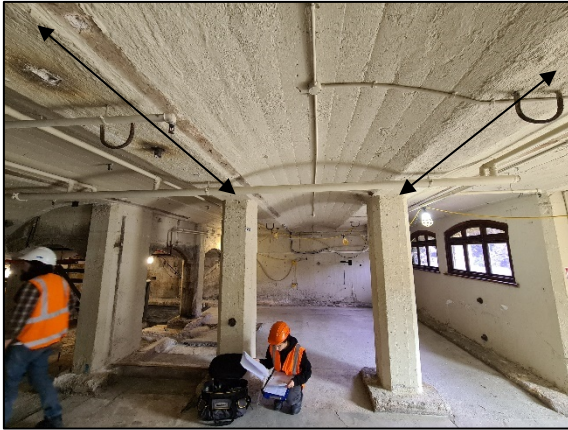
Figure 4-4. Observations regarding Zone 3 mezzanine floor structure

Ground floor slab

- The floor slab consists of the historical unreinforced pumice concrete two-way vaulted slab, which is supported on unreinforced pumice concrete columns and walls at basement level, see [Figure 4-5b](#). At its thinnest point (i.e., apex of arch), the slab measures approx. 260mm thick. At its thickest point, the slab measures approx. 430-440mm thick, see [Figure 4-5c](#). Note that these measurements are relative to Zone 3 only, floor thicknesses appeared to vary throughout the original zones.
- The raised section of slab at the east side of the building is a one-way arched slab with arches spanning east-to-west. SPR scanning indicated the presence of steel beams spanning north-to-south between columns within the eastern area where the slab is at a greater height, see [Figure 4-5a](#).
- The historical slab in the main atrium space is overlain by an approx. 30-40mm concrete overlay with wire mesh reinforcing and newer tiling – it is unclear when this was undertaken. It is believed that the historical slab was releveled prior to placement of the overlay and tiles.
- A detailed investigation was made regarding the extensive level of cracking (see for example [Figure 4-5d-g](#)) of the vaulted floor slab in Zone 3 and other original zones (Zones 2, 4 and 5). For a detailed analysis of the slab cracking and associated deformations, please refer to [§5](#).
- At the underside of the vaulted floor slab in Zone 3, localised areas were identified where previous attempts were undertaken to remediate the slab cracking via injection of a resin material to try and seal the cracks. This has not appeared to have been successful due to the cracks having opened further, see [Figure 4-5d-f](#).

Foundations

- The foundations typically consist of unreinforced pumice concrete walls at the perimeter of the Zone 3 structure and columns of the same material to support the internal region of the vaulted floor slab, see [Figure 4-5a, b, h](#).
- The footings for the north and south concrete gable walls of Zone 3 were approx. 350mm deep and 850-950mm wide, see [Figure 4-5h](#).
- It was observed that a significant number of columns have experienced deterioration and loss in cross section over the years due to the continual ingress of geothermal gases through the ground. This observation was made largely throughout the original basement area.



(a) Steel beams identified (from SPR) spanning E-W between columns in raised slab area



(b) General view of Zone 3 basement space with historic fountain foundation to the left



(c) Slab thickness measured from basement of Zone 3 – approx. 250-260mm at thinnest point, 430-440mm at junction with wall



(d) Walkway along north wall, close up of slab soffit in (d) and (e)



(e) Closeup of slab soffit showing cracking along apex of arch



(f) Evidence of past attempts to repair cracks via resin material injections



(g) Cracking of concrete at east end of basement – upward movement of ground



(h) Strip footing for north and south concrete gable walls approx 850-950mm wide

Figure 4-5 Observations regarding Zone 3 floor slab and foundation structures

4.2.3 Roof and ceiling

- The roof of the structure comprises heavy ceramic tiles on timber sarking, in turn supported on hammerbeam trusses which span from north to south, see **Figure 4-6a, b**.
- The sarking is connected to the trusses via nailed connections. The timber trusses have pinned bolt and plate connections at the member joints, see **Figure 4-6c-e**. At some point over the life of the structure, some of the trusses appear to have been strengthened with the addition of steel members, see **Figure 4-6f**.
- The ceiling structure is composed of approx. 25mm thick tongue and groove sarking which spans from the ceiling beams that in turn span between the trusses. The ceiling diaphragm currently spans between the west wall and the central spine wall (eastern wall of the main floor)
- A large turret is located at the approximate centre of the roof structure. The turret has been strengthened via the use of eight steel rods which have been tensioned at each end and fixed down into the beams which support the turret, see **Figure 4-6g, h**.



(a) General view of ceiling structure and hammerbeam trusses spanning north-south (facing west)



(b) Truss support at north wall



(c) View within roof space (facing west)



(d) Roof space at southeast side of Zone 3 (i.e., approx. above southern staircase)



(e) Gable wall and roof structure on south side



(f) Addition of steel member to strengthen truss



(g) View inside main turret showing steel tensioned rods



(h) Rods bolted into beams at base of turret structure

Figure 4-6. Observations for roof and ceiling of Zone 3 structure

4.3 Zone 2

4.3.1 Walls

- The east and west walls of the Zone 2 structure are of the same composition as the east and west walls of the Zone 3 structure, comprised of a concrete nib at the base of the walls, and timber framing with pumice concrete infill panels atop the nib, see [Figure 4-7a, c](#). For further details regarding the construction, refer to the description of the east and west walls in [§4.2.1](#).

- Due to the linings largely being removed in this zone, a black layer of material was identified between the top of the concrete nib and the bottom plate of the timber framed wall, likely an effort to create a waterproofing barrier between the concrete (which would have been more regularly exposed to moisture due to the building's original use), and the timber framing.

- The bottom plate of the timber framed wall is bolted to the concrete nib wall using approx. 12mm bolts and 30mm washers at approx. 1800mm centres. Localised deterioration of this baseplate was observed in some locations, see [Figure 4-7d](#).

- The north wall of Zone 2 is shared with Zone 3 and is an approx. 500mm thick (at GF level) unreinforced pumice concrete gable wall extending to a height of approx. 10m and topped by a timber framed wall with infill panels, see [Figure 4-7e](#).

- The south elevation of Zone 2 contains two large openings to facilitate access between Zones 1 and 2. The central portion of the elevation between the openings comprises a steel cross braced frame. The east and west sides of the elevation contain remnants of an unreinforced pumice concrete wall (considered part of Zone 1) and steel cross braced framing (added 2010-11), see [Figure 4-7f](#).

- At the ground floor level, it was observed that there is a seismic separation of at least 50mm between Zones 1 and 2.

- In addition to the steel framing described above, Zone 2 contains steelwork along the east, west and north elevations which was added to strengthen Zone 2 as part of development works undertaken in 2010-11 to complete the Zone 1 structure, see [Figure 4-7a-c, i](#).

- The framing comprises I-section steel columns and SHS beams with typically Ø20mm or Ø25mm Reidbrace cross bracing connected to the columns with Reidbrace end connections. The framing is connected to the walls only at the roof eaves level.

- The end connections are typically connected to the flanges of the columns, resulting in an eccentric connection, see [Figure 4-7g](#). In some locations, it was observed that the brace on one side of the column was connected to the outer flange, whereas the brace on the other side of the column was connected to the inner flange, resulting in a torsional action on the column should both braces be stressed, see [Figure 4-7i](#).
- The Reidbrace end connections were observed to be poorly installed, with significant gaps between the plates of the end connection and the column flange, see [Figure 4-7h](#). The gaps were deemed large enough to cause the connecting pin to undergo partial bending under seismic action, instead of solely shear.
- The end connections were considered to have insufficient edge distance from the pin to the end of the column flange in multiple locations, see [Figure 4-7h, j](#).



(a) View of west wall of Zone 2 showing concrete nib and timber framed wall above with infill panels as per Zone 3, and steel framing in front



(b) Steel framing connected to original west wall at roof eaves level



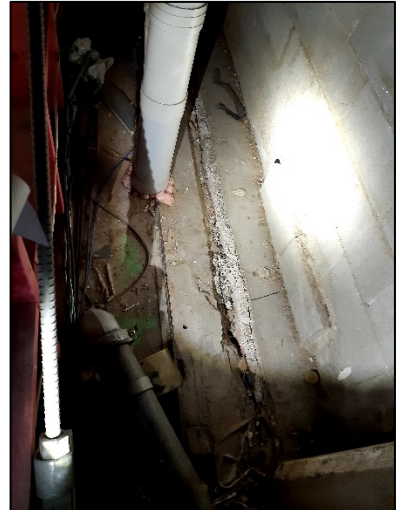
(c) View of east wall showing same build up – original timber framed wall with infill panels and newer steel framing in front



(d) Localised deterioration of timber baseplate of historic timber framed wall



(e) East side of north wall of Zone 2 (shared with Zone 3)



(f) South side of Zone 2 showing concrete block wall of Zone 1, steel framing of Zone 2 and gap in flooring with polystyrene in between



(g) Reidbrace end connections eccentrically connected to I-section column flanges



(h) Gap between flange and end connection causing pin to bend under load. Approx. 20mm edge distance of connection



(i) Reidbrace end connections on opposite flanges, creating effect of torsion of column under load



(j) Reidbrace end connection to corner column



(k) Exposed railway line at south basement wall of Zone 2. Also identified via SPR in north and south basement walls of Zone 3



(l) North wall of Zone 2 showing steel framing continuing down to basement

Figure 4-7 Observations regarding Zone 2 wall structures

4.3.2 Floors and foundations

Ground floor slab

- The ground floor slab of the Zone 2 structure is the same as that of the main floor area of Zone 3 i.e., two-way arched unreinforced pumice concrete slab.
- The slab contains an approx. 90-100mm concrete overlay with wire mesh reinforcing, which is believed to have been added c. 1967, see [Figure 4-8a](#).
- A detailed investigation was made regarding the extensive level of cracking of the vaulted floor slab in Zone 2 and other original zones (Zones 3, 4 and 5). For a detailed analysis of the slab cracking and associated deformations, please refer to [§5](#).

Foundations

- The foundations resemble that of the foundations encountered for the Zone 3 structure i.e., unreinforced pumice concrete perimeter foundation walls and columns to support the vaulted floor slab, see [Figure 4-8b, c](#).
- Measurements of the historical column foundations in this zone indicated approx. 780mm square by 360mm deep unreinforced pads, see [Figure 4-8d, e](#). Remnants of a black, slate-like layer was identified at the base of the pad footing, which may represent efforts during the original construction to provide a waterproofing layer for the foundations, see [Figure 4-8f](#).
- Newer pad footings have been added for the 2010-11 steel frame columns, see [Figure 4-8b](#) – full drawings for these are available (see DOC568v1 – Structural Alterations to Existing Tarawera Gallery as built drawings, Sigma Consultants, 2010).



(a) Approx. 90-100mm thick topping with 665 mesh atop vaulted floor slab (topping added c. 1967)



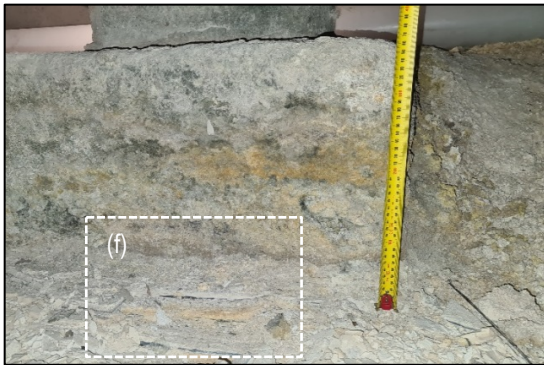
(b) View of Zone 2 basement adjacent to north wall with steel frame columns with newer footings in front of historic foundation wall



(c) Typical historic column and footing with evidence of slab cracking just above



(d) Approx. 780mm square unreinforced pumice concrete pad footing



(e) Approx. 380mm deep footing



(f) Presence of flat, black layer beneath footing as possible waterproofing layer for pad foundation

Figure 4-8 Observations regarding Zone 2 floor and foundation structures

4.3.3 Roof and ceiling

- The roof of the structure comprises heavy ceramic tiles on timber sarking, in turn supported on a twin gable roof structure with trusses spanning east to west, see [Figure 4-9c, d, f](#).
- The trusses are seated on the east and west perimeter walls and are supported by horizontal steel trusses running North-South on either side of the central walkway, see [Figure 4-9a, b](#). These trusses in turn are supported on steel frames at the north and south elevations of the structure.
- The ceiling of Zone 2 is composed of approx. 45mm x 20mm battens supporting plywood and fibrous plasterboard overlays, much of which has been removed to leave just the framing, see [Figure 4-9e](#).



(a) View of Zone 2 ceiling showing lined steel trusses and horizontal steel truss running N-S



(b) View through lining of steel truss composed of UC sections and diagonal steel angles



(c) North side of Zone 2 showing gable wall, roof trusses and sarking boards



(d) Cross braces of roof trusses



(e) Ceiling linings largely removed leaving only framing



(f) Roof trusses and ceiling supported on historic east and west walls

Figure 4-9 Observations regarding Zone 2 roof and ceiling structures

4.4 Zone 4

4.4.1 Walls

- The east and west walls of the Zone 4 structure are of the same composition as the east and west walls of the Zone 3 structure, comprised of a concrete nib at the base of the walls, and timber framing with pumice concrete infill panels atop the nib, see [Figure 4-10a](#). For further details regarding the construction, refer to the description of the east and west walls in [§4.2.1](#).

- Due to the linings largely being removed in this zone, a black layer of material was identified between the top of the concrete nib and the bottom plate of the timber framed wall, likely an effort to create a damp proof course between the concrete (which would have been more regularly exposed to moisture as a result of the building's original use), and the timber framing, see [Figure 4-10d](#).

- The bottom plate of the timber framed wall is bolted to the concrete nib wall using approx. 12mm bolts and 30mm washers at approx. 1800mm centres, see [Figure 4-10b, c](#).

- The south wall of Zone 4 is shared with Zone 3 and is the approx. 500mm thick (at GF level) unreinforced pumice concrete gable wall that extends to a height of approx. 10m and topped by a timber framed wall with infill panels, see [Figure 4-10g](#).

- The north wall of Zone 4 is shared with Zone 5 and is an approx. 300mm thick (at GF level) unreinforced pumice concrete gable wall extending to a height of approx. 10m, see [Figure 4-10e](#). There is no timber framed section of wall atop this wall.

- Steel frames are located on the east and west sides of the central walkway that runs North-South. These frames consist of approx. 200mm deep I-section beams and columns, see [Figure 4-10f](#). The columns bear directly onto columns at the basement level.



(a) General view of east (RHS) and west (LHS) walls showing concrete nib topped with timber framed wall and infill panels



(b) Baseplate of timber framed wall bolted to concrete nib with approx. Ø12.4mm bolt and 30mm washer



(c) Bolts connecting baseplate to concrete nib spaced at approx. 1800mm centres

(d) Evidence of black layer between baseplate and concrete nib as a possible waterproofing layer



(e) View of north (A6) wall (west side of Zone 4) – shared with Zone 5. Wall is approx. 300mm thick at GF

(f) Steel frame on west side of corridor (east side also) supporting roof loads and timber framed mezzanine floor wall



(g) South wall of Zone 4 (A5 wall) – shared with Zone 3

Figure 4-10. Observations regarding Zone 4 wall structures

4.4.2 Floors and foundations

Mezzanine floor

- Zone 4 contains a timber mezzanine floor along the eastern side of the structure which spans the full length between the north and south walls, see [Figure 4-11a](#).
- The floor is supported on timber stringers at the north and south walls, and on steel I-section beams inter-span which span to the east wall and to the steel frame on the eastern side of the central walkway, see [Figure 4-11b, c](#).
- It was noted that the I-section beams were provided with additional support via another I-section beam placed directly underneath and columns at the east wall, see [Figure 4-11c](#). The timber joists also appeared to have steel strips added to their underside, see [Figure 4-11b, c](#).

Ground floor slab

- The ground floor slab of the Zone 4 structure is the same as that of the main floor area of Zone 3 i.e., two-way arched unreinforced pumice concrete slab.
 - The slab in Zone 4 contains a large void at the southwest corner of the zone where there is a historic bath structure, known as the Rachel bath.
- A detailed investigation was made regarding the extensive level of cracking of the vaulted floor slab in Zone 4 and other original zones (Zones 2, 3 and 5). For a detailed analysis of the slab cracking and associated deformations, please refer to [§5](#).

Foundations

- The foundations typically consist of unreinforced pumice concrete walls at the perimeter of the Zone 4 structure and columns of the same material to support the internal region of the vaulted floor slab, see [Figure 4-11d](#). Newer columns and foundations were added to support the steel frames added along the central corridor, see [Figure 4-11e](#).

- It was observed that a significant number of columns have experienced deterioration and loss in cross section over the years (see [Figure 4-11g](#)) due to the continual ingress of geothermal gases through the ground. This observation was made largely throughout the original basement area.

- An active theatre was established along the eastern side of the zone in the 1990s, which resulted in new columns and foundations being added at the basement level to support the theatre.



(a) Underside of timber mezzanine floor structure – joists spanning North-South



(b) Joists supported on timber stringer at north concrete gable wall



(c) Joists supported within span by steel beams. Note beams have been provided with additional support via additional I section beams and column at the east wall



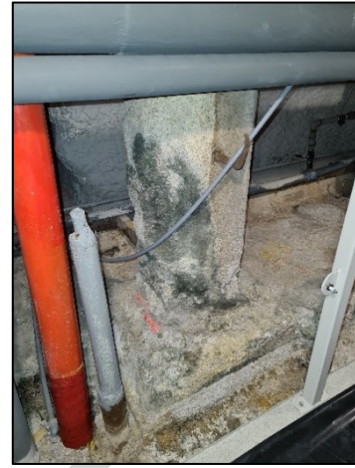
(d) View of Zone 4 basement level



(e) Corridor at basement level showing new columns/foundations added to support steel frame above



(f) Approx. 700-750mm wide footing for 350mm wide north (A6) wall of Zone 4



(g) Example of concrete deterioration observed throughout historic basement structure

Figure 4-11. Observations regarding Zone 4 floor and foundation structures

4.4.3 Roof

- The roof of the structure comprises heavy ceramic tiles on timber sarking, in turn supported on a twin gable roof structure with trusses spanning east to west, similar to the Zone 2 roof structure, see [Figure 4-12](#). The trusses are seated on the east and west perimeter walls and are supported by the steel frames along both sides of the corridor at the interior.
- The roof end rafters are connected to the north and south concrete gable walls with bolts.



(a) Zone 4 truss structure (similar to Zone 2)



(b) View upward showing timber sarking supporting on truss rafters

Figure 4-12 Observations regarding Zone 4 roof structure

4.5 Zone 5

4.5.1 Walls

- The east and west wings of Zone 5 contain perimeter timber framed walls that are of the same construction as the Zone 3 east and west walls i.e., timber framing connected via mortise-tenon joints and 50mm thick pumice concrete infill panels, sitting on top of approx. 240mm thick concrete nib walls, see [Figure 4-13a, b](#).
- The north and south walls of Zone 5 are approx. 10m tall unreinforced pumice concrete gable walls. The south (A6) wall is integral with Zone 4, and the north wall is independent of Zone 6, with evidence of a seismic gap between the two

zones. Both walls are thinner than the A4 and A5 walls (north and south walls of Zone 3), measuring approx. 350mm at ground floor (see [Figure 4-13g](#)).

- Internal full height walls are composed of a concrete nib wall topped with timber framing and lined with lath and plaster, see [Figure 4-13c](#).
- Several partition walls on the east side of the gallery are not to full height and consist of an approx. 1.5m tall concrete nib with a short timber framed section above, see [Figure 4-13i](#).
- The main corridor running North-South and the corridor on the eastern side running East-West both contain approx. 1.2m wide doorways which have later been filled, see [Figure 4-13c, d, h](#).
- On the west side of the gallery, load bearing walls which likely will have formed a corridor (i.e., mirroring the east side), have been removed and replaced with steel trusses, see [Figure 4-13a](#).
- The north wall of Zone 5 was observed to have a separation with the south wall of Zone 6, see [Figure 4-13j](#).



(a) West wall of Zone 5 – concrete nib wall topped with timber framed wall and infill panels, lath and plaster internal lining



(b) Closeup of west wall showing tiles overlaying concrete nib and lath and plaster overlaying timber framed portion of wall



(c) West wall of main corridor running N-S (photo taken on west side), showing 3 door openings filled with timber framing



(d) Door openings approx. 1200mm wide



(e) Approx. 150mm thick timber wall studs, approx. 18-20mm thick lath and plaster linings. Baseplate approx. 80mm thick



(f) View of north concrete gable wall from bath room (room G5N)



(g) South wall of Zone 5 measured approx. 320-330mm thick (from roof space)



(h) View of north wall of corridor in east side of Zone 5 running E-W and showing filled door openings



(i) Typical partial height partition walls in east side of Zone 5



(j) Polystyrene identified between Zone 5 and Zone 6 walls

Figure 4-13 Observations regarding walls of Zone 5 structure

4.5.2 Floors and foundation

Ground floor slab

- The ground floor slab of the Zone 5 structure is the same as that of the main floor area of Zone 3 i.e., two-way arched unreinforced pumice concrete slab, see [Figure 4-14a, b](#).
- The slab in Zone 5 contains three voids at the eastern end against the north wall, belonging to three historic bath rooms, see [Figure 4-14c](#).
- In several locations (including other zone basements), the presence of a fibrous, cementitious material (resembling gannite or sprayed concrete) was observed on the vaulted floor slab soffit, see [Figure 4-14a, b](#). The layer is approx. 10-20mm thick and is believed to have been added during the 1990s, possibly to mitigate further propagation of the slab cracking (though it has not succeeded).
- A detailed investigation was made regarding the extensive level of cracking of the vaulted floor slab in Zone 5 and other original zones (Zones 2, 3 and 4). For a detailed analysis of the slab cracking and associated deformations, please refer to [§5](#).

Foundations

- The foundations typically consist of unreinforced pumice concrete walls at the perimeter of the Zone 4 structure and columns of the same material to support the internal region of the vaulted floor slab.
- Localised areas of the Zone 5 basement contain various braces and/or steel elements to strengthen the historic foundations, such as timber supports and circular hollow section diagonal members. It is unclear how effective the pipe sections are due to their end connections, see [Figure 4-14d-f](#).
- It was observed that a significant number of columns have experienced deterioration and loss in cross section over the years (see [Figure 4-14g](#)) due to the continual ingress of geothermal gases through the ground. This observation was made largely throughout the original basement area.



(a) View of Zone 5 basement area (looking west), sprayed cementitious material visible on soffit



(b) Zone 5 basement adjacent to south concrete gable wall, sprayed cementitious material visible on soffit



(c) Void in vaulted floor slab at east side of Zone 5 for historic baths (two more adjacent to this one)



(d) Timber support elements encountered in west side of Zone 5 basement



(e) Circular hollow section 'pipe' elements encountered in west side of Zone 5 basement



(f) End connections of diagonal pipe sections



(g) Example of column cross section deterioration, likely due to ingress of geothermal gases from ground

Figure 4-14 Observations regarding floor and foundation structures of Zone 5

4.5.3 Roof and ceiling

- The roof of the Zone 5 structure comprises heavy ceramic tiling supported on 20-30mm thick timber sarking which is in turn supported on timber rafters and trusses.

- The trusses in the east and west wings (see [Figure 4-15b](#)) of Zone 5 span onto the perimeter timber framed walls at north and south ends, and internally on timber framed walls on the east side, and onto beams in the west side of the zone.
- The central section of the Zone 5 roof contains queen post trusses. The ends of the truss chords are pocketed into the concrete gable walls at north and south ends and the trusses are supported inter-span by internal timber walls at ground floor. Bolted plate connections were observed for the bottom chords of the trusses and strengthening of the rafter connection to the bottom chord was observed for some trusses (see [Figure 4-15c, d](#)).
- It was observed that the sarking and rafters have experienced some decay due to moisture ingress and some nailed connections have corroded, see [Figure 4-15a](#). This observation is supported by previous timber assessments within the roof structure (see Prendos timber deterioration assessment report dated 20 June 2017, ref: 023908003).



(a) Truss structure at south gable wall of Zone 5, showing some moisture ingress and deterioration of timber members



(b) Truss structure for southeast wing of Zone 5 (northeast and west wing structures similar)



(c) Bolted plate connections on truss chord pocketed into gable wall



(d) Steel plates to strengthen rafter to bottom chord truss connection

Figure 4-15 Observations regarding Zone 5 roof structure

4.6 Zone 1

- Zone 1 (see [Figure 4-16a](#)) is predominantly newer than the zones described in the previous sections. Construction of the zone first began in 1911, followed by additional works in 1983, before final completion in 2010-11.
- Detailed as built drawings were available for this zone; these were used in conjunction with on-site observations to investigate and confirm the zone's configuration.
- The roof of the zone comprises heavy ceramic tiling on timber purlins, which are in turn supported on structural steel rafters.

- The upper structure of the building is principally steel framed. The ground floor consists of an approx. 110mm thick flat slab along the eastern perimeter of the building, and timber flooring supported on 215mm thick Stahlton ribs, see [Figure 4-16f](#).
- The ground floor is supported on reinforced concrete frames. The zone contains a central 'core' of reinforced concrete masonry walls and reinforced concrete frames that form the central stairwell and lift shaft, see [Figure 4-16b](#).
- The north portion of the western perimeter wall of the zone was found to be a full height unreinforced concrete wall up to the roof eaves level – this wall was likely part of the 1911 construction period, see [Figure 4-16c](#). The south portion of this wall was found to be composed of a reinforced concrete masonry nib wall topped with a timber framed wall above to resemble the historic timber framed walls of the other zones, see [Figure 4-16e](#). The gable portion of the wall (i.e., above roof eaves level) is composed of timber framing and steel trusses, see [Figure 4-16d](#).
- The ground floor was found to contain voids along the southernmost side of the extension and along the west side of the larger portion of the zone, to allow ventilation ducts to pass through (as per drawings), see [Figure 4-16g, h](#).
- The foundations of the zone comprise reinforced concrete isolated footings and shallow strip footings.
- All elevations of the zone contain the Reidbrace cross bracing system. The framing comprises I-section steel columns and SHS beams with typically Ø20mm or Ø25mm Reidbrace cross bracing connected to the columns with Reidbrace end connections.
 - Several columns were observed to have insufficient baseplate connections, see [Figure 4-16j](#). Some of the Reidbrace end connections were also observed to be connected to the columns via a cantilevered plate, which by inspection appeared to be inadequately detailed, see [Figure 4-16i](#).
- Zone 1 was observed to have a seismic separation from Zone 2 at the mid-section and at the east side of the interface between the two zones. It could not be confirmed whether the seismic separation exists at the west side of the interface between the zones.



(a) View of Zone 1 structure with entryways to larger section (view facing northeast)



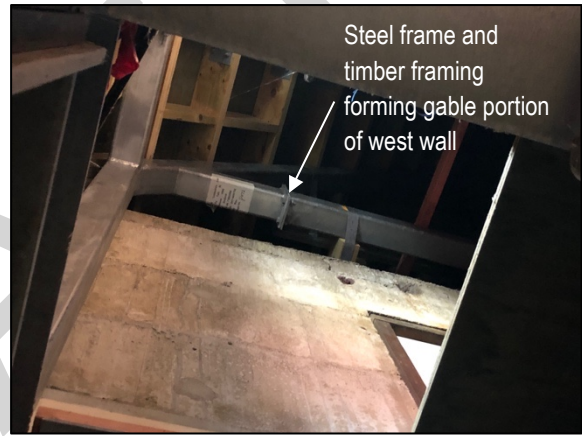
(b) View from larger section of Zone 1, looking southwest, entry to stairwell to the right



(c) Reinforced concrete masonry walls and RC frames comprising central lift and stair core



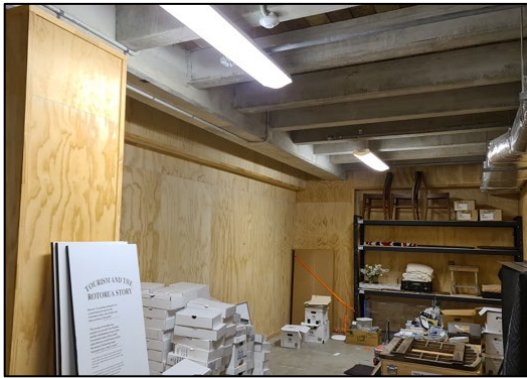
(d) North portion of west perimeter wall – full height concrete wall connected to steel framing



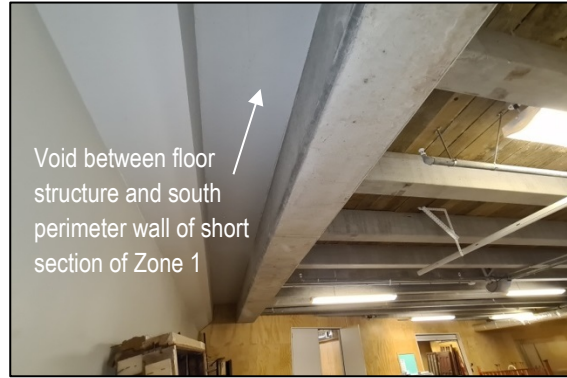
(e) Concrete wall reaches roof eaves level, gable wall composed of steel frame and timber framing



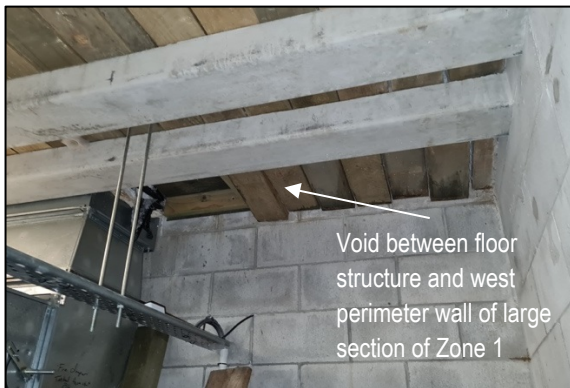
(f) Southern portion of west perimeter wall, reinforced concrete block nib wall topped with timber framing



(g) Floor structure comprised of timber infill and precast Stahlton ribs spanning N-S



(h) Void in floor structure along south elevation of smaller section of Zone 1



(i) Voids in floor structure along west side of larger section of Zone 1



(j) Reidbrace end connected to cantilevered welded plate on column



(k) Column baseplate with insufficient number of bolts

Figure 4-16 Observations regarding Zone 1 structure

4.7 Zone 6

- Zone 6 of the museum was designed and constructed c. 2007-8 as an extension to the main building. Detailed as built drawings were available for this zone; these were used in conjunction with on-site observations to investigate and confirm the zone's configuration.

- The roof structure of the zone comprises heavy ceramic tile roofing supported on timber purlins which are in turn supported on Reidbrace steel framing and cross bracing, see [Figure 4-17a](#).
- The zone contains a ventilation structure which resembles the various turrets located across the roof structure of the building. The structure is laterally supported by the roof ridge beam, and by the Reidbrace system at lower height, see [Figure 4-17c](#).
- The walls of the zone consist of steel portal frames composed of square hollow sections and plywood layers, creating a 'composite' wall structure, see [Figure 4-17d](#). From assessment of the drawings, the frames are considered to typically have pinned connections.
- The floor of the zone comprises timber infills and precast Stahlton ribs (see [Figure 4-17g, h](#)), with an approx. 75mm concrete topping reinforced with 665 mesh. The floor was found to contain voids along the north elevation of the zone to allow ventilation ducts to pass through.
- At basement level, the perimeter walls are reinforced concrete block walls, see [Figure 4-17h, i](#). A single reinforced concrete block wall also runs North-South along the central axis of the zone, see [Figure 4-17h](#).
- The foundations for the perimeter walls are approx. 2.5m wide by 400mm deep reinforced concrete strip footings.
- The interface between Zones 5 and 6 was observed to have a gap filled with polystyrene, indicating a seismic separation between the two zones, see [Figure 4-17j](#).



(a) Roof structure composed of timber purlins on rafters supported on steel frames and Reidbrace cross bracing.



(b) Rafters connected to steel frames via bolted plate connections



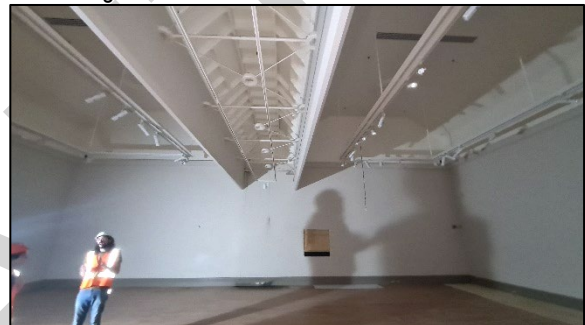
(c) View of cross bracing system to support central turret



(d) Wall structure composed of SHS sections comprising steel frames and plywood layers. Approx. 60-70mm gap to Zone 5 north concrete gable wall



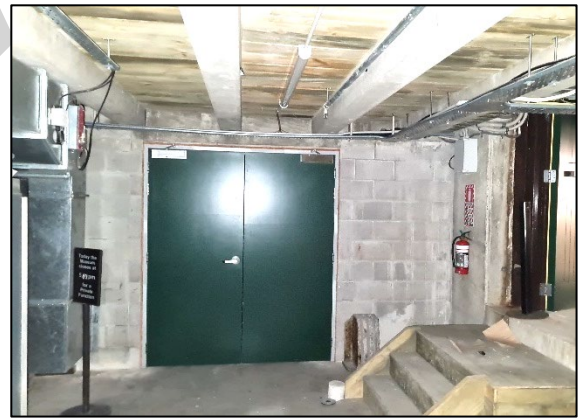
(e) View of Zone 6 structure, looking northeast



(f) Looking north, showing roof support structure and horizontal truss



(g) Timber infill and Stahlton rib floor structure



(h) Basement level (looking east), central concrete block wall running N-S. Entry from Zone 5 to the right



(i) Basement entry to Zone 6, showing polystyrene between Zone 5 and 6 interface

Figure 4-17 Observations regarding Zone 6 structure

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5 CRACK INVESTIGATIONS & MONITORING

5.1 Previous crack investigations

Several efforts to record the occurrence of cracking have been undertaken in previous years. Kerslake and Partners in 1985 reported an assessment (DOC547-v1, DOC548-v1 and DOC549-v1) of the state of the basement ceiling floor, columns and walls. Significant cracking that had propagated through the full depth of the slab was identified, with crack widths varying from 1-3 mm. It was concluded that due to the pattern of the cracking, the cause was excessive shrinkage and that a difference in elevation between two adjacent cracked slabs indicated differential settlements. The broader conclusion was that the source of cracking could not be accurately determined and that due to the structure's resilience to date (being 80 years old) and the absence of any major foundation settlement or collapse, another crack inspection in 5 years was the only required action. Additional crack investigations were attempted in later years, however these generally consisted of undertaking crack mapping without any comprehensive investigation into possible causes.

5.2 Point Cloud

Laser scanning to document the structure was undertaken in 2016 and 2020: these scans were utilised to investigate whether any structural deformations had occurred in those four years, such as vertical or horizontal translation, or rotation of vertical elements due to settlement. The structural elements which were checked for deformation were the floor, basement columns and the north and south concrete gable walls of Zone 3, as well as the vaulted floor for Zones 2, 4 and 5.

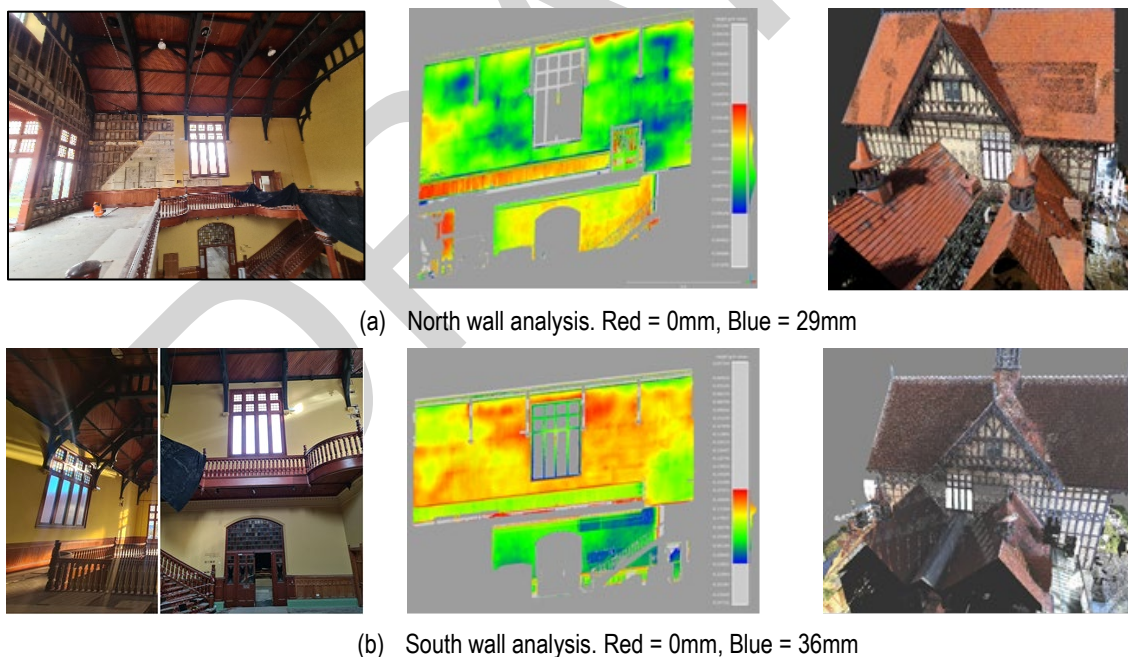
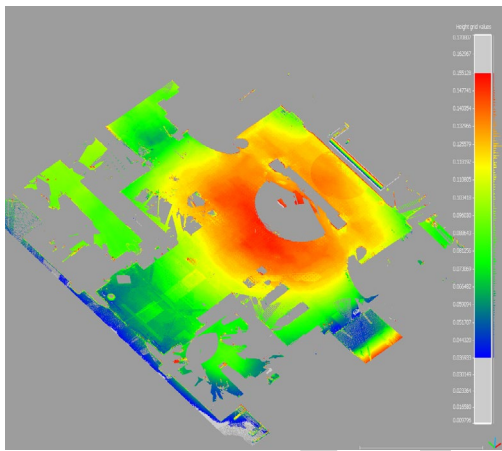
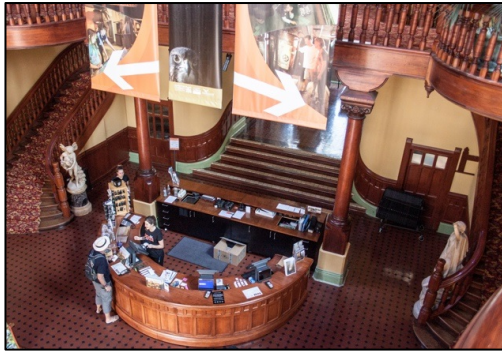


Figure 5-1. Analysis of Zone 3 north and south concrete gable walls (ASBUILT 2016 point cloud)

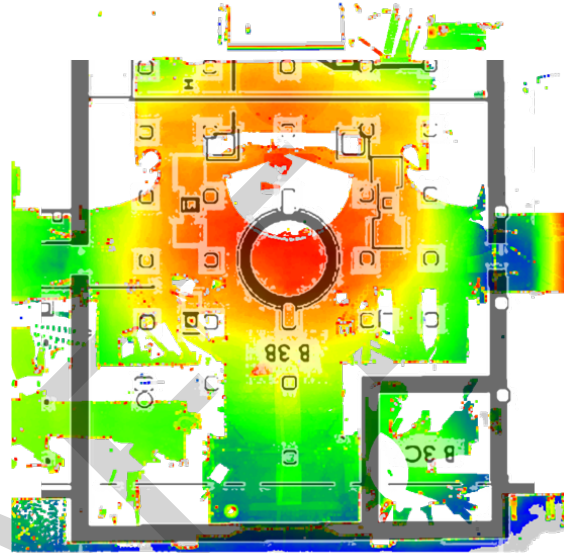
No evident out-of-plane deformations were observed in the walls. In the investigated deformations, the deformations caused by the thrust of the gable over the wall and the thrust due to the adjacent Zone 2 and 4 pitched roofs can be observed.

Figure 5-3 to Figure 5-5 report the displacements of the ground level for Zones 2 to 5 relative to the middle of Zone 3 at the location of the fountain foundation, which was observed to be the highest point and was therefore selected as the origin. This study was undertaken to understand the level of relative displacement of various regions of the floor structure in an attempt to validate the observed crack patterns and hypothesised failure mechanisms. Note that the reported

numbers may not necessarily reflect true relative displacement values, as they do not account for the possibility of variation in ground level/foundation depths at the time of construction. Note also that the data utilised for this study was not collected for the purposes of ground settlement estimates, and may therefore not have been collected in a method so as to provide a single datum from which relative displacements can be estimated to a high level of accuracy.



Deformations (Blue = -118mm, Red = 0mm)



Deformations (Blue = -118mm, Red = 0mm)

Figure 5-2. Deformation measurements undertaken for the vaulted floor at ground floor level in Zone 3

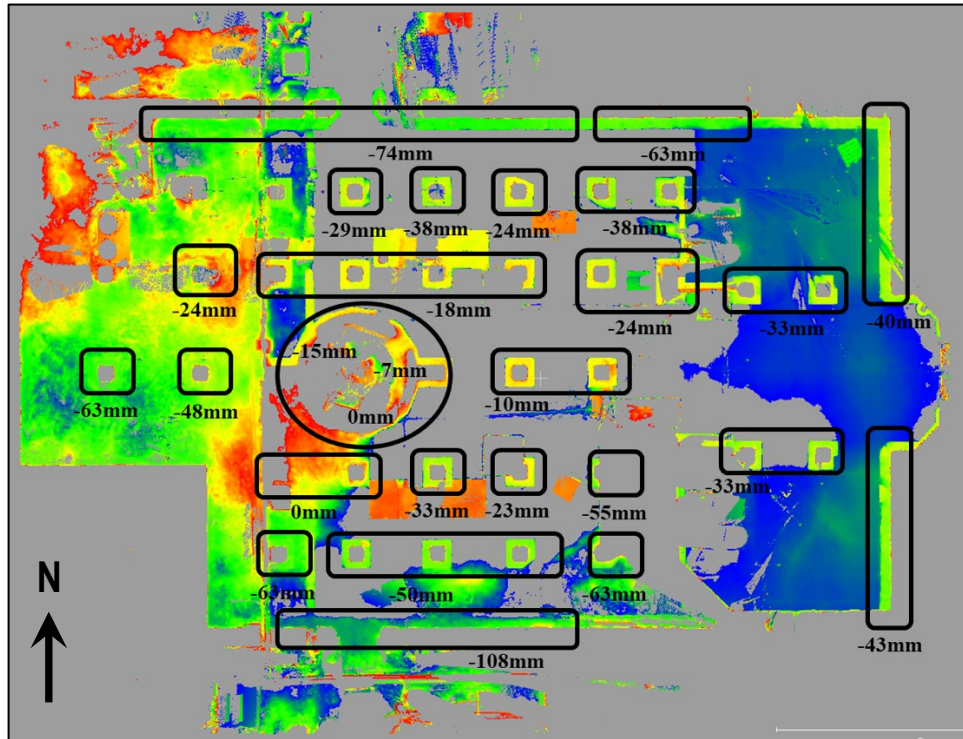


Figure 5-3 Relative displacements for vaulted floor of Zone 3 (taken from the top surface of column and wall footings)

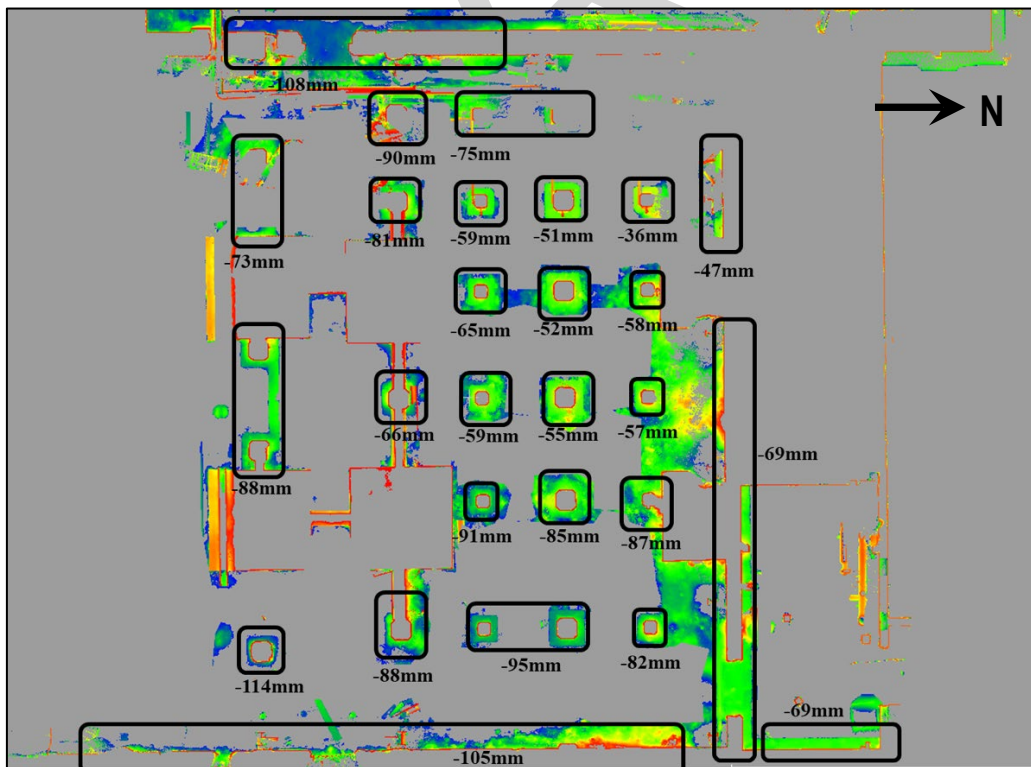


Figure 5-4 Relative displacements for basement level of Zone 2 (taken from the top surface of column and wall footings)

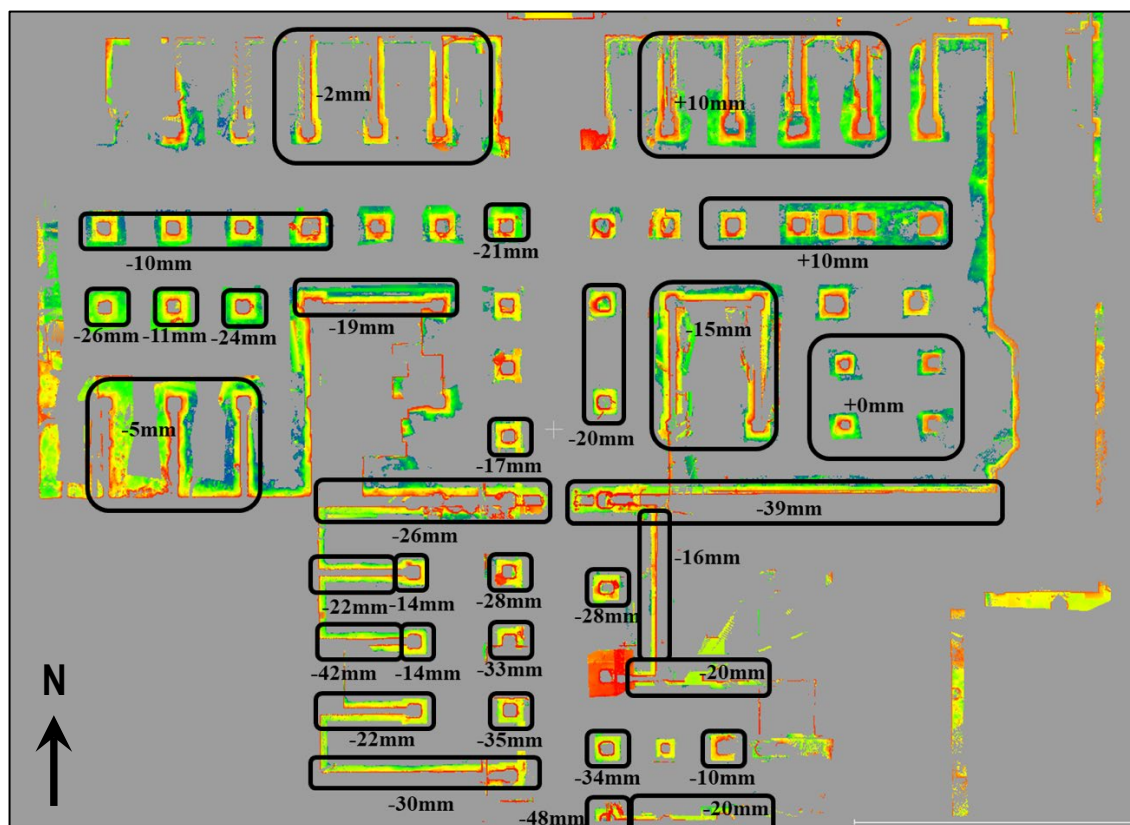


Figure 5-5 Relative displacements for vaulted floor of Zones 4 and 5 (taken from the top surface of column and wall footings)

As can be seen in **Figure 3-1c**, it appears that the basement columns were constructed prior to the vaulted floor, which likely resulted in cold joints between the columns and the floor. Although the large, unreinforced concrete gable walls will have had some structural stability provided from the vaulted geometry of the floor structure, it is unlikely that they had any significant connections to the floor. Based on the assumption that the original foundations were constructed with a reasonable level of consistency in elevation, and by observing the consistency in displacements measured at the ground floor level and at the top of the footings, it is seen that the north and south concrete gable walls (A4 and A5 walls) of Zone 3 have experienced significant settlements relative to the foundation of the historic fountain, which appears minimal. Given that the only 'pathway' between these two points is the vaulted floor structure, it is hypothesised that the stress concentrations from the high level of movement between the two points have been released in the form of damage to the vaulted floor. In addition, the change in cross sectional area from the vaulted floor to the basement columns results in high stress concentrations at this interface, which when coupled with the presence of cold joints between the columns and the floor (due to possibly being constructed at different times), may explain the resulting cracks that are observed at the tops of the columns where they meet with the vaulted floor.

Explanations and diagrams of the hypothesised cracking mechanisms are provided in terms of four crack groups (CG) in **Figure 5-6**. A sketch to describe the hypothesised cracking mechanisms in relation to the observed vertical displacements and cracking of the vaulted floor and walls is shown in **Figure 5-7**, with respect to cracking observed at the north side of the Zone 3 basement level.

CG1: Differential settlement cracks (see **Figure 5-6a**).

CG2: The longitudinal cracks observed in the vault running parallel to the gable wall may be explained by **Figure 5-6c** (Acikgoz et al. (2017)). The thrust line and hinges shown which create cracking comparable to the cracks observed on site are seen in schematic (a) of **Figure 5-6c**, where a third hinge (in addition to those at the interface with the columns or wall supports) will have been formed at the extrados (apex) of the arch, caused by one of the supports settling. The cold joint likely to exist between the columns and the vaulted floor

also creates a weak plane where cracks are likely to form, and these also coincide with the location of the base hinges in the mechanisms shown in **Figure 5-6c**.

CG3: Cracks on the gable walls which are observed at ground floor but do not propagate down to the basement level. This may be due to contribution from the railway lines at basement level assisting with tensile stresses at the thinnest part of the vertical crack to prevent propagation.

CG4: Differential settlements between columns and possible horizontal displacement may result in cracking mechanisms described by schematics (c) and (d) of **Figure 5-6c**.

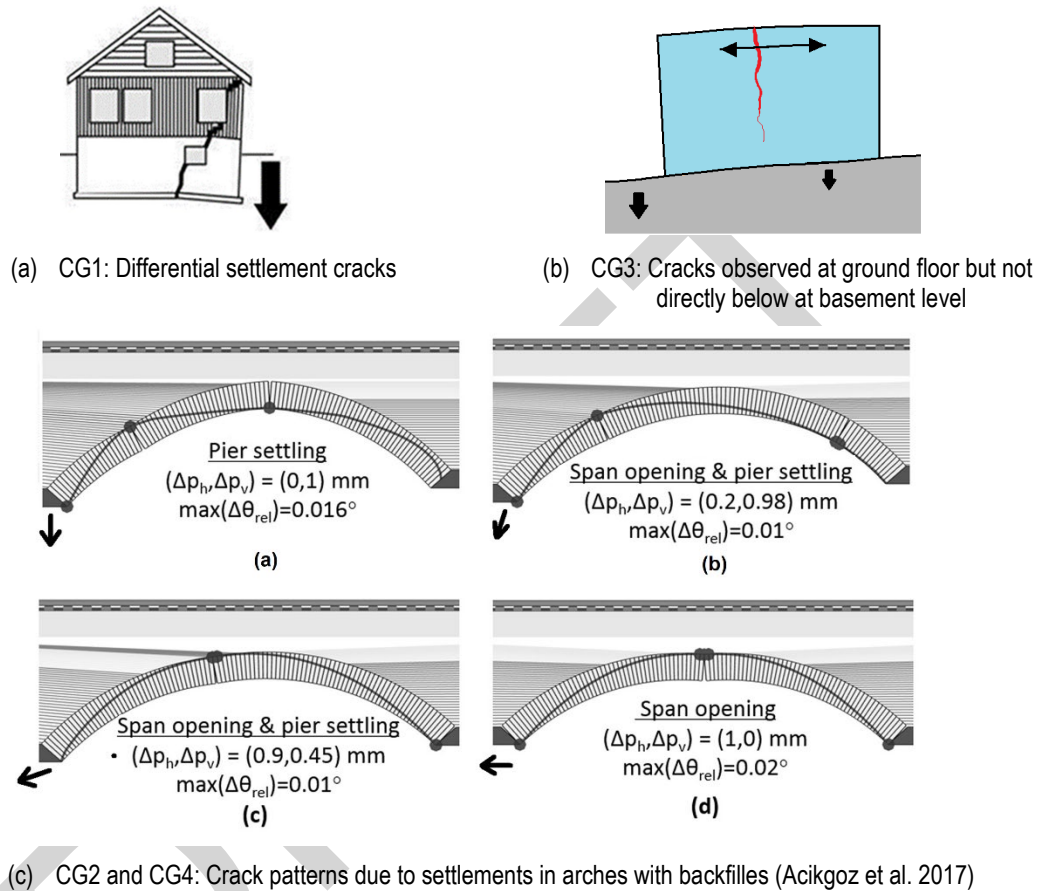


Figure 5-6. Summary of possible cracking mechanisms observed

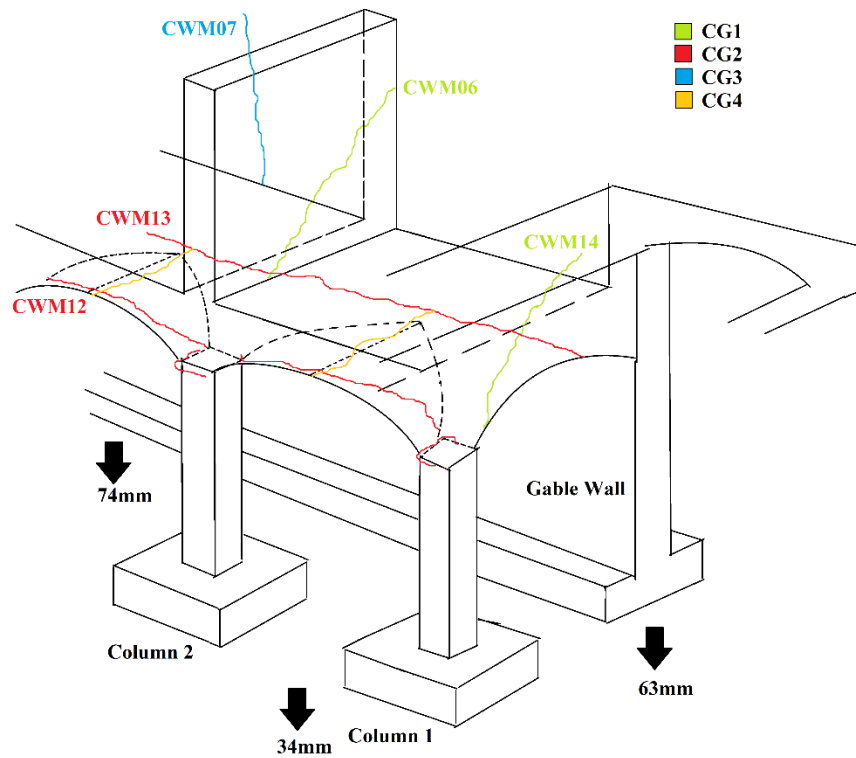


Figure 5-7. Schematic describing possible cracking mechanisms in relation to observed displacements and crack patterns for vaulted floor and concrete gable walls (schematic reflects north gable wall of Zone 3 and adjacent walkway, and observed cracking in this region)

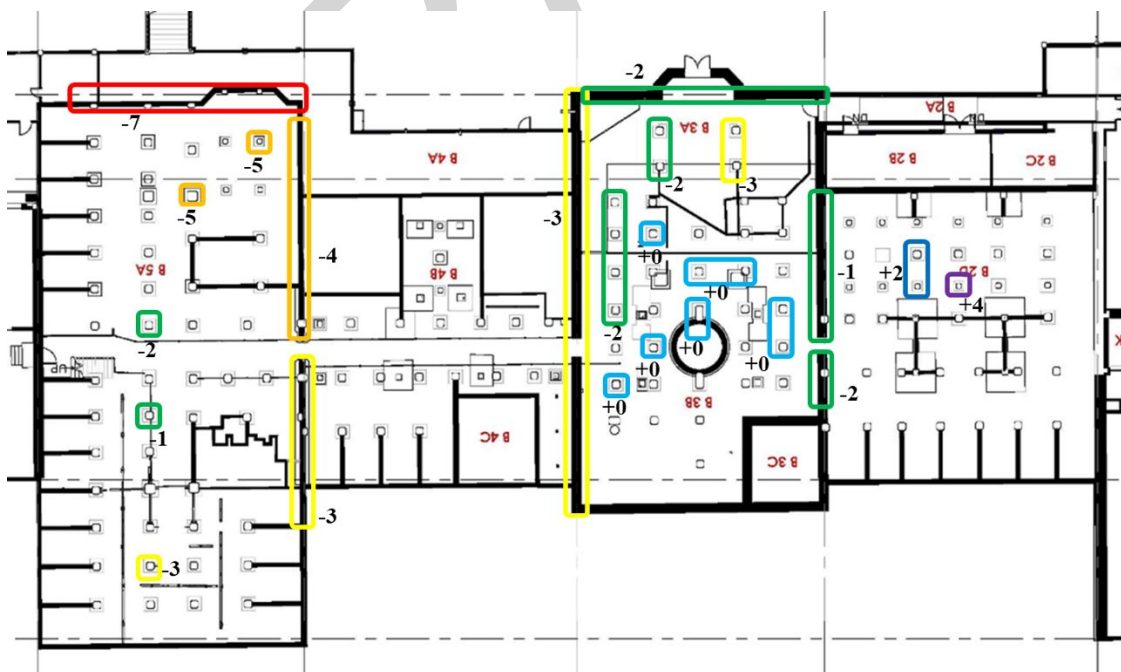


Figure 5-8. Difference in footing elevation between 2016 and 2020 (min. error $\pm 0.5\text{mm}$, max. error $\pm 2\text{mm}$)

Figure 5-9 shows a comparison of footing elevations between the two point clouds collected in 2016 and 2020. The observed differences provide evidence that the structure has continued to experience settlement in recent years.

5.3 Crack monitoring

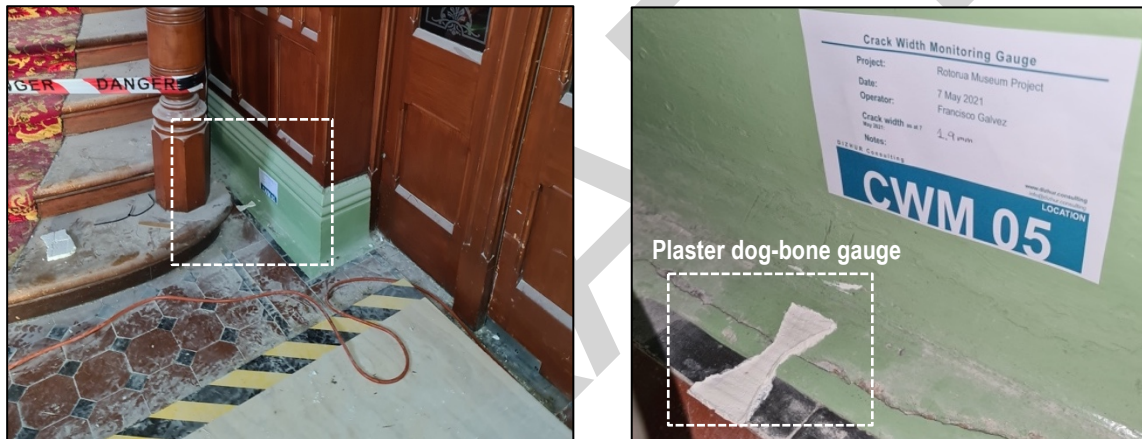
Due to the observations made regarding the extensive cracking of the vaulted floor and a comparative lack of information regarding whether the building was continuing to experience movement and if so, the rate at which it was moving, a comprehensive crack monitoring study was commenced, as described by the subsequent sections.

5.3.1 Manual crack monitoring gauges

As part of on-site investigation works undertaken on 5-7 May 2021, DIZHUR Consulting placed 20 crack monitoring gauges throughout the superstructure of the building on 7 May 2021 to determine over a period of time whether the cracks are active or dormant. An additional 21 crack monitoring gauges were placed throughout the structure in November 2021, resulting in a total of 41 gauges.

The gauges were made using tape placed over the width of the crack, on top of which a dog-bone shaped 5-10mm thick layer of plaster was placed with the narrow neck localised over the crack to facilitate that the gauge brakes at the desire spot. The crack width as at the date of installation of the gauges was recorded.

Examples of crack width gauges installed on various building elements at locations throughout the building are shown in **Figure 1-1**. See **Appendix 3** for the full crack monitoring report which details all crack width monitoring gauges and their locations, and the crack width measurement log.



(a) Example of floor separation monitoring in Zone 4 adjacent to south gable wall (CWM 5)



(b) Typical example of wall crack monitoring (CWM 6)

Figure 5-9. Examples of installed crack width gauges at various building elements at locations throughout the building

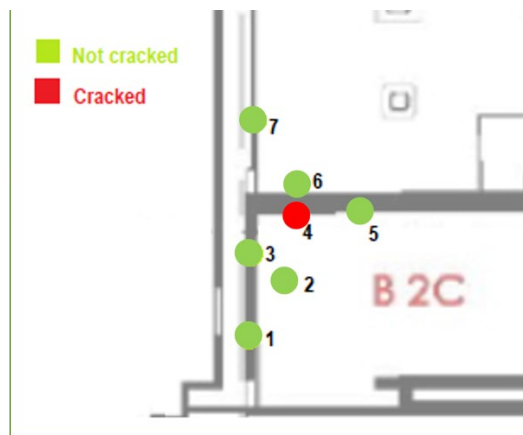
5.3.2 Existing gauges

Existing glass crack gauges (i.e., not placed by DIZHUR Consulting) were found in the South side of Zone 2 placed on the 23 January 2016. The location and whether they were found to have cracked or not are reported in [Figure 5-5](#).

At gauge 4, approx. 2.5mm of relative displacement between the two sides of the crack had occurred, which describes the relative movement that occurred between the southern wall of Zone 2 and the perpendicular wall that the gauge was placed on (see [Figure 5-5a](#)).

Gauge 6 was placed in the same crack as gauge 4, but on the opposite side of the wall. This gauge had detached from the wall on one side of the crack, however considering the significant relative movement experienced by gauge 4, it is hypothesised that this detachment could have been caused by wall movement. Gauges 1, 3, 5 and 7 did not exhibit any type of failure.

Based on these observations, it appears that the south wall of Zone 2 is moving differentially to the orthogonal wall – in turn, this difference in movement is likely to be inducing stresses within the vaulted concrete floor which results in cracking of the slab to relieve itself of the induced stress concentrations.



(a) Plan of glass gauges locations – all are focused around southeast corner of Zone 2



(b) 1 – Not cracked



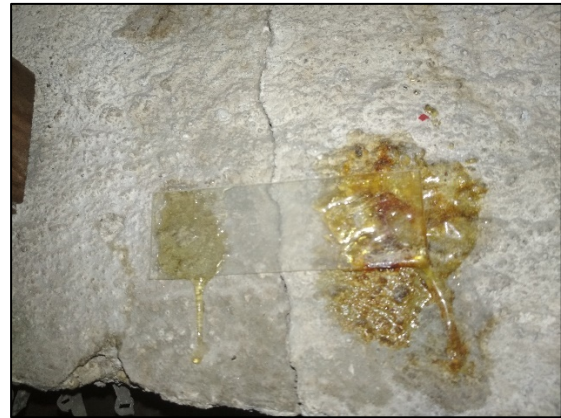
(c) 2 – not cracked (one side detached from wall)



(d) 3 – Not cracked



(e) 4 – Cracked



(f) 5 – Not cracked



(g) 6 – not cracked (one side detached from wall)



(h) 7 - Not cracked

Figure 5-10. Crack width monitoring gauges (pre-existing i.e., not installed by DIZHUR Consulting) with new cracking identified (photos taken 17 November 2021)

5.3.3 Digital crack monitoring

The observations made with respect to the plaster crack monitoring gauges installed in May and November 2021 were sufficient for the purpose of providing evidence of movement and, in turn, a high-level estimate of rate of movement. Considering the relatively short timeframe in which movement was identified following the installation of the first set of plaster gauges in May 2021 (approx. 5 weeks), a more robust crack monitoring system composed of displacement transducers connected to a fully automated acquisition system was installed in the structure on the 17 December 2021. 5 digital gauges were installed in the building, the locations of which are provided in [Figure 5-10](#).

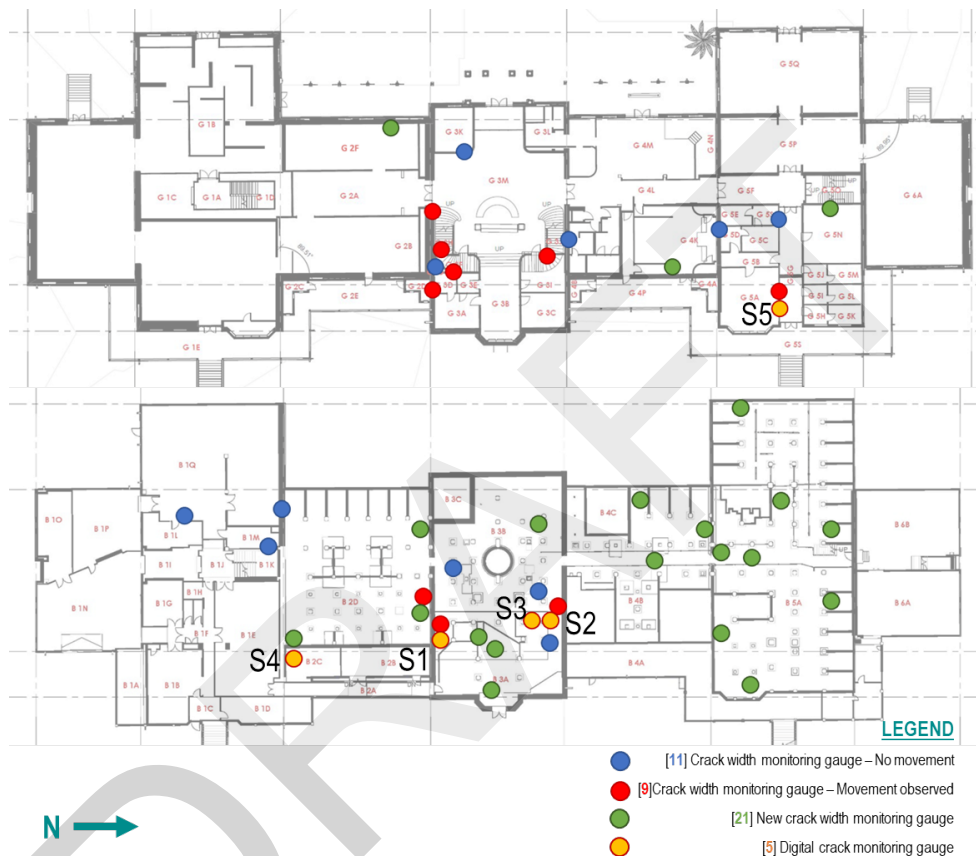


Figure 5-11. Crack gauge locations with identification of gauges that have exhibited movement (note: data is from Nov 2021, does not consider movement observations for new plaster gauges (green) installed during Nov 2021, nor for the digital ones (yellow))

Continual monitoring over a period of eight weeks (see [Figure 5-11](#)) has shown daily cycles of expansion and contraction of approx. 0.01mm. With regard to cracks opening, it was observed that all the sensors installed experienced fluctuations in movement of between 0.1mm and 0.15mm of movement per month, which was similar to the estimates made using the plaster gauges. Sensor S2, corresponding to the connection between the Zone 3 north wall (A5 wall) and the vaulted floor, showed a monthly opening and closing of approx. 0.1mm. Sensor S5, corresponding to a crack located in Zone 5, appeared to be opening at a steady rate of approx. 0.1mm per month, also in line with observations regarding plaster gauges.

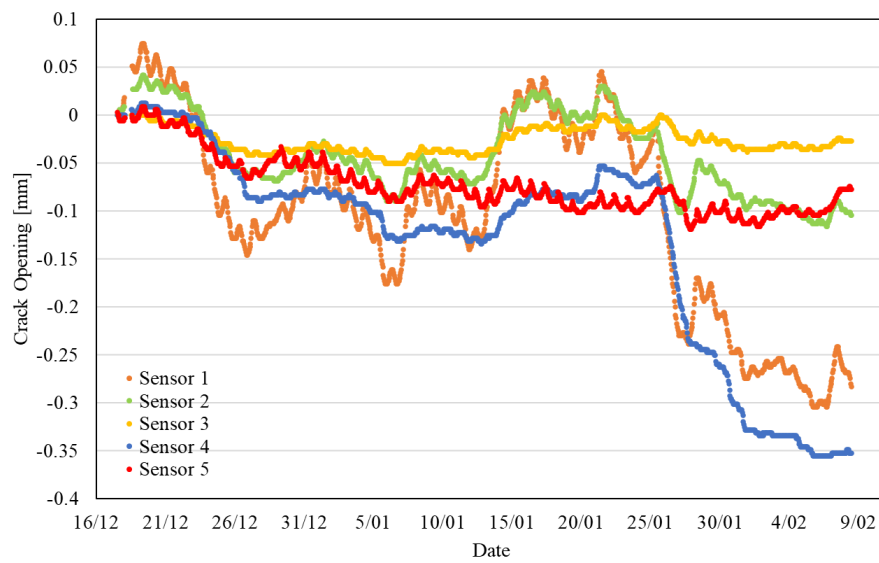


Figure 5-12. Data obtained over a period of 8 weeks from digital gauges installed at crack locations within building (recorded December 2021 – February 2022)

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6 MATERIAL TESTING

6.1 Concrete core testing

6.1.1 Concrete material quality

During the extraction of concrete core samples in the original zones of the building (see [Figure 6-1](#)), it was observed that the concrete quality was generally poor, see [Figure 6-2](#). The concrete was generally found to contain voids and was poorly compacted. Samples extracted from the vaulted floor slab also broke into pieces upon extraction, indicating that the slab had possibly been placed in layers which were not well integrated. Some samples extracted from walls were also found to contain cracks through the cross section of the core sample, which was not discernible from the wall surface.

For compressive strength testing, a total of 19 core samples were extracted: 16 from basement and ground floor walls, and 3 from the vaulted floor slab.

For split tensile strength testing, a total of 17 core samples were extracted: 16 from basement and ground floor walls, and 1 from the vaulted floor slab.

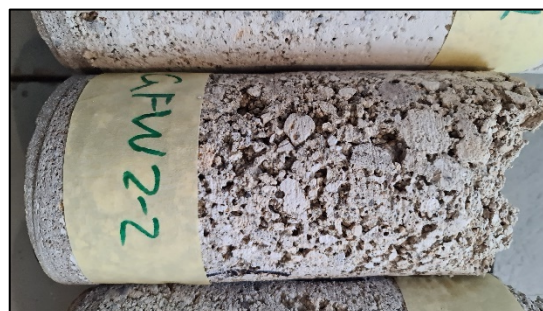
All concrete core extraction locations are provided in [Appendix 2.1](#).



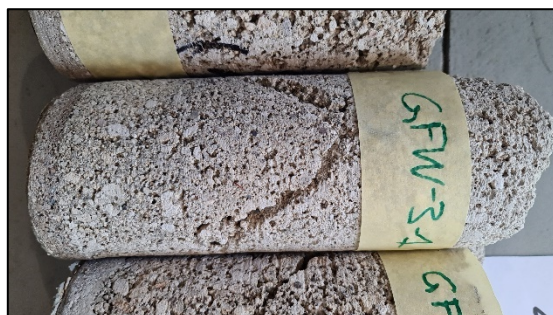
Figure 6-1: All collected concrete core samples



(a) B-S-1 – showing reasonably well compacted concrete



(b) GF-W-2-2 – showing poorly compacted concrete with voids



(c) GF-W-3-1 – showing voids/cracks in cross section of core



(d) Core samples showing large variability in compactness of material



(e) Cores BS-2-1 and BS-1 (floor slab cores from Zones 4 and 5, respectively) showing 'layers' of poorly compacted concrete causing core to break apart

Figure 6-2: Observations showing poor concrete quality/large variability in material

6.1.2 Concrete compressive strength, density, and modulus of elasticity

The core sample length was made parallel using a wet saw, dried at ambient temperature and capped using gypsum plaster prior to testing in compression and measuring the density. The compressive strength was measured according to ASTM C39 (2018) while the modulus of elasticity was measured according to ASTM C469 (2014). The cylinder dimensions were approximately 120/180 L × 93/94 D mm, using a 1.5 (or higher) aspect ratio as suggested by the standard. Sample weight was measured prior to testing.

Table 6-1. Concrete compressive strength properties of the subject building

Location	COMPRESSIVE STRENGTH		DENSITY ρ (kg/m ³)*	MODULUS OF ELASTICITY E (GPa)*
	Cylindrical f'_c (MPa)*	CoV		
Basement walls	10.7 (11)	0.38	1340.9	6.4
Ground floor walls	6.0 (5)	0.22	1272.1	3.7
Vaulted floor slab	8.2 (3)	0.00	1399.1	4.2
Reference standard or method	ASTM C39 (2018)			ASTM C469 (2014)

Note: (#) – Number of samples tested; CoV – Coefficient of variation; * Average batch value

Samples extracted from the basement walls resulted in average compressive strength of 10.7 MPa (CoV 0.38), average modulus of elasticity of 6.4 GPa (CoV 0.33), and average density at the time of testing was 1340.9 kg/m³ (CoV 0.05).

Samples extracted from the ground floor walls resulted in an average compressive strength of 6.0 MPa (CoV 0.22) and average modulus of elasticity of 3.7 GPa (CoV 0.01). The average density at the time of testing was 1272.1 kg/m³ (CoV 0.03).

Samples extracted from the vaulted floor slab resulted in an average compressive strength of 8.2 MPa (CoV 0.00) and average modulus of elasticity of 4.2 GPa. Note that two of the samples had a height to diameter ratio of less than 1.5,

therefore were not included in the average elastic modulus calculation. The average density at the time of testing was 1399.1 kg/m^3 (CoV 0.01).

Figure 6-4 shows typical failure modes during compressive tests. A summary of the results conducted on the extracted concrete samples is presented in **Table 6-1** and individual sample test results are included for completeness in **Appendix 2.4**.

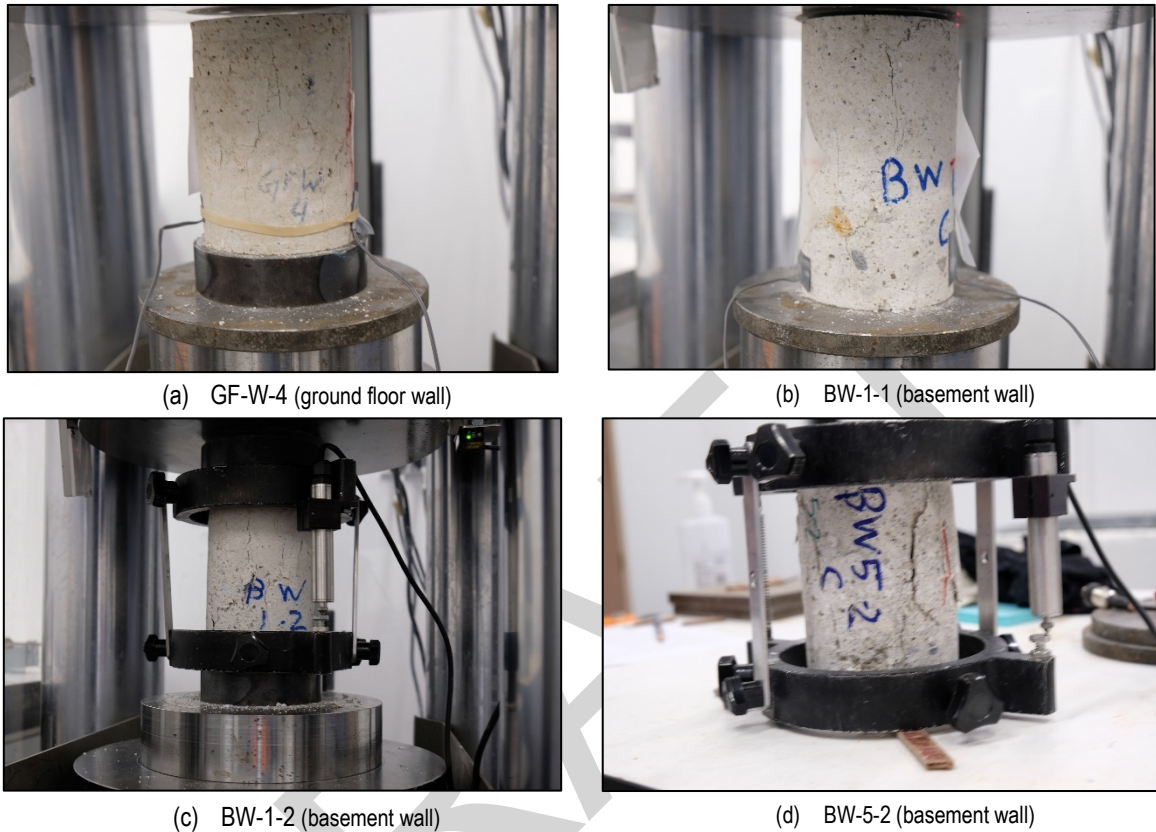


Figure 6-3. Typical failure modes for concrete cylindrical compressive strength samples

6.1.3 Concrete split tensile strength

The core sample length was made parallel using a wet saw, dried at ambient temperature and capped using gypsum plaster prior to testing in compression and measuring the density. The tensile strength was measured according to ASTM C496 (2017). The cylinder dimensions were approximately 140 L × 93/94 D mm, using a 1.5 (or higher) aspect ratio as suggested by the standard. Sample weight was measured prior to testing.

Table 6-2. Concrete tensile strength properties of the subject building

Location	TENSILE STRENGTH		DENSITY
	Cylindrical f_t (MPa)*	CoV	
			ρ (kg/m ³)*
All locations	0.8 (17)	0.42	1252.0
Reference standard or method	ASTM C496 (2017)		
Note: (#) – Number of samples tested; CoV – Coefficient of variation; * Average batch value			

The 17 samples extracted from walls and floor slab resulted in an average tensile strength of 0.8 MPa (CoV 0.42), and average density at the time of testing was 1252.0 kg/m³ (CoV 0.07).

Figure 6-4 shows typical failure modes during tensile tests. A summary of the results conducted on the extracted concrete samples is presented in Table 6-2 and individual sample test results are included for completeness in Appendix 2.5.



Figure 6-4. Typical failure modes for concrete cylindrical tensile strength samples

As was observed in the compressive strength study, a correlation between density and tensile strength was also observed (see Figure 6-5).

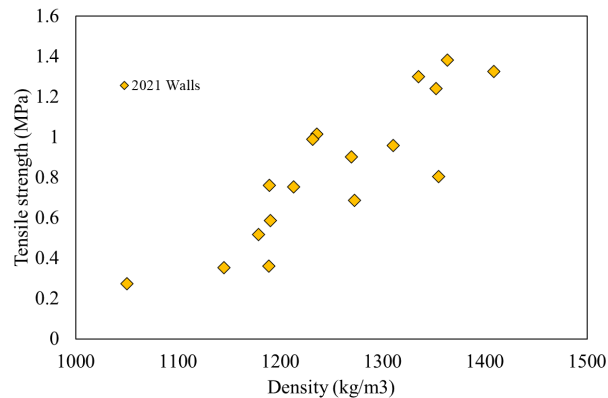


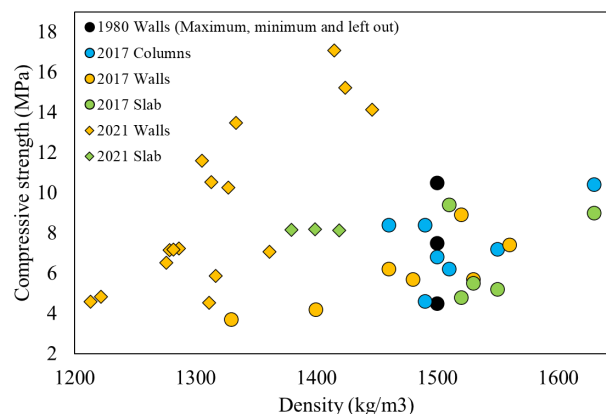
Figure 6-5. 2021 pumice concrete split tensile testing campaign results

6.1.4 Comparison of compressive strength & density with previous core extractions

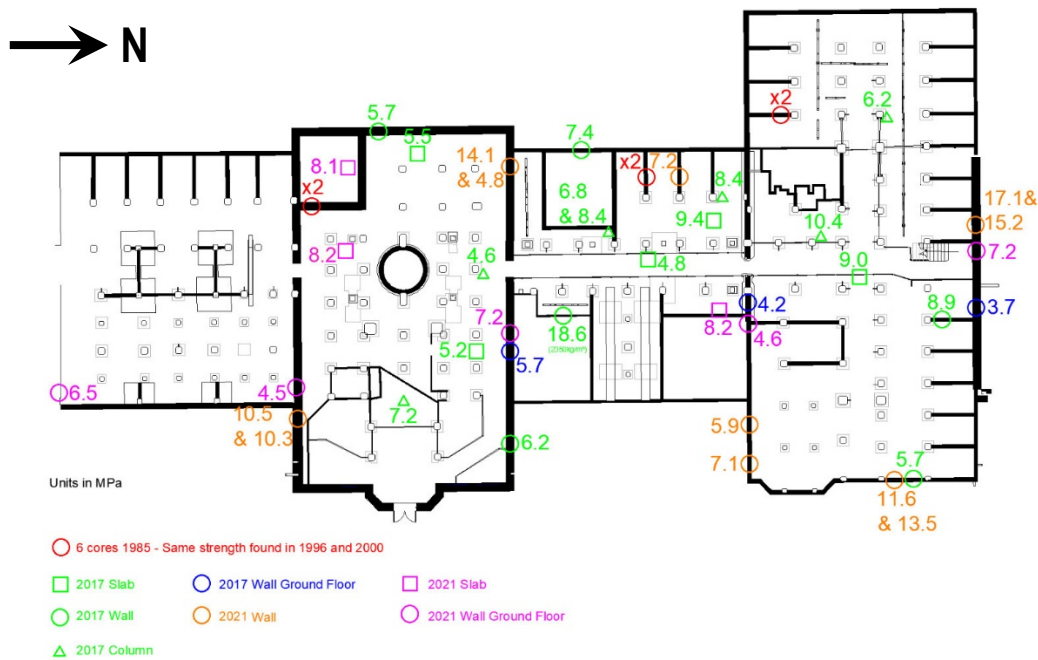
In 1985, Kerslake and Partners extracted seven concrete core samples (100mm diameter, 200mm length) from the walls to determine the average compressive strength (refer DOC547-v1, DOC548-v1, DOC549-v1). This was found to be approx. 8.4MPa, with a minimum of 4.5MPa and maximum of 10.5MPa, thus exhibiting a large variability in the quality of the material. The density of the concrete was calculated as approx. 1500kg/m³. Additional cores extracted in 1991 indicated similar strength and density to the 1985 cores.

In 2017, an invasive testing campaign involving the extraction of 21 core samples (75mm diameter, 150mm length) was undertaken by GDC Consulting and Opus Consultants (refer DOC795-v1). An average density of 1538kg/m³ was reported, in line with 1985 findings. The average compressive strength was found to be approx. 6.7 MPa.

With respect to cores extracted by DIZHUR Consulting in 2021, the calculated densities are typically lower than those reported for the 1985, 1991 and 2017 cores (between 1252-1399kg/m³ (2021 cores) compared to 1500-1538kg/m³). For both historic and new samples, a generally positive trend between compressive strength and density was observed despite large variability, see Figure 6-6a. As can be observed in Figure 6-6b, values from ground floor walls generally appeared to be lower than those at basement level, which may be due to slightly improved compaction at the time of construction of the concrete at basement level as a result of greater overburden weight. It is also observed that some tests (both historic and new) which were only approx. 1m apart, still resulted in widely different compressive strengths (for example 4.1MPa vs. 14.1MPa from two cores extracted at Zone 3 north gable wall at the west side (2021 campaign), 11.6MPa/13.5MPa vs. 5.7MPa at Zone 5 basement east perimeter wall (2021 and 2017 respectively)). These observations reinforce the large variability of the concrete quality.



(a) Graph of compressive strength against density for pumice concrete samples extracted in previous years + 2021 samples extracted by DIZHUR Consulting



(a) Schematic showing distribution of historic and 2021 concrete core sample extractions and compressive strengths. Note: Wall keys refer to basement walls unless otherwise stated.

Figure 6-6. Data comparing compressive strength, density and locations of samples extracted in 2021 against historically extracted samples

6.2 Pumice concrete infill panel testing

As part of investigation works undertaken on 5-7 May 2021, pumice concrete infill panel samples were extracted from the building and subjected to compressive strength testing to assist in determining the current strength of the structure and to aid the concept design of the retrofit intervention.

Pumice concrete infill panel samples were extracted from the east wall of Zone 4 at ground floor of the building where they were exposed. The panels were generally painted on the exterior, and on the interior varied between being exposed in some regions, covered in lath/plaster and/or lining in other regions.

The building has been constructed with timber framing with approx. 40-50mm thick infill panels consisting of a more lightweight mix of concrete made using pumice. The infill panels were found to contain chicken wire mesh at approx. mid-section of the panels, which has assisted in holding the panels together despite cracking/localised deterioration.

Aside from localised cracks observed in some infill panels, they were generally observed to be in good condition. Small voids were observed in the cross section of the infill panels, as well as small pieces of pumice concrete. The aggregate was generally small sized, as was expected considering the geometry of the panels.



a) General construction of east & west perimeter walls with concrete nib wall at base and timber frame wall above with pumice concrete infill panels



b) Closeup of infill panels



c) Typical sample extraction, see also d)



(d) Typical sample extraction



(e) Closeup of infill samples for testing showing small voids and pumice material in concrete mix

Figure 6-7. Typical timber framing/pumice concrete infill panel construction encountered for east and west perimeter walls of the subject building

6.2.1 Results of the laboratory testing

Compression tests were undertaken on infill panel samples following the procedure described in Válek & Veiga (2005), which suggests cutting irregular specimens into approximately cubical shapes with two parallel sides (top and bottom) that are capped using gypsum plaster prior to testing (see [Figure 6-8](#)). The compressive strength results were then normalised following the procedures detailed in Section C8 of the New Zealand Society for Earthquake Engineering Guidelines (MBIE/NZSEE 2017) to account for irregular sample geometry.

A summary of the results (average value) of the compression tests conducted on the extracted material samples is presented in [Table 6-3](#). Individual infill panel sample test results, as well as group averages are included for completeness in [Appendix 2.2](#). The overall average infill panel compression strength is 30.7 MPa (CoV 0.05), and average density of the infill panels is 1530 kg/m³.

The degree of variation in the compressive strength results (see coefficient of variation (CoV) in [Table 6-3](#)) is well within the range of variability that is expected for early concrete mixes. The compression strength results agree with the visual observations made regarding the pumice concrete mix condition.



(a) Extracted and prepared mortar samples



(b) Typical failure observation

Figure 6-8. Typical example of infill panel compression tests

Table 6-3. Material properties of the infill panels

Location	Level	f_c (MPa) *	CoV (%)	Density (kg/m ³)
Zone 4 east wall	Ground floor	30.67 (18)	5.5	1527.6
Reference standard or method		Válek & Veiga (2005)		

Note: (#) – Number of samples tested; CoV – Coefficient of variation; * Average compressive strength value

6.3 Timber species identification

As part of investigation works undertaken on 5-7 May 2021, timber samples were extracted from the building for identification of the species. Seven timber samples were extracted from Zones 2, 3 and 4 of the building for species identification, see [Figure 3](#). Previous timber investigations had predominantly extracted samples from the roof space of the historic regions of the structure, and those samples had typically been identified as Kauri. The timber samples for this investigation were extracted from the walls at ground floor, with one sample extracted from the mezzanine floor in Zone 3. [Table 2](#) below provides a summary of the results of the species identification, with a summary from the timber specialist also provided in [Appendix 2.1](#).

**Figure 6-9.** Timber samples extracted for species identification**Table 6-4.** Results of timber species identification

SAMPLE ID	LOCATION	MEMBER	SPECIES
TS1	Zone 4 east perimeter wall	Smaller horizontal stud	Totara
TS2	Zone 4 east perimeter wall	Main vertical member	Totara
TS3	Zone 3 west perimeter wall	Main vertical member	Totara
TS4	Zone 3 west perimeter wall	Diagonal brace	Totara
TS5	Zone 3 west perimeter wall	Primary horizontal member	Totara
TS6	Zone 3 mezzanine floor	Floor joist	Totara
TS7	Zone 2 east perimeter wall	Timber baseplate	Totara

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7 NUMERICAL MODEL DEFINITION AND ANALYSIS APPROACH FOR ZONES 2-5

The seismic response of the structure is determined through a series of Nonlinear Static Analyses (pushover analyses NLSPA) as recommended by the most recent international standards for seismic assessment and rehabilitation of existing structures (ASCE SEI 41/17, Eurocode 8, NTC2018 and The Seismic Assessment of Existing Buildings (the Guidelines) (2017).

7.1 Seismic action

In accordance with the NZS 1170.5 (section 3.1), the elastic site hazard spectrum for horizontal loading $C(T)$ is defined as:

$$C(T) = C_h(T) Z R N(T, D)$$

Where:

- For a class D shallow soil, the spectral shape factor $C_h(T)$ was determined as follows (for the modal response spectrum (MRS)):

Class D deep or soft soil

The values for the MRS and NITH methods are given by:

$$C_h(0) = 1.12$$

For $0 < T < 0.1$

$$C_h(T) = 1.12 + 1.88 (T/0.1)$$

For $0.1 \leq T < 0.56$

$$C_h(T) = 3.0$$

For $0.56 \leq T \leq 1.5$

$$C_h(T) = 2.4 (0.75/T)^{0.75}$$

For $1.5 < T \leq 3$

$$C_h(T) = 2.14/T$$

For $3 < T$

$$C_h(T) = 6.42/T^2$$

For the equivalent static method, the plateau extends back to 0.0 s period:

For $0 \leq T < 0.56$

$$C_h(T) = 3.0$$

- $Z=0.24$ is the hazard factor for the Rotorua site according to Table 3.3 (Section 3.1.4)
- $R_u=1.3$ is the return period factor for the Ultimate Limit State (ULS) according to the table reported below.

TABLE 3.5
RETURN PERIOD FACTOR

Required annual probability of exceedance	R_s or R_u
1/2500	1.8
1/2000	1.7
1/1000	1.3
1/500	1.0
1/250	0.75
1/100	0.5
1/50	0.35
1/25	0.25
1/20	0.20

The parameters used for the definition of the elastic site spectra for horizontal loading for the ultimate limit state are summarised in the following table:

Seismic action parameters (ULS)		
Importance level	3	Table 3.1 NZS 1170.0
Design working life	50 years	
Annual probability of exceedance	1/1000	3.4 NZS 1170.5
Site subsoil class	D	
Spectral shape factor for $T=0$ s	$Ch(0)=1.12$ g	3.1.2 NZS 1170.5
Hazard factor	$Z=0.24$	3.1.4 NZS 1170.5
Return period factor (ULS)	$R_u=1.3$	3.1.5 NZS 1170.5
Near fault factor	$N(T,D)=1$	3.1.6 NZS 1170.5

The results are presented in the figures below (for a damping value of $\xi = 5\%$):

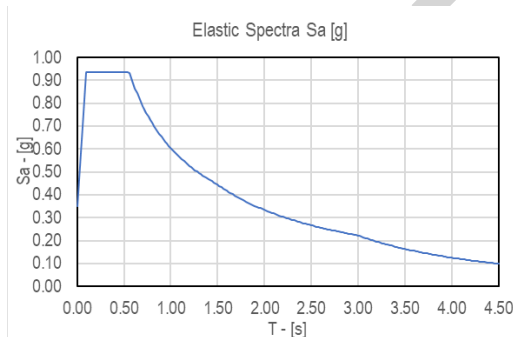


Figure 7-1. Elastic acceleration spectrum

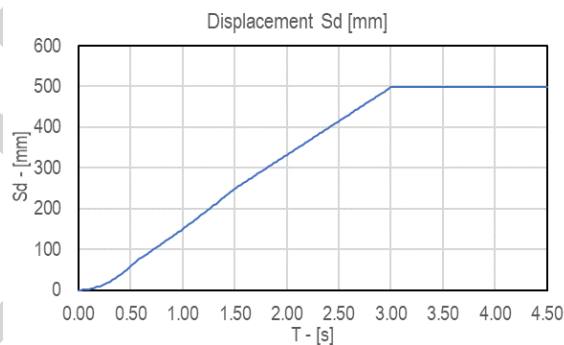


Figure 7-2. Elastic displacement spectrum

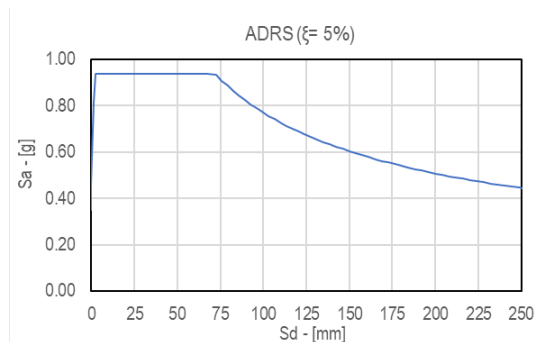


Figure 7-3. Acceleration displacement response spectrum

7.2 Structural analysis approach

This section reports the process of study and analysis adopted to evaluate the seismic capacity of the superstructure for the historic section of the Bath House building. Given the large dimension of the structure and the complexity of its composition (**Figure 7-4**), the building is divided in 6 zones which have been examined individually (**Figure 7-5**).



Figure 7-4: Isometric view of a 3D model representation of the Bath House building – Rotorua Museum

The following chapter specifically will treat Zones 2, 3, 4 and 5, because these are the portions comprising the original structure (**Figure 7-6** and **Figure 7-7**). These zones, due to the limited strength properties of some of the original materials and the outdated construction methods used for their fabrication, are likely to be more prone to damage under intense seismic action in comparison to the zones of modern construction (Zones 1 and 6).

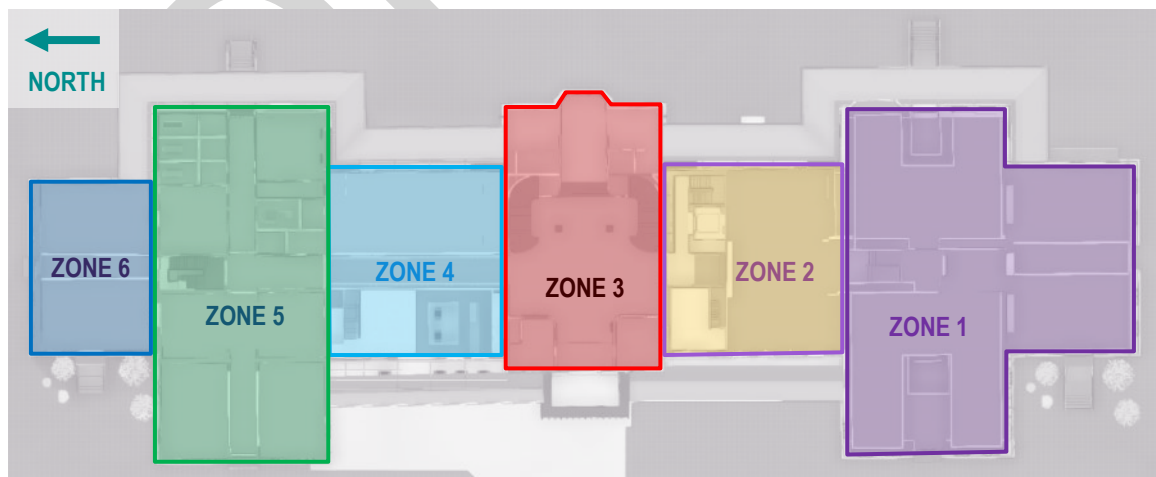


Figure 7-5: Identification and labelling of the various zones adopted for the structural analysis of the building

From on-site investigation and from a desk survey of the available design documentation of the modern additions, it was found that the new structures (i.e., Zone 1 and Zone 6) were built adopting appropriate seismic gaps with the adjacent historic building. Therefore, disregarding the influence of the modern additions on the lateral behaviour of the original structure can be deemed as a reasonable assumption.

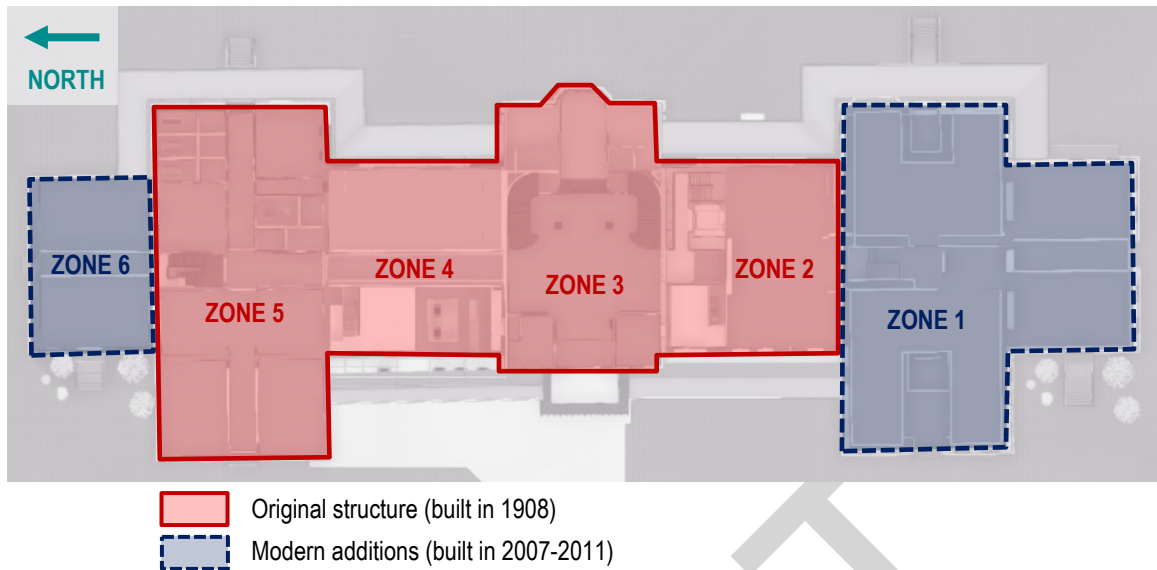


Figure 7-6: Date of construction of different sections of the building. The original structure is highlighted in red, and examined in detail in the following chapter

Zones 2-3-4-5 underwent several structural modifications over the course of the building life-span. **Figure 7-8** reports the current layout of the main vertical structural elements which are supposed to contribute to the lateral capacity of the building.

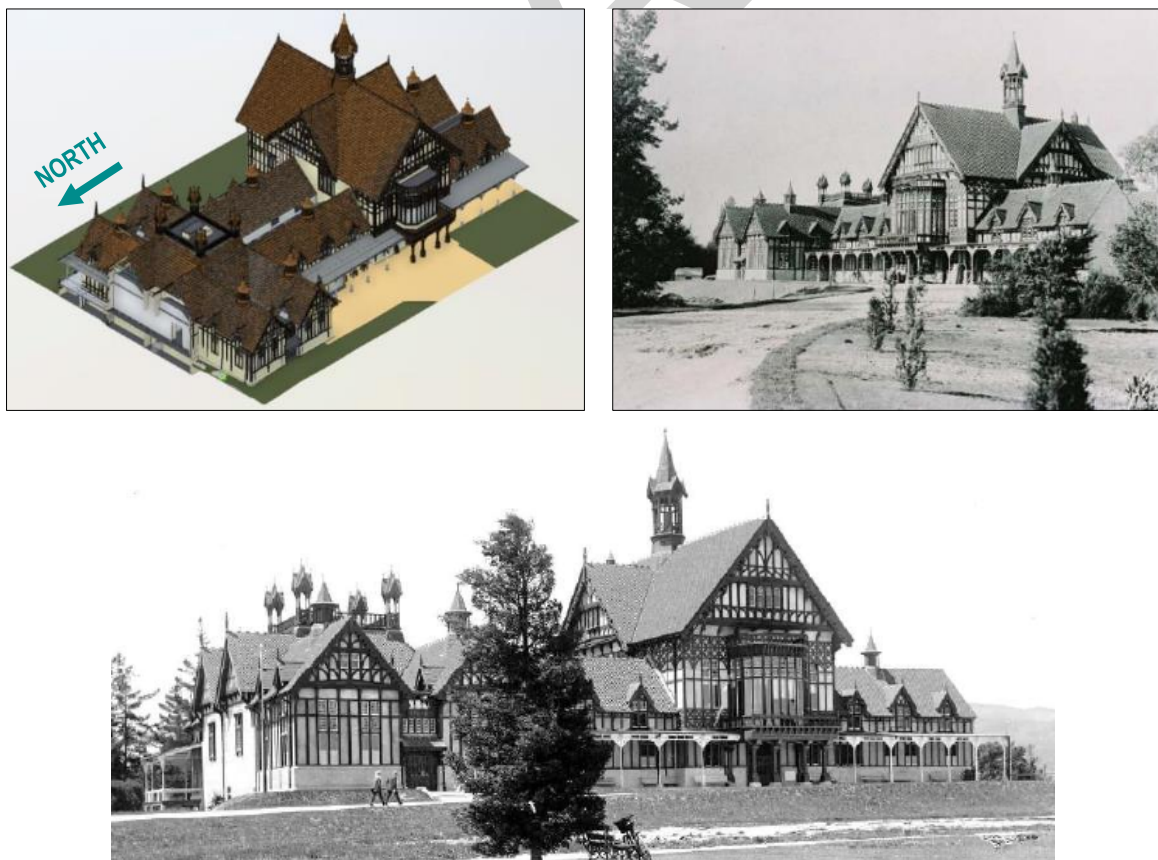


Figure 7-7: Isometric view of the original structure and historic photos taken shortly after work completion

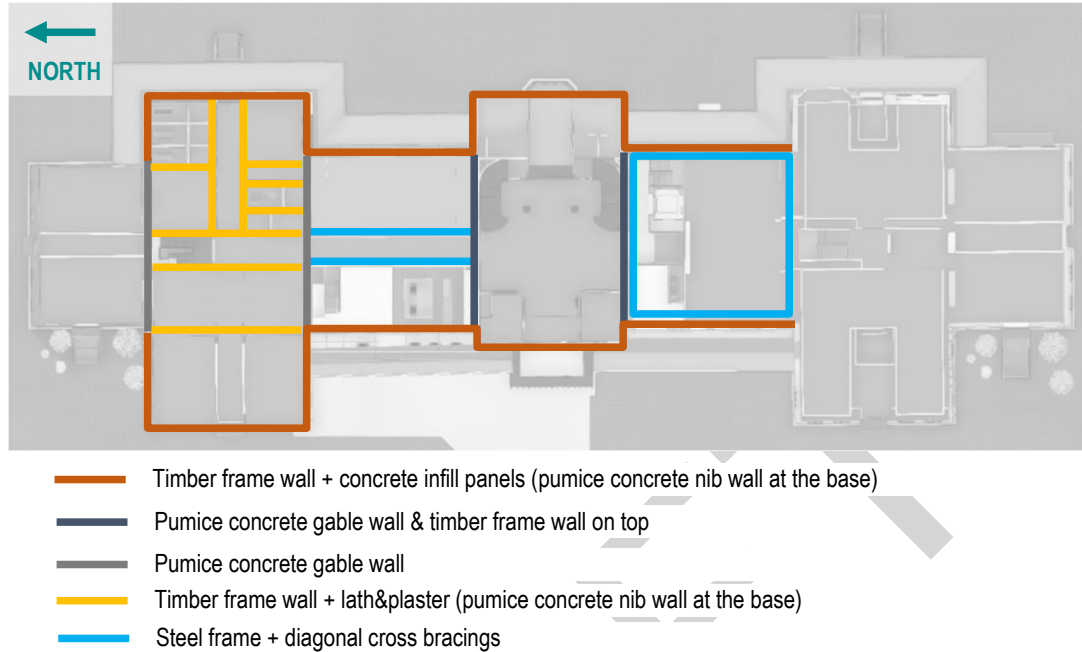


Figure 7-8: Current structural layout of the original zones of the building

7.3 Numerical model details

7.3.1 Methods of analysis and modelling strategies

The building and most of its components are modelled adopting the Finite Element (FE) software SAP2000. Given the complex nature of the structure and of its loading conditions, numerous models were defined which ranged from single structural elements to the building in its entirety (considering only the original zones of the structure as defined in the previous section). The building was modelled from the foundation level including also the basement structure, however in parallel, another model neglecting the basement and floor structure was defined and used for the analyses as will be explained below.

Nonlinear static pushover analyses (NLSPA) were performed to determine the probable global displacement capacity of the building. Specifically, the procedure presented in section C2.4.3 of NZSEE 2017 (MBIE-NZSEE 2017a) was used.

The typical outcome of a pushover analysis is the capacity curve (or pushover curve), which is a curve representing the building shear capacity versus the displacement of a control point. The safety level of the structure is defined by the % of **new building standard (NBS)** achieved.

To verify the structure, the acceleration displacement response spectrum was compared with the capacity curve of the equivalent SDOF system (Single Degree of Freedom), derived from the curve of the MDOF system (Multi Degrees of Freedom).

The SDOF curve is determined based on the indications contained in the “*Technical Guidelines for Engineering Assessment – Part C2: Assessment Procedures and Analysis Techniques*”. In accordance with step D3 of the Standard document, the effective mass m_{eff} is assumed equal to the total mass of the building and the effective height h_{eff} is assumed equal to $0.67 \cdot h$, h being the building height measured from the foundation level to the level of the considered control point of the analysis. In the case of the local analyses on the single walls, the SDOF curves are determined assuming the mass of the wall as the effective mass m_{eff} , including the tributary load from the horizontal structures (e.g., floor tributary areas), and assuming the effective height h_{eff} as a value of $0.67 \cdot h$, with h equal to the total height of the

wall. Unless otherwise specified the MDOF capacity curves of the single walls are derived from the global model, hence considering the interaction effects between the various elements.

The structure is considered to meet the safety requirements when the SDOF capacity curve intersects the acceleration displacement response spectrum (ADRS). The %NBS coefficient can be calculated as the ratio between the ultimate displacement capacity of the equivalent SDOF system Δ_{Prob} and the displacement demand Δ_{ULS} , obtained from the intersection of the effective period T_{eff} line with the ADRS demand curve (Figure 7-9).

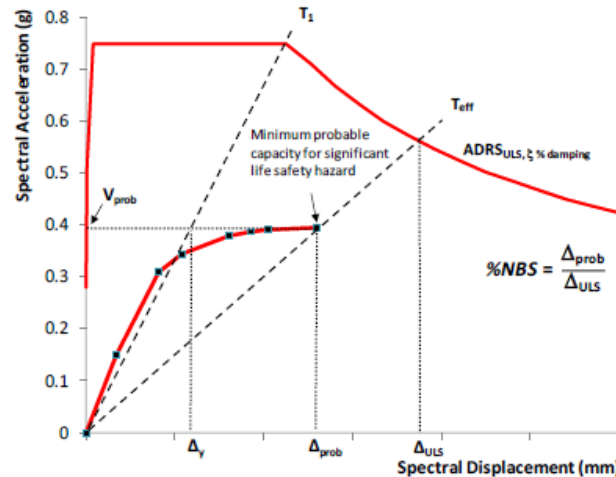
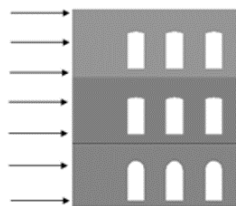


Figure 7-9. Nonlinear static pushover assessment according to NZSEE 2017

In the following a summary of the hypotheses and assumptions made for the nonlinear static pushover analysis is reported:

- The capacity curves are determined in relation to multiple reference points (Control Points) due to the different responses of the structure observed when selecting a single reference point.
- To take into account the effects of non-symmetrical distribution of mass and stiffness, the pushover analyses are performed considering two opposite directions for the seismic action, that from here on will be referred to as “+” or positive and “-” or negative.
- Lateral loads are applied to the numerical model in proportion to the distribution of inertia forces resulting from a uniform acceleration to analyse the structure performance beyond the plastic threshold (see Figure 7-10). Because of the difference in mass distribution along the building height (heavy pumice concrete base and light timber frame wall extension), additional models are examined neglecting the basement and ground floor structure and adopting again the uniform acceleration load pattern. This is done with the intention of reducing the influence of the heavy basement structure on the total base shear of the structure when applying a uniform acceleration load. The effective mass m_{eff} is then assumed equal to the total mass of the examined building portion.
- To account for the irregularity of the building, the nonlinear static pushover analyses were examined both from a global perspective (considering the whole building) and from a local perspective (studying the response of the single walls).



Load pattern proportional to the inertia force distribution

Figure 7-10: Pushover loading regime, simplified scheme

7.3.2 Numerical model elements and materials

This section lists all the materials and elements adopted to simulate the structural behaviour of the building and of the proposed retrofit intervention.

- All the structural elements made of the original pumice concrete material (i.e., gable walls in the west-to-east direction, nib walls and infill panels of the exterior timber framed walls) are modelled as layered shell elements using a modified version of the “Darwin-Pecknold” constitutive behaviour. The model is a co-axially rotating smeared-crack concrete model that considers cracking and crushing of the material. The “Darwin-Pecknold” model implemented in the software assumes as default a zero-tensile strength (Figure 1-11). This assumption, albeit conservative, tends to lead to an overestimation of the cracked material area which makes it difficult to identify the failure modes with certainty. Therefore, an additional material layer is introduced to simulate tensile strength. This “tension-layer” works in parallel with the “Darwin-Pecknold” layer and is characterized by a “tension-only” elastic behaviour with a linear softening once the tensile stress reaches the value selected for tensile strength. Compressive strength reduction based on perpendicular tensile strain is accounted for as described in Vecchio and Collins (1986).

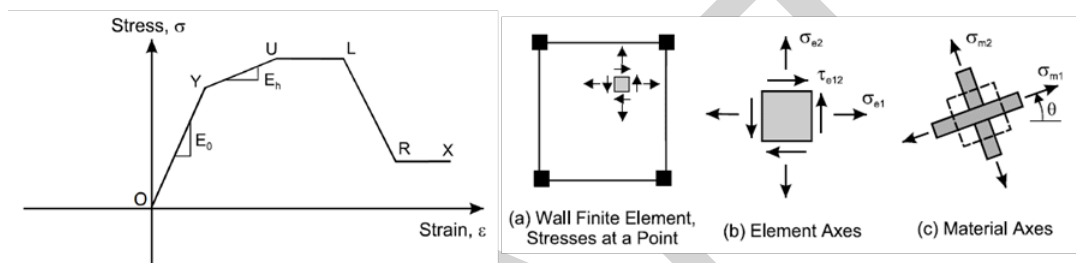


Figure 7-11. Uniaxial stress-strain relationship and principal material axes

The compressive strength and the density values are assumed equal to the mean values of the core sample testing undertaken in previous years, prior to the definition of the numerical model (compressive strength $f_c = 6.00$ MPa and density $\rho = 1500$ kg/m³). The concrete tensile strength f_{ctm} is estimated firstly from the formulation proposed by the CEB-FIP Model Code for concrete structures (1990). Using the compressive strength obtained from the experimental tests as a reference value, the tensile strength was equal to 1.00 MPa. This was then confirmed by further testing done on 17 new specimens extracted by DIZHUR Consulting (mean value of 0.83 MPa and standard deviation equal to 0.35 MPa). Nevertheless, to consider the high variability of the tensile strength property of concrete (neglecting the contribution of the original barbed wire steel reinforcement), additional analyses were run using a reduced tensile strength value equal to 0.10 MPa. The modulus of elasticity E_c is assumed equal to 6000 MPa, derived from tests done on 19 new specimens extracted by DIZHUR Consulting (mean value of 5698 MPa and standard deviation equal to 2198 MPa).

- All the timber elements of the original structure are modelled as linear elastic frame elements. The eventual safety checks are performed a-posteriori to ensure that the capacity of each structural element is sufficient for the given load level. As from historic documentation and on-site investigations, three different species are assumed for the timber structures: *Rimu* timber for ceiling and mezzanine floor joists, *Totara* timber for the wall framing and *Kauri* timber for the main roof and truss framing.

Species	Density [kg/m ³]	Modulus of elasticity [MPa]	Bending strength* [MPa]
Rimu	590	9600	19
Totara	480	6400	19**
Kauri	560	9100	19**

* For a-posteriori safety checks

** In absence of specific data (to the best of the author's knowledge) the characteristic bending strength for the Totara and Kauri timber species is assumed equal to the Rimu bending strength derived from the NZ Standard

- The steel elements currently present in the building (steel braced frame of Zones 2 and 4) and the steel elements that are planned to be inserted as part of the retrofit intervention were modelled as frame elements. The assumed material properties were taken equal to a steel Grade 300 according to the New Zealand standard AS/NZS 3679.1 (density $\rho = 7850 \text{ kg/m}^3$, modulus of elasticity $E = 205000 \text{ Mpa}$, minimum yield stress $f_y = 300 \text{ Mpa}$, minimum tensile stress $f_u = 440 \text{ Mpa}$). The non-linear behaviour of the material was accounted for by defining plastic hinges for each element, in particular axial hinges for the cross-bracing components associated with a compression limit equal to 0 kN, to account for tension-only rod bracing.
- To simulate the in-plane stiffness of the existing diaphragms (e.g., mezzanine floor of Zone 3 made of timber joists and orthogonal timber boards) and of the lath-and-plaster interior walls of Zone 5, linear elastic shell elements were used with a thickness of 100 mm and a shear modulus of 1.5 Mpa for the existing diaphragms and a shear modulus of 0.7 Mpa for the lath-and-plaster interior walls. These assumptions provide equivalent shear stiffnesses of the modelled elements equal to 150 kN/m and of 70 kN/m respectively (see Giongo et al 2018 and BRANZ 2013). It should be noted that in the retrofit proposal of the building, most of the lath-and-plaster interior walls will be substituted or coupled with new light-timber framed shear walls. Therefore, the shear stiffness of the original walls is purposely assumed to be on the low end of the spectrum of possible values.
- A Fabric-Reinforced Cementitious Matrix (FRCM) system is proposed as the strengthening intervention for the original unreinforced pumice concrete walls (the existing steel reinforcement is largely undersized when considering current standards). The FRCM system is modelled as an additional layer for the layered shell elements which simulate the pumice concrete elements. The thickness of the layer is taken as equal to the equivalent fibre thickness of the FRCM system t_f (usually provided by the manufacturer). For the constitutive behaviour of the material a debonding failure mode is assumed for the system. This is defined by the two parameters of conventional stress limit $\sigma_{lim,conv}$ and conventional strain limit $\epsilon_{lim,conv}$, as indicated by the guidelines of the Italian National Research Council CNR-DT 215 (2018). In the model, the three parameters needed for the characterization of the FRCM system are taken as $t_f = 0.17 \text{ mm}$, $\epsilon_{lim,conv} = 0.2\%$ and $\sigma_{lim,conv} = 380 \text{ MPa}$.
- To reinforce the in-plane and out-of-plane behaviour of several existing timber framed walls, a strengthening solution is proposed which connects the original wall to Cross-Laminated Timber (CLT) panels adequately anchored at the ground floor level. The CLT panels are modelled via linear elastic shell elements which replicate the structural behaviour of CLT using stiffness modifiers and assuming an orthotropic material calibrated with equivalent mechanical properties. The CLT panels are assumed to be 3-layered, 600 mm thick elements made of timber boards (200 mm thick and 200 mm wide) having a modulus of elasticity parallel to the fibres $E_{0,mean} = 11000 \text{ MPa}$, a modulus of elasticity perpendicular to the fibres $E_{90,mean} = 370 \text{ MPa}$, a shear modulus $G_{090,mean} = 690 \text{ MPa}$, and a rolling shear modulus $G_{9090,mean} = 69 \text{ MPa}$. The shell elements are then modelled as 600 mm thick shells using a membrane stiffness modifier along the main axis of the element $f_{11} = 0.69$, a membrane stiffness modifier along the weak axis of the element $f_{22} = 9.00$ and a twisting moment stiffness modifier $m_{12} = 1.01$. The mechanical properties for the equivalent orthotropic material are reported in the table below.

E_1 [MPa]	E_2 [MPa]	E_3 [MPa]	G_{12} [MPa]	G_{13} [MPa]	G_{23} [MPa]
10593	407	0	546	101	58

- Finally, plywood sheathing is adopted to strengthen the roof and ceiling diaphragms and the rest of the timber framed walls i.e., the ones which were not addressed with the CLT panel retrofit. The plywood overlay is simulated by using linear elastic shell elements with an equivalent in-plane stiffness of 1400 kN/m (see Rizzi et al. 2020).

The mechanical properties of the connections and other specific element or material properties (used for additional analyses on isolated structural member) will be reported in subsequent report sections on a zone-by-zone basis.

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8 ZONE 3 | NONLINEAR STATIC ANALYSES

8.1 Zone 3 | Isolated structure analyses

To examine the seismic behaviour of Zone 3 (i.e., Atrium section of the Rotorua Museum) the first analyses are performed assuming the main lateral resisting walls as self-standing isolated structures, after which the structure is analysed as a whole. The aim is to deduce information about the maximum lateral capacity and about the expected failure modes of the single structural components to propose a tailor-made retrofit intervention.

8.1.1 Z3 – West wall | Timber framed wall + infill panels

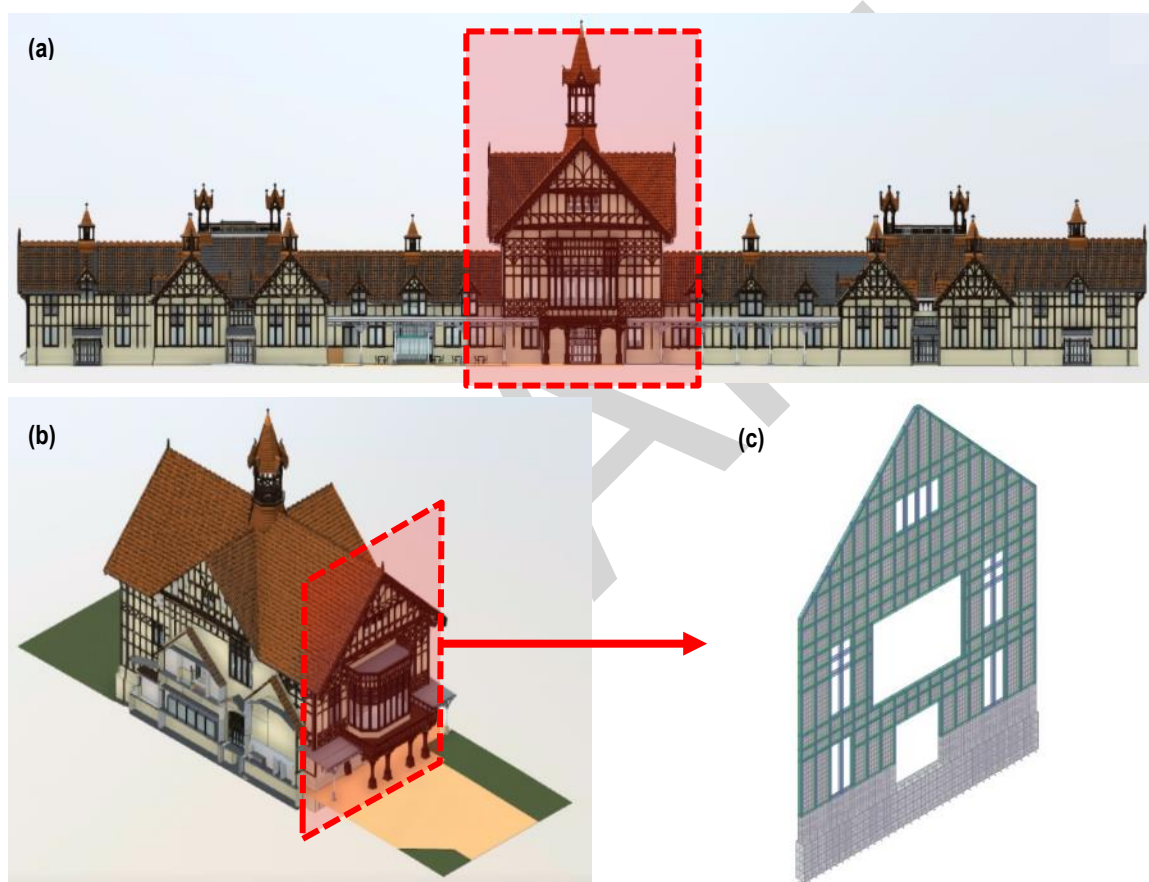


Figure 8-1: Front view of the Rotorua Museum (a) – Zone 3 (b) – FEM model geometry of front façade wall (c)

The main assumptions for the analysis of the West wall of Zone 3 (front façade of the Atrium section) are:

- In-plane analysis.
- Fixed restraints at the base of the wall (pinned restraint for each joint on the bottom edge of the base shell elements).
- Neglect the jettying of the gable portion of the wall.
- Neglect the front porch and the large, extruded window portion on top of it.

The timber elements are modelled as elastic frame elements assuming Totara for the timber species, while the pumice concrete elements are modelled as layered shell elements (more details about the materials used in the model are reported in the relative section of the previous chapter regarding the analysis approach). The timber framed structure is modelled with continuous horizontal members and interrupted vertical studs to simulate the frame arrangement observed during on-site investigations. In addition, to simulate the original connection between the horizontal members and the

studs, moment end-releases with a tension limit equal to 3.85 kN are applied to each stud. The tension limit value of 3.85 kN is set based on a literature reference which examines the maximum force that a round wooden dowel with a diameter of 10 mm is able to transfer in shear. The wooden dowel is the fastener element of the tenon-mortise joint between the timber studs and the horizontal members; the capacity of the system to withstand tensile loading is then assumed as equal to the minimum between the shear capacity of the connection and the tensile strength of the stud. The contribution of the pumice concrete infill panels to the lateral capacity of the timber framed wall (which otherwise should be considered as unstable) is simulated using gap link elements connecting the timber frame elements to the infill panel shell element (**Figure 8-2**). Gap links are chosen to simulate the “compression-only” interaction between the timber frame structure and the infill panels.

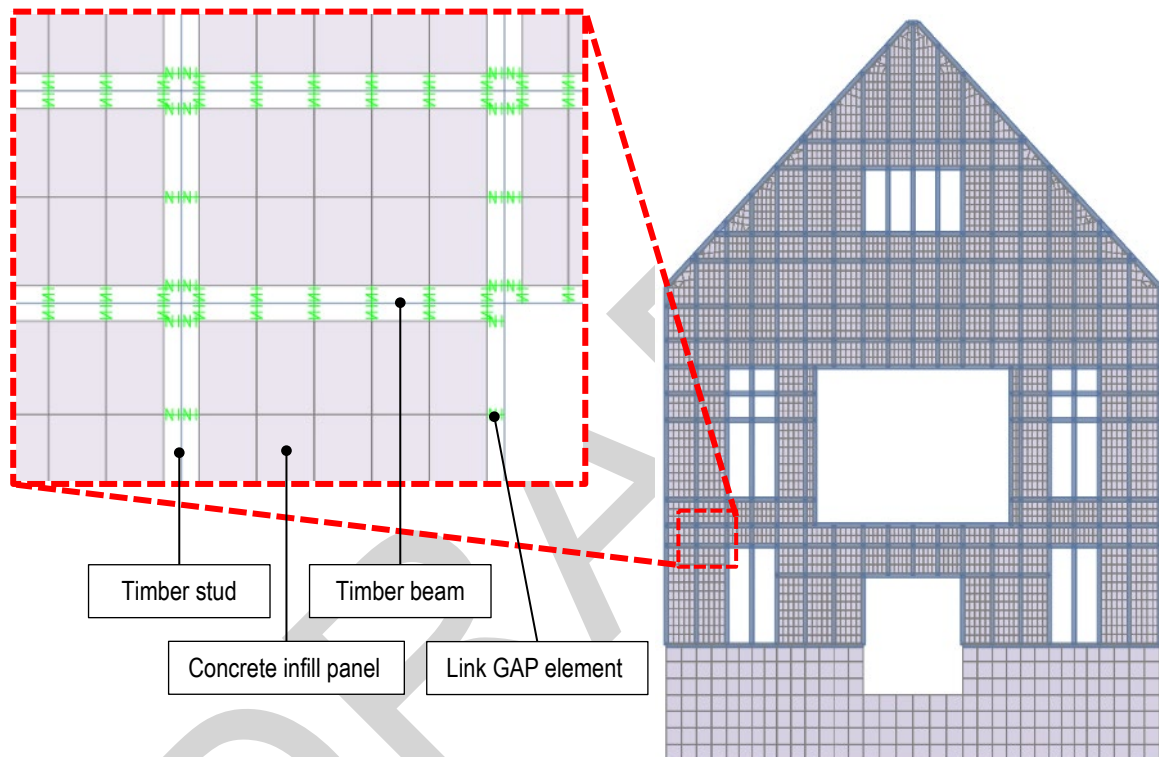


Figure 8-2: Zone 3 - West wall | Details of the numerical model adopted to study the lateral behaviour of the historic timber framed wall

Figure 8-3 summarises the outcomes of the pushover analyses performed to examine the lateral load-carrying capacity of the wall. Different loading patterns are assumed which range from uniform acceleration to a lateral load proportional to the first fundamental mode (similar to an inverted triangle distribution of forces). These analyses exhibit large differences in terms of global pushover curves and total base shear. However, it can be seen that adopting the appropriate value of effective mass for the calculation of the equivalent Single Degree Of Freedom (SDOF) entails almost identical results in terms of Acceleration Displacement Response Spectra (ADRS) curves. For the uniform acceleration cases the effective masses used for the calculation of the SDOF curves are the total masses of the examined structures (i.e., entire wall, nib wall + timber framed wall, and only timber framed wall). On the other hand, for the load pattern proportional to the first fundamental mode, the formulation proposed in Section C2.4.2 of NZSEE 2017 (MBIE-NZSEE 2017a) is used for the calculation of the effective mass. The large value of total base shear for the uniform acceleration case considering the entire wall is due to the loading pattern being proportional to the vertical mass distribution: the basement concrete wall and nib wall are loaded with higher forces and are able to contribute more to the total base shear because of their large thickness and stocky structure.

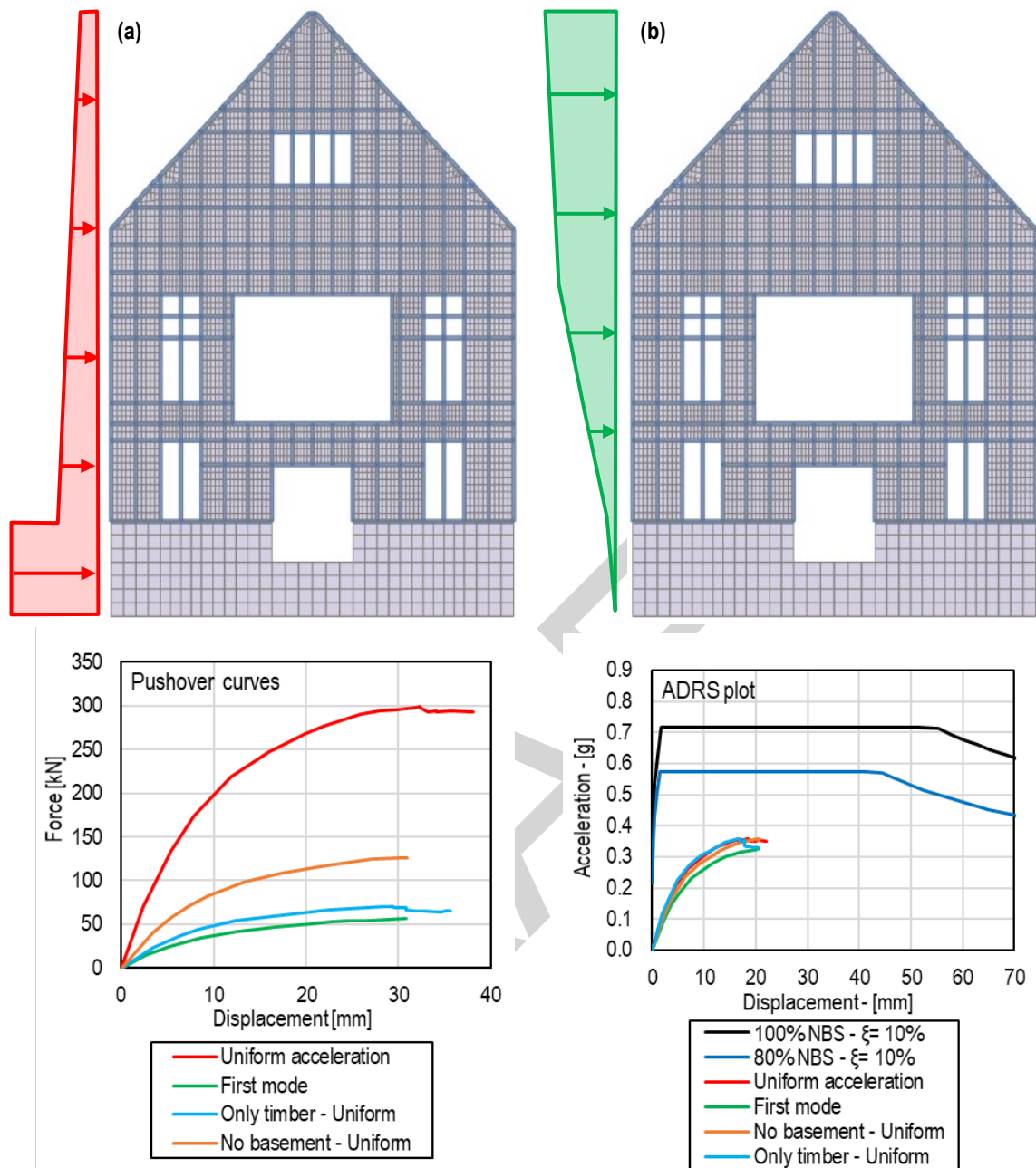


Figure 8-3: Zone 3 - West wall | Comparison of NLSPA loading pattern in terms of pushover curves and ADRS plots. Schematics of uniform acceleration loading pattern (a) and first mode loading pattern (b)

Figure 8-4 shows the principle tensile strain of the pumice concrete elements at the last step of the pushover analysis and the deformed shape of the structure for the uniform acceleration case (figures for the other load patterns are not reported due to being largely similar to Figure 8-4). The failure mode of the wall can be estimated as a rocking mechanism of the “piers” placed at ground floor and mezzanine floor level. The lateral load-carrying capacity of the wall is governed by the capacity of the connection between the timber studs and the horizontal members and by the (compression-only) interaction between the timber framed structure and the infill panels.

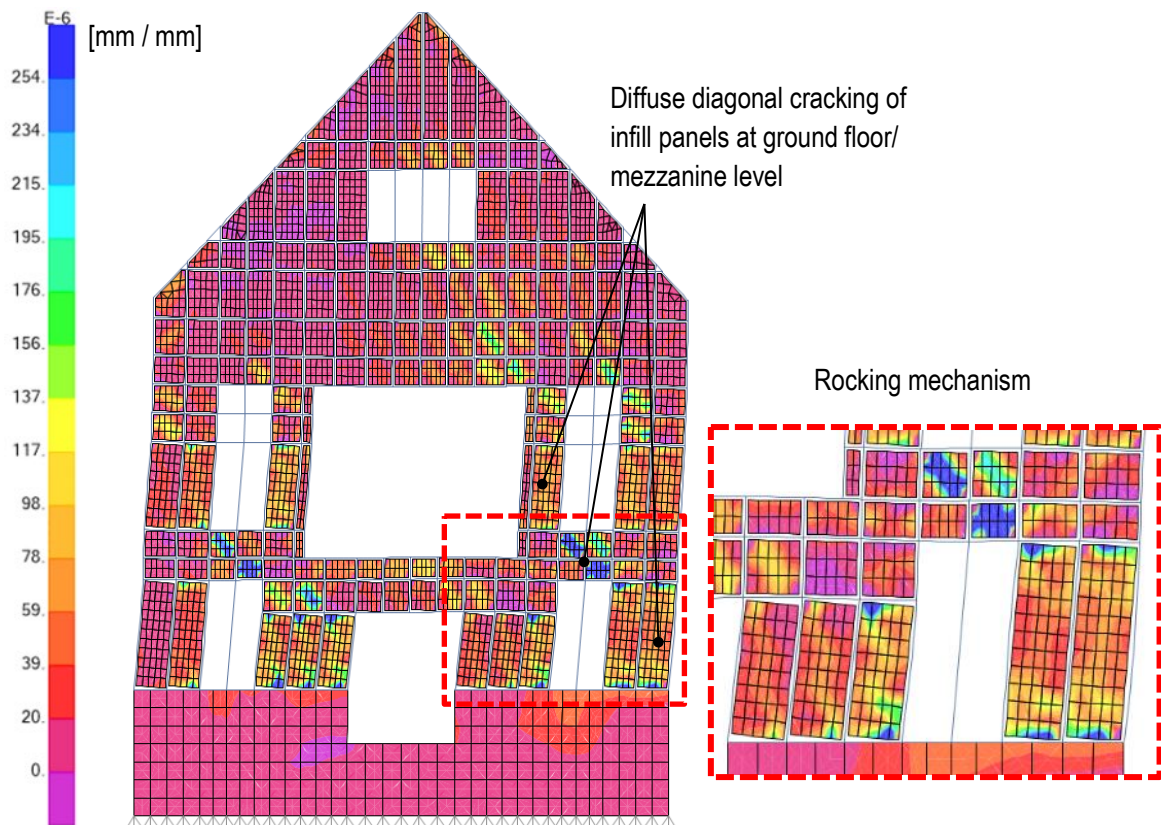


Figure 8-4: Zone 3 - West wall | Principle tensile strain on the pumice concrete material at the last step of analysis (uniform acceleration) | Deformed shape scale factor: 15

In addition to the detailed model (the numerical model simulating the concrete infill panels connected via the gap link elements), an ulterior, simplified model was defined (Figure 8-5). The aim of this additional model is to reduce the computation time needed for the calculation of the timber frame-to-infill panels interaction when examining the entire Zone 3 structure. The infill panels are modelled without gap link element connections and are instead directly attached to the frame elements of the timber framed wall. The in-plane shear transfer between the two elements is disregarded by defining in-plane shear edge release. The compression-only behaviour of the system is accounted for by calibrating the material properties of these new shell elements based on the outcomes of the detailed model.

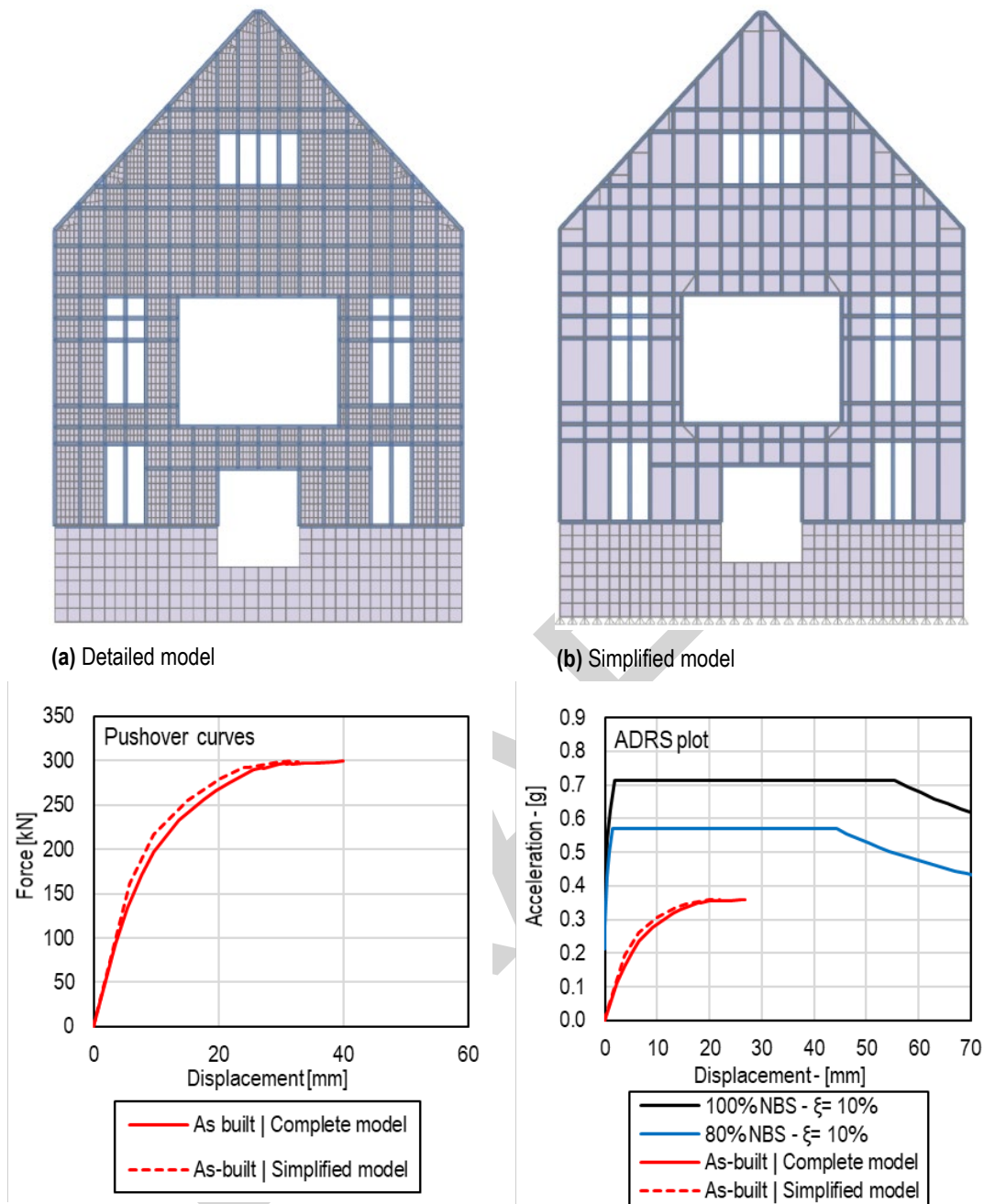


Figure 8-5: Zone 3 - West wall | Comparison of the detailed numerical model and of the calibrated simplified model in terms of pushover curves and ADRS plots (uniform acceleration loading pattern)

Finally, as the last step for the isolated wall analysis of the front façade wall, the results of the retrofit model are reported because the as-built structure is not able to reach the required level of seismic capacity (80% NBS). The proposed retrofit is based on the connection of Cross-Laminated Timber (CLT) panels to the existing timber framed structure, ensuring an increase in both in-plane and out-of-plane capacity. **Figure 8-6** shows the assumed panels arrangement, which has accounted for practical considerations such as the maximum panel dimension which can be handled inside the building. The connection of the CLT panels to the timber framed wall is modelled using multilinear elastic link elements (simulating the shear capacity of a single screw) placed at a centre distance of 250 mm. The base connection of the panels and the connection between the individual panels are simulated using multilinear elastic elements calibrated against typical commercial items.

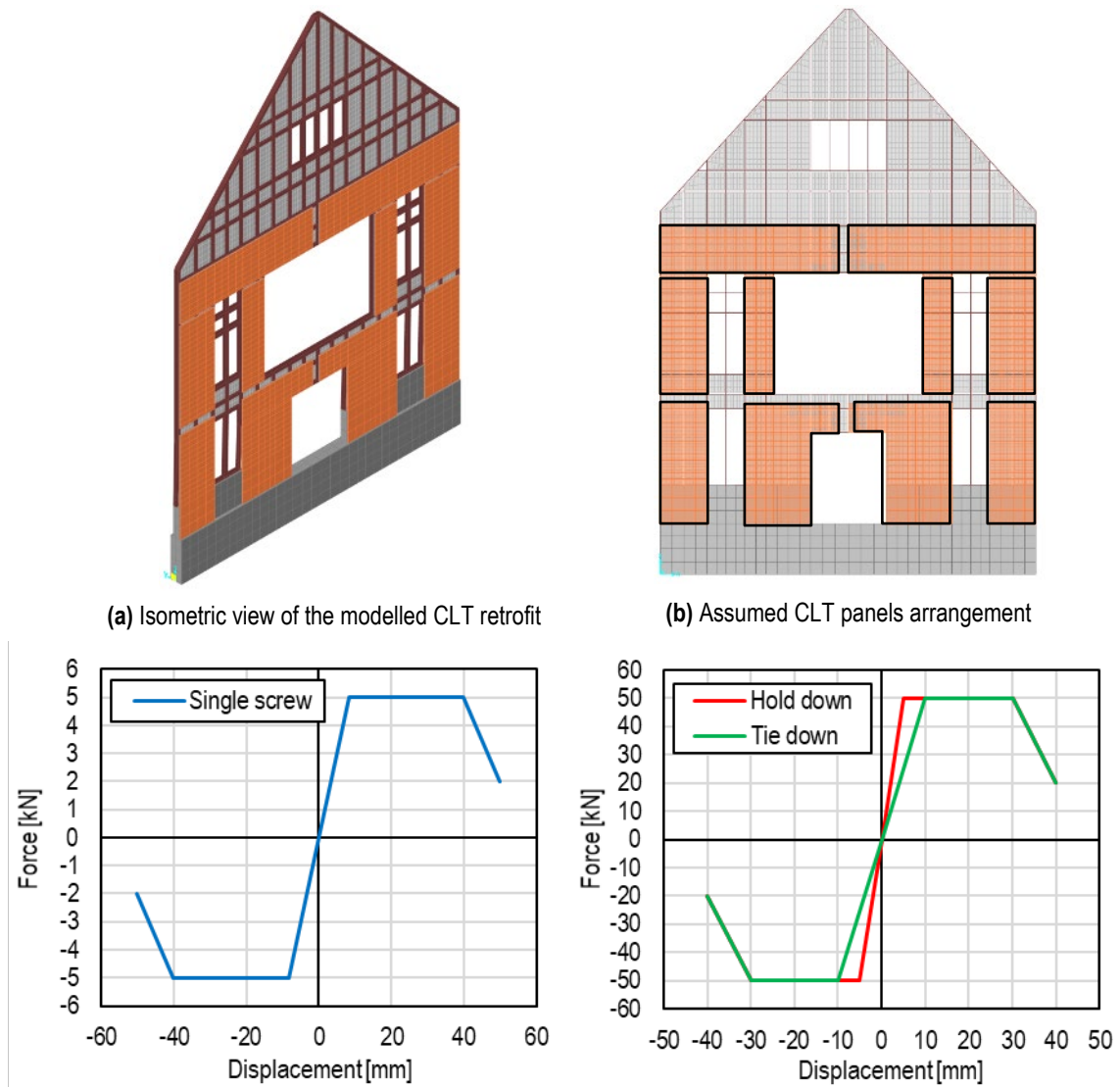


Figure 8-6: Zone 3 - West wall | CLT panels retrofit proposal and force-displacement curves assigned at the multilinear elastic links adopted to connect the CLT panels to the timber framed wall and to the base

As can be observed from [Figure 8-7](#), the CLT panels retrofit appears to be sufficient to ensure that the front façade wall is able to resist the seismic load as a self-standing structure. The extra lateral load capacity level highlighted by the graphs will be exploited when considering the Zone 3 entire model, with the front façade wall acting as a shear wall to restrain the out-of-plane movement of the large concrete gable walls and tributary load to the front façade wall. The retrofit model will then be complemented by strengthening of the studs-to-horizontal member connection and by the addition of plywood sheathing for the gable portion of the front façade wall.

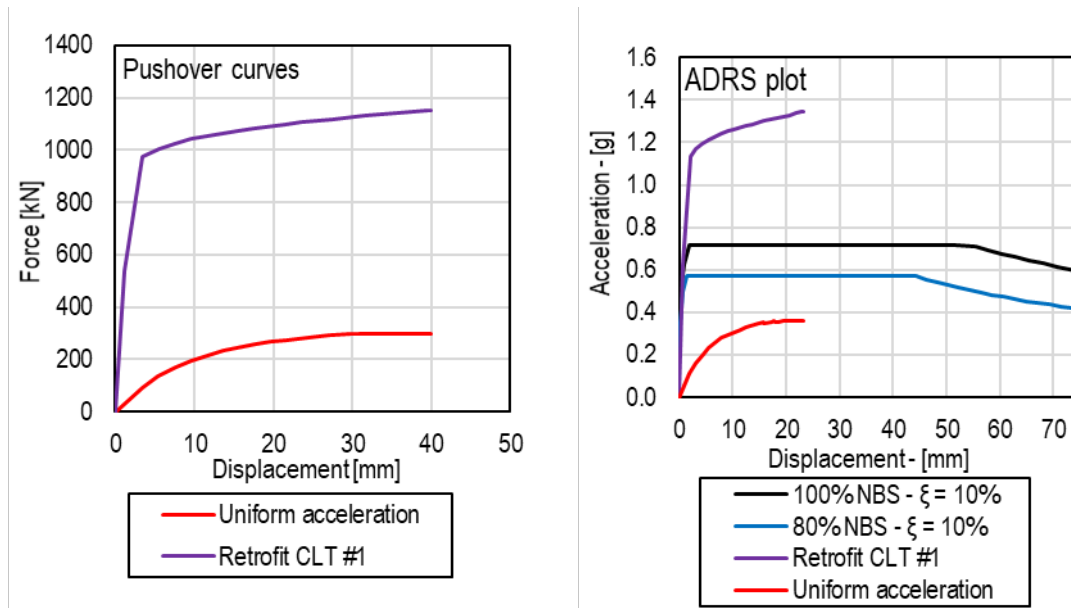


Figure 8-7: Zone 3 - West wall | Comparison of the as-built model and of the retrofit model in terms of pushover curves and ADRS plots (uniform acceleration loading pattern)

8.1.2 Z3 – North wall | Pumice concrete gable wall & timber framed wall

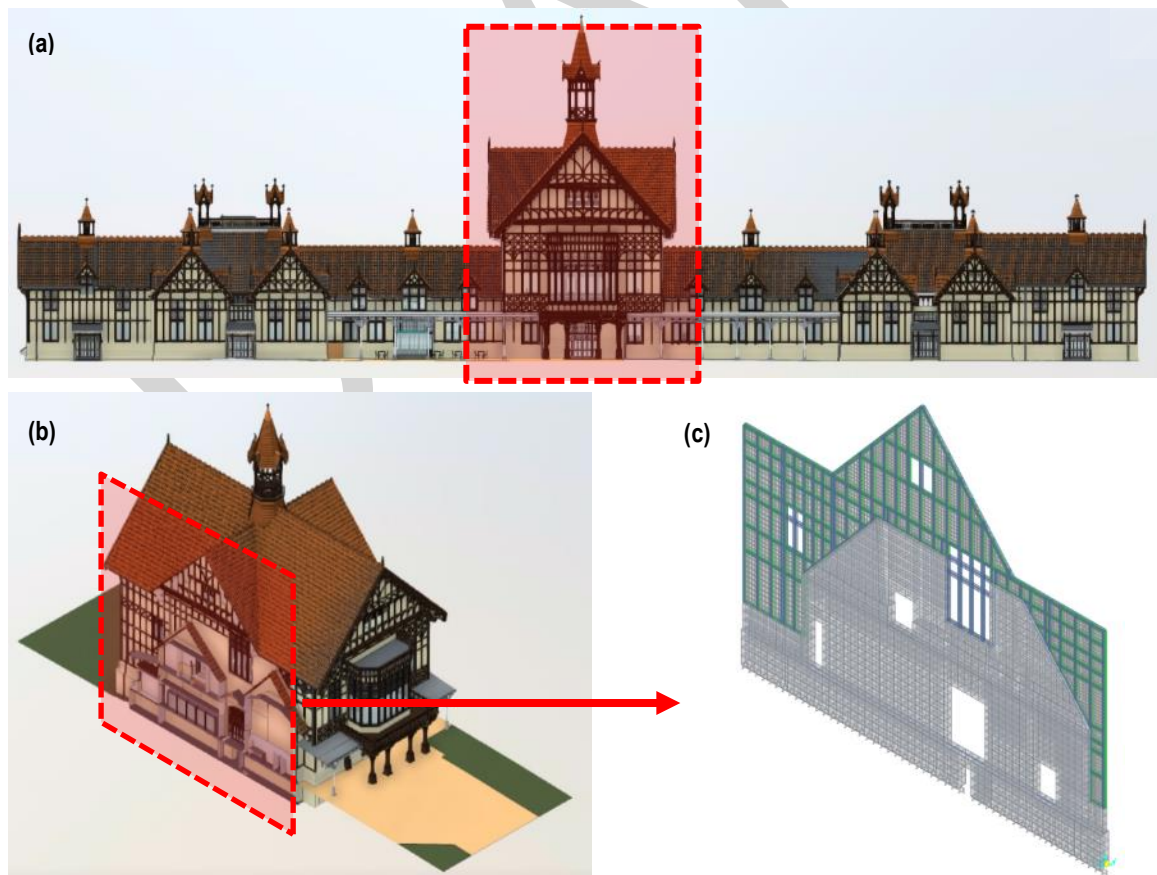


Figure 8-8: Front view of the Rotorua Museum (a) – Zone 3 (b) – FEM model geometry of North gable wall (c)

This section will report the results of the isolated wall analyses carried out for the north wall of Zone 3, made of a large pumice concrete wall at the base and a timber framed wall atop the concrete wall with a structure similar to that of the front façade wall (**Figure 8-8**). Different from the front wall study, the analyses reported here is done for both the in-plane and out-of-plane behaviour and the results will already account for the tributary mass which that is to be considered for the wall (**Figure 8-9**).

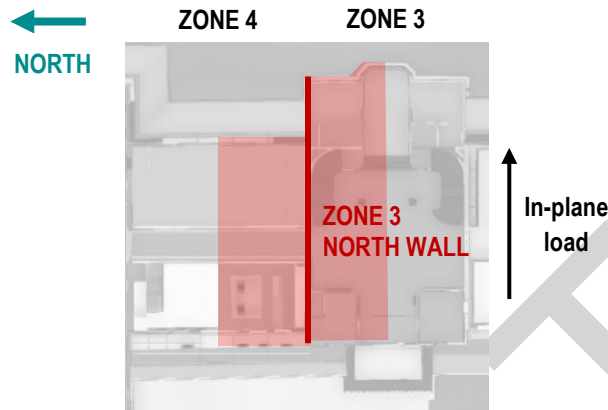


Figure 8-9: Zone 3 - North wall | Tributary mass area to obtain the ADRS plot for the North wall

Figure 8-10 shows the principle tensile strain of the concrete elements at the last analysis step, highlighting a failure mode related to the failure of the concrete infill panels in the row just above the concrete gable wall upper edge.

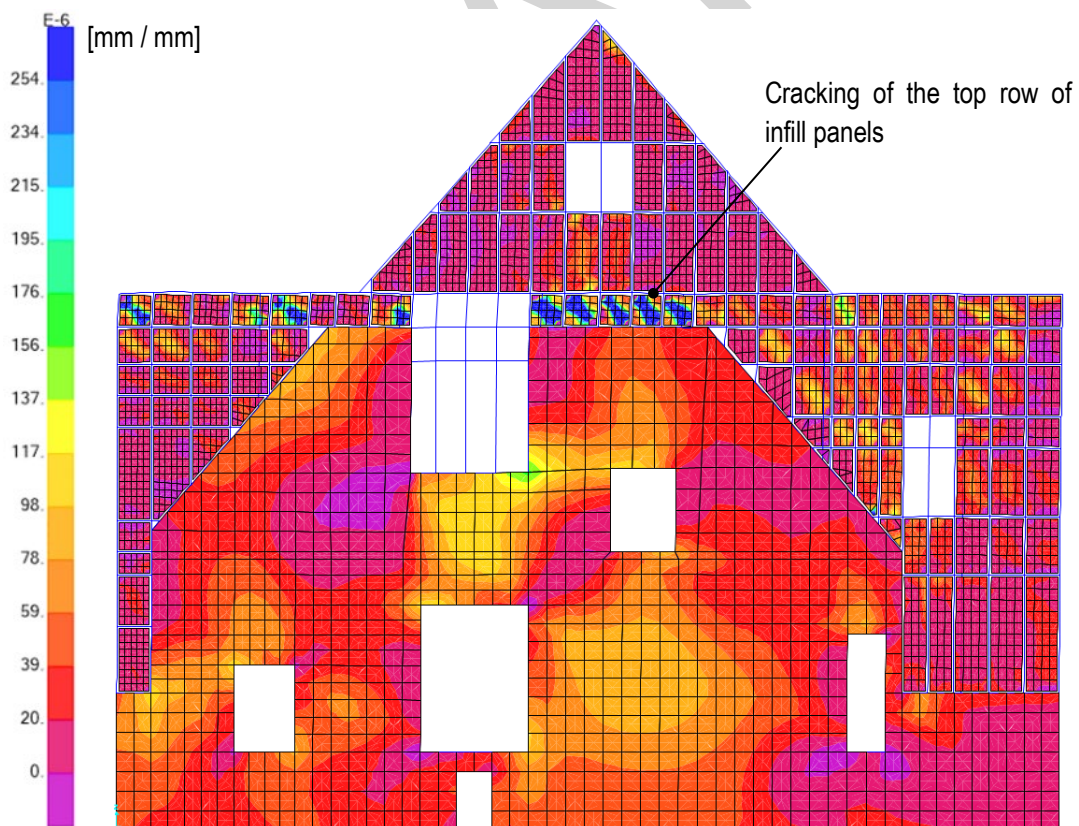


Figure 8-10: Zone 3 - North wall | Principle tensile strain on the pumice concrete material at the last step of analysis (uniform acceleration) | Deformed shape scale factor: 10

Figure 8-11 reports the comparison between the detailed model and simplified model.. For the rest of the analyses on the structure the simplified model was selected, which had a comparable value of maximum capacity and initial stiffness because the challenging irregular arrangement of infill panels avoided a perfect match between models.

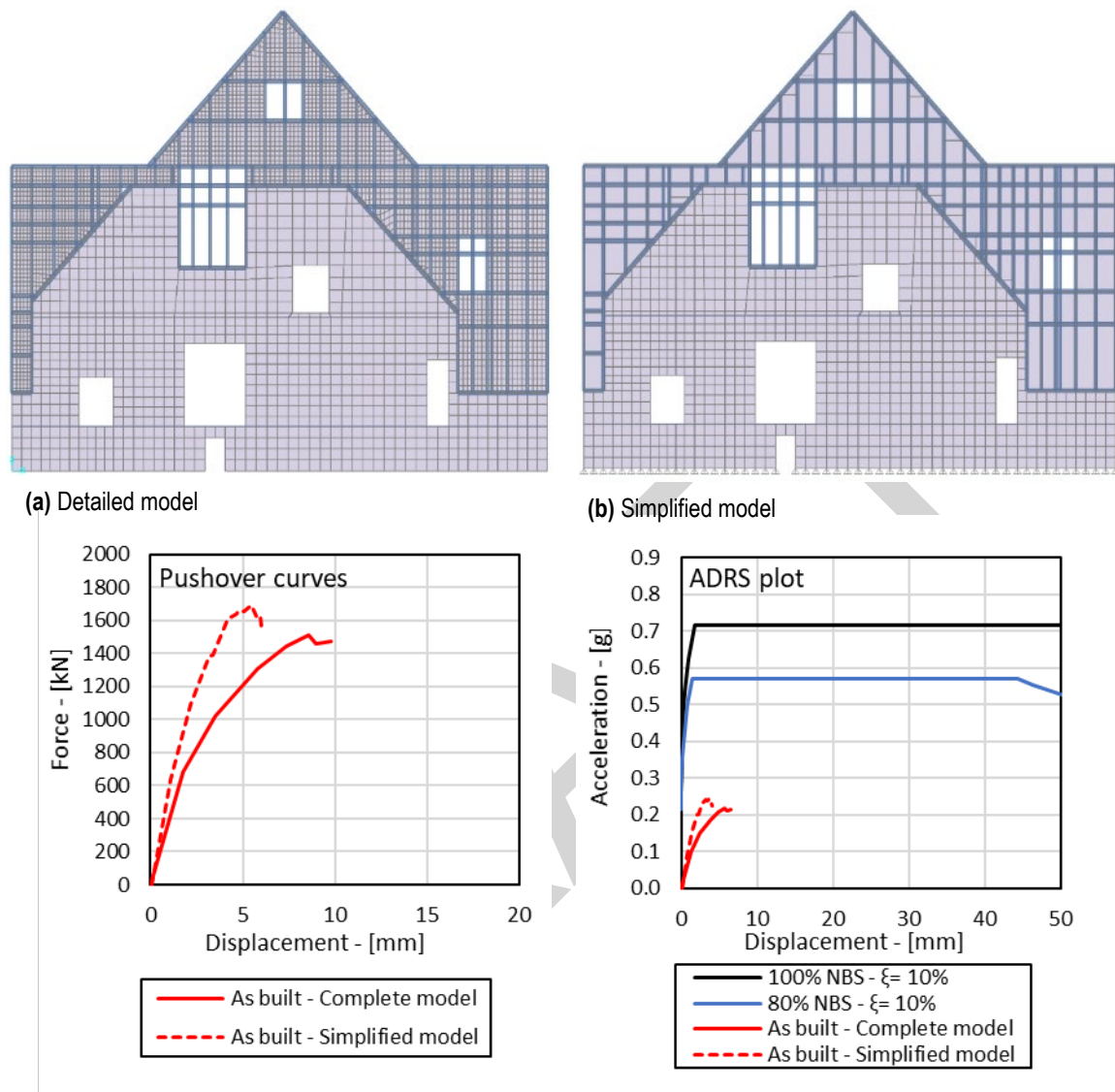


Figure 8-11: Zone 3 - North wall | Comparison of the detailed numerical model and of the calibrated simplified model in terms of pushover curves and ADRS plots (uniform acceleration loading pattern)

Finally, the out-of-plane (OOP) behaviour of the isolated wall was studied by considering for the sake of clarity only the concrete base structure (the upper portion of the wall and the tributary mass are nevertheless accounted for as applied vertical loads). **Figure 8-12** displays the scheme adopted for the analyses and the principle tensile strain at the yielding point of the pushover curve showing the failure mechanism of the cantilevered wall due to the formation of the ground floor hinge. **Figure 8-13** reports the outcomes of the OOP analyses for both the as-built and retrofitted model (see *Section 8.2.2 8.2.2Z3 | Retrofit solution* for more details about the proposed strengthening intervention) considering two boundary values for the concrete tensile strength of 1.0 and 0.1 MPa. In both cases the FRCM coating entails a significant increase in terms of ultimate OOP strength of the wall. The required level of seismic capacity (80% NBS) is not reached even for the retrofitted model, although it should be considered that the reported analyses neglect the contribution of the mezzanine floor, roof diaphragm, and of the return shear walls, which largely contribute to the concrete structure capacity. This issue will be addressed in *Section 8.2.2 Z3 | Retrofit solution* when examining the pushover analyses of the Zone 3 entire structure and in *Section 12 Zone 2-3-4-5 | Nonlinear Static Analyses* when reporting the outcomes of the pushover analyses carried out on the model combining all the original parts of the building.

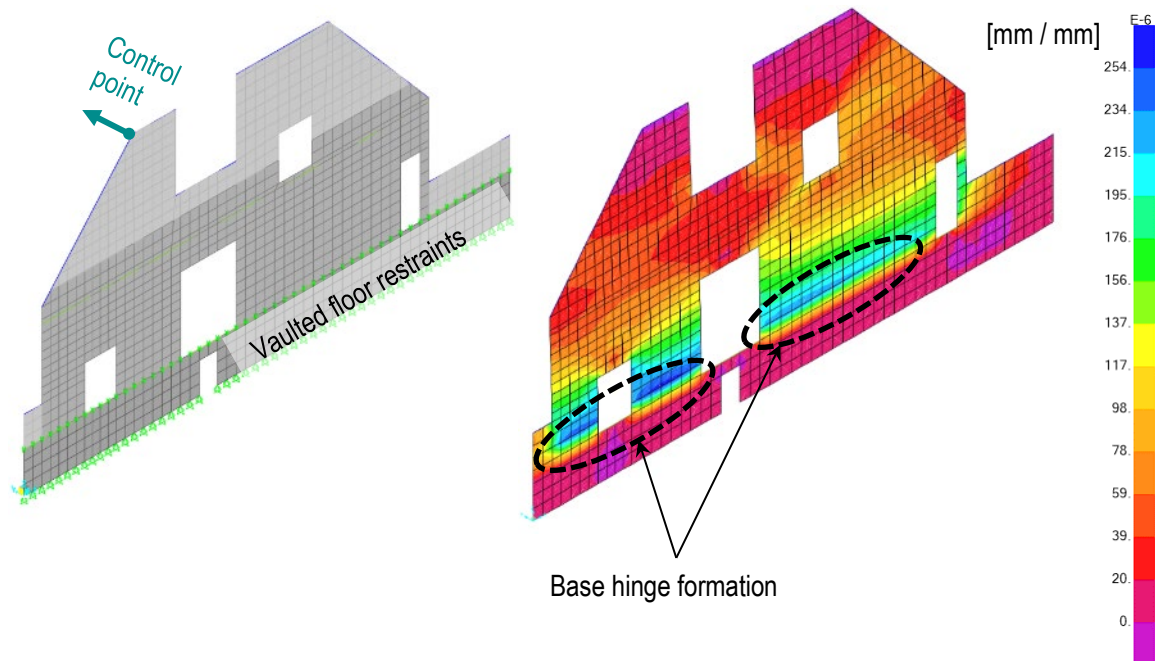


Figure 8-12: Zone 3 - North wall | Out-of-plane (OOP) analysis of the pumice concrete structure considered as a self-standing structure and principle tensile strain at the yielding point of the pushover curve

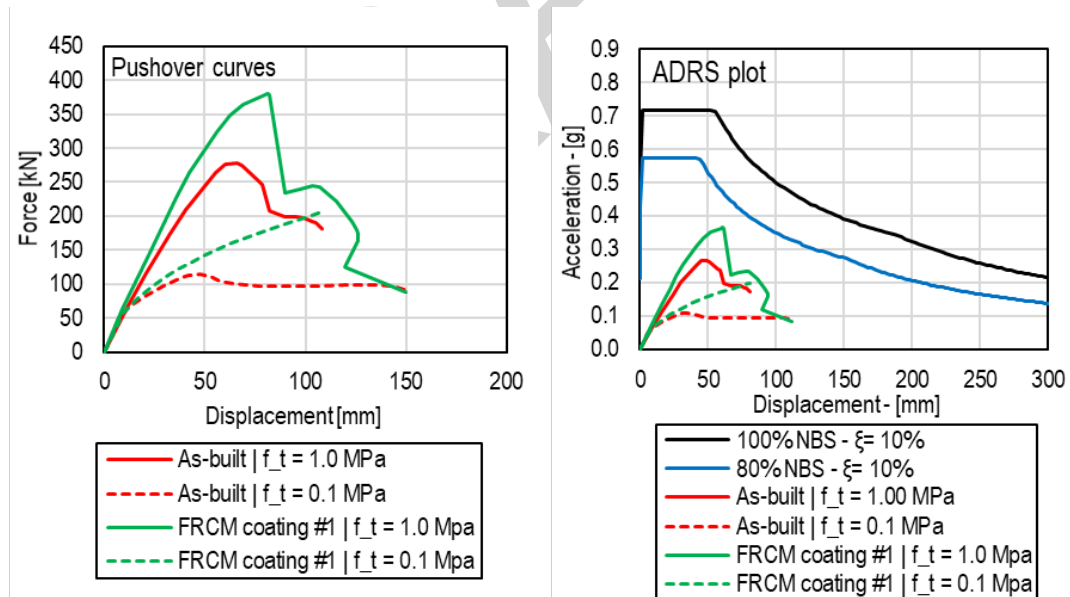


Figure 8-13: Zone 3 - North wall | Out of plane analysis results | Load pattern: Uniform acceleration

8.1.3 Z3 – Mezzanine floor | In-plane analyses

The mezzanine floor plays a significant role in the lateral capacity of the Atrium structure as it is the only horizontal structure that connects the vertical lateral resisting elements between the ground floor and roof levels (**Figure 8-14**).

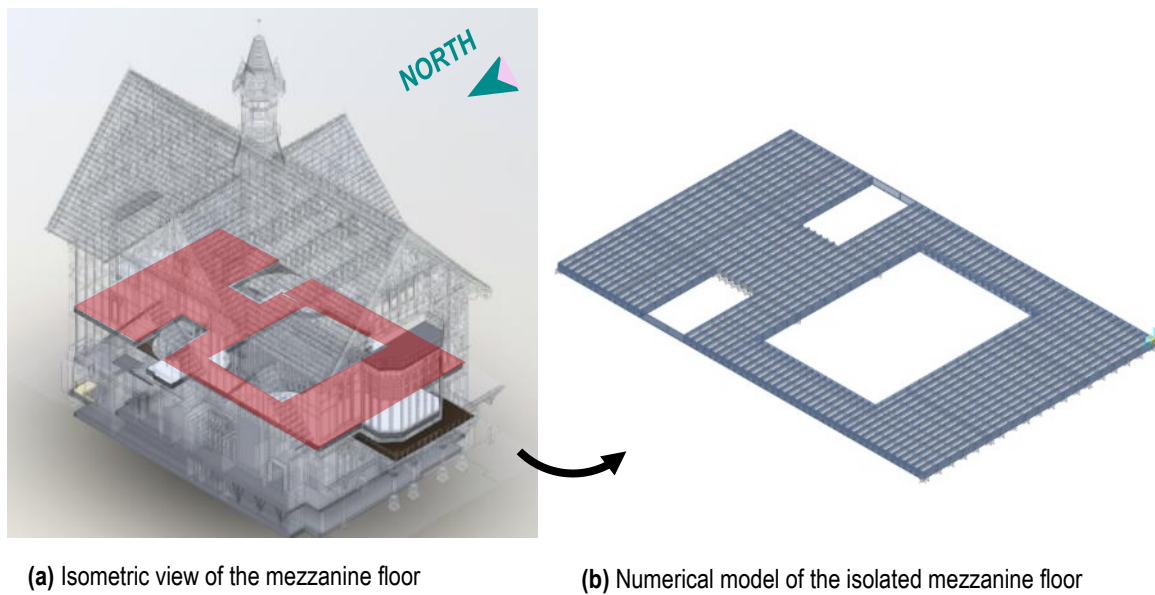


Figure 8-14: Zone 3 – Mezzanine floor | (a) Isometric view of the mezzanine floor taken from the BIM model (b) Numerical model of the isolated mezzanine floor

In the as-built condition, the diaphragm action is ensured only by the original timber boards of the floor fastened to the joists. The large central opening and the two stair openings greatly reduce the diaphragm's in-plane stiffness and capacity, thus requiring a specific retrofit design to improve diaphragm capacity. In-plane pushover analyses were carried out assuming the isolated floor structure is restrained at the western and eastern edges, and two loading patterns were adopted: a load uniformly distributed along the diaphragm span and a parabolic shaped load according to FEMA-356 (Figure 8-15).

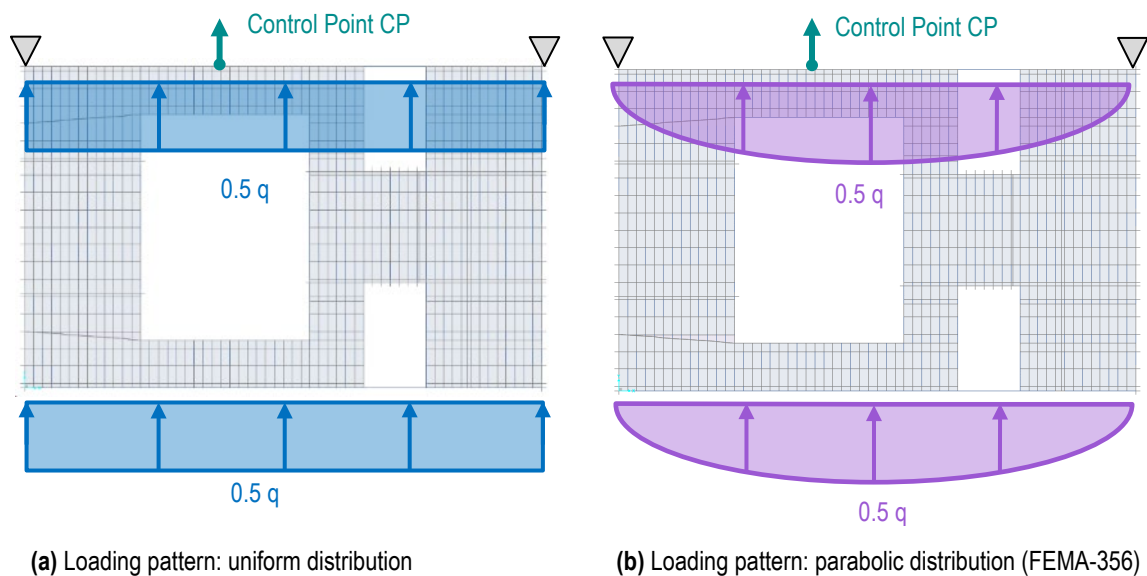


Figure 8-15: Zone 3 – Mezzanine floor | Loading pattern used for the analyses: (a) uniform distribution (b) parabolic distribution (according to FEMA-356)

The analyses were carried out in both the as-built and the retrofitted conditions. The retrofit consists of two systems: a “bottom” reinforcement made of diagonal steel strips and orthogonal timber blocking elements connected to the joists using nails, and a layer of 21 mm thick plywood panels on top of the steel strips and timber blocking (Figure 8-16).

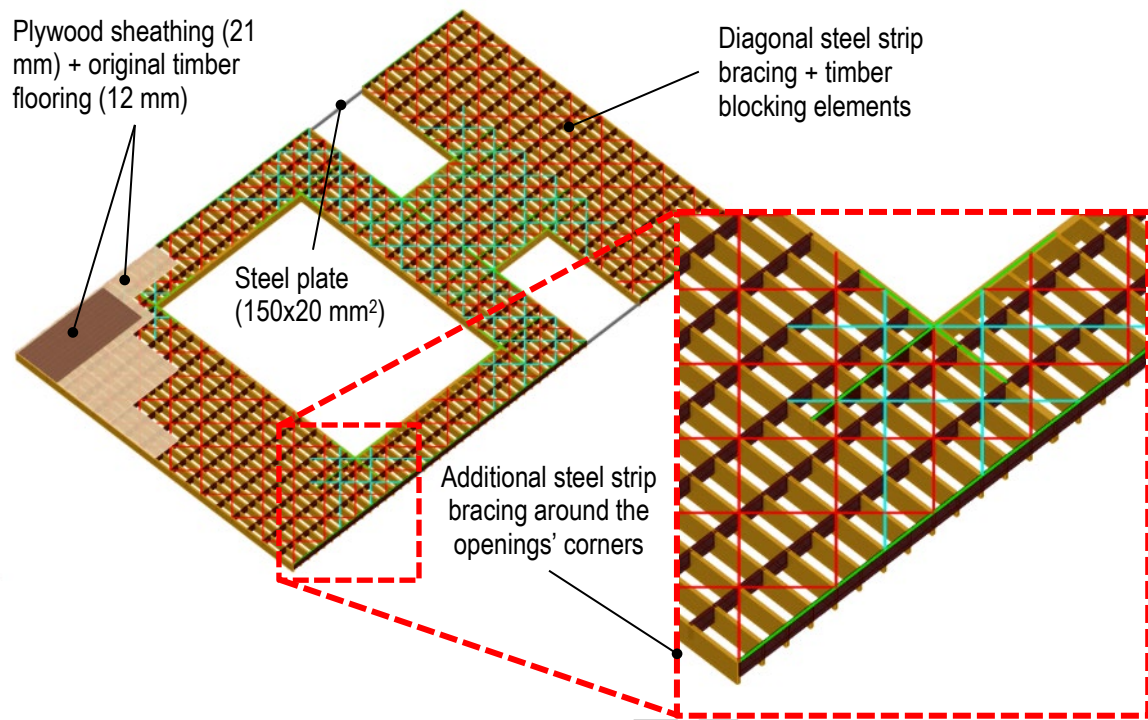


Figure 8-16: Zone 3 – Mezzanine floor | Schematic of the retrofit work for the mezzanine floor structure

Figure 8-17 reports the outcomes of the analyses done for the as-built and the retrofit model, where a considerable increase of in-plane stiffness and capacity is observed as a result of the proposed strengthening. An additional model is defined which simplifies the effect of the various strengthening elements into an equivalent shell element with a shear stiffness equal to 3000 kN/m. This calibrated shell element will then be adopted for the analyses of the entire Zone 3 structure.

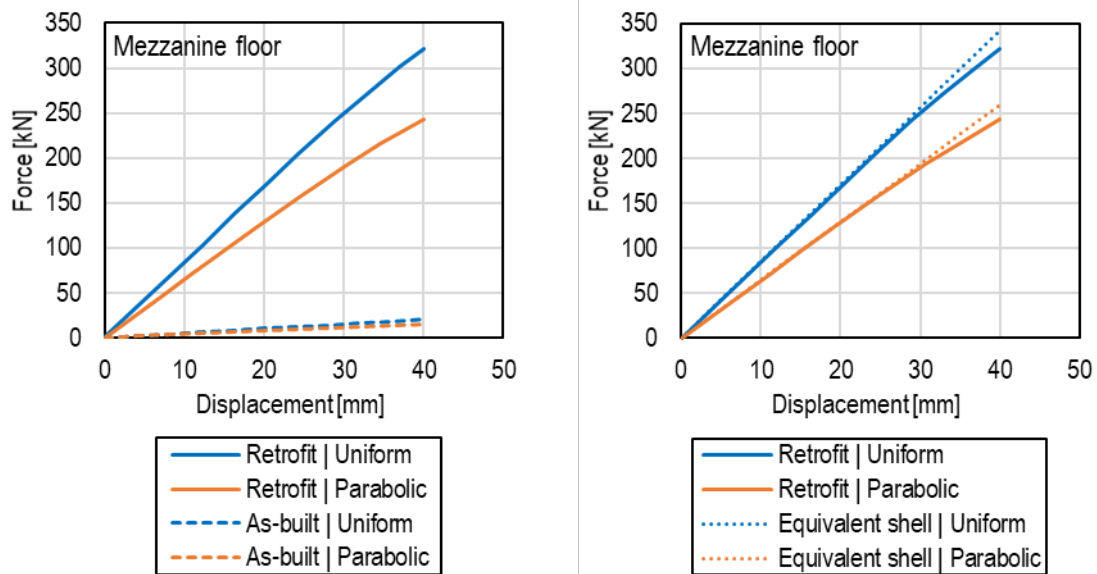


Figure 8-17: Zone 3 – Mezzanine floor | Pushover curves comparison of the: (a) retrofit and as-built configuration - detailed model (b) retrofit configuration – detailed and equivalent shell model

8.2 Zone 3 | Entire structure analyses

This section reports the outcomes of the nonlinear static pushover analyses performed on the entire Atrium structure (Zone 3). For the sake of simplicity, the turret, which rises from the central section of the roof, is neglected in the numerical model and is accounted merely as additional mass. For the lateral loading pattern the uniform acceleration option was selected. Because the fundamental mode of vibration exhibits a low level of modal mass participating ratio ($< 30\%$), the lateral loading pattern proportional to the first mode was not used. As an alternative, another model was defined which eliminates the basement structure (labelled as “No basement model”, [Figure 8-18](#)). The no basement model was examined using the uniform acceleration load pattern. This is done to provide additional insight into the behaviour of the structure without the influence of the basement structure, which may be large due to the loading condition being proportional to the mass distribution.

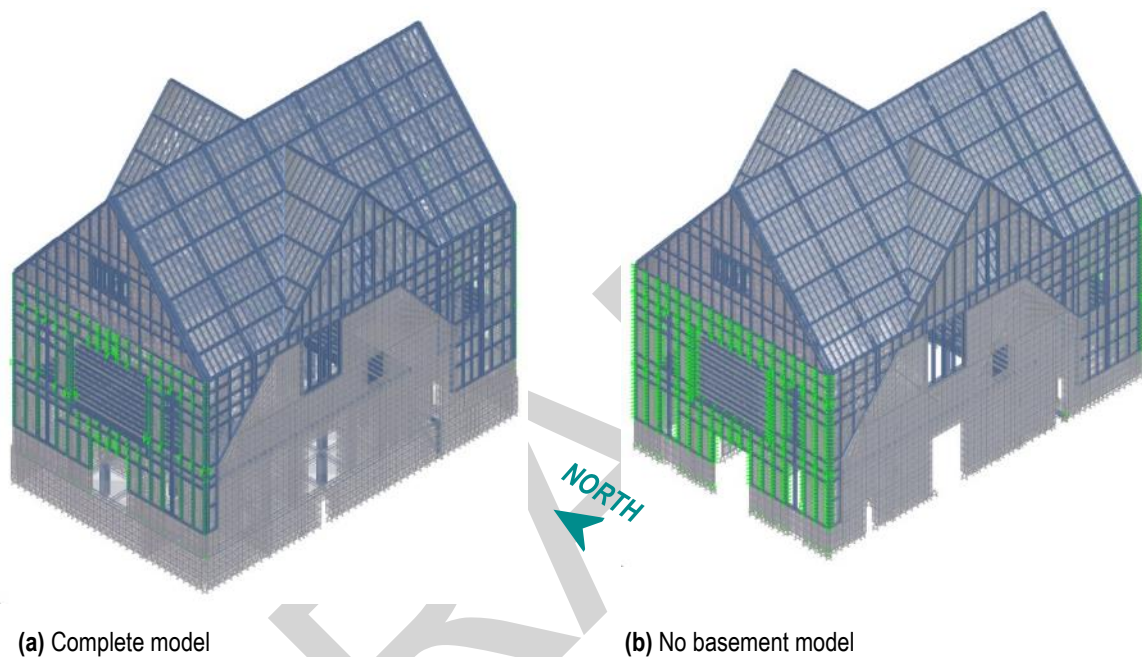


Figure 8-18: Numerical models used for the analysis of the whole structure: (a) Complete model (b) No basement model

[Figure 8-19](#) shows the tributary mass areas adopted to obtain the SDOF curves in the ADRS plots. In the X direction the North wall tributary area comprises half of Zone 3 and half of Zone 4 due to the direct connection of these two zones. On the other side, the South wall is loaded only by half of Zone 3 because of the presence of the parallel steel frame in Zone 2 which relieves the South wall from taking on lateral load from Zone 2. For loads oriented in the Y direction, the mass tributary area includes only Zone 3, adequately divided between the three main lateral load-carrying systems in that direction (West and East wall, and central spine wall near the stairs).



Figure 8-19: Zone 3 - Complete structure | Tributary mass areas used to define the ADRS plot | Loading direction along the X and Y direction

Figure 8-20 reports the control points selected to display the pushover analyses outcomes. The displacements are taken at the highest portion of the building and are spread across the footprint of the structure to account for any possible irregular behaviour of the structure during the interpretation of the results. The labels W, C, and E stands for West, Central, and East; for the sake of clarity in the pushover curves and ADRS plots charts, the mean displacement values of each selected group of control points are represented. The same control points are used to represent the outcomes of the as-built model and of the retrofit model.

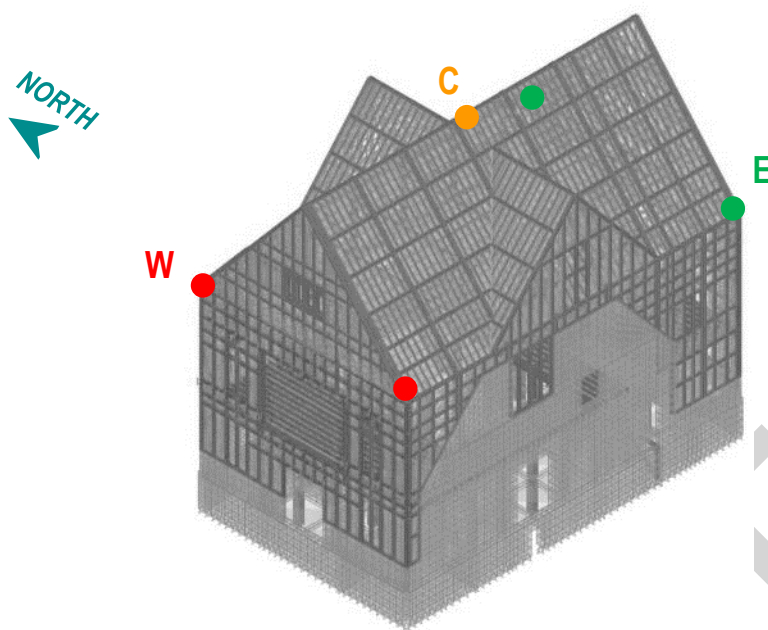


Figure 8-20: Zone 3 - Complete structure | Control points adopted for the nonlinear static pushover analyses

8.2.1 Z3 | As-built condition

This section presents the outcomes of the as-built model nonlinear static pushover analyses. These results are used to locate the main area of weakness of the original structure to design an appropriate retrofit intervention. **Figure 8-21** and **Figure 8-22** report the outcomes of the analyses for the +X and +Y loading directions in terms of both pushover curves and ADRS plots. Furthermore, the deformed shapes obtained at the last step of the analysis are reported for a quick visual assessment of the stiffness (relative to lateral loads) of different parts of the building.

In both of the two loading directions the as-built structure does not achieve the required seismic performance level. In detail, the east wall appears to be more deformable with respect to the west wall (front façade of the building, **Figure 8-21**), likely due to the larger openings present at the rear of the Atrium structure. The relatively low in-plane stiffness of the existing roof diaphragm results in larger deformation of the central portion of the roof for loads directed in the X direction (**Figure 8-22**). For loads acting perpendicular to the two main concrete gable walls (i.e., Y direction), the larger deformation experienced by the upper central portion of the building is likely due to the lack of intermediate shear walls spanning the entire height of the building. A last observation drawn from the as-built model outcomes regards the out-of-plane behaviour of the west and east façades of the structure (**Figure 8-22**). The bottom part of the gable section may be subjected to large displacement demand during an earthquake due to its slenderness and the absence of diaphragm elements close to it. The retrofit for the West and East walls should then incorporate strengthening for both the in-plane and out-of-plane directions.

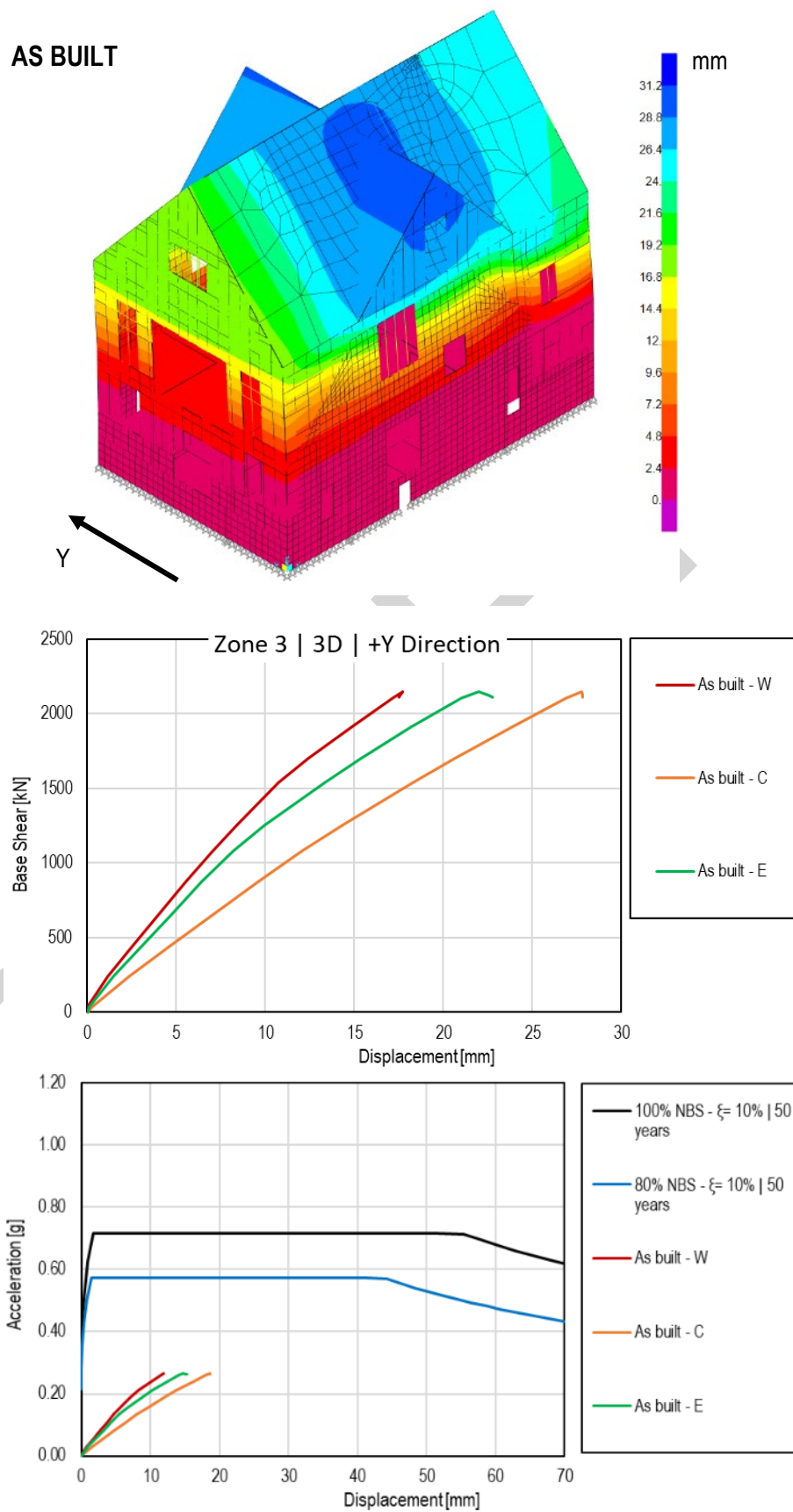


Figure 8-21: Zone 3 - Complete structure | As-built model deformed shape at the last analysis step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

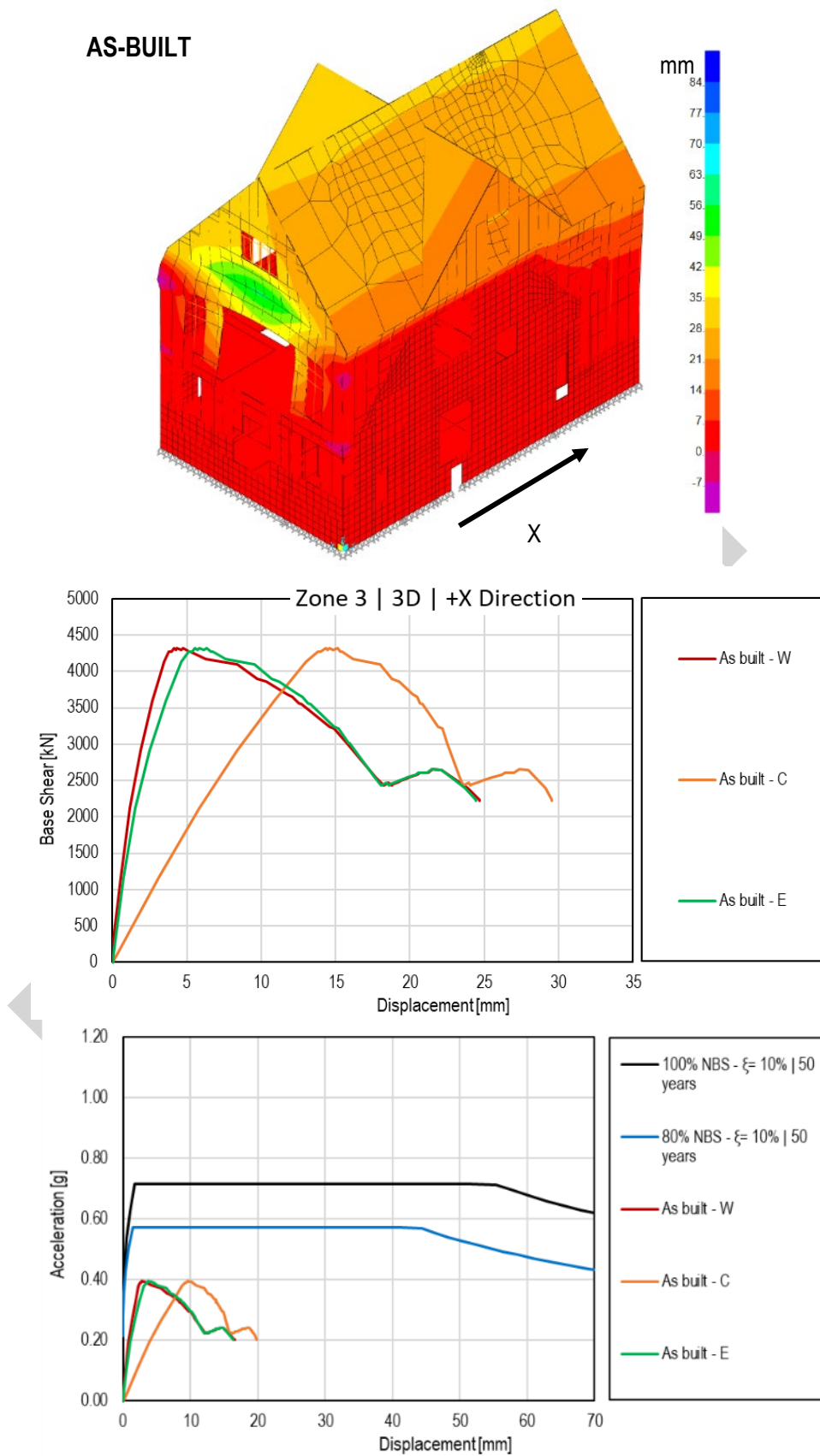


Figure 8-22: Zone 3 - Complete structure | As-built model deformed shape at the last analysis step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

8.2.2 Z3 | Retrofit solution

This section presents the outcomes of the retrofit model nonlinear static pushover analyses. **Figure 8-23** shows a summary of all the strengthening works implemented in the numerical model to increase the lateral capacity of the structure to at least a seismic load level equivalent to 80% NBS.

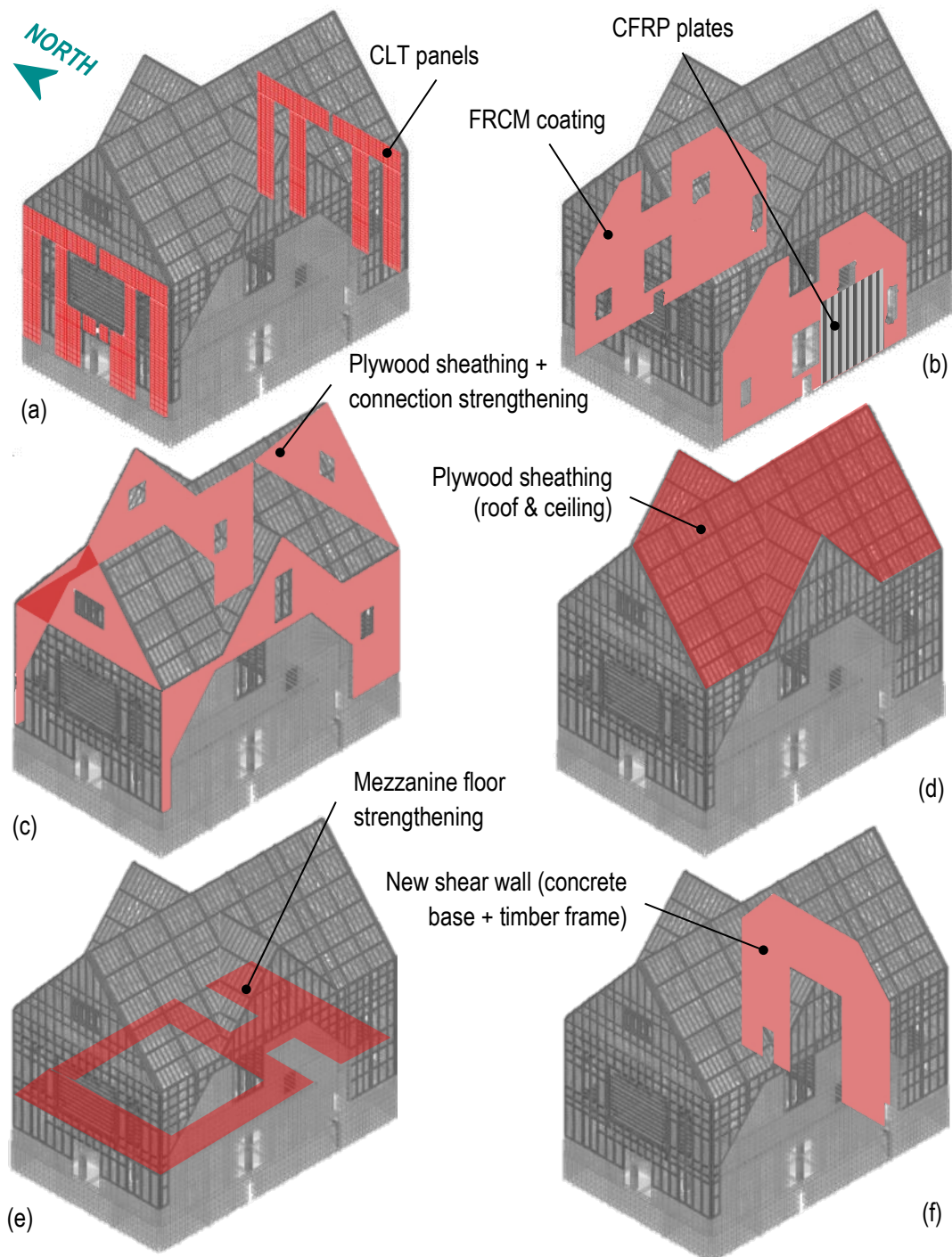


Figure 8-23: Zone 3 - Complete structure | Retrofit works implemented in the numerical model (structural elements contributing to the lateral capacity of the entire structure)

These strengthening works comprise:

- CLT panels (**Figure 8-23a**) connected to the timber frame of the West and East walls using screw fasteners with a spacing of 250 mm. Each CLT panel is then connected with the adjacent panel(s); the base panels are considered properly anchored at the ground floor level.
- FRCM coating (**Figure 8-23b**) applied on both sides of the pumice concrete gable walls. For a section of the southern face of the South wall, the FRCM coating is replaced by a series of vertically placed Carbon-Fibre-Reinforced-Polymers (CFRP) plates (150 mm wide and spaced at a centre distance of 1.6 m).
- The timber framed wall parts of the structure not addressed by the CLT retrofit, are strengthened by a plywood sheathing (**Figure 8-23c**). The connection between the vertical studs and horizontal members of the original timber framed wall is also strengthened by nailed steel strips to increase the tension limit of the frame elements from the original 3.85 kN to 10 kN. Similarly, the connection between the horizontal members and the diagonal elements at the edge of the concrete gable walls is reinforced by nailed steel strips (tension limit = 20 kN). Lastly, the connection of the bottom timber frame member with the underlying pumice concrete wall is reinforced via grouted steel bars (20 mm diameter and 200 mm embedment depth) inserted and spaced at 400 mm centres.
- The roof pitches and the ceiling platform on top of the hammer-beam trusses are also strengthened with a plywood overlay (**Figure 8-23d**). This is done to stiffen and strengthen the top diaphragm structure, improving the effectiveness of its lateral load distributing function.
- The mezzanine floor is reinforced by a combination of diagonal nailed steel strips and plywood sheathing (**Figure 8-23e**). The behaviour of the diaphragm is evaluated via the isolated floor detailed model previously described; for the global model a shell element with an equivalent in-plane stiffness is defined and used for the analyses.
- Finally, a new shear wall in the direction perpendicular to the two heavy pumice concrete walls is introduced in the retrofit model (**Figure 8-23f**). For its definition the presence of an already existing wall next to the two mezzanine floor stair wells has been used. The ground floor structure of the wall is composed on the south side by an original pumice concrete wall strengthened by a FRCM coating, and on the north side by a new built Reinforced Concrete (RC) wall. The wall structure above the mezzanine floor level consists of a timber frame assembly reinforced by a plywood overlay for in-plane loading conditions.

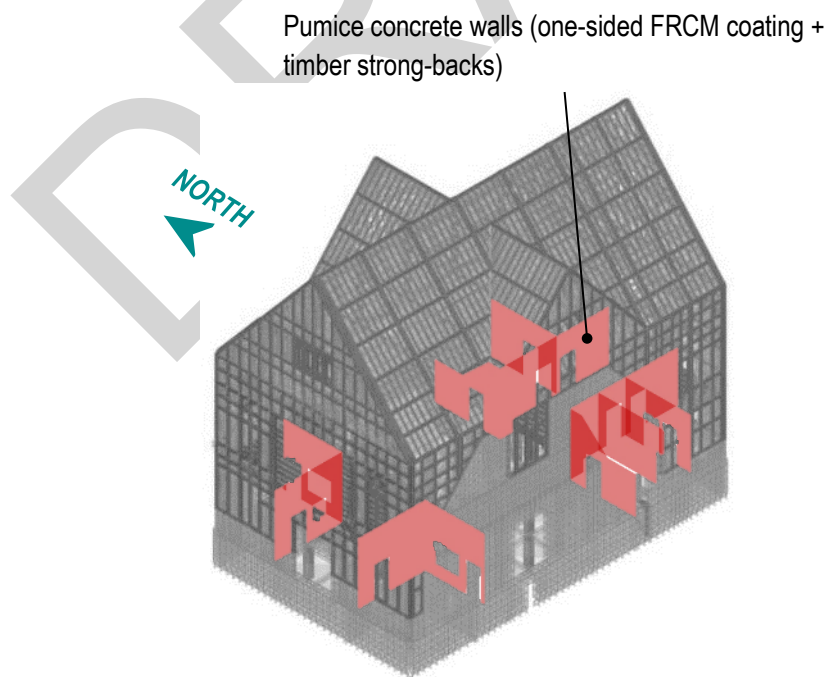


Figure 8-24: Zone 3 - Complete structure | Proposed retrofit for the ground floor concrete walls (secondary structural elements)

Figure 8-24 shows the scheme of the retrofit for the internal ground floor walls. These walls are considered as secondary structural elements because their contribution to the lateral capacity of the building is regarded as negligible with respect to the main lateral resisting structures. Nevertheless, their primary function is to support the mezzanine floor structure; hence, a retrofit work for both the in-plane (one-sided FRCM coating) and the out-of-plane (timber strong-backs) is proposed which aims to reduce the damage that may arise from earthquake induced drifts. For the pushover analyses of the retrofitted structure 3 cases are considered (all for the “no basement” model):

- Nonlinear behaviour of the ground floor secondary concrete walls.
- Linear behaviour of the ground floor secondary concrete walls.
- Ground floor secondary concrete walls removed from the model and substituted by roller connection at the mezzanine floor level.

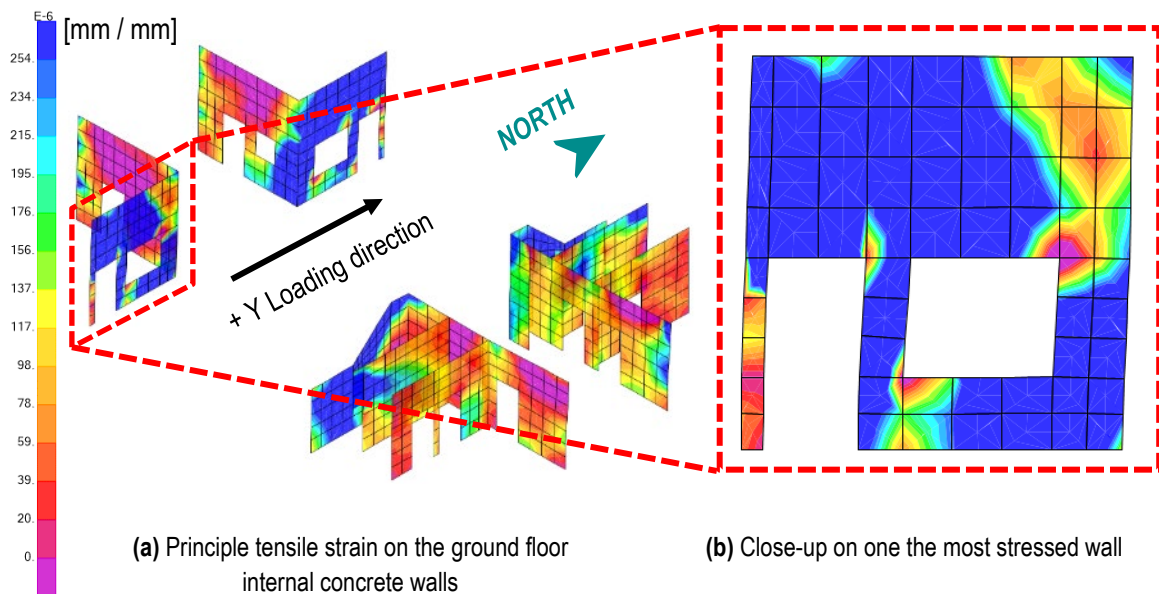


Figure 8-25: Zone 3 – Ground floor internal concrete walls | Principal tensile strain level at the performance point step of analysis (80% NBS) for the model considering the nonlinear behaviour of the internal walls (retrofit case | one-sided FRCM coating)

Figure 8-25 shows the principal tensile strain pattern on the ground floor internal concrete walls taken at the performance point step of the analysis in the +Y direction. The maximum colour scale level is set equal to the strain at the maximum tensile stress resistance assumed for the concrete material (i.e., 1.0 MPa). The level of damage expected on the walls appears to be high. However, it should be noted that this model of the isolated Zone 3 structure does not account for the contribution of the other zones (especially Zone 4) which is likely to considerably reduce the force demand on the ground floor walls.

Figure 8-26 to **Figure 8-29** report the outcomes of the nonlinear static analyses carried out on the retrofit model in terms of both pushover curves and ADRS plots. Furthermore, the deformed shapes obtained at the performance point (80%NBS) step of the analyses are reported for a quick visual assessment of the building behaviour subjected to lateral load. The graphs reported in **Figure 8-26** to **Figure 8-29** display the pushover analyses results of the model assuming a linear behaviour for the ground floor concrete walls. The nonlinear behaviour model used to extract the principle tensile strain shown in **Figure 8-25** exhibited an analysis convergence issue right after achieving a force level equivalent to the required seismic demand (80% NBS). The linear model was then used to show that the rest of the structure is able to withstand forces greater than the 80% NBS limit (again, note that the ground floor internal walls will be less stressed once considering the entire building model, i.e. Zones 2-3-4-5 together). In addition, **Figure 8-26**, **Figure 8-27**, and **Figure 8-28** report the outcomes of a supplementary model, run without the presence of the internal walls. Unsurprisingly, the model exhibits a larger deformability, but still reaches the required seismic capacity level. From the

analyses carried out in the X direction (parallel to the two main concrete gable walls) it is evident that the plywood strengthening of the timber framed wall sections addresses the infill panels failure that was highlighted in the as-built model. Regarding the out-of-plane (OOP) deformation of the West and East wall gables, it can be observed that the compound action of the existing timber frame and the applied CLT panel reinforcement induces a considerable increase in both OOP stiffness and capacity.

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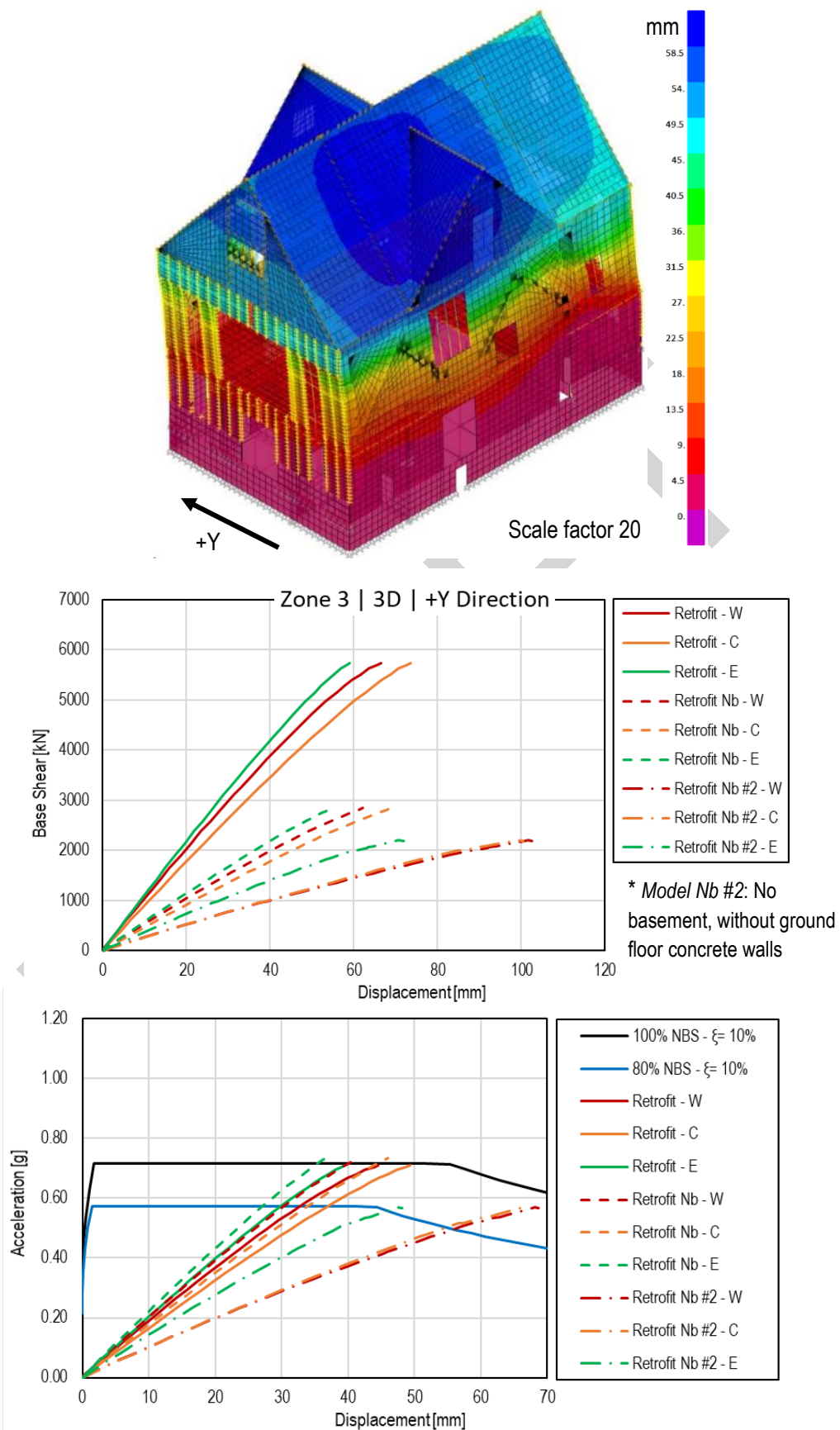


Figure 8-26: Zone 3 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

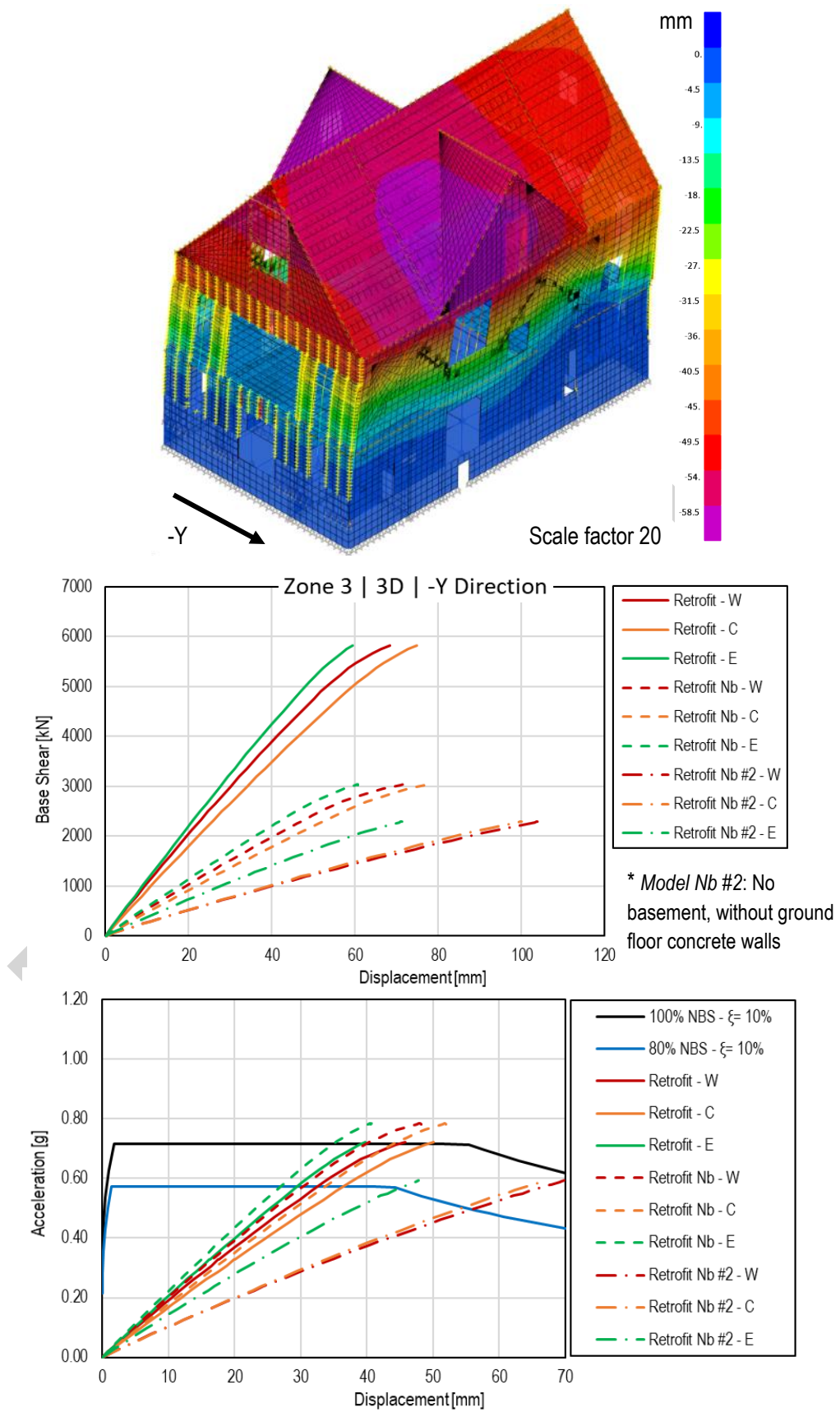


Figure 8-27: Zone 3 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -Y Loading direction | Load pattern: Uniform acceleration

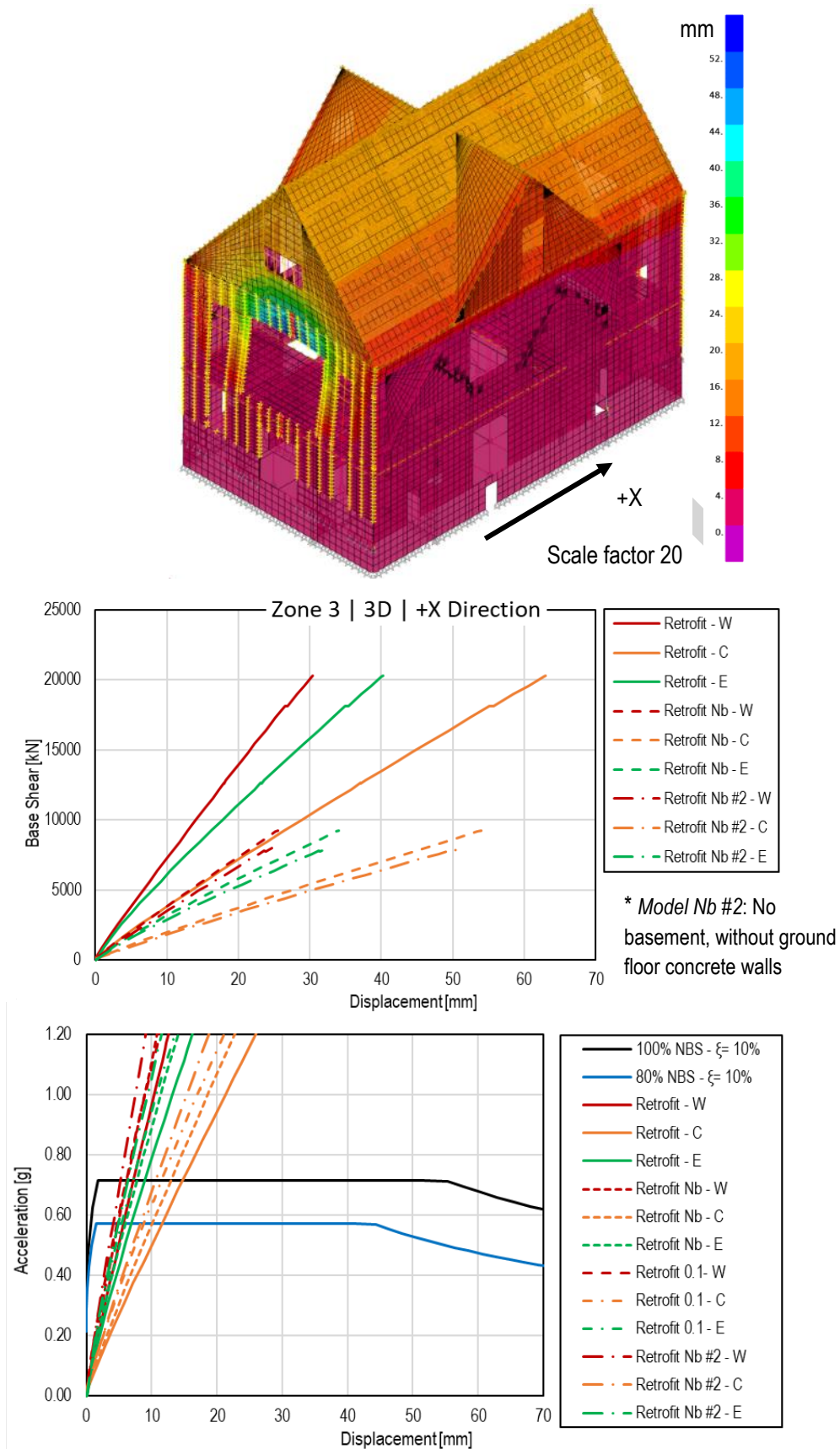


Figure 8-28: Zone 3 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

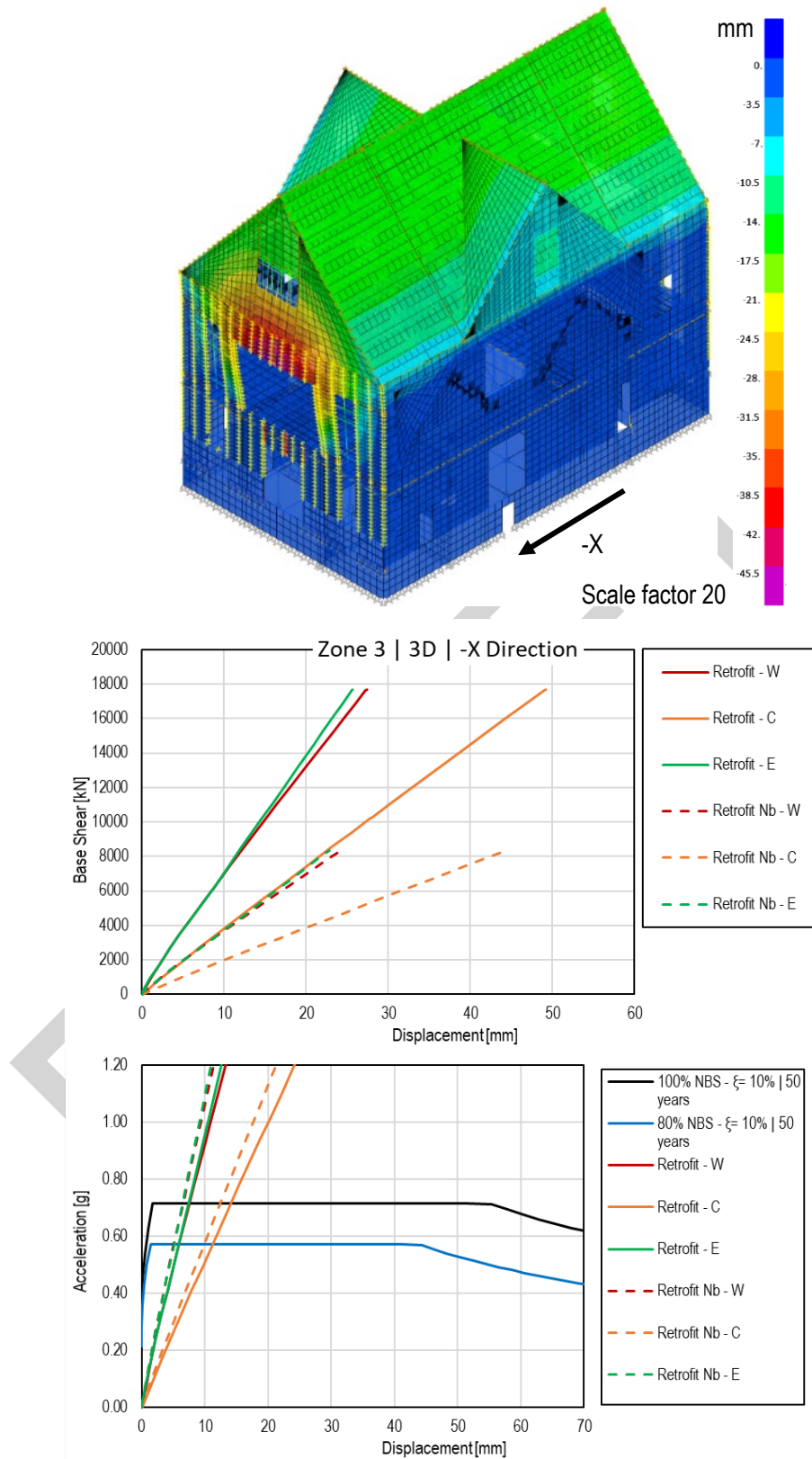


Figure 8-29: Zone 3 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -X Loading direction | Load pattern: Uniform acceleration

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9 ZONE 2 | NONLINEAR STATIC ANALYSES

9.1 Zone 2 | Entire structure analyses

This section reports the outcomes of the nonlinear static pushover analyses performed on the Intermediate South Wing structure (Zone 2). For the lateral loading pattern, the uniform acceleration option was selected (for a more detailed explanation about the selection of lateral loading pattern please refer to the Zone 3 section). In addition, another model was defined which eliminates the basement structure (labelled as “No-basement model”, [Figure 9-1](#)). The no-basement model was also examined using the uniform acceleration load pattern. This was done to provide additional insight into the behaviour of the structure without the influence of the basement structure, which can be large due to the loading condition being proportional to the mass distribution. The hole in the roof which can be observed in the east portion of the building is due to the presence of a newly planned lift structure. The lift structure itself was neglected for the evaluation of the lateral capacity of the retrofitted structure (i.e., it is disconnected from the rest of the building), nevertheless the effects of its presence are accounted for (i.e., holes in both the ground floor and roof structure).

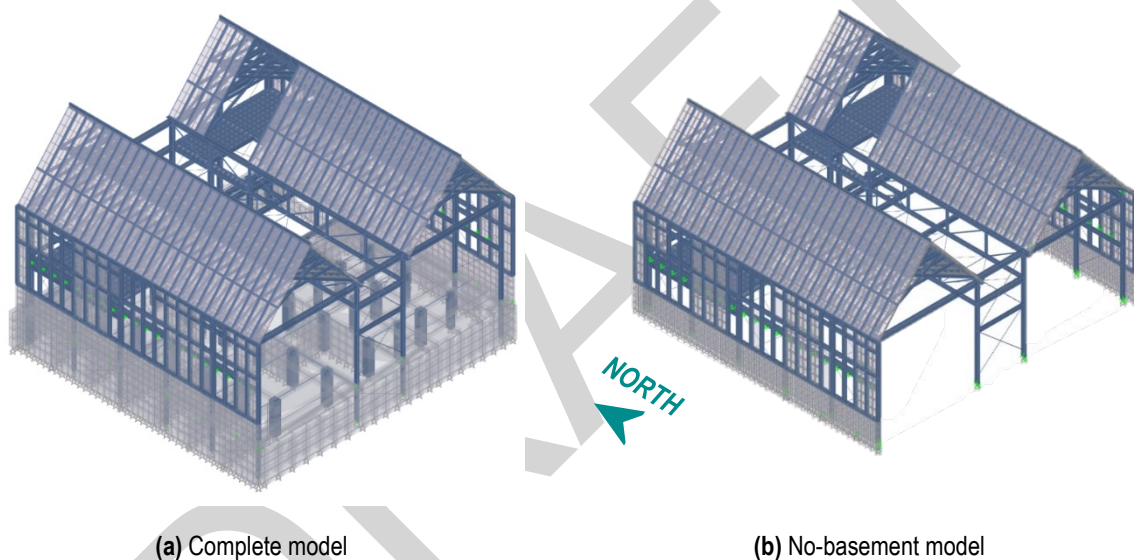


Figure 9-1: Numerical models used for the analysis of the whole zone 2 structure: (a) complete model (b) No-basement model (note that the pictures refer to the retrofitted structure)

Figure 9-2 displays the tributary mass areas adopted to obtain the SDOF curves in the ADRS plots. In both the X and Y direction the tributary areas comprise only the Zone 2 structure. This is because the steel frame structure (added during the past retrofit/development work executed on the building in 2010-11) is disconnected from both the Atrium and South Wing structures (Zone 3 and Zone 1 respectively) and can be treated as a separate structure. **Figure 9-2** also shows the lateral resisting elements (above the ground floor level) in the two main directions, which are considered to transfer the load into the basement structure.

Figure 9-3 reports the control points selected to display the pushover analyses outcomes. The displacements are taken at the highest portion of the building and spread across the structure's footprint to account for any possible irregular behaviour of the structure during the interpretation of the results. The labels W, C, and E stand for West, Central, and East; for the sake of clarity in the pushover curves and ADRS plots charts, the mean displacement values of each selected group of control points are presented. The same control points are used to represent the outcomes of the as-built model and of the retrofit model.



Figure 9-2: Zone 2 - Complete structure | Tributary mass areas used to define the ADRS plot | Loading direction along the X and Y direction

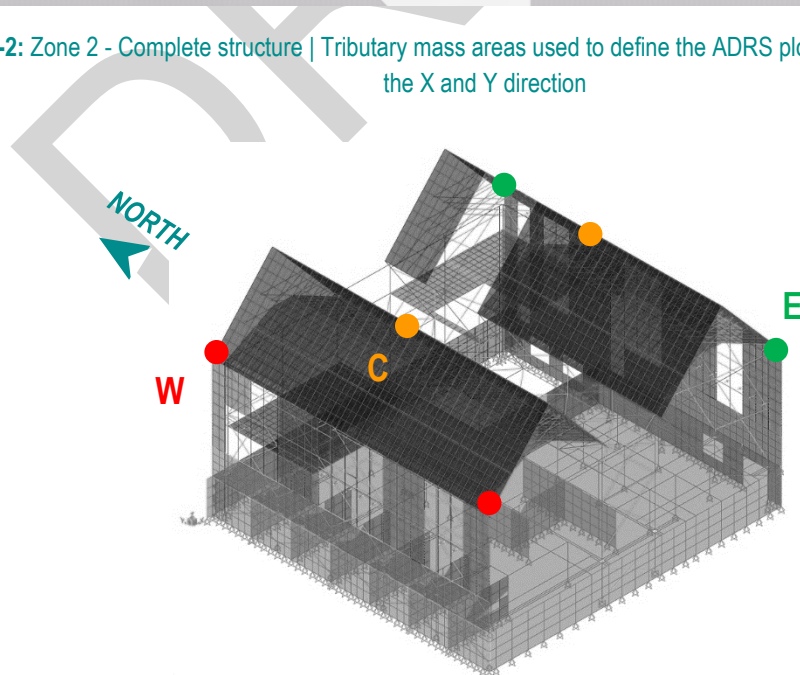


Figure 9-3: Zone 2 - Complete structure | Control points adopted for the nonlinear static pushover analyses

9.1.1 Z2 | As-built condition

This section presents the outcomes of the nonlinear static pushover analyses for the as-built model. These results are used to locate the main area(s) of weakness of the original structure, to design an appropriate retrofit intervention. **Figure 9-4** and **Figure 9-5** report the outcomes of the analyses for the +X and +Y loading directions in terms of deformed shapes (taken at the performance point step of the analyses), pushover curves, and ADRS plots. Cases -X and -Y are neglected due to symmetry reasons, however both positive and negative loading directions will be used for the study of the strengthened building. Although the structure appears to meet the seismic requirements in terms of ADRS plots, the presence of the original timber framed walls still requires consideration in the south-to-north direction. The in-plane resistance of these walls is solely provided by the infill concrete panels, which under large seismic demand may crack (risk of falling debris, life safety risk to occupants). Another aspect requiring consideration is the large deformability of the structure in the X direction (east-to-west). In particular, the resultant high load applied in the out-of-plane of the existing timber framed walls need to be addressed.

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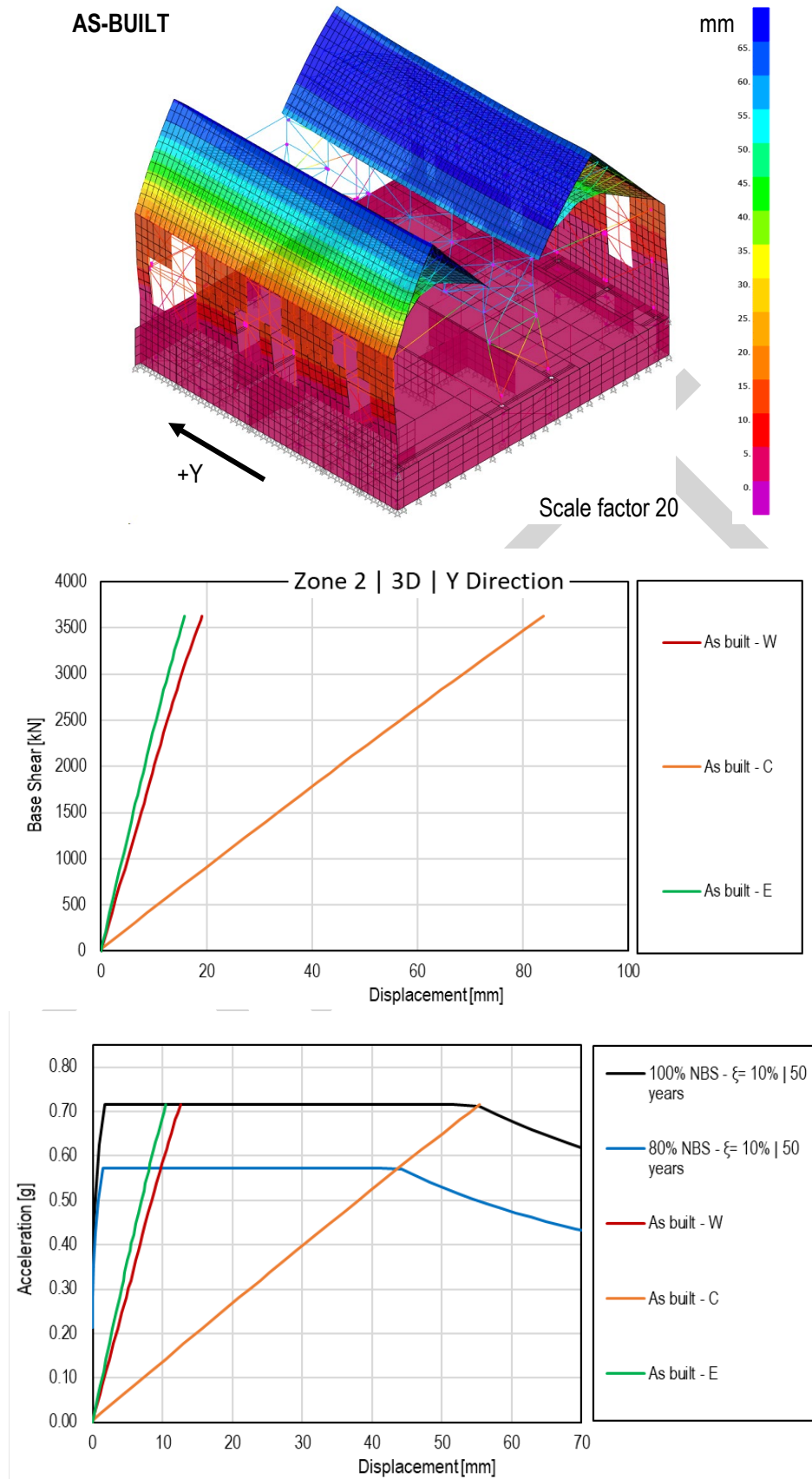


Figure 9-4: Zone 2 - Complete structure | As-built model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

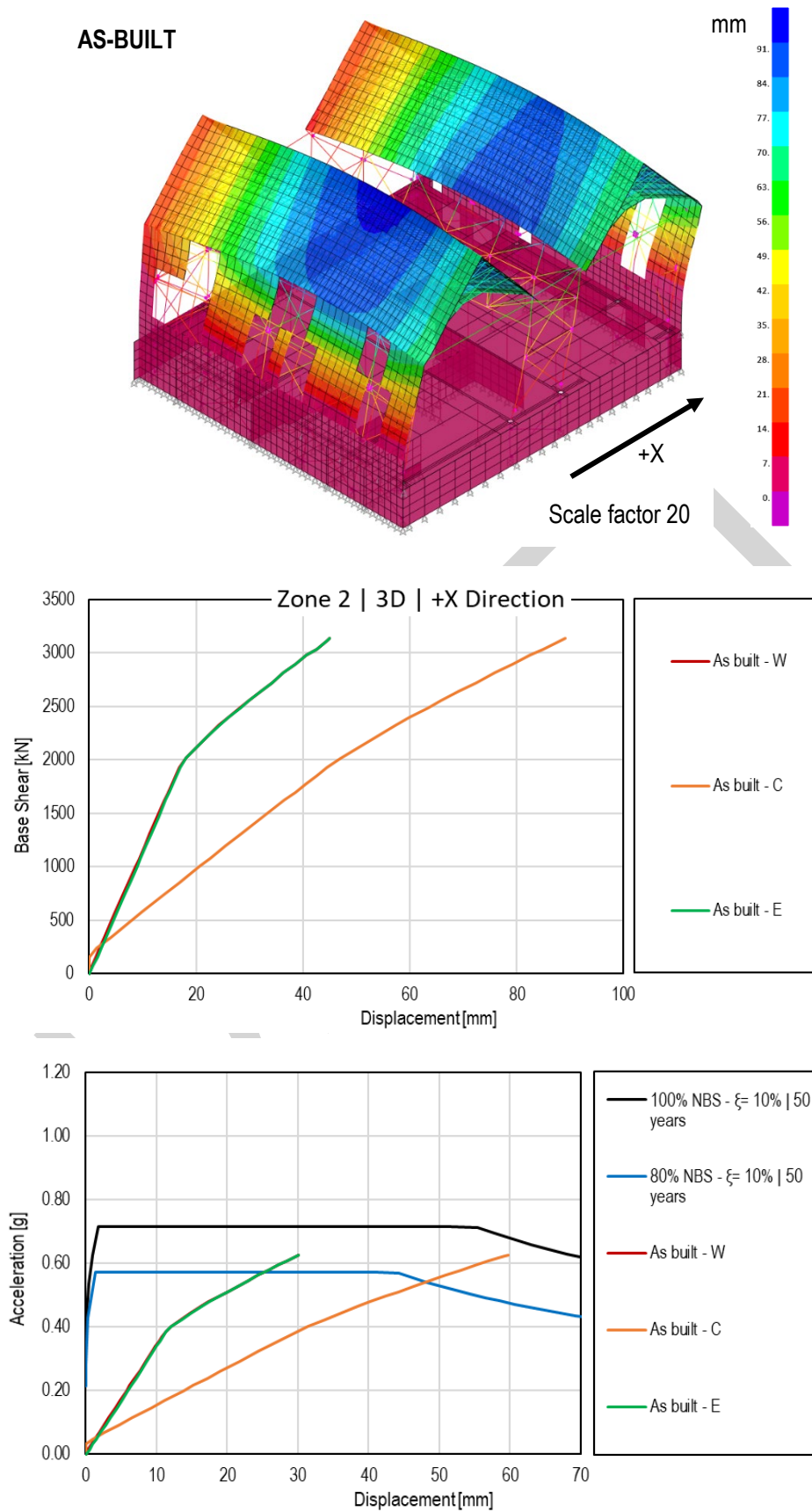


Figure 9-5: Zone 2 - Complete structure | As-built model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

9.1.2 Z2 | Retrofit solution

This section presents the outcomes of the nonlinear static pushover analyses for the retrofit model. **Figure 9-6** presents a summary of all the strengthening works implemented in the numerical model to increase the lateral capacity of the building and to address the weak points of the as-built structure highlighted in the previous section.

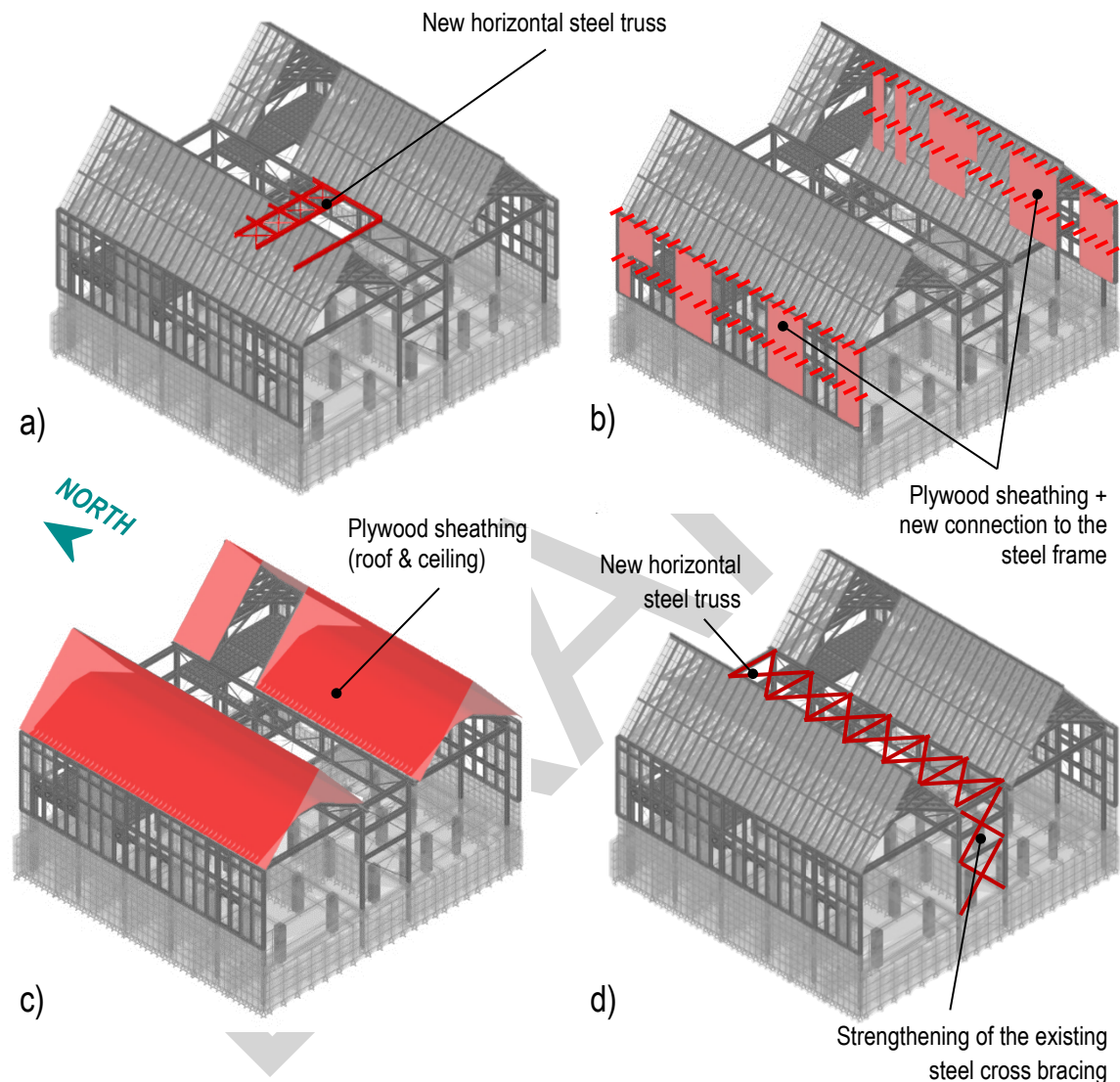


Figure 9-6: Zone 2 – Complete structure | Retrofit intervention implemented in the numerical model

These strengthening works comprise:

- A horizontal steel truss at the ground floor level in the north-east corner where it is planned to build new stairs and the new lift structure (see **Figure 9-6a**). The purpose of this truss is to partially restore the in-plane stiffness of the removed portion of concrete floor (**Figure 9-7**) and to provide an adequate out-of-plane restraint for the Atrium (Zone 3) south concrete gable wall.
- The original timber framed walls are strengthened by a plywood sheathing (see **Figure 9-6b**). The connection between the vertical studs and horizontal members of the original timber framed wall is also strengthened by nailed steel strips incrementing the tension limit of the frame elements from the original 3.85 kN to 10 kN (10 kN being the shear capacity of the steel-to-timber nailed connection). Similarly, the connection between the horizontal members (at each interruption) is reinforced by nailed steel strips (tension limit = 20 kN). Lastly, the

connection of the bottom plate of the timber wall framing with the underlying pumice concrete nib wall is reinforced via grouted steel bars (20 mm diameter and 200 mm embedment depth) inserted adopting a 400 mm spacing.

- To improve the out-of-plane capacity of the original timber framed wall, its connection with the adjacent steel frame is improved by adding an additional row of fasteners at the mid-height of the steel frame structure. In doing so, the OOP capacity of the system now also depends on the capacity of the steel columns connection present at the ground floor level. In the model this is accounted for by implementing a link element with equivalent force-displacement and moment-rotation curves (Figure 9-8).
- The roof and the ceiling structures are strengthened with a plywood overlay (see Figure 9-6c). A new steel truss is introduced connecting the roof eaves, to improve the coupling and load transfer capability between the west and east roof portions (see Figure 9-6d).
- Lastly, the capacity of the steel cross braced frame at the south elevation is improved by increasing the size of the diagonal elements.

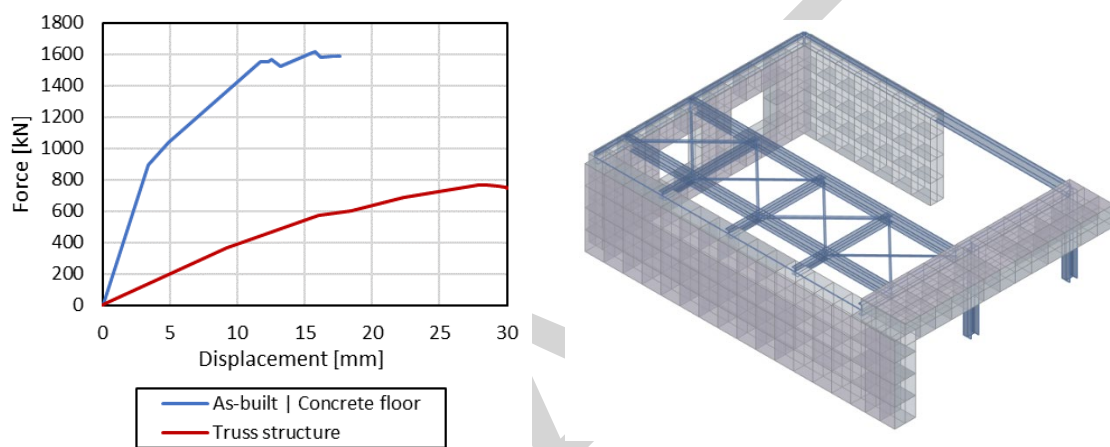


Figure 9-7: Zone 2 – Steel horizontal truss | Pushover curve comparison of the as-built concrete floor and the substituting steel truss and isometric view of the numerical model

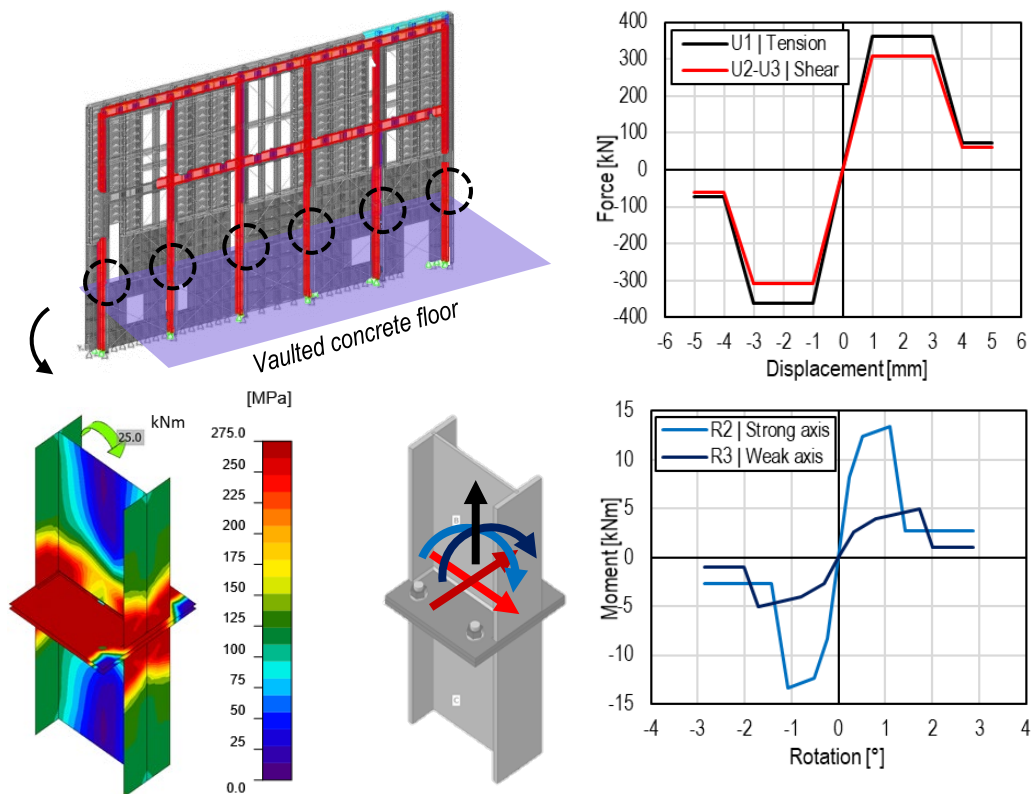


Figure 9-8: Zone 2 – Steel columns connection | Detail analysis to evaluate the out-of-plane capacity of the steel frame. Force-displacement and moment-rotation curve of the adopted equivalent link element

Figure 9-9 to **Figure 9-12** report the outcomes of the nonlinear static analyses carried out on the retrofit model in terms of both pushover curves and ADRS plots. Furthermore, the deformed shapes obtained at the performance point (80%NBS) step of the analyses are reported for a quick visual assessment of the building behaviour when subjected to lateral load.

From the deformed shape diagrams, it can be observed that the concrete infill panels have been neglected in the definition of the Zone 2 retrofit model, with the lateral capacity of the timber framed wall being solely provided by the plywood sheathing reinforcement.

Figure 9-9 and **Figure 9-10** show the effect of the retrofit works on the lateral behaviour of the structure in the Y direction (North-to-South). In both directions the capacity of the retrofitted structure appears to greatly exceed the required demand. By comparing these results to the outcomes of the as-built configuration (**Figure 9-4**), it is observed that the stiffness of the central portion of the roof (control point C) is greatly improved after the implementation of the retrofit works (despite the large opening introduced for the lift structure). This improvement will help in reducing any potential pounding effect onto the adjacent Zone 3 structure, in addition to enhancing the overall response of the Zone 2 structure. Finally, the addition of the plywood sheathing strengthening on the existing timber framed walls and the improvement of the connection with the parallel steel frames relieve the concrete infill panels by taking on the in-plane load that was previously only being resisted by the panels, resulting in a reduced applied stress and in turn, potential damage.

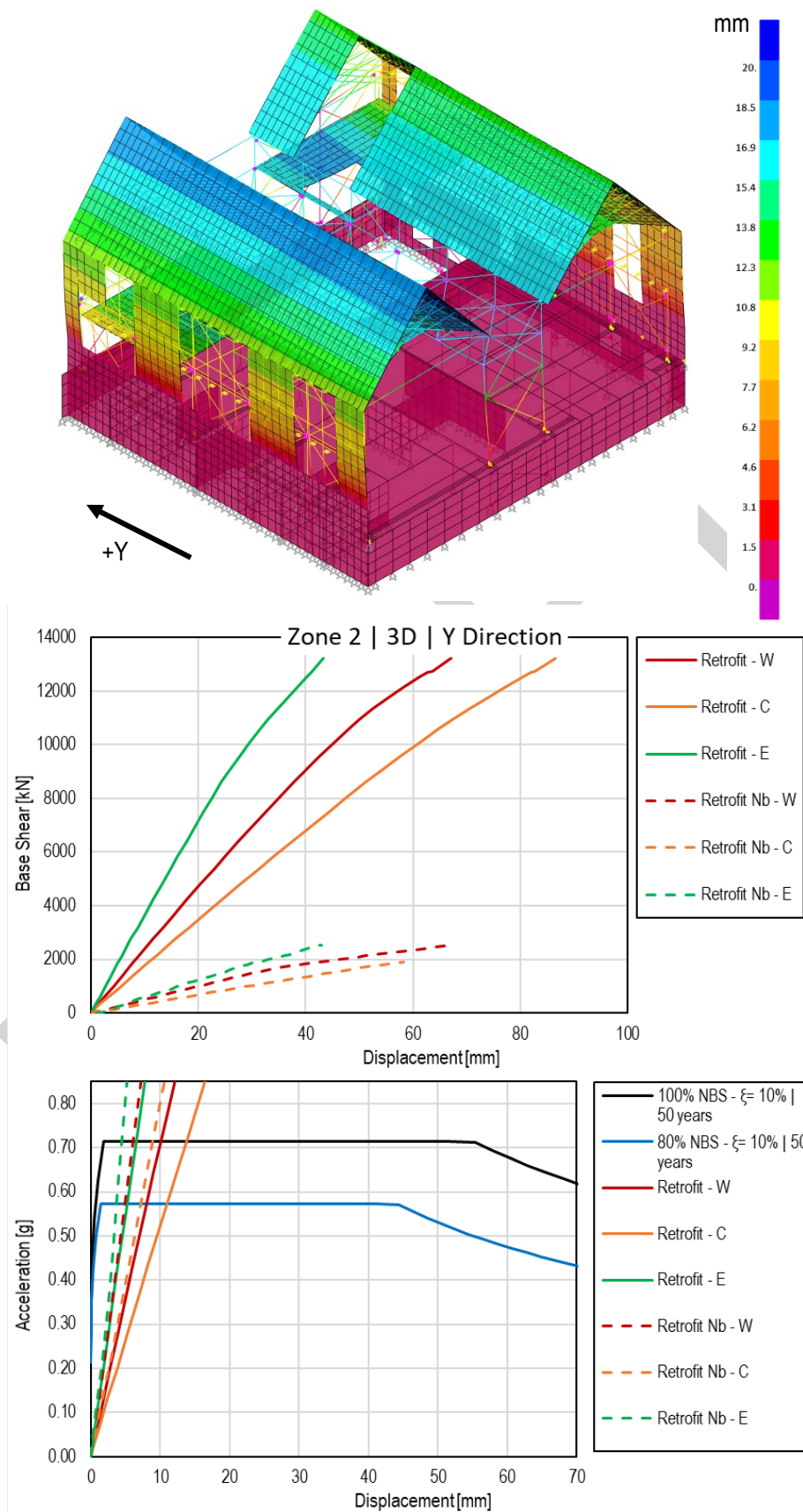


Figure 9-9: Zone 2 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

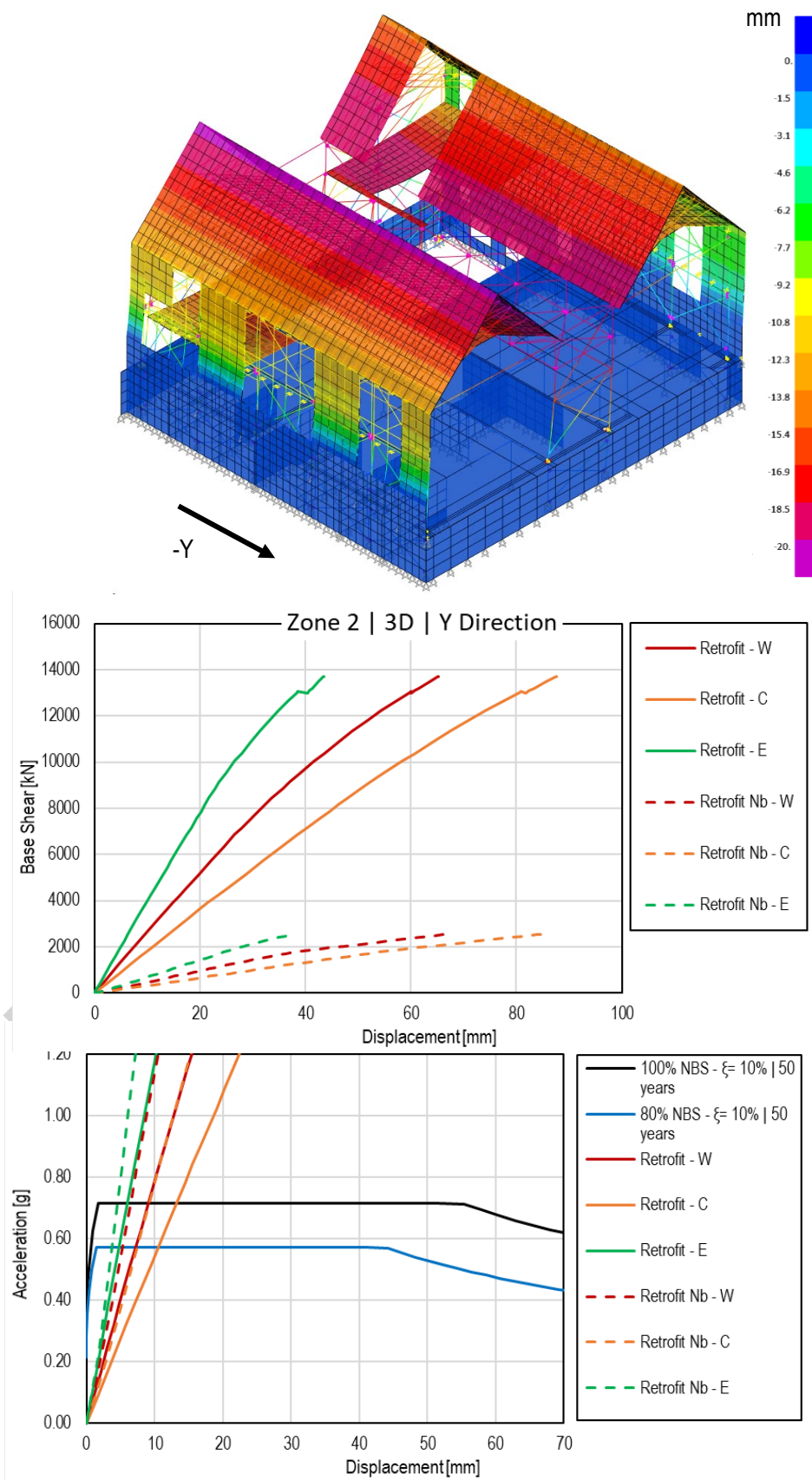


Figure 9-10: Zone 2 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -Y Loading direction | Load pattern: Uniform acceleration

Figure 9-11 and **Figure 9-12** present the outcomes of the retrofit model for the X direction (East-to-West). The big difference, in terms of total base shear of the pushover curves, between the complete model and the no-basement model is due to the presence, at the basement level, of several concrete walls oriented in the east-to-west direction. However, accounting for the appropriate effective mass for each model (in the definition of the SDOF equivalent system) leads to similar results in the ADRS plots. As intended, the strengthening of the roof diaphragm and the improvement of the connection between the original timber framed walls and the steel frames enhances the out-of-plane response of the West and East walls. Lastly, the improvement of the steel cross braced frame at the south elevation via the replacement of existing diagonal members with larger elements assists in decreasing the disparity of displacement demand between the north and south part of Zone 2.

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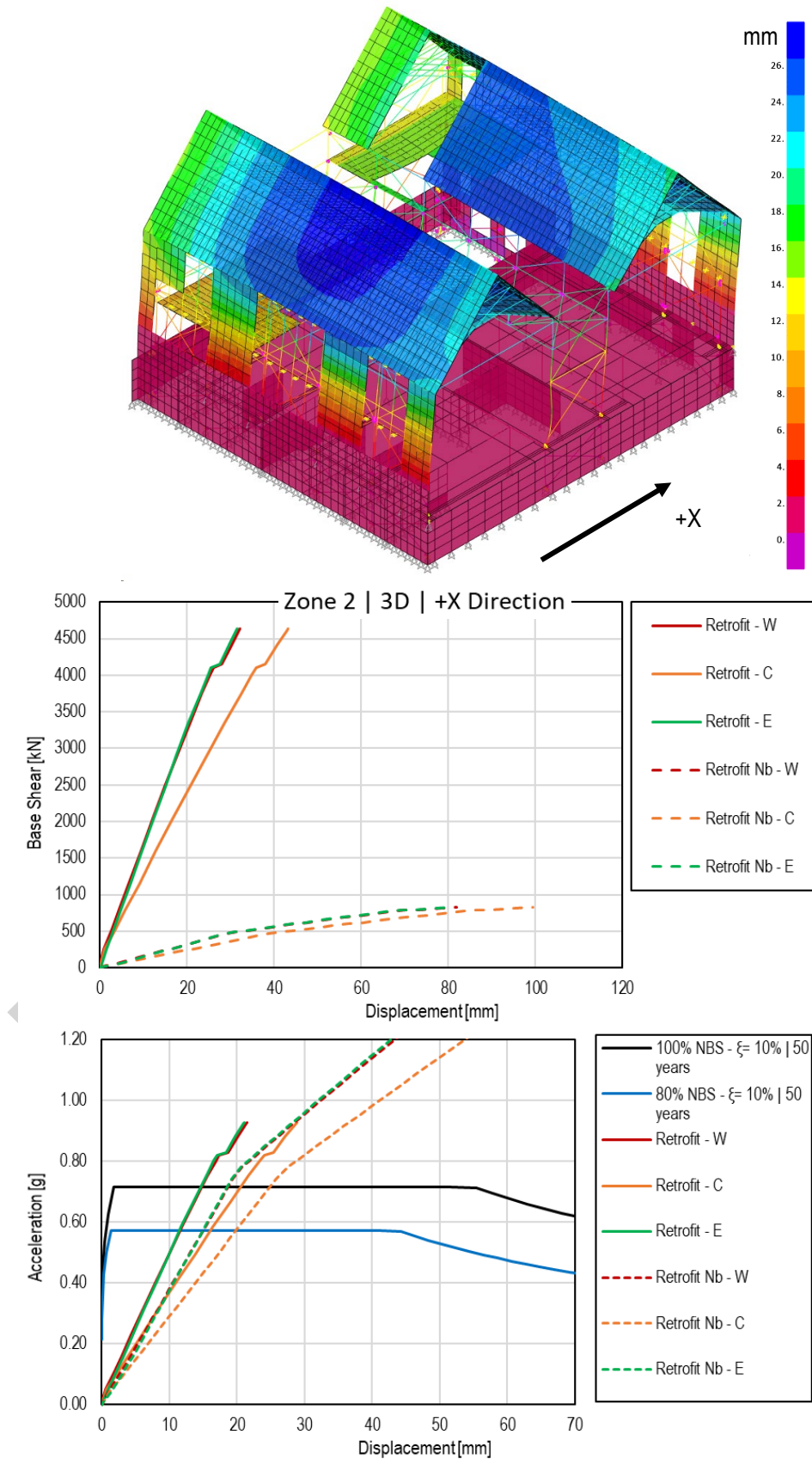


Figure 9-11: Zone 2 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

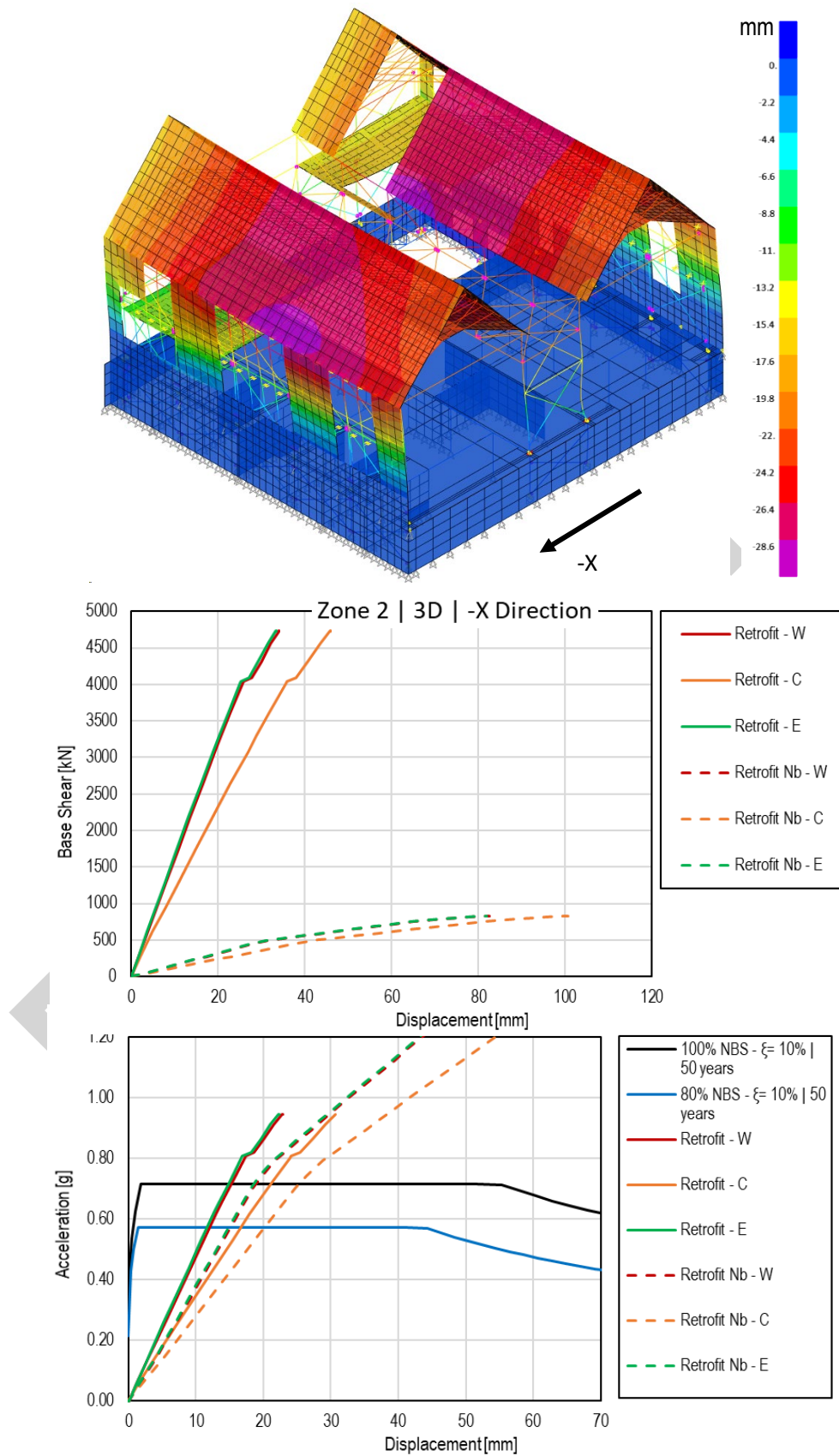


Figure 9-12: Zone 2 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -X Loading direction | Load pattern: Uniform acceleration

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10 ZONE 4 | NONLINEAR STATIC ANALYSES

10.1 Zone 4 | Entire structure analyses

This section reports the outcomes of the nonlinear static pushover analyses performed on the Intermediate North Wing structure (Zone 4). For the lateral loading pattern, the uniform acceleration option was selected (for a more detailed explanation about the selection of lateral loading pattern please refer to the Zone 3 section). In addition to that, another model is defined eliminating the basement structure (labelled as “No-basement model”, [Figure 10-1](#)). The no-basement model is also examined using the uniform acceleration load pattern. This is done to provide additional insight into the behaviour of the structure without the influence of the basement structure, which can be large due to the loading condition being proportional to the mass distribution.

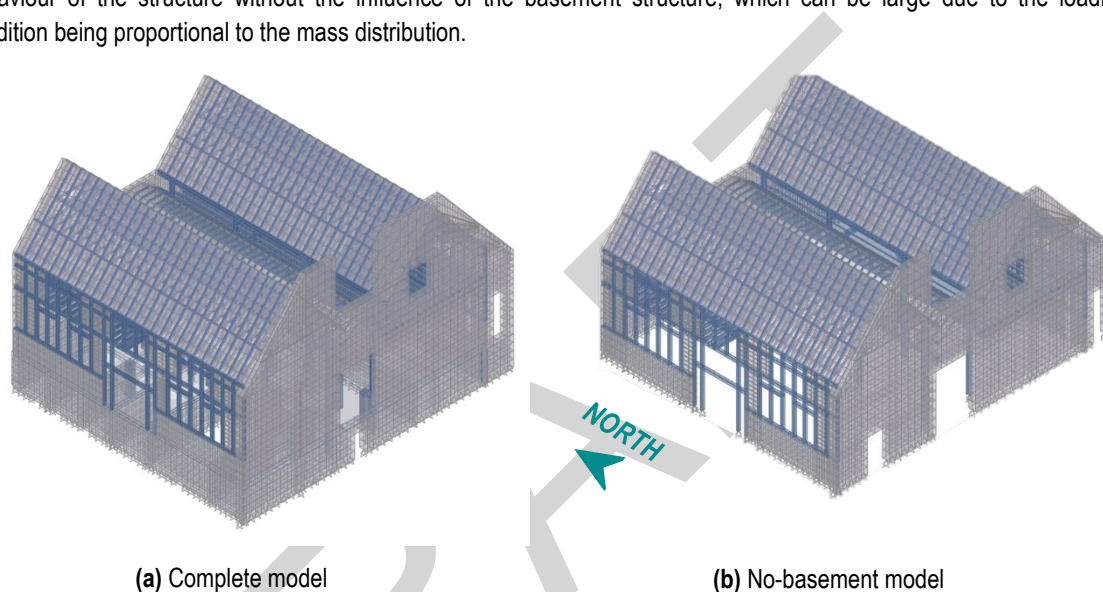


Figure 10-1: Numerical models used for the analysis of the whole zone 4 structure: (a) Complete model (b) No-basement model

[Figure 10-2](#) shows the tributary mass areas adopted to obtain the SDOF curves in the ADRS plots. In the X direction the South wall tributary area comprises half of Zone 3 and half of Zone 4 due to the direct connection of these two zones. Same for the North wall, which is loaded by half of Zone 4 and half of Zone 5. For loads oriented in the Y direction, the mass tributary area includes only Zone 4 (accounting also for the mass of the two large concrete gable walls), adequately divided between the four main lateral load-carrying systems in that direction (West and East wall, and the two steel braced frames in the central hallway).

[Figure 10-3](#) reports the control points selected to display the pushover analyses outcomes. The displacements are taken at the highest portion of the building and spread across the structure's footprint to account for any possible irregular behaviour of the structure during the interpretation of the results. The labels W, C, and E stands for West, Central, and East; for the sake of clarity in the pushover curves and ADRS plots charts, the mean displacement values of each selected group of control points are presented. The same control points are used to represent the outcomes of the as-built model and of the retrofit model.



Figure 10-2: Zone 4 - Complete structure | Tributary mass areas used to define the ADRS plot | Loading direction along the X and Y direction

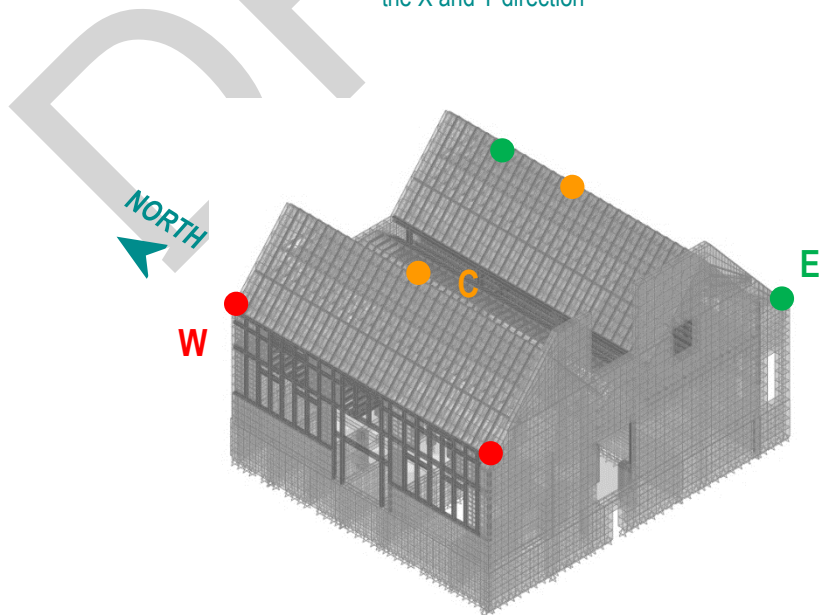


Figure 10-3: Zone 4 - Complete structure | Control points adopted for the nonlinear static pushover analyses

10.1.1 Z4 | As-built condition

This section presents the outcomes of the nonlinear static pushover analyses for the as-built model. These results are used to locate the main area of weakness of the original structure, to design an appropriate retrofit intervention. **Figure 10-4** and **Figure 10-5** report the outcomes of the analyses for the +X and +Y loading directions in terms of deformed shapes (taken at the performance point step of the analyses), pushover curves, and ADRS plots. Cases -X and -Y are neglected due to symmetry reasons, however both positive and negative loading directions will be used for the study of the strengthened building. In the Y direction (North-to-South) the building in the as-built condition doesn't achieve the required seismic performance level. This is due to the limited in-plane capacity of the existing timber framed walls and of the existing steel frames. In turn, this limited capacity causes an increase in the out-of-plane demand on the two large unreinforced concrete gable walls. In the X direction (East-to-West) the structure appears to meet the seismic requirements in terms of ADRS plots. However, the large displacement of the central portion of the structure requires a reduction as the existing timber framed walls are unable to withstand an out-of-plane seismic demand of that intensity.

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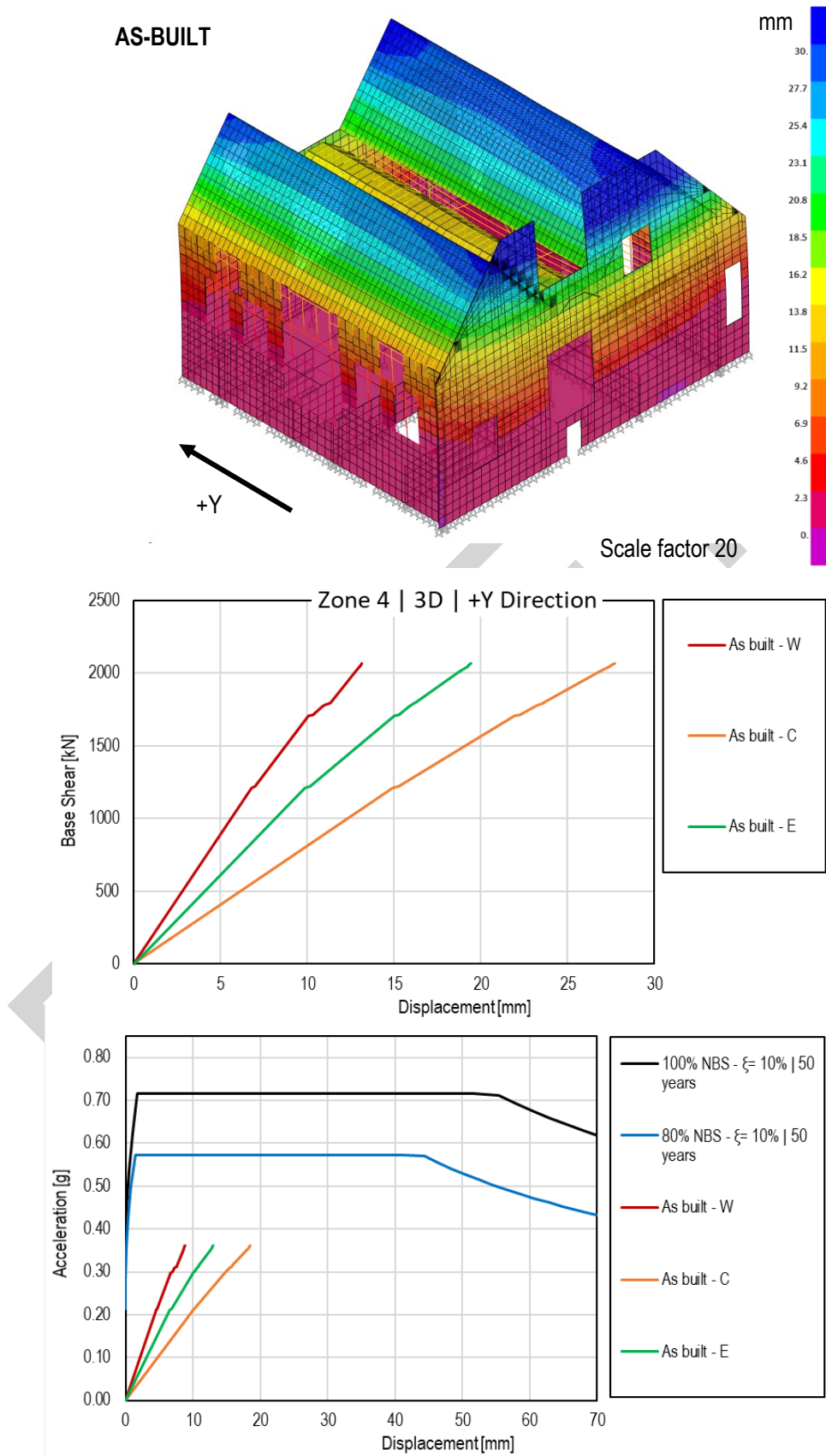


Figure 10-4: Zone 4 - Complete structure | As-built model deformed shape at the last analysis step | +Y Loading direction | Load pattern: Uniform acceleration

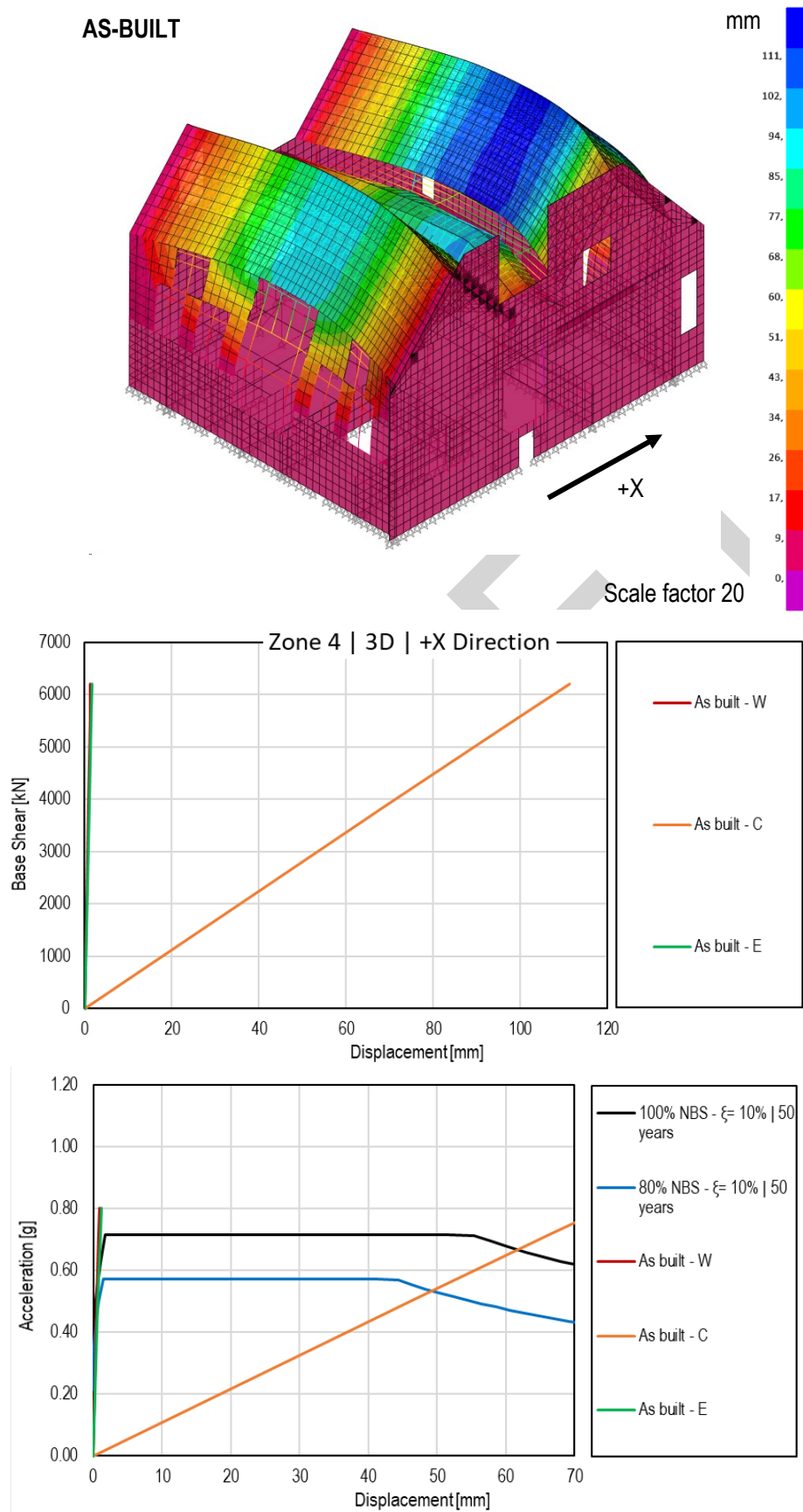


Figure 10-5: Zone 4 - Complete structure | As-built model deformed shape at the last analysis step | +X Loading direction | Load pattern: Uniform acceleration

10.1.2 Z4 | Retrofit solution

This section presents the outcomes of the nonlinear static pushover analyses for the retrofit model. **Figure 10-6** shows a summary of all the strengthening works implemented in the numerical model to increase the lateral capacity of the building and to address the weak points of the as-built structure highlighted in the previous section.

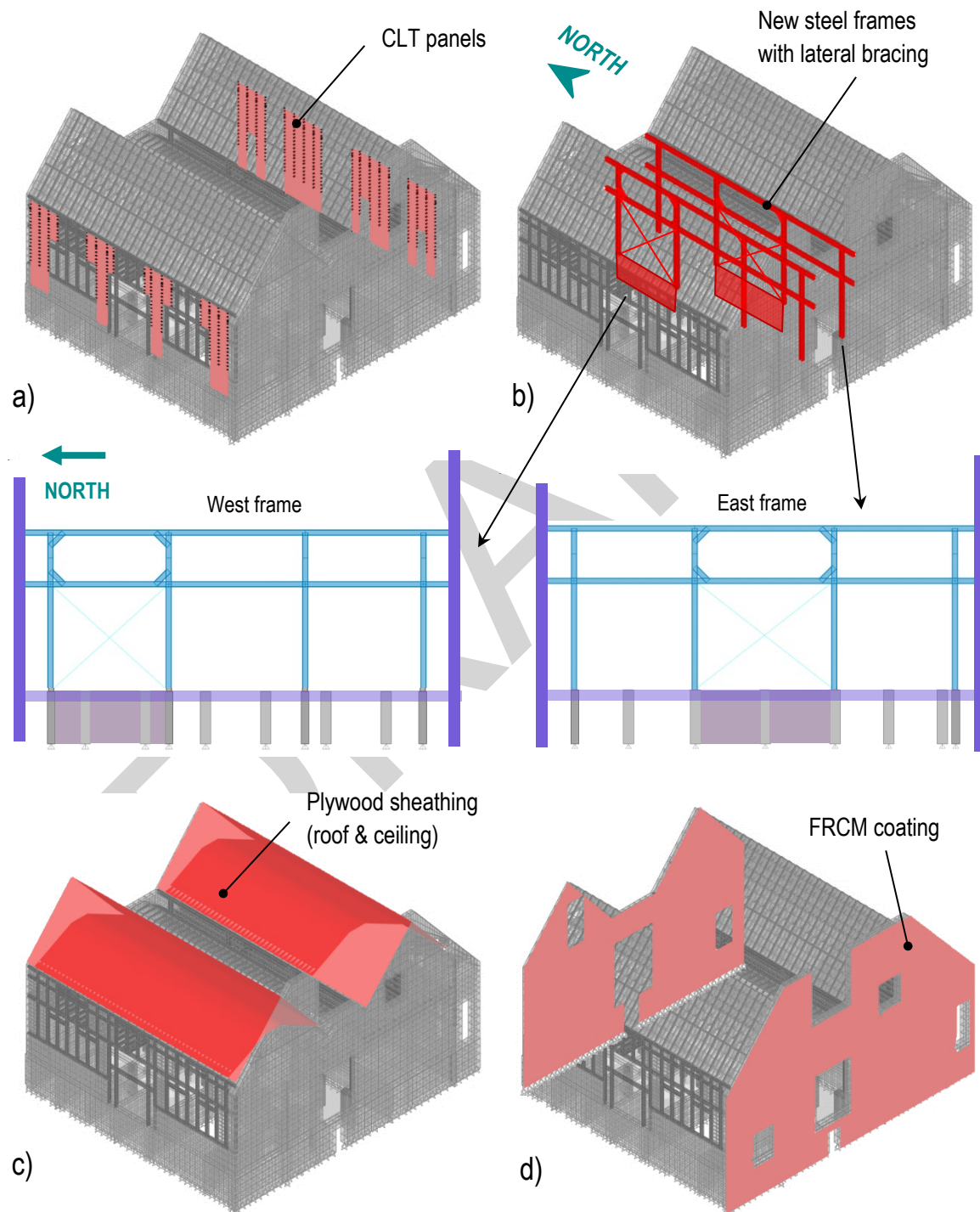


Figure 10-6: Zone 4 - Complete structure | Retrofit intervention implemented in the numerical model (structural elements contributing to the lateral capacity of the entire structure)

These strengthening works comprise:

- CLT panels connected to timber frame of the West and East wall using screw fasteners with a spacing of 250 mm (see [Figure 10-6a](#)). The panels are considered properly anchored at the ground floor level.
- Substitution of the existing moment-resisting steel frames with new steel frames having cross braces at the ground floor level and a moment-resisting extension at the mezzanine level (see [Figure 10-6b](#)). The lateral load is then transferred at the foundation level by including two new RC walls at the basement level under the braced spans.
- The roof and the ceiling structures are strengthened with a plywood overlay (see [Figure 10-6c](#)).
- FRCM coating applied on both side of the pumice concrete gable walls (North and South wall) (see [Figure 10-6d](#)).

[Figure 10-7](#) to [Figure 10-10](#) report the outcomes of the nonlinear static analyses carried out on the retrofit model in terms of both pushover curves and ADRS plots. Furthermore, the deformed shapes obtained at the performance point (80%NBS) step of the analyses are reported for a quick visual assessment of the building behaviour when subjected to lateral load.

From the deformed shape pictures, it is observed that the concrete infill panels have been neglected in the definition of the Zone 4 retrofit model, with the lateral capacity of the timber framed wall solely provided by the plywood sheathing.

[Figure 10-7](#) and [Figure 10-8](#) show the effect of the retrofit works on the lateral behaviour of the structure in the Y direction (North-to-South). The improvement of the in-plane capacity of the timber framed walls and of the central steel frames, added to the increased out-of-plane capacity of the pumice concrete gable walls, leads to an improvement of the lateral capacity of the structure beyond the required 80% NBS level.

[Figure 10-9](#) and [Figure 10-10](#) present the outcomes of the retrofit model for the X direction (East-to-West). The strengthening of the out-of-plane (OOP) behaviour of the West and East walls using CLT panels, and the reinforcement of the roof structure via plywood sheathing, enables a significant reduction of the OOP displacement demand placed on the West and East walls themselves. Lastly, [Figure 10-11](#) and [Figure 10-12](#) report the axial forces on the new steel frames cross-bracing, derived from the retrofit model at the performance point step of the analysis (80% NBS) and used to cross check the required capacity of the cross-bracing elements section according to NZS 3404:1997 Parts 1 and 2.

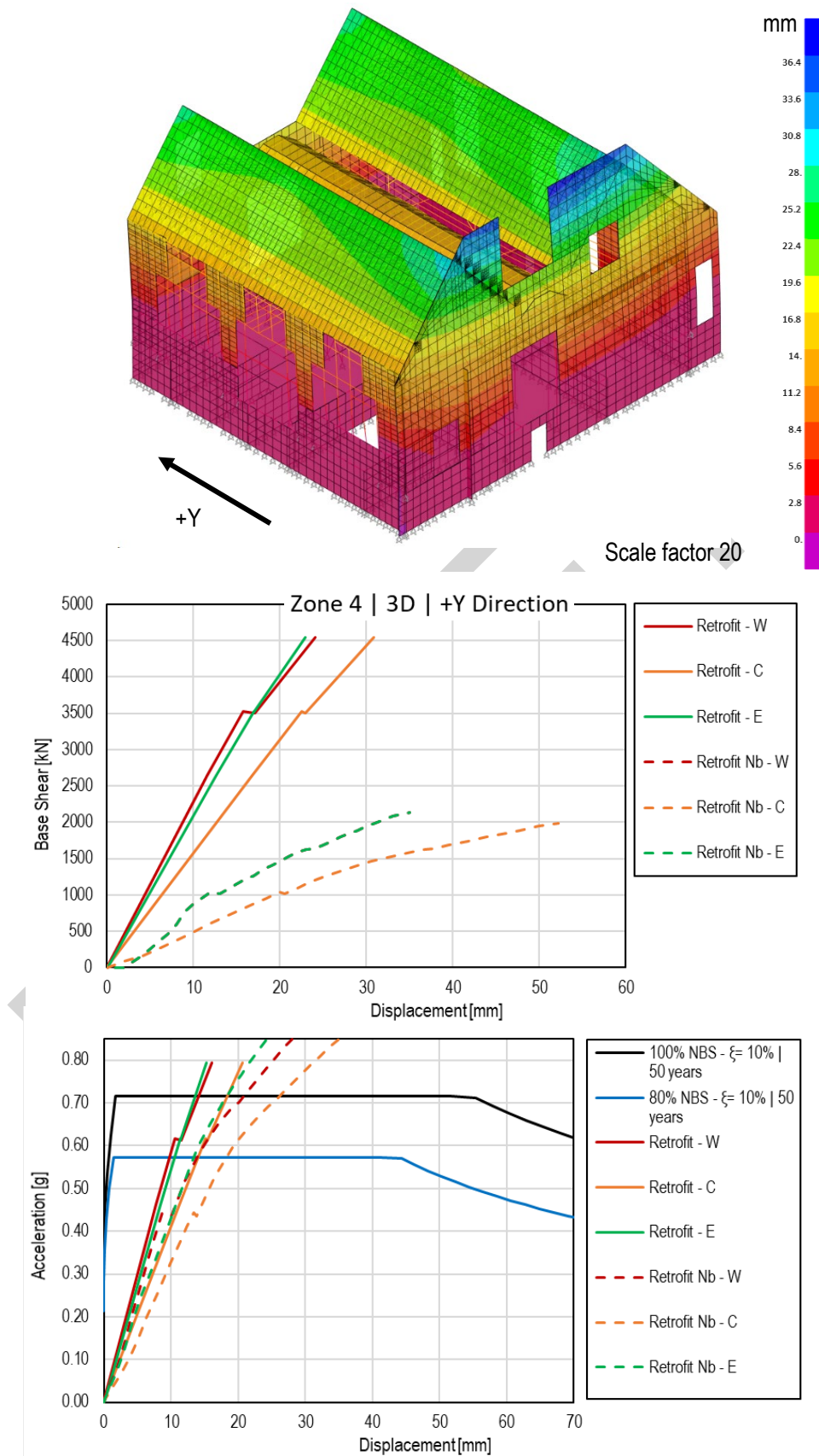


Figure 10-7: Zone 4 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

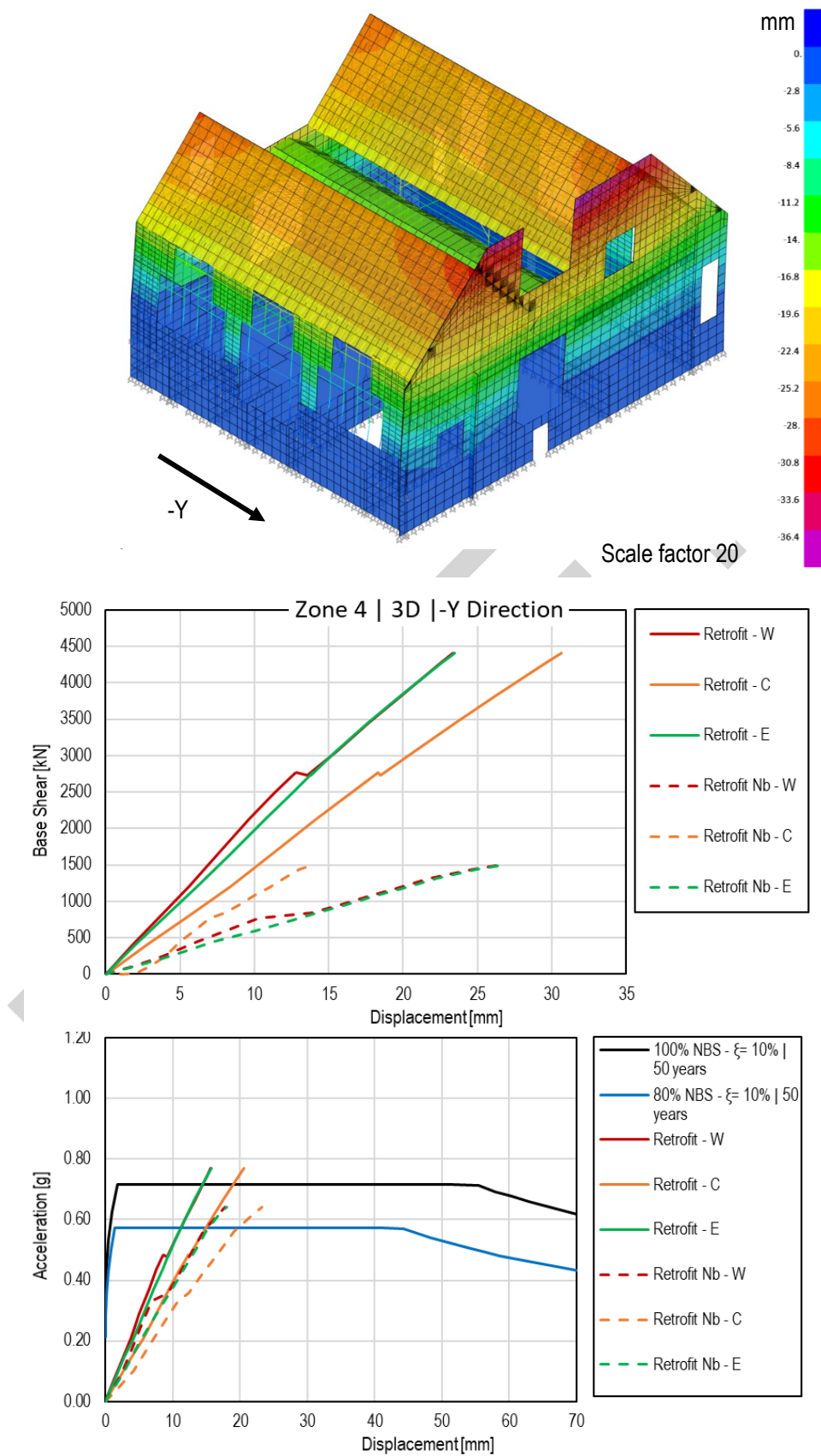


Figure 10-8: Zone 4 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -Y Loading direction | Load pattern: Uniform acceleration

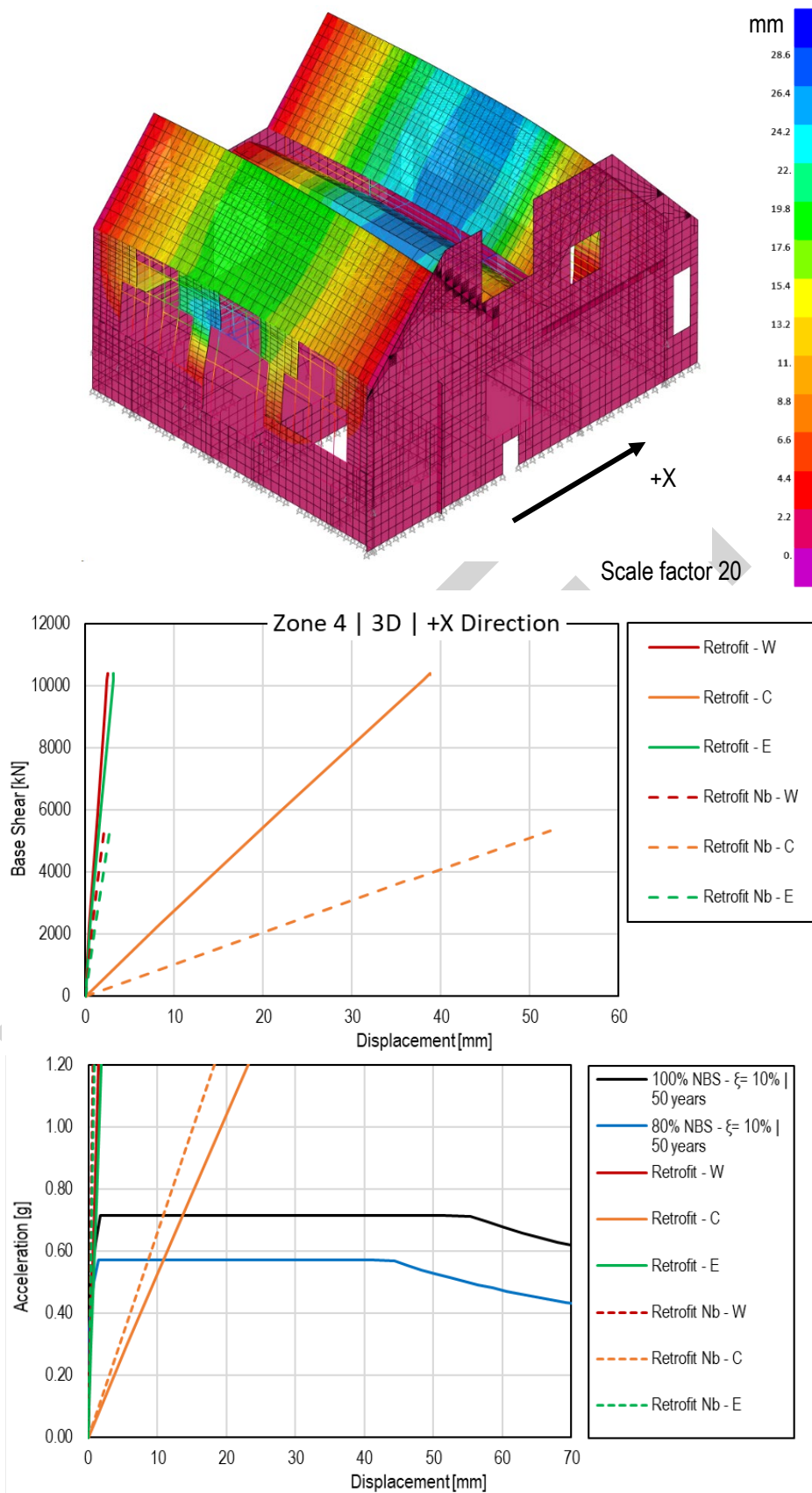


Figure 10-9: Zone 4 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

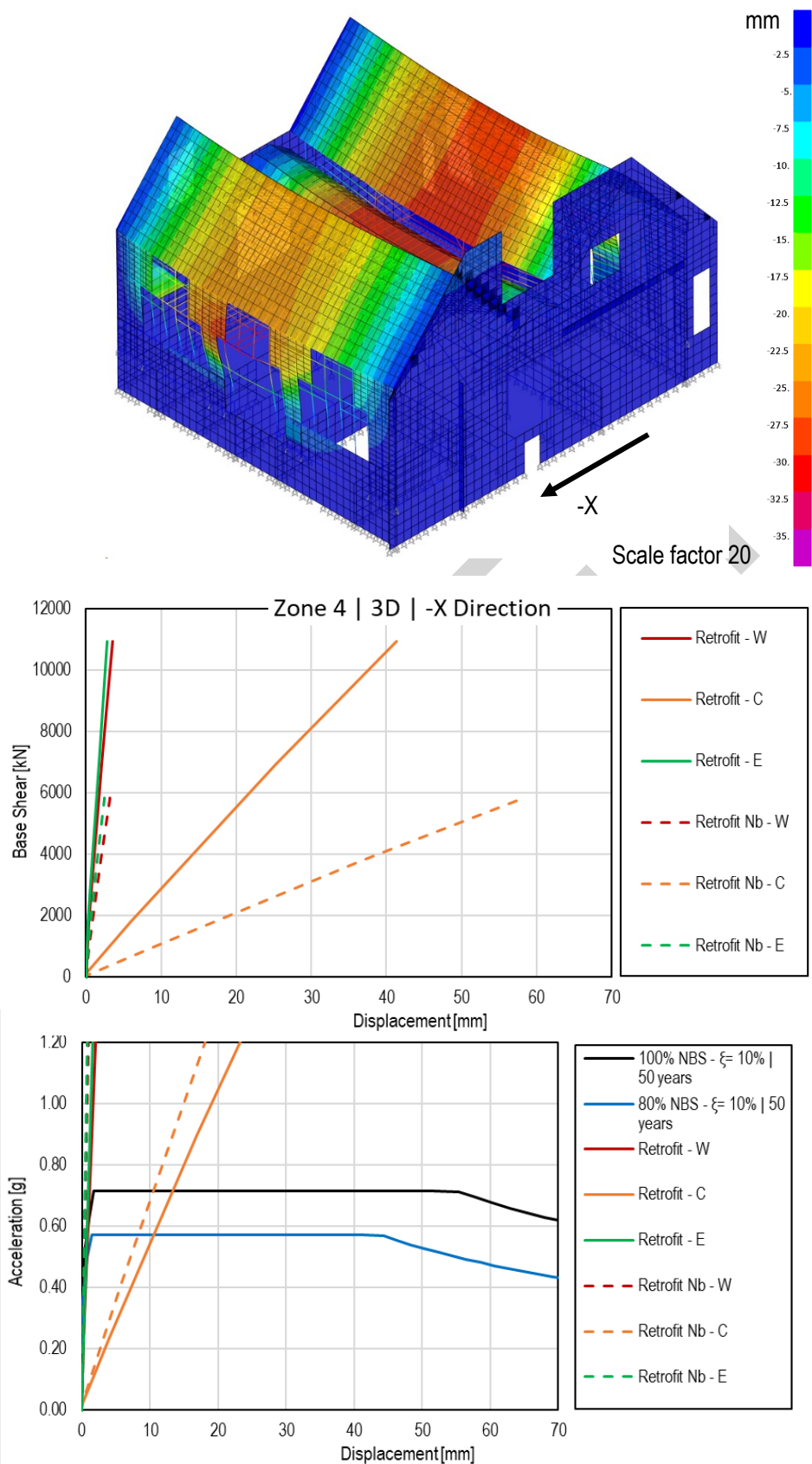


Figure 10-10: Zone 4 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -X Loading direction | Load pattern: Uniform acceleration

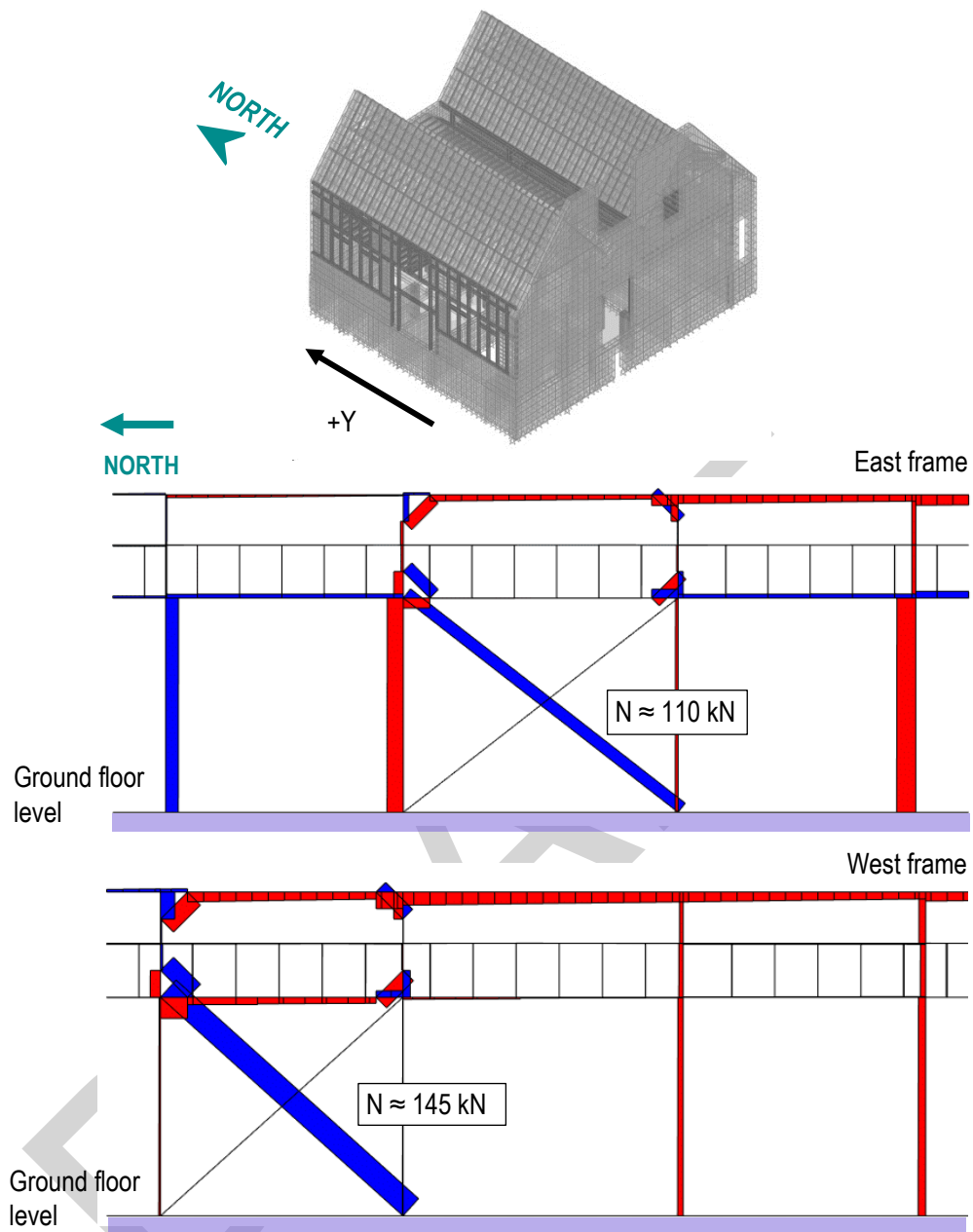


Figure 10-11: Zone 4 – Steel frames | Axial forces at the performance point step | +Y Loading direction

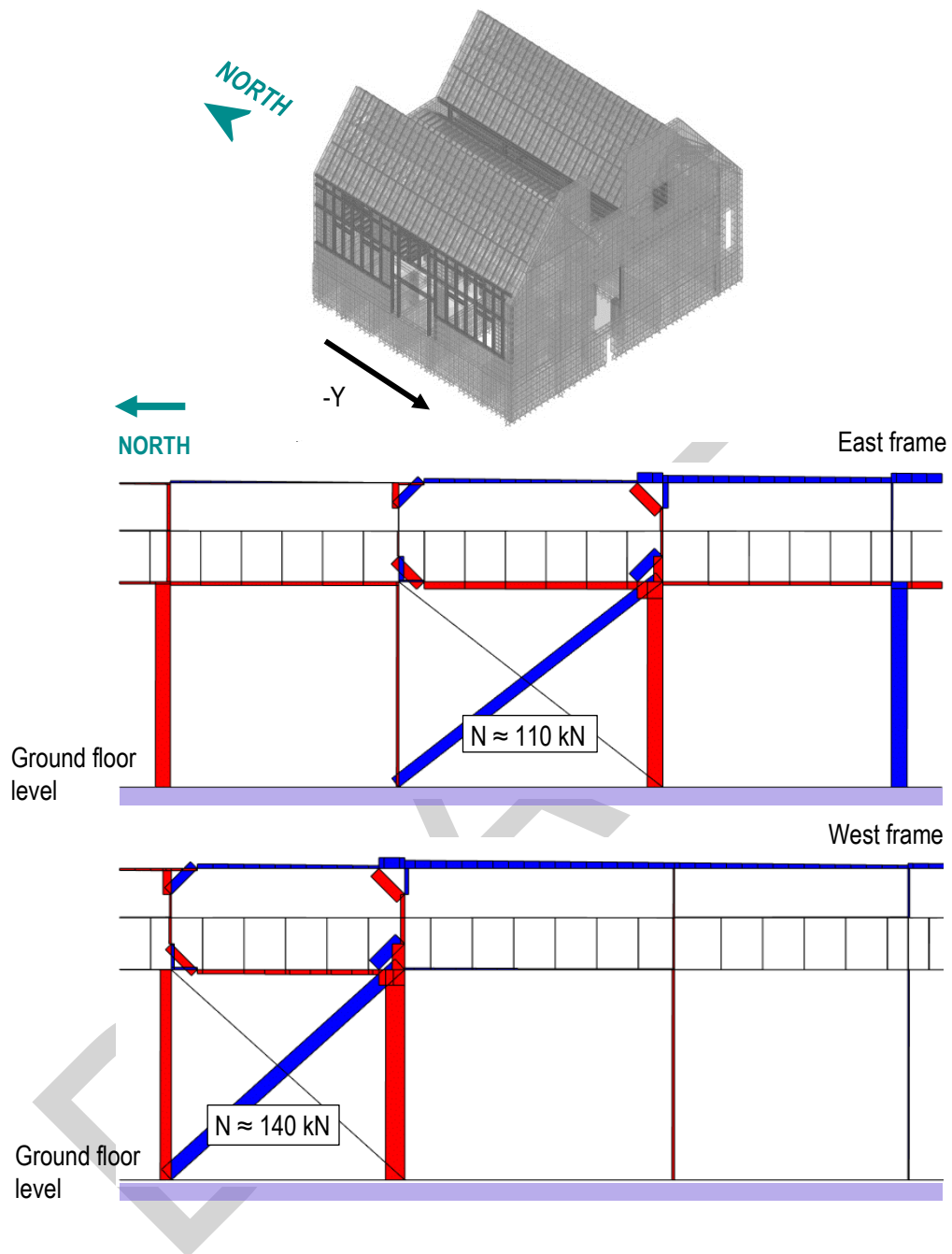


Figure 10-12: Zone 4 – Steel frames | Axial forces at the performance point step | -Y Loading direction

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11 ZONE 5 | NONLINEAR STATIC ANALYSES

11.1 Zone 5 | Entire structure analyses

This section reports the outcomes of the nonlinear static pushover analyses performed on the North Wing structure (Zone 5). For the lateral loading pattern, the uniform acceleration option was selected (for a more detailed explanation about the selection of lateral loading pattern please refer to the Zone 3 analysis section). In addition, another model was defined eliminating the basement structure (labelled as “No-basement model”, **Figure 11-1**). The no-basement model was also examined using the uniform acceleration load pattern. This was done to provide additional insight into the behaviour of the structure without the influence of the basement structure, which can be large due to the loading condition being proportional to the mass distribution.

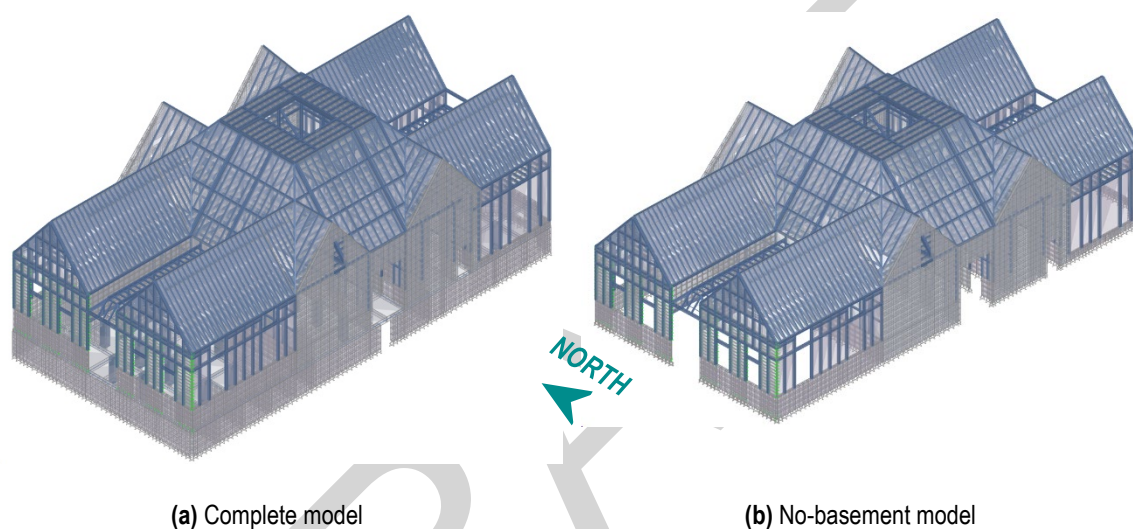


Figure 11-1: Numerical models used for the analysis of the whole zone 5 structure: (a) Complete model (b) No-basement model

Figure 11-2 shows the tributary mass areas adopted to obtain the SDOF curves in the ADRS plots. In the X direction the South wall tributary area comprises half of Zone 4 and half of Zone 5 due to the direct connection of these two zones. On the other hand, the North wall tributary area includes only half of Zone 5 because the adjacent Zone 6 is a modern addition (constructed c. 2007), built separate from the rest of the historic building. For loads oriented in the Y direction, the mass tributary area includes only Zone 5 (accounting also for the mass of the two large concrete gable walls). The dark red lines in the figure identify the walls that have been selected to be the lateral resisting elements in that direction and that will be adequately strengthened or overlain and connected to new wall skins.

Figure 11-3 reports the control points selected to display the pushover analyses outcomes. The displacements are taken at the highest portion of the building and spread across the structure's footprint to account for any possible irregular behaviour of the structure during the interpretation of the results. The labels W, C, and E stands for West, Central, and East; for the sake of clarity in the pushover curves and ADRS plots charts the mean displacement values of each selected group of control points are presented. The same control points are used to represent the outcomes of the as-built model and of the retrofit model.



Figure 11-2: Zone 5 - Complete structure | Tributary mass areas used to define the ADRS plot | Loading direction along the X and Y direction

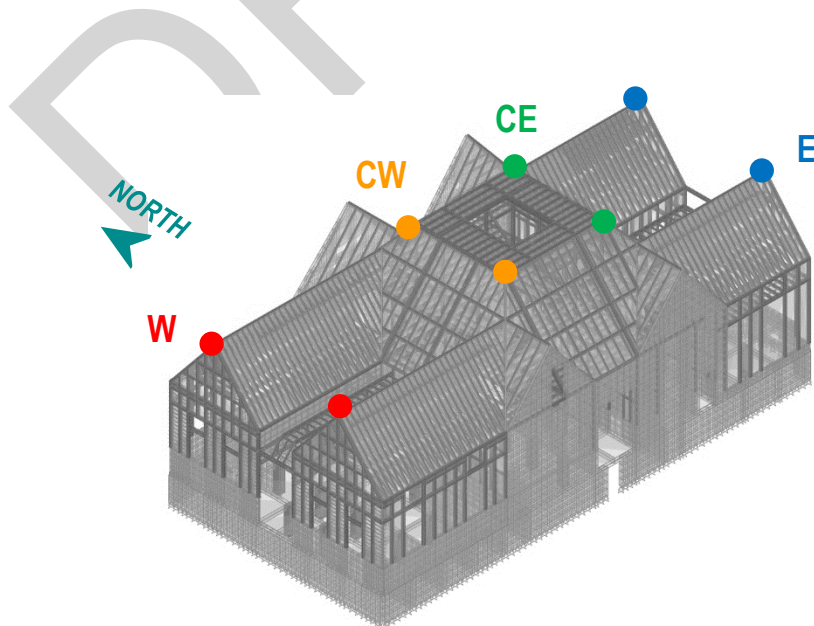


Figure 11-3: Zone 5 - Complete structure | Control points adopted for the nonlinear static pushover analyses

11.1.1 Z5 | As-built condition

This section presents the outcomes of the nonlinear static pushover analyses for the as-built model. These results are used to locate the main area of weakness of the original structure, to design an appropriate retrofit intervention. **Figure 11-5** and **Figure 11-6** report the outcomes of the analyses for the +X and +Y loading directions in terms of deformed shapes, pushover curves, and ADRS plots. The deformed shapes taken at the performance point step of the analysis (+Y direction) and the associated principle tensile strain on the concrete South gable wall are also shown in **Figure 11-4**. Even though the structure appears to meet the safety requirements at the 80% NBS level, it is evident that the displacement demand on the single structural elements is excessively large (in particular the West section, as visible from **Figure 11-4**). Therefore, the retrofit work shall address the out-of-plane behaviour of the two large concrete gable walls and the out-of-plane behaviour of the two “extensions” made of timber framed walls (West and East sections). In the X direction (East-to-West) the structure appears to meet the seismic requirements in terms of ADRS plots. The out-of-plane of the West and East façades seems then to be less concerning in terms of maximum displacement with respect to other parts of the building (e.g., Zone 2 and Zone 4), likely due to the orientation of the roof pitches in the East-to-West direction.

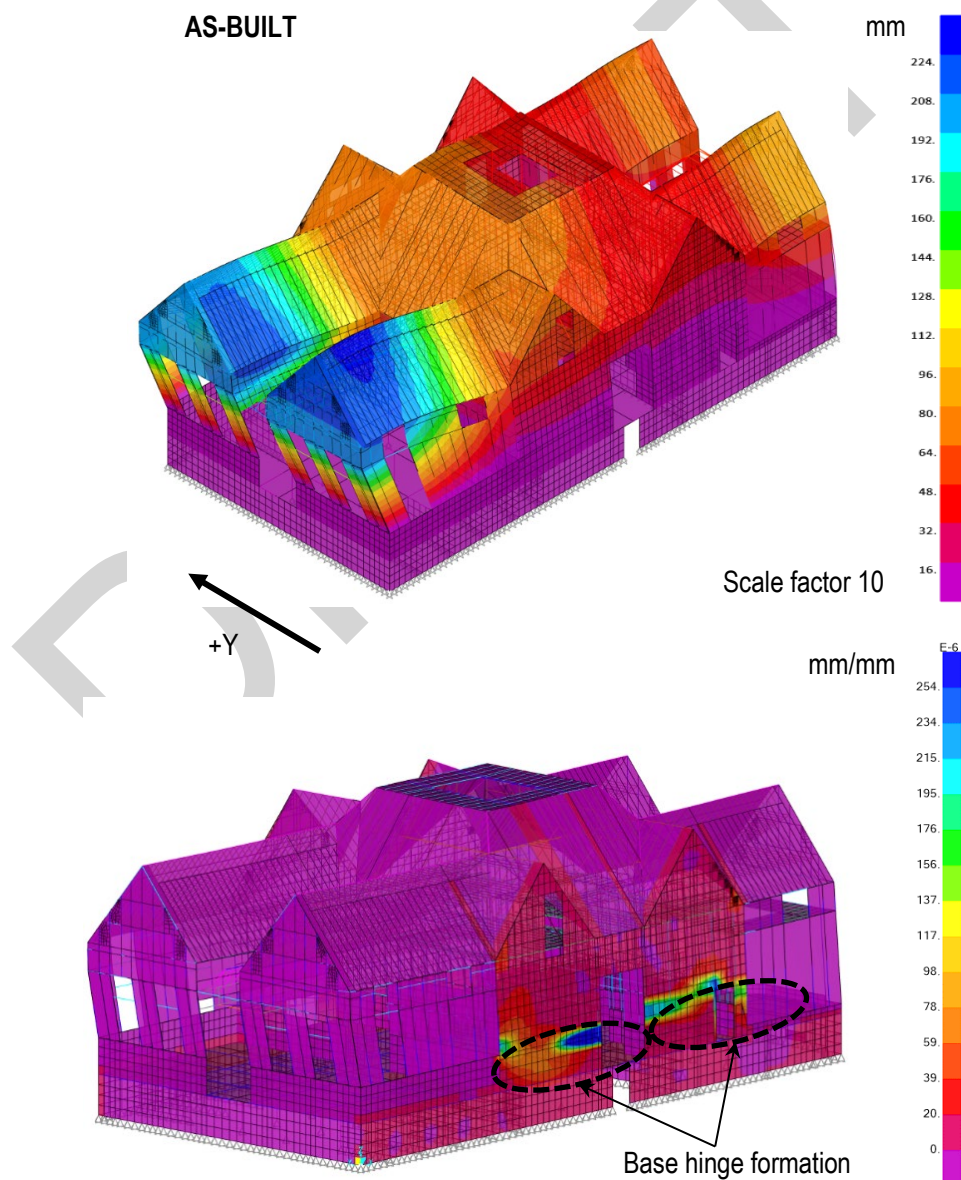


Figure 11-4: Zone 5 - Complete structure | As-built model deformed shape and principle tensile strain at the last analysis step | +Y Loading direction | Load pattern: Uniform acceleration

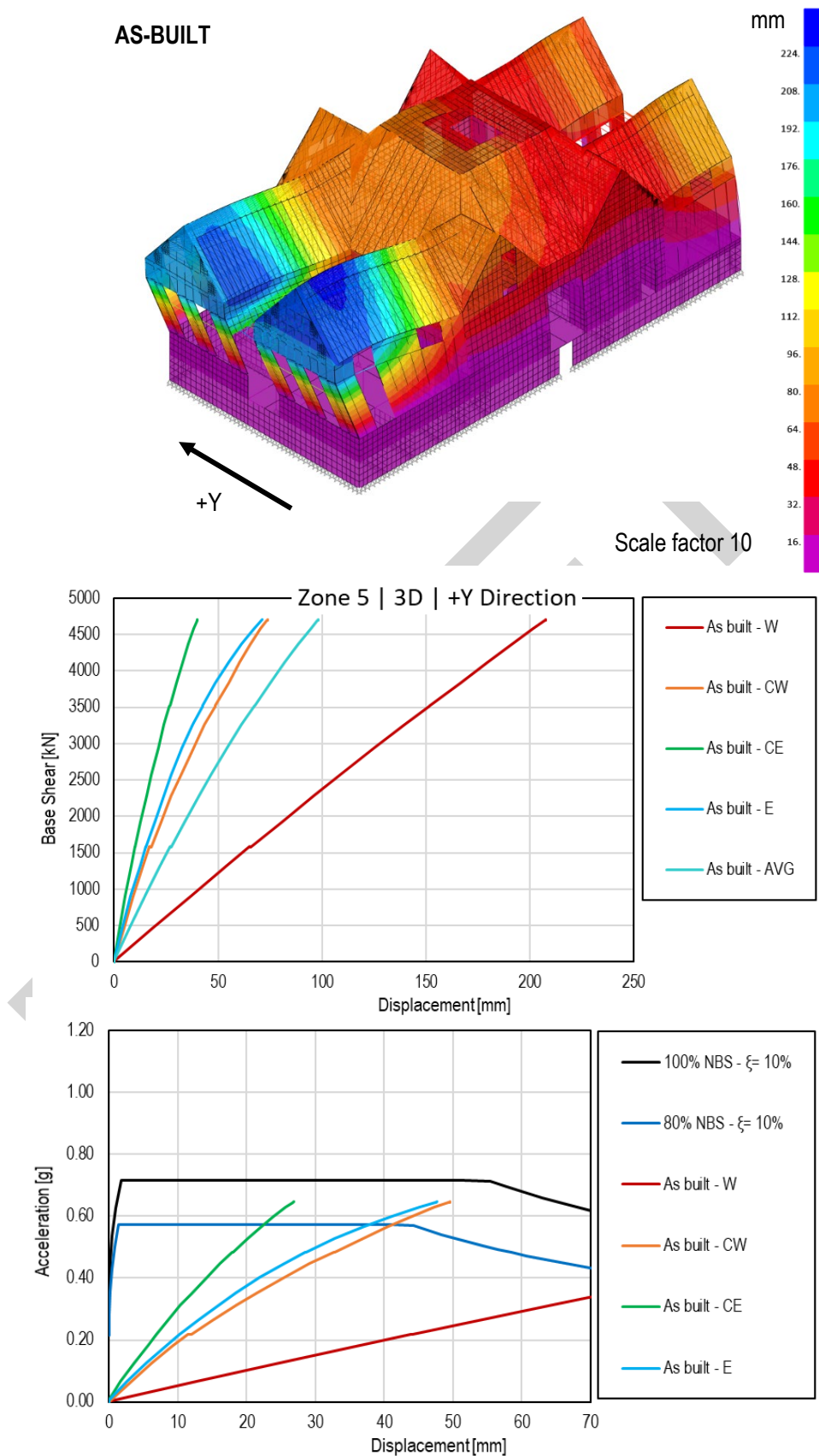


Figure 11-5: Zone 5 - Complete structure | As-built model deformed shape at the last analysis step | +Y Loading direction | Load pattern: Uniform acceleration

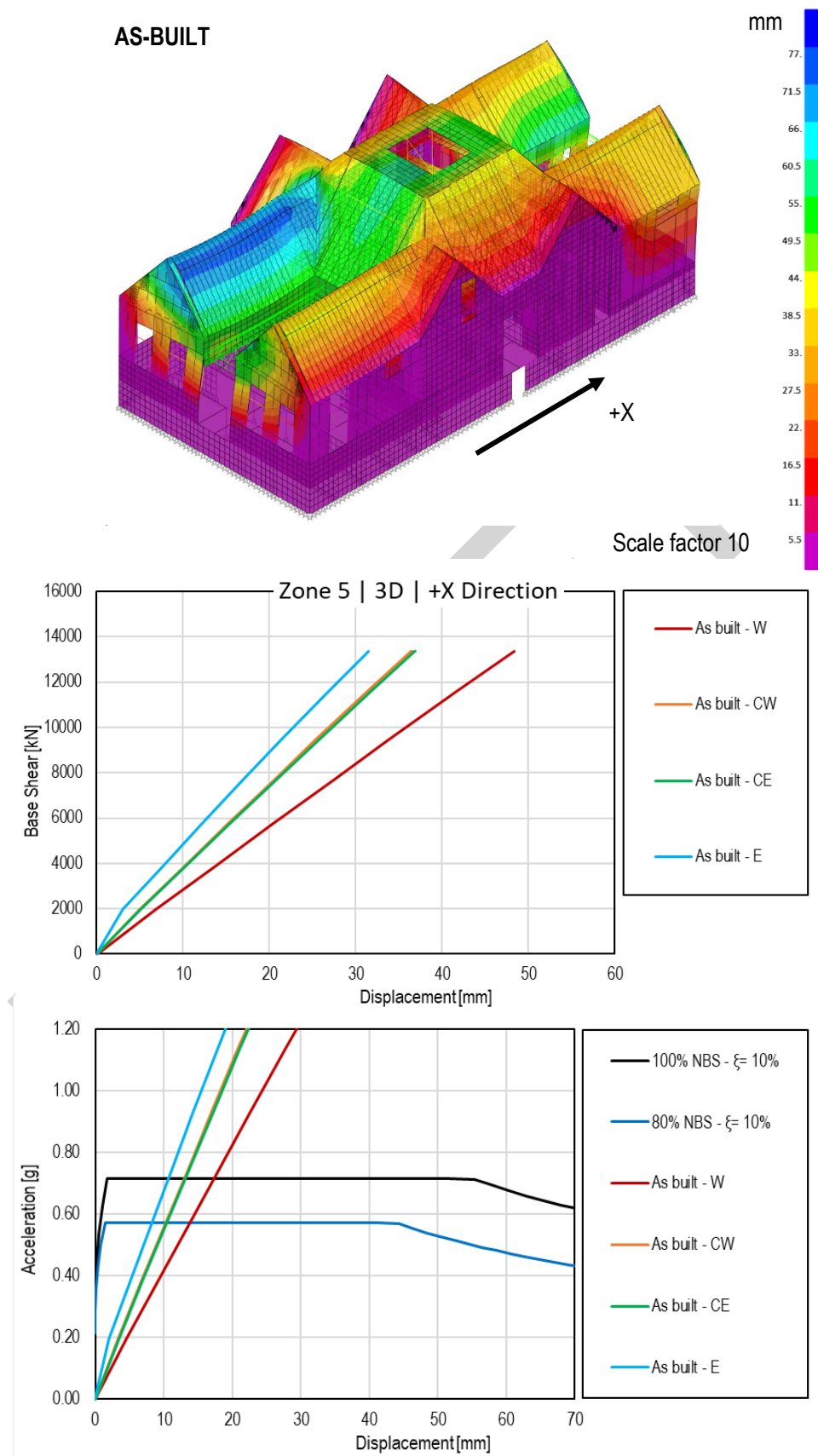


Figure 11-6: Zone 5 - Complete structure | As-built model deformed shape at the last analysis step | +X Loading direction | Load pattern: Uniform acceleration

11.1.2 Z5 | Retrofit solutions

This section presents the outcomes of the nonlinear static pushover analyses for the retrofit model. **Figure 11-7** shows a summary of all the strengthening works implemented in the numerical model to increase the lateral capacity of the building and to address the weak points of the as-built structure highlighted in the previous section.

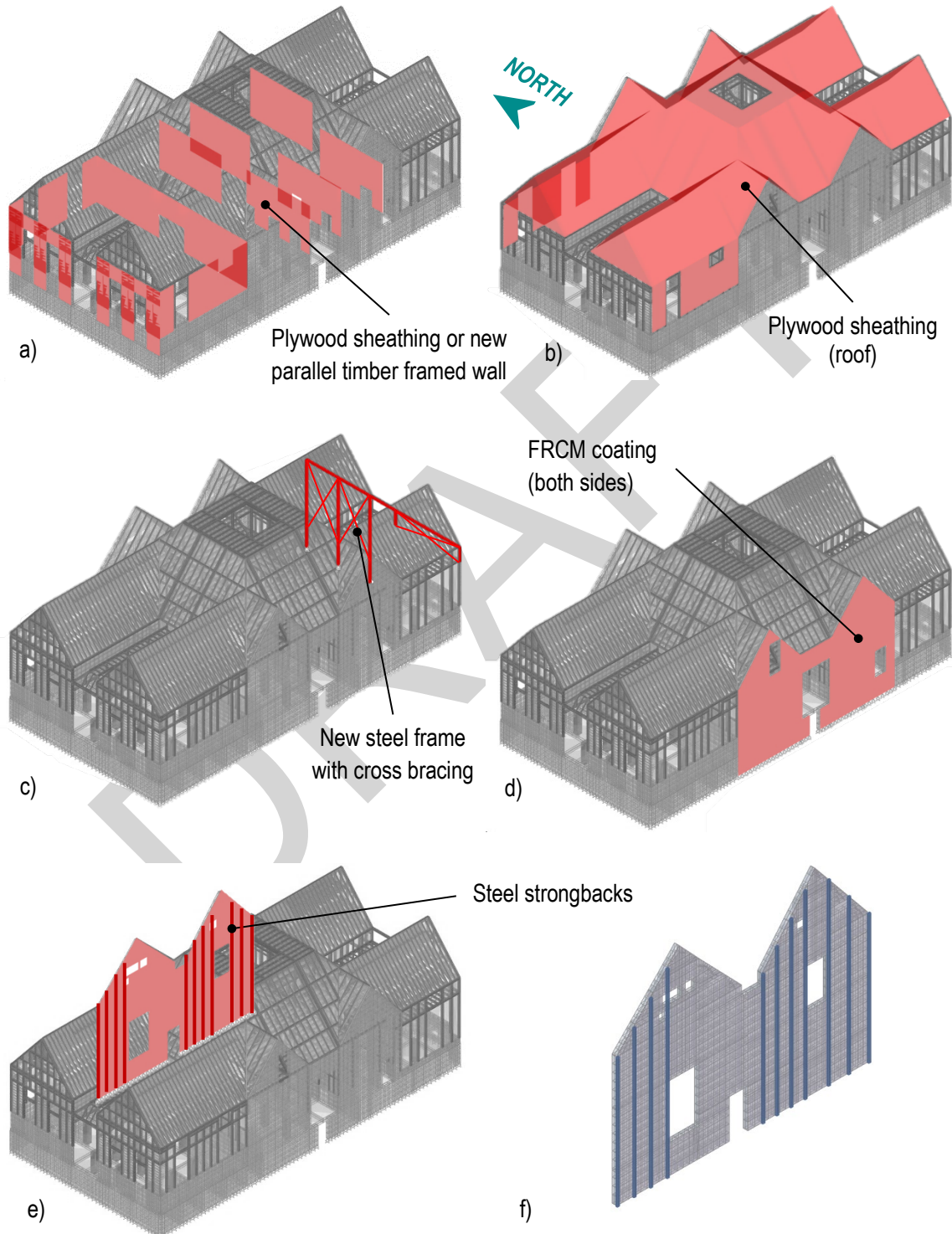


Figure 11-7: Zone 5 - Complete structure | Retrofit intervention implemented in the numerical model

These strengthening works comprise:

- Plywood sheathing overlay or new light timber framed walls (see [Figure 11-7a](#)) to relieve the existing timber framed wall (i.e., exterior timber framed wall + concrete infill panels and interior timber framed wall + lath&plaster coating) from lateral load-resisting tasks. The newly created shear walls (running in the north-south direction) are then to be adequately connected to the North and South concrete gable walls.
- Plywood sheathing reinforcement for the roof structure (see [Figure 11-7b](#)) aiming at increasing its in-plane capacity and diaphragm action to improve the diaphragm lateral load-sharing ability between the various vertical lateral resisting elements.
- New steel frame with cross bracing placed at the edge of the east extension (see [Figure 11-7c](#)). Because of the high heritage value of this portion of the building, this type of retrofit work was preferred over a more invasive solution (e.g., application of plywood panels directly over the existing timber framing). The main steel frame, which transfers the load to the foundation, is placed on the north-east corner, and it is connected to the north-west roof portion via a top horizontal beam.
- FRCM coating applied on both side of the south pumice concrete gable wall (see [Figure 11-7d](#)).
- Steel strongbacks (RHS 150x100x5 spaced every 1.2 m) applied on the south face of the north pumice concrete gable wall (see [Figure 11-7e](#) and [Figure 11-7f](#)). The steel strongbacks are connected to the concrete wall using mechanical fasteners. The grouted bars are implemented in the model via a multilinear elastic link simulating the shearing and tensile actions force-displacement curves.

[Figure 11-8](#) to [Figure 11-11](#) report the outcomes of the nonlinear static analyses carried out on the retrofit model in terms of both pushover curves and ADRS plots. Furthermore, the deformed shapes obtained at the performance point (80%NBS) step of the analyses are reported for a quick visual assessment of the building behaviour subjected to lateral load.

From the deformed shapes of the pushover analyses carried out in the Y direction ([Figure 11-8](#) and [Figure 11-9](#)) it is evident that the proposed retrofit enables a significant reduction of the large displacement demand applied to the West and East extensions in the as-built models. Additionally, the strengthening works also improve the behaviour of the building under lateral load oriented in the X direction (East-to-West). Particularly, the reinforcement of the roof diaphragm induces a more uniform response across the various parts of the structure.

Lastly, in [Figure 11-12](#) and [Figure 11-13](#) is reported the study of the out-of-plane (OOP) behaviour of the isolated North pumice concrete wall. The aim of the study is to evaluate the effect of the steel strongbacks retrofit work without the influence of other factors. The OOP analyses are carried out considering two boundary values for the concrete tensile strength of 1.0 and 0.1 MPa. When the retrofit is applied on the “tensile side” of the wall for the OOP bending action (i.e., analyses in the +Y direction) the increase in maximum capacity of the wall is significant for both the 1.0 MPa and 0.1 MPa concrete tensile strength cases. On the other hand, when the retrofit is applied on the “compressive side” of the wall (i.e., analyses in the -Y direction), the increase in maximum capacity, as expected, is much smaller for the 0.1 MPa concrete tensile strength case. This emphasizes the importance of the structural reinforcement of the shear walls running in the north-south direction (i.e., existing walls strengthened with plywood overlay or overlain and connected to new timber light-frame wall) to reduce the OOP demand applied to the concrete gable walls.

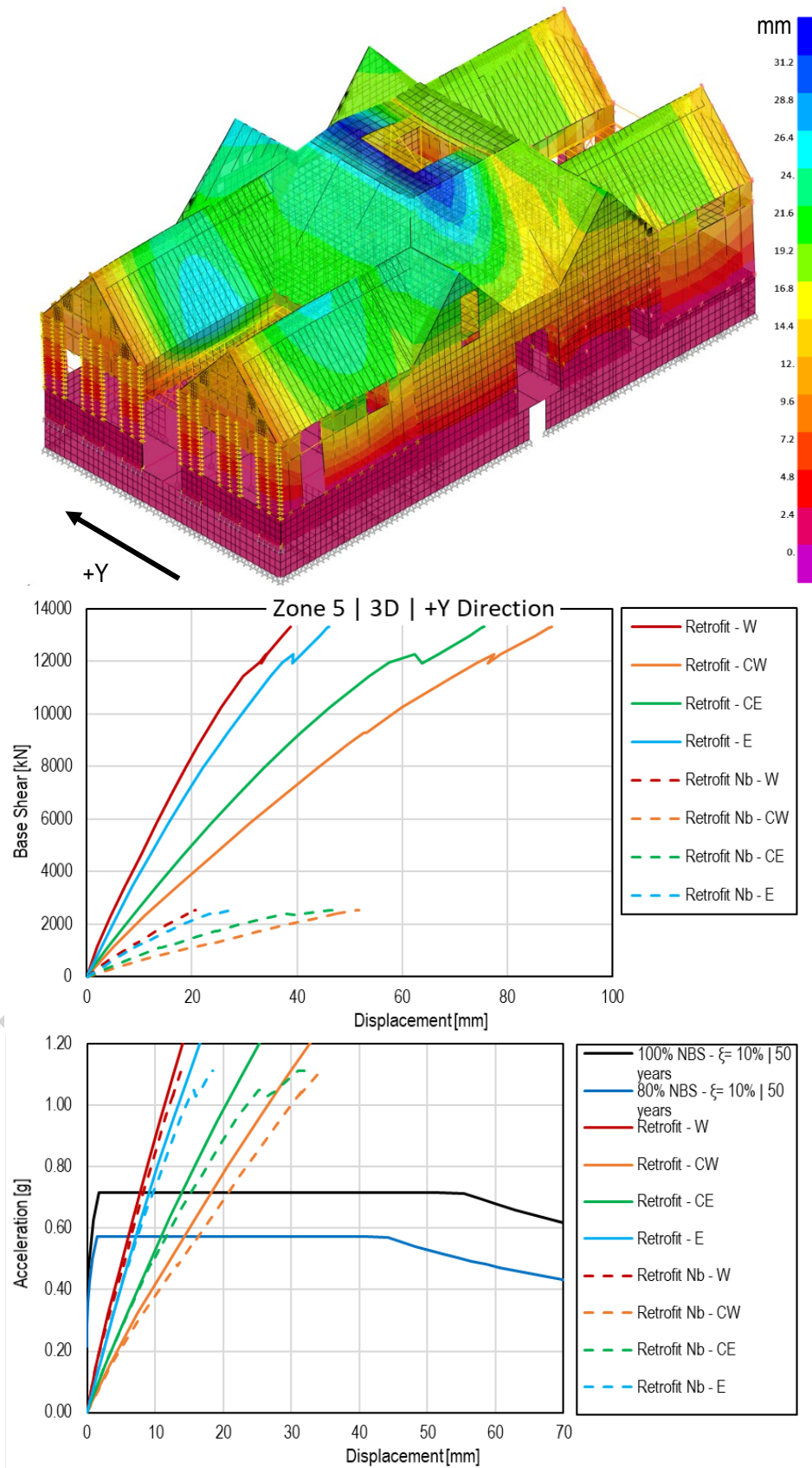


Figure 11-8: Zone 5 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

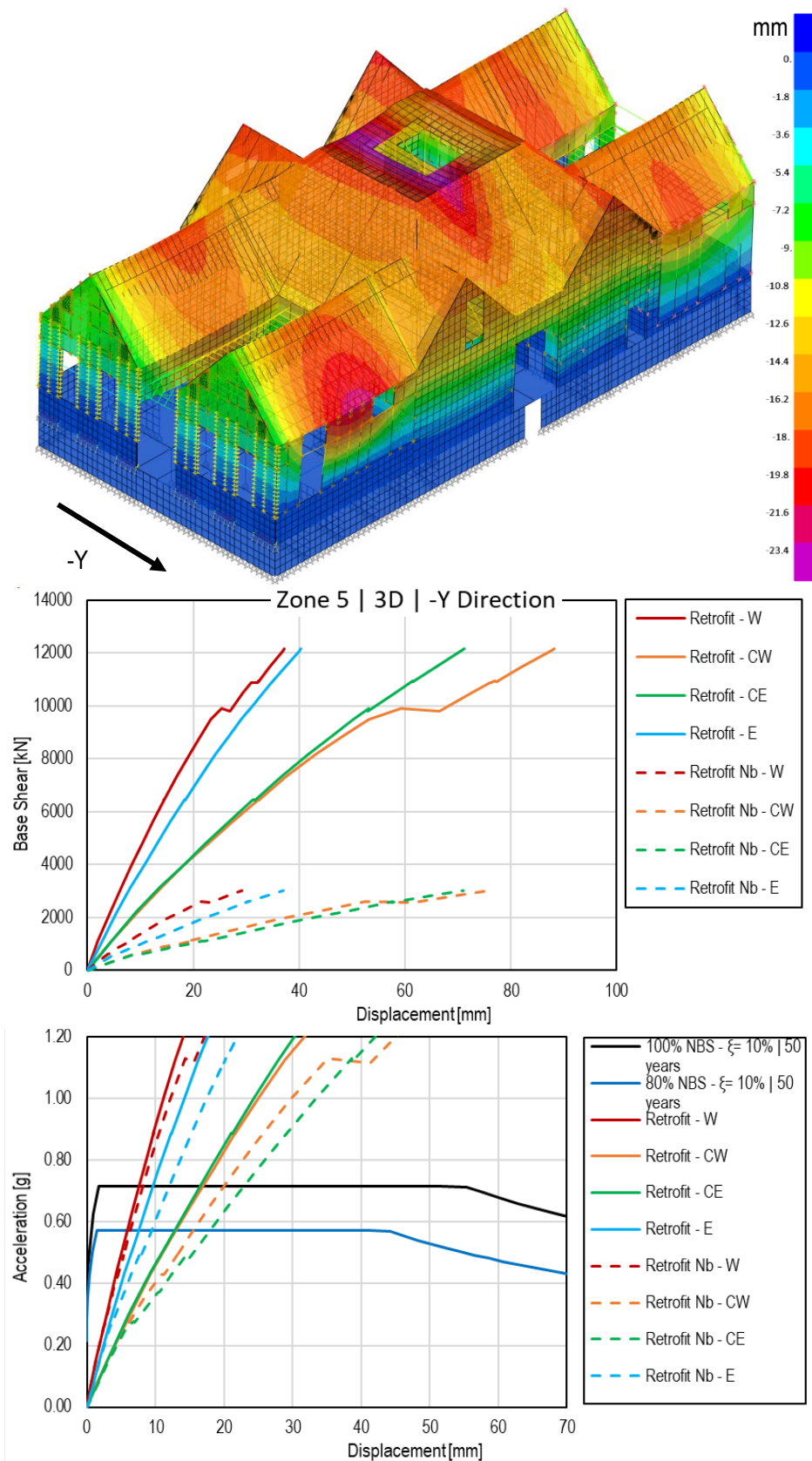


Figure 11-9: Zone 5 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -Y Loading direction | Load pattern: Uniform acceleration

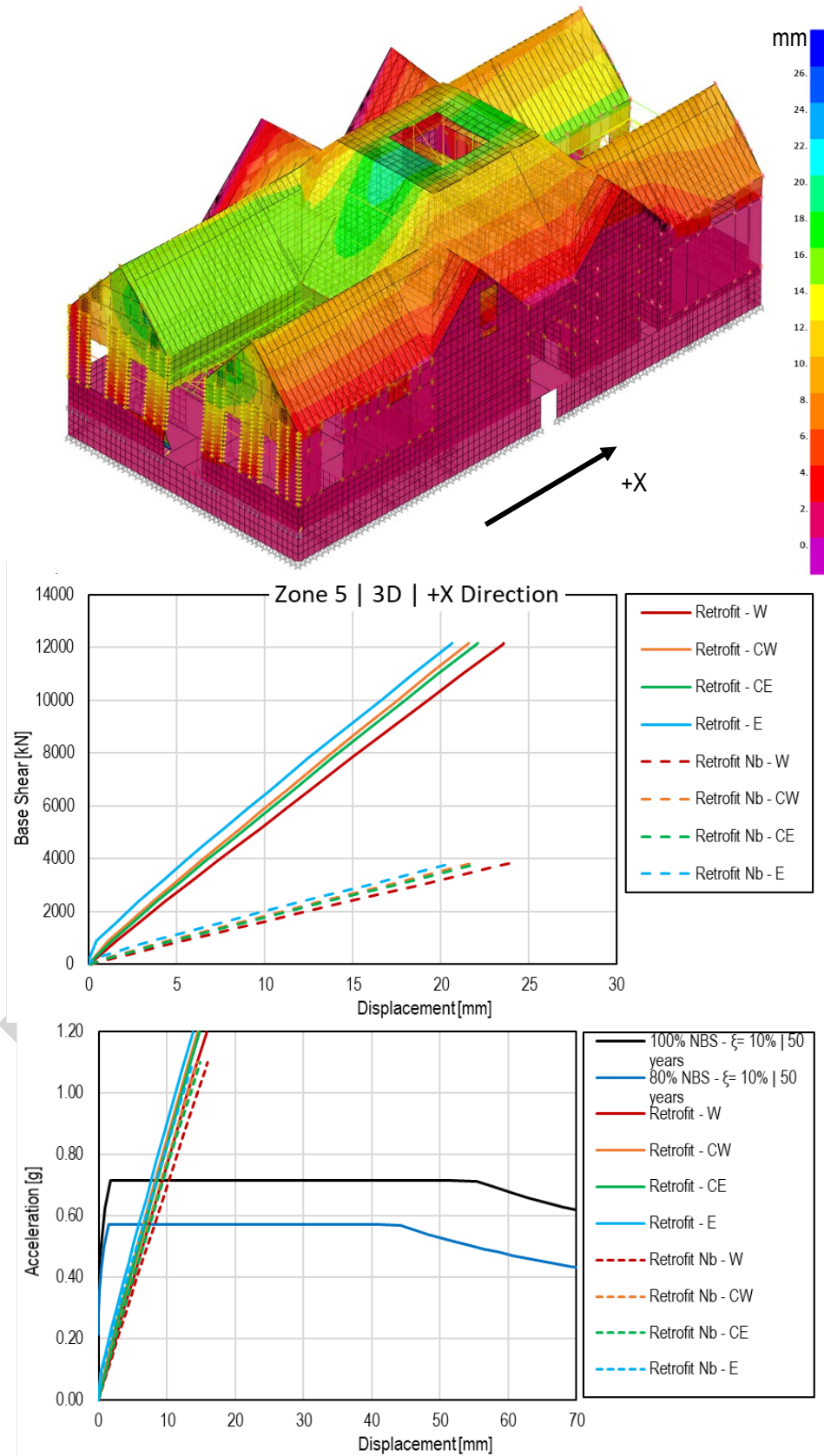


Figure 11-10: Zone 5 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

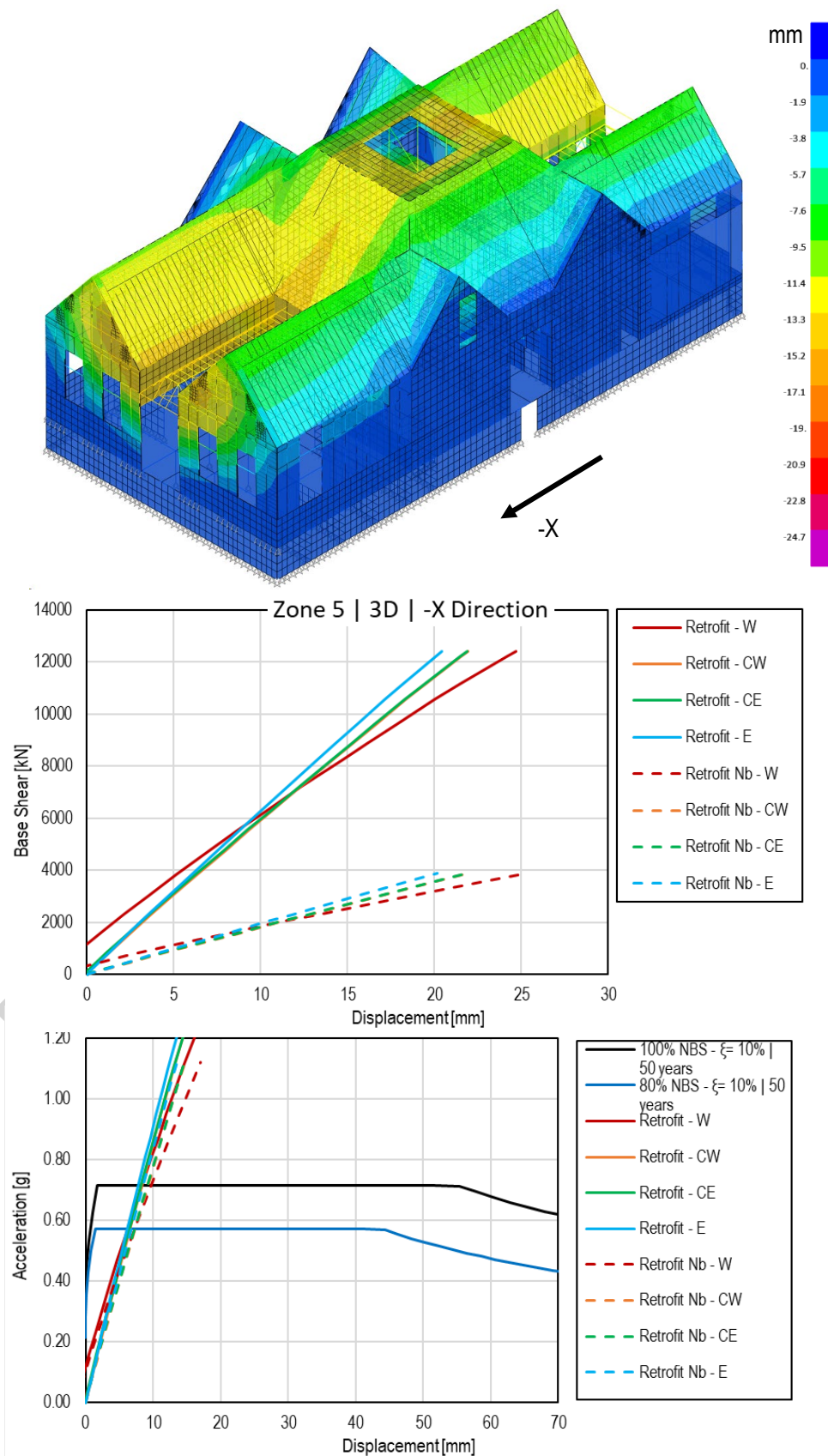


Figure 11-11: Zone 5 - Complete structure | Retrofit model deformed shape at the performance point step | Pushover curves and ADRS plots for the considered control points | -X Loading direction | Load pattern: Uniform acceleration

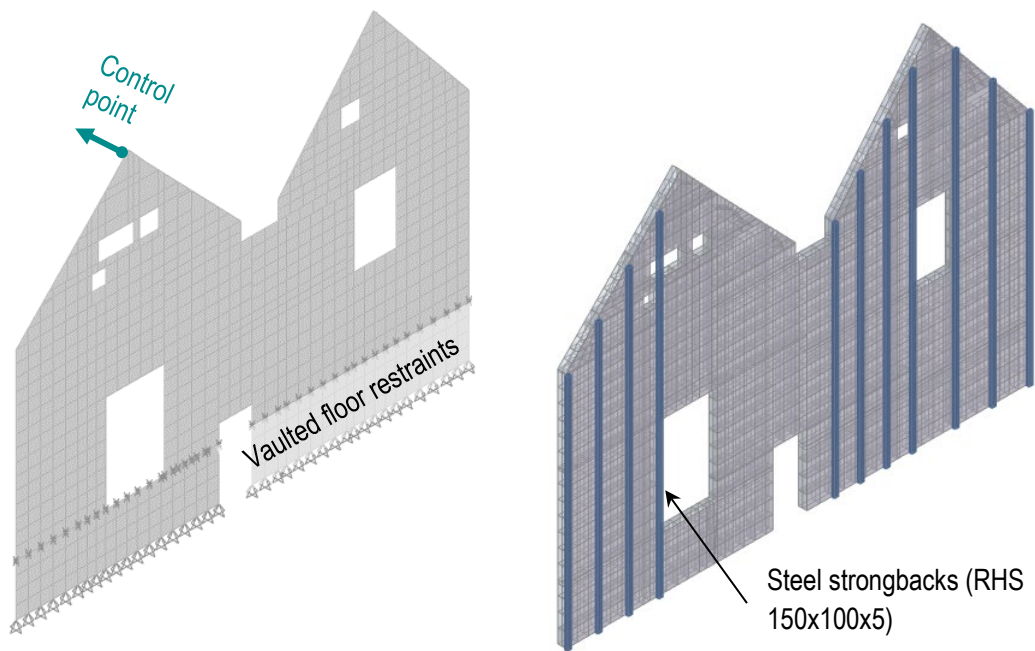


Figure 11-12: Zone 5 - North wall | Out-of-plane (OOP) analysis of the pumice concrete structure considered as a self-standing structure

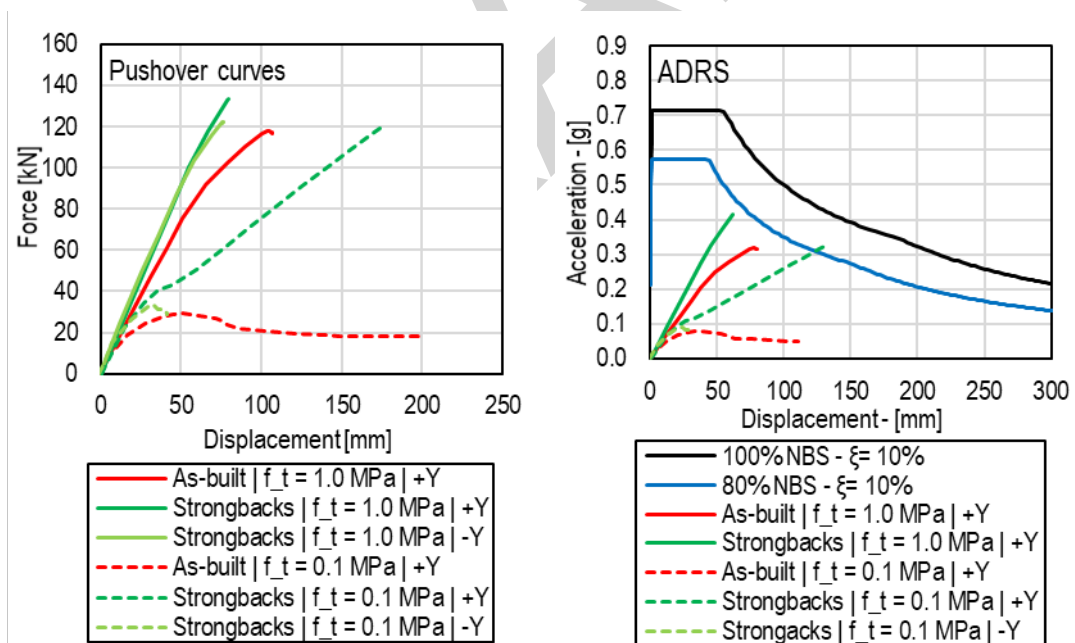


Figure 11 13: Zone 5 - North wall | Out of plane analysis results | Load pattern: Uniform acceleration

12 ZONE 2-3-4-5 | NONLINEAR STATIC ANALYSES

12.1 Zone 2-3-4-5 | Entire structure analyses (Retrofit model)

This section reports the outcomes of the nonlinear static pushover analyses performed on the original structure of the Rotorua Museum (Zones 2-3-4-5), considering that the two modern additions (Zone 1 built c. 2011, and Zone 6 built c. 2007) have been built adopting appropriate seismic gaps with the adjacent historic building and are treated as separated self-standing structures. For the lateral loading pattern the uniform acceleration option was selected. The vertical distribution of lateral load proportional to the fundamental mode could not be used because the modal participating mass ratio is below 75% in both directions. Also, it was not possible to define a lateral load pattern proportional to a combination of modal responses because, given the complexity of the structure, it was unviable to capture 90% of the total mass of the building within a reasonable number of modes. More details about the dynamic response of the building are provided in the Nonlinear Dynamic Analysis chapter (see Section 13 of the report). In addition to the complete model, another model was defined which eliminated the basement structure (labelled as “No-basement model”, [Figure 12-1](#)). The no-basement model was examined again using the uniform acceleration load pattern. This was done to provide additional insight into the behaviour of the structure without the influence of the basement structure, which can be large due to the loading condition being proportional to the mass distribution.

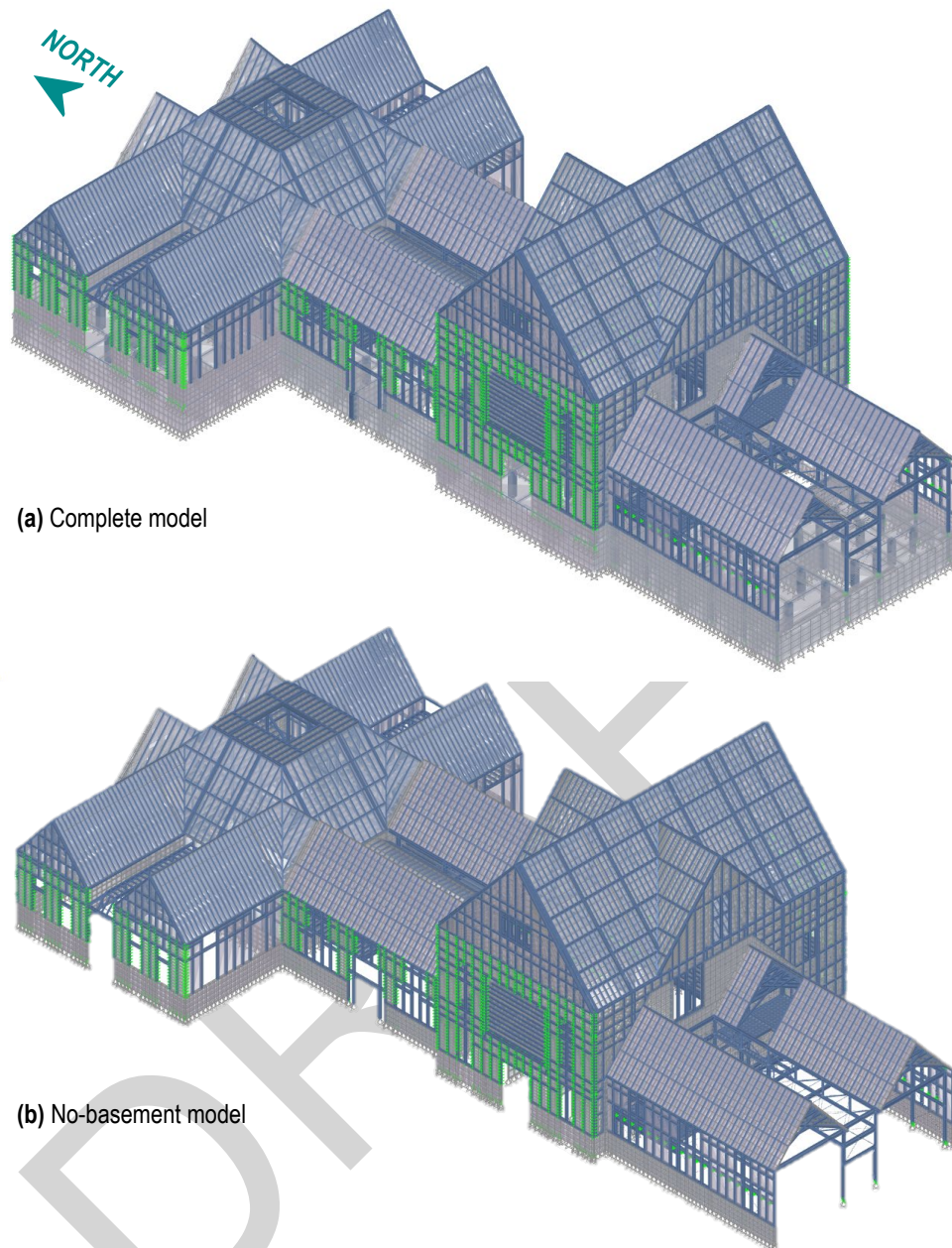


Figure 12-1: Numerical models used for the analysis of the original structure of the Rotorua Bath House building (zone 2-3-4-5): (a) Complete model (b) No-basement model

Figure 12-2 shows the tributary mass areas adopted to obtain the SDOF curves in the ADRS plots. In both directions the tributary area comprises exclusively the mass of the historic portion of the building (Zones 2-3-4-5). All the walls acting as the lateral load resisting system are presented with dark red lines and the tributary area considered for the loading is highlighted in light red colour. Note that the interface between Zone 3 and Zone 2 has parallel lateral systems. The south wall of Zone 3 is a concrete shear wall and the lateral system for Zone 2 at its north side (that is parallel to the south wall of Zone 3) is steel framing with cross-bracing.

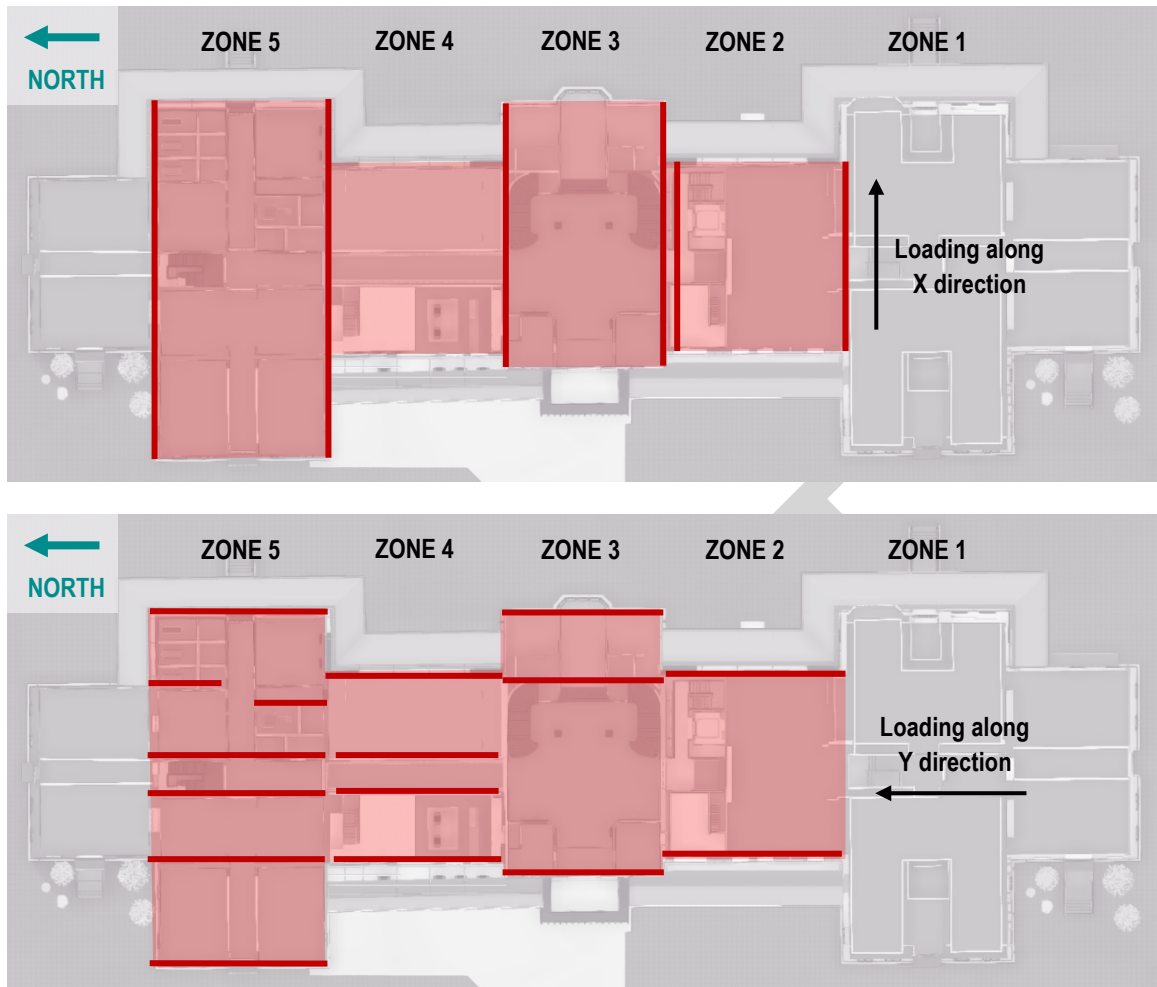


Figure 12-2: Zone 2-3-4-5 - Complete structure | Tributary mass areas used to define the ADRS plot | Loading direction along the X and Y direction

Figure 12-3 reports the control points selected to display the pushover analyses outcomes. The displacements are taken at the highest portion of the building and spread across the structure's footprint to account for any possible irregular behaviour of the structure during the interpretation of the results. The selection of the control points shown in **Figure 12-3** matches the ones adopted for the analyses of the single isolated zones. Therefore, it is possible to make a direct comparison between the outcomes of the retrofit models of the single isolated zones and the retrofit model of the entire structure reported in this section. For the sake of clarity in the pushover curves and ADRS plots charts the mean displacement values of each selected group of control points are represented.

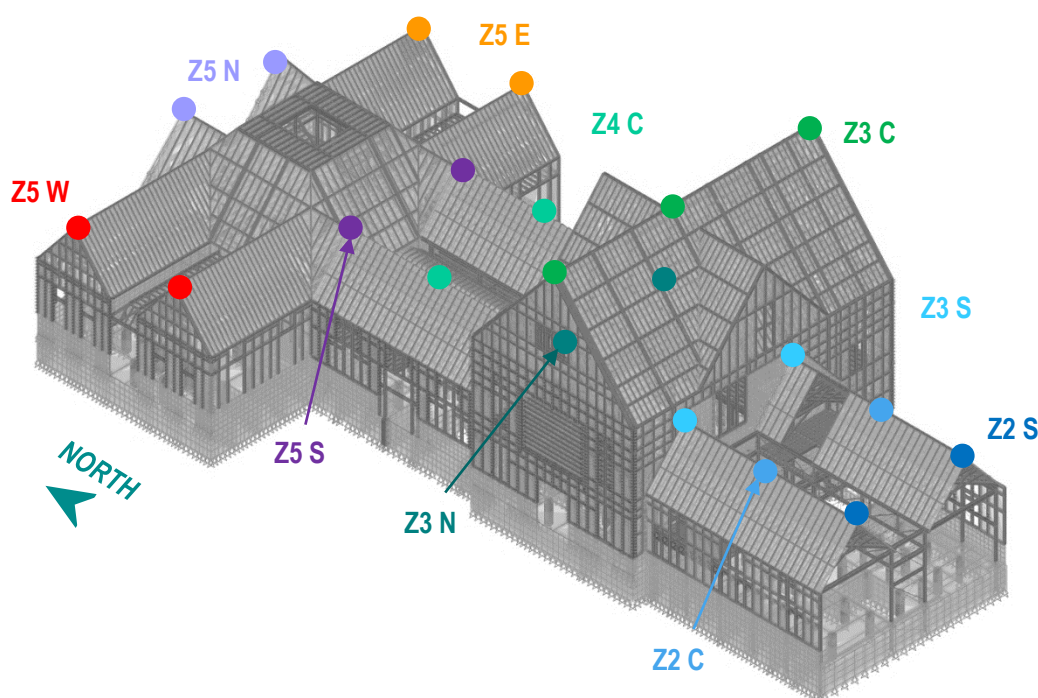


Figure 12-3: Zone 2-3-4-5 – Complete structure | Control points adopted for the nonlinear static pushover analyses (NLSPA)

The outcomes of the nonlinear static analyses carried out on the retrofit model are reported from [Figure 12-4](#) to [Figure 12-11](#). For a clear and thorough representation of the results the deformed shapes of the building (taken at the performance point of the analysis for 80% NBS) are displayed from two different viewpoints: western view and eastern view ([Figure 12-4](#), [Figure 12-6](#), [Figure 12-8](#), and [Figure 12-10](#)). It should be noted that the scale factor adopted to represent the displacements is extremely high (deformed shape amplified 60 times) to make the actual deformation distinguishable from the undeformed condition.

It can be observed that, not surprisingly, the main deformation patterns retrace what was already reported for the single isolated zones. For lateral loads acting in the Y direction (South-to-North) it is evident that the main deformation pattern is the displacement of the Zone 3 roof portion ([Figure 12-4](#) and [Figure 12-6](#)). From a retrofit design perspective this means that the strengthening of the connection between the timber framed roof portion and the underlying concrete gable walls could be of great significance for the safety of the building. Modelling the complete Zone 2-3-4-5 structure also enables an examination of the contribution of the Intermediate South (Zone 2) and Intermediate North (Zone 4) wings on the out-of-plane behaviour of the two large pumice concrete gable walls of Zone 3. For a quick comparison, the performance point displacement measured at the top of the concrete gable wall (i.e., control points Z3N and Z3S as reported in [Figure 12-3](#)) is equal to 52 mm for the Zone 3 isolated model and equal to 26 mm for the Zone 2-3-4-5 complete structure model. Another remark can be done observing the pushover curves for the different control points selected for the structure. The West and East façades of the North wing section (Zone 5) are the part of the entire building which exhibit the smallest displacement values when interpreted from the uniform acceleration pushover curves ([Figure 12-5](#) and [Figure 12-7](#)). However, this is mainly due to the fact that most of the mass of the building is concentrated along the “corridor” delineated by the West and East walls of the two intermediate wings (Zones 2 and 4). Hence, all the structural elements comprised in that “corridor” sector are more loaded during the pushover analysis carried out with the uniform acceleration pattern. Note that, for the design of the structural reinforcement of the West and East façades of Zone 5, the outcomes from the Zone 5 isolated model and from the nonlinear dynamic analysis annex are used, to account for the worst-case scenario.

For lateral loads acting in the X direction (East-to-West) the main deformation patterns visible from the model ([Figure 12-8](#) and [Figure 12-10](#)) are the out-of-plane displacements of the walls with the largest unsupported span (i.e., Zone 3 bottom portion of the West and East façade gables and Zone 4 East and West walls). As expected the most deformable sections of the building are the central portion of Zone 2, Zone 3 and Zone 4 (control points Z2 C, Z3 C and Z4 C in

Figure 12-9 and **Figure 12-11**) and the South end of Zone 2 (control points Z2 S in **Figure 12-9** and **Figure 12-11**). A possible explanation for this is that the remaining zones can count on the large in-plane stiffness of the four pumice concrete gable walls (control points Z3S, Z3N, Z5S and Z5N). On the other hand, the central portion of Zones 2, 3 and 4 must rely on the out-of-plane capacity of the western and eastern walls and on the diaphragm action of the joining horizontal structure in order to transfer part of the load to the four pumice concrete gable walls. The deformability of the Zone 2 south end is due to the limited stiffness of the steel cross-bracing compared to the other portion of the building with concrete walls. Nevertheless, its lateral displacement level at the performance point of the analysis can be considered as acceptable ($\approx 10 \div 15$ mm, corresponding drift levels $\approx 0.1 \div 0.15\%$).

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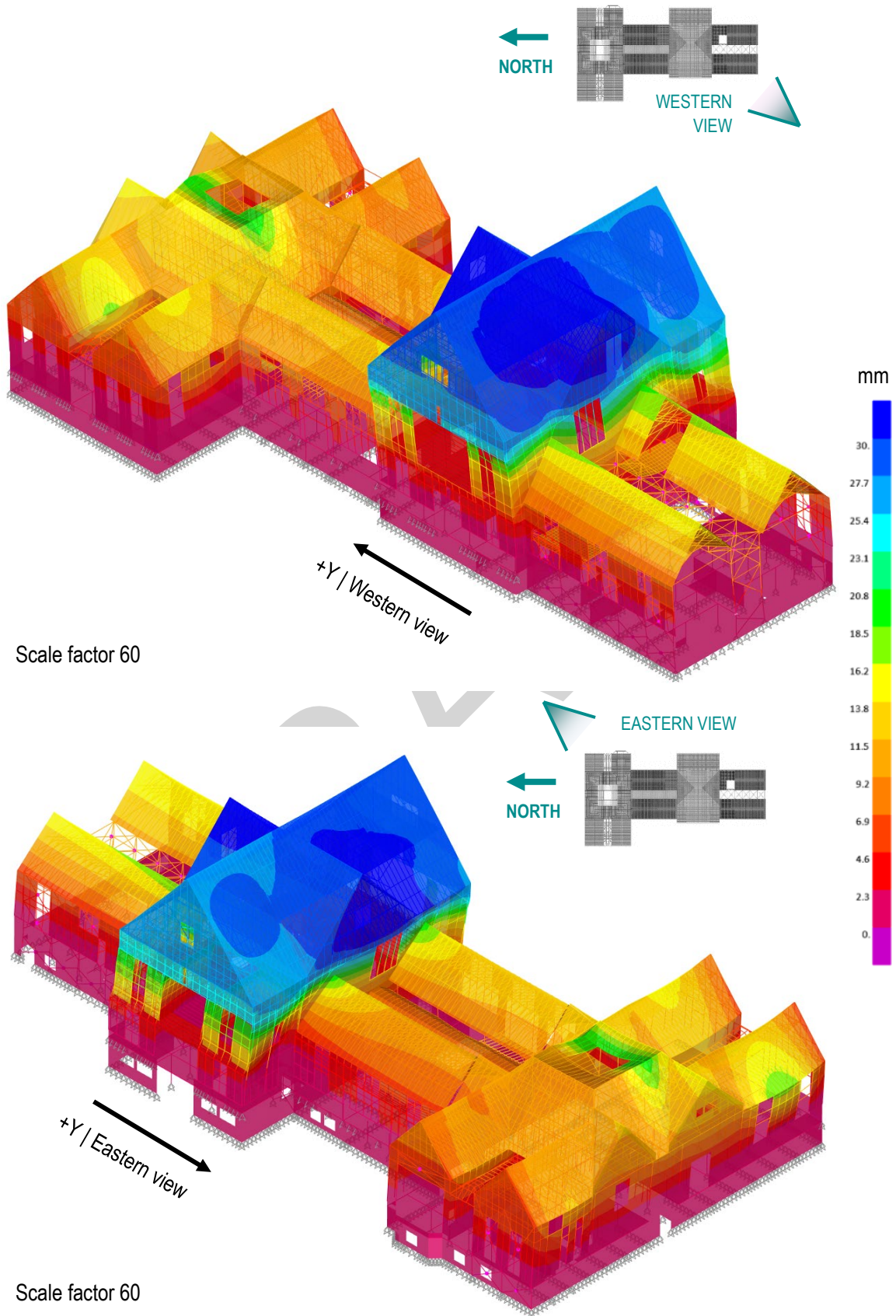


Figure 12-4: Zone 2-3-4-5 - Complete structure | retrofit model deformed shape taken at performance point of the analysis (scale factor 60) | +Y Loading direction | Load pattern: Uniform acceleration

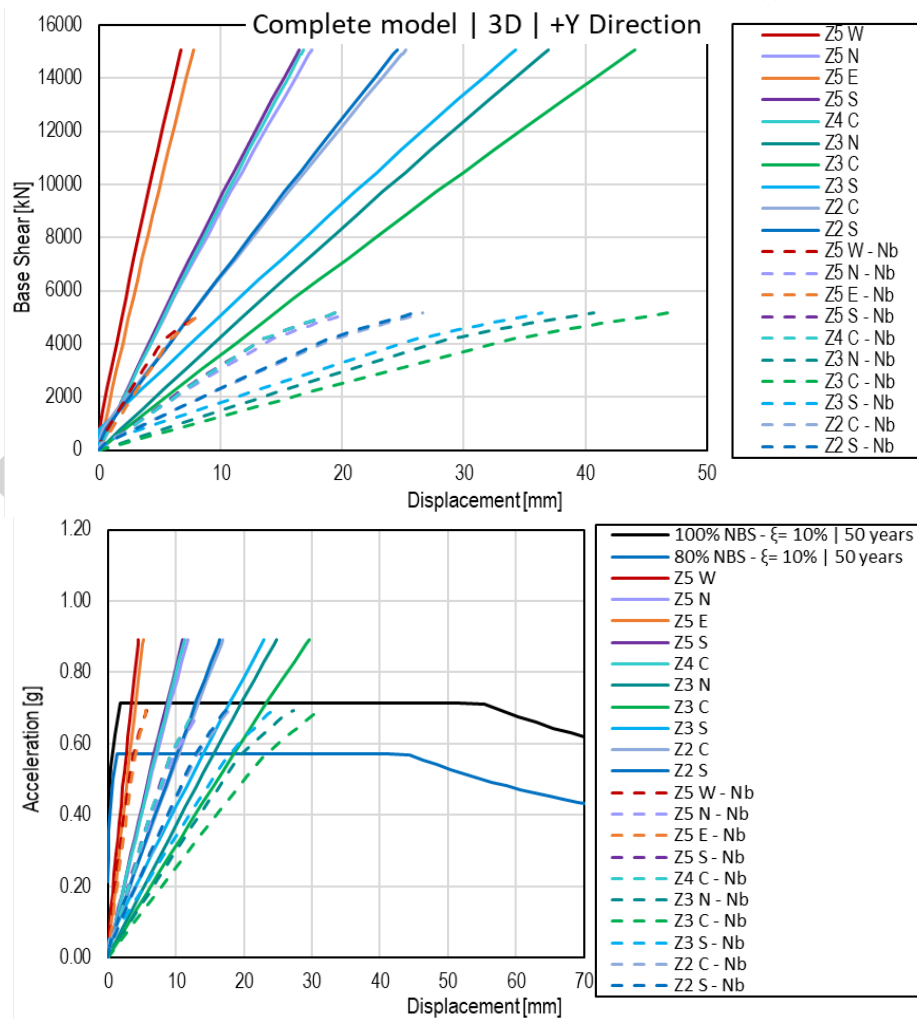
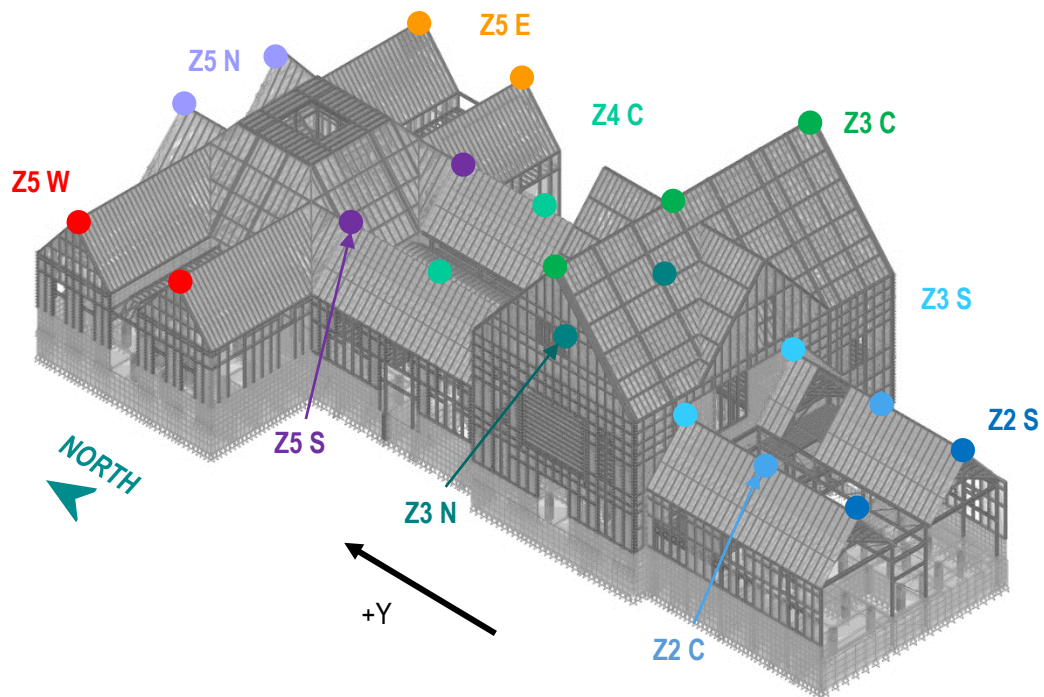


Figure 12-5: Zone 2-3-4-5 - Complete structure | Retrofit model pushover curves and ADRS plot for the considered control points | +Y Loading direction | Load pattern: Uniform acceleration

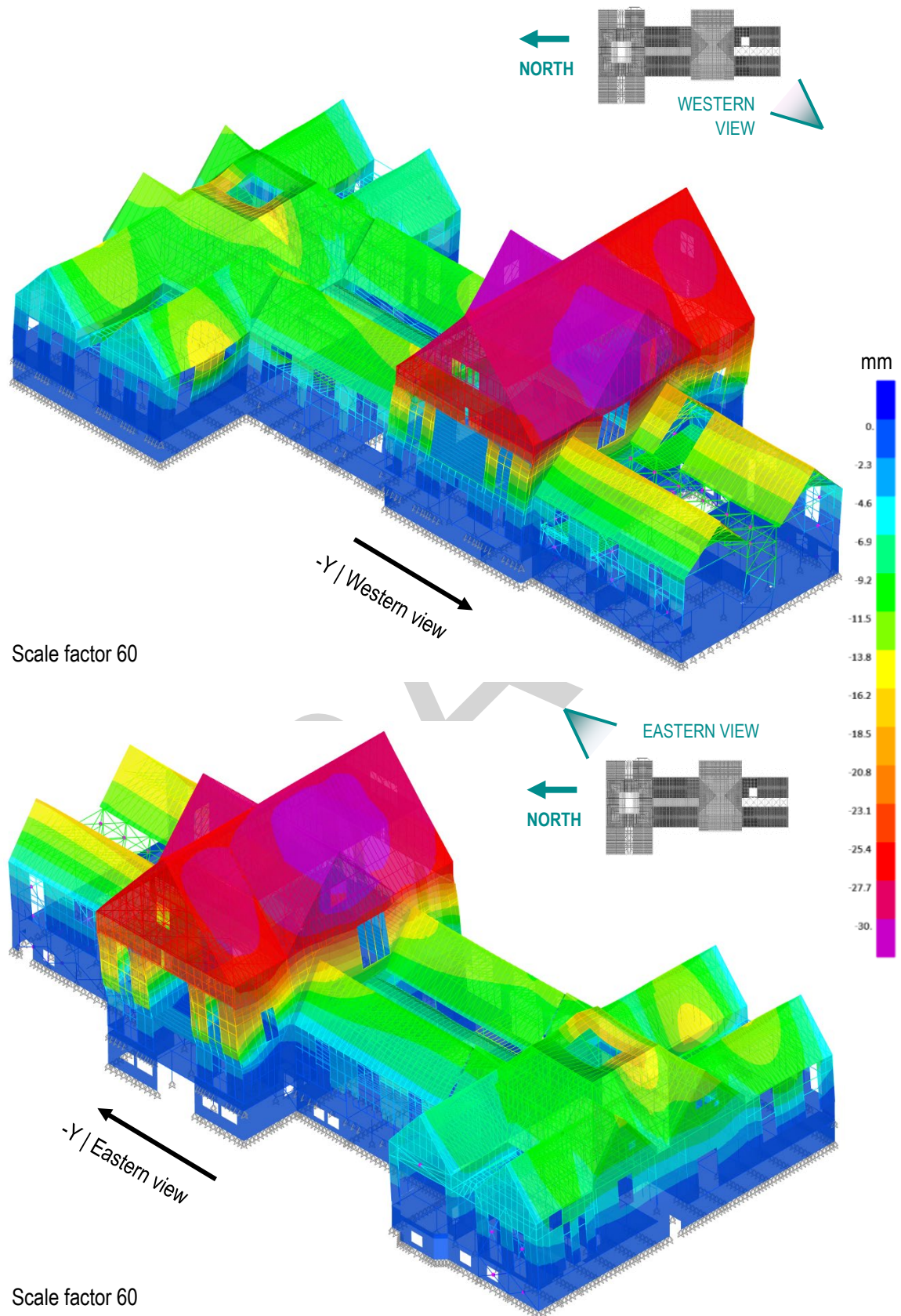


Figure 12-6: Zone 2-3-4-5 - Complete structure | retrofit model deformed shape taken at performance point of the analysis (scale factor 60) | -Y Loading direction | Load pattern: Uniform acceleration

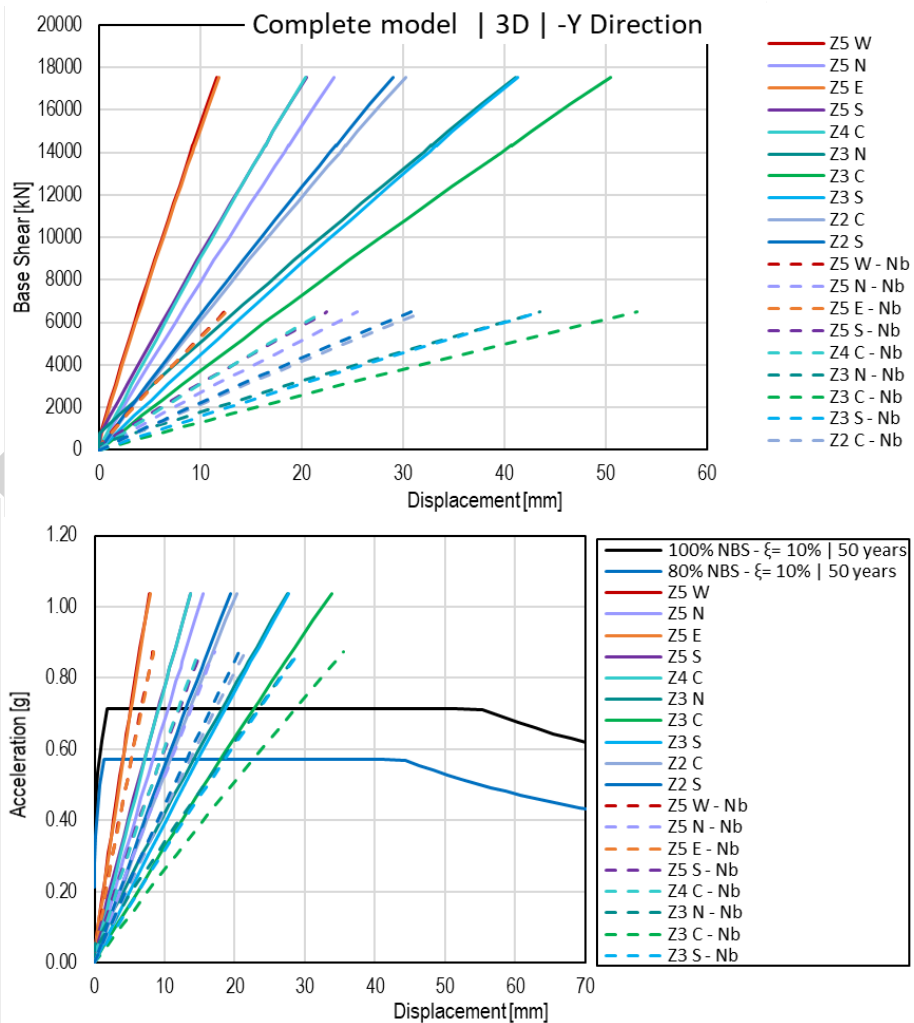
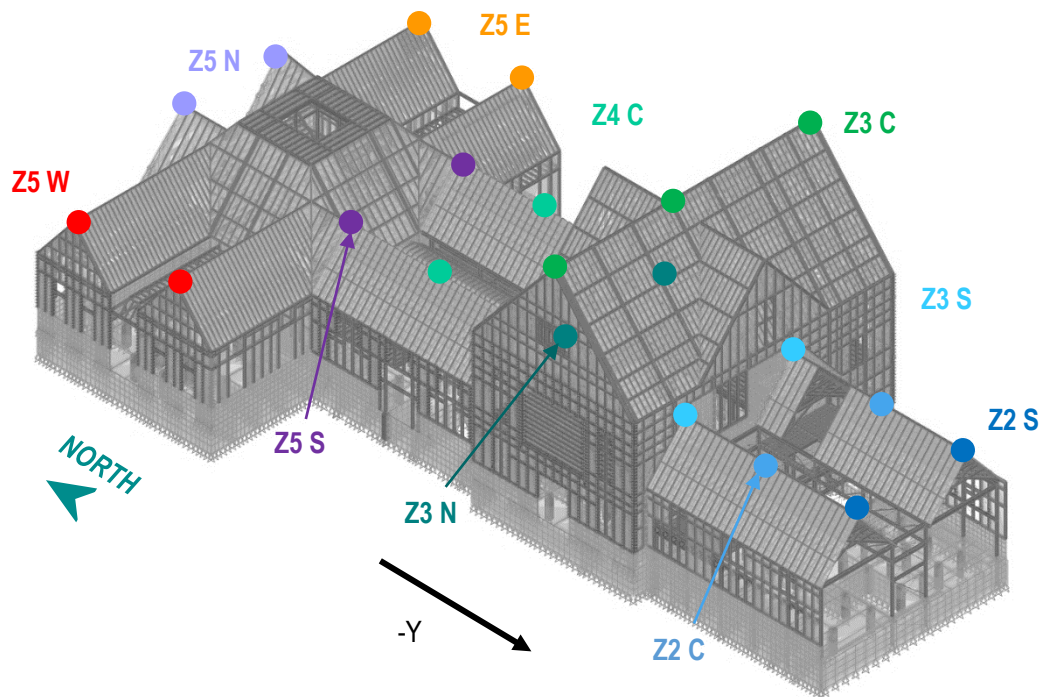


Figure 12-7: Zone 2-3-4-5 - Complete structure | Retrofit model pushover curves and ADRS plot for the considered control points | -Y Loading direction | Load pattern: Uniform acceleration

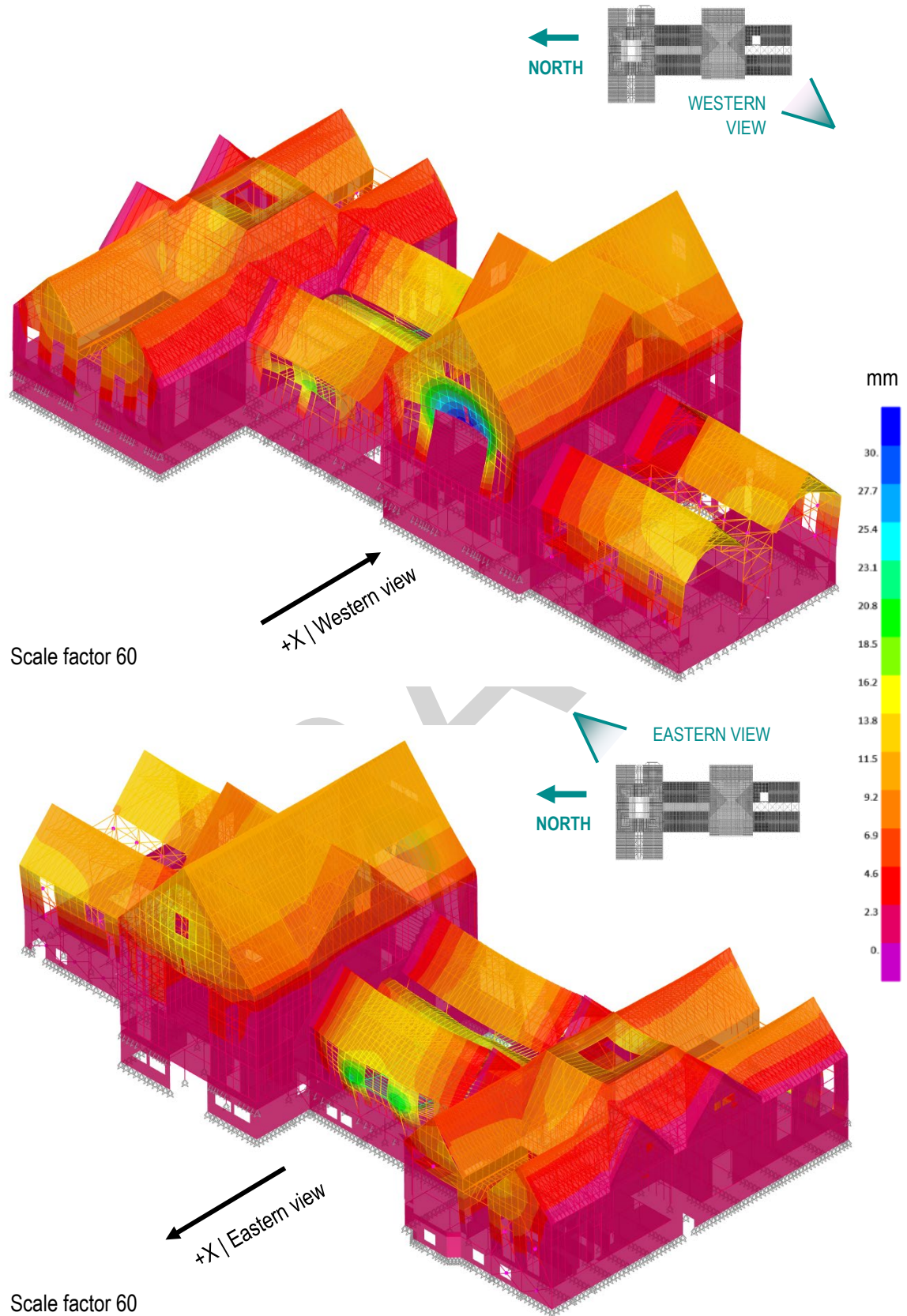


Figure 12-8: Zone 2-3-4-5 - Complete structure | retrofit model deformed shape taken at performance point of the analysis (scale factor 60) | +X Loading direction | Load pattern: Uniform acceleration

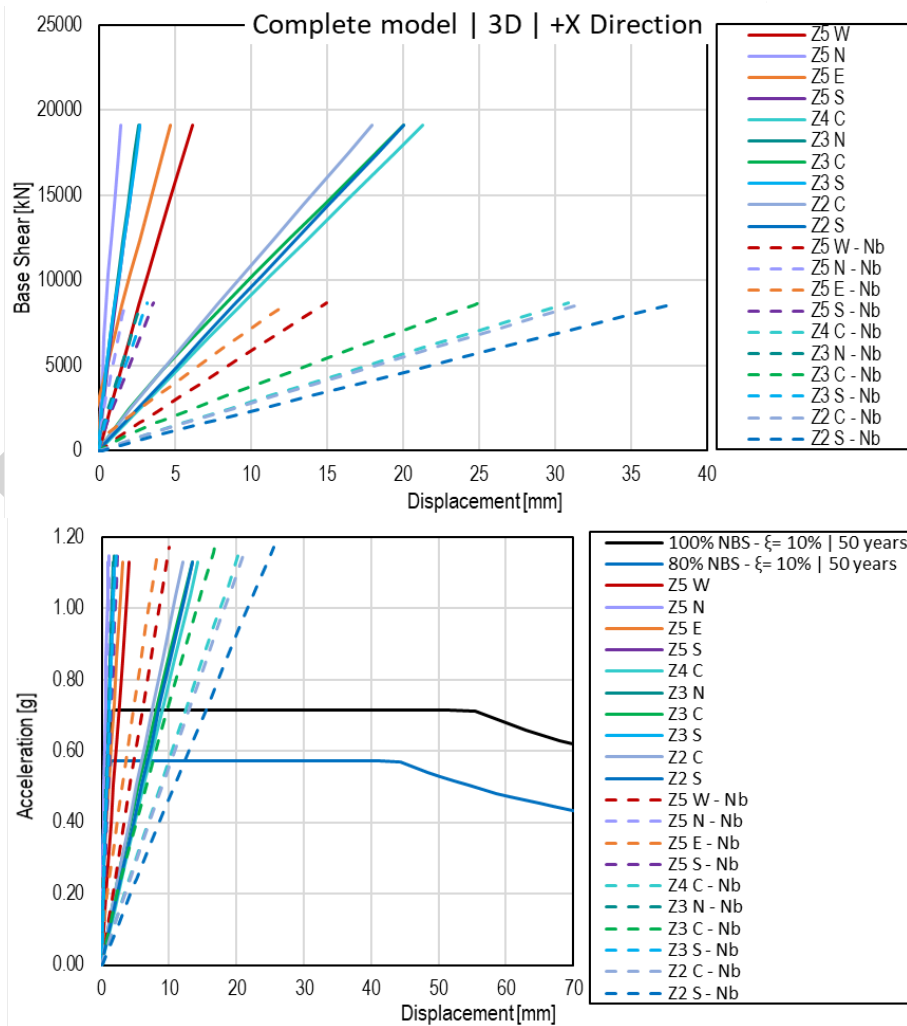
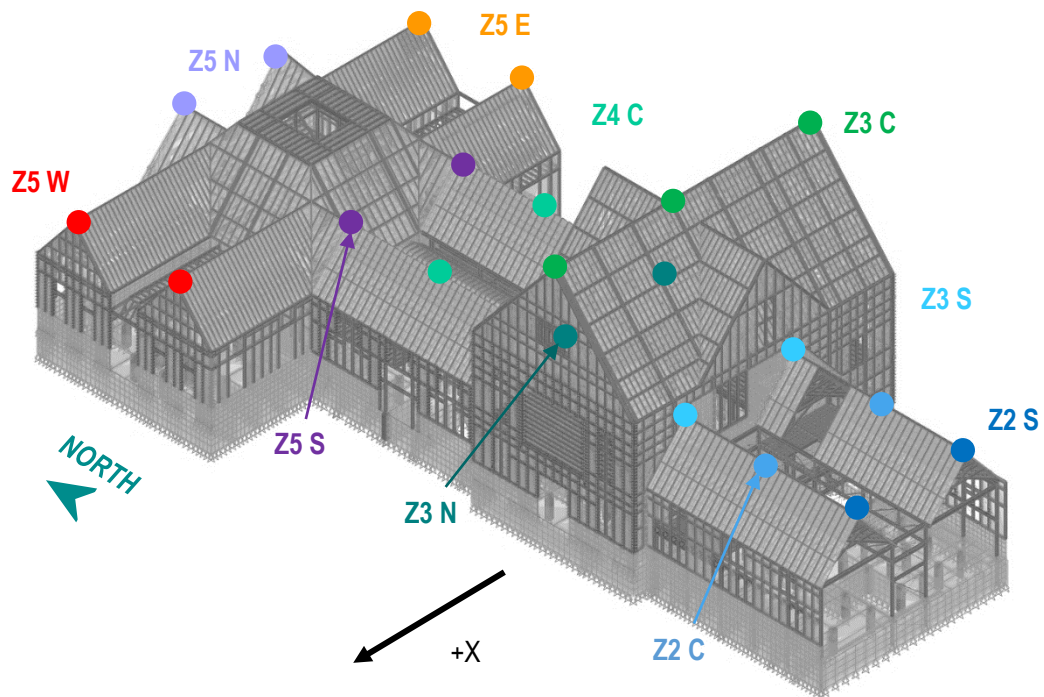


Figure 12-9: Zone 2-3-4-5 - Complete structure | Retrofit model pushover curves and ADRS plot for the considered control points | +X Loading direction | Load pattern: Uniform acceleration

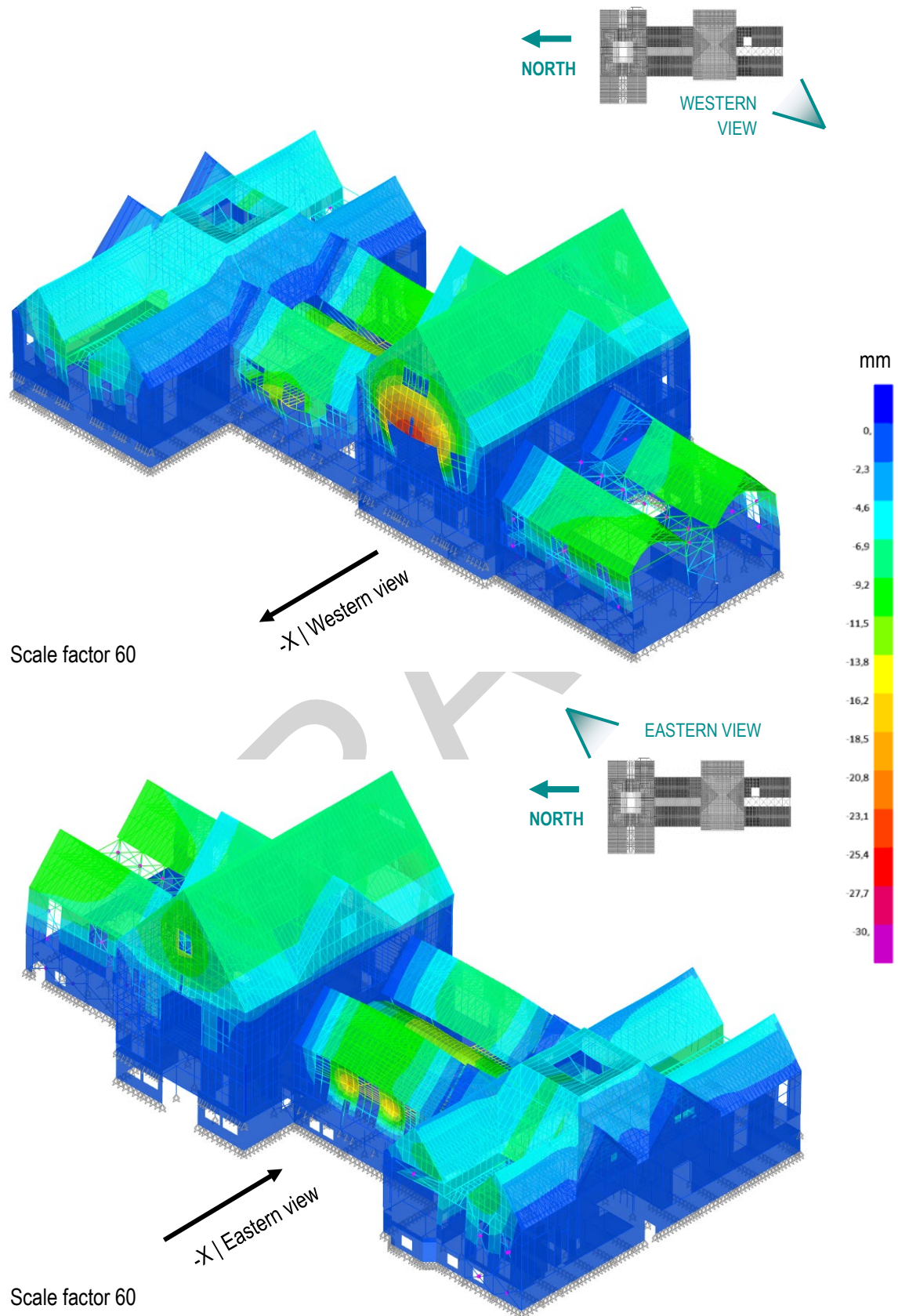


Figure 12-10: Zone 2-3-4-5 - Complete structure | retrofit model deformed shape taken at performance point of the analysis (scale factor 60) | -X Loading direction | Load pattern: Uniform acceleration

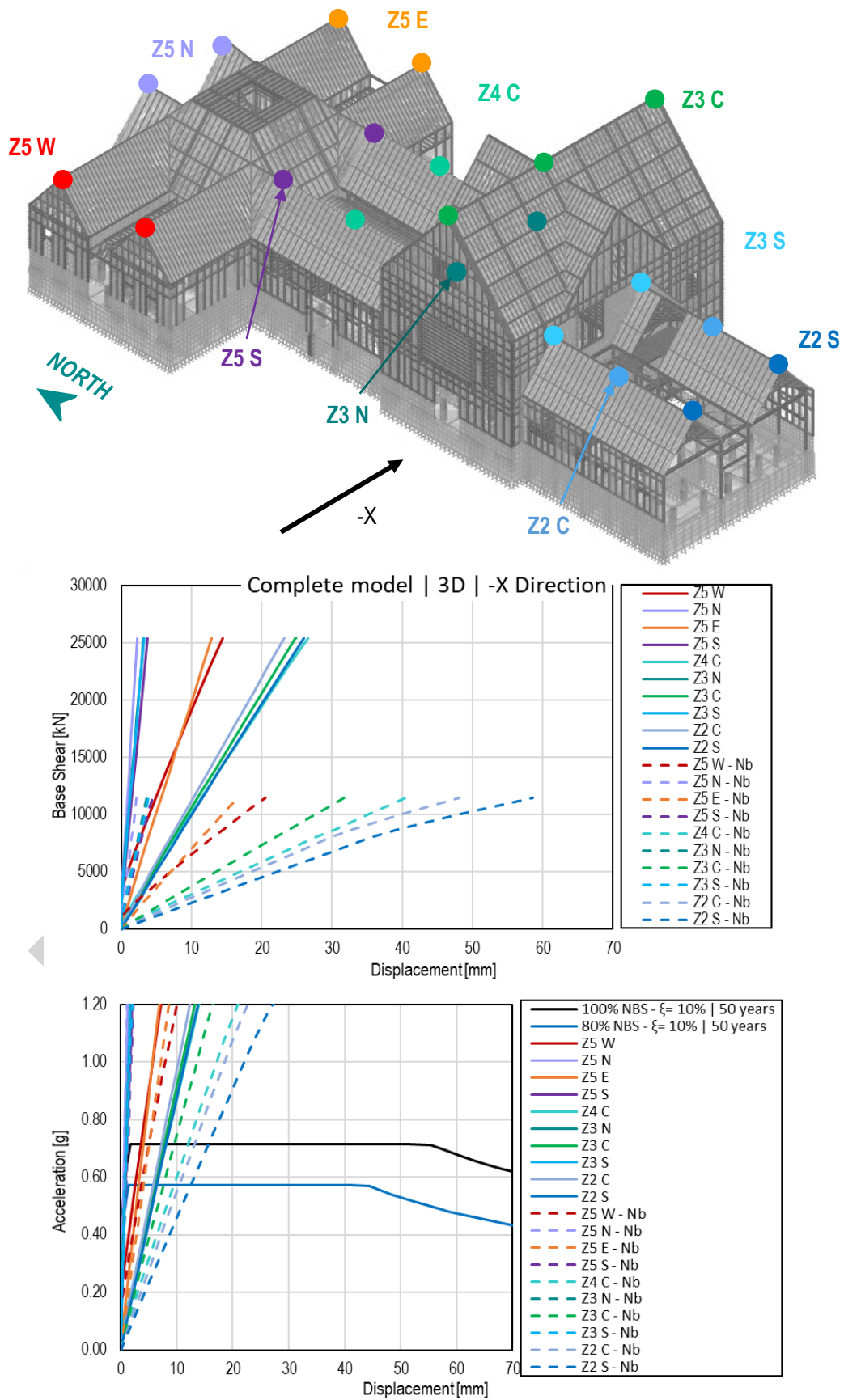


Figure 12-11: Zone 2-3-4-5 - Complete structure | Retrofit model pushover curves and ADRS plot for the considered control points | -X Loading direction | Load pattern: Uniform acceleration

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13 ZONE 2-3-4-5 | NONLINEAR DYNAMIC ANALYSES

This section reports the outcomes of the nonlinear time history analyses (NLTHA) performed on the original (retrofitted) structure of the Rotorua Museum building (Zone 2-3-4-5), considering that the two modern additions (Zone 1 built c. 2011, and Zone 6 built c. 2007) have been built adopting appropriate seismic gaps with the adjacent historic building and are treated as separated self-standing structures. The purpose of these analyses is to cross verify the results obtained via the nonlinear static pushover analyses (NLSPA), which was the main tool adopted for the design of the retrofit intervention, and to provide supplementary information on the (estimated) dynamic behaviour of the building subjected to seismic load.

13.1 Modal analysis

The modal analysis is carried out with a two-fold goal in mind. First, as a preliminary study of the structure's main dynamic properties. Secondly, the modal analysis serves as a basis for the definition of various parameters used in the NLTHA. The period of the first translational mode is used to calculate the period range of interest T_{range} necessary for scaling the ground motion records. It should be noted that in both the X and Y directions the largest translational mode period of the strengthened structure is smaller than 0.4 s, therefore the period range of interest is calculated adopting $T_1 = 0.4$ s as per NZS 1170.5 (2004). The outcomes of the modal analysis are used also to estimate the period range for the definition of the damping ratio according to the Rayleigh damping model.

Figure 13-1 to **Figure 13-5** display the deformed shapes of the building for the five main modes and provide the modal participating mass ratios (identified from the global model where the concrete basement structure is included). **Figure 13-1** also reports the modal participating mass ratio in the Y direction obtained from the “No-basement” model to highlight the large influence of the basement structure mass on the definition of the modal participating mass ratios of the global model.

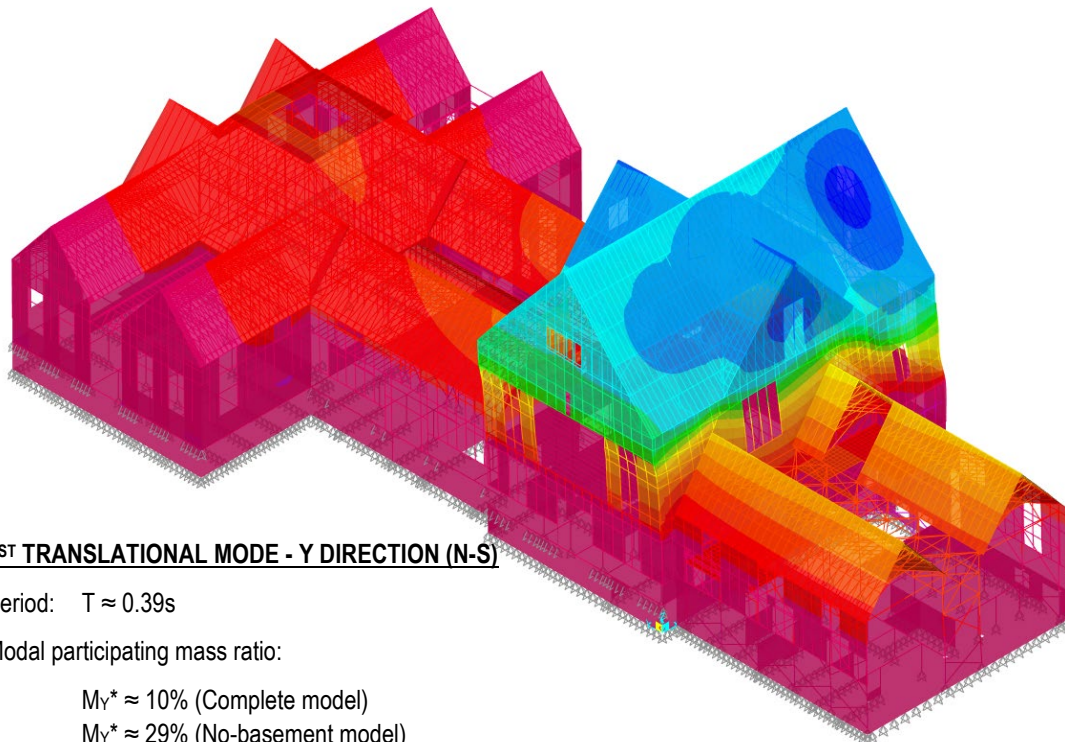


Figure 13-1: Modal analysis | Complete model deformed shape for the 1st translational mode in the Y direction (North-to-South) | Mode 1 of the analysis

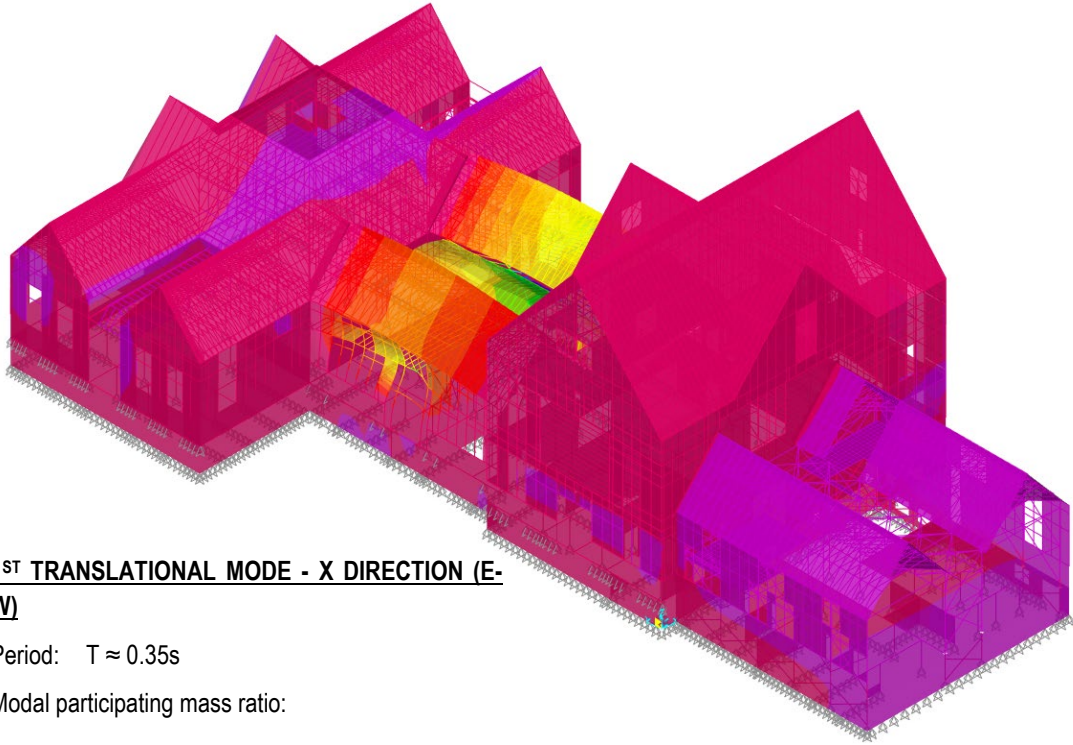


Figure 13-2: Modal analysis | Complete model deformed shape for the 1st translational mode in the X direction (West-to-East) | Mode 3 of the analysis

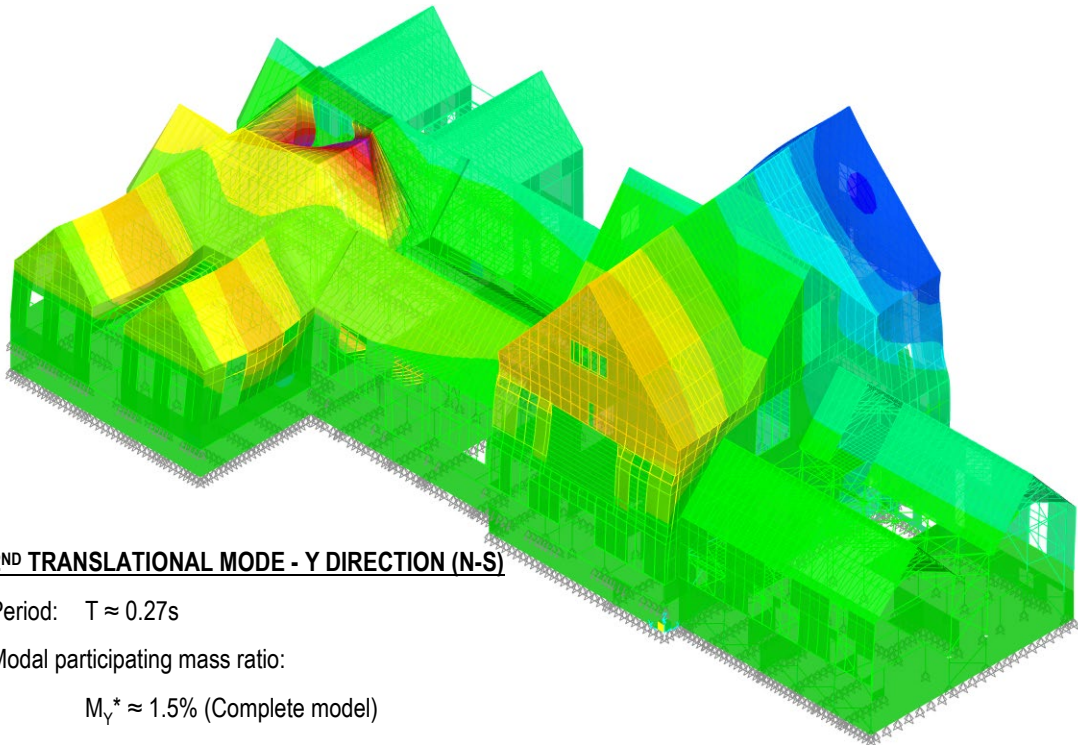


Figure 13-3: Modal analysis | Complete model deformed shape for the 2nd translational mode in the Y direction (North-to-South) | Mode 10 of the analysis

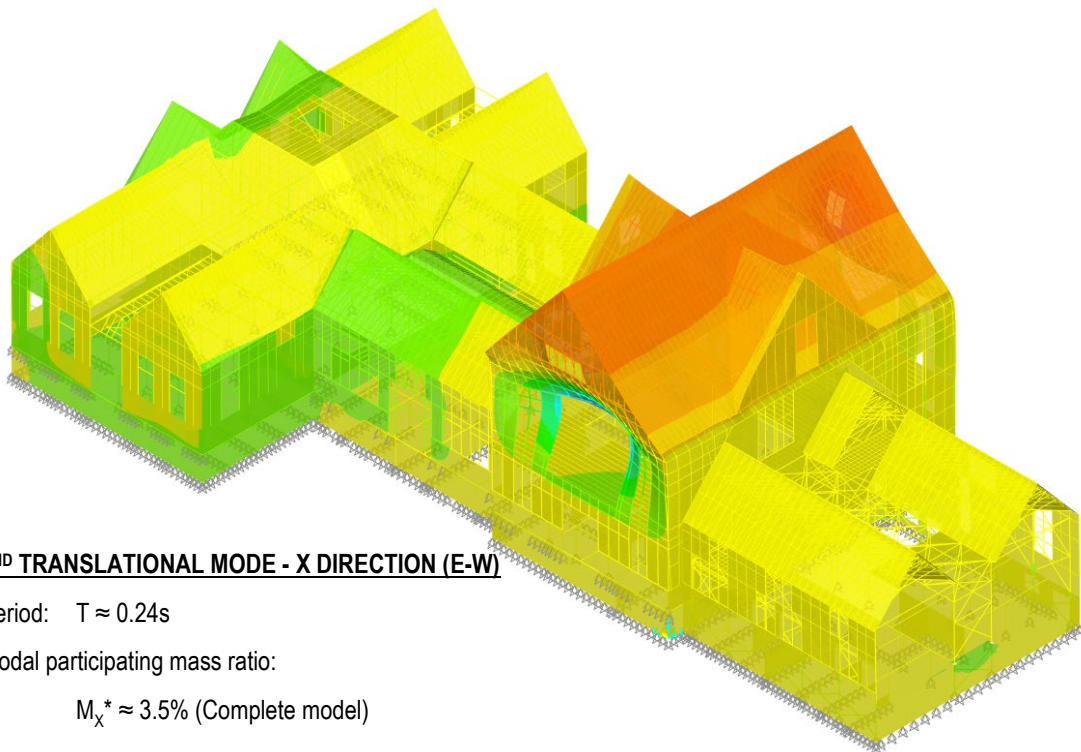


Figure 13-4: Modal analysis | Complete model deformed shape for the 2nd translational mode in the X direction (West-to-East) | Mode 17 of the analysis

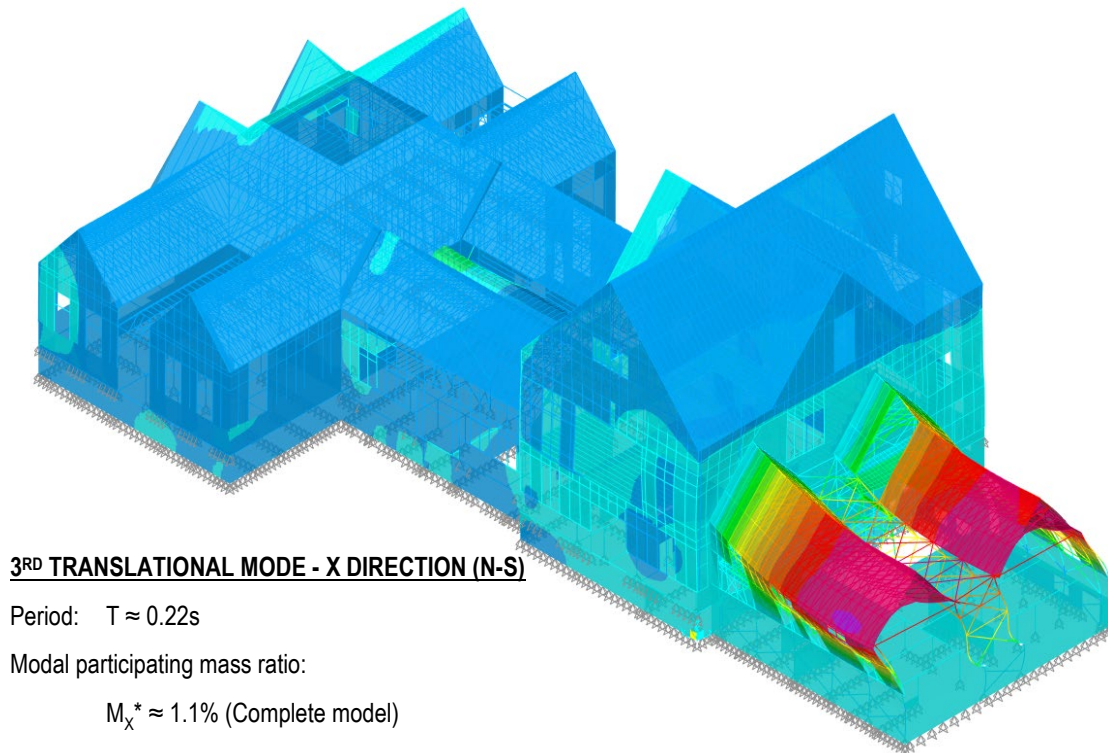


Figure 13-5: Modal analysis | Complete model deformed shape for the 3rd translational mode in the X direction (West-to-East) | Mode 21 of the analysis

13.2 Zones 2-3-4-5 | Entire structure NLTHA outcomes (Retrofit model)

Figure 13-6 shows a schematic of the settings adopted for the NLTHA of the original structure of the Rotorua Museum building (Zones 2-3-4-5). The principal and secondary components of the selected ground motion record are identified based on the scale factor k_1 derived in the ground motion record scaling procedure according to NZS 1170.5 (2004). The scale factor k_1 is determined minimizing in a least mean square sense the function $\log(k_1 SA_{\text{component}} / SA_{\text{target}})$ over the period range of interest (being $SA_{\text{component}}$ and SA_{target} the spectral pseudo-accelerations of the ground motion record component and the target spectrum). The principal component is applied to the direction of the first translational mode (Y direction, South-to-North) together with the secondary component applied to the orthogonal direction.

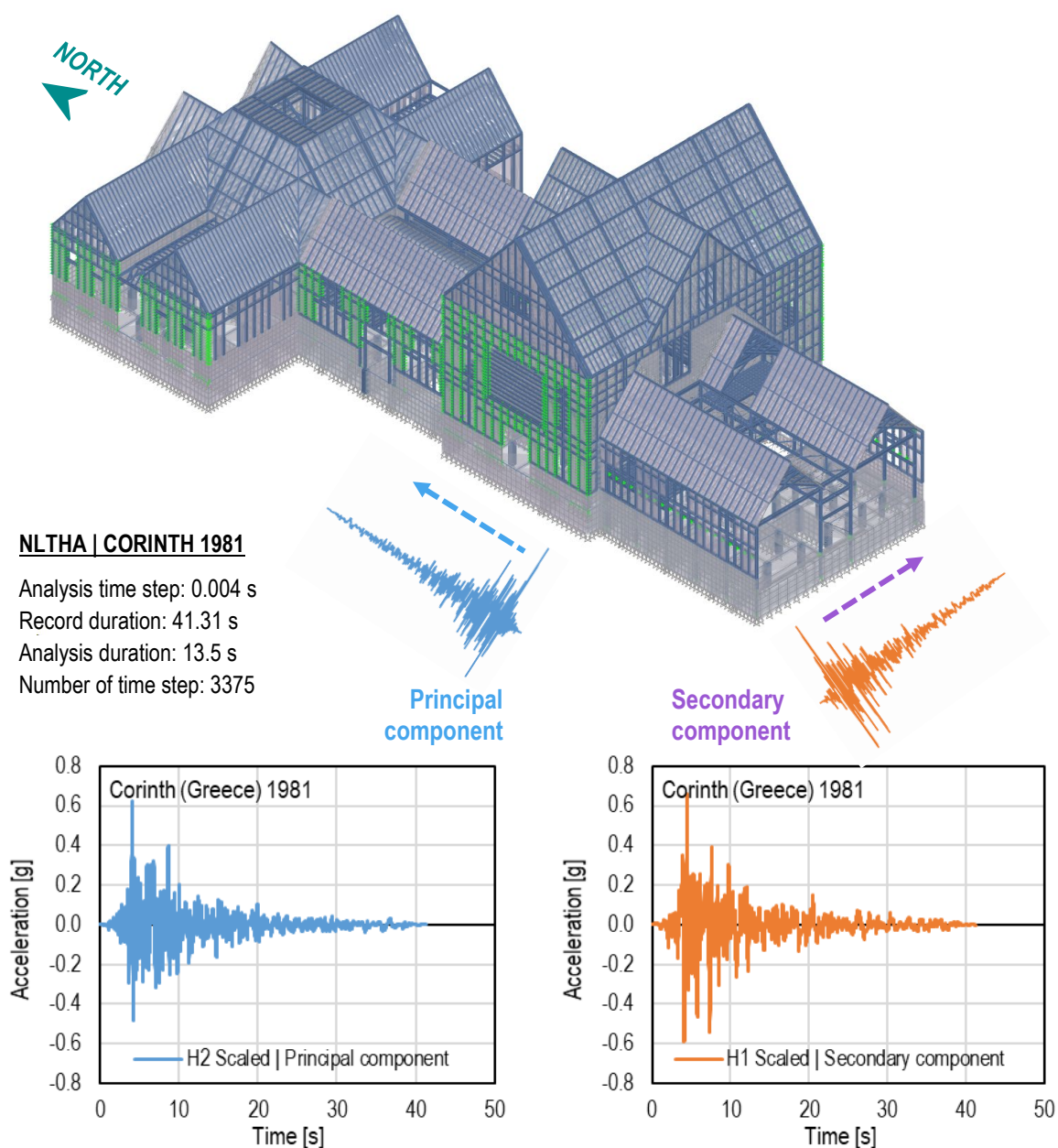


Figure 13-6: Non-linear time history analysis (NLTHA) setting: Corinth (1981) scaled ground motion record

The selection of the ground motion acceleration records used for the time-history analysis was performed according to NZS 1170.5 (2004). The earthquake events of Imperial Valley - USA 1940 (also known as El Centro; Magnitude $M = 6.95$, Arias intensity $AI = 1.6$ m/s), Corinth - Greece 1981 ($M = 6.60$, $AI = 0.9$ m/s), and Kalamata - Greece 1986 ($M = 6.20$, $AI = 0.7$ m/s) were identified as compatible with the seismic characteristics of the Taupo Volcanic Zone region (Zone North V, Oyarzo-Vera et al. 2012).

All the records were scaled by using the factors k_1 and k_2 (NZS 1170.5, 2004) in order to ensure compatibility with the target spectrum (when determining the spectrum, the Structural Performance Factor S_p was conservatively assumed to be equal to 1) and also to ensure that the energy content of at least one of the records exceeds that of the target spectrum over the period range of interest T_{range} (i.e. $T_{min} = 0.16$ s and $T_{max} = 0.52$, see Figure 13-7).

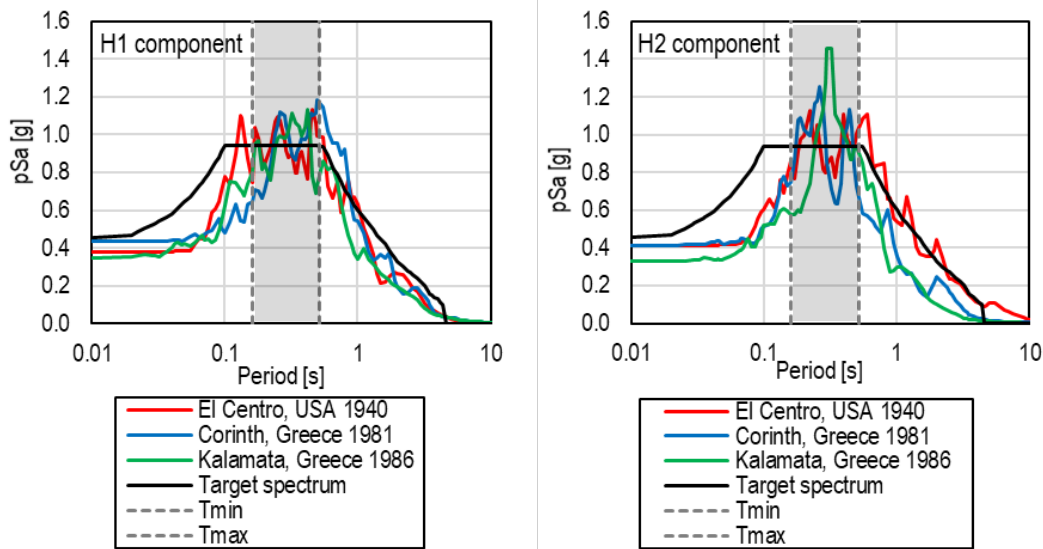


Figure 13-7: Target spectrum and scaled spectra according to (NZS 1170.5, 2004) for both horizontal components

The first aim for the nonlinear dynamic analysis was to carry out three different direct-integration NLTHA using all three ground motion records identified as compatible with the seismic characteristic of the Rotorua region. However, after various attempts varying the analysis parameters (e.g., analysis time step, tolerance, and integration method) it was decided to use exclusively the “Corinth” ground motion record. Moreover, to maintain the NLTHA computation time and hard drive space requirement within feasible limits, the analysis was carried out using the first 13.5 seconds of registered signal. It has been demonstrated by Reyes et al 2017 that trimming the signal of the weak trailing part has a negligible impact on the NLTHA outcomes. Nonetheless, the NLTHA of the complete model took 171 hours (≈ 7 days) to run and required 867 GB of hard-drive space using a powerful PC.

Viscous damping was simulated referring to the Rayleigh damping model, which considers damping proportional to a linear combination of mass and stiffness. The two Rayleigh damping coefficients $\alpha = 0.8976$ 1/s (mass proportional coefficient) and $\beta = 0.00136$ s (stiffness proportional coefficient) were selected in order to have a damping ratio of approximately 5% in the period range 0.1 s and 0.6 s. **Figure 13-8** shows the damping ratio graph as a function of both period and frequency to provide a clearer representation of both the mass and stiffness proportional coefficients.

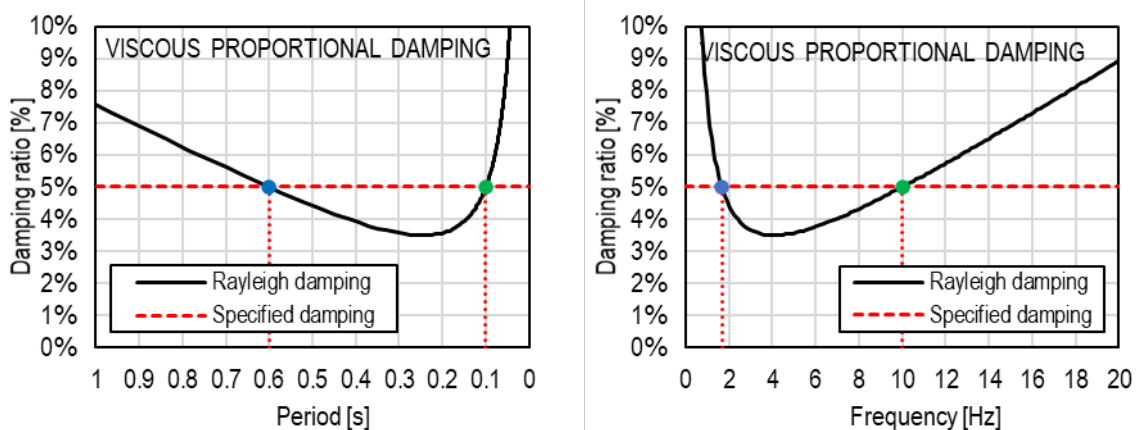


Figure 13-8: Rayleigh damping function (in terms of period and frequency) and adopted 5% damping ratio period interval (0.6 s – 0.1 s)

The 5% damping ratio period interval ($T = 0.6$ s to 0.1 s) was selected to include the principal modes of the building. The first interval value (0.6 s) which was selected refers to the first translational mode of the structure in the Y direction, North-to-South ($T = 0.39$ s, see **Figure 13-1**). The second value (0.1 s) was identified so that the relevant high frequency

modes of the structure are included in the selected period range. From the outcomes of the modal analysis the number of modes comprised in the selected period range (0.6 s to 0.1 s) is 106 for the Complete model and 115 for the “No-basement” model. The modal participating mass ratio in the two directions is equal to 11% in the X direction (West-to-East) and 22% in the Y direction (North to South) for the Complete model; these values increase up to 33.3% and 62.9% for the “No-basement” model.

For further validation of either the ground motion records scaling procedure and the choice of the Rayleigh damping period interval, **Figure 13-9** reports the pseudo acceleration spectra comparison for the two horizontal components of the Corinth (1981) ground motion record. The blue curves represent the ground motion spectra fitted to the target spectrum in the period range of interest (as already reported in **Figure 13-7**). The red curves on the other hand are obtained by the direct integration of the equation of motion using the finite difference method and applying the scaled ground motion record as imposed acceleration time history.

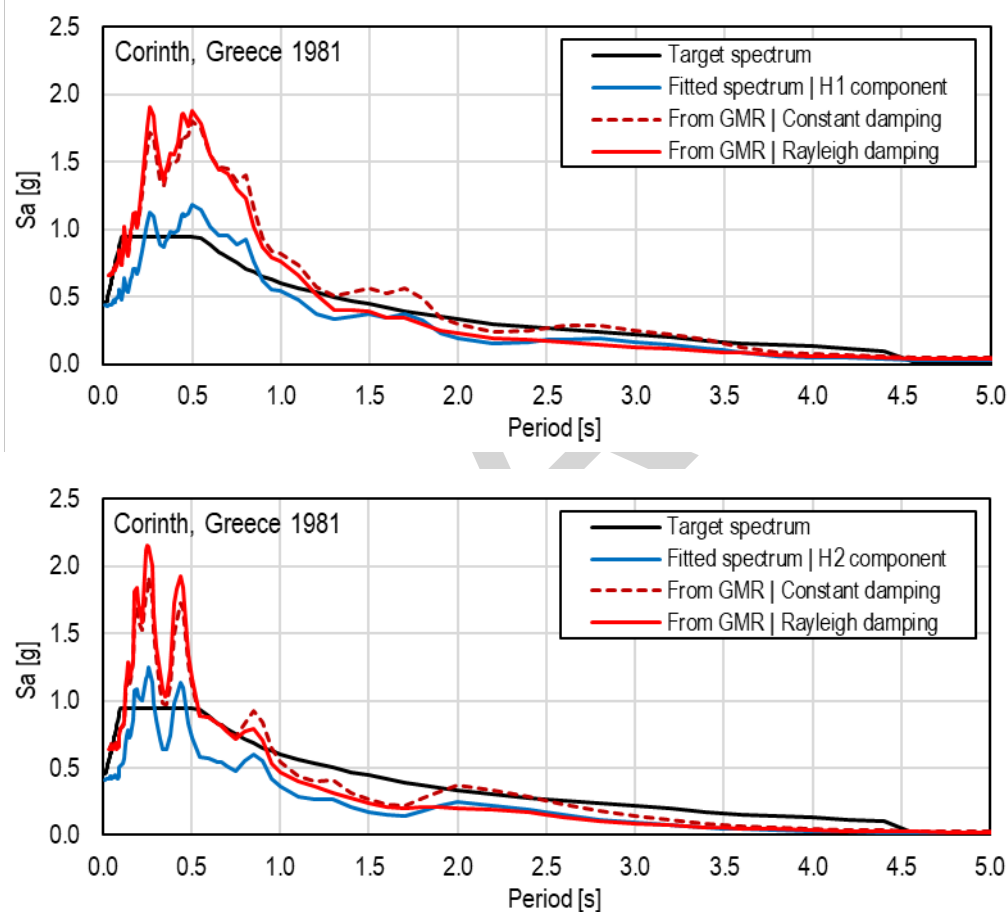


Figure 13-9: Spectra comparison for the two horizontal components of the Corinth 1981 Ground Motion Record (GMR): the two red curves are obtained from the scaled GMR

The hysteretic response of the unreinforced pumice concrete structural elements was reproduced by exploiting the cyclic behaviour of the “Darwin-Pecknold” constitutive behaviour and adopting the Takeda hysteresis model for the additional “tensile strength” layer and for the FRCM equivalent material layer, as reported in **Figure 13-10**. The dissipative properties of the timber-framed structure were included in the numerical model exclusively based on the elastic damping. The reason for such a conservative assumption is due to an absence of reliable information on the hysteretic behaviour of this construction system. Also, the complexity of the interaction between the various components of the timber frame structure (e.g., concrete infills and timber elements) together with the uncertainty regarding the connections make any attempt of simulating the hysteretic behaviour highly challenging and most likely fruitless.

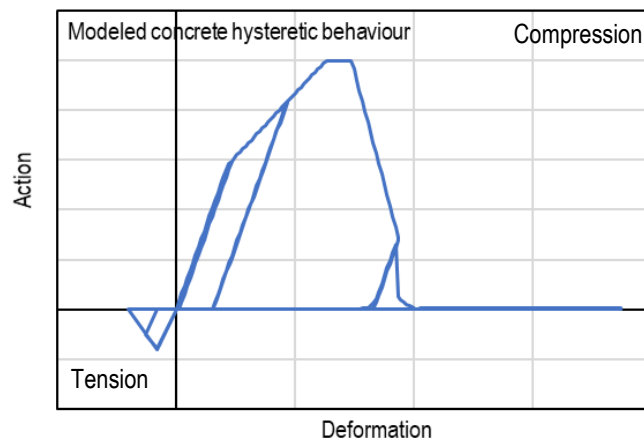


Figure 13-10: Concrete hysteretic behaviour: Darwin-Pecknold material cyclic behaviour and Takeda model for the “tensile strength” and FRCM additional layers

Figure 13-11 shows the Control points adopted to examine the outcomes of the time history analyses. Displacements, velocities and accelerations are measured at the highest portion of the building and spread across the structure’s footprint to account for any possible irregular behaviour of the structure in the interpretation of the results.

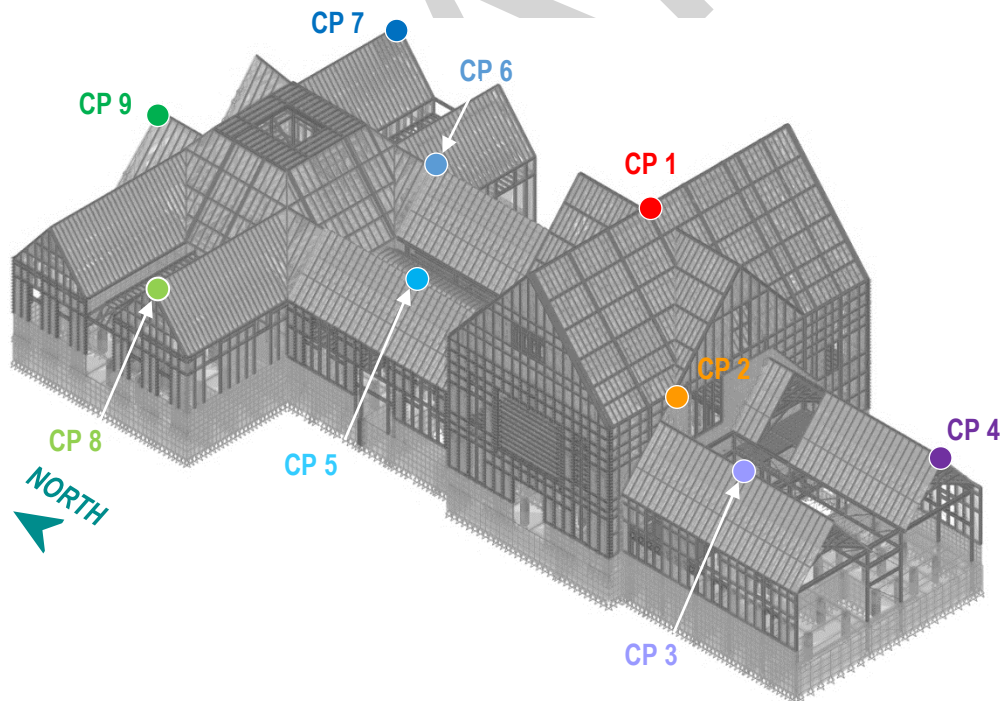


Figure 13-11: Zone 2-3-4-5 - Complete structure | Control points adopted for the nonlinear time history analyses (NLTHA)

For a visual assessment of the building behaviour under lateral loads, **Figure 13-12** to **Figure 13-15** report the deformed shapes of the FE model subjected to the Corinth (1981) ground motion record. The pictures are taken at the time steps of the analysis equivalent to the maximum positive and negative displacements of Control Point 1 in either the Y direction (North-to-South) and the X direction (East-to-West).

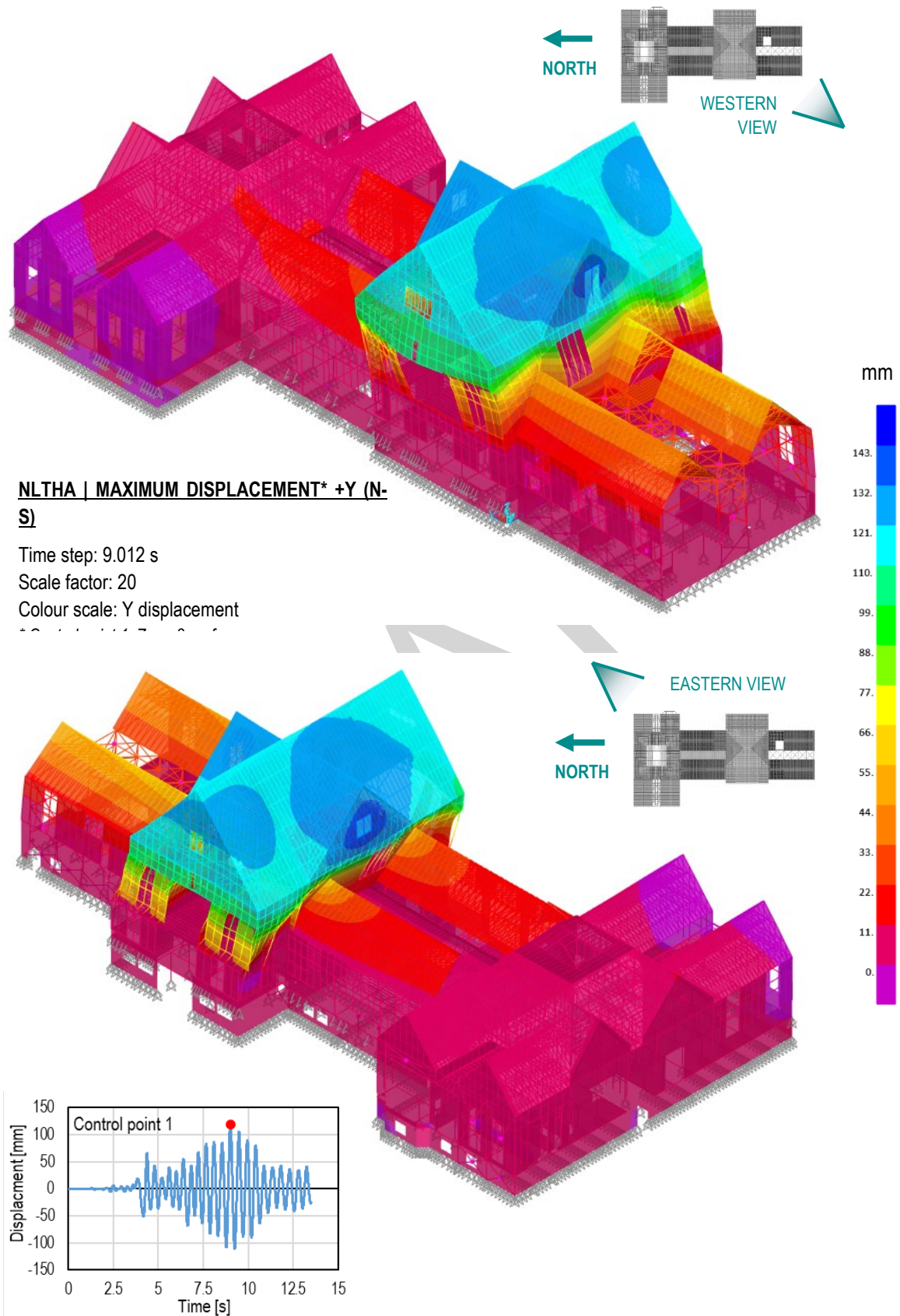


Figure 13-12: NLTHA | Corinth 1981 | Deformed shape at the maximum +Y displacement of CP1

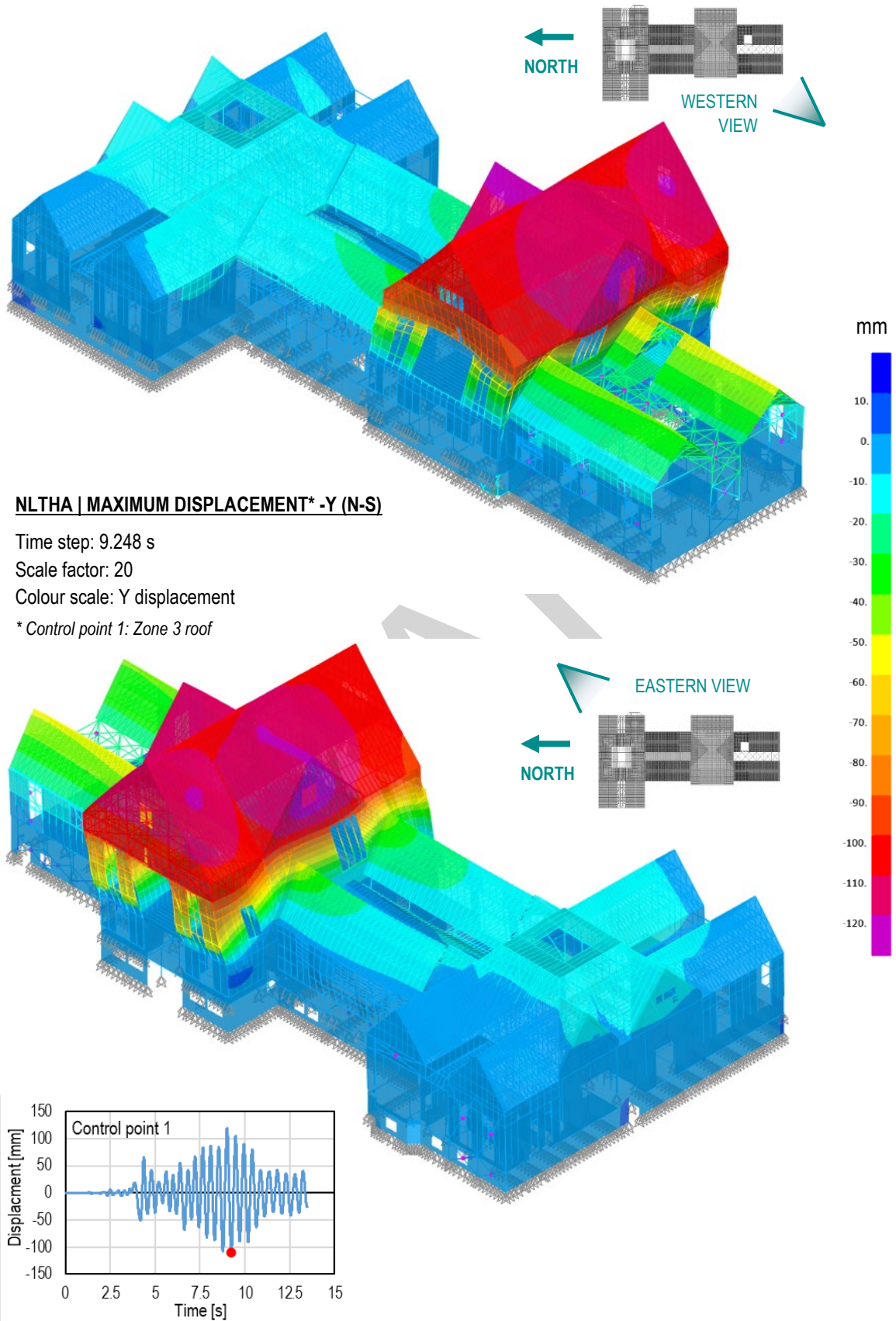


Figure 13-13: NLTHA | Corinth 1981 | Deformed shape at the maximum -Y displacement of CP1

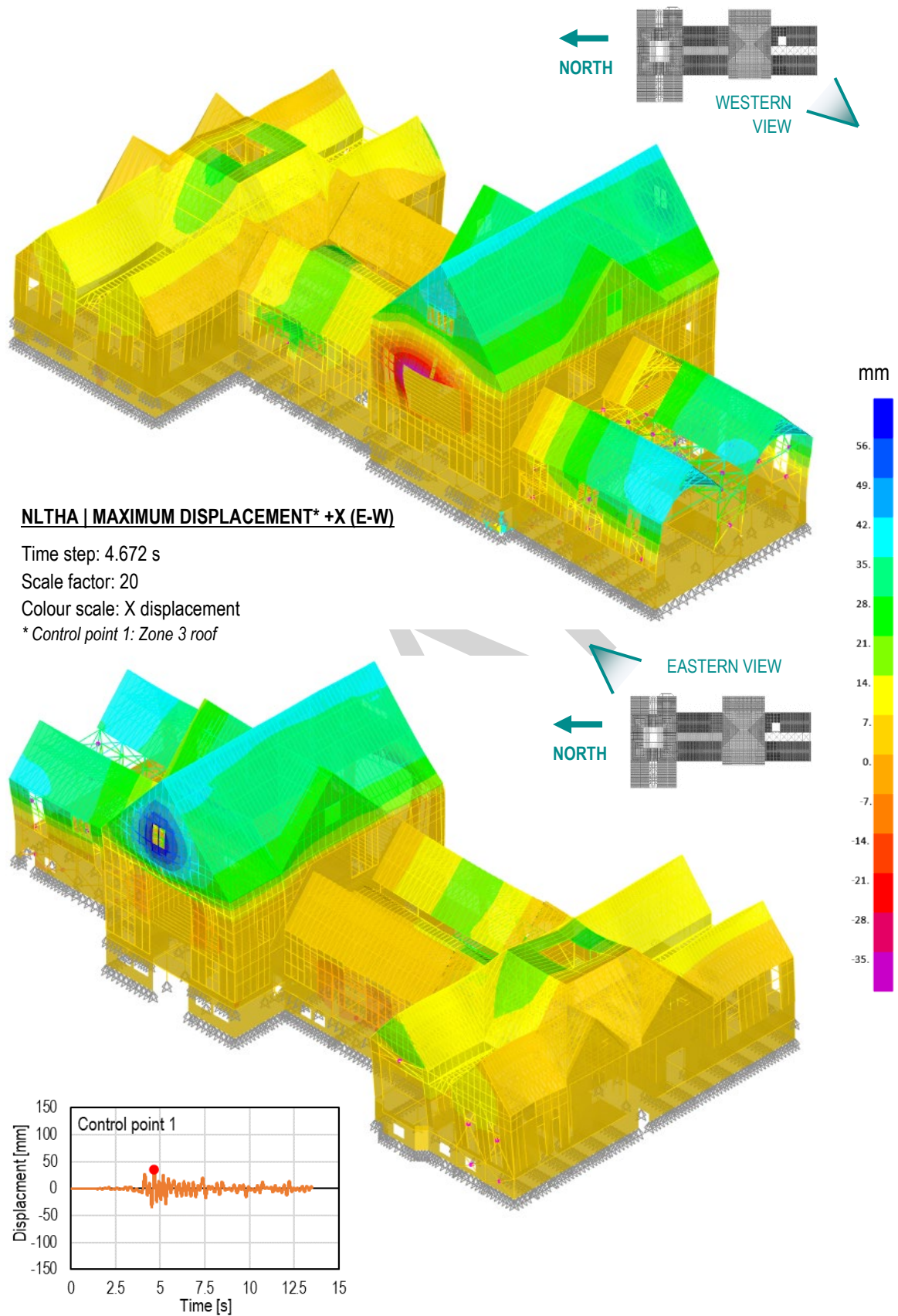


Figure 13-14: NLTHA | Corinth 1981 | Deformed shape at the maximum +X displacement of CP1

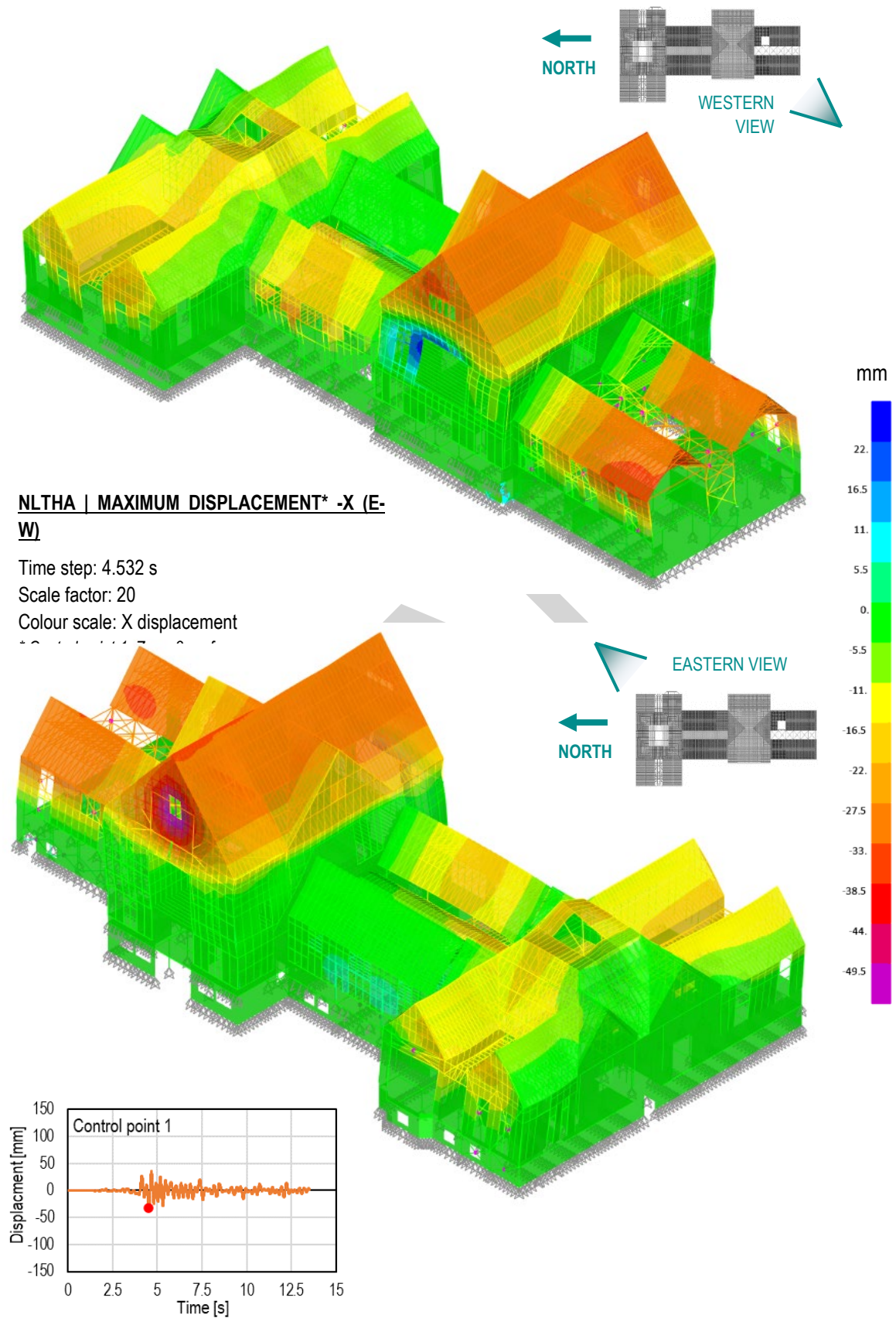


Figure 13-15: NLTHA | Corinth 1981 | Deformed shape at the maximum -X displacement of CP1

Table 13-1 shows the comparison between the Nonlinear Static Pushover Analyses (NLSPA) outcomes (for both the complete model and the no-basement model) and the Nonlinear Time History Analysis results in terms of total base shear and maximum horizontal displacement for the Control Point CP 1, 3, 5, and 9 (taken as representatives of Zone 3, 2, 4 and 5, respectively). It can be observed that the main differences between the NLSPA and the NLTHA results are in terms of displacement, particularly for CP1 in the Y direction (N-S) which matches the direction of the first translational mode of the building (**Figure 13-16**). These differences could be explained considering that:

- The NLSPA were performed using the uniform acceleration load pattern, which applies a lateral load to the structure with a vertical pattern proportional to the building mass distribution. It was not possible to adopt the modal load pattern (neither the fundamental mode pattern nor the one determined by a combination of modal responses) to carry out the NLSPA. The complexity of the structure and the large difference in mass distribution between the concrete and timber portion of the building didn't allow to capture, within a reasonable number of modes, the percentage of modal participating mass ratio required by the Standard (i.e., 75% for the fundamental mode load pattern and 90% for the modal combination load pattern). It is worth noting that the uniform acceleration load pattern is associated with larger forces at the base of the building, usually the most critical part for seismic design and in this specific case also the most vulnerable portion of the structure due to the presence of the unreinforced pumice concrete walls.
- Nonetheless, it was decided to carry out the NLTHA to provide additional insight into the dynamic behaviour of the structure in absence of the usual modal pattern NLSPA. The displacement demand of the NLTHA was found to be larger than that obtained from the NLSPA, especially for CP1 (**Figure 13-16**).
- The assessment of the results from the NLTHA should be undertaken with respect to the following remarks:
 - The scaling procedure for the ground motion records used in the NLTHA entails a quite strong seismic demand with respect to the target spectrum. As can be seen in **Figure 13-9**, the NLTHA appears extremely conservative, having (in the period range of interest) acceleration values two times larger than the target spectrum (as a result of the k_2 factor).
 - The choice of the period range for the definition of the damping according to the Rayleigh model results in a slight underdamping of the first translational mode of the structure with respect to the selected customary damping ratio value equal to 5% (period $T = 0.39$ s, Rayleigh modal damping ratio $\xi \approx 4\%$ as from **Figure 13-8**). This small change can greatly influence the results of the NLTHA considering that the first mode of the building (i.e., translation of the Zone 3 roof structure in the Y direction, North-to-South, as from **Figure 13-1**) exhibits a resonant response. **Figure 13-12** and **Figure 13-13** shows that the maximum displacement U_2 (i.e., displacement in the Y, north-south direction) of CP1 is reached several seconds after the peak of maximum base shear in the same direction (which matches the peak in acceleration of the ground motion records). The maximum displacement is observed at the second peak of the base shear (smaller than the first one), due to a displacement built up related to a resonant response of the structure.
 - To better examine the resonance phenomenon of the roof structure of Zone 3, observed during the NLTHA, it was decided to study the variation over time of the frequency content of the seismic input signal using the Continuous Wavelet Transform (CWT). **Figure 13-18** and **Figure 13-19** report the outcomes of this study. It can be observed that the signal magnitude in the time-frequency domain exhibits two main peaks: the first peak at approximately 4 seconds after the start, matching the arrival time of the acceleration peak in the ground motion record. The bulk of the energy of this first peak is centred between the frequency 3.5 Hz and 4.9 Hz, exciting a large number of high frequency modes of the structure ($T \approx 0.2\div0.3$ s) and resulting in the peak of base shear registered in the Y direction (north-south). On the other hand, the second peak is reached at the time interval between approximately 5 to 10 seconds when the observed resonance phenomenon for CP 1 is more evident. Indeed, the energy of this second peak is concentrated at the 2.0 Hz centre value matching almost exactly the frequency value of the fundamental mode of the structure (period $T = 0.39$ s and frequency $f = 2.56$ Hz).
 - In the absence of reliable information on the hysteretic properties of the timber framed structure it was decided not to include them into the analysis. This is another conservative assumption, because

it is unlikely that a complex system such as the timber framed walls of the building will respond to dynamic excitation without showing any energy dissipation (additional to the 5% elastic damping).

- The direct comparison between the NLTHA and the pushover curves at the level of displacement corresponding to the maximum demand (observed for CP1) is not possible. Given the loading pattern of the NLSPA, pushing the top part of the structure up to a displacement equal to the maximum of the NLTHA would result in a base shear unrealistically larger than the base shear demand from the NLTHA.
- The maximum displacement demand from the NLTHA is reached without any evident decrease in the capacity of the structure, denoting that the ultimate building conditions have not been reached. **Figure 13-20** and **Figure 13-21** display the principal tensile strain on the concrete structure of the Rotorua Museum building respectively at the time step of maximum base shear in the Y (north-south) direction and at the time step of maximum CP 1 displacement in the Y (north-south) direction. In the first case (**Figure 13-20**), it can be observed that the tensile strain on the four main concrete walls (related to their out-of-plane OOP response) is below the conventional strain limit of the FRCM reinforcement applied to the concrete wall, indicating no visible damage. In the second case (**Figure 13-21**), the larger displacements experienced by the Zone 3 roof structure cause an increase in tensile strain demand for the FRCM reinforcement, which exceeds the strain limit and causes the formation of horizontal cracks at a level corresponding to the roof top height of adjacent Zone 2 and Zone 4. However, the 70 mm displacement demands for the wall top portions, isolated by the aforementioned cracks, appears well within the overturning limit associated with their rocking response. **Figure 13-22** shows the deformed shape of the North wall of the Zone 3 at the time step of maximum CP 1 displacement in the Y (north-south) direction.

Table 13-5: NLTHA | Maximum reaction force values (F) – Maximum displacement values (d) [CP 1 | Zone 3 Roof, CP 3 | Zone 2, CP 5 | Zone 4, CP 9 | Zone 5]. For the NLSPA forces and displacement are related to the Performance Point of the analysis

	NLTHA Corinth - 1981		NLSPA Complete model Performance Point		NLSPA No-basement model Performance Point	
Global	Fx	Fy	Fx	Fy	Fx	Fy
Max [kN]	13167	13922	10400	9723	4685	4204
Min [kN]	-12615	-9287	-9465	-9943	-4858	-4269
CP 1	dx	dy	dx	dy	dx	dy
Max [mm]	35.71	119.65	10.24	27.77	12.86	35.35
Min [mm]	-32.94	-110.28	-8.91	-27.62	-13.25	-35.14
CP 3	dx	dy	dx	dy	dx	dy
Max [mm]	31.96	45.76	10.37	15.38	20.77	19.17
Min [mm]	-27.53	-45.58	-9.52	-15.99	-21.47	-19.80
CP 5	dx	dy	dx	dy	dx	dy
Max [mm]	24.31	22.90	10.83	11.20	14.93	14.80
Min [mm]	-17.95	-25.50	-8.40	-11.82	-14.54	-15.16
CP 9	dx	dy	dx	dy	dx	dy
Max [mm]	1.39	35.65	0.56	11.90	0.65	16.34
Min [mm]	-1.64	-29.61	-0.69	-13.74	-0.88	-17.76

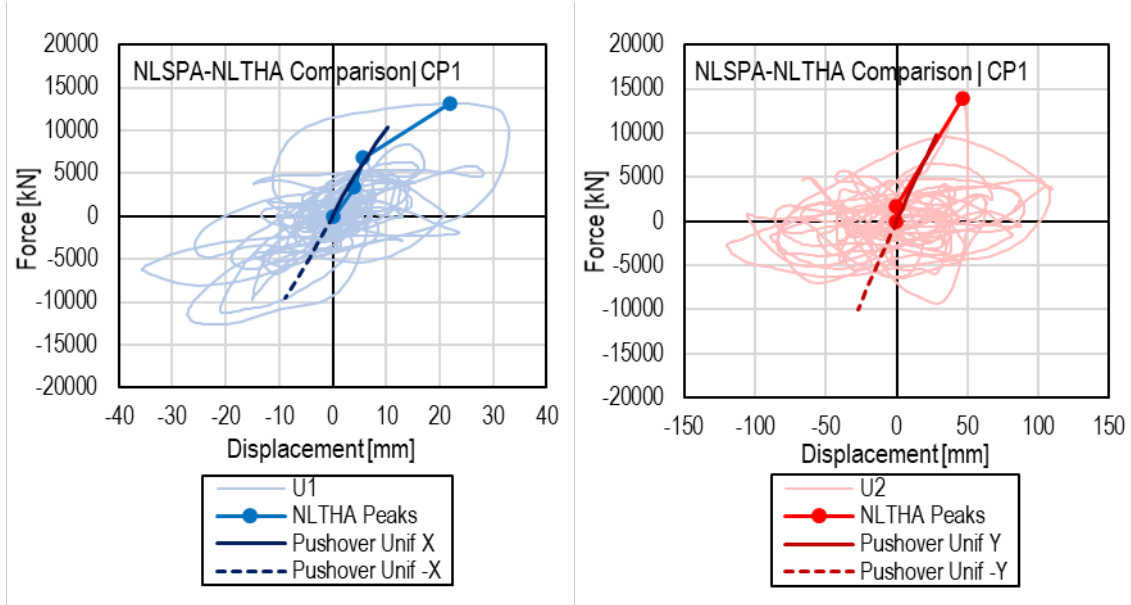


Figure 13-16: Comparison of NLSPA and NLTHA outcomes: NLTHA base shear-displacement cycles, NLTHA peaks selection and NLSPA pushover curves (stopped at the Performance Point of analysis) | CP 1 – Zone 3 Roof

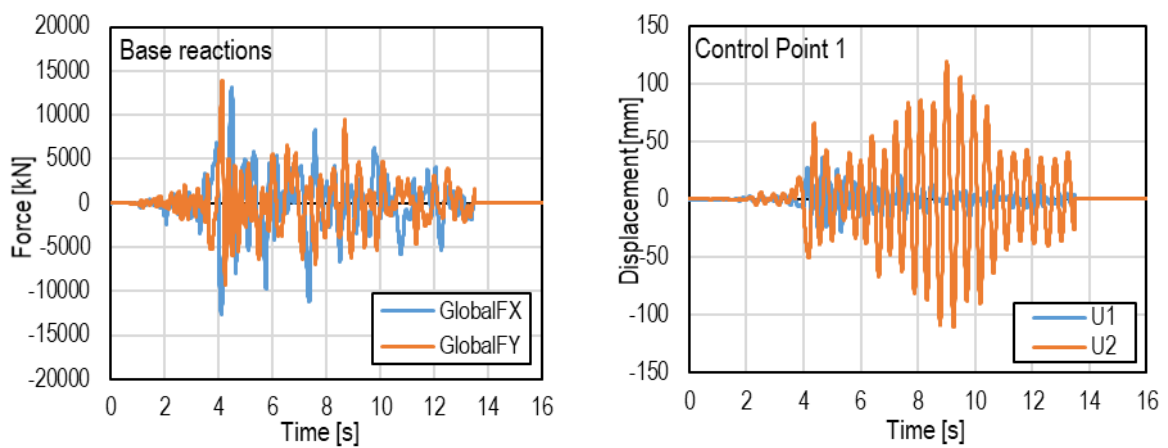
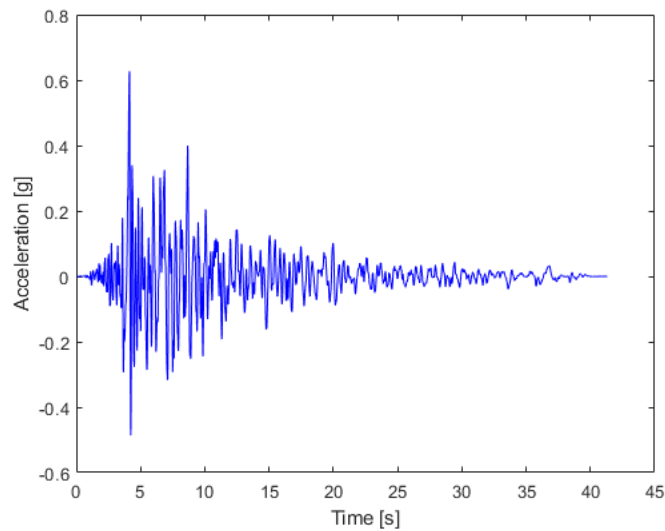


Figure 13-17: NLTHA | Corinth 1981 | Global base reactions and CP1 displacement time histories

CORINTH | PRINCIPAL COMPONENT – Y (NORTH-SOUTH)

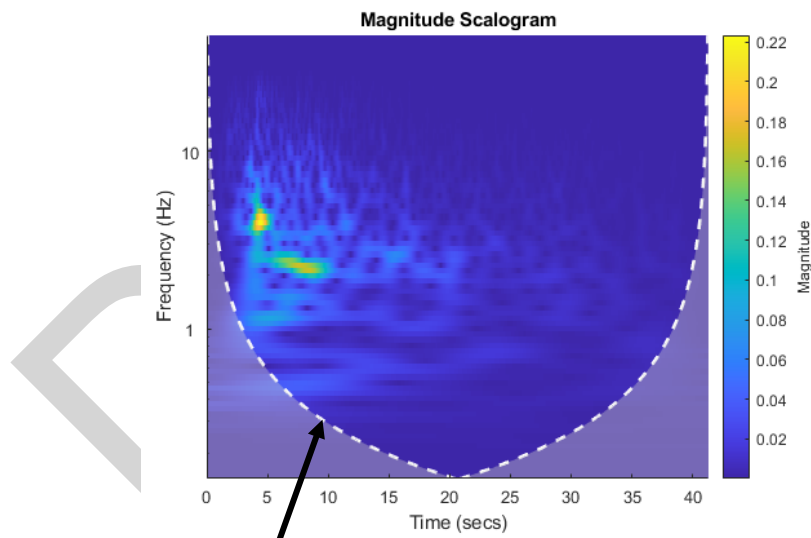


CONTINUOUS WAVELET TRANSFORM (CWT)

Morse wavelet

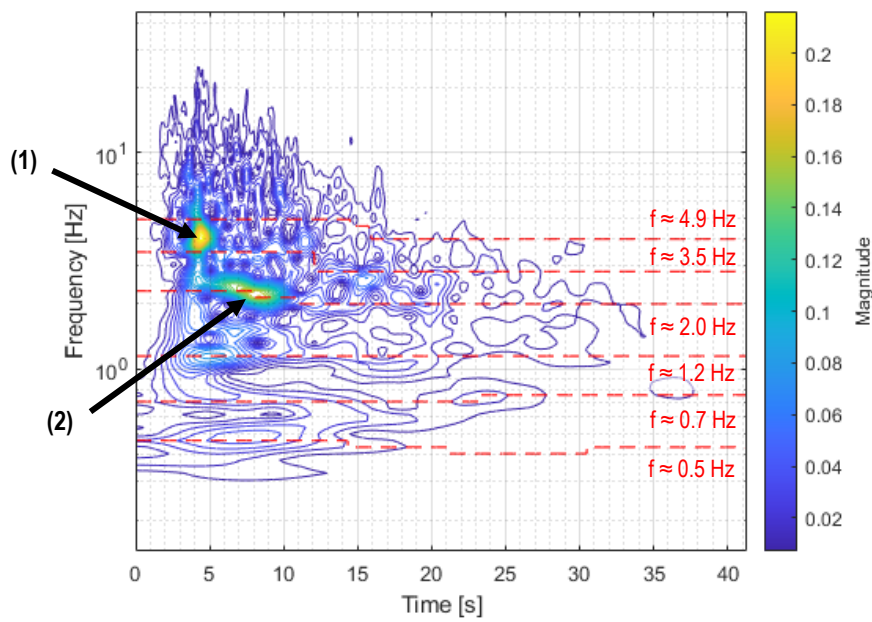
Symmetry parameter: $\gamma = 3$

Time-bandwidth parameter: $P^2 = 120$



Dashed white line = cone of influence: area potentially affected by edge-effect artifacts

Figure 13-18: Scalogram evaluation via Wavelet analysis: study of the frequency and energy content variation over time of the Corinth ground motion record principal component (Y, north-south direction)



Scalogram time-frequency magnitude peaks:

(1) Period: $T \approx 0.2 \div 0.3$ s Duration: $4 \text{ s} < t < 5 \text{ s}$

Base shear peak: shorter periods excite the high frequency modes of the building generating larger reaction forces

(2) Period: $T \approx 0.4 \div 0.5$ s Duration: $6 \text{ s} < t < 10 \text{ s}$

Resonance: frequency content of the input signal matches the frequency of the fundamental mode of the building

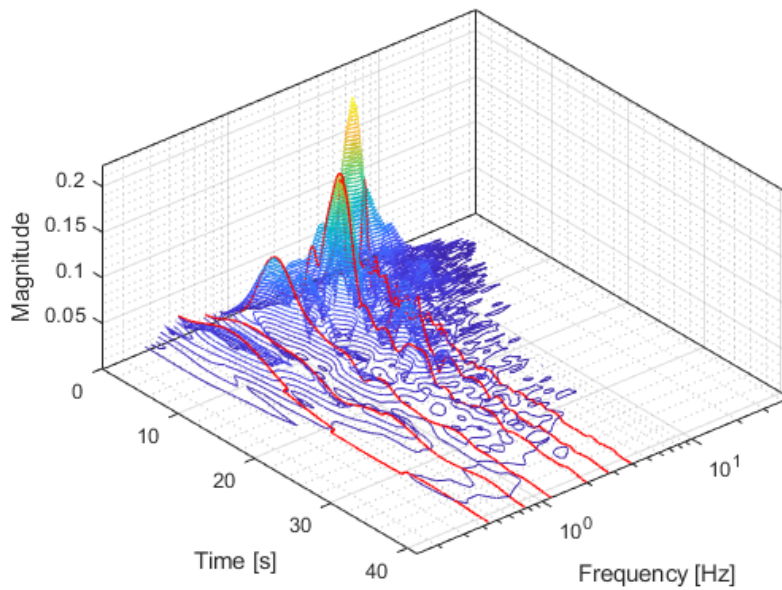
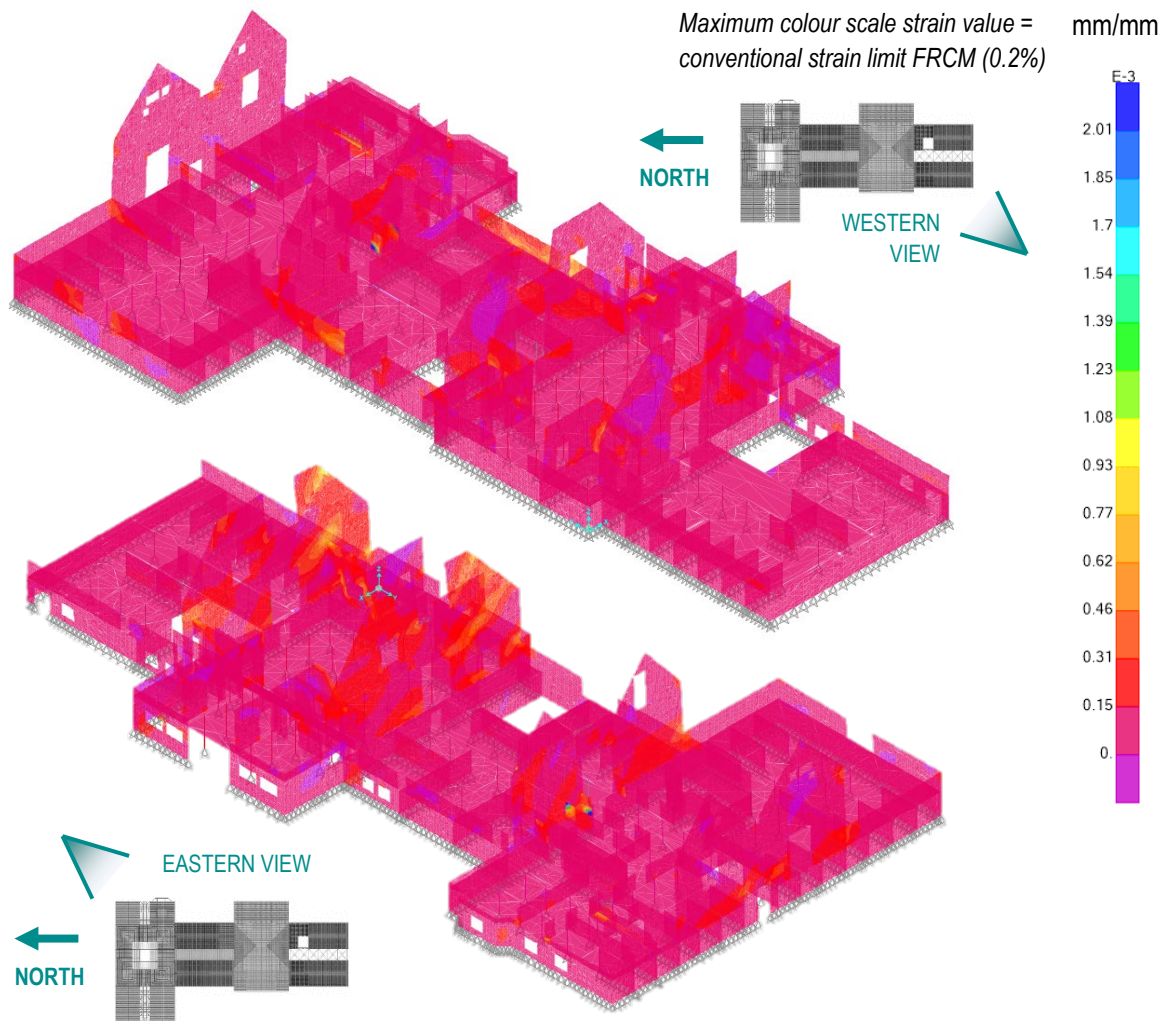


Figure 13-19: Scalogram representation of the Corinth principal component using a 2D contour plot and 3D contour plot (a.k.a. Waterfall plot). The dashed red lines and the continuous red lines represent 6 maximum-energy time-frequency ridges (reported numerical values are the mode values of the ridge frequencies)



NLTHA | CONCRETE STRUCTURE PRINCIPAL TENSILE STRAIN

Time step: 4.14 s (Maximum base shear north-south: FY = 13922 kN)

Deformed shape scale factor: 20

Colour scale: Principal tensile strain

Signal energy peak = high frequency content modes

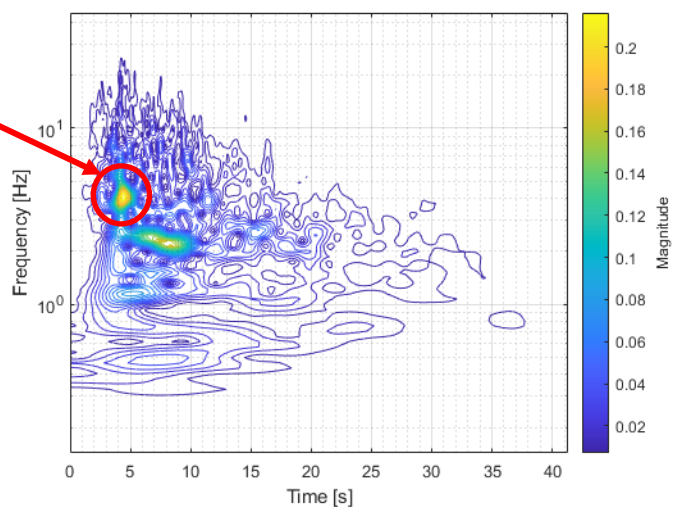
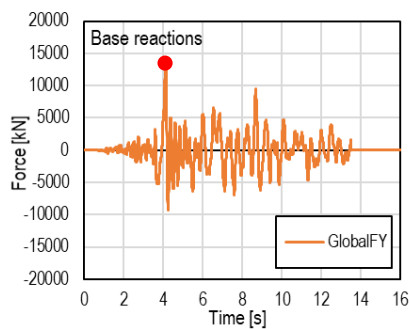
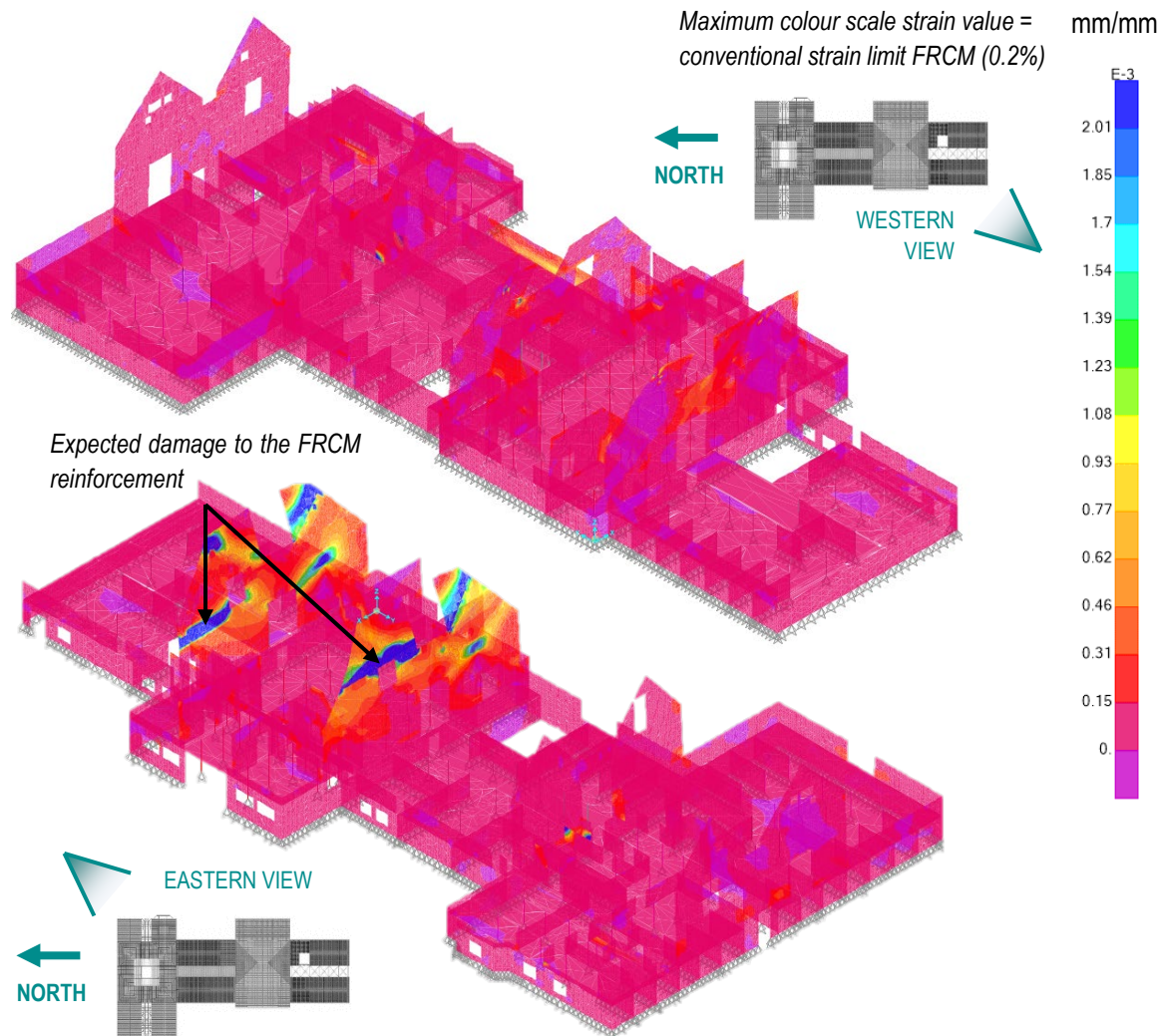


Figure 13-20: Principal tensile strains on the concrete structure of the Rotorua Museum building from the NLTHA at the time step of registered maximum base shear in the Y direction (north-south)



NLTHA | CONCRETE STRUCTURE PRINCIPAL TENSILE STRAIN

Time step: 9.248 s (Maximum CP-1 displacement north-south: U2 = 110.3 mm)

Deformed shape scale factor: 20

Colour scale: Principal tensile strain

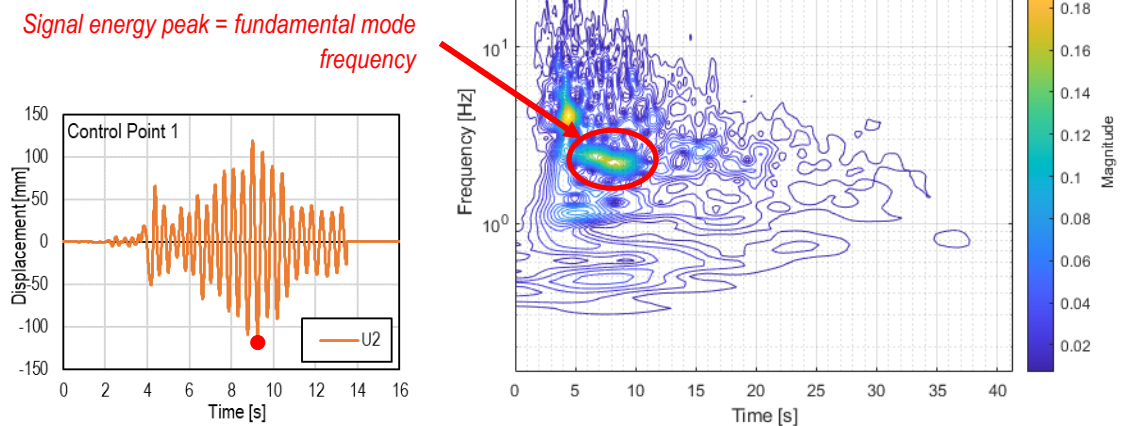
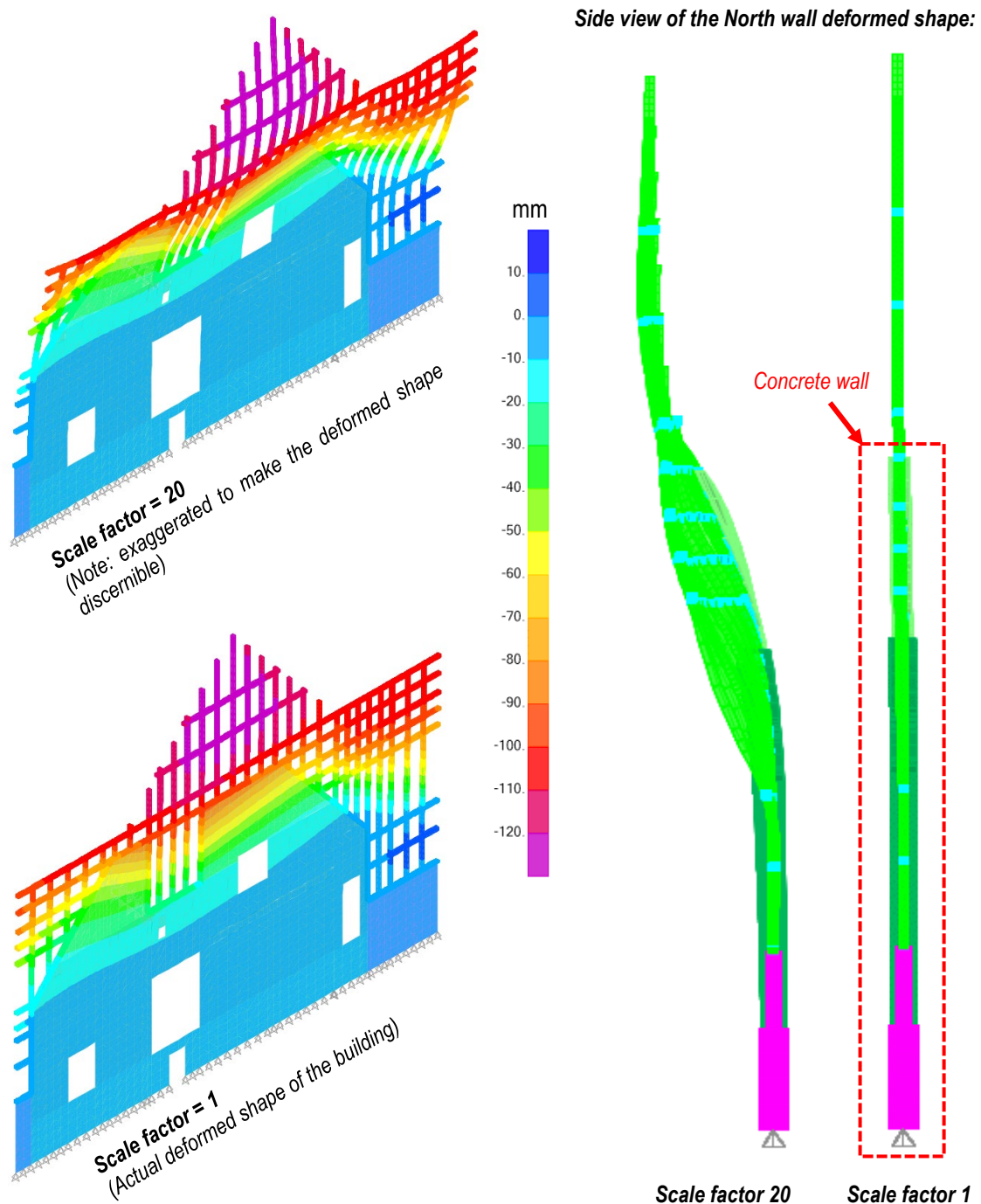


Figure 13-21: Principal tensile strains on the concrete structure of the Rotorua Museum building from the NLTHA at the time step of registered maximum CP 1 displacement in the Y direction (north-south)



NLTHA | ZONE 3 NORTH WALL DEFORMED SHAPE

Time step: 9.248 s (Maximum CP-1 displacement north-south: U2 = 110.3 mm)

Colour scale: U2 displacement (Y direction) for the isometric view of the wall

Figure 13-22: Deformed shapes of the Zone 3 North wall from the NLTHA at the time step of registered maximum CP 1 displacement in the Y direction (north-south).

As an additional evaluation of the NLTHA results, [Figure 13-23](#) to [Figure 13-25](#) report the various steps of the analysis process required for the estimation of the Frequency Response Function (FRF) related to CP1. The purpose of this study

was to confirm that the modal response of the structure (already estimated via linear dynamic analysis) is actually captured during the execution of the NLTHA. The steps for the calculation of the FRF are the following:

- The considered Outputs are the two horizontal displacements (i.e., the displacements in the X, east-west, and Y direction, north-south, here defined as U1 and U2 respectively) of the CP1 (Compliance FRF). As Inputs are adopted the total base shear actions (in the X, east-west, and Y, north-south, direction), each of one related to the respective horizontal displacement.
- The recorded signals (3376 samples) are extended to the closest power of 2 ($2^{12} = 4096$ samples) using the zero-padding technique to make possible the use of the Fast Fourier Transform (FFT) algorithm. The obtained Input and Output spectra are shown in **Figure 13-23**. Note that for a clearer representation of the outcomes, both the abscissa and ordinate axis are represented using a logarithmic scale.
- **Figure 13-24** reports the Bode plots of the Output/Input Compliance FRF. Despite the presence of some noise in the results toward the high frequency zone, it is possible to identify one resonance peak in the U1 FRF (East-West direction) and two resonance peaks and one anti-resonance valley in the U2 FRF (North-South direction).
- **Figure 13-25** shows the deformed shapes of the modes, extracted from the modal analysis, with period values close to the period values of the identified FRF resonance peaks and antiresonance valley. It can be seen that these actually represent the first modes of the Zone 3 roof structure. This confirms the capability of the NLTHA to account for a combination of all the various modal responses of the model, allowing for a more realistic representation of the dynamic behaviour of the structure.

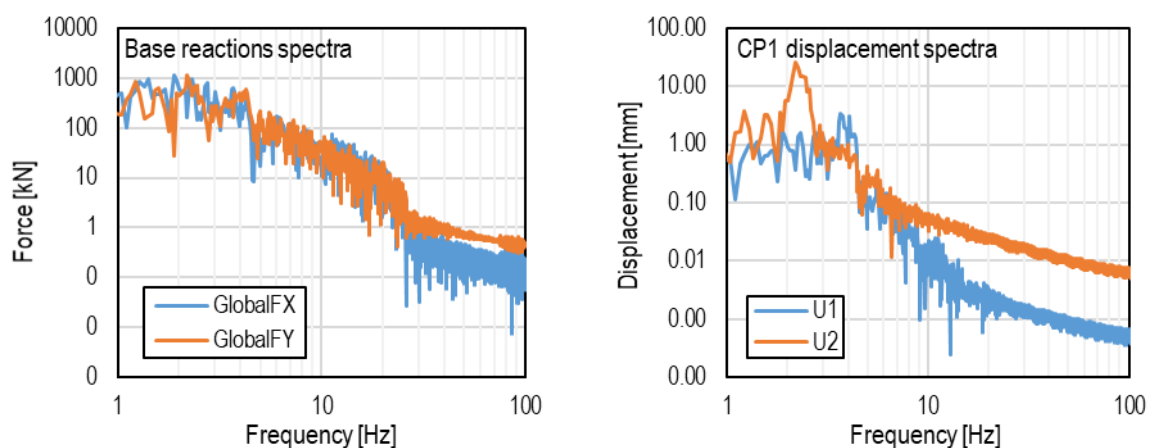


Figure 13-23: NLTHA | Corinth 1981 | Global base reactions and CP1 displacement spectra

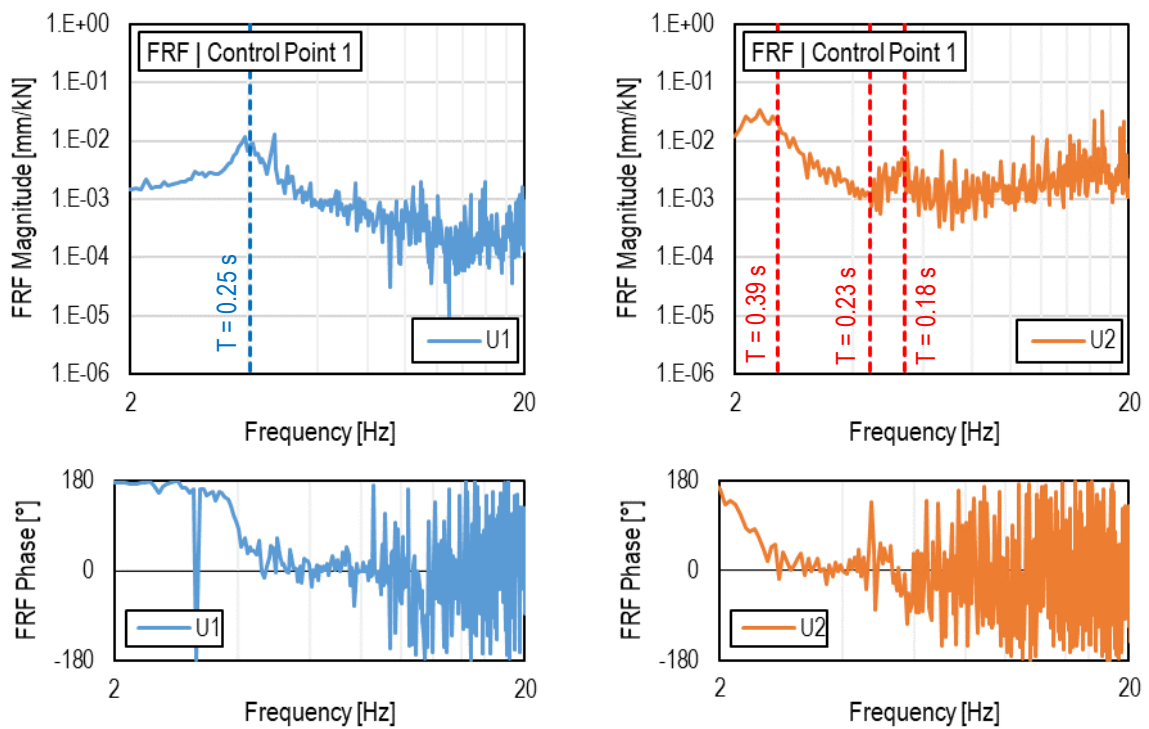


Figure 13-24: NLTHA | Corinth 1981 | FRF estimation and identification of resonance peaks and antiresonance valleys. The dashed lines represent the periods of the fundamental modes identified via linear dynamic analysis

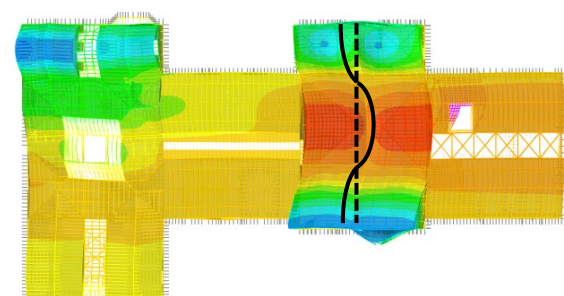
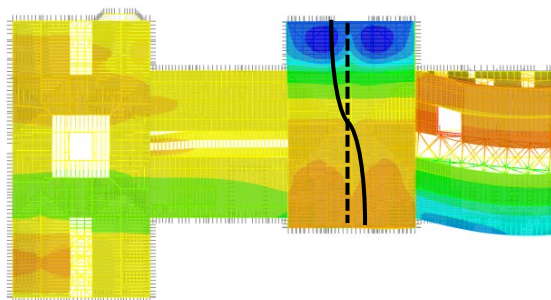
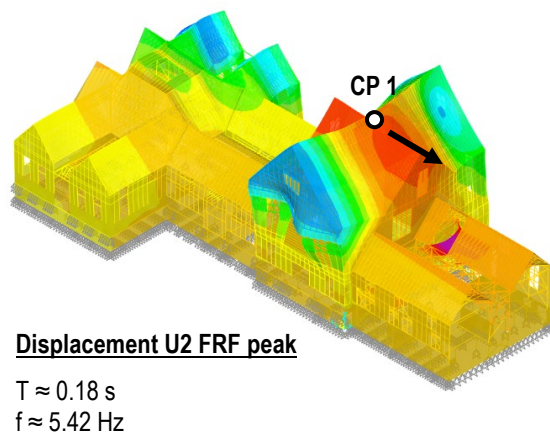
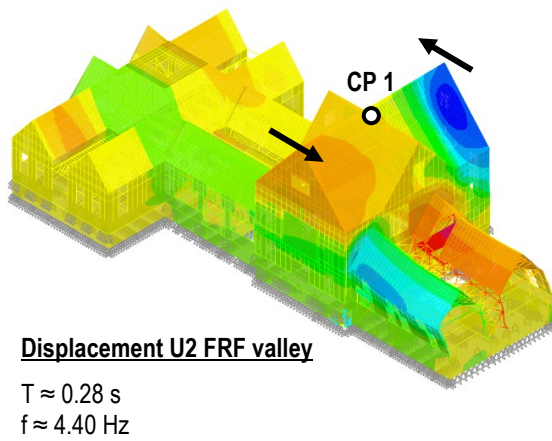
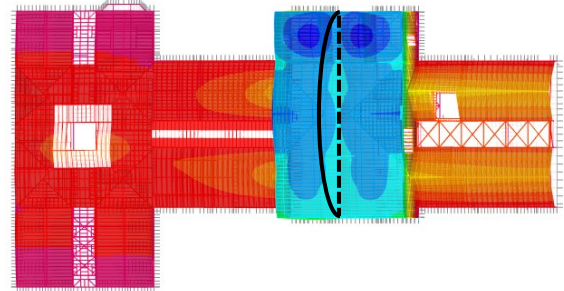
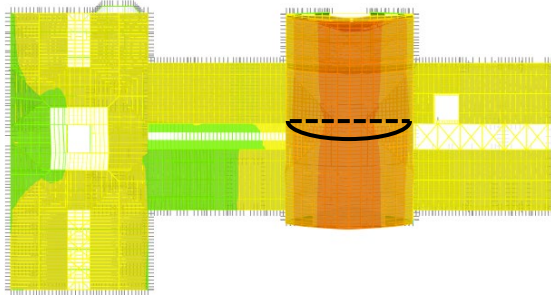
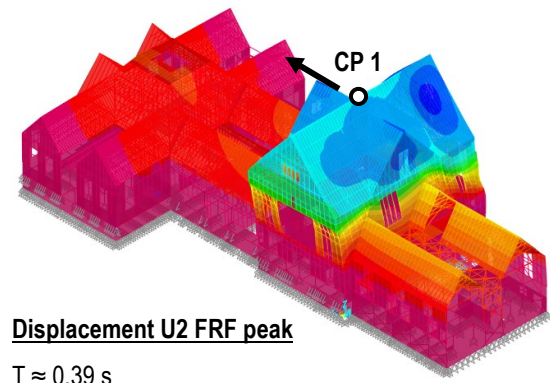
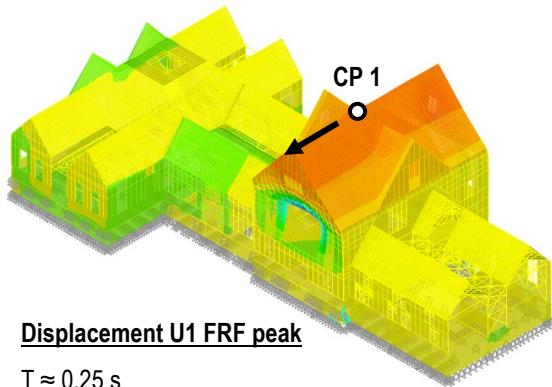


Figure 13-25: NLTHA | Corinth 1981 | Principal modes of the Atrium roof structure (zone 3) derived from the FEM model modal analysis and compared to the FRF resonance peaks and antiresonance valleys

Finally, **Figure 13-26** displays the comparison between the imposed ground motion records and the acceleration time histories measured at the highest point of the building (i.e., CP1). The resonant response of the fundamental mode of the building (i.e., translation of the Zone 3 roof structure in the Y direction, North-to-South, as from **Figure 13-1**) is clearly recognisable from the acceleration charts measured in the Y direction. The acceleration peak for the CP1 ($a \approx 2.29$ g) is reached at around 8.75 s, several seconds after the maximum acceleration value of the input ground motion record in the same direction ($a \approx 0.63$ g reached at 4.14 s). This corroborates the observation done when examining the base shear and CP-1 displacement recordings, evidencing the presence of a dynamic amplification phenomenon experienced by the Zone 3 roof structure. **Table 13-2** reports the maximum input and output (relative to CP1) acceleration values for both the horizontal directions (East-to-West and North-to-South) and the Roof Acceleration Amplification (R.A.A.) factors calculated as the ratio between the peak roof acceleration and the peak ground acceleration.

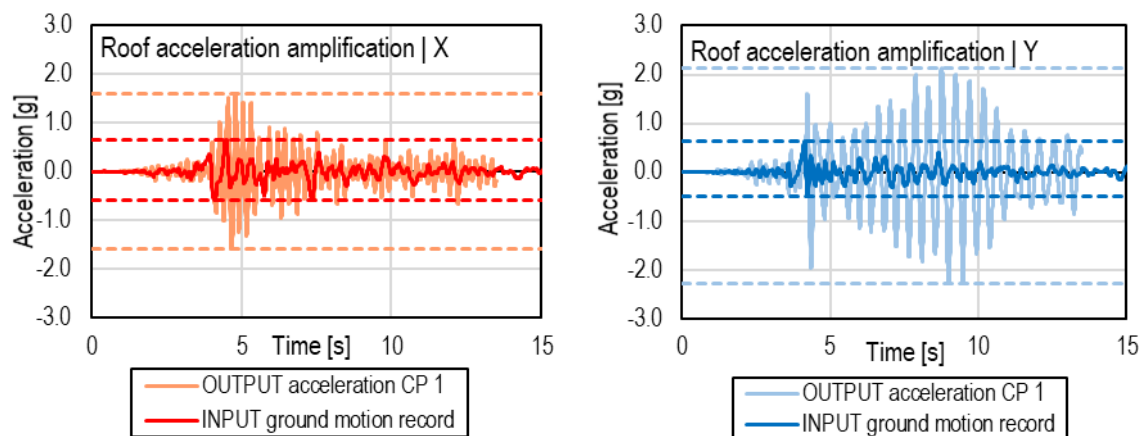


Figure 13-26: Roof acceleration amplification time history charts for Control Point 1 (Zone 3 - Roof) | Corinth 1981

Table 13-6: Roof acceleration amplification factor maximum values | Corinth 1981

Direction	Max INPUT acc. [g]	Max OUTPUT acc. CP1 [g]	R.A.A. factor [-]
X east-west	0.66	1.60	2.43
Y north-south	0.63	2.29	3.65

DRAFT

14 DETAILED SEISMIC ASSESSMENT AND CONCEPT RETROFIT (ZONES 1 AND 6)

14.1 Zone 1

Zone 1 is a four-storey building constructed circa 2010 as an extension to Zone 2 (South Wing). The building has no basement with all its four levels located above the ground.

The heavy Monier terracotta roof of the building is supported on timber purlins that are in turn supported on structural steel rafters. The structure between the roof and the first suspended floor is principally steel framed with plywood floors supported on timber joists. The first suspended floor of the building consists of 110mm thick Uni-Floor flat slab along the eastern perimeter of the building and a 215mm thick Stahlton rib & timber in-fill floor slab. Gravity load support to the first floor is provided by Reinforced Concrete (RC) frames. The building is founded on RC isolated footings and strip shallow foundations.

Lateral force resistance to seismic shaking in both principal directions of the ground floor of the building is provided by RC walls and block walls. A combination of RC block walls and Reidbrace cross-bracing provide lateral force resistance in the upper floors. The roof has Reidbrace cross-bracing as its lateral force resisting system.

For the purposes of analysis, Zone 1 was deemed to be seismically separated from Zone 2 on the northern side by a 50 mm seismic gap.



Figure 14-1. Site plan showing Zone 1 section of the building

14.1.1 Standards and Guidelines

The DSA of Zone 1 has been undertaken in accordance with the latest version of the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines). The Guidelines form part of the new framework for managing earthquake-prone buildings that result from the Earthquake-prone Buildings Amendment Act 2016.

In addition to the Guidelines, the following documents were also referenced for the DSA:

- AS/NZS 1170.0: 2000 – Structural Design Actions. Part 0: General principles
- AS/NZS 1170.1: 2001 - Structural design actions. Part 1: Permanent, imposed and other actions
- NZS 1170.5: 2004 - Structural Design Actions. Part 5: Earthquake Actions – New Zealand
- Technical Proposal to Revise the Engineering Assessment Guidelines, C5 (November 2018)
- Summary of Outcomes from Reidbrace Testing at the University of Auckland (2019)

- NZS 4230: 2004 - Design of reinforced concrete masonry structures

14.1.2 Building Design Life

The building design life is assumed to be 50 years.

14.1.3 Ultimate Limit State (ULS)

This DSA is based on the assumption that the building is considered to be an Importance Level 3 (IL 3) structure, as defined by NZS 1170.0:2002. Importance Level 3 structures, designed to current standards, are required to provide a high margin against collapse from earthquake shaking with a return period of 1,000 years, which is referred to the ULS earthquake shaking herein. A new building designed to this level of earthquake shaking may be damaged after a major earthquake, perhaps beyond repair, but it is expected to provide a high margin of safety for its occupants and allow them to safely exit the building after such an earthquake. A building that meets the minimum requirements of NZS 1170.5: 2004 and the associated material standards achieves at least 100%NBS (IL3).

14.1.4 DSA Assumptions

All gravity loads were derived based on AS/NZS 1170.1: 2002 or as otherwise noted within the DSA calculations. The summary of general seismic parameters used is presented in Table 1-1.

Table 14-1. Seismic Parameters

FACTOR	CLAUSE USED	VALUE SELECTED
Soil Class	NZS1170.5 Cl. 3.1.3	D
Hazard Factor (Z)	NZS1170.5 Cl. 3.1.4	Z = 0.24
Near-Fault Factor N(T,D)	NZS1170.5 Cl.3.1.6	N(T,D) = 1.0
Return Period Factor (R _u)	NZS1170.5 Cl.3.1.5	R _u = 1.3

Material properties used when undertaking the DSA are presented in Table 1-2.

Table 14-2. Material Properties

MATERIAL	STRENGTH	COMMENT
Concrete	45 MPa	Table C5.3 of the Guidelines (2018)
Blockwalls	12 MPa	Type B – Assumed
Reinforcing bars (G300)	324 MPa	Table C5.4 of the Guidelines (2018)
Reinforcing bars (G500)	540 MPa	Table C5.4 of the Guidelines (2018)
Structural steel (G300)	345 MPa	Table C6.2 of the Guidelines (2017)

14.1.5 General Assessment Methodology

A staged analysis approach was adopted for the seismic assessment of the building, starting with simpler analysis methods where appropriate and then undertaking an incrementally increasing level of sophistication of analysis in order to establish the expected seismic performance of the building. The techniques used are generally outlined in the Guidelines. The methodology is briefly summarised below:

- Review of the available structural documentation, to identify the main structural elements and “potential critical structural weaknesses” that may significantly reduce the seismic performance of the building.
- Assess capacity and ductility of RC walls and frame members using the Guidelines – Section C5 (2018).
- Assess capacity of RC block walls – NZS 4230 (2004).
- Assess capacity of floor diaphragm using a simplified method in accordance with the provisions of the Guidelines – Section C5 (2018).

- Determine the seismic demands on the upper floors and the roof structure using parts loadings in accordance with NZS 1170.5.
- Assess Reidbrace cross-bracing system using recent testing results from the University of Auckland (2019). It was assumed that the end connectors and the pins of the Reidbraces have been replaced with recent & compliant components as part of the 2020 seismic strengthening works.
- Assess brace cleat connections using the Guidelines – Section C6.
- Assess steel frame members and their connections using the Guidelines – Section C6.
- Build a three-dimensional computer model of the building to undertake a seismic analysis of the building, see **Figure 14-2**. The foundation of the building was modelled using compression-only springs given the lack of foundation hold down capacity. The stiffness of the foundation was based on upper/lower-bound subgrade modulus (non-liquifiable) provided for the subsoil of the building by Tonkin & Taylor. The Reidbrace cross-bracing was modelled using tension-only frame elements.
- Calculation of the expected gravity and seismic loadings on the building following current New Zealand loading standards (NZS 1170.0:2002 and NZS 1170.5:2004).
- Determine the seismic demands on the main structure using a non-linear static analysis in accordance with Section C2 of the guidelines (2017). A non-linear static analysis was considered appropriate given the compression-only foundation and the tension-only Reidbrace cross-bracing. The structure was assumed to have a nominally ductile response.
- Determination of the expected seismic performance of the building compared with the minimum expected of an equivalent new building at the site based on review of existing documentation and limited inspection.

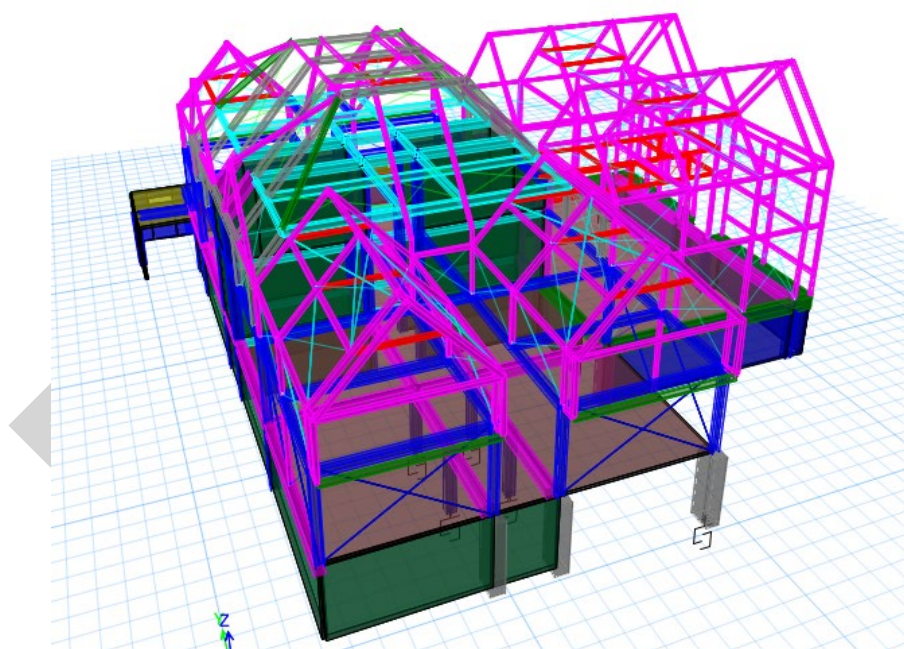


Figure 14-2. 3D model of Zone 1 structure for seismic analysis

14.1.6 Assessment Findings

The assessment identified that the capacity of the main lateral load resisting elements of the building is likely to be exceeded at 40%NBS (IL3). Therefore, Zone 1 is considered to currently achieve an earthquake score of 40%NBS (IL3) and the building does not meet one of the criteria that could get it categorised as an Earthquake Prone Building by RDC.

A summary of the assessment of the seismic capacity of the elements of the building is provided in the following tables. Seismic capacities are assessed in terms of percentage of New Building Standard (%NBS). The elements have been scored against IL3 requirements.

Table 14-3 Longitudinal element capacities scoring <100% NBS (IL3)

Location	Element	IL3 %NBS	Comment
Roof	Reidbraces RB16	75	The capacity of the braces is limited by their tensile strength. If these braces are connected to a 12mm or thicker plate (G300), the braces could develop a higher level of ductility and score 100%NBS.
Level 3 floor	Brace R20	90	The capacity of these braces is limited by their tensile strength.
Level 3	Reidbraces RB20	85	The capacity of these braces is limited by their tensile strength.
Level 1	Reidbraces RB25	80	The capacity of these braces is limited by their tensile strength. Details of the end connections of these braces is not provided in the structural drawings. It is assumed that the plate these braces are connected to would have a minimum of 330mm long weld (6FW) to the supporting structural elements. The base connections of the columns these braces are connected also scored 70%NBS.
Level 2 (Grids 4 & 5)	RC Column 600x600mm	95	The capacity of these columns is limited by their shear strength.
Level 2 (Grids 4 & 5)	RC Column 400x400mm	80	The capacity of these columns is limited by their flexural strength.
Ground floor (Grid 1 & 3)	RC Walls	80	The capacity of these shear walls is limited by their flexural strength.
Ground floor	Floor diaphragm	40	The capacity of the floor diaphragm is limited by the strength of its connection to the wall on Grid A. The connection to the wall on Grid 7 scored 55%NBS. The structural drawings indicate there are 500mm wide penetrations along these walls leaving minimal connections between the walls and the floor diaphragm.

14.1.7 Recommendations for strengthening

There are some areas in the building which would need to be seismically upgraded to increase the capacity of the buildings to greater than 80%NBS:

- **RB25 Reidbraces:**
Provide new epoxy anchors to the existing base-plate connections of the columns the RB25 braces are connected to in order to increase the strength of the connections.
- **First floor diaphragm:**
In-fill portions of the existing long & wide floor penetrations along gridlines A & 7 to improve the floor-wall connections and enable adequate transfer of forces to the walls.

Drawings to illustrate the potential strengthening schemes are included in [Appendix 4](#).

14.2 Zone 6

Zone 6 of Rotorua Museum has been designed in circa. 2007 and constructed shortly thereafter. This portion of the building was designed as an extension to the main building, see [Figure 14-3](#). The building is seismically separated from the main building on the South. The building's gravity structure consists of steel portal frames (mainly composed of 150x150x6 SHS sections) supported over the basement perimeter walls. The frame connections are mostly pinned connections with limited moment resisting ability, mainly due to the thin 10mm flange plates used for splice detailing.

The basement structure consists of perimeter reinforced concrete walls and a single block wall running N-S in the centre. The basement lid structure is formed from rib and infill slab with 75mm of topping concrete reinforced with non-ductile 665 mesh. However, due to the regular geometry of the building, the diaphragm is considered deemed to comply as per the provisions of C5- Technical Proposal to revise the Engineering Assessment Guidelines (referred to as the yellow book).

The foundations for basement walls are 2.5m wide by 400mm deep reinforced concrete strip footings. The footings are tied with the basement slab. There is an opening in the ground floor slab to allow for services, which detaches the diaphragm from basement wall located on the North elevation.

The roof structure is braced using the Reidbrace system as cross bracing. The cross bracing is responsible to collect the roof and ceiling inertial loads and transfer this as an in-plane load on the plywood + steel SHS composite shear walls.

The ventilation structure is laterally supported by the ridge beam acting as a top prop followed by the Reidbrace system in the roof acting as the lower prop. 12mm F11 grade plywood shear walls with spans separated by 150x150x6 SHS framing are the main lateral resisting system for the structure. The walls are located on all four sides of the structure.



Figure 14-3. Site plan showing Zone 6 section of the building

14.2.1 Standards and Guidelines

Reference has been made to NZS 1170 for calculating loadings and to the relevant sections of *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines) for calculating member capacities. The lateral load resisting system of the building has been checked based on Equivalent Static Analysis. The roof bracing checks are based on loads obtained from the Parts and Components provisions of NZS1170.5.

In addition to the above, the following documents were also referenced for the DSA:

- Original design calculations by Sigma Consultants Ltd., dated September 2007;
- Strengthening calculations by GDC Consultants Ltd, dated May 2020;
- Architectural drawings by Sigma Consultants Ltd., dated September 2001;
- North wings bracing alterations at ceiling level by Kerslake and Partners Consulting Engineers, dated August 1997;

- Inspection records by Terrane Consultants, dated March 2007;
- Original structural drawings by Sigma Consultants Ltd., dated December 2007;
- Strengthening drawings by GDC Consultants Ltd., dated May 2020;
- Design features report by GDC Consultants Ltd., dated May 2020.

14.2.2 Building Design Life

The building design life is assumed to be 50 years.

14.2.3 Ultimate Limit State (ULS)

This DSA is based on the assumption that the building is considered to be an Importance Level 3 (IL 3) structure, as defined by NZS 1170.0:2002. Importance Level 3 structures, designed to current standards, are required to provide a high margin against collapse from earthquake shaking with a return period of 1,000 years, which is referred to the ULS earthquake shaking herein. A new building designed to this level of earthquake shaking may be damaged after a major earthquake, perhaps beyond repair, but it is expected to provide a high margin of safety for its occupants and allow them to safely exit the building after such an earthquake. A building that meets the minimum requirements of NZS 1170.5:2004 and the associated material standards achieves at least 100%NBS (IL3).

14.2.4 DSA Assumptions

All gravity loads were derived based on AS/NZS 1170.1: 2002 or as otherwise noted within the DSA calculations. The summary of general seismic parameters used is presented in [Table 14-4](#).

Table 14-4. Seismic Parameters

FACTOR	CLAUSE USED	VALUE SELECTED
Soil Class	NZS1170.5 Cl. 3.1.3	D
Hazard Factor (Z)	NZS1170.5 Cl. 3.1.4	Z = 0.24
Near-Fault Factor N(T,D)	NZS1170.5 Cl.3.1.6	N(T,D) = 1.0
Return Period Factor (R _u)	NZS1170.5 Cl.3.1.5	R _u = 1.3

Material properties used when undertaking the DSA are presented in [Table 14-5](#).

Table 14-5. Material Properties

MATERIAL	STRENGTH	COMMENT
Concrete	37.5 MPa	Zone 6 as built drawings
Block walls	18 MPa	Type B – Assumed
Reinforcing bars (G300)	324 MPa	Table C5.4 of the Guidelines (2018)
Timber		Section C9 of the Guidelines, assumed between Radiata Pine & Douglas Fir
Structural steel (G300)	287.5 MPa	Table C6.2 of the Guidelines (2017)
Plywood	12mm F11 Grade	NZS 3603:1993, Table 6.14
Structural steel (SHS sections) (G350)		NZS 3404:1997

As per the original design philosophy, the structure has been designed for an elastic response in order to have minimal cracking for events less than the design earthquake, referring to Serviceability Limit State (SLS) level events. For the purpose of this DSA, the main consideration would be 'life safety'. The DSA process does not directly assess damage caused to cladding systems of the building and maintenance required after a SLS level earthquake event. The process

however addresses the 'life safety' risk of any damage caused to heavy cladding, secondary structural and non-structural elements if these are to collapse and/or block exit routes in a design level earthquake event.

The primary lateral system for this building consists of plywood diaphragm walls with orthogonal members formed from 150SHS steel sections. There are inevitable deformations associated with nail slip at plywood to steel connections which will contribute to (at least) a nominal ductile response from the structure. The steel detailing as inspected from the available structural drawings is deemed sufficient and based on judgment this should be compatible with the nominally ductile response of the plywood shear walls. Therefore, for the purpose of this assessment a ductility factor (m_u) of 1.25 and a structural performance factor (S_p) of 0.9 has been adopted.

The present roof diaphragm consists of tension only Reidbrace. As such, the roof diaphragm is flexible and the seismic load on each shear wall shall be based on tributary areas.

14.2.5 General Assessment Methodology

The DSA has been undertaken based on simple hand checks for load takedown and for the purposes of calculating seismic demands on the roof, the plywood superstructure and the basement concrete structure. Sophisticated FEM models are developed to check the plywood shear wall capacities composite with the steel SHS framing members and to accurately assess the roof steel structure. Steps outlined below were followed sequentially to complete the DSA for zone 6.

- Review of the available structural documentation, to identify the main structural elements and "potential critical structural weaknesses" that may significantly reduce the seismic performance of the building.
- Assess capacity and ductility of RC walls and frame members using the Guidelines – Section C5 (2018).
- Assess capacity of RC block walls – Section C7 (2017).
- Assess capacity of floor diaphragm using a simplified method in accordance with the provisions of the Guidelines – Section C5 (2018).
- Determine seismic demands on the plywood box structure and on the roof-structure using the equivalent static analysis approach and parts loadings in accordance with NZS 1170.5 respectively.
- Assess Reidbrace cross-bracing capacity using recent testing results from the University of Auckland (2018). It was assumed that the end connections for the roof Reidbrace and pins can undertake demands corresponding to 80%NBS. This assumption will be confirmed with Reid in the detailed design phase.
- Calculation of the expected gravity and seismic loadings on the building following current New Zealand loading standards (NZS 1170.0:2002, NZS1170.1:2002 and NZS 1170.5:2004+A1).
- Determine the seismic demands in accordance with Section C2 of the guidelines (2017).
- Build a 2D strut & tie model to accurately assess the plywood shear wall capacity (Moroder et al., 2015).
- Build a 3D model of the roof steel structure to accurately assess the tension demands in the Reidbraces.
- Assess steel frame members and their connections using the Guidelines – Section C6 (2017).
- Assess basement reinforced concrete structure for elastic demands from the superstructure and demands from the dead and imposed loading on ground level podium slab.
- Determine the expected seismic performance of the building in terms of %NBS, IL3, when compared with the minimum expected of an equivalent new building at the site based on our review and limited inspection.

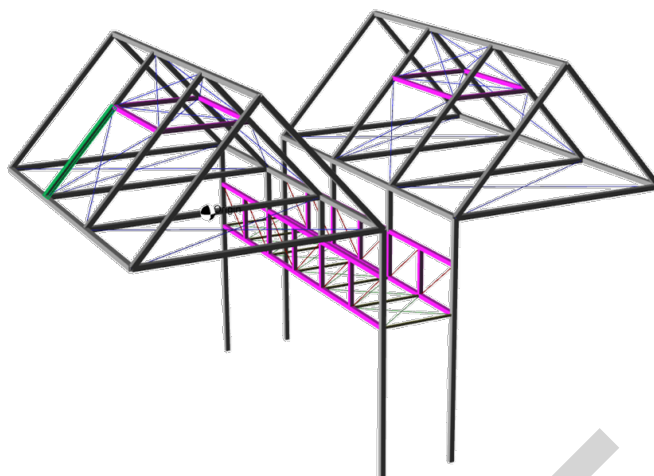


Figure 14-4. FEM model for modelling and analysis of Zone 6 roof steel bracing

14.2.6 Assessment findings

The detailed seismic assessment identified that the capacity of the main lateral load resisting elements of the building is likely to be exceeded at 80%NBS (IL3). Therefore, Zone 6 is considered to currently achieve an earthquake score of 80%NBS (IL3) with an alpha rating of “A” as per Table A3.1 of Part A – Assessment Objectives and Principles, the Guidelines.

A summary of the assessment outcome for the elements of the building is given in the following table. Seismic capacities for the existing building elements are mentioned in terms of percentage of New Building Standard (%NBS) score when compared to an identical element designed to current standards. The elements have been scored against IL3 level seismic demands at the site.

Table 14-6 Element capacities in terms of %NBS (IL3)

Location	Element	IL3 %NBS	Comment
Roof	Reidbraces	80	The capacity of the braces is limited by their tensile strength. The yield strength of historic RB20 has been limited to 157kN (for this DSA) as noted from Auckland University test results.
Plywood walls composite with SHS framing	Shear walls	91	The capacity is limited by the capacity of 140*45 SG8 studs.
Basement reinforced concrete shear walls	Shear walls	100	NA
Basement reinforced concrete masonry 20-series walls	Shear wall	100	NA
Basement diaphragm	<i>Podium diaphragm</i>	100	Rectangular regular diaphragm. The service voids for A/C ducting on the north side of the building need to be filled to have shear transfer between the podium diaphragm and the north basement RC wall. %NBS score contingent on diaphragm infilling being

			undertaken.
Foundations	Strip footings	100	NA

14.2.7 Recommendations for strengthening

All the building elements score more than 80%NBS, IL3. However, the following items need to be addressed to maintain the assumptions made during the detailed seismic assessment.

- Roof Reidbraces: Obtain confirmation from Reid on whether the installed end connections and pins are compliant for capacities obtained from University of Auckland test results.
- Ground floor diaphragm: The service voids for A/C ducting on the north side of the building need to be filled to have shear transfer between the podium diaphragm and the north basement RC wall. Additional epoxy dowels need to be provided in these voids in order to develop adequate shear friction transfer capacity.

Drawings to illustrate the proposed ground floor diaphragm strengthening are included in [Appendix 4](#).

15 CONCRETE FLOOR AND RETROFIT STUDY

This section reports the analyses performed to examine the behaviour of the existing unreinforced pumice concrete floor under static gravity loads and describes the two proposed interventions of structural reinforcement. **Figure 15-1** shows a reference image of the existing concrete floor structure (more details from the on-site assessment can be found in Section 4 of the report). Most of the original part of the structure still features the original pumice concrete floor with vaulted floor geometry. Zone 2 (the intermediate South Wing of the building) experienced a structural modification to the original floor structure in 1967 when the original floor was topped by an additional 90 mm thick slab reinforced with HRC 665 mesh, still present to this day. Zone 3 has also experienced a modification to the original floor structure (exact date unknown), where the original floor was topped with an approx. 30-40mm overlay with wire mesh and newer tiling.

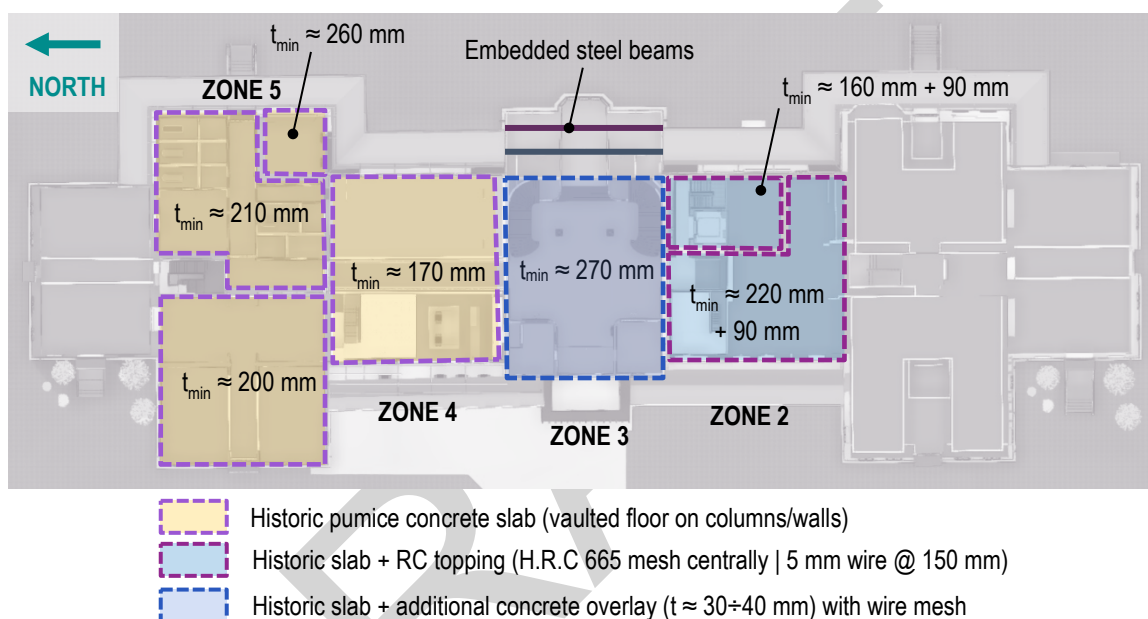


Figure 15-1: Concrete floor minimum thickness and structural changes over the years for the original part of the building (zone 2-3-4-5)

15.1 As-built assessment

To study the structural behaviour of the concrete floor in order to facilitate development of a suitable remediation to the extensive cracking, the analysis was carried out on an isolated module of the floor which was considered representative for the global static behaviour of the structure (i.e., vaulted slab portion supported by four columns as displayed in **Figure 15-2**). The floor module was modelled adopting the Finite Element (FE) software SAP2000 and the FE software TNO DIANA (Manie and Kikstra, 2014). For the SAP2000 model the vaulted floor geometry is simplified to a 'straight plate' geometry adopting layered shell elements modelled using a modified version of the "Darwin-Pecknold" constitutive behaviour (as already described in Section 7 of the report). This is a reasonable assumption due to the relatively small curvature of the vaults' arches. Nevertheless, the outcomes from the TNO DIANA model are used as a reference to assess the plausibility of this assumption. For the TNO DIANA model the vaulted floor geometry is considered in its entirety by using 3D tetrahedral solid elements (**Figure 15-2**). A total strain constitutive model with smeared cracking was adopted (Selby and Vecchio, 1993). Both the compressive and tensile behaviours are based on fracture energy and simulated using a parabolic curve and a linear function respectively. Fracture energy values for tension and compression ($G_f = 0.02$ Nmm/mm²), were determined consistently with CEB-FIP Model Code for concrete structures (1990).

The compressive strength and the Modulus of Elasticity of concrete were assumed equal to 6.00 MPa and 6000 MPa respectively, in agreement with the average values observed in the experimental testing campaign (see Section 6 of the

report). Regarding the value of concrete tensile strength a sensitivity analysis was performed by carrying out 3 different simulations for each model with tensile strength values equal to 1.00 MPa, 0.10 MPa and 0.05 MPa. For the boundary conditions of both models the base joints of the columns had all the translational restraints fixed and rotational restraints released to simulate a pinned condition.. The joints on the lateral edges of the floor had the horizontal translational DOFs (U_x , U_y) fixed and the vertical (U_z) released, in order to replicate the continuation of the concrete floor slab outside the limits of the assumed structural module.

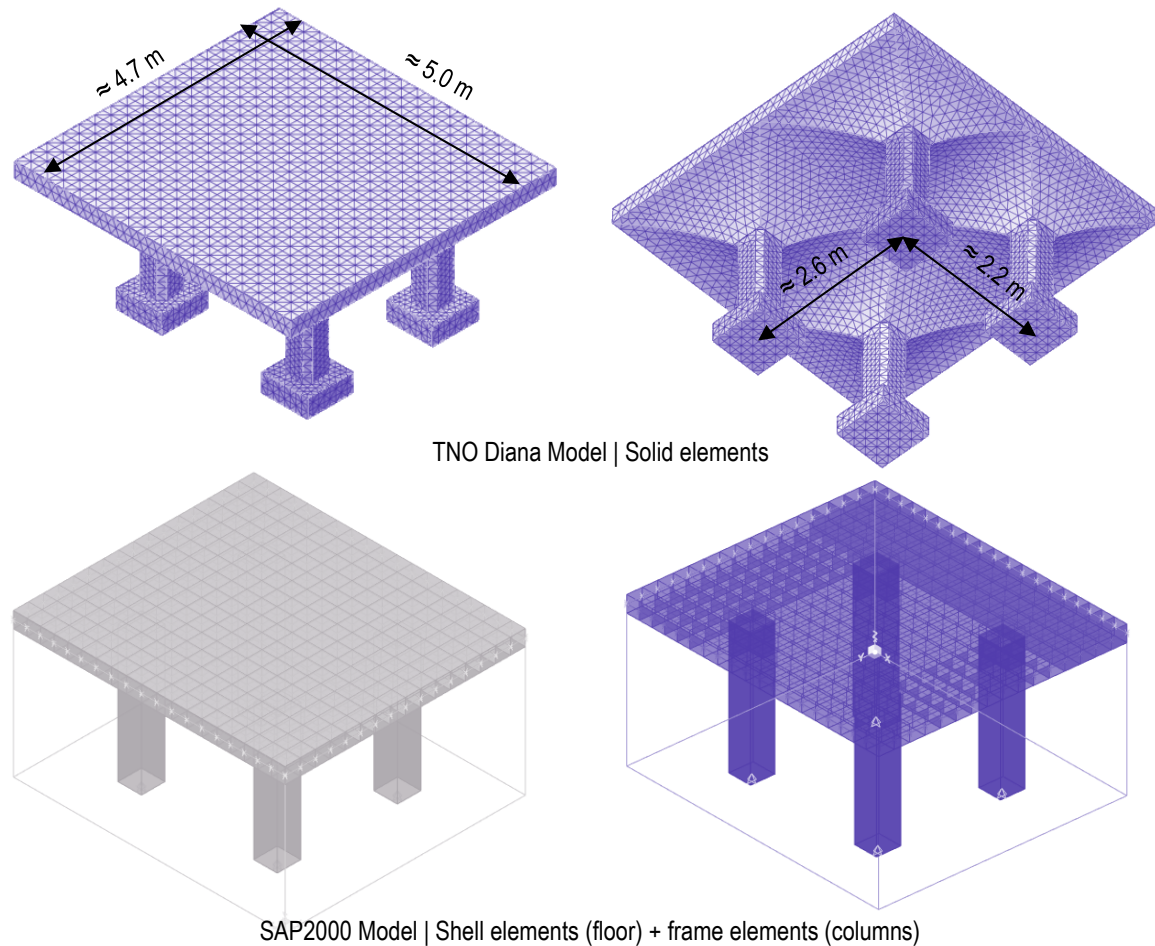


Figure 15-2: Numerical models used to assess the as-built condition of the selected concrete floor module

The selected loading condition for the assessment of the as-built concrete floor is presented in **Figure 15-3**. It considers the static load as from the Standard (NZS 1170.1) for museum floors and art galleries for exhibition purposes. For each of the analyses the load (comprising also the dead load) is applied progressively starting from the unloaded structure.

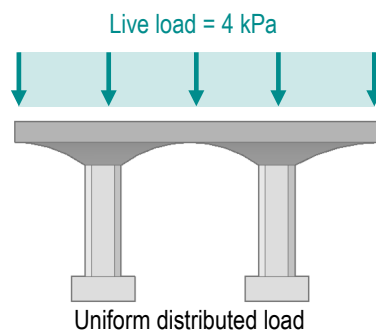


Figure 15-3: Loading condition adopted for the as-built assessment

UNIFORM DISTRIBUTED LOAD (4 kPa) | Last analysis step

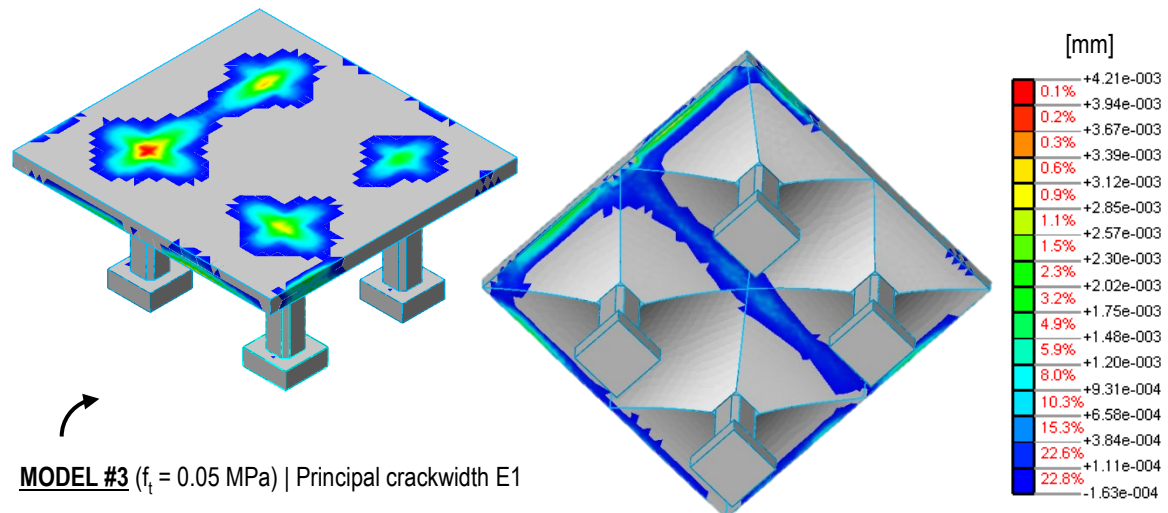
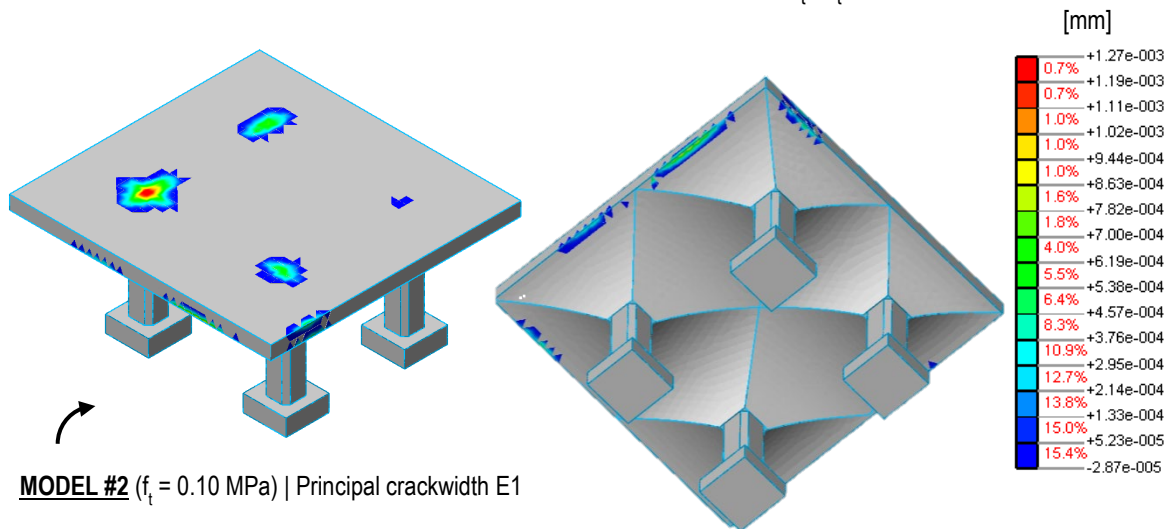
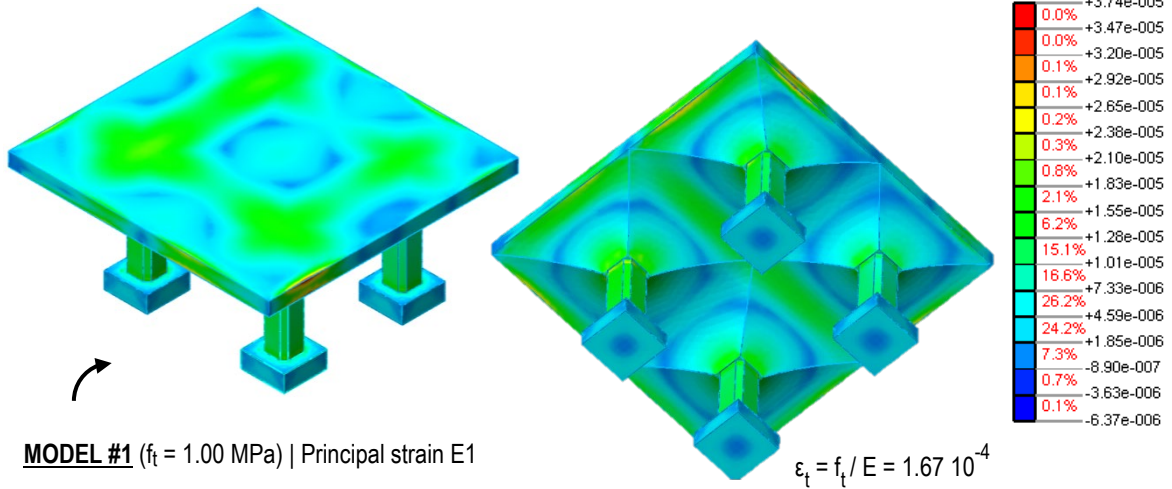


Figure 15-4: TNO DIANA model: outcomes for the uniform distributed load condition (4 kPa) in terms of principal strain (positive tensile strain convention) and principal crack width for different concrete tensile strength value assumptions: 1.00 MPa, 0.10 MPa and 0.05 MPa | Note: numerical values in the chart legends are reported in black colour; in red colour are reported the proportions of finite elements comprised in the interval between the two values

Figure 15-4 reports the outcomes obtained from the TNO DIANA model under the uniform distributed load condition in terms of principal strain and principal crack width. It is seen that if the tensile strength of concrete is assumed equal to 1.00 MPa the principal tensile strains are below the limit strain value and the structure remains uncracked. Considering the second option of tensile strength equal to 0.10 MPa, cracks appear mainly on the top surface of the vaulted floor in the vicinity of the supporting columns underneath, where the negative bending moment reaches its maximum. The schematics reported in **Figure 15-4** display the condition at the last step of analysis; the first cracks however appear at a step of the analysis where the applied uniform distributed load is equal to approximately 1.6 kPa. Finally, the last analysis was performed with a concrete tensile strength equal to 0.05 MPa. As expected, it is seen that the cracking pattern is similar, but more widespread, to the one obtained from the model with $f_t = 0.10$ MPa. Large cracks are evident also on the bottom surface of the vaulted floor along the midspan section line between the column supports. Also of note is that the cracks on the top surface appear before the whole dead load of the structure is applied (approximately at a step corresponding to the 75% of the dead load). The average tensile strength obtained from the experimental campaign was 0.8 MPa, closer to the 1MPa assumption where principal tensile strains are below the limit strain value and the structure remains uncracked for 4kPa.

UNIFORM DISTRIBUTED LOAD (4 kPa) | Last analysis step

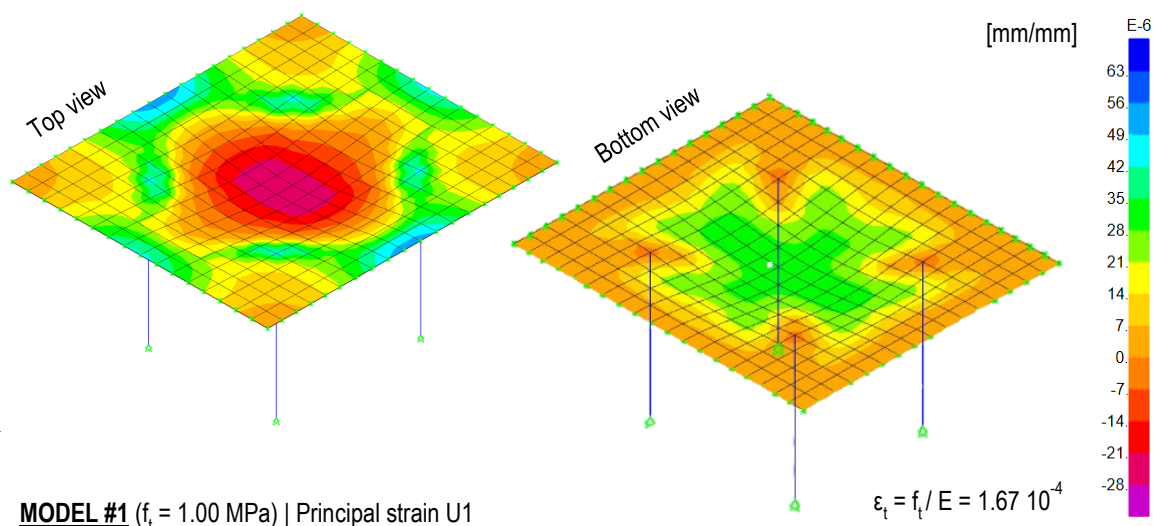
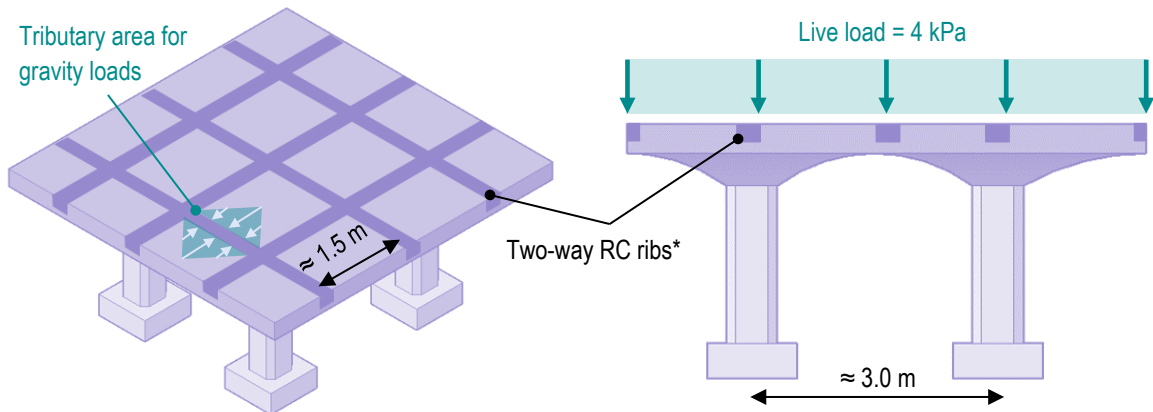


Figure 15-5: SAP2000 model: outcomes for the uniform distributed load condition (4 kPa) in terms of principal strain ($f_t = 1.00$ MPa) | Positive tensile strain convention

Figure 15-5 shows the principal strains for the SAP2000 model considering the tensile strength equal to 1.00 MPa and the uniform distributed load condition. Despite some minor variability in the values (e.g., on the top surface over the columns supports: tensile strain equal to $21 \div 28 \mu\epsilon$ for the SAP2000 model and $15 \div 21 \mu\epsilon$ for the TNO DIANA model), the strain pattern is similar to the one obtained via the TNO DIANA model. On the top surface the maximum tensile strains are located in the vicinity of the column supports, while on the bottom surface the maximum tensile strains are situated along the midspan section lines between the column supports.

15.2 Retrofit proposal

This section reports the two proposed retrofit interventions aimed at improving the structural behaviour of the existing concrete floor for both static and seismic loading conditions. The two proposed interventions differentiate between the part of the floor structure which can be accessed for structural reinforcement work from top (Zone 2-3-4 and the West part of Zone 5) and the part of the floor structure where the top surface shall remain untouched for architectural heritage purposes (East part of Zone 5).



* Plus an additional FRCM topping, and anchoring fasteners

Figure 15-6: Schematic representation of the selected retrofit intervention for Zone 2-3-4 and the west part of Zone 5

Figure 15-6 exhibits a schematic view of the floor strengthening achievable by working from the top surface: two Reinforced Concrete (RC) beams (section area $A = 150 \times 100 \text{ mm}^2$) carved inside the existing slab. As a further measure to enhance the mechanical behaviour of the floor, an additional FRCM topping connected to both the existing floor and the RC beams is proposed. In doing so it is possible to improve the load-carrying capacity for vertical loads of the floor as well as the diaphragm action for horizontal forces acting during earthquake events. In addition, this solution prevents a large increase in floor thickness of the existing floor, which in turn avoids cost and construction complexities.

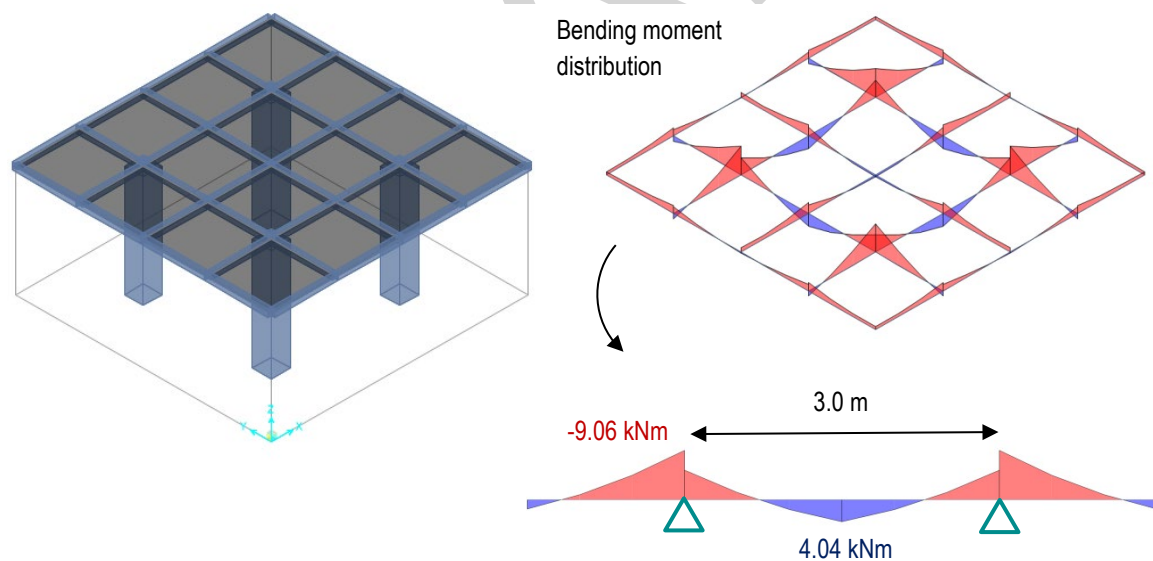
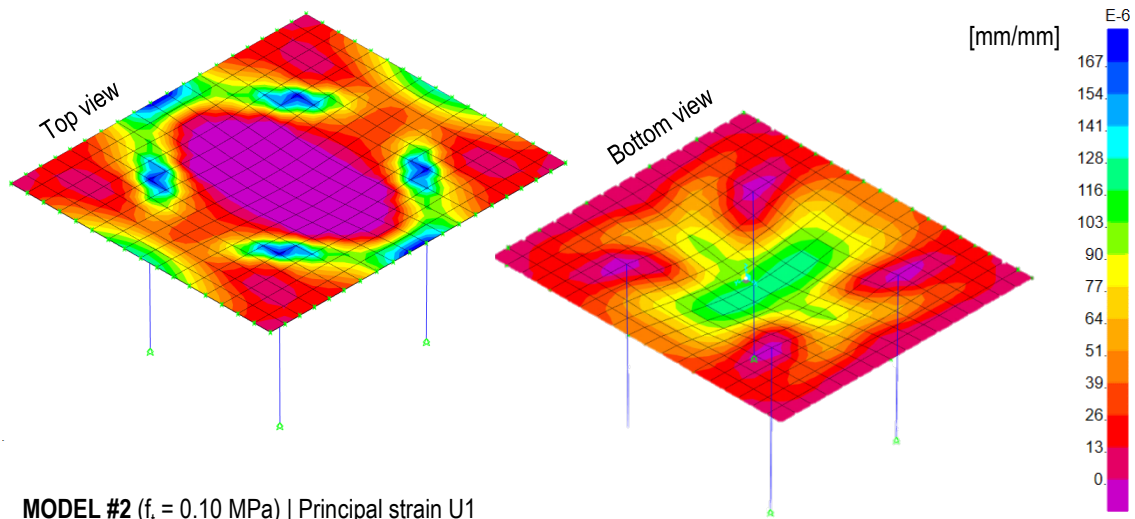


Figure 15-7: Bending moment distribution on the two-way RC ribs under the 4 kPa uniform distributed load condition

Figure 15-7 shows the SAP2000 model used to estimate the bending moment demand on the two-way RC beams assuming that all the uniform distributed load equal to 4 kPa is applied to them. From this simplified scheme it is possible to extract the maximum acting bending moment expected on a single RC element. A preliminary design of the reinforcement is proposed based on the geometric properties of the RC beam previously reported (width of 150 mm and depth of 100 mm) and assuming an effective cover from the top edge of 40 mm (and of 20 mm from the bottom edge, in contact with the existing concrete floor structure). Then, considering a concrete grade of 20 MPa (NZS 3101) and a rebar steel grade of G300E (AS/NZS 4671), the top and bottom reinforcement consist both of 5 longitudinal steel bars with a diameter equal to 16 mm. The associated resisting bending moments of the concrete section are equal to approximately 11.8 kNm for negative bending action and equal to 10.0 kNm for positive bending action.

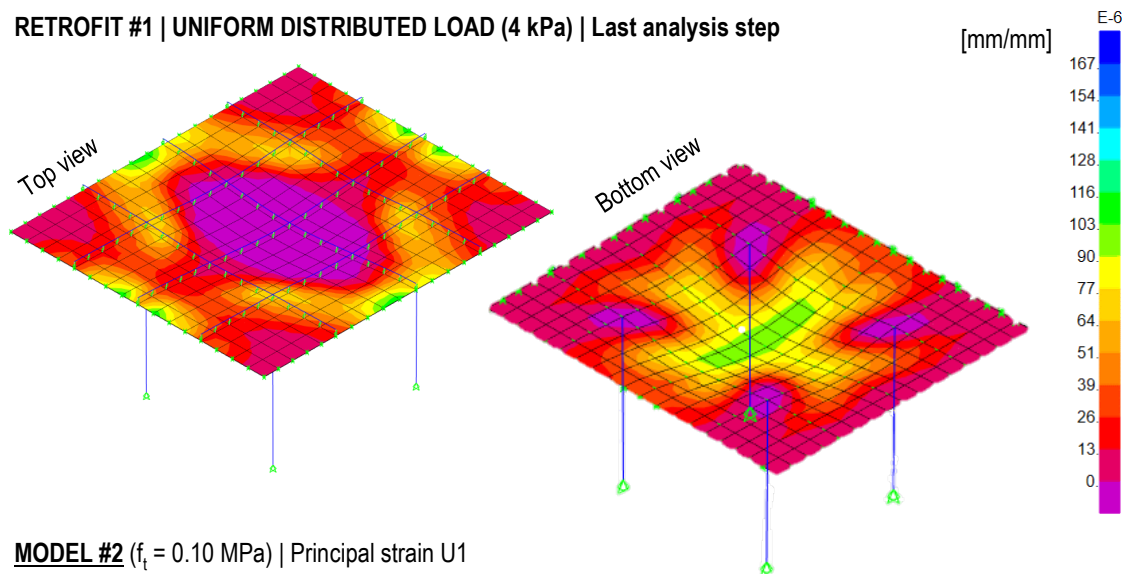
Figure 15-8 displays the comparison in terms of principal tensile strain between the as-built and the retrofit model using the software SAP2000 and assuming a concrete tensile strength equal to 0.10 MPa. Even considering the lower bound tensile strength value equal to 0.10 MPa, the structural reinforcement proposed is able to withstand the required static load demand. It can be observed that, as expected, the presence of the reinforcement helps in reducing the tensile strain acting on the existing unreinforced concrete floor. The improvement of tensile capacity is more evident on the top surface of the floor, where most of the new steel bars are located, but as a secondary effect there is beneficial impact of the reinforcement also on the inner surface of the vaulted floor.

AS-BUILT | UNIFORM DISTRIBUTED LOAD (4 kPa) | Last analysis step



MODEL #2 ($f_t = 0.10$ MPa) | Principal strain U1

RETROFIT #1 | UNIFORM DISTRIBUTED LOAD (4 kPa) | Last analysis step

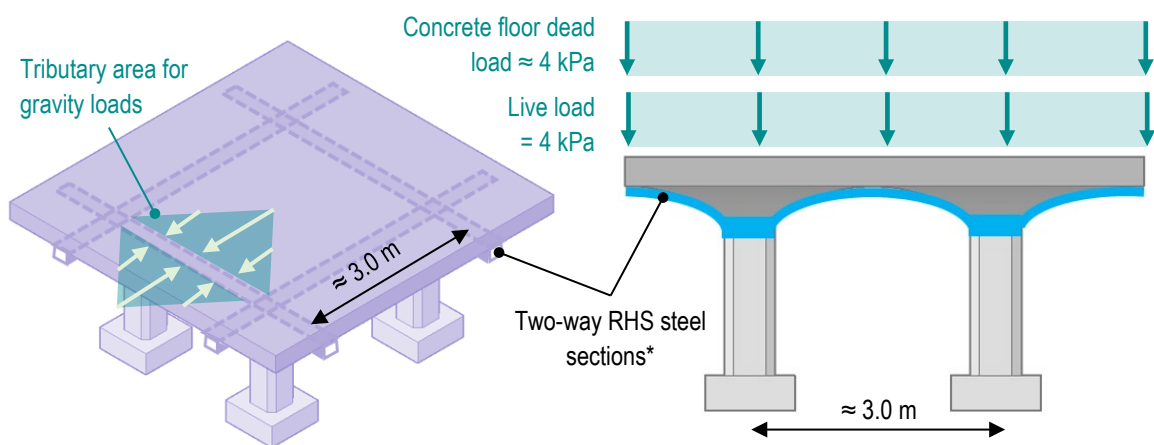


MODEL #2 ($f_t = 0.10$ MPa) | Principal strain U1

Figure 15-8: SAP2000 models: results comparison of as-built condition and retrofit proposal in terms of principal tensile strain

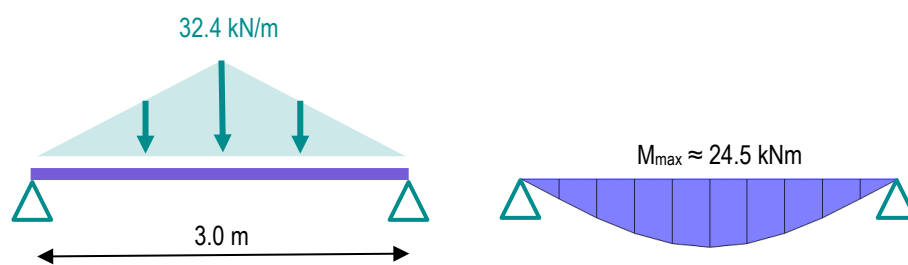
Finally, **Figure 15-9** shows the structural reinforcement proposed for the existing vaulted floor of the east portion of Zone 5 (North wing of the building). Due to the heritage significance of this region, the retrofit cannot be undertaken from the top surface of the floor. The proposed solution is hence to insert Rectangular Hollow Section (RHS) steel profiles at the basement level along the supporting column grid lines (**Figure 15-9**). The RHS steel elements will be bent to adapt to the vaulted floor inner surface and will be connected to the existing concrete floor using mechanical fasteners and to the columns via specifically designed steel “collars”.

For a preliminary design of the RHS section the steel element is assumed to be in simply supported boundary conditions between the span of the two column supports (**Figure 15-10**), ignoring the curvature of the steel profile (which in reality would increase the resistance of the profile due to arching effects). Because of the grid geometry of the structural reinforcement the load applied to the element is a linear variable load with a maximum located at the midspan section. Assuming a steel material with a nominal yield strength of 300 MPa (AS 3679) two loading conditions are considered for the design of the beam section: only the 4 kPa live load acting on the steel retrofit structure and a total load of 8 kPa comprising of both the live load and the concrete floor dead load (adopting the appropriate load combination factors as from NZS 1170.0). In the first case (i.e., 6 kPa) the maximum bending moment is equal to 13.7 kNm and a RHS steel section 125x75x6 (width x height x thickness in mm) is proposed (resisting bending moment of approximately 20.42 kNm). For the second loading option (i.e., 10.8 kPa) the maximum bending moment is equal to 24.5 kNm and a RHS steel section 150x100x6 is suggested (resisting bending moment of approximately 35.24 kNm).



* Bent to adapt the vaulted floor inner surface curvature and connected to the floor via mechanical fasteners

Figure 15-9: Schematic representation of the selected retrofit intervention for the east part of Zone 5



Linear load on a single RHS element (From 10.8 kPa uniformly distributed)

Figure 15-10: Bending moment distribution for a single RHS steel section element under the "dead load + live load" uniform distributed load condition

16 SUMMARY & CONCLUSIONS

The on-site investigation, detailed seismic assessment and concept retrofit work of Rotorua Museum was undertaken by DIZHUR Consulting throughout the period from May – December 2021, and the conclusions are summarised below:

Zones 2 to 5: The museum today is comprised of six zones, which are a combination of historical and newer constructions: Zones 2 to 5 comprise the original historic areas. Zone 3 is the Central Atrium, and is flanked by Zone 2 to the south, and Zone 4 to the north. Both these structures are largely original, and are separated from Zone 3 by approx. 10m tall concrete gable walls. The east and west walls of all three zones comprise approx. 1.5m tall concrete nibs topped with timber framed walls infilled with approx. 40-50mm thick pumice concrete panels. Zone 5 lies to the north of Zone 4, separated by another 10m tall concrete wall, and is of largely the same construction as the previously mentioned zones, whilst also containing a number of internal partition walls of similar configuration to the east and west perimeter walls, with lath and plaster internal linings. Zones 2 to 5 all sit on a pumice concrete two-way vaulted floor slab supported on original unreinforced pumice concrete columns, walls and foundations, with localised newer basement construction having been undertaken in various regions over the years. The roof structure across these zones typically consists of heavy ceramic roof tiling supported on timber sarking and trusses. Detailed information regarding Zones 2 to 5 can be found in §4.2 to §4.5.

Zones 1 and 6: Zone 1 lies to the south of Zone 2, and was partially undertaken at around the original time of construction. It was extended in 1983, and again in 2011. This zone is composed of a combination of reinforced concrete masonry walls and steel framing, with a central 'core' of reinforced concrete masonry walls enclosing the lift and staircase structures. It contains a concrete slab on grade, with the use of precast reinforced concrete ribs and timber flooring at upper levels. Zone 6 lies to the north of Zone 5, and was constructed circa. 2007-2008. It was built predominantly using cast in-situ reinforced concrete walls and plywood timber walls, with steel portal framing supporting timber roof rafters and clay roof tiling, and concrete masonry walls at basement level. Detailed information regarding Zones 1 and 6 is provided in §4.6 and §4.7.

Crack investigations and monitoring: An in-depth investigation into the existing cracking observed within the vaulted floor and numerous walls of the structure was undertaken, in an effort to understand the experienced deformations and possible causes, and to develop a possible remediation methodology for the vaulted floor structure. Point cloud analyses of laser scanning data was undertaken to identify the amount of deformation experienced and the associated possible cracking mechanisms, and concluded that large amounts of differential settlement had been experienced by the structure, most notably the relative settlement of the large concrete gable walls with respect to the rest of the structure's foundations. Due to the observations made regarding the extensive cracking of the vaulted floor, a comprehensive crack monitoring study was commenced with the installation of several manual crack monitoring gauges in May 2021. The gauges provided evidence within 5 weeks that the structure was still experiencing movement. Considering the relatively short timeframe in which movement was identified, a more robust crack monitoring system composed of displacement transducers connected to a fully automated acquisition system was installed in the structure on the 17 December 2021. This system will be utilised to monitor the building throughout its future life stages and to continually inform the detailed design and construction stages of the project. Detailed information regarding cracking investigations and monitoring is provided in §5 and Appendix 3.

Material testing: Testing of the pumice concrete collected from the infill panels and of the unreinforced pumice concrete utilised for the walls and vaulted floor of the structure was undertaken. Samples of timber were also extracted from the perimeter wall framing at several locations for identification of the timber species. Detailed information about material testing is provided in §5.1 and Appendix 2.

Concrete material quality: The concrete was generally found to contain voids and was poorly compacted. Several core samples extracted from walls were found to contain cracks through the cross section of the core sample, which was not discernible from the wall surface, see photographic examples in §6.1.1.

Concrete compressive strength:

A total of 19 core samples were extracted, laboratory test results are as follows (see §6.1.2 and Appendix 2):

- The average **compressive strength of cylindrical concrete core** samples is 9.0 MPa (CoV 41%).
- The average **concrete density** is 1332 kg/m³ (CoV 5%).
- The average **modulus of elasticity** is 6.0 GPa (CoV 37%).

Concrete tensile strength:

A total of 17 core samples were extracted, laboratory test results are as follows (see §6.1.3 and Appendix 2):

- The average **tensile strength of cylindrical concrete core** samples is 0.8 MPa (CoV 42%).
- The average **concrete density** is 1252 kg/m³ (CoV 7.5%).

Concrete (for infill panels) compressive strength:

A total of 18 cube samples were extracted, laboratory test results are as follows (see §6.2 and Appendix 2):

- The average **compressive strength of cubic concrete samples** is 30.7 MPa (CoV 5.5%).
- The average **concrete density** is 1530 kg/m³ (CoV 3%).

Timber species identification:

Timber samples were extracted from several locations of the original timber framed portion of the perimeter walls and the mezzanine floor structure belonging to Zone 3. It was concluded that all samples were found to be Totara.

Detailed Seismic Assessment (DSA): The detailed seismic assessment of the structure was undertaken with multiple approaches to address the newer and the original historic zones. In addition, given the large dimension of the structure and the complexity of its composition, the building was divided into the 6 zones above described, which have been examined individually.

Zones 2 to 5 (original zones) (see §7 to §13):

Analysis: The seismic response of the structure was determined through a series of Nonlinear Static Analyses (pushover analyses NLSPA) as recommended by the most recent international standards for seismic assessment and rehabilitation of existing structures (ASCE SEI 41/17, Eurocode 8, NTC2018 and The Seismic Assessment of Existing Buildings (the Guidelines) (2017)). The building and most of its components are modelled adopting the Finite Element (FE) software SAP2000. Given the complex nature of the structure, its irregularities and of its loading conditions, numerous models were defined so that the nonlinear static pushover analyses were examined both from a global perspective (considering the building in its entirety i.e., a single model comprising Zones 2 to 5), and from a local perspective (studying the response of single structural elements). The building was modelled from the foundation level including also the basement structure, however in parallel, another model neglecting the basement and floor structure was defined and used for the analyses.

The typical outcome of a pushover analysis is the capacity curve (or pushover curve), which is a curve representing the building shear capacity versus the displacement of a control point. The safety level of the structure is defined by the % of new building standard (NBS) achieved.

Following NLSPA, nonlinear time history analyses (NLTHA) was performed on the zones comprising the original structure (i.e., single model comprising Zones 2 to 5). The purpose of these analyses were to substantiate the results obtained via NLSPA which was the main assessment approach adopted for the design of the retrofit intervention, as well as to provide supplementary information about the estimated dynamic behaviour of the building when subjected to seismic load.

Retrofit philosophy and strengthening concept: The philosophy adopted for the retrofit scheme (to achieve 80% NBS (IL3)) of the original zones was to utilise as much of the existing structural capacity as possible, with supplementation provided by 'surface applied' materials (typically Cross Laminated Timber (CLT) panels, plywood, Fibre Reinforced Cementitious Matrix (FRCM) coatings, vertical strong-backs) to improve the capacity of individual structural elements. Connections between vertical and horizontal structural elements are then improved so as to enhance the overall global

behaviour of the structure. This approach entails the use of materials which are compatible with those of the existing structure. As with the assessment approach, retrofit schemes were developed for each individual zone, with certain retrofit techniques implemented in multiple locations and in multiple zones (within the original zones) where possible to optimise the scheme as well as future project stages. The retrofit scheme intends to maintain the existing level of interdependency of the zones i.e., it has not been designed to increase or decrease the dependency of one zone on another. Specialised retrofit techniques have been utilised in certain areas of heritage significance (such as the east wing of Zone 5) where the use of surface applied materials was not considered appropriate. See [Appendix 4](#) for detailed concept strengthening drawings.

Zone 1 (see §14.1):

The DSA of Zone 1 has been undertaken in accordance with the latest version of the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines). The Guidelines form part of the new framework for managing earthquake-prone buildings that result from the Earthquake-prone Buildings Amendment Act 2016. A staged analysis approach was adopted for the seismic assessment of the building, starting with simpler analysis methods where appropriate and then undertaking an incrementally increasing level of sophistication of analysis to establish the expected seismic performance of the building.

Recommendations for strengthening to Zone 1 are as follows:

- Provide new epoxy anchors to the existing base-plate connections of the columns that the ReidBrace cross braces are connected to, in order to increase the strength of the connections.
- In-fill portions of the existing long & wide floor penetrations along gridlines A & 7 to improve the floor-to-wall connections and enable adequate transfer of forces to the walls.

Zone 6 (see §14.2):

The DSA of Zone 6 has been undertaken in accordance with relevant sections of the latest version of the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017 (the Guidelines) for calculating member capacities. Reference has been made to NZS 1170 for calculating loadings. The lateral load resisting system of the building has been checked based on Equivalent Static Analysis. The roof bracing checks are based on loads obtained from the Parts and Components provisions of NZS1170.5.

Recommendations for strengthening to Zone 6 are as follows:

- In-fill portions of the existing long & wide floor penetrations along gridline A8 (north side of Zone 6) to improve the floor-to-wall connections and enable adequate transfer of forces to the walls.

Concrete floor retrofit study (see §15):

Based upon the observations regarding the extensive slab cracking of the unreinforced pumice concrete vaulted floor found in Zones 2 to 5, analyses were undertaken to examine the behaviour of the floor under static gravity loading. The analysis was carried out on an isolated module of the floor which was considered representative for the global static behaviour of the structure (i.e., vaulted slab portion supported by four columns). The floor module was modelled adopting the Finite Element (FE) software SAP2000 and the FE software TNO DIANA. Following the analyses of the floor in the as-built condition, two retrofit interventions were proposed. The two interventions differentiate between the part of the floor structure which can be accessed for structural reinforcement work from the top of the floor (Zone 2-3-4 and the West part of Zone 5) and the part of the floor structure where the top surface shall remain untouched for architectural heritage purposes (East part of Zone 5).

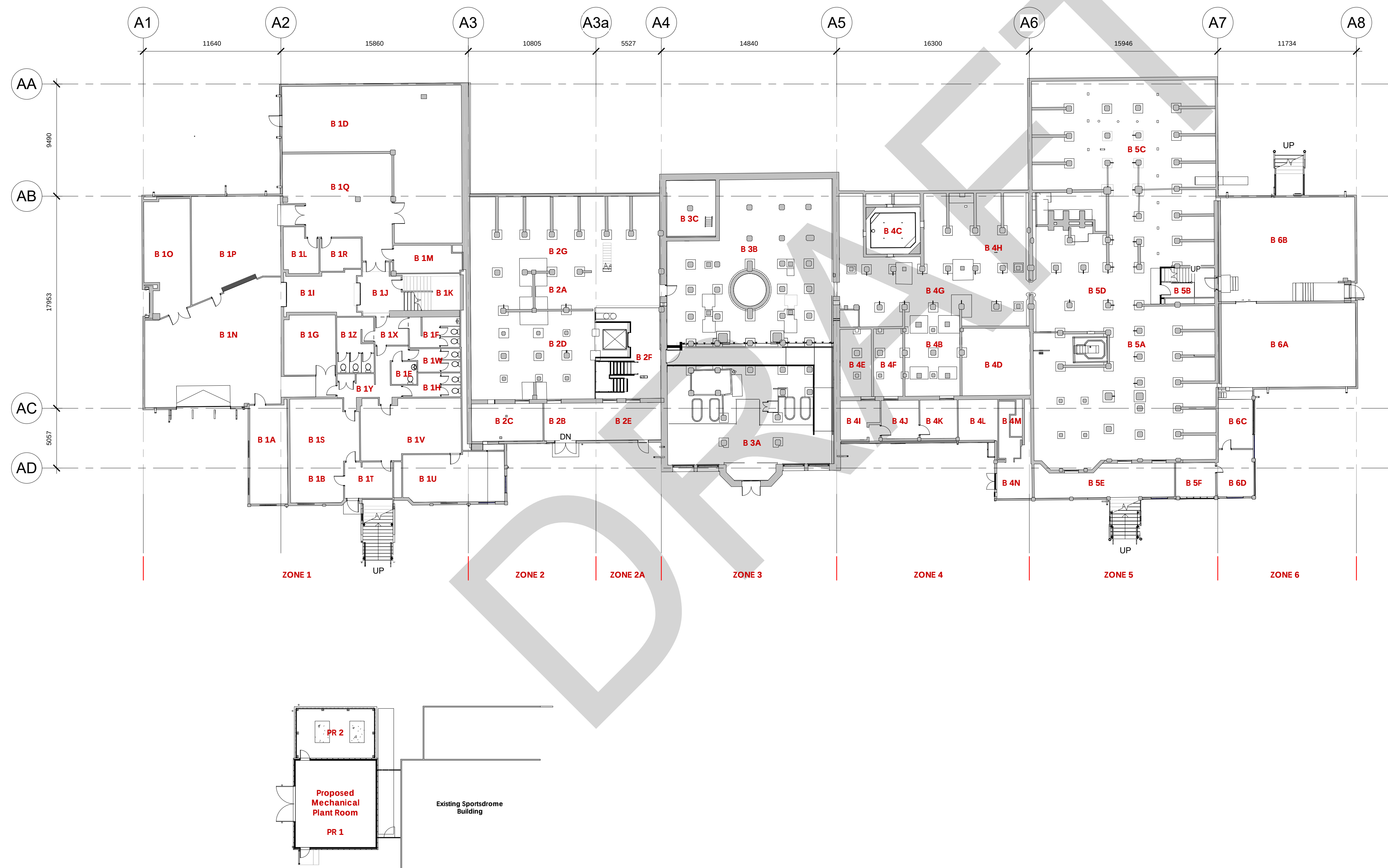
The retrofit for Zones 2 to 4 and the west part of Zone 5 comprises a grid of reinforced concrete ribs carved inside the existing slab. An additional FRCM topping connected to both the existing floor and the new ribs is also proposed. The retrofit for the east wing of Zone 5 consists of inserting Rectangular Hollow Section (RHS) steel profiles from basement level onto the soffit of the slab, along the supporting column grid lines. The RHS steel elements are to be bent to adapt to the vaulted floor soffit and will be connected to the existing concrete floor using mechanical fasteners and to the columns via purposely designed steel “collars”.

REFERENCES

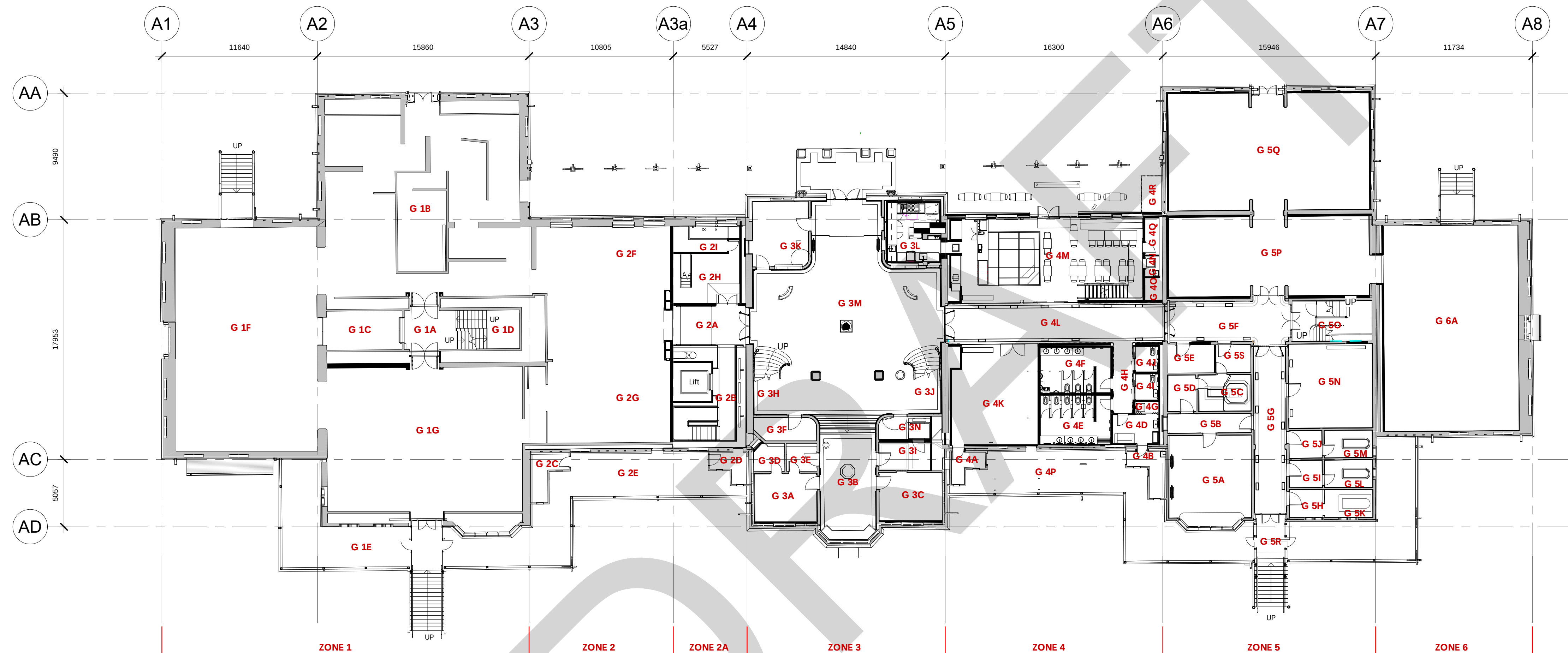
- ACI 318 (2008). *Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute, United States.
- Acikgoz, S., Soga, K., Woodhams, J. (2017). "Evaluation of the response of a vaulted masonry structure to differential settlements using point cloud data and limit analyses." *Construction and Building Materials* 150: 916-931. doi:10.1016/j.conbuildmat.2017.05.075
- American Society of Civil Engineers (ASCE), 2000. "Pre-standard and Commentary for the Seismic Rehabilitation of Buildings." Prepared for the SAC Joint Venture, published by the Federal Emergency Management Agency, FEMA-356, Washington, D.C.
- AS/NZS 1170.0 (2011). *Structural Design Actions. Part 0: General Principles*. Australian/New Zealand Standard.
- AS/NZS 1170.1 (2002). *Structural Design Actions. Part 1: Permanent, Imposed and Other Actions*. Australian/New Zealand Standard.
- ASTM C39 (2018). *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. USA: American Society for Testing and Materials. doi:10.1520/C0039.
- ASTM C469 (2014). *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*. ASTM International. USA: American Society for Testing and Materials. doi:10.1520/C0469.
- Bottari, A., De Domenico, D., Giannino, F., Marino, A., & Teramo, A. (2003). "On a GPR Survey to Characterize the Structural Configuration of a Masonry Building." In *3rd International Conference on NDT*.
- BRANZ (2013). Study report SR 305 Bracing ratings for non-proprietary bracing walls, BRANZ, Wellington, NZ.
- CEB-FIP. 1990. Model code for concrete structures. Comité Euro-International du Béton – Fédération Internationale de la Précontrainte, Thomas Telford Services Ltd, Lausanne, Switzerland.
- Clifton, G. C. (2018). "Summary of Outcomes from ReidBrace Testing at the University of Auckland Structures Test Hall." [technical report dated 28 August 2018].
https://reids.co.nz/upload/modules/document_loader/Summarised_ReidBrace_testing_including_bearing_in_Ply_r_economondation.pdf. Retrieved 16 Feb 2021.
- CNR-DT 215 (2018). "Guide for the design and construction of externally bonded fibre reinforced inorganic matrix systems for strengthening existing structures." National Research Council, Rome.
- CSI (2017). SAP 2000 Analysis Reference Manual. Computers and Structures Inc., Berkeley
- CSI (2015). Technical Note Modified Darwin-Pecknold 2-D Reinforced Concrete Material Model. Computers and Structures, Inc., Berkeley.
- Daniels, D. J. (1996). "Surface-Penetrating Radar." *Electronics & Communication Engineering Journal* 8 (4): 165–82.
- DPA Architects (2019). "Rotorua Bath House - Conservation Plan."
- Frontini, F., Siem, J., Renmæmo, R. (2020). "Load-Carrying Capacity and Stiffness of Softwood Wooden Dowel Connections." *International Journal of Architectural Heritage*, 14:3, 376-397, DOI: 10.1080/15583058.2018.1547798
- Giongo I., Rizzi E., Ingham J., Dizhur D., 2018, "Numerical modelling strategies for the in-plane behavior of straight sheathed timber diaphragms", *Journal of Structural Engineering*, vol. 144, No.10.
- Giunta, G., & Calloni, G. (2000). "Ground Penetrating Radar Applications on the Facade of St. Peter's Basilica in Vatican." In *15th World Conference on Nondestructive Testing*. (Roma, 15-21 October), Italy. <http://www.ndt.net/article/wcndt00/papers/idn706/idn706.htm>.
- Manie J., Kikstra W. P. (2014). "DIANA - Finite Element Analysis." *User's Manual release 9.5*, Delftechpark 19a, 2628 XJ Delft, The Netherlands: Published by TNO DIANA BV, 2014.
- MBIE-NZSEE (2017a). *The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments. Section C2 – Assessment Procedures and Analysis Techniques*. Wellington, New Zealand: Co-ordinated by a Project Technical Group on behalf of NZSEE, SESOC, NZGS, MBIE and EQC.
- MBIE-NZSEE (2017b). *The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering*

- Assessments. Section C8 – Unreinforced Masonry Buildings.* Wellington, New Zealand: Co-ordinated by a Project Technical Group on behalf of NZSEE, SESOC, NZGS, MBIE and EQC.
- MBIE-NZSEE (2017c). *The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments. Section C9 – Timber Buildings.* Wellington, New Zealand: Co-ordinated by a Project Technical Group on behalf of NZSEE, SESOC, NZGS, MBIE and EQC.
- Moroder, D., Smith, T., Pampanin, S., Buchanan, A.H., 2015, "An equivalent truss method for analysis of timber diaphragms", *Proceedings of the Tenth Pacific Conference on Earthquake Engineering: Building an Earthquake-Resilient Pacific*, November 2015, Sydney, Australia.
- NZS 1170.5 (2004). *Structural Design Actions. Part 5: Earthquake Actions - New Zealand.* Wellington, New Zealand Standard.
- Oyarzo-Vera, C. O., McVerry, G. H., & Ingham, J. M. (2012). Seismic Zonation and Default Suite of Ground-Motion Records for Time-History Analysis in the North Island of New Zealand. *Earthquake Spectra*, 28(2), 667-688. doi:10.1193/1.4000016
- Ranalli, D., Scozzafava, M., & Tallini, M (2004). "Ground Penetrating Radar Investigations for the Restoration of Historic Buildings: The Case Study of the Collemaggio Basilica (L'Aquila, Italy)." *Journal of Cultural Heritage* 5: 91–99. doi:10.1016/j.culher.2003.05.001.
- Reyes, J.C., Kalkan, E., Sierra, A. (2017). "Fast nonlinear response history analysis" *Proceedings of the 16th World Conference on Earthquake WCEE*, Santiago, Chile.
- Rizzi E., Giongo I., Ingham J., Dizhur D., 2020, "Testing and Modeling In-Plane Behavior of Retrofitted Timber Diaphragms", *Journal of Structural Engineering*, v. 2020, vol. 146, n. Issue 2 (2020). - DOI: 10.1061/1943-541.0002473.
- Selby R. G., Vecchio F. J. (1993). "Three-dimensional Constitutive Relations for Reinforced Concrete." [technical report] 93-02, Univ. Toronto, dept. Civil Eng., Toronto, Canada.
- Standards Australia, 2010a, AS/NZS 3679.1 Structural Steel, Hot rolled bars and sections, Sydney, NSW, Australia.
- Technical Note Modified Darwin-Pecknold 2-D Reinforced Concrete Material Model Computers And Structures, Inc., July 2015
- Vecchio, F.J. and Collins, M.P., "The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear", *ACI Journal*, Paper No. 83-22, March-April 1986.

DRAFT



Rev. Description		Date
dpa architects 83 Victoria Road, PO BOX 32-318 Devonport , Auckland 0744 www.dpaarchitects.co.nz ph: +649 445 8544		
Job Title		
ROTORUA LAKES DISTRICT COUNCIL		
Rotorua Museum Development		
Government Gardens, Oruawhata Dr, Rotorua 3046		
Basement Floor Room Identification (Proposed)		
Drawn : Author		Checked/Checker
Job # : 1273		
Printed		7/12/2021 08:50:52
Project Status		WIP
Scale : 1 : 200 @ A1 Reduced 50% @ A3		Rev. A100
All dimensions to be verified on site before commencing any work or making any shop drawings. Do not scale measure dimensions from drawings.		



Rev.	Description	Date

dpa
architects

83 Victoria Road, PO BOX 32-318
Devonport, Auckland 0744
www.dpaarchitects.co.nz
ph: +649 445 8544

Job Title

ROTORUA LAKES DISTRICT COUNCIL

**Rotorua Museum
Development**

Government Gardens, Oruawhata Dr,
Rotorua 3046

**Ground Floor Room
Identification (Proposed)**

Drawn : Author	Checked Checker
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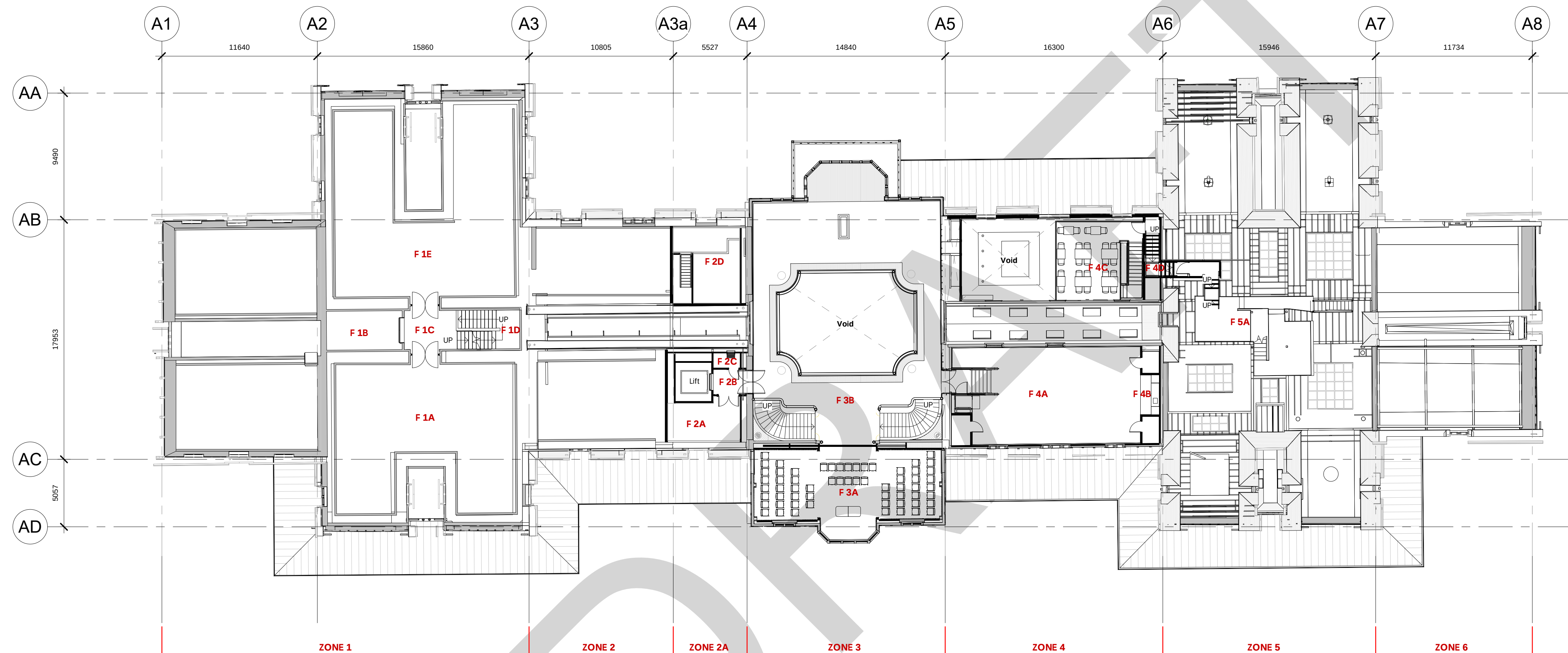
Job # : 1273

Printed : 7/12/2021 08:51:15

Project Status	WIP
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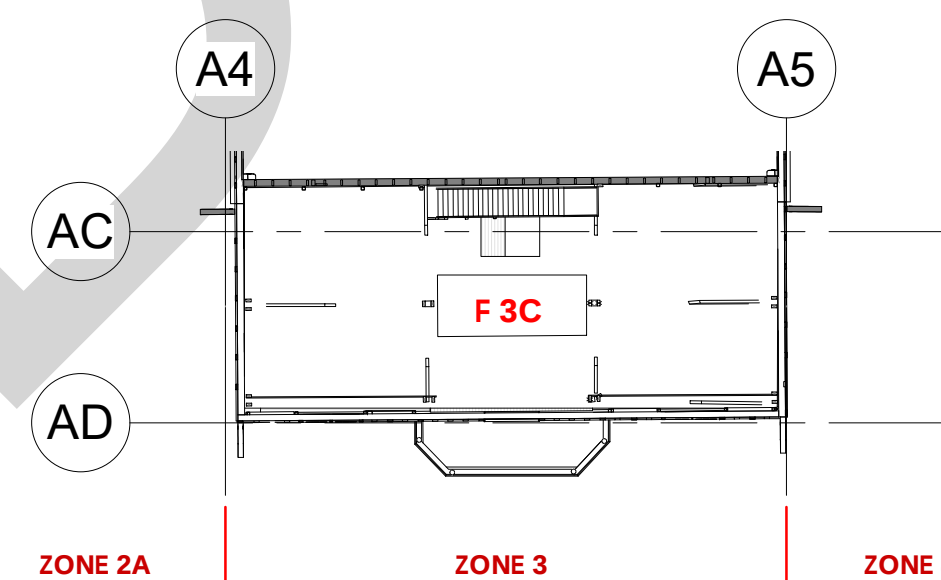
Scale : 1 : 200 @ A1 Reduced 50% @ A3	Rev.	A101
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All dimensions to be verified on site before commencing any work or making any shop drawings.
Do not scale measure dimensions from drawings.



1 Proposed First Floor Plan

1 : 200



2 Zone 3 Attic Floor Plan

1 : 200

Rev.	Description	Date



83 Victoria Road, PO BOX 32-318
Devonport , Auckland 0744
www.dpaarchitects.co.nz
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Job Title

ROTORUA LAKES DISTRICT COUNCIL

**Rotorua Museum
Development**

Government Gardens, Oruawhata Dr,
Rotorua 3046

**First Floor Room
Identification (Proposed)**

Drawn : Author	Checked Checker
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Job # : 1273

Printed : 7/12/2021 08:51:41

Project Status

WIP

Scale : 1 : 200 @ A1 Reduced 50% @ A3	Rev.	A102
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All dimensions to be verified on site before commencing any work or making any shop drawings.
Do not scale measure dimensions from drawings.

DRAFT

Thursday, 20 May 2021

Dmytro Dizhur
Lev10, Zurich House
21 Queen Street
AUCKLAND, 1010

Re: Wood identification – Rotorua Museum

Dear Dmytro

Thankyou for sending your wood samples for identification. The samples are all either Rimu or Totara. These two timbers are very similar apart from differences in durability. Given the age of the building and the apparent good condition of the timber, including absence of any obvious borer pinholes, all the timber samples are more likely to be Totara than Rimu. Sample TS2 is more typical of Totara in appearance than the others.

Sample TS1 - Intermediate North Wing (Zone 4) east perimeter wall, smaller horizontal stud - Totara

Sample TS2 - Intermediate North Wing (Zone 4) east perimeter wall, main vertical member - Totara

Sample TS3 - Atrium (Zone 3) west perimeter wall mezzanine level, main vertical member - Totara

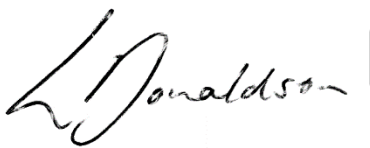
Sample TS4 - Atrium (Zone 3) - west perimeter wall mezzanine level, diagonal brace - Totara

Sample TS5 - Atrium (Zone 3) - west perimeter wall mezzanine level, primary horizontal member - Totara

Sample TS6 - Atrium (Zone 3) - mezzanine floor joist (southern end) - Totara

Sample TS7 - Intermediate South Wing (Zone 2), east perimeter wall timber baseplate - Totara

Yours Sincerely



Dr Lloyd Donaldson
Senior Scientist

Infill Panel Compressive Strength

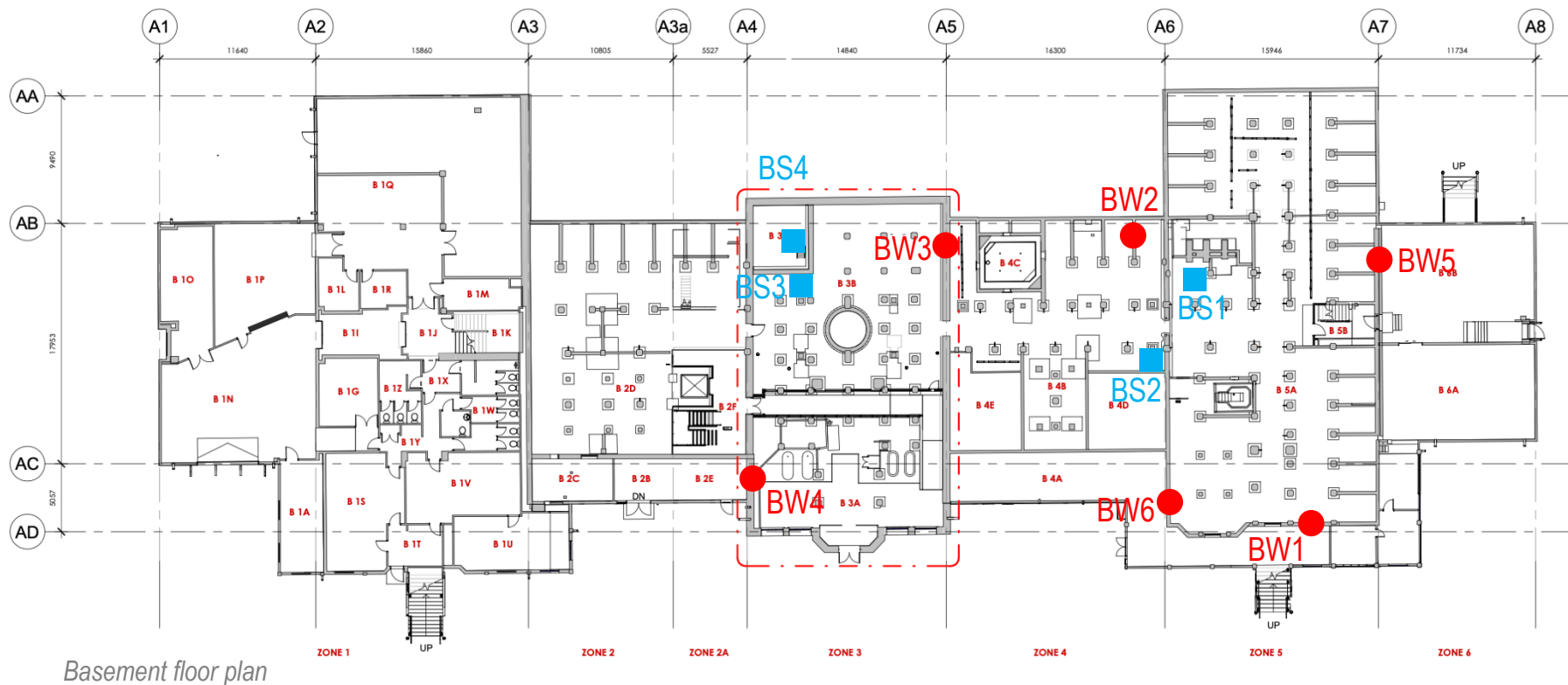
Project:	Rotorua Museum
Extracted by:	DIZHUR Consulting
Tested by:	DIZHUR Consulting
Date extracted:	7/05/21
Date tested:	28/05/21
Date released:	12/07/21

Location T1L1

Overall Average:	30.67	MPa
Standard Dev / CoV:	1.70	5.5%
Density	1527.58	kg/m3

Sample extraction location ID	Sample ID	Length avg (mm)	Width avg (mm)	Height avg (mm)	Weight (g)	Density (kg/m3)	Force (kN)	Stress (MPa)	α_{ht}	α_{tl}	$f'_j = \alpha_{tl}\alpha_{ht}f'_{jl}$ (MPa)	Group average (MPa)	STD	CoV
Z4-GF-EW1	A	40.57	38.68	43.80	103.00	1498.31	50.00	31.86	1.08	0.98	33.85	33.52	1.71	0.05
	B	40.13	41.84	45.31	110.00	1445.91	49.60	29.54	1.06	1.02	31.93			
	C	39.92	38.85	42.20	102.00	1558.49	52.40	33.78	1.06	0.99	35.52			
	D	38.71	40.00	43.46	103.00	1530.60	48.00	31.00	1.06	1.01	33.43			
	E	38.37	40.89	42.22	104.00	1570.03	50.00	31.87	1.03	1.03	33.65			
	F	40.41	42.14	41.42	110.00	1559.42	53.20	31.24	0.98	1.02	31.30			
	G	41.53	40.56	42.42	111.00	1553.18	55.40	32.89	1.04	0.99	33.79			
	H	40.17	41.81	43.11	111.00	1533.19	54.60	32.51	1.03	1.02	33.94			
	I	39.14	40.13	41.00	100.00	1552.71	55.80	35.53	1.02	1.01	36.58			
Z4-GF-EW2	J	41.02	39.66	40.52	105.00	1592.71	50.60	31.10	1.02	0.99	31.24	27.10	1.69	0.06
	A	42.86	44.90	44.18	131.00	1540.69	56.20	29.20	0.99	1.02	29.35			
	B	41.07	44.50	44.64	123.00	1507.76	50.40	27.58	1.00	1.04	28.63			
	C	43.94	43.93	43.00	132.00	1590.56	56.40	29.22	0.98	1.00	28.64			
	D	42.51	41.63	45.69	121.00	1496.24	44.40	25.09	1.07	0.99	26.60			
	E	39.79	41.80	45.35	115.00	1524.78	37.20	22.37	1.06	1.02	24.28			
	F	43.06	41.57	44.81	118.00	1470.80	46.20	25.81	1.06	0.99	26.93			
	G	41.26	43.75	45.00	120.00	1477.04	45.60	25.26	1.02	1.03	26.53			
	H	42.86	40.44	45.56	118.00	1493.94	42.40	24.46	1.08	0.98	25.83			

With reference to NZSEE (2017) and Valek & Veige (2005)



Note:

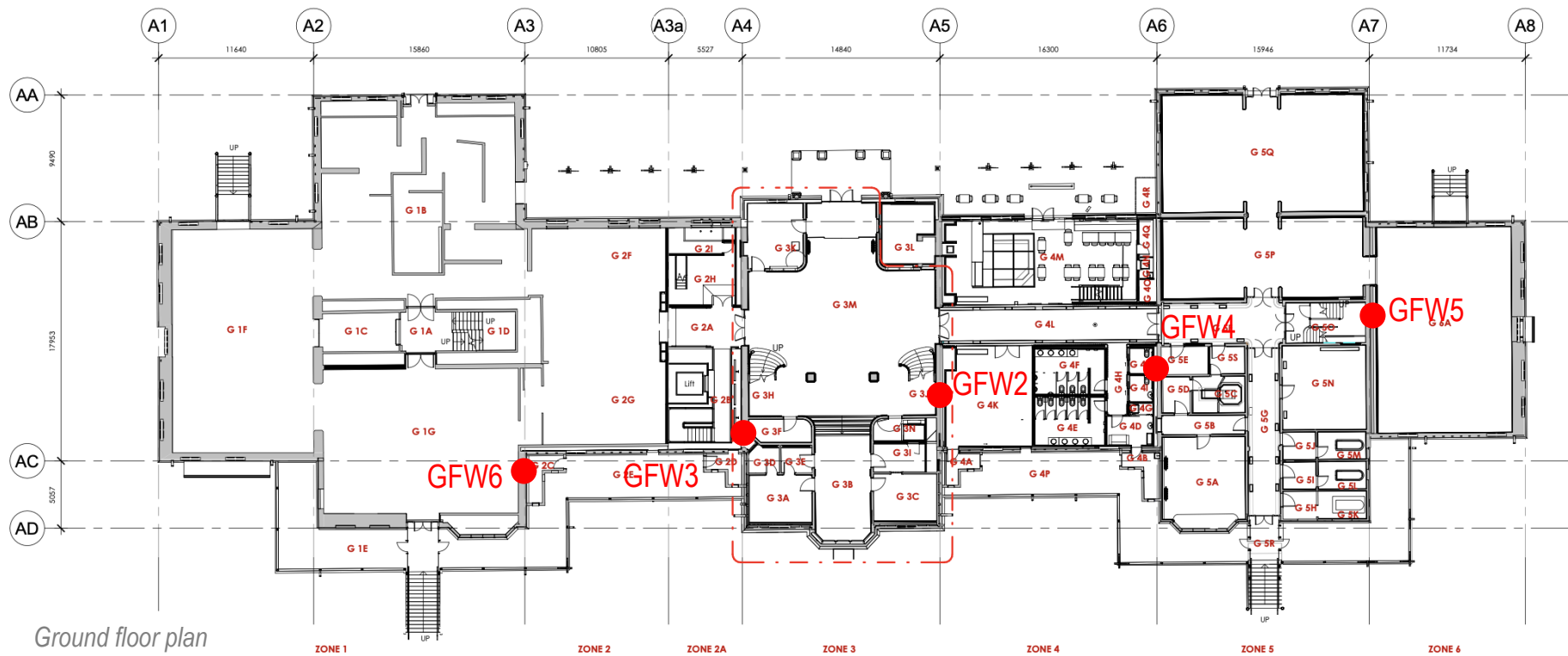
- All maps are oriented → N
- Sample naming: A-B-C-D, A = Building level, B = Structural element (wall or slab), C = sample number D = sample number if more than one core in same location

Core extraction from floor slab



Core extraction from wall





Note:

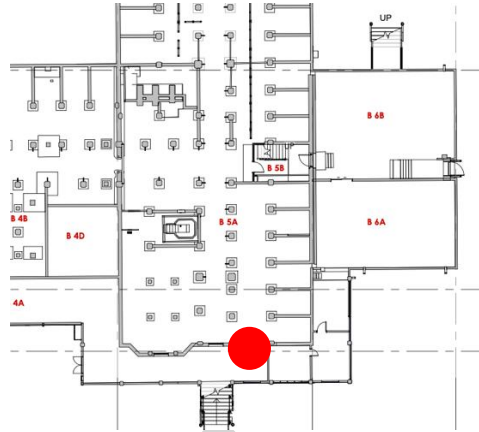
- All maps are oriented → N
- Sample naming: A-B-C-D, A = Building level, B = Structural element (wall or slab), C = sample number D = sample number if more than one core in same location

Core extraction from floor slab

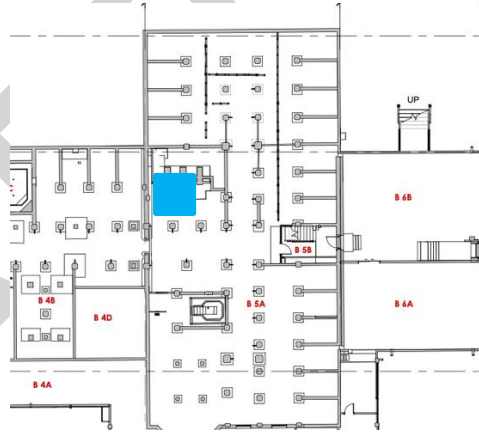


Core extraction from wall

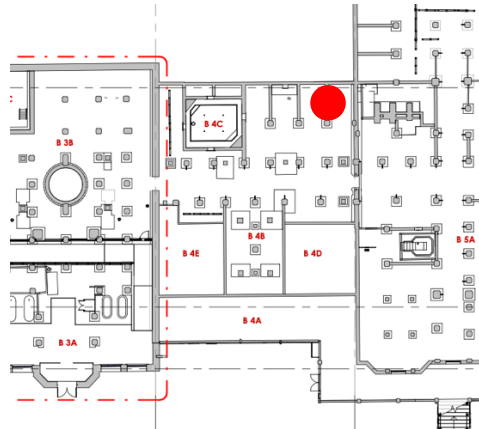
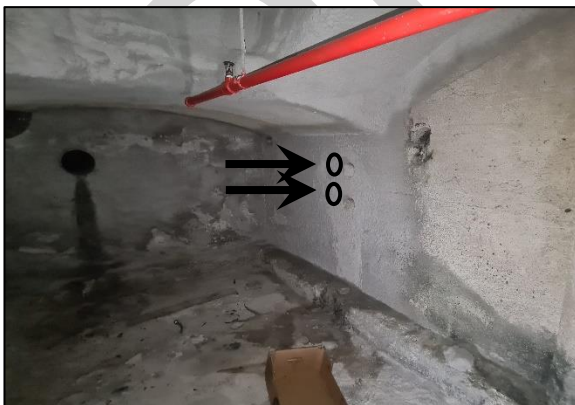




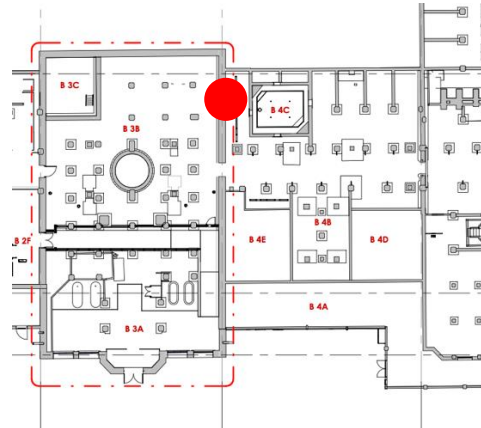
B-W-1-1, B-W-1-2 – Zone 5 east basement wall (2 cores extracted)



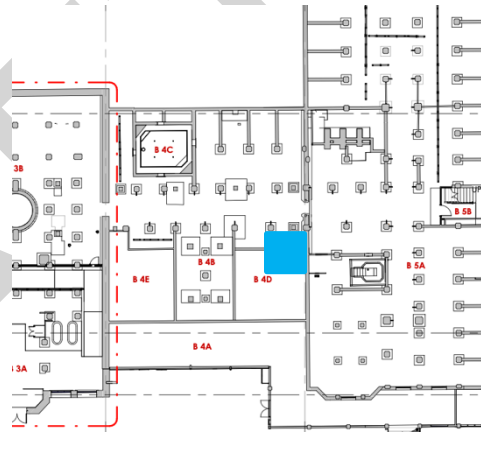
B-S-1 – Floor slab, Zone 5 south side (within G5P zone) (done from below, closer to column to avoid penetrating timber floor overlay)



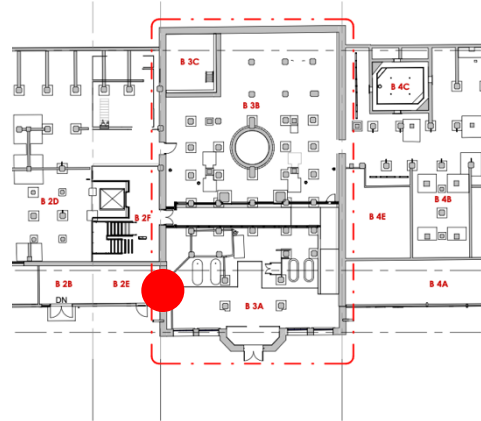
B-W-2-1, B-W-2-2 – basement wall parallel to B4C north wall, Zone 4 west side (2 cores extracted)



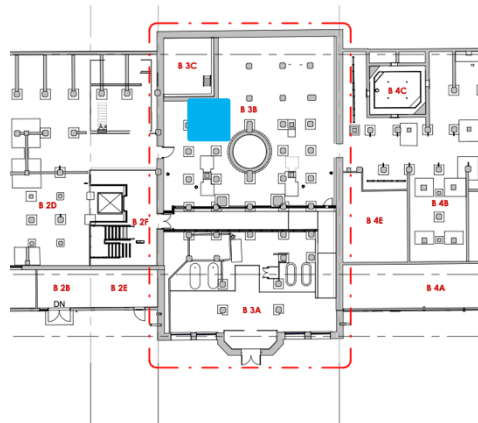
B-W-3-1, B-W-3-2 – main pumice concrete wall between Zones 3 and 4 (photo looking eastward at wall from Zone 4)
(2 cores extracted)



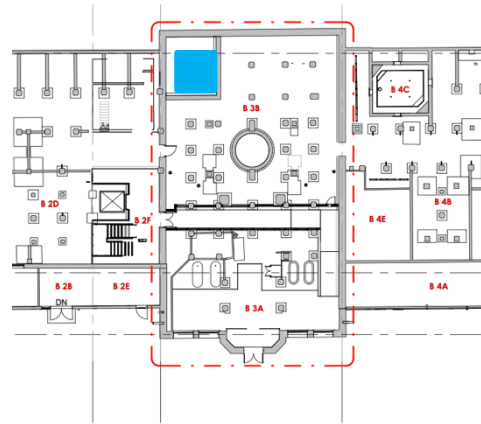
B-S-2 – floor slab near north wall of Zone 4, done close to basement column to avoid penetrating through, done from above and avoiding G4L (corridor)



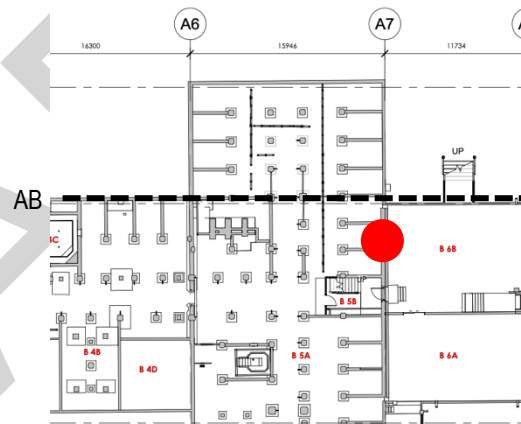
B-W-4-1, B-W-4-2 – basement wall between Zones 2 and 3 (2 cores extracted). Photos show general location for coring - room B2E, Zone 2. Coring done from room B2E



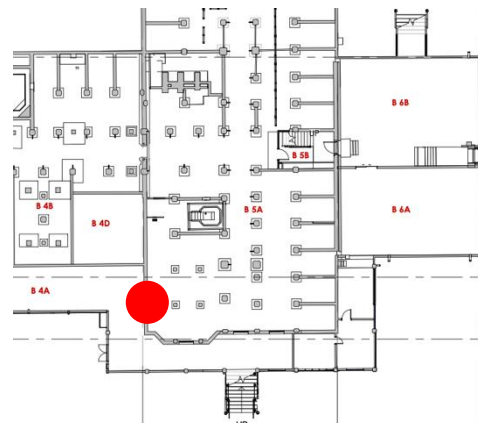
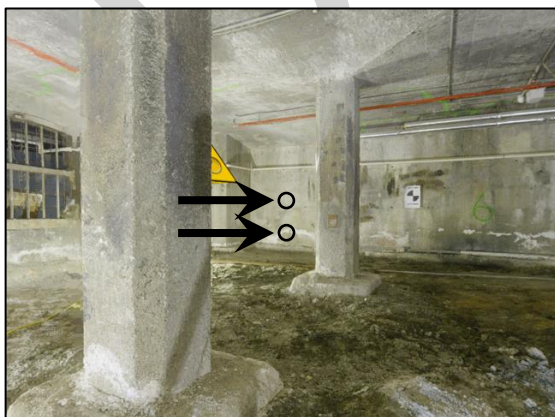
B-S-3 – floor slab at western end near B3C, Zone 3 – western side of column (due to tiles above, coring done from beneath slab)



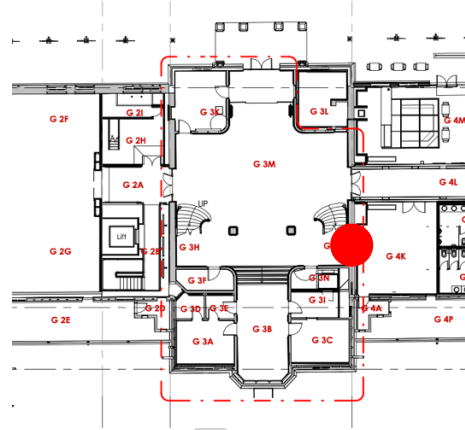
B-S-4 - floor slab in Zone 3 near B3C (done from above)



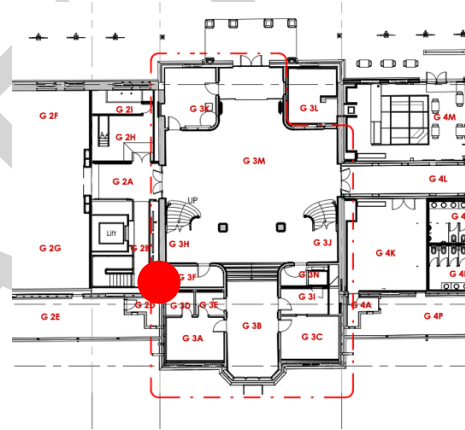
B-W-5-1, B-W-5-2 – basement, main north wall of Zone 5 (not visible in photo, shows general location – done from Zone 5 & east of gridline AB) (2 cores extracted)



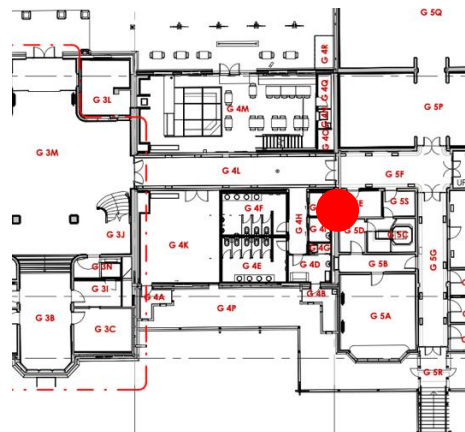
B-W-6-1, B-W-6-2 – basement, south wall at southeast corner of Zone 5 (2 cores extracted)



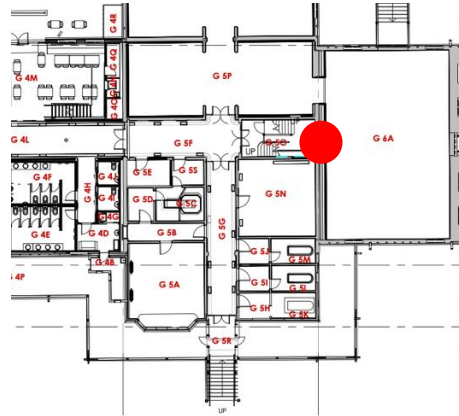
GF-W-2 – gable wall between Zones 3 and 4, ground floor, beneath staircase (coring done from Zone 4 G4K, without penetrating fully though)



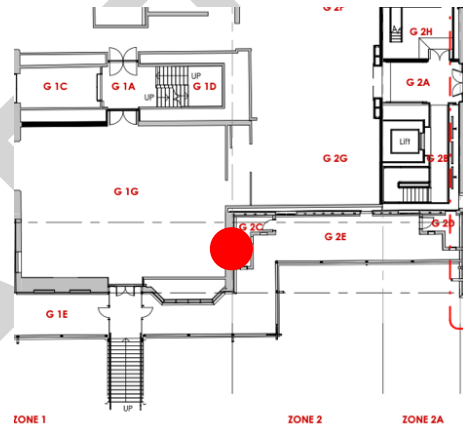
GF-W-3 – main pumice concrete wall between Zones 2 and 3 (photo is looking south in room G3F) – coring done from G3F



GF-W-4 – gable wall between Zones 4 and 5, ground floor (coring done from Zone 4 side)



GF-W-5 – gable wall between Zones 5 and 6 (coring done from Zone 6, G6A, without penetrating through)



GW-F-6 – historic wall remnant at interface between Zones 1 and 2. 3 samples extracted

Concrete Cylinder Compressive Strength

Project:	Rotorua Museum
Extracted by:	DIZHUR Consulting Ltd
Tested by:	DIZHUR Consulting Ltd
Date extracted:	17-18/11/2021
Date tested:	13-14/12/2021
Date releasead:	13-14/12/2021

Overall average			CoV
Average compressive stress (cylindric):	9.0	MPa	41.1%
Average density:	1332	kg/m3	5.1%
Average modulus of elasticity:	6	GPa	36.6%

N.B.: Extraction locations are wrt. Room IDs established by DPA Architects, file "RMD_2021-12-07_Room IDs".

	Concrete extraction location	Sample ID	Diameter avg (mm)	Height avg (mm)	Density (kg/m3)	Modulus of elasticity (GPa) **	Force (kN)	Aspect ratio correction factor	Stress (MPa)	Average (MPa)	STD	CoV
Basement Walls	Z5 - wall AD north end	BW-1-1	93.76	163.23	1305.4	6.97	81.782	0.98	11.6	10.7	4.06	0.38
	Z5 - wall AD north end	BW-1-2	93.86	164.52	1333.6	8.5	95.203	0.98	13.5			
	Z4 bath partition wall	BW-2-1	93.76	175.2	1286.4	6.19	50.476	0.99	7.2			
	Z3 - A5 wall west end	BW-3-1	93.92	180.46	1446.3	8.57	98.939	0.99	14.1			
	Z3 - A5 wall west end	BW-3-2	93.9	199.63	1222.0	3.54	33.524	1.00	4.8			
	Z3 - A4 wall east end	BW-4-1	93.97	196.15	1312.9	5.75	73.029	1.00	10.5			
	Z3 - A4 wall east end	BW-4-2	93.84	190.34	1327.3	4.39	70.959	1.00	10.3			
	Z5 - A7 wall	BW-5-1	93.95	170.54	1414.7	9.02	119.638	0.99	17.1			
	Z5 - A7 wall	BW-5-2	93.92	167.35	1423.7	8.95	107.706	0.98	15.2			
	Z5 - A6 wall east end	BW-6-1	93.58	165.79	1316.8	3.76	41.329	0.98	5.9			
	Z5 - A6 wall east end	BW-6-2	93.45	167.19	1361.0	4.96	49.626	0.98	7.1			
Ground Floor Walls	Z3 - A5 wall east end	GFW-2-1	93.91	169.22	1278.5	3.71	50.601	0.98	7.2	6.0	1.33	0.22
	Z3 - A4 wall east end	GFW-3-1	93.79	143.73	1311.5	3.75	32.717	0.96	4.5			
	Z5 - A6 wall west end	GFW-4	93.22	131.82	1213.0	N/A	32.935	0.95	4.6			
	Z5 - A7 wall	GFW-5	93.56	136.96	1282.0	N/A	51.413	0.96	7.2			
	Z2 - A3 wall east end	GFW-6-2	93.84	126.56	1275.7	N/A	48.002	0.94	6.5			
Floor Slab	Floor in G4J	BS-2-2	93.86	102.3	1398.9	N/A	63.802	0.89	8.2	8.2	0.03	0.00
	Floor in G3M	BS-3	93.81	172.25	1379.5	4.15	57.063	0.99	8.2			
	Floor in G3K	BS-4	93.72	103.1	1418.9	N/A	63.145	0.89	8.1			

With reference to ASTM C39 (2018), ASTM C42 (2020) and ASTM C469 (2014) documents

** n/a when height to diameter ratio is lower than 1.5

Concrete Cylinder Tensile Strength

Project:	Rotorua Museum
Extracted by:	DIZHUR Consulting Ltd
Tested by:	DIZHUR Consulting Ltd
Date extracted:	17-18/11/2021
Date tested:	13-14/12/2021
Date releasead:	13-14/12/2021

Overall average			CoV
Average tensile strength (cylindric):	0.8	MPa	41.9%
Average density:	1252	kg/m3	7.5%

N.B.: Extraction locations are wrt. Room IDs established by DPA Architects, file "RMD_2021-12-07_Room IDs".

	Concrete extraction location	Sample ID	Diameter avg (mm)	Height avg (mm)	Density (kg/m3)	Force (kN)	Tensile Strength (MPa)	Average (MPa)	STD	CoV
Basement Walls	Z5 - wall AD north end	BW-1-1	93.74	88.29	1235.6	13.19	1.0	0.9	0.33	0.36
	Z5 - wall AD north end	BW-1-2	93.81	93.71	1310.6	13.25	1.0			
	Z4 bath partition wall	BW-2-1	93.91	187.68	1409.1	36.68	1.3			
	Z3 - A5 wall west end	BW-3-1	93.94	189.72	1354.7	22.54	0.8			
	Z3 - A5 wall west end	BW-3-2	93.85	180.3	1189.0	9.57	0.4			
	Z3 - A4 wall east end	BW-4-1	94.04	197.6	1272.9	20.06	0.7			
	Z3 - A4 wall east end	BW-4-2a	93.87	157.64	1269.7	21	0.9			
	Z3 - A4 wall east end	BW-4-2b	93.92	112.61	1231.9	16.44	1.0			
	Z5 - A7 wall	BW-5-1	93.91	131	1363.5	26.73	1.4			
	Z5 - A7 wall	BW-5-2	93.76	131.1	1335.0	25.09	1.3			
	Z5 - A6 wall east end	BW-6-1	92.79	96.3	1190.6	8.25	0.6			
	Z5 - A6 wall east end	BW-6-2	92.79	100.1	1178.8	7.56	0.5			
Ground Floor Walls	Z3 - A5 wall east end	GFW-2-2	93.8	125.52	1050.2	5.06	0.3	0.5	0.26	0.48
	Z3 - A4 wall east end	GFW-3-2	93.65	160.07	1145.0	8.29	0.4			
	Z2 - A3 wall east end	GFW-6-1	93.78	135.1	1189.5	15.14	0.8			
	Z2 - A3 wall east end	GFW-6-3	93.83	126.8	1213.2	14.1	0.8			
Floor Slab	Floor in G4J	BS-2-1	93.8	107.9	1352.6	19.7	1.2	1.2	-	-

With reference to ASTM C496 (2017) documents

DIZHUR



CRACK WIDTH MONITORING REPORT

ROTORUA MUSEUM

GOVERNMENT GARDENS, ORUAWHATA DRIVE, ROTORUA

June 2021

Document revision summary

Document	File name	Prepared by	Revised by	Issued to
May 2021 version	21_0517_RMD_Crack Report_V1.pdf	DS	FG, DD	Kareen McLaren
June 2021 version	21_0614_RMD_Crack Report_V2.pdf	DS	DD	Kareen McLaren
August 2021 version	21_0823_RMD_Crack Report_V3.pdf	DS	DD	Kareen McLaren
February 2022 version	22_0208_RMD_Crack Report_V4.pdf	FG	DS, DD	Kareen McLaren

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Auckland, August 2021

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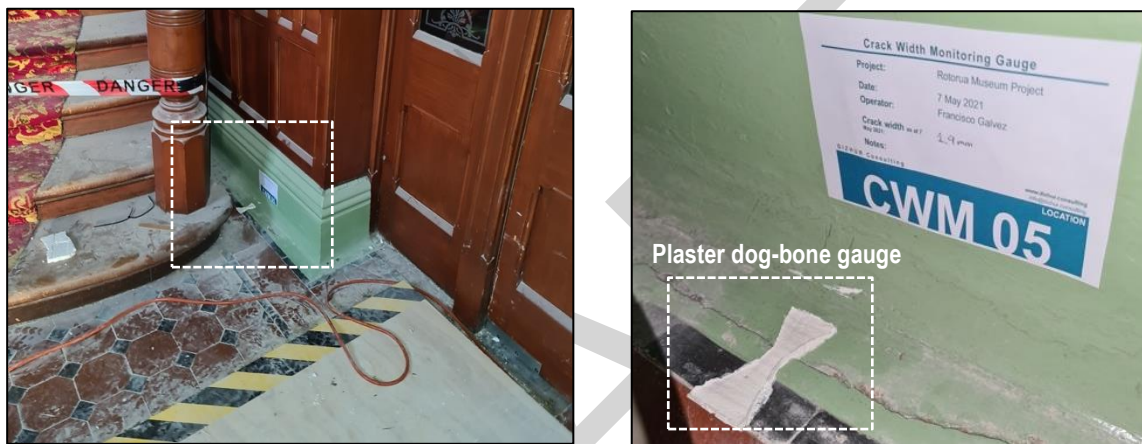
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1 CRACK WIDTH MONITORING

As part of on-site investigation works undertaken on 5-7 May 2021, DIZHUR Consulting placed several crack monitoring gauges throughout the superstructure of the building on 7 May 2021 to determine over a period of time whether the cracks are active or dormant.

The gauges were made using tape placed over the width of the crack, on top of which a dog-bone shaped 5-10mm thick layer of plaster was placed with the narrow neck localised over the crack to facilitate that the gauge brakes at the desire spot. The crack width as at the date of installation of the gauges was recorded.

Examples of crack width gauges installed on various building elements at locations throughout the building are shown in **Figure 1-1**. See **Appendices 1-3** for all crack width monitoring gauges and their locations.



(a) Example of floor separation monitoring in Zone 4 adjacent to south gable wall (CWM 5)

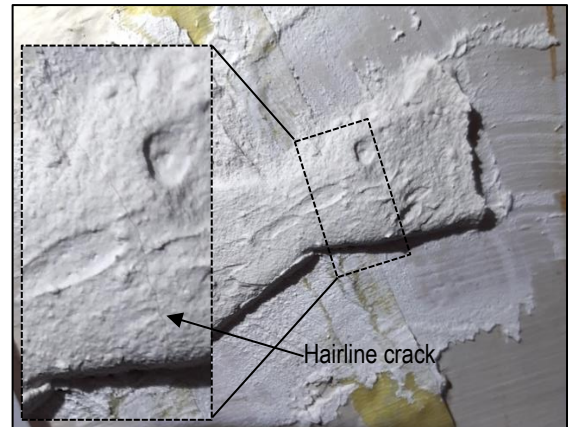
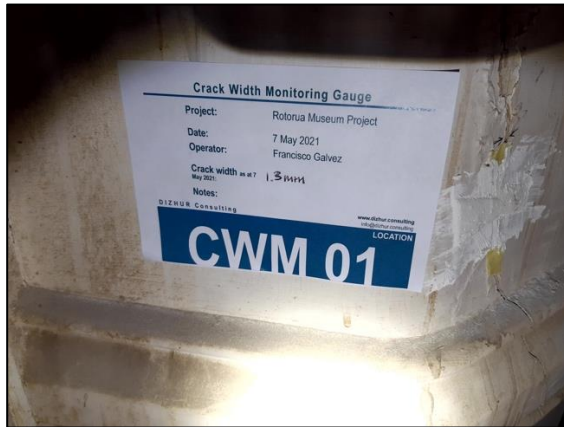


(b) Typical example of wall crack monitoring (CWM 6)

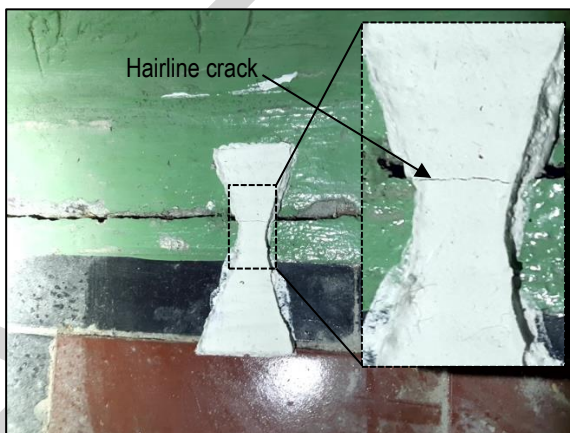
Figure 1-1. Examples of installed crack width gauges at various building elements at locations throughout the building

1.1 June 2021 inspection

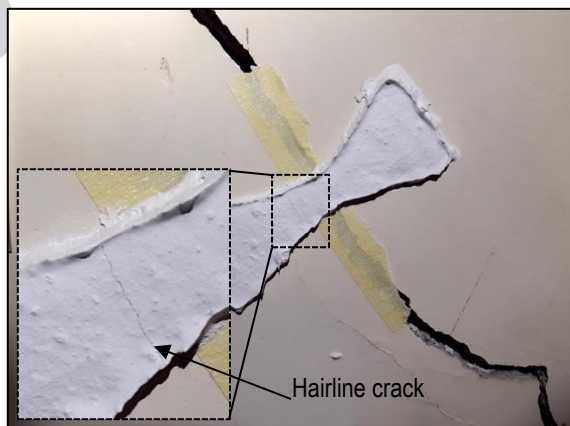
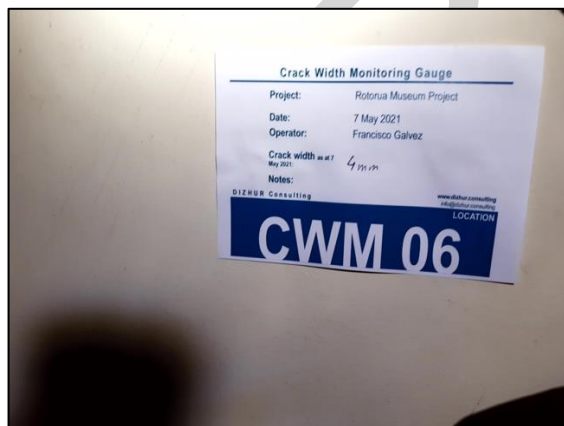
As part of a site visit undertaken on the 14 June 2021, all gauges were investigated for any sign of movement of the cracks. The following gauges exhibited hairline cracks (immeasurable width, <0.1mm) in the plaster: CWM 1, 6, 13, and 15, see **Figure 1-2a, c-e**. Gauge CWM 05 exhibited a crack of approx. 0.1mm, see **Figure 1-2b**. All gauges which exhibited crack movement were located in Zone 3. No cracking of the plaster could be identified in the other gauges. Refer to **Appendix 4** for all photos of gauges taken on 14 June 2021.



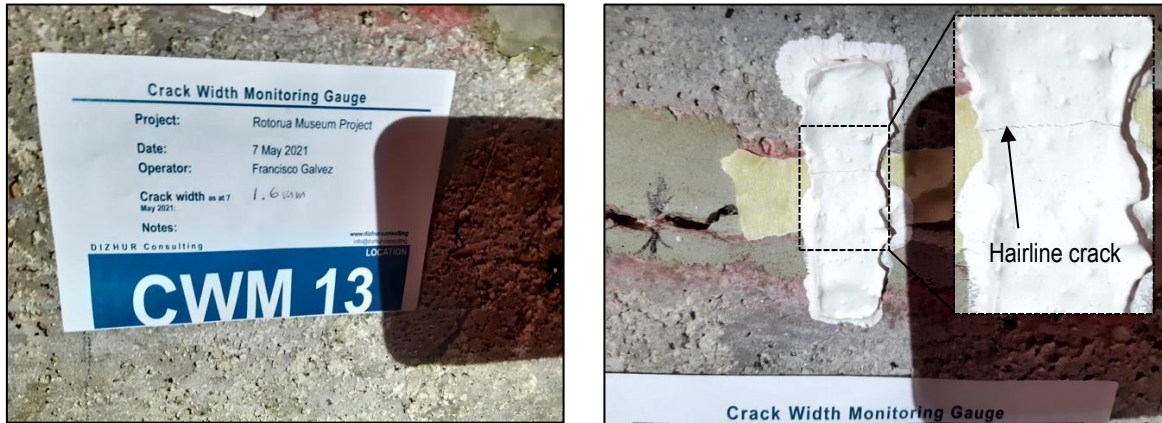
(a) CWM 01 – hairline crack ($<0.1\text{mm}$) identified



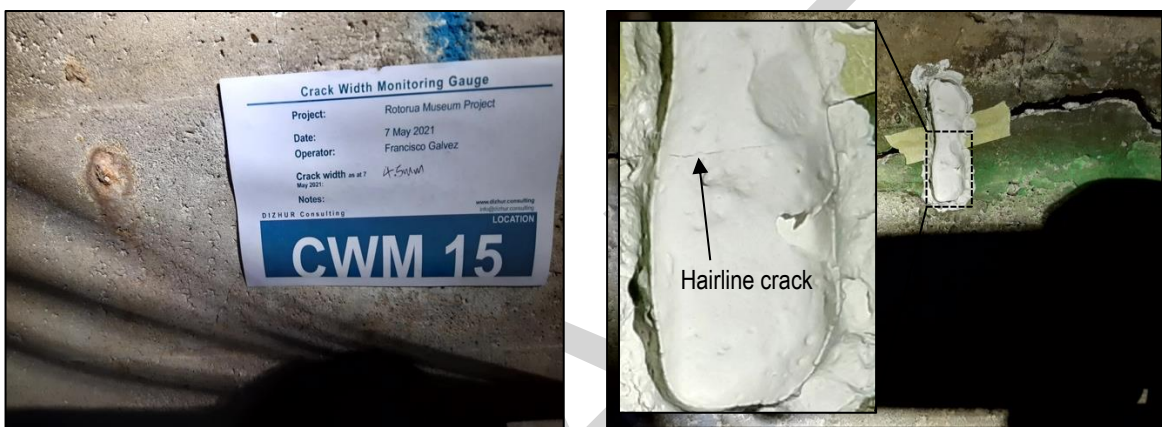
(b) CWM 05 – 0.1mm crack identified in plaster, new total crack width = approx. 2.0mm



(c) CWM 06 – hairline crack ($<0.1\text{mm}$) identified



(d) CWM 13 – hairline crack (<0.1mm) identified



(e) CWM 15 – hairline crack (<0.1mm) identified

Figure 1-2. Crack width monitoring gauges with cracking identified

1.2 August 2021 inspection

As part of a site visit undertaken on the 17 August 2021, all gauges were investigated for any sign of movement of the cracks. The following gauges exhibited new cracks in the plaster, i.e., cracks which were not present during the May 2021 inspection: CWM 3, 8, 11. The cracks in these gauges were all hairline cracks (immeasurable width, <0.1mm), see [Figure 1-3](#). CWM 3 and 8 were located within the ground floor of Zone 3, and CWM 11 was located on the southern wall of the ground floor corridor of Zone 5.

CWM 5 which exhibited a crack in the plaster of approx. 0.1mm in the May inspection, was observed to have increased in width to approx. 0.2mm. All other gauges which had previously exhibited cracks (CWM 1, 6, 13 and 15, all located in Zone 3) remained immeasurable (i.e., <0.1mm width) or exhibited approx. 0.1mm crack width. Refer to [Appendix 5](#) for all photos of gauges taken on 17 August 2021.



CWM 03 – hairline crack (<0.1mm) identified

CWM 08 – hairline crack (<0.1mm) identified

CWM 11 – hairline crack (<0.1mm) identified

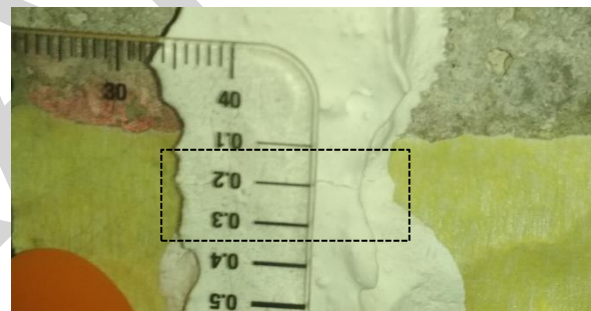
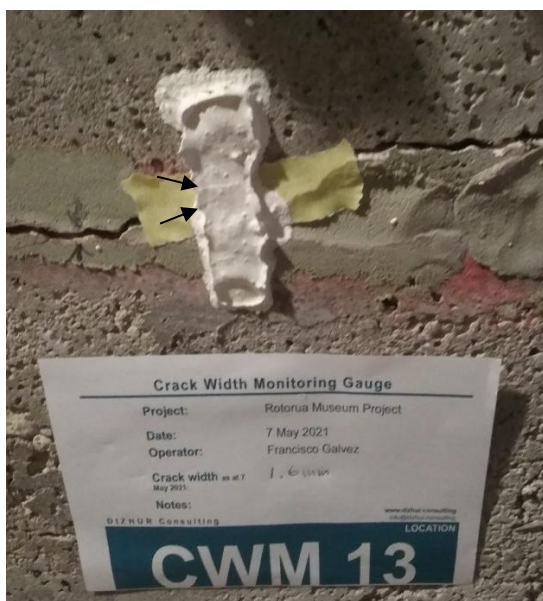
Figure 1-3. Crack width monitoring gauges with new cracking identified

1.3 November 2021 inspection

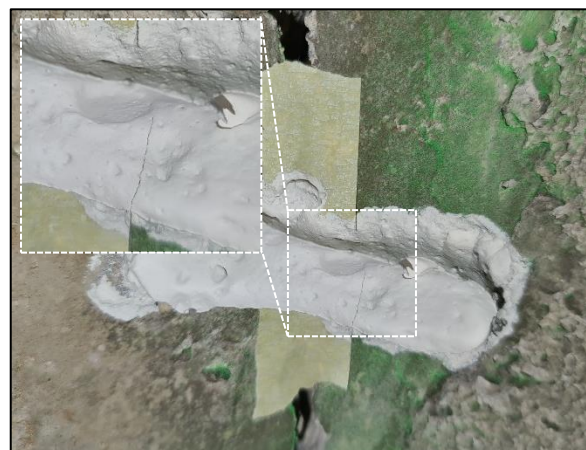
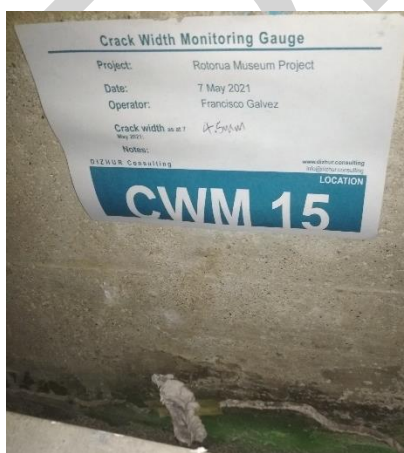
As part of a site visit undertaken on the 21 November 2021, all the already installed gauges were investigated for any sign of movement of the cracks.

The following gauges exhibited new cracks in the plaster or widening of existing cracks, i.e., cracks which were not present during the August 2021 inspection: CWM 13, 15, 17. The cracks in these gauges were all hairline cracks (immeasurable width, <0.1mm), see [Figure 1-3](#). CWM 13 and 15 were located at the Basement of Zone 3, and CWM 17 was located at the Basement of Zone 2.

CWM 13 which exhibited a hairline crack in the plaster in the August inspection, was observed to have increased in width to approx. 0.2mm and the vertical displacement was estimated as 0.3mm. A new hairline crack was observed parallel to the existing 0.2mm crack coinciding with the width of the crack in the concrete ceiling. CMW 15 was measured to have moved vertically 0.4mm, which was not detected before. Refer to [Appendix 6](#) for all photos of gauges taken on 17 November 2021.

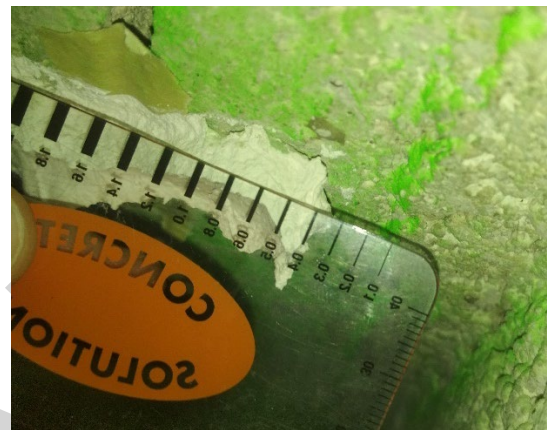
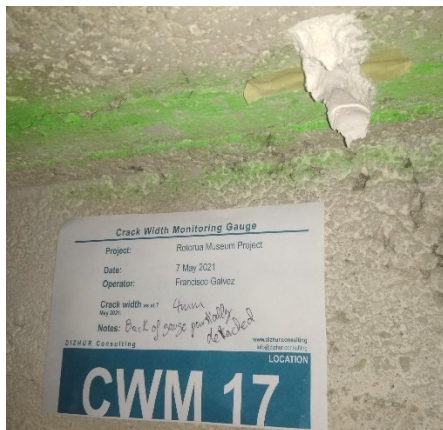


CWM 13 – 0.2mm crack opening + 0.3mm vertical movement detected. Parallel double crack.





CWM 15 – hairline crack ($<0.1\text{mm}$) (Photo not updated because of quality) – 0.4mm detachment from top/vertical movement detected



CWM 17 – no movement identified – Back detachment hard to see if movement (new gauge placed)

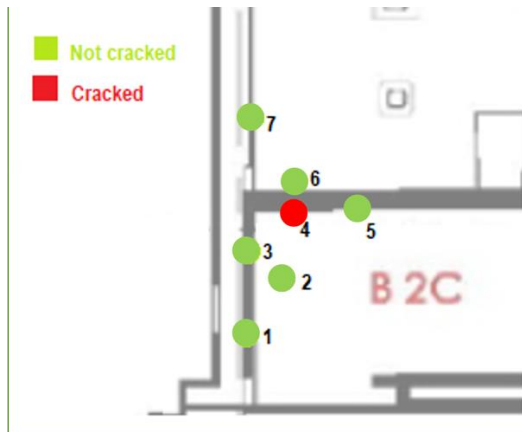
Figure 1-4. Crack width monitoring gauges with new cracking identified

Existing glass crack gauges (i.e., not placed by DIZHUR Consulting) were found in the South side of Zone 2 placed on the 23rd of January 2016. The location and whether they were found to have cracked or not is reported in Figure 1-5.

At gauge 4, approx. 2.5mm of relative displacement between the two sides of the crack had occurred, which describes the relative movement that occurred between the southern wall of Zone 2 and the perpendicular wall that the gauge was placed on (see Figure 1-5a).

Gauge 6 was placed in the same crack as gauge 4, but on the opposite side of the wall. This gauge had detached from the wall on one side of the crack, however considering the significant relative movement experienced by gauge 4, it is hypothesised that this detachment could have been caused by wall movement. Gauges 1, 3, 5 and 7 did not exhibit any type of failure.

Based on these observations, it appears that the south wall of Zone 2 is moving differentially to the orthogonal wall – in turn, this difference in movement is likely to be inducing stresses within the vaulted concrete floor which results in cracking of the slab to relieve itself of the induced stress concentrations.



(a) Plan of glass gauges locations – all are focused around southeast corner of Zone 2



(b) 1 – Not cracked



(c) 2 – not cracked (one side detached from wall)



(d) 3 – Not cracked



(e) 4 – Cracked



(f) 5 – Not cracked



(g) 6 – not cracked (one side detached from wall)



(h) 7 - Not cracked

Figure 1-5. Crack width monitoring gauges (pre-existing i.e., not installed by DIZHUR Consulting) with new cracking identified (photos taken 17 November 2021)

As part of on-site investigation works undertaken on the 21 November 2021, DIZHUR Consulting placed several crack monitoring gauges throughout the superstructure of the building on the spots where in previous campaigns no gauges were placed to determine over a period of time whether the cracks are active or dormant.

See [Appendices 7](#) for all crack width monitoring gauges and their locations and Table 2 for the crack width measurement log.

APPENDIX 1. CRACK MEASUREMENT SUMMARY

Table 1. Crack width measurement log - Round 1

ID	Zone	Location description	Existing crack width (mm) measured on: 7 May 2021	Dog-bone crackwidth inspected and measured on: 14 June 2021	Dog-bone crackwidth inspected and measured on: 17 August 2021	Dog-bone crack width inspected and measured on: 21 November 2021
CWM 1	Zone 3	GF – South gable wall	1.3	Hairline crack <0.1mm	Hairline crack <0.1mm	Hairline crack <0.1mm
CWM 2	Zone 3	GF – South gable wall	0.9	No movement	No movement	No movement
CWM 3	Zone 3	GF – Beneath south staircase	1.8	No movement	Very small hairline crack	Very small hairline crack
CWM 4	Zone 3	GF – southwest room, east wall	0.4	No movement	No movement	No movement
CWM 5	Zone 3	GF – floor next to south wall doorway	1.9	Crack, +0.1mm = 2.0mm new width	Crack, 0.2mm crack in plaster = 2.1mm new width	Crack, 0.2mm
CWM 6	Zone 3	GF – beneath north staircase	4	Hairline crack <0.1mm	0.1mm crack	0.1mm crack
CWM 7	Zone 3	GF – beneath north staircase	1.2	No movement	No movement	No movement
CWM 8	Zone 3	GF – south gable wall, southeast corner	1	No movement	Hairline crack	Hairline crack
CWM 9	Zone 4	GF – north gable wall adjacent to doorway	0.9	No movement	No movement	No movement
CWM 10	Zone 5	GF – corridor south wall	0.3	No movement	No movement	No movement
CWM 11	Zone 5	GF – corridor south wall	0.35	No movement	Hairline crack	Hairline crack
CWM 12	Zone 3	Basement – floor slab (overhead) near north gable wall	0.7	No movement	No movement	No movement
CWM 13	Zone 3	Basement – floor slab (overhead) near north gable wall	1.6	Hairline crack <0.1mm	Hairline crack + evidence of vertical displacement	Crack 0.2mm (2 nd hairline crack parallel) Vertical offset 0.3mm
CMW 14	Zone 3	Basement – floor slab extrados (overhead) near north gable wall	1.3	No movement	No movement	No movement
CMW 15	Zone 3	Basement – floor slab (overhead) near south gable	4.5	Hairline crack <0.1mm	Hairline crack <0.1mm	Hairline crack (<0.1mm) Detachment from one side top/vertical

		wall				movement 0.4mm
CMW 16	Zone 3	Basement – floor slab (overhead) near south gable wall	1.2	No movement	No movement	No movement
CMW 17	Zone 2	Basement –north gable wall near entryway	4	No movement	No movement	Possible top detachment – hard to see (new gauge placed)
CMW 18	Zone 1	Basement – CMU wall under staircase	0.55	No movement	No movement	No movement
CMW 19	Zone 1	Basement – floor slab near doorway	0.9	No movement	No movement	No movement
CMW 20	Zone 1	Basement – northern CMU wall	0.7	No movement	No movement	No movement

Table 2. Crack width measurement log – Round 2

ID	Zone	Location description	Existing crack width (mm) measured on: 17 November 2021			
CWM 21	Zone 2	GF – West Wall	2.5			
CWM 22	Zone 4	GF – East Wall	1.2			
CWM 23	Zone 5	GF – West Wall room G5N	2.0			
CWM 24	Zone 5	Basement – Wall at West from stairs	1.6			
CWM 25	Zone 5	Basement – Wall at East from stairs	1.3			
CWM 26	Zone 5	Basement – Floor East side under cooling room next to main East wall	1.8			
CWM 27	Zone 5	Basement – Floor under cooling room next to gable wall	1.1 + vertical movement			
CWM 28	Zone 5	Basement – Floor next to gable wall in the N-S central corridor	2.5 + vertical movement			
CWM 29	Zone 5	Basement – Floor in the N-S central corridor next to CWM 28	2.8			
CWM 30	Zone 5	Basement – Floor in West side in the W-E central corridor next to swimming pool wall	1.0			
CWM 31	Zone 5	Basement – Floor West side next to main West wall	0.3			
CWM 32	Zone 4	Basement – Floor in the middle of the N-S central corridor	3.0			
CWM 33	Zone 4	Basement – Floor at North side next to the swimming pool wall	0.5			
CWM 34	Zone 3	Basement – Floor at West side by the second column next to North gable wall	1.4			
CWM	Zone	Basement – Floor	1.5			

35	3	next to main entrance by the East wall				
CWM 36	Zone 3	Basement – Floor in third N-S bay at change in ceiling elevation	1.4			
CWM 41	Zone 2	Basement – Wall next to South gable wall inside room B2C	2.0			
CWM 42	Zone 4	Basement – Floor in West side next to North gable wall	1.2			

APPENDIX 2. PHOTOGRAPHS OF ALL CRACK WIDTH GAUGES INSTALLED 7 MAY 2021



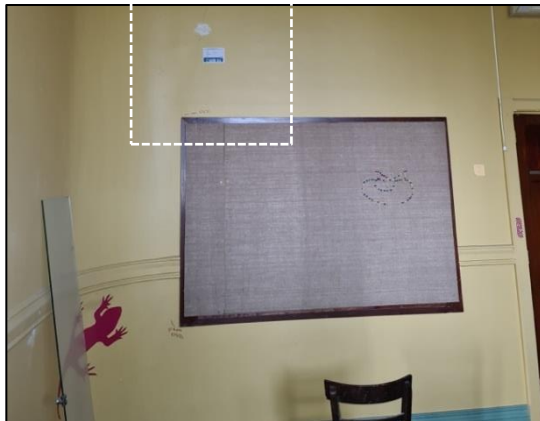
CWM 1 – crack width at time of gauge placement = 1.3mm



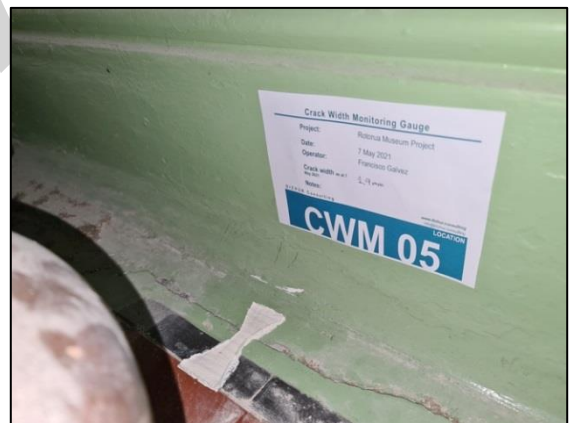
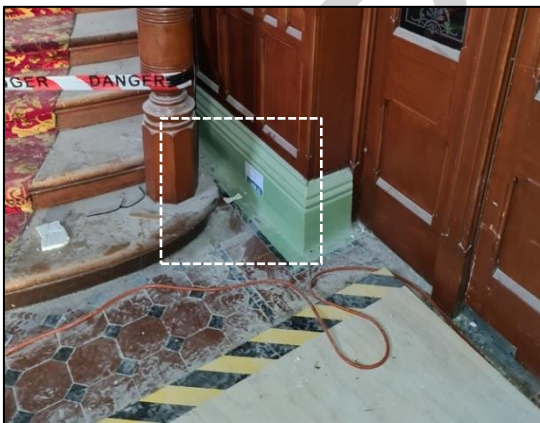
CWM 2 – crack width at time of gauge placement = 0.9mm



CWM 3 – crack width at time of gauge placement = 1.8mm



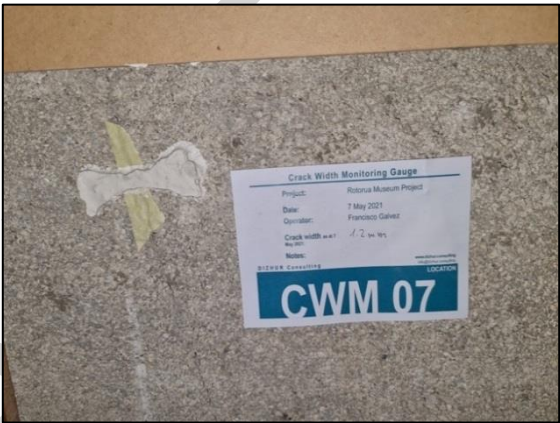
CWM 4 – crack width at time of gauge placement = 0.4mm



CWM 5 – crack width at time of gauge placement = 1.9mm



CWM 6 – crack width at time of gauge placement = 4mm



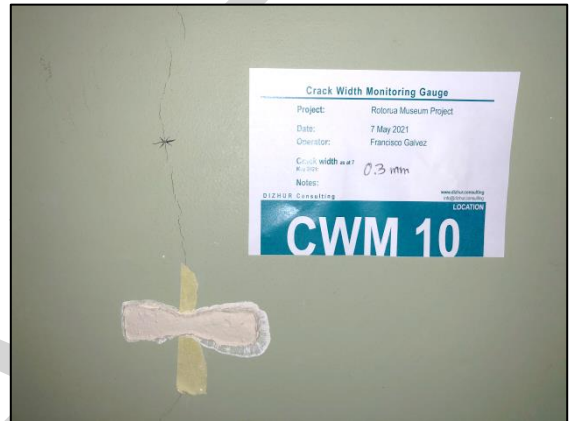
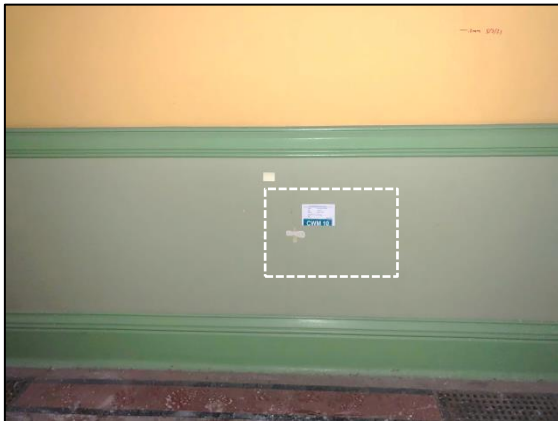
CWM 7 – crack width at time of gauge placement = 1.2mm



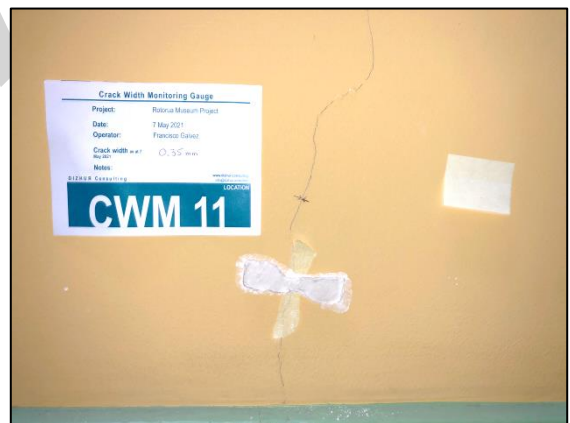
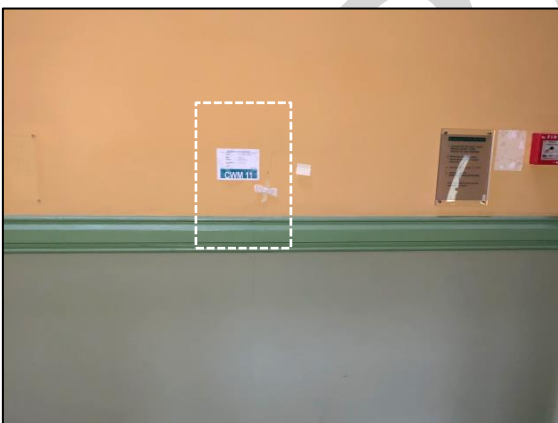
CWM 8 – crack width at time of gauge placement = 1mm



CWM 9 – crack width at time of gauge placement = 0.9mm



CWM 10 – crack width at time of gauge placement = 0.3mm



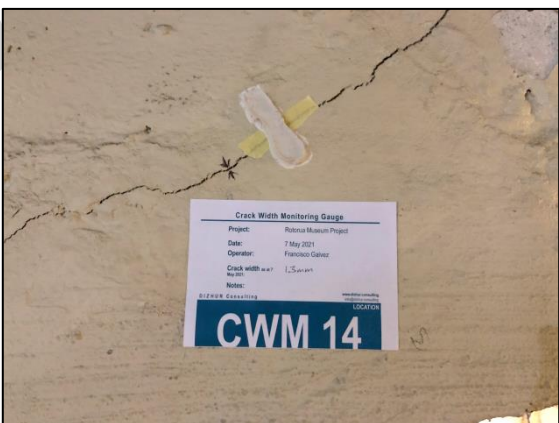
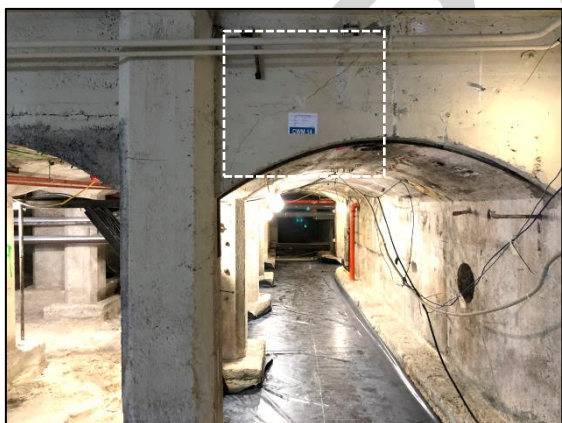
CWM 11 – crack width at time of gauge placement = 0.35mm



CWM 12 – crack width at time of gauge placement = 0.7mm



CWM 13 – crack width at time of gauge placement = 1.6mm



CWM 14 – crack width at time of gauge placement = 1.3mm



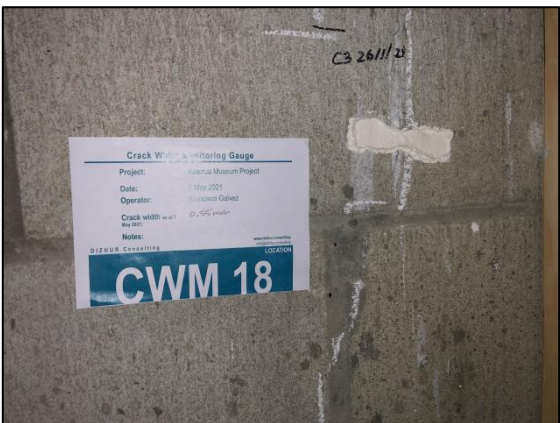
CWM 15 – crack width at time of gauge placement = 4.5mm



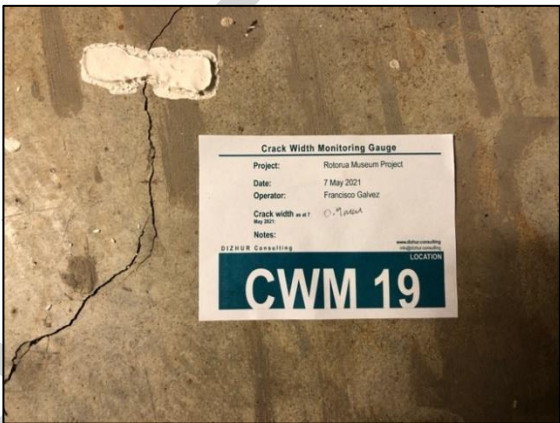
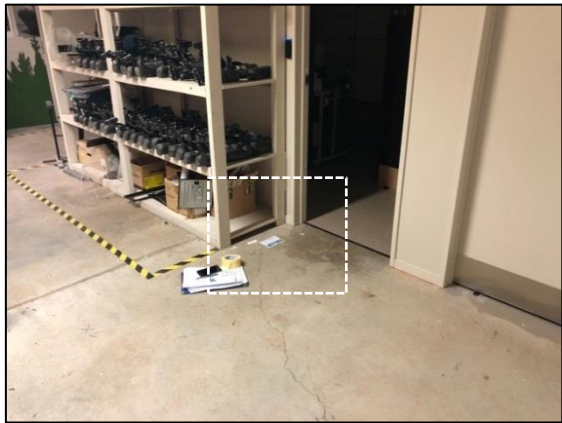
CWM 16 – crack width at time of gauge placement = 1.2mm



CWM 17 – crack width at time of gauge placement = 4mm [Notes: Back of the gauge partially detached because of difficulty during installation]



CWM 18 – crack width at time of gauge placement = 0.55mm



CWM 19 – crack width at time of gauge placement = 0.9mm



CWM 20 – crack width at time of gauge placement = 0.7mm

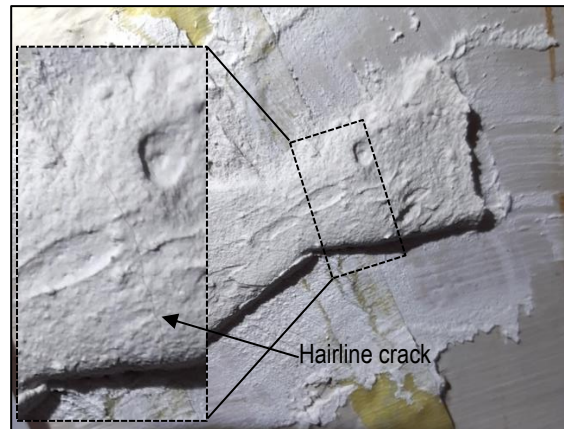
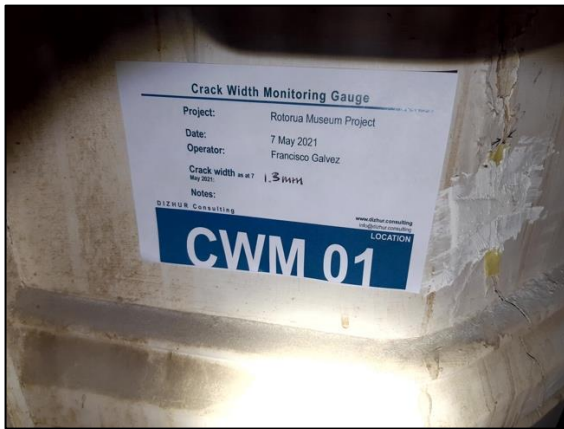
APPENDIX 3. CRACK WIDTH GAUGE LOCATIONS AND ID

A3.1 – Basement plan

A3.2 – Ground floor plan

DRAFT

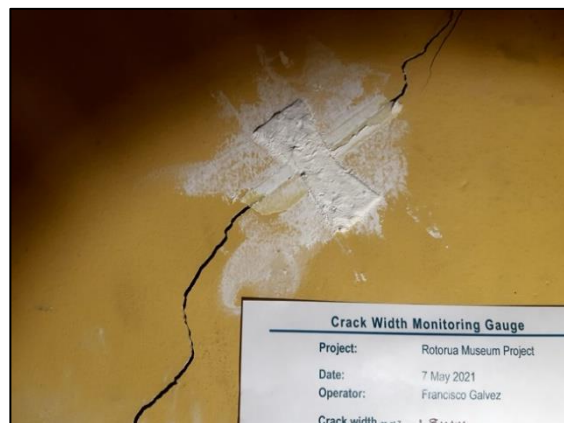
APPENDIX 4. PHOTOGRAPHS OF GAUGES TAKEN 14 JUNE 2021



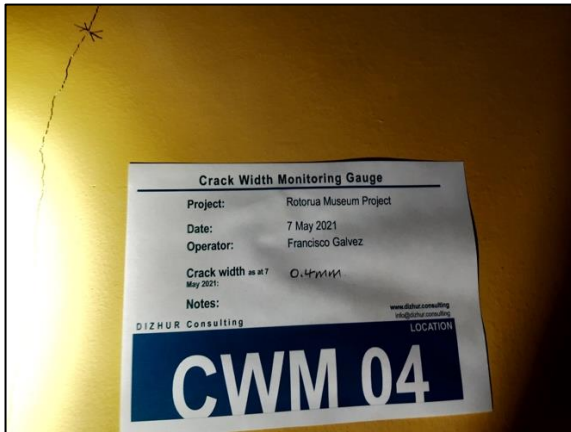
(a) CWM 01 – hairline crack (<0.1mm) identified



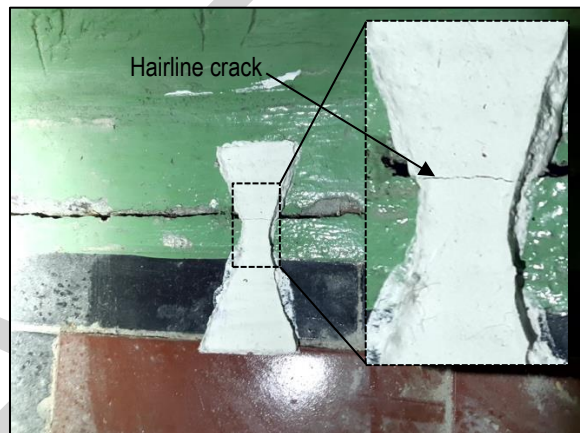
(b) CWM 02 – no movement identified



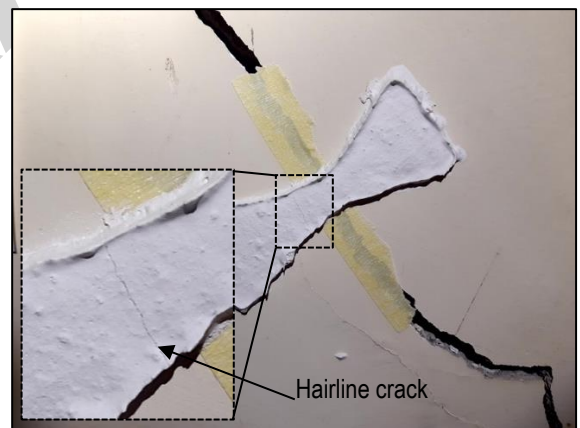
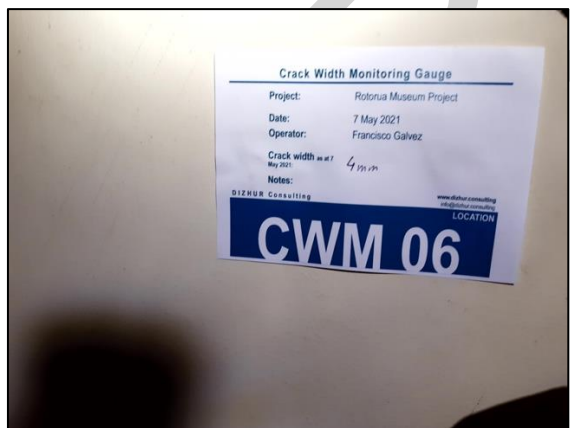
(c) CWM 03 – no movement identified



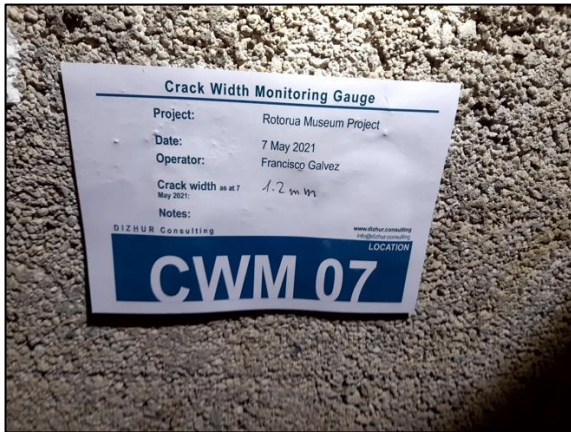
(d) CWM 04 – no movement identified



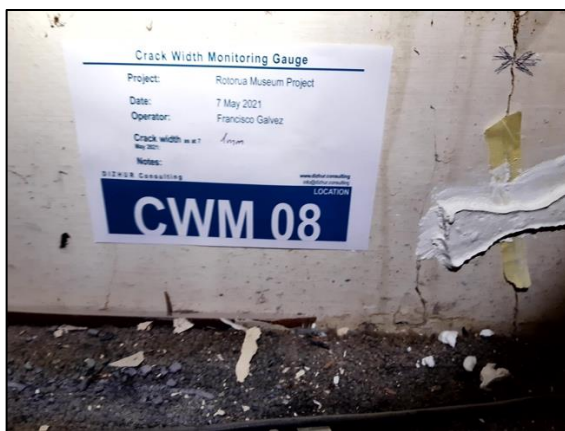
(e) CWM 05 – 0.1mm crack identified in plaster, new total crack width = 23pprox.. 2.0mm



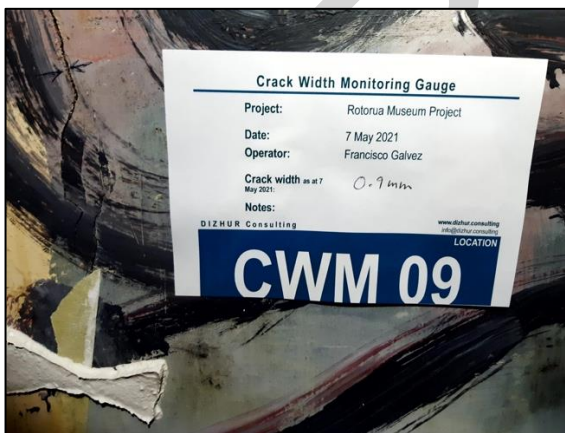
(f) CWM 06 – hairline crack (<0.1mm) identified



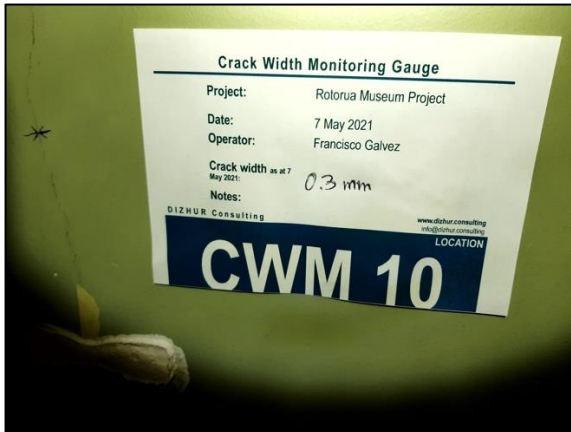
(g) CWM 07 – no movement identified



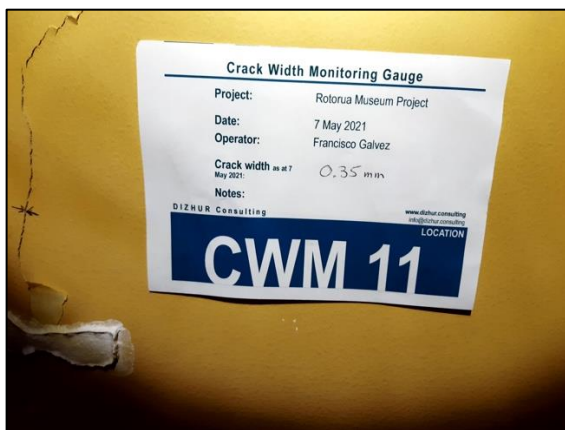
(h) CWM 08 – no movement identified



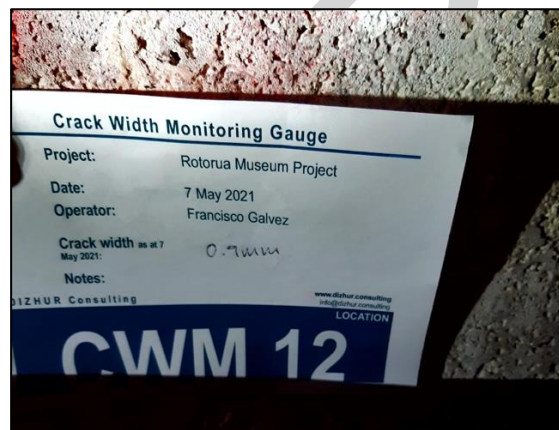
(i) CWM 09 – no movement identified



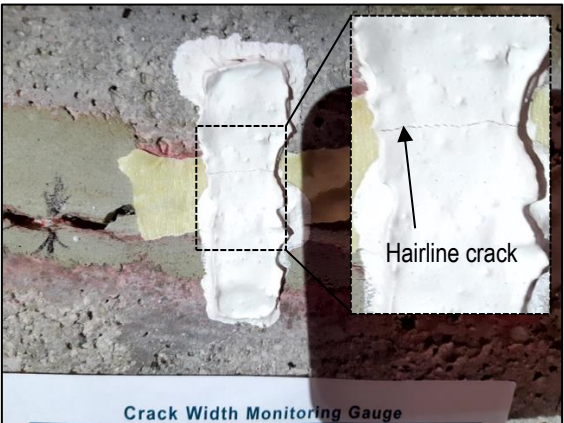
(j) CWM 10 – no movement identified



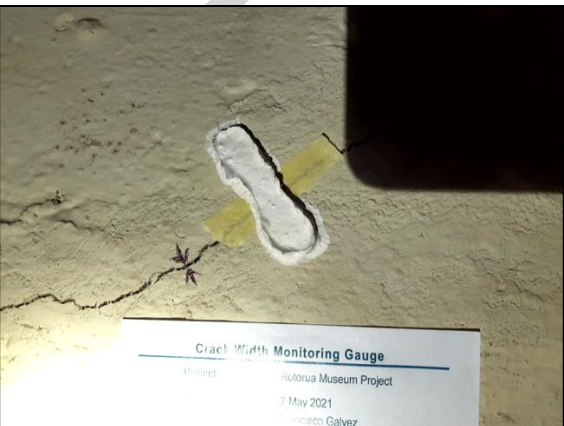
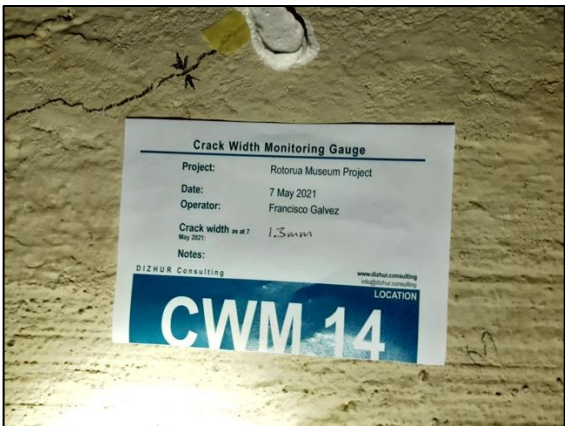
(k) CWM 11 – no movement identified



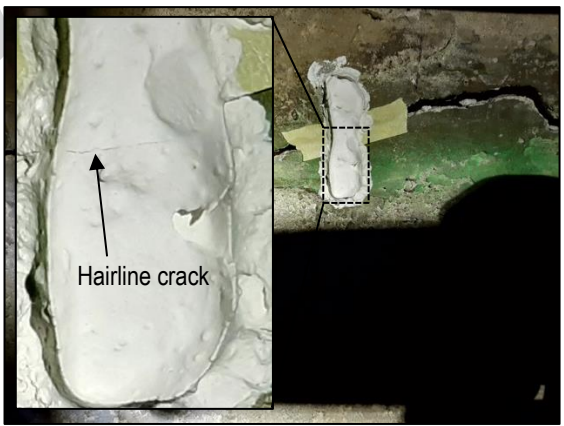
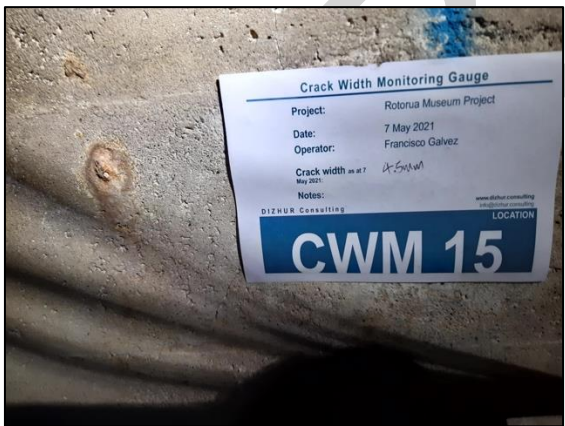
(l) CWM 12 – no movement identified



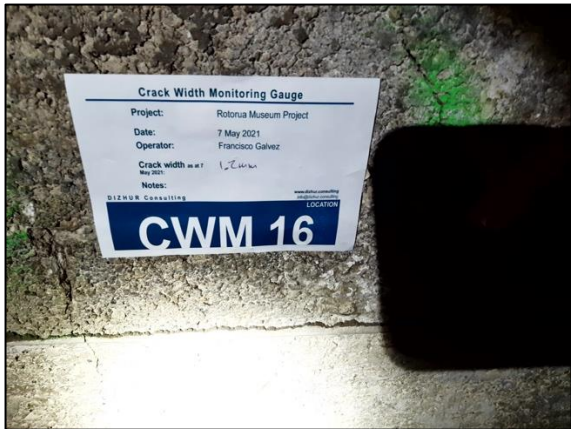
(m) CWM 13 – hairline crack (<0.1mm) identified



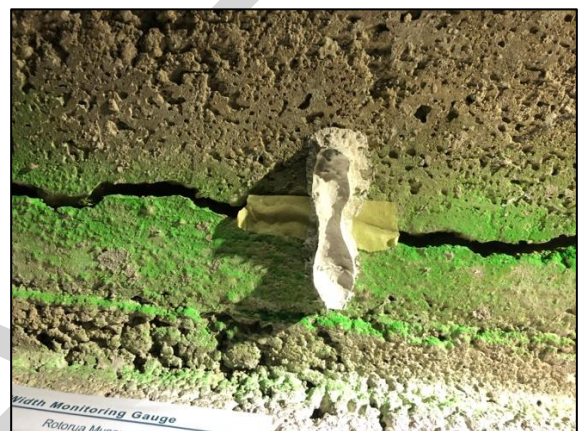
(n) CWM 14 – no movement identified



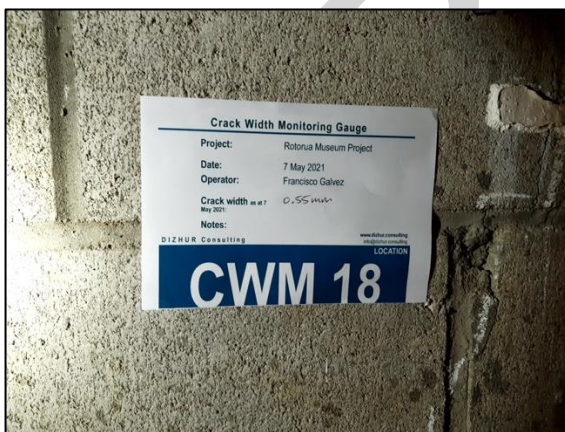
(o) CWM 15 – hairline crack (<0.1mm) identified



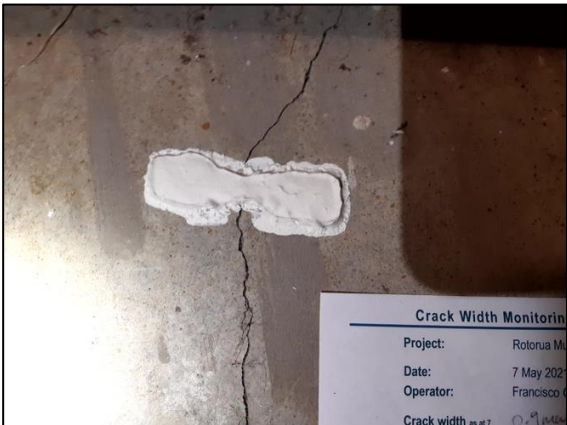
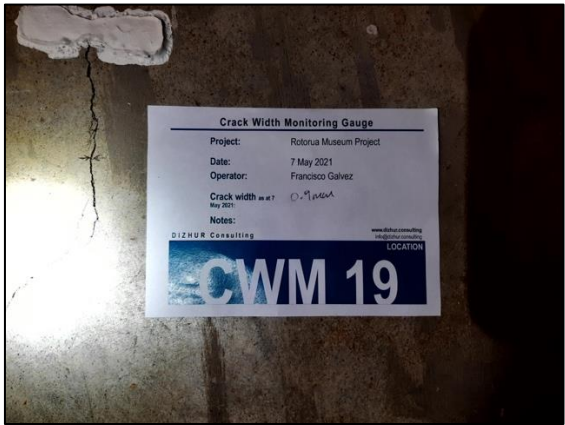
(p) CWM 16 – no movement identified



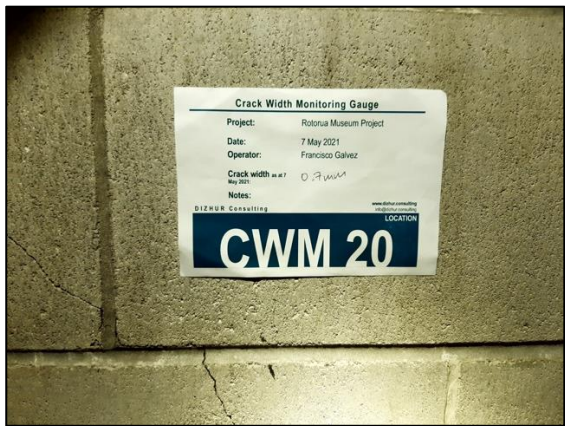
(q) CWM 17 – no movement identified



(r) CWM 18 – no movement identified

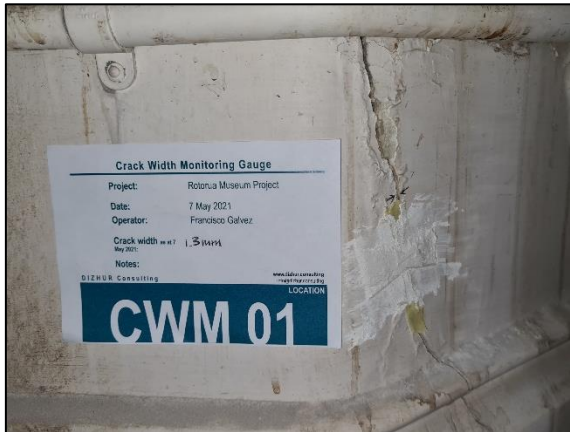


(s) CWM 19 – no movement identified

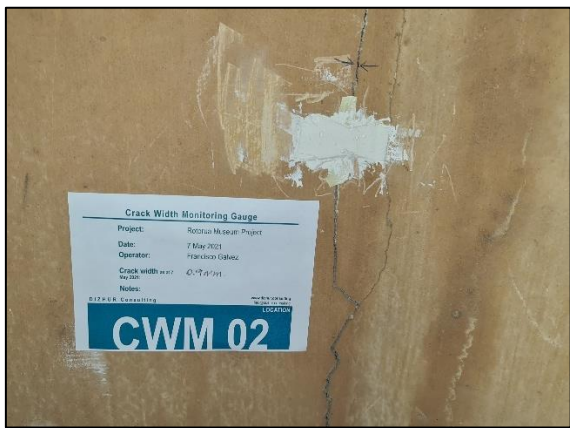


(t) CWM 20 – no movement identified

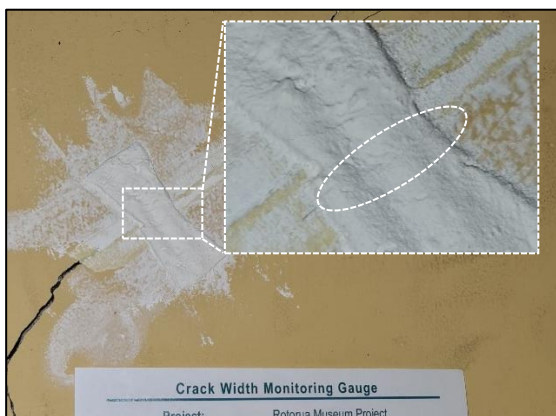
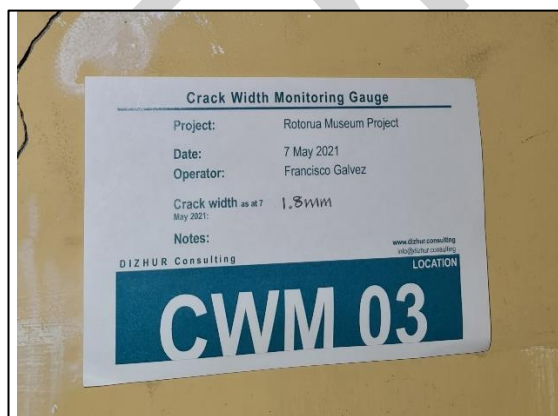
APPENDIX 5. PHOTOGRAPHS OF GAUGES TAKEN 17 AUGUST 2021



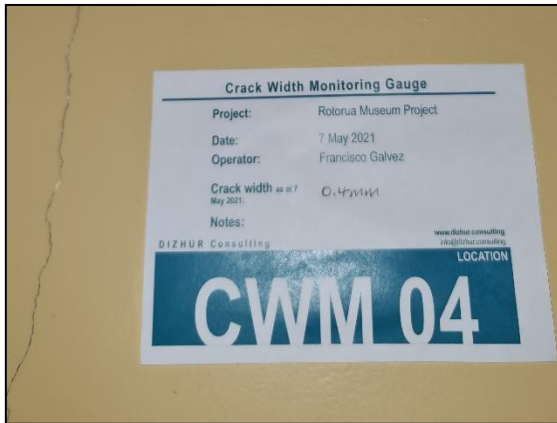
CWM 01 – hairline crack (<0.1mm) as per previous measurement



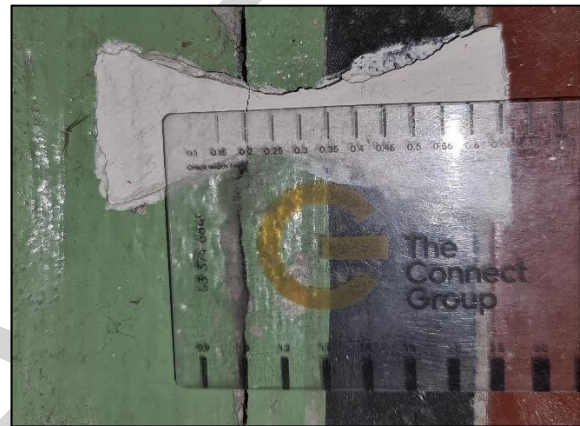
CWM 02 – no movement identified



CWM 03 – hairline crack (<0.1mm) identified



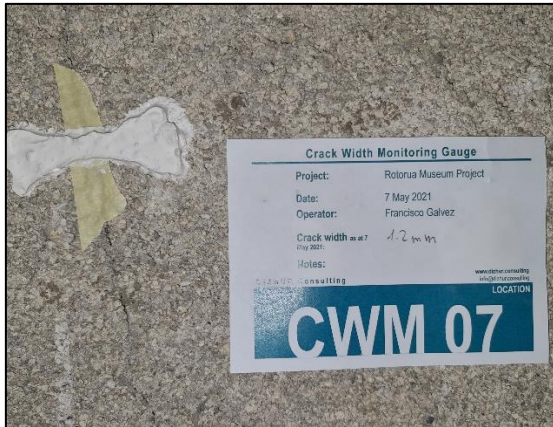
CWM 04 – no movement identified



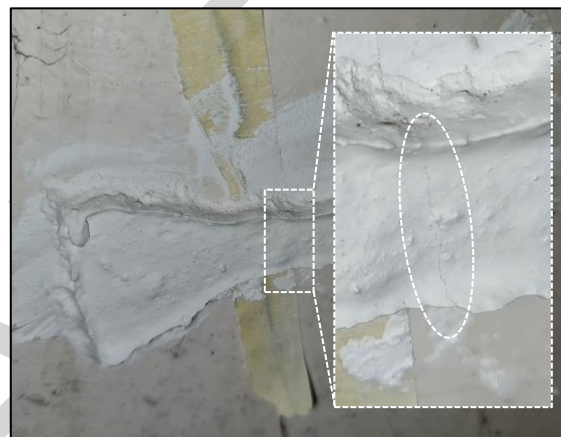
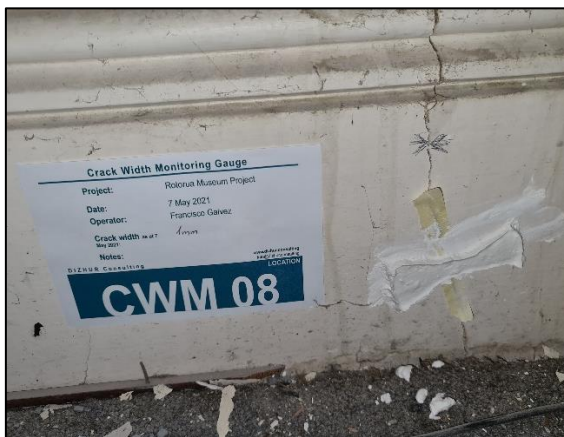
CWM 05 – 0.2mm crack, +0.1mm from previous measurement



CWM 06 – 0.1mm crack identified, previous measurement = hairline crack (<0.1mm)



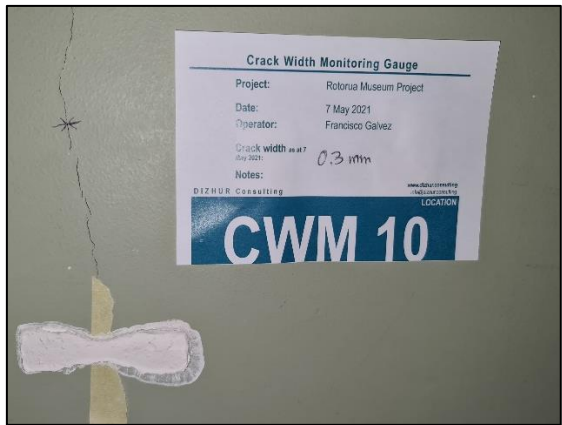
CWM 07 – no movement identified



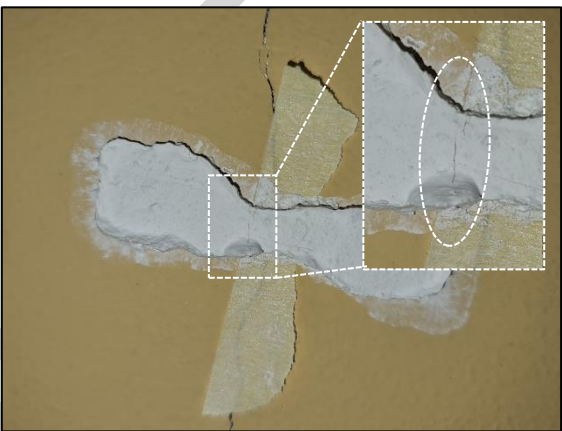
CWM 08 – hairline crack ($<0.1\text{mm}$) identified



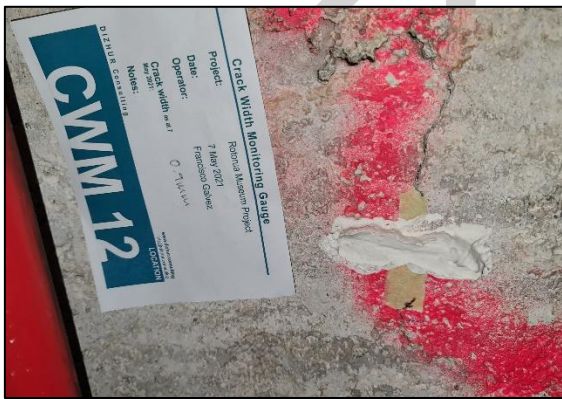
CWM 09 – no movement identified



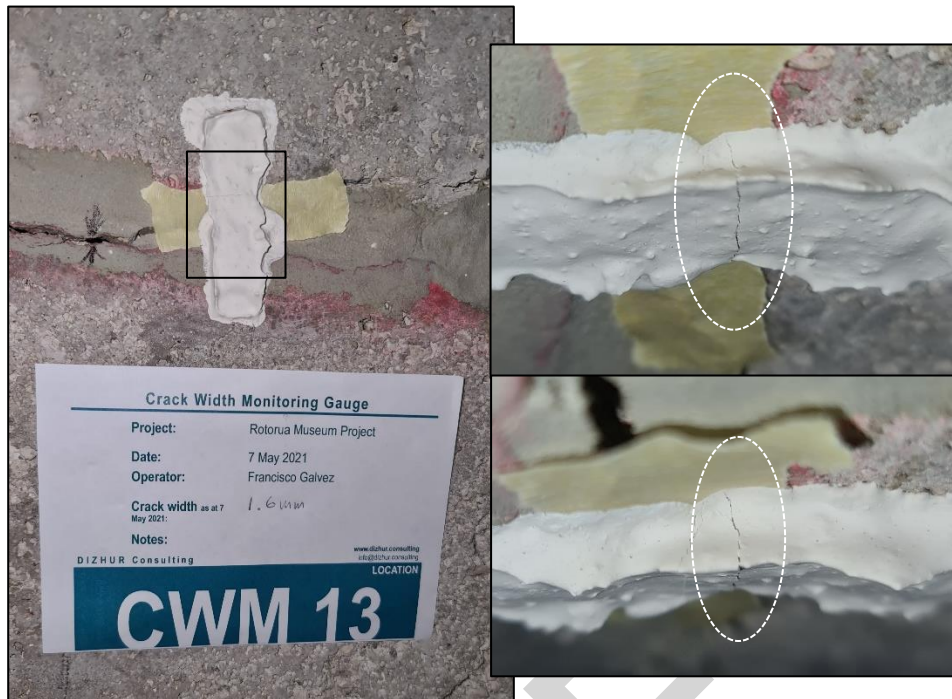
CWM 10 – no movement identified



CWM 11 – hairline crack (<0.1mm) identified



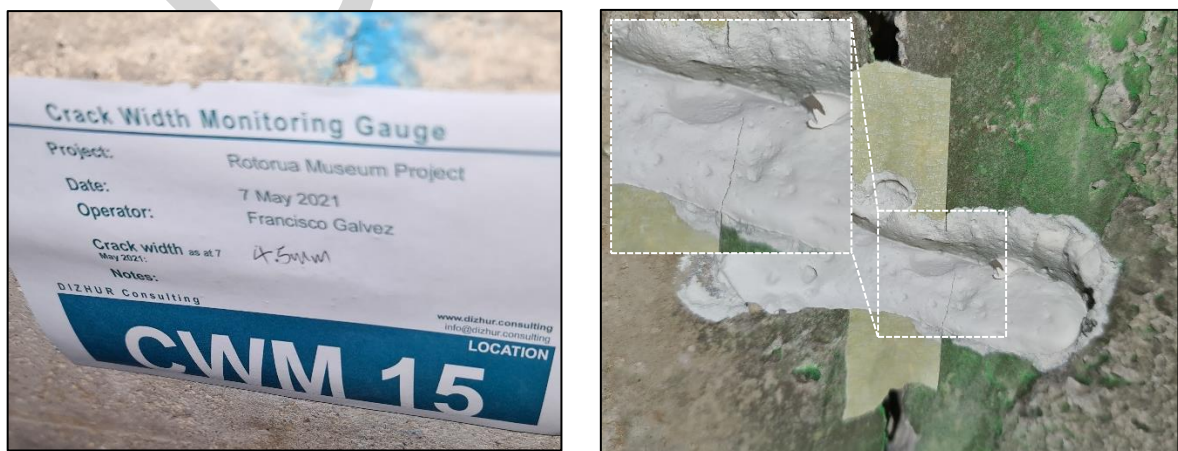
CWM 12 – no movement identified



CWM 13 – hairline crack + vertical displacement – as per previous measurement



CWM 14 – no movement identified



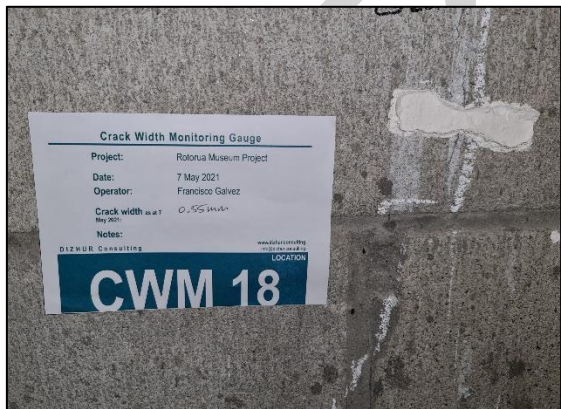
CWM 15 – hairline crack (<0.1mm) – as per previous measurement



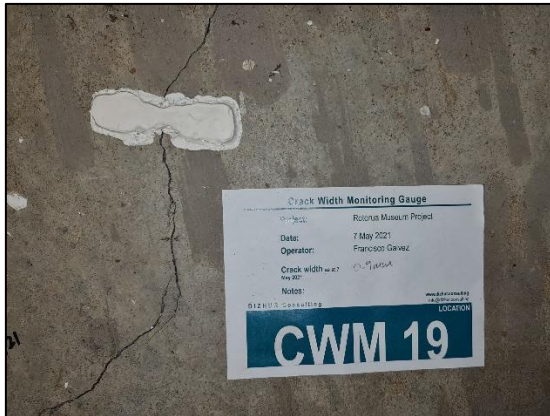
CWM 16 – no movement identified



CWM 17 – no movement identified



CWM 18 – no movement identified



Crack Width Monitoring Gauge

Project: Rotorua Museum Project

Date: 7 May 2021

Operator: Francisco Galvez

Crack width as at 7 May 2021: 0.7mm

Notes:

DIZHUR Consulting

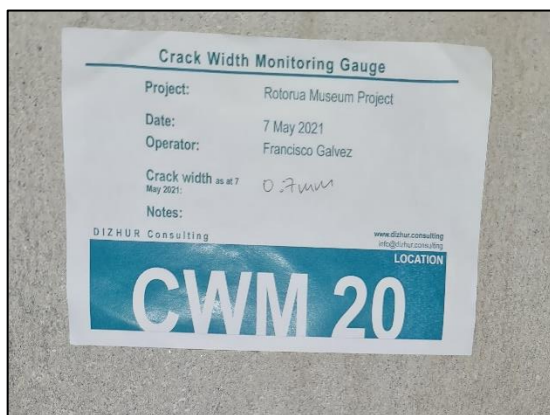
www.dizhurconsulting.co.nz

LOCATION

CWM 19



CWM 19 – no movement identified



Crack Width Monitoring Gauge

Project: Rotorua Museum Project

Date: 7 May 2021

Operator: Francisco Galvez

Crack width as at 7 May 2021: 0.7mm

Notes:

DIZHUR Consulting

www.dizhurconsulting.co.nz

LOCATION

CWM 20

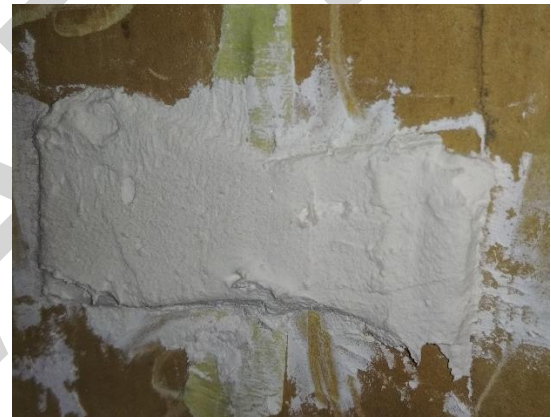


CWM 20 – no movement identified

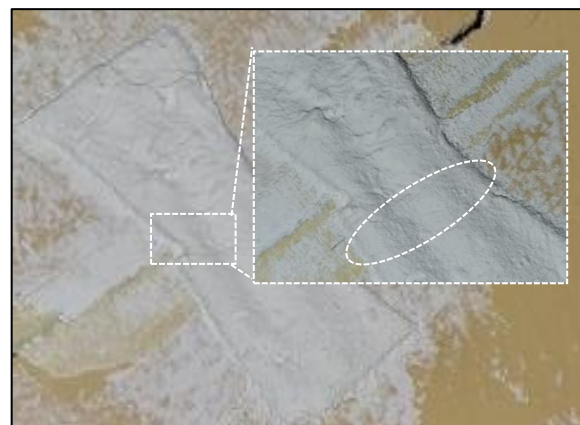
APPENDIX 6. PHOTOGRAPHS OF GAUGES TAKEN 18 NOVEMBER 2021



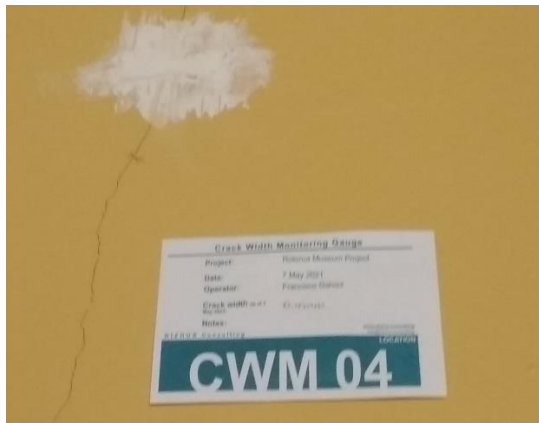
CWM 01 – hairline crack (<0.1mm) as per previous measurement (Photo not updated because of quality)



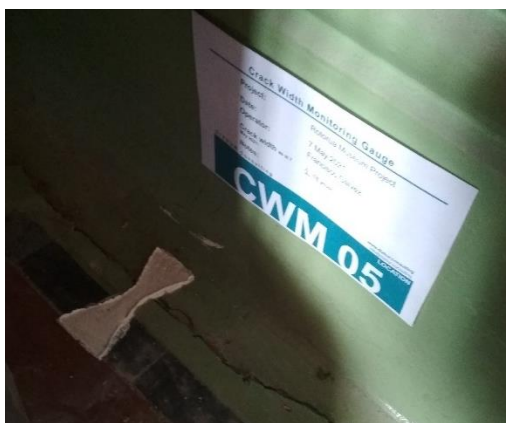
CWM 02 – no movement identified



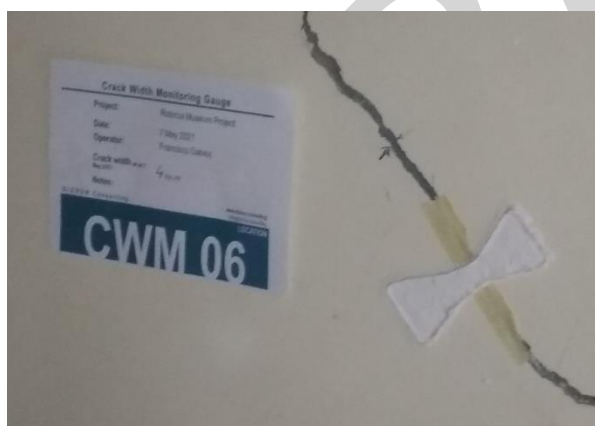
CWM 03 – hairline crack (<0.1mm) as per previous measurement (Photo not updated because of quality)



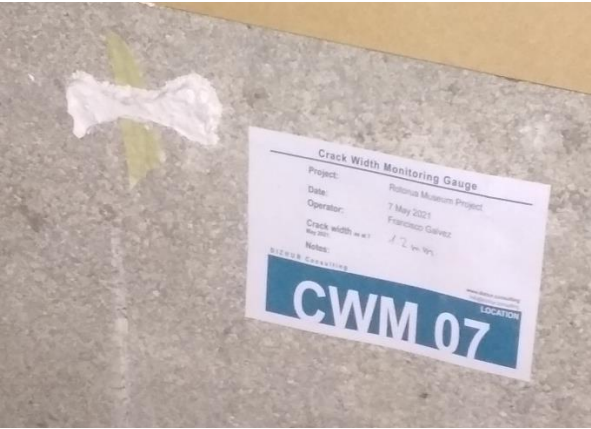
CWM 04 – no movement identified



CWM 05 – 0.2mm crack as per previous measurement

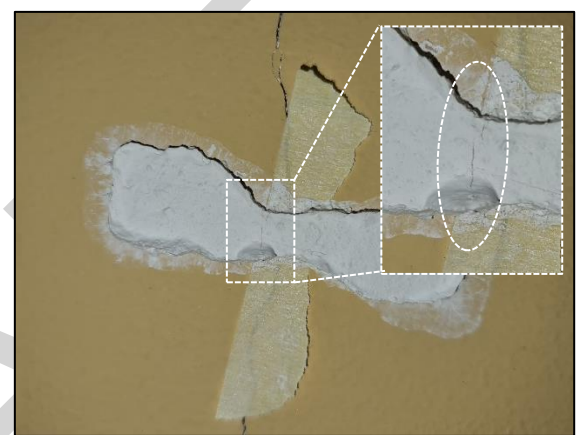
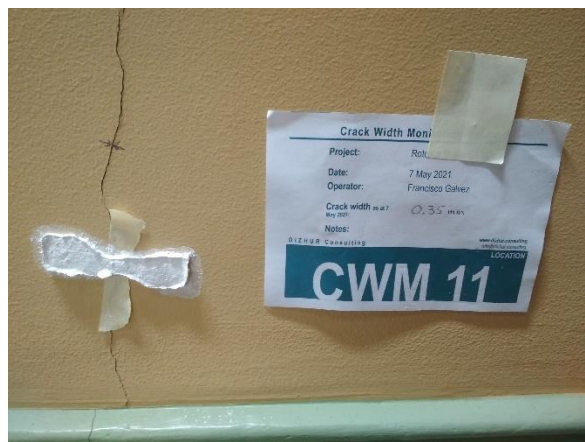


CWM 06 – 0.1mm crack as per previous measurement (Photo not updated because of quality)





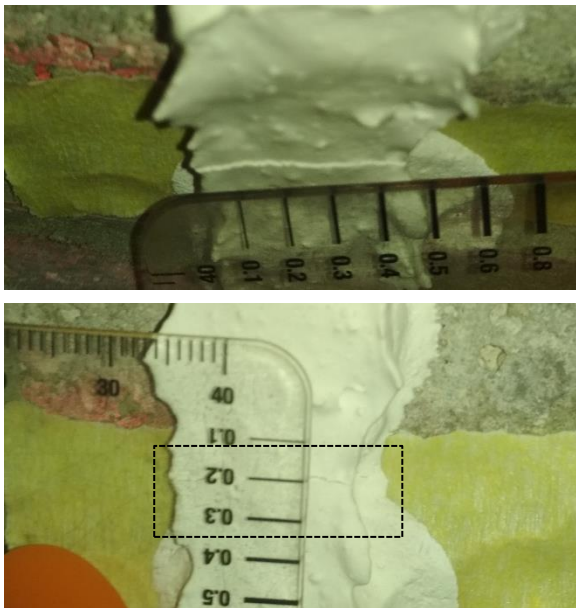
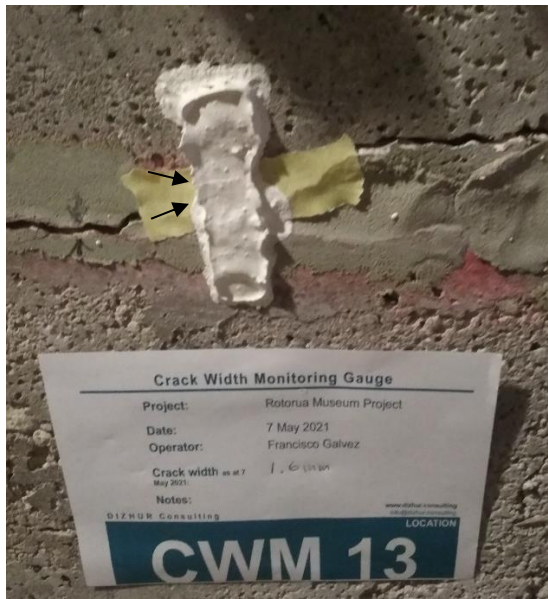
CWM 10 – no movement identified



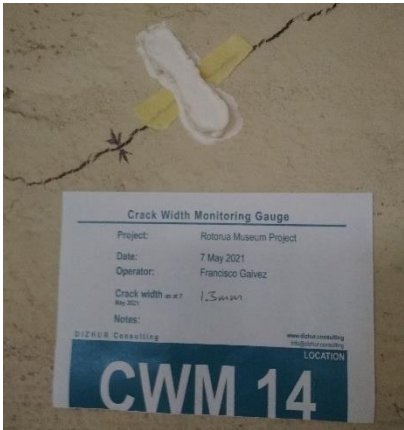
CWM 11 – hairline crack (<0.1mm) as per previous measurement



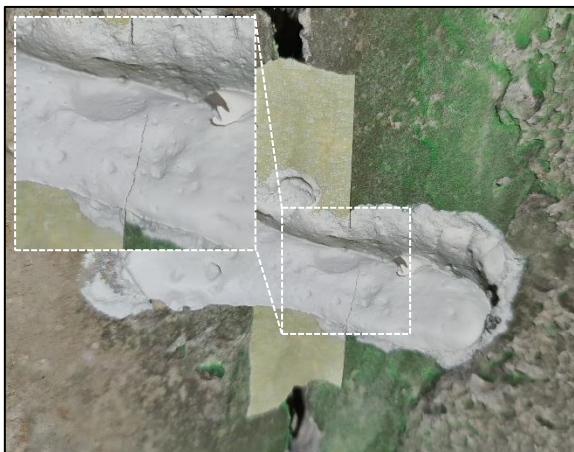
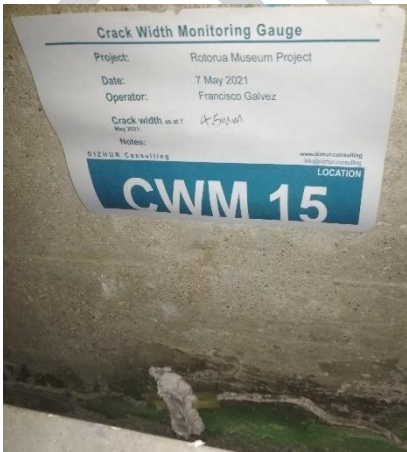
CWM 12 – no movement identified



CWM 13 – 0.2mm crack opening + 0.3mm vertical movement detected. Parallel double crack.



CWM 14 – no movement identified

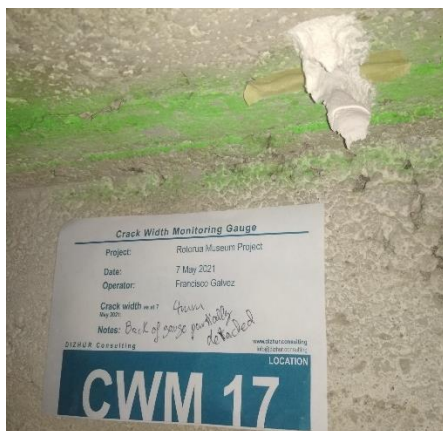




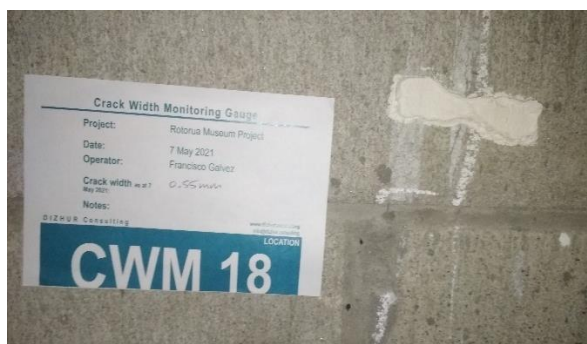
CWM 15 – hairline crack (<0.1mm) (Photo not updated because of quality) – 0.4mm detachment from top/vertical movement detected



CWM 16 – no movement identified



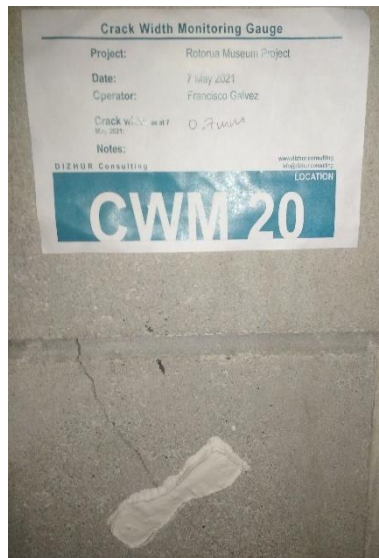
CWM 17 – no movement identified – Back detachment hard to see if movement (new gauge placed)



CWM 18 – no movement identified



CWM 19 – no movement identified



CWM 20 – no movement identified

APPENDIX 7. PHOTOGRAPHS OF ALL CRACK WIDTH GAUGES INSTALLED 17 NOVEMBER 2021



CWM 21 – crack width at time of gauge placement = 2.5mm



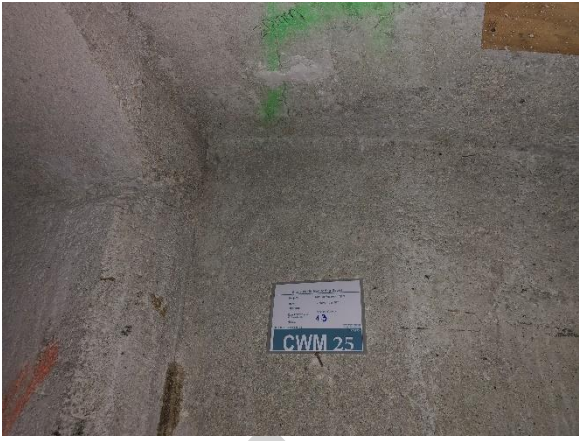
CWM 22 – crack width at time of gauge placement = 1.2mm



CWM 23 – crack width at time of gauge placement = 2.0mm



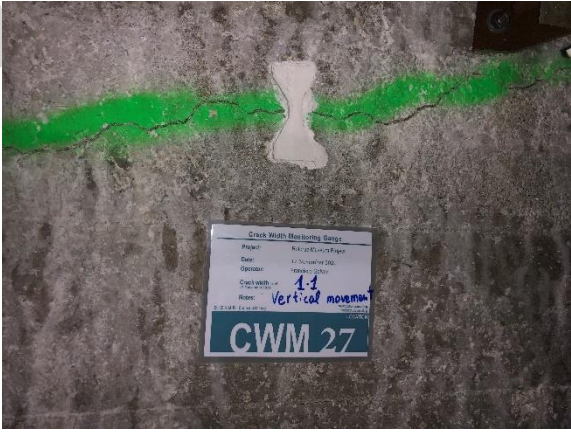
CWM 24 – crack width at time of gauge placement = 1.6mm



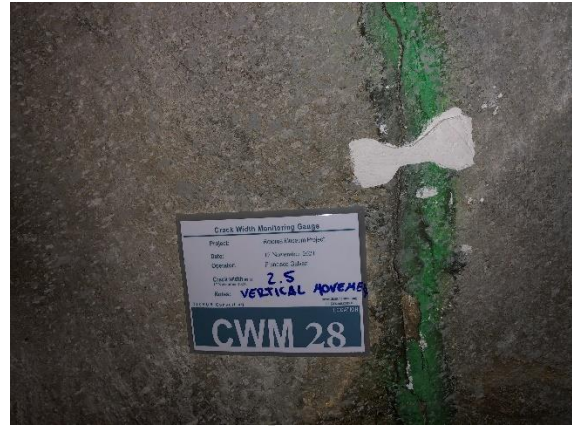
CWM 25 – crack width at time of gauge placement = 1.3mm



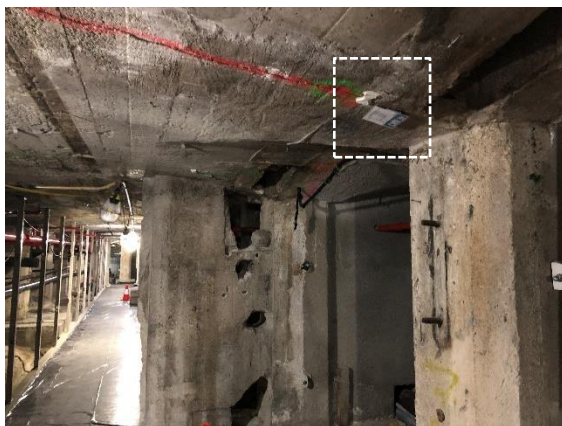
CWM 26 – crack width at time of gauge placement = 1.8mm



CWM 27 – crack width at time of gauge placement = 1.1mm + vertical movement



CWM 28 – crack width at time of gauge placement = 2.5mm + vertical movement



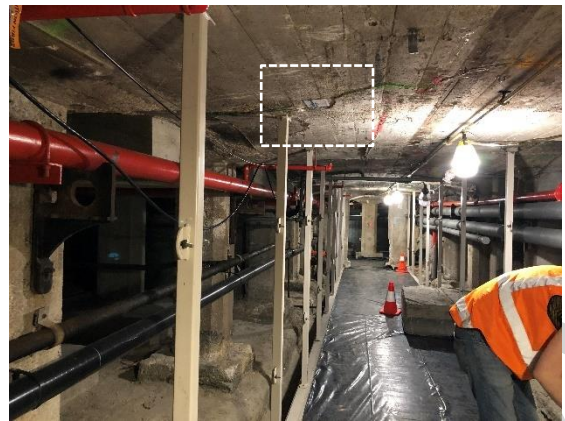
CWM 29 – crack width at time of gauge placement = 2.8mm



CWM 30 – crack width at time of gauge placement = 1mm



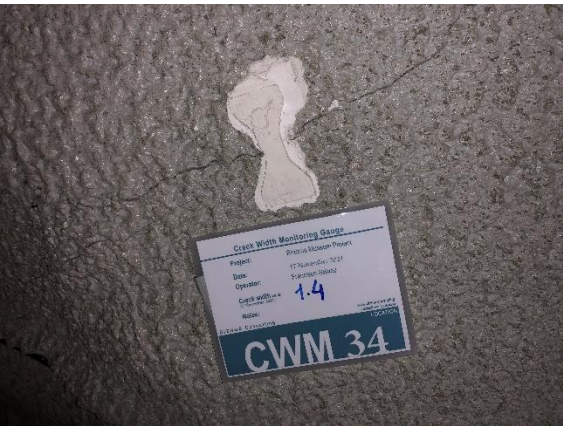
CWM 31 – crack width at time of gauge placement = 0.3mm



CWM 32 – crack width at time of gauge placement = 3mm + vertical movement



CWM 33 – crack width at time of gauge placement = 0.5mm



CWM 34 – crack width at time of gauge placement = 1.4mm



CWM 35 – crack width at time of gauge placement = 1.5mm



CWM 36 – crack width at time of gauge placement = 1.4mm



CWM 41 – crack width at time of gauge placement = 2mm



CWM 42 – crack width at time of gauge placement = 1.2mm

ROTORUAMUSEUM DEVELOPMENT DRAWING LIST			
SHEET NO.	SHEET	REVISION	DATE
S000	COVER PAGE		

ZONE 1 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S1-101	GROUND FLOOR PLAN	A	03.12.21

ZONE 2 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S2-101	GROUND FLOOR PLAN	A	29.11.21
S2-102	BRIDGE WALKWAY	A	29.11.21
S2-103	CEILING PLAN	A	29.11.21
S2-201	SECTIONS	A	29.11.21
S2-202	SECTIONS	A	29.11.21
S2-203	SECTIONS	A	29.11.21
S2-301	DETAILS	A	29.11.21

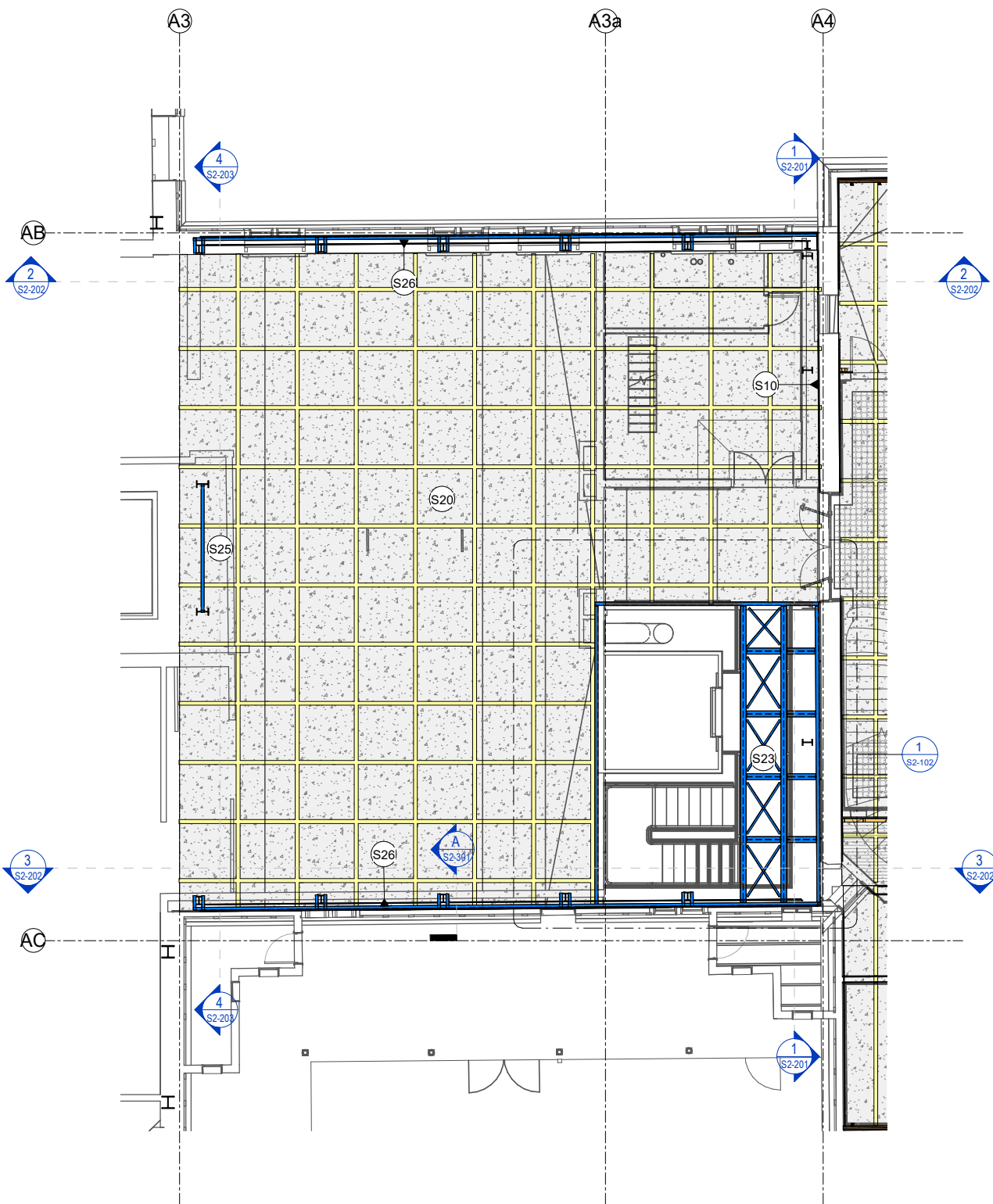
ZONE 3 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S3-101	FOUNDATION PLAN	A	23.11.21
S3-102	GROUND FLOOR PLAN	A	23.11.21
S3-103	MEZZANINE FRAMING PLAN	A	23.11.21
S3-104	MEZZANINE FLOOR PLAN	A	23.11.21
S3-105	CEILING PLAN	A	23.11.21
S3-106	ROOF PLANS	A	23.11.21
S3-201	SECTIONS	A	23.11.21
S3-202	SECTIONS	A	23.11.21
S3-203	SECTIONS	A	23.11.21
S3-204	SECTIONS	A	23.11.21
S3-205	SECTIONS	A	23.11.21
S3-301	DETAILS	A	23.11.21
S3-302	DETAILS	A	23.11.21

ZONE 4 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S4-101	FOUNDATION PLAN	A	29.11.21
S4-102	GROUND FLOOR PLAN	A	29.11.21
S4-103	MEZZANINE PLAN	A	29.11.21
S4-104	CEILING PLAN	A	29.11.21
S4-201	SECTIONS	A	29.11.21
S4-202	SECTIONS	A	29.11.21
S4-203	SECTIONS	A	29.11.21
S4-301	DETAILS	A	29.11.21

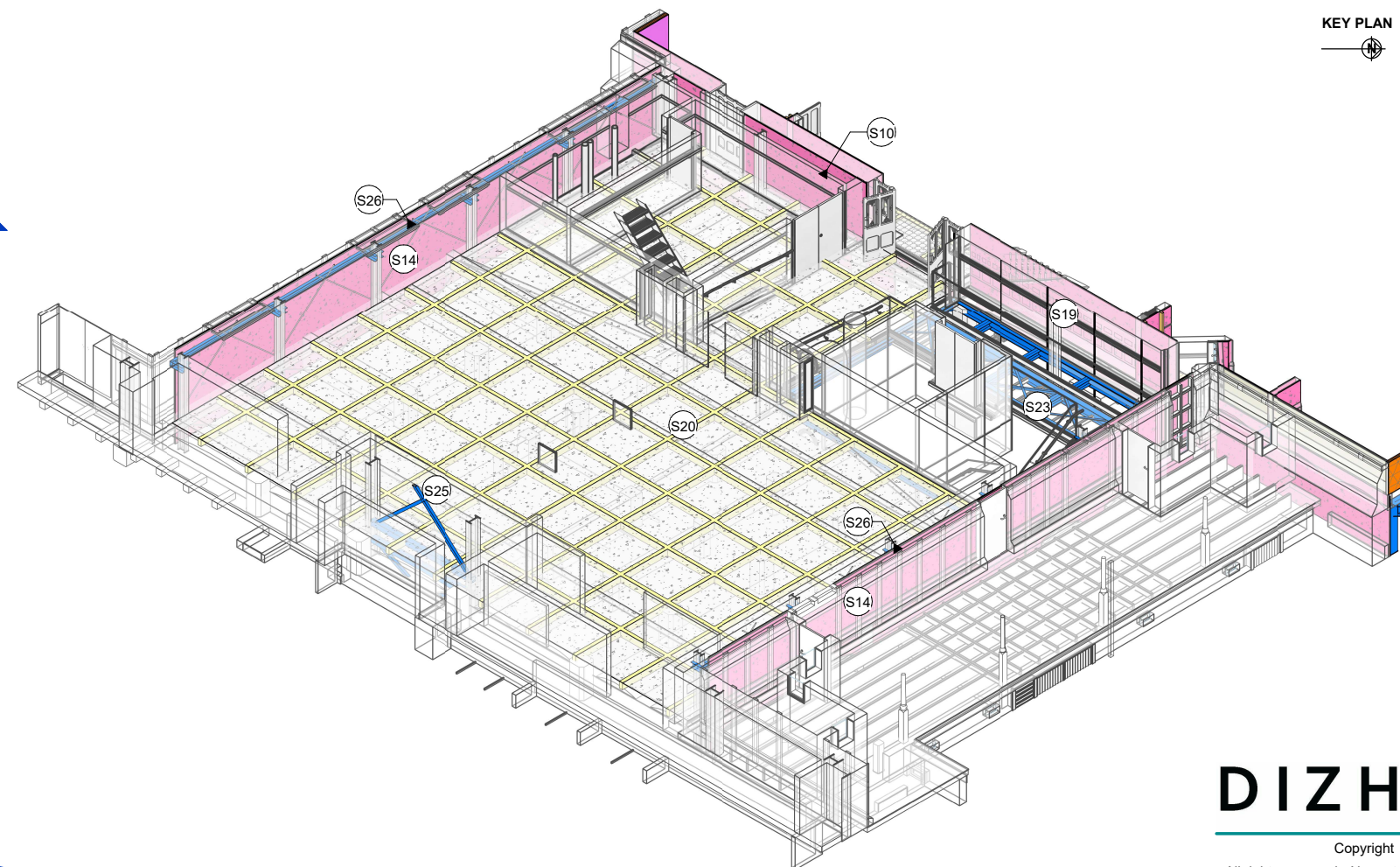
ZONE 5 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S5-101	FOUNDATION PLAN	A	03.12.21
S5-102	GROUND FLOOR PLAN	A	03.12.21
S5-103	ROOF PLAN	A	03.12.21
S5-201	SECTIONS	A	03.12.21
S5-202	SECTIONS	A	03.12.21
S5-203	SECTIONS	A	03.12.21
S5-301	DETAILS	A	03.12.21

ZONE 6 CONCEPT DRAWINGS			
SHEET NO.	SHEET	REVISION	DATE
S6-101	GROUND FLOOR PLAN	A	03.12.21





ZONE 2 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S19	NSM CFRP STRIPS AT 1.6m MAX SPACINGS
S20	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S2-301
S21	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOISTS WITH NOGGING TO SUIT AS NEEDED
S22	12mm PLYWOOD SHEATHING OVER EXISTING SARKING, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S23	NEW STEEL FRAMING WITH CROSS BRACING TO FORM BRIDGE WALKWAY
S24	NEW HORIZONTAL STEEL TRUSS
S25	NEW 100SHS BRACING TO EXISTING VERTICAL TRUSS
S26	NEW 150PPC TO EXISTING NIB WALL AND EXISTING STEEL COLUMNS
S27	NEW STEEL BEAM FOR OOP SUPPORT



EXISTING REIDBRACE REMEDIATION NOTES:

1. WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO THE SAME FLANGE ON EITHER SIDE OF THE COLUMN AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT EITHER OF THE REMEDIAL SOLUTIONS FROM BELOW:
 - A. WELD ADDITIONAL STEEL PACKER PLATE TO COLUMN FLANGE IN ORDER TO CLOSE THE NON-COMPLIANT GAP BETWEEN THE COLUMN FLANGE AND INTERNAL FACE OF RBRACE-END;
 - B. FABRICATE AND BOLT CUSTOM DESIGNED STEEL GUSSET PLATE TO THE COLUMN SUCH THAT THE CONNECTION OF RBRACE-END IS CONCENTRIC TO THE COLUMN SECTION. THIS WILL SHIFT THE REID BRACES FROM COLUMN FLANGES TO CENTRE OF COLUMN WEB.
2. WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO OPPOSITE FLANGES ON EITHER END OF THE COLUMN WEB AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT THE FOLLOWING REMEDIAL SOLUTION:

FABRICATE AND BOLT CUSTOM DESIGNED STEEL GUSSET PLATE TO THE COLUMN SUCH THAT THE CONNECTION OF RBRACE-END IS CONCENTRIC TO THE COLUMN SECTION. THIS WILL SHIFT THE REID BRACES FROM COLUMN FLANGES TO CENTRE OF COLUMN WEB.
3. WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO THE SAME FLANGE ON EITHER SIDE OF THE COLUMN AND THE CONNECTION IS CLOSE TO A BEAM SUPPORTED ON THE COLUMN, ADOPT A COMBINATION OF SOLUTIONS FROM BELOW:
 - A. WELD ADDITIONAL STEEL PACKER PLATE TO COLUMN FLANGE IN ORDER TO CLOSE THE NON-COMPLIANT GAP BETWEEN THE COLUMN FLANGE AND INTERNAL FACE OF RBRACE-END;
 - B. ADD ADDITIONAL STEEL STIFFENER PLATES TO COLUMN AND REVIEW TORSIONAL CONNECTION WITH THE CONNECTING BEAMS. IF THE TORSIONAL CONNECTION DOES NOT EXIST, BOLT ADDITIONAL STEEL BRACKETS FROM BEAM TO COLUMN.

NOTE: THE CONCEPTUAL OPTIONS DESCRIBED ABOVE ARE FOR PRELIMINARY COSTING PURPOSES AND WILL NEED TO BE REFINED ON A CASE-BY-CASE BASIS. THE CONCEPTUAL OPTIONS DESCRIBED ABOVE AND THEIR REFINEMENT SHALL BE SUBJECT TO CONSTRUCTABILITY, SITE CONSTRAINTS AND QUANTITY OF PERMISSIBLE SITE HOT WORKS



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[illegible]

ROTORUA
LAKE COUNCIL
Te Kauriwha o ngāi Roto o Rotorua

ROTORUA MUSEUM
DEVELOPMENT

ZONE 2

BRIDGE WALKWAY

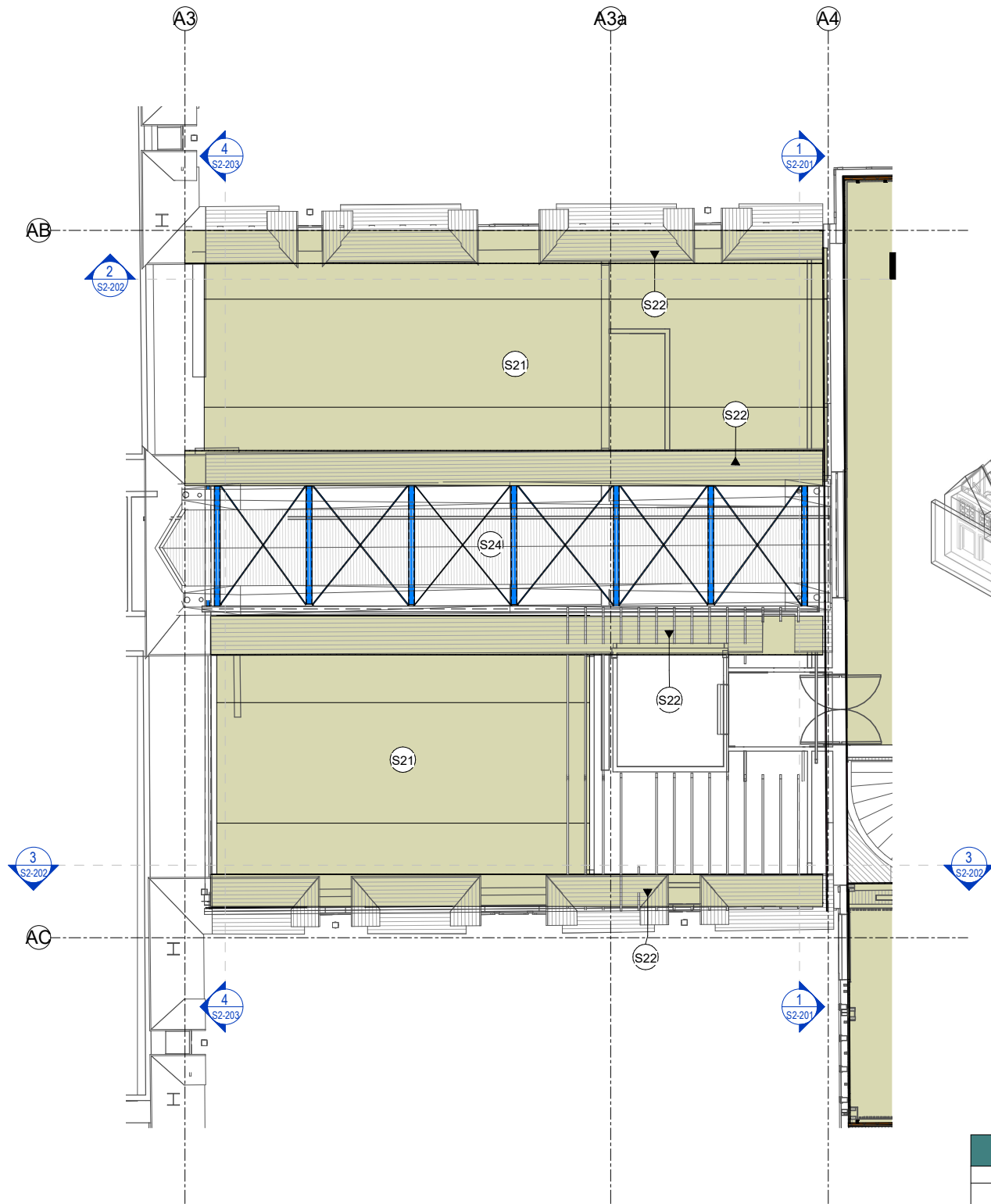
PRELIMINARY CONCEPT
80% NBS (IL3)

SCALE 1:25 @ A1
1:50 @ A3

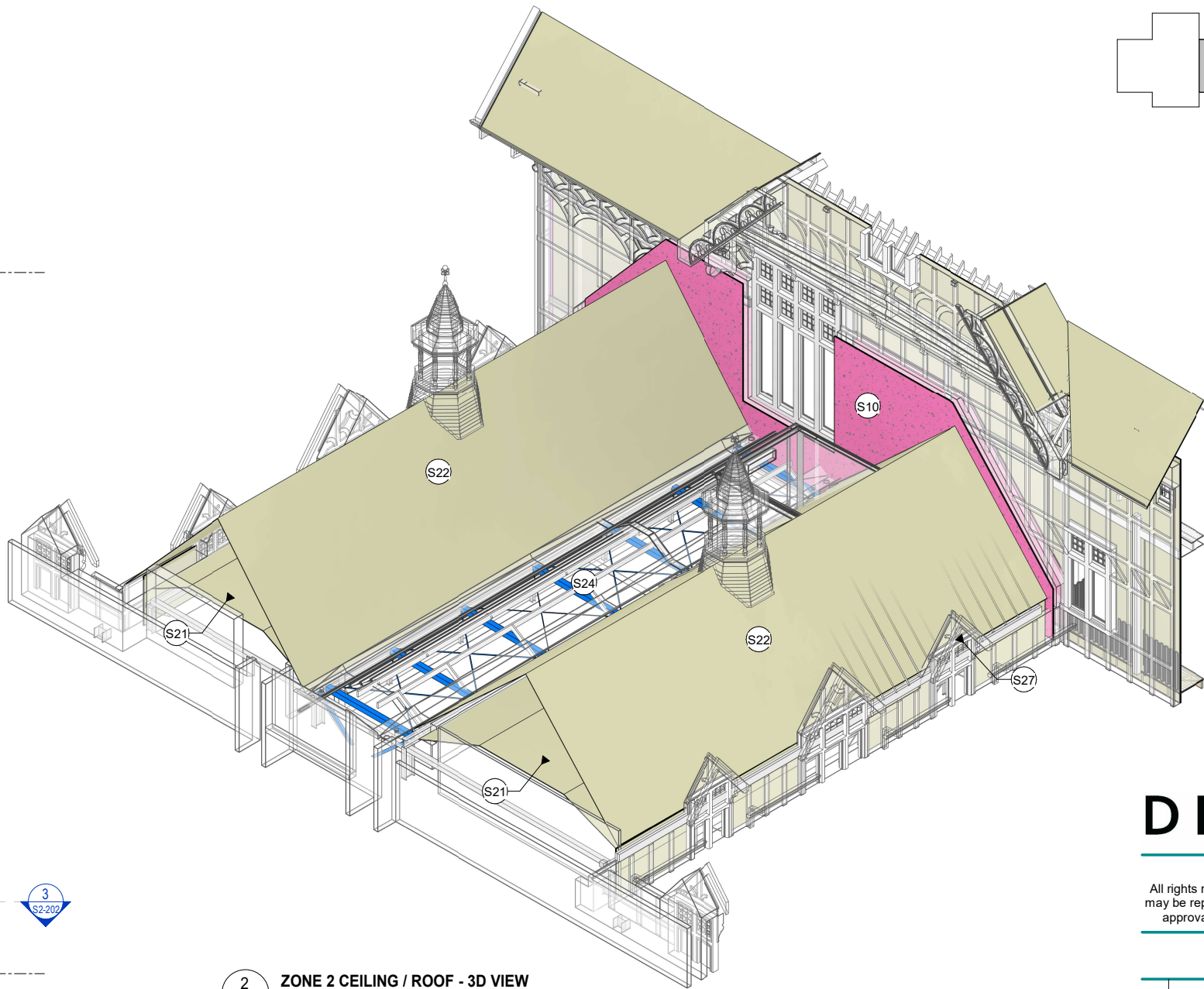
JOB No.	DWG No.	REVISION
J2187	S2-102	A



NOTE: THE CONCEPTUAL OPTIONS DESCRIBED ABOVE ARE FOR PRELIMINARY COSTING PURPOSES AND WILL NEED TO BE REFINED ON A CASE-BY-CASE BASIS. THE CONCEPTUAL OPTIONS DESCRIBED ABOVE AND THEIR REFINEMENT SHALL BE SUBJECT TO CONSTRUCTABILITY, SITE CONSTRAINTS AND QUANTITY OF PERMISSIBLE SITE HOT WORKS.



1 ZONE 2 CEILING PLAN
1:75 @ A1 A3 Scale = 2 x A1



2 ZONE 2 CEILING / ROOF - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 2 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S19	NSM CFRP STRIPS AT 1.6m MAX SPACINGS
S20	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S2-301
S21	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOISTS WITH NOGGING TO SUIT AS NEEDED
S22	12mm PLYWOOD SHEATHING OVER EXISTING SARKING, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S23	NEW STEEL FRAMING WITH CROSS BRACING TO FORM BRIDGE WALKWAY
S24	NEW HORIZONTAL STEEL TRUSS
S25	NEW 100SHS BRACING TO EXISTING VERTICAL TRUSS
S26	NEW 150PFC TO EXISTING NIB WALL AND EXISTING STEEL COLUMNS
S27	NEW STEEL BEAM FOR OOP SUPPORT

EXISTING REIDBRACE REMEDIATION NOTES:

- WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO THE SAME FLANGE ON EITHER SIDE OF THE COLUMN AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT EITHER OF THE REMEDIAL SOLUTIONS FROM BELOW:

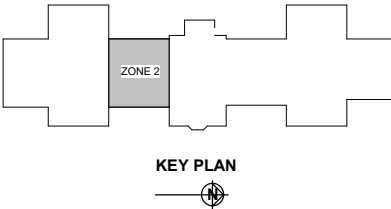
A. WELD ADDITIONAL STEEL PACKER PLATE TO COLUMN FLANGE IN ORDER TO CLOSE THE NON-COMPLIANT GAP BETWEEN THE COLUMN FLANGE AND INTERNAL FACE OF RBRACE-END;

B. FABRICATE AND BOLT CUSTOM DESIGNED STEEL GUSSET PLATE TO THE COLUMN SUCH THAT THE CONNECTION OF RBRACE-END IS CONCENTRIC TO THE COLUMN SECTION. THIS WILL SHIFT THE REID BRACES FROM COLUMN FLANGES TO CENTRE OF COLUMN WEB.
- WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO OPPOSITE FLANGES ON EITHER END OF THE COLUMN WEB AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT THE FOLLOWING REMEDIAL SOLUTION:

FABRICATE AND BOLT CUSTOM DESIGNED STEEL GUSSET PLATE TO THE COLUMN SUCH THAT THE CONNECTION OF RBRACE-END IS CONCENTRIC TO THE COLUMN SECTION. THIS WILL SHIFT THE REID BRACES FROM COLUMN FLANGES TO CENTRE OF COLUMN WEB.
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A. WELD ADDITIONAL STEEL PACKER PLATE TO COLUMN FLANGE IN ORDER TO CLOSE THE NON-COMPLIANT GAP BETWEEN THE COLUMN FLANGE AND INTERNAL FACE OF RBRACE-END;
B. ADD ADDITIONAL STEEL STIFFENER PLATES TO COLUMN AND REVIEW TORSIONAL CONNECTION WITH THE CONNECTING BEAMS. IF THE TORSIONAL CONNECTION DOES NOT EXIST, BOLT ADDITIONAL STEEL BRACKETS FROM BEAM TO COLUMN.

NOTE: THE CONCEPTUAL OPTIONS DESCRIBED ABOVE ARE FOR PRELIMINARY COSTING PURPOSES AND WILL NEED TO BE REFINED ON A CASE-BY-CASE BASIS. THE CONCEPTUAL OPTIONS DESCRIBED ABOVE AND THEIR REFINEMENT SHALL BE SUBJECT TO CONSTRUCTABILITY, SITE CONSTRAINTS AND QUANTITY OF PERMISSIBLE SITE HOT WORKS.



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CONCEPT

A	29.11.21	CONCEPT DESIGN ISSUE	DD
Rev	Date	Description	App

CLIENT
ROTORUA LAKES COUNCIL
Te Kaunihera o ngāi Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 2

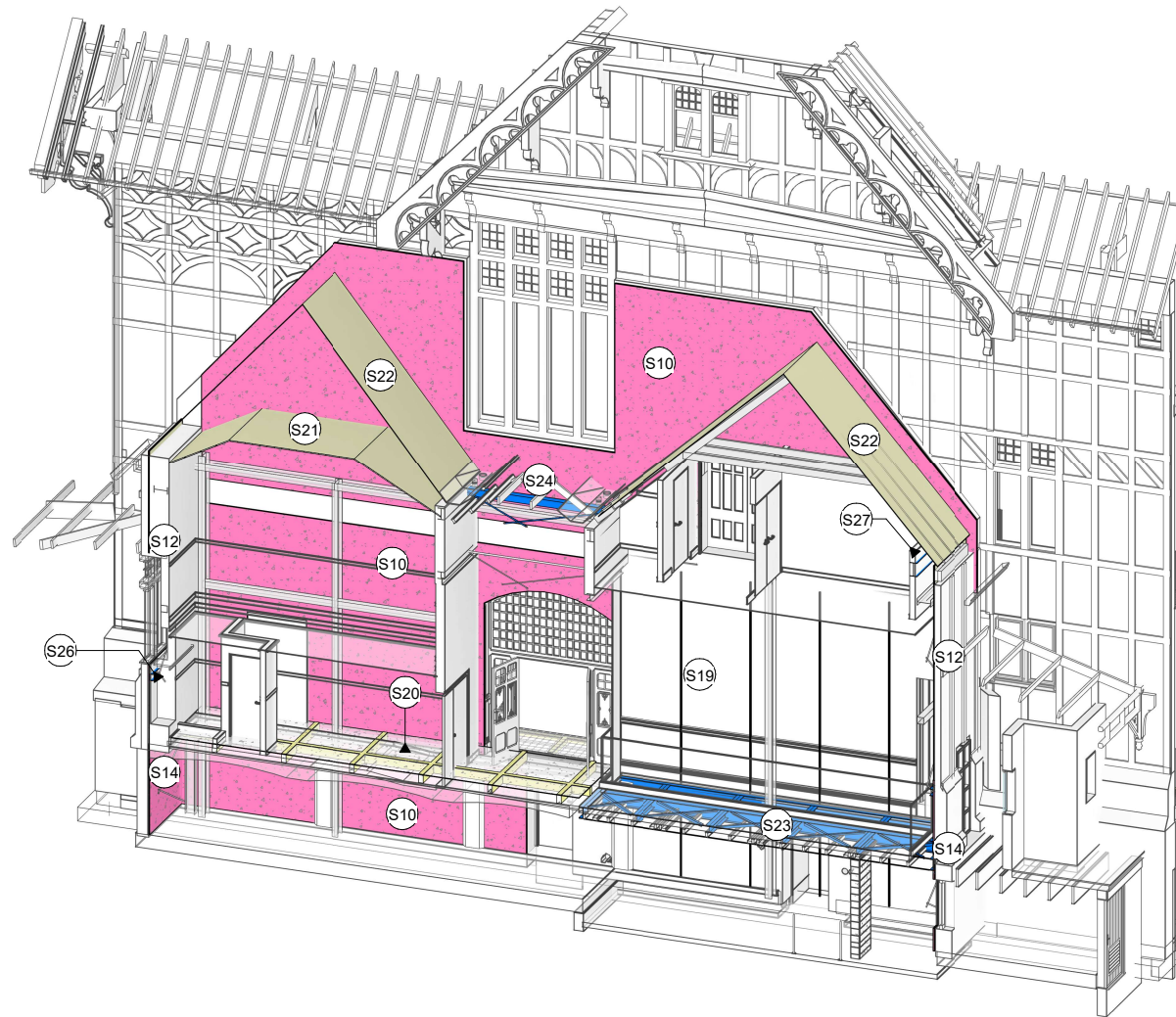
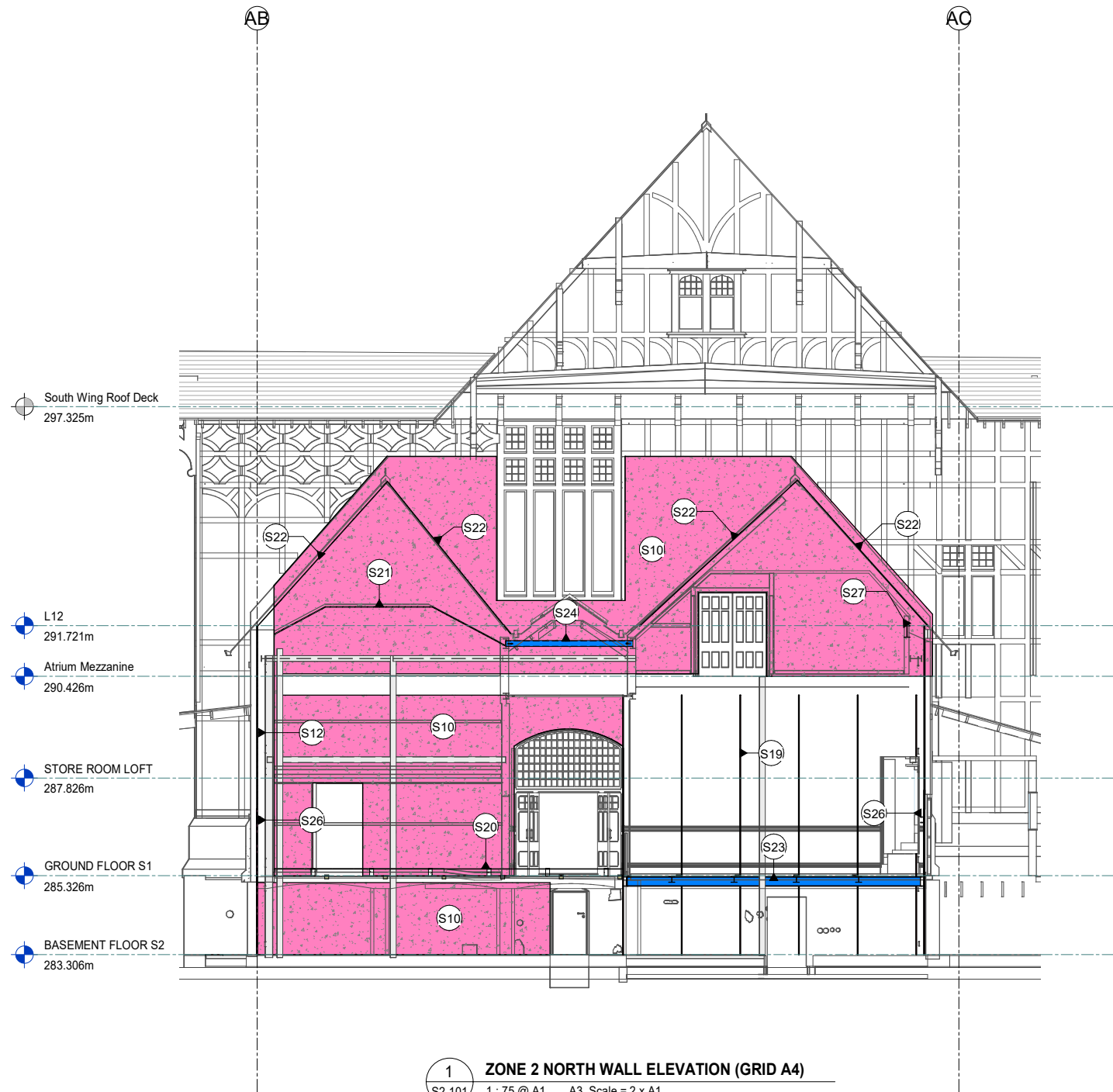
CEILING PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE
1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S2-103	A



2 ZONE 2 NORTH WALL ELEVATION (GRID A4) - 3D VIEW
@ A1 A3 Scale = 2 x A1

EXISTING REIDBRACE REMEDIATION NOTES:

- WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO THE SAME FLANGE ON EITHER SIDE OF THE COLUMN AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT EITHER OF THE REMEDIAL SOLUTIONS FROM BELOW:

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B. FABRICATE AND BOLT CUSTOM DESIGNED STEEL GUSSET PLATE TO THE COLUMN SUCH THAT THE CONNECTION OF RBRACE-END IS CONCENTRIC TO THE COLUMN SECTION. THIS WILL SHIFT THE REID BRACES FROM COLUMN FLANGES TO CENTRE OF COLUMN WEB.
- WHERE THE RBRACE-END ON EITHER SIDE OF THE COLUMN CONNECTS TO OPPOSITE FLANGES ON EITHER END OF THE COLUMN WEB AND THE CONNECTION IS CLOSE TO THE COLUMN BASEPLATE, ADOPT THE FOLLOWING REMEDIAL SOLUTION:

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ZONE 2 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S19	NSM CFRP STRIPS AT 1.6m MAX SPACINGS
S20	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S2-301
S21	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOISTS WITH NOGGING TO SUIT AS NEEDED
S22	12mm PLYWOOD SHEATHING OVER EXISTING SARKING, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S23	NEW STEEL FRAMING WITH CROSS BRACING TO FORM BRIDGE WALKWAY
S24	NEW HORIZONTAL STEEL TRUSS
S25	NEW 100SHS BRACING TO EXISTING VERTICAL TRUSS
S26	NEW 150PFC TO EXISTING NIB WALL AND EXISTING STEEL COLUMNS
S27	NEW STEEL BEAM FOR OOP SUPPORT

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CONCEPT

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CLIENT
ROTORUA LAKES COUNCIL
Te Kaunihera o ngāi Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 2

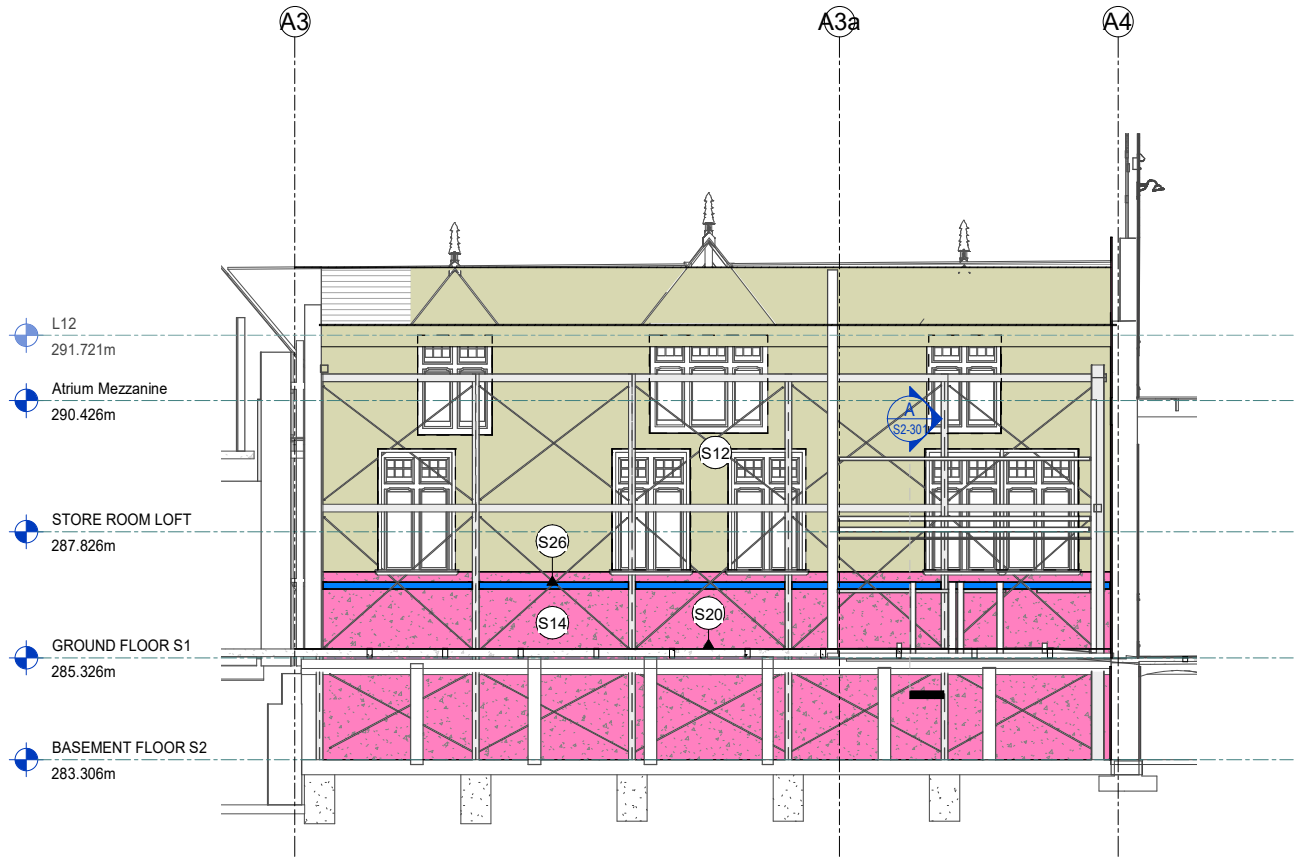
SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE
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1:150 @ A3

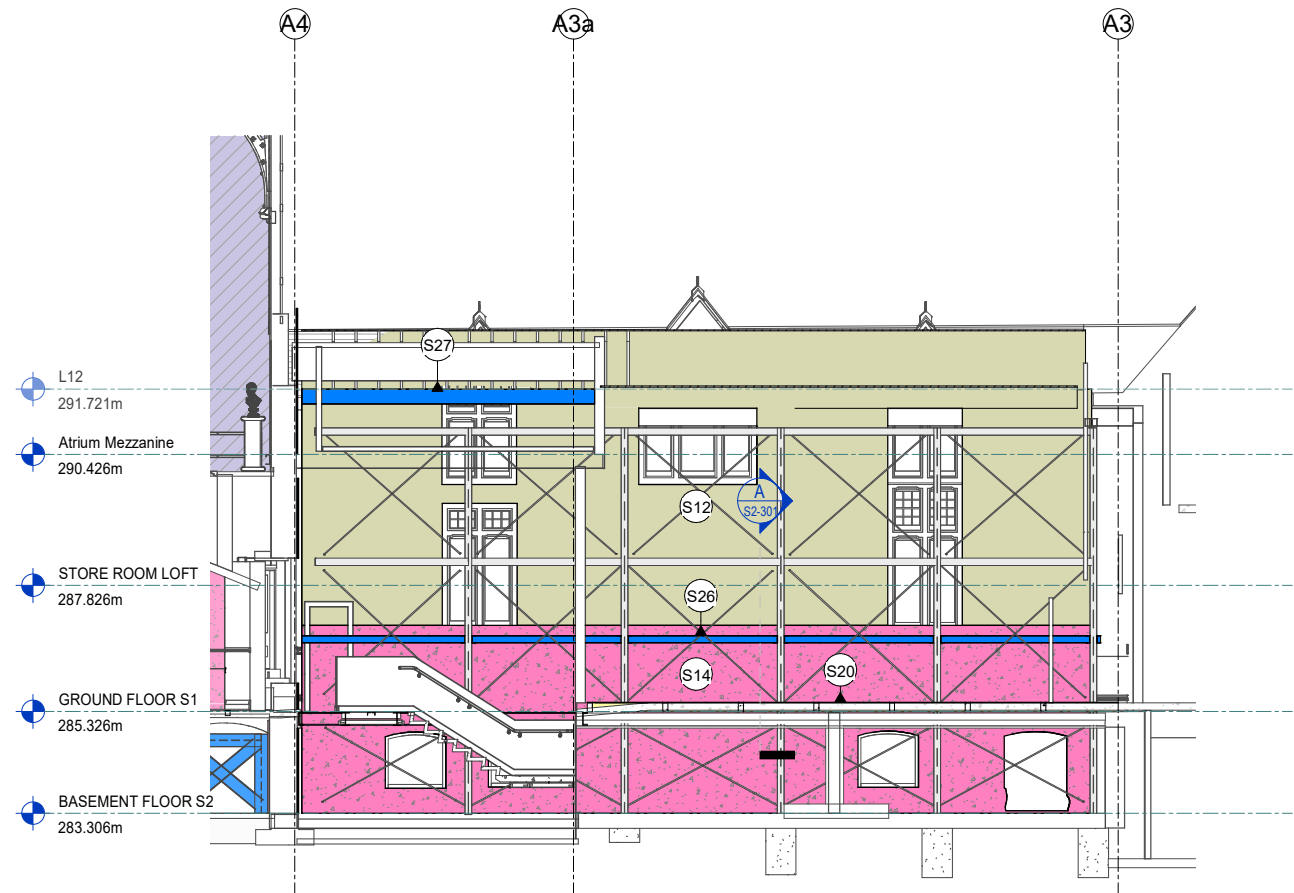
JOB No.	DWG No.	REVISION
J2187	S2-201	A



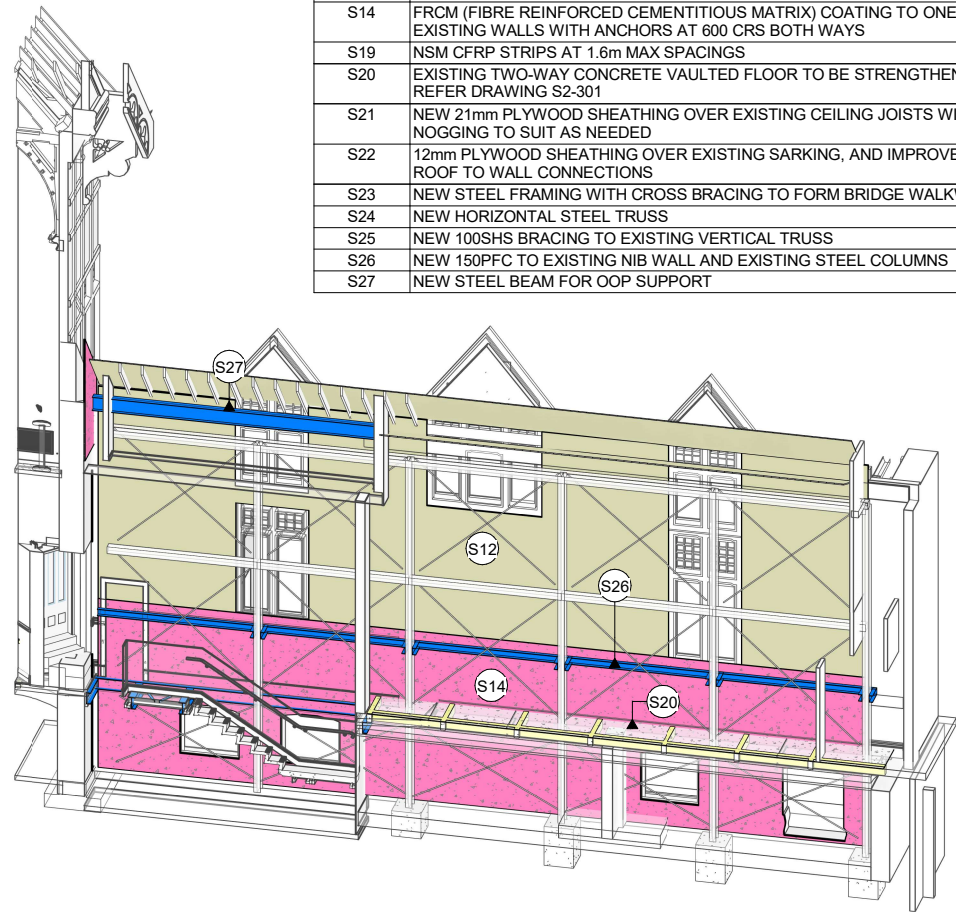
2 ZONE 2 WEST WALL ELEVATION (GRID AB)
S2-101 1: 75 @ A1 A3 Scale = 2 x A1



2A ZONE 2 WEST WALL ELEVATION (GRID AB) - 3D VIEW
@ A1 A3 Scale = 2 x A1



3 ZONE 2 EAST WALL ELEVATION (GRID AC)
S2-101 1: 75 @ A1 A3 Scale = 2 x A1



3A ZONE 2 EAST WALL ELEVATION (GRID AC) - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 2 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
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S24	NEW HORIZONTAL STEEL TRUSS
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S26	NEW 150PFC TO EXISTING NIB WALL AND EXISTING STEEL COLUMNS
S27	NEW STEEL BEAM FOR OOP SUPPORT

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CONCEPT

A	29.11.21	CONCEPT DESIGN ISSUE	DD
Rev	Date	Description	App

CLIENT

**ROTORUA
LAKES COUNCIL**
Te Kaitiaki o ngā Roto o Rotorua

PROJECT

**ROTORUA MUSEUM
DEVELOPMENT**

SHEET TITLE

ZONE 2

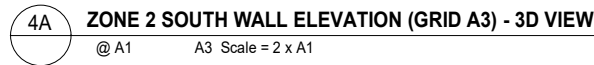
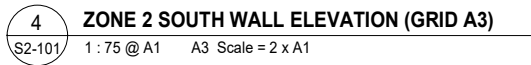
SECTIONS

**PRELIMINARY CONCEPT
80% NBS (IL3)**

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE 1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S2-202	A



EXISTING REIDBRACE REMEDIATION NOTES:

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ROTORUA
LAKES COUNCIL
Te Kaurihera o ngāi Roto o Rotorua

PROJECT

ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE

ZONE 2

SECTIONS

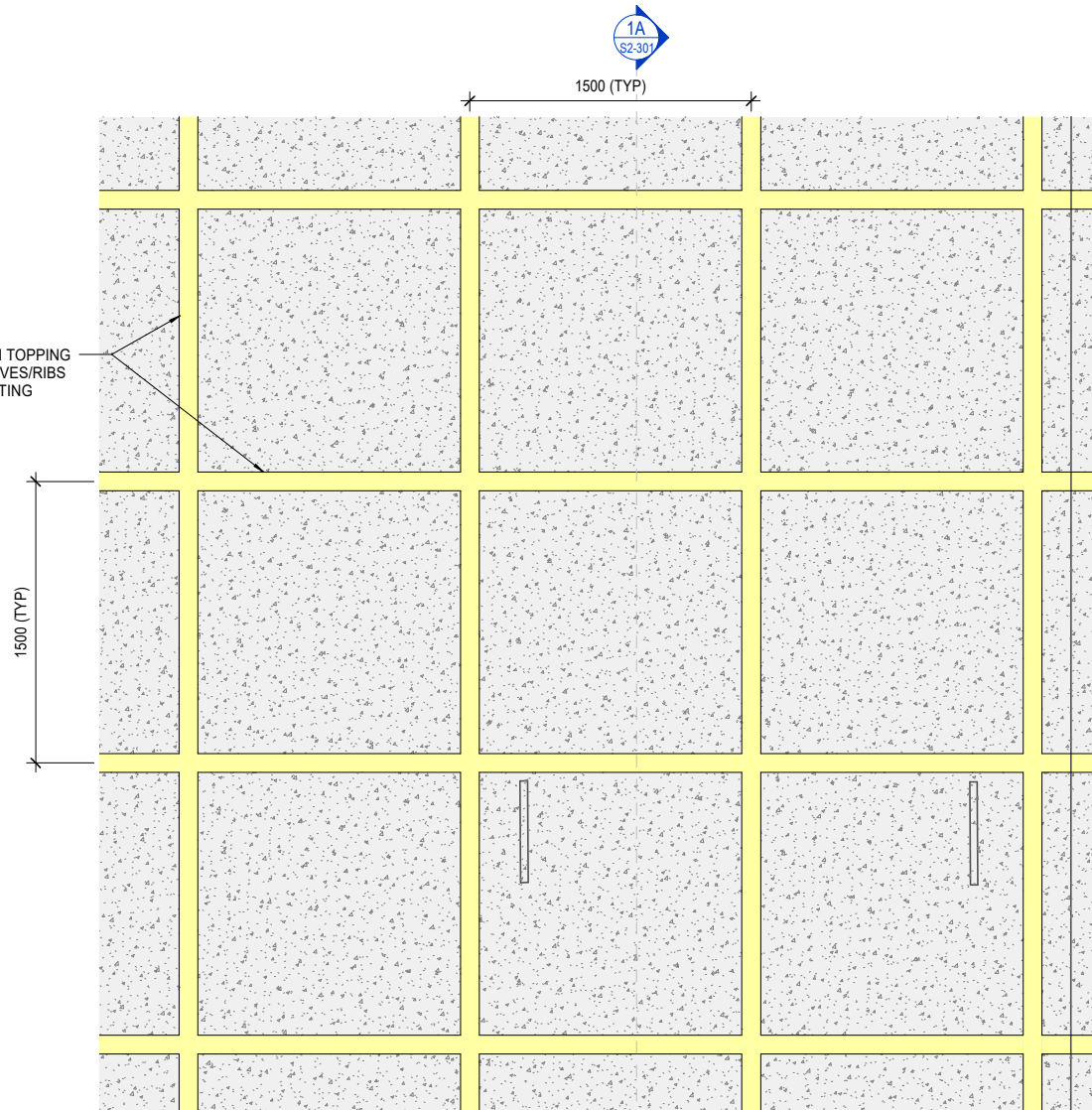
PRELIMINARY CONCEPT
80% NBS (IL3)

SCALE 1:75 @ A1

1:150 @ A3

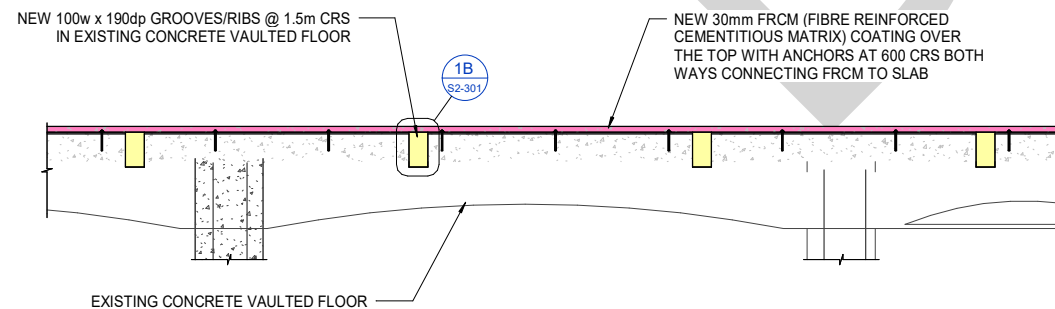
JOB No.	DWG No.	REVISION
J2187	S2-203	A

NEW 100w x 190dp (I.E. THROUGH TOPPING AND INTO VAULTED SLAB) GROOVES/RIBS @ 1.5m CRS BOTH WAYS IN EXISTING CONCRETE VAULTED FLOOR

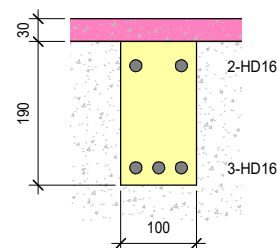


NOTE:
EXISTING CONCRETE TO BE CAREFULLY REMOVED UNDER EXISTING INTERNAL WALLS TO ENSURE RIB CONTINUITY

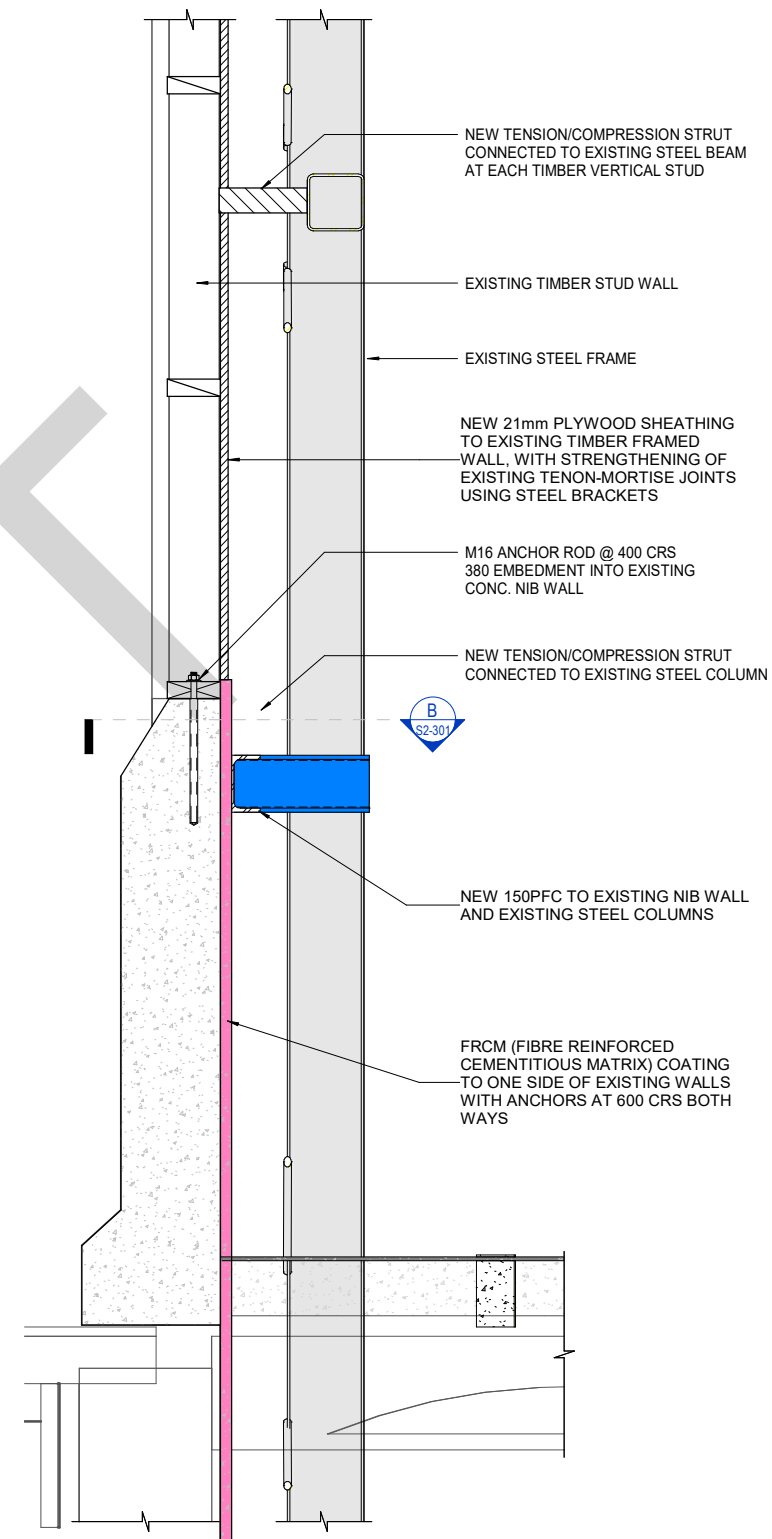
1 EXISTING CONCRETE VAULTED FLOOR STRENGTHENING - PLAN
1 : 20 @ A1 A3 Scale = 2 x A1



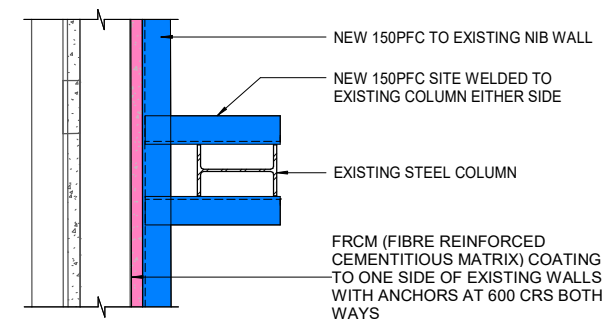
1A SECTION 1A
S2-301 1 : 20 @ A1 A3 Scale = 2 x A1



1B RIB REINFORCING
S2-301 1 : 5 @ A1 A3 Scale = 2 x A1



A SECTION A
S2-101 1 : 10 @ A1 A3 Scale = 2 x A1



B SECTION B
S2-301 1 : 10 @ A1 A3 Scale = 2 x A1

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ROTORUA LAKES COUNCIL
Te Kaunihera o ngā Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 2

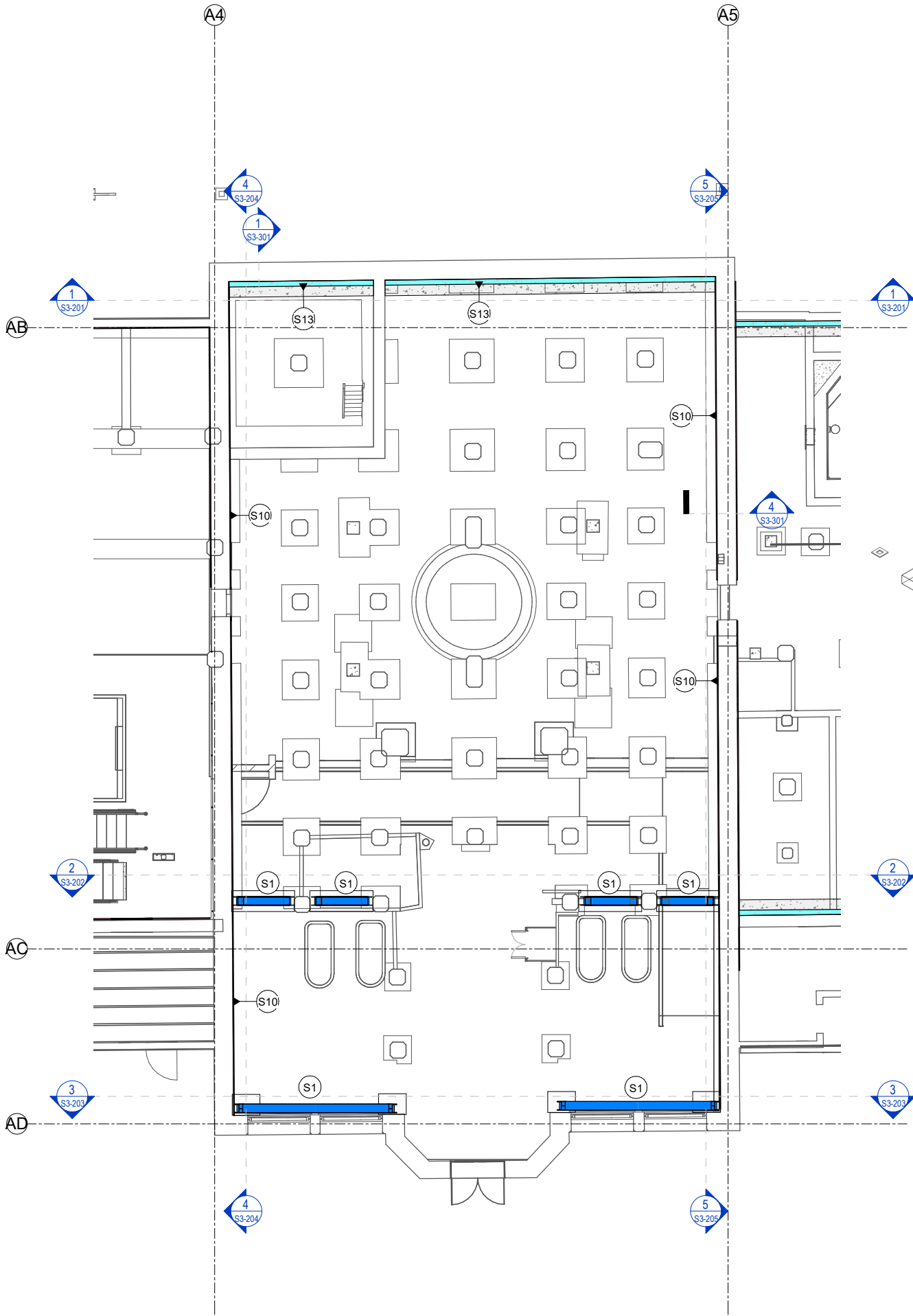
DETAILS

PRELIMINARY CONCEPT
80% NBS (IL3)

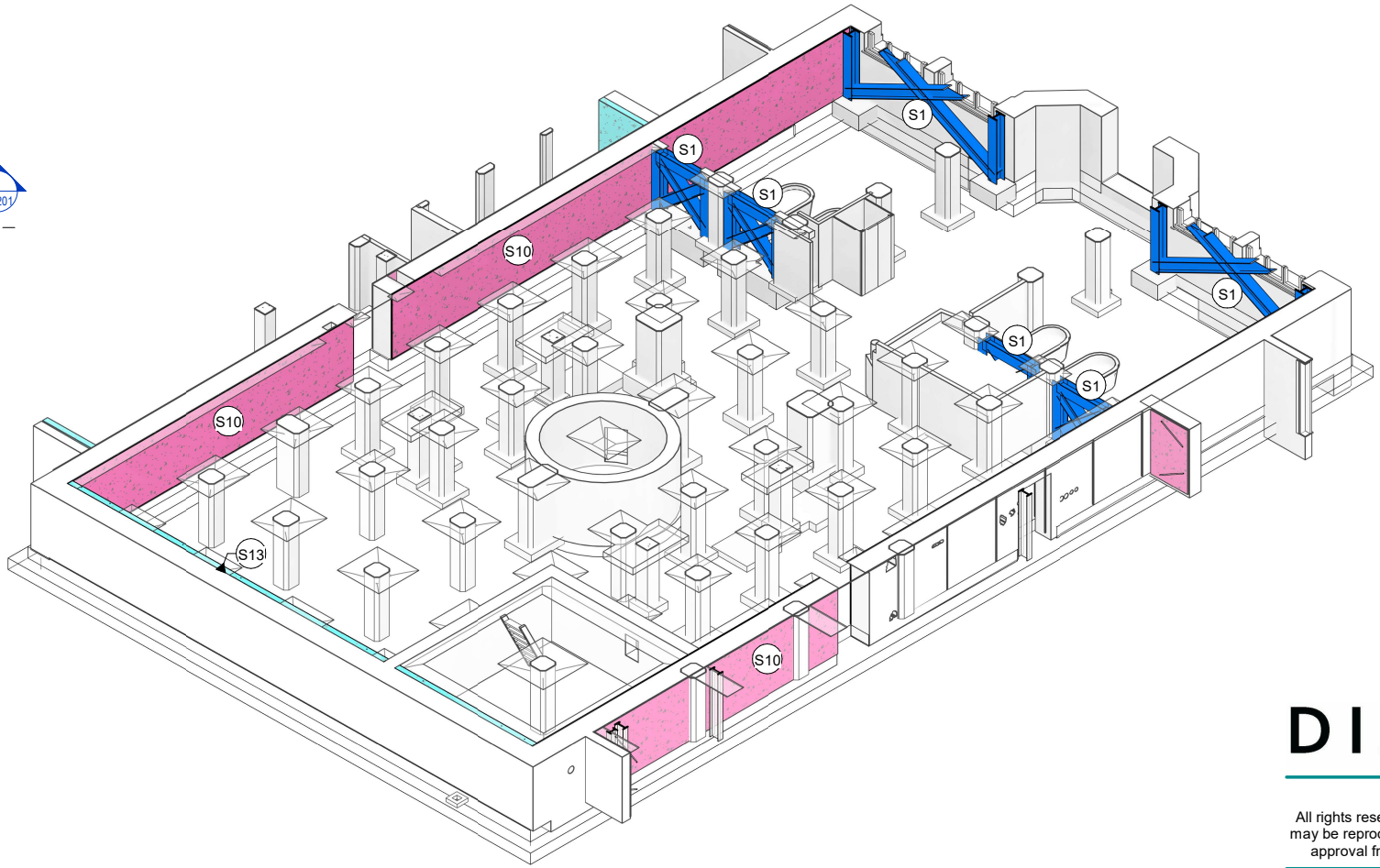
DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE
1:10 @ A1
1:20 @ A3

JOB No.	DWG No.	REVISION
J2187	S2-301	A

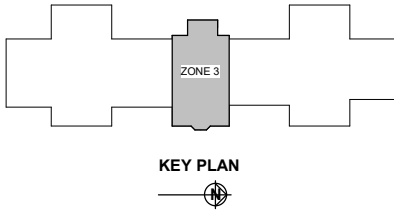


1 ZONE 3 FOUNDATION PLAN
1 : 75 @ A1 A3 Scale = 2 x A1



2 FOUNDATION - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE.
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 150 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS



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CONCEPT

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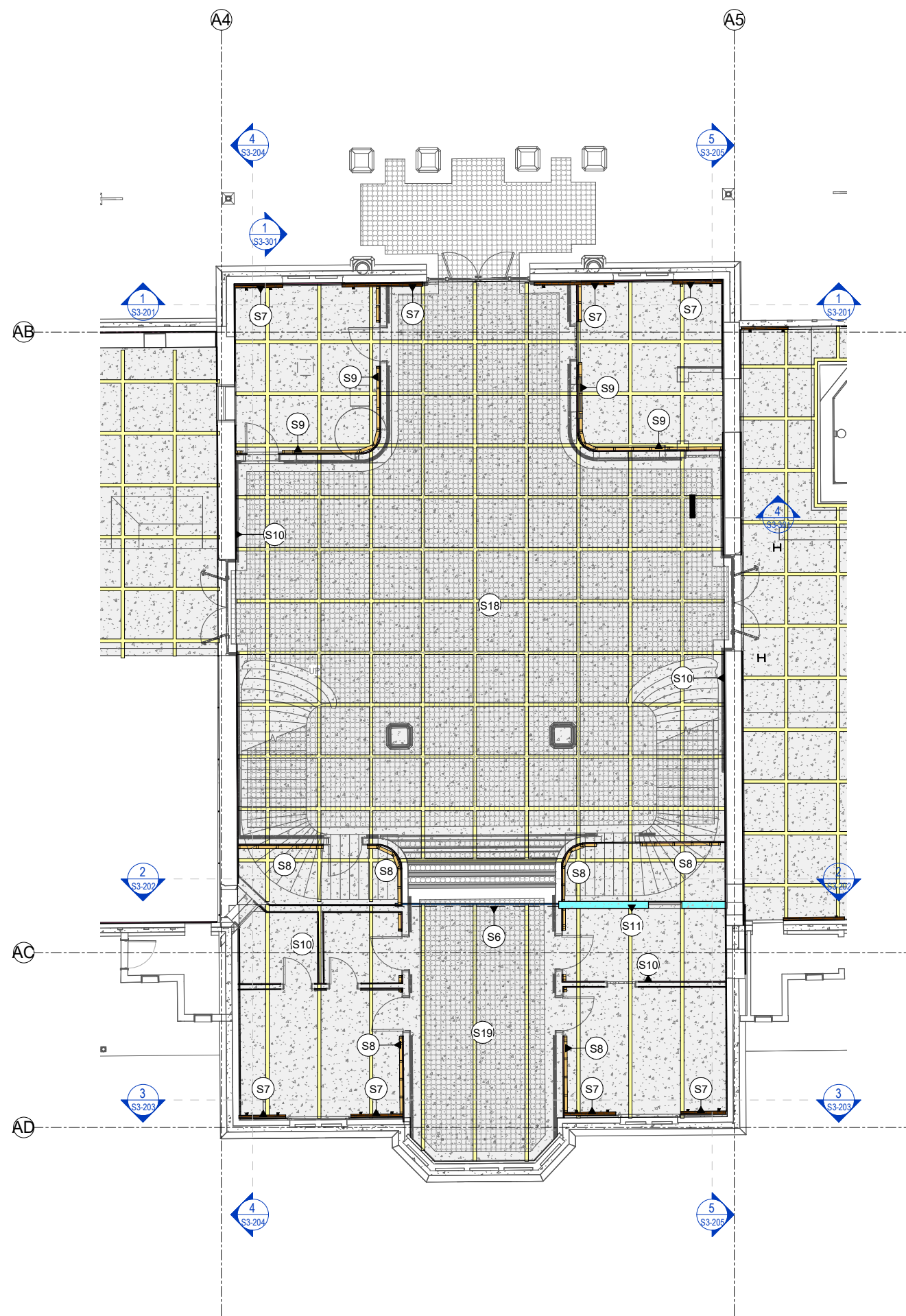
PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 3

FOUNDATION PLAN

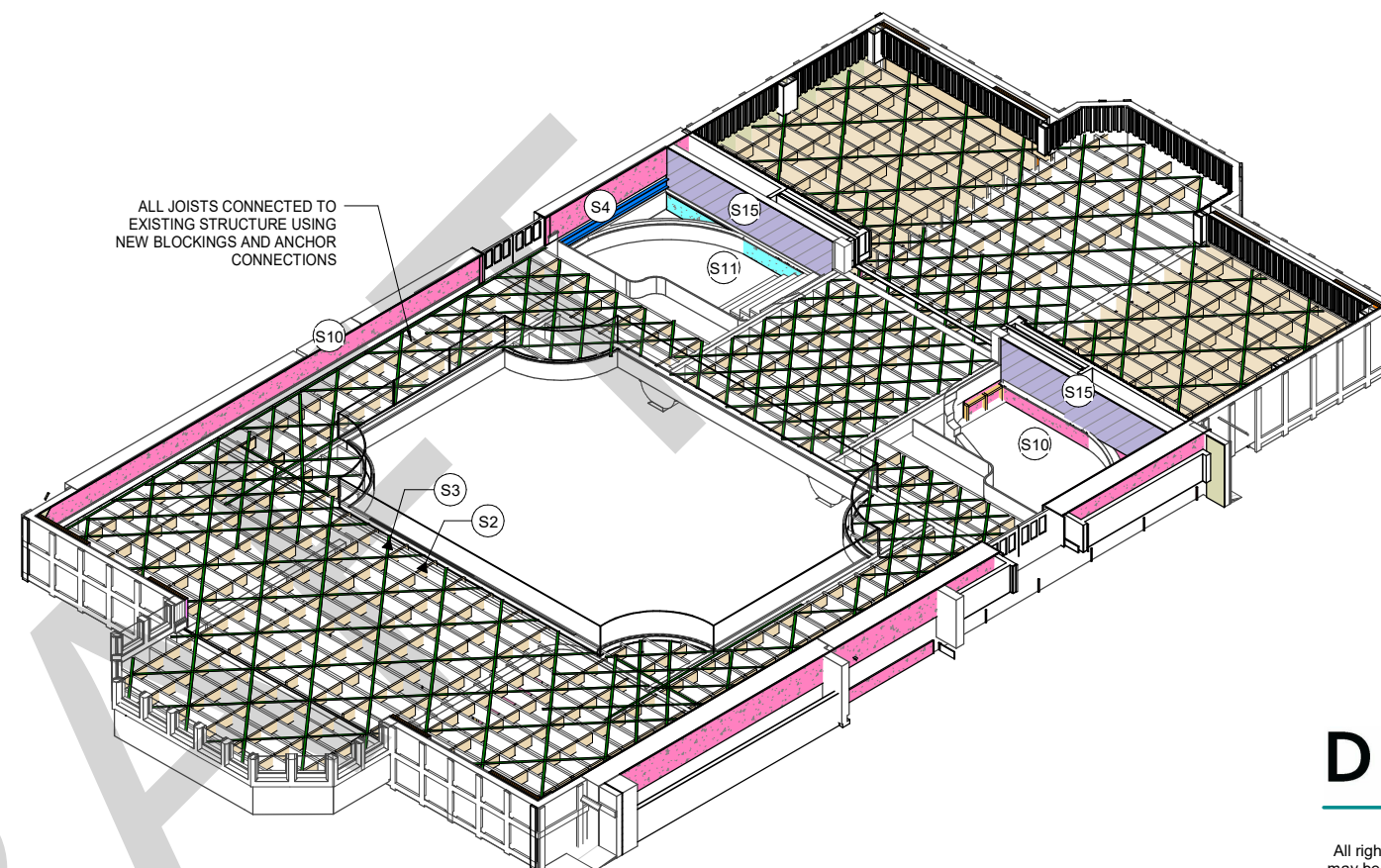
PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD
SCALE	1:75 @ A1 1:150 @ A3		
JOB No.	DWG No.	REVISION	
J2187	S3-101	A	



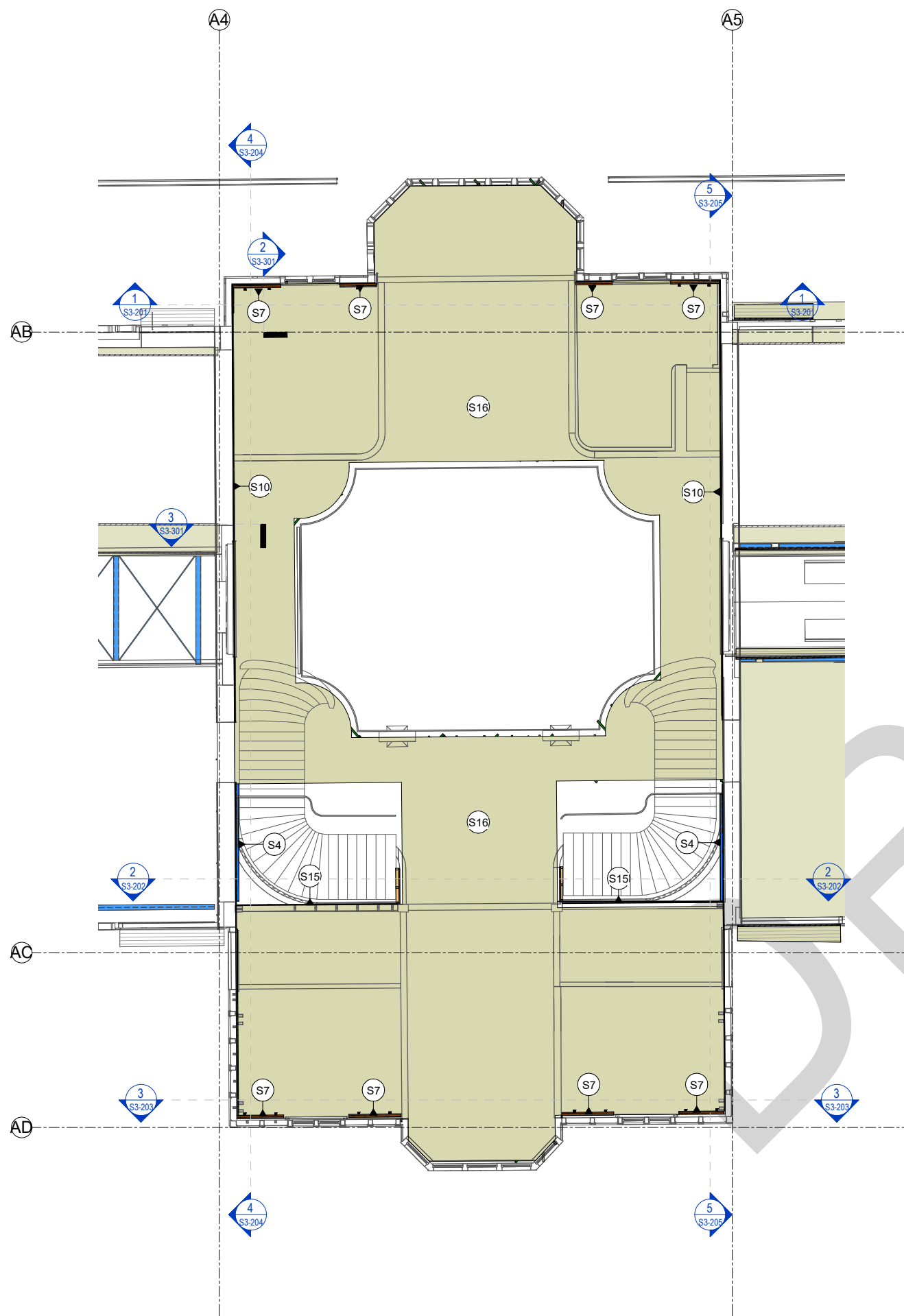
ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
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S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
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S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

JOB No.	DWG No.	REVISION
J2187	S3-102	A



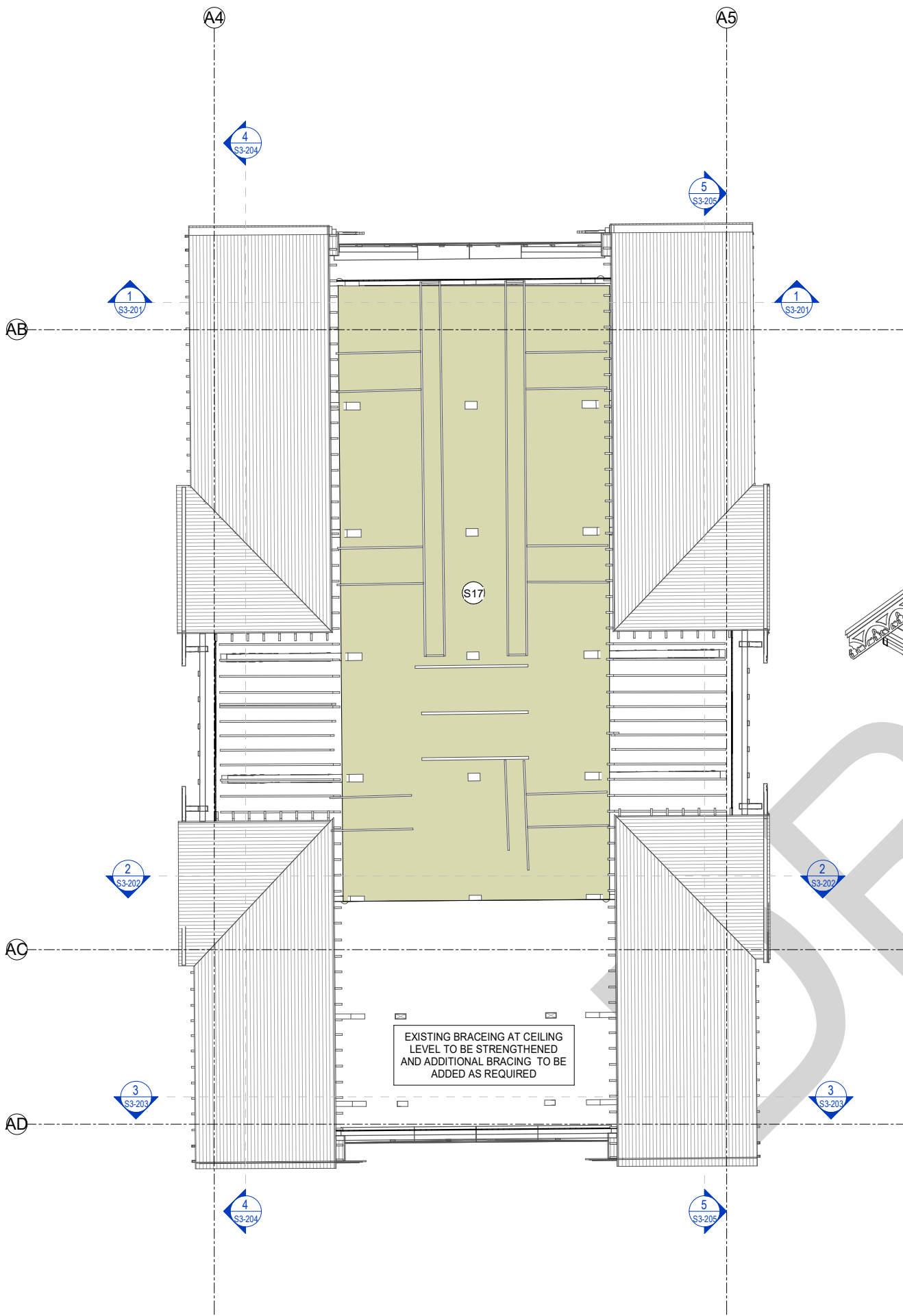
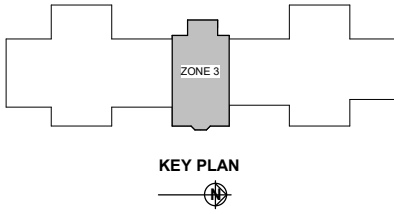
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S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 15 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

JOB No.	DWG No.	REVISION
J2187	S3-103	A

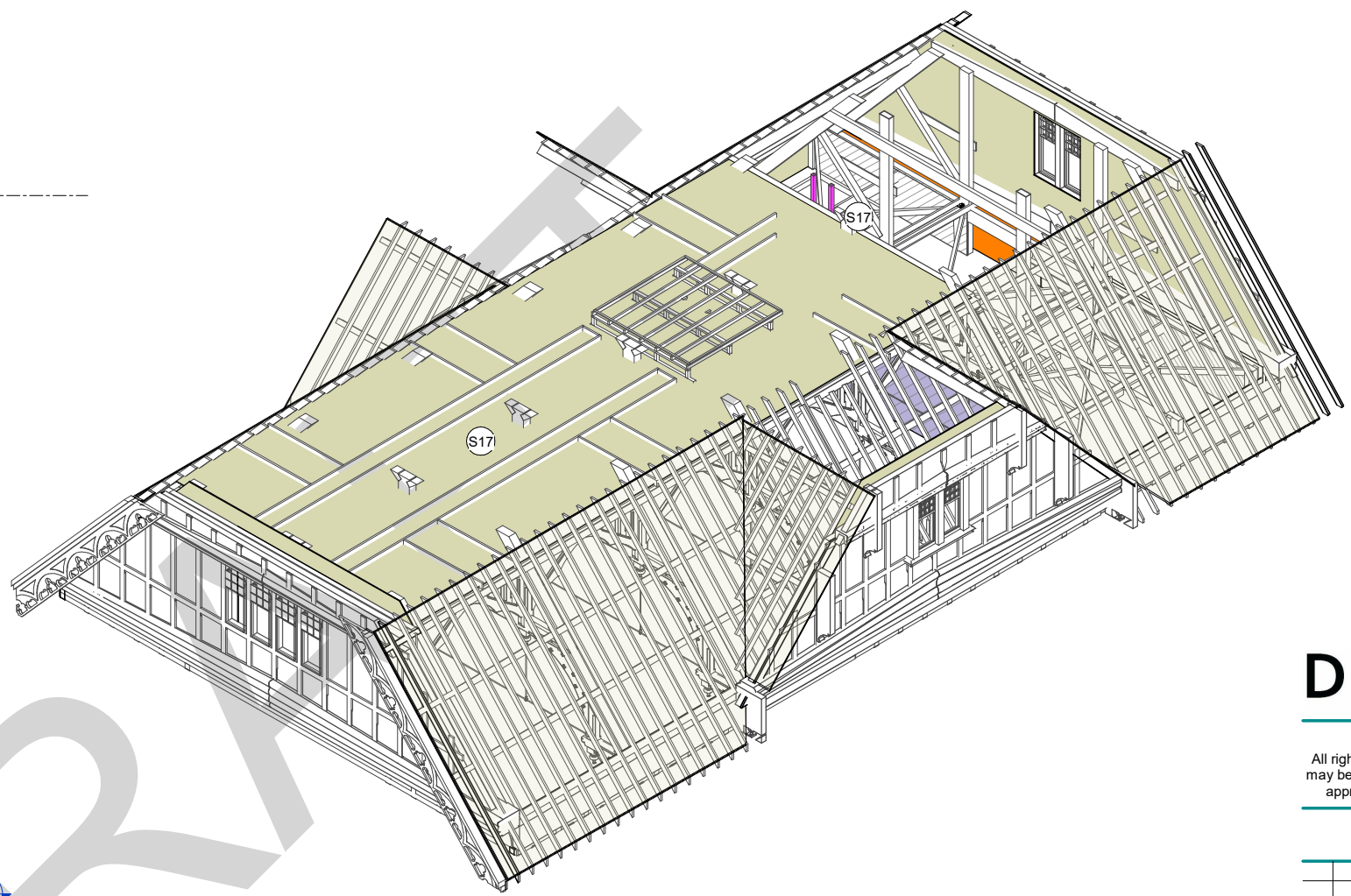


ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m ² OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 15 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

JOB No.	DWG No.	REVISION
J2187	S3-104	A



1
S3-201
1 : 75 @ A1 A3 Scale = 2 x A1



2
CEILING PLAN - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE.
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 150 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
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S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

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CONCEPT

Rev	Date	Description	App
A	23.11.21	CONCEPT DESIGN ISSUE	DD



PROJECT
ROTORUA MUSEUM DEVELOPMENT

ZONE 3

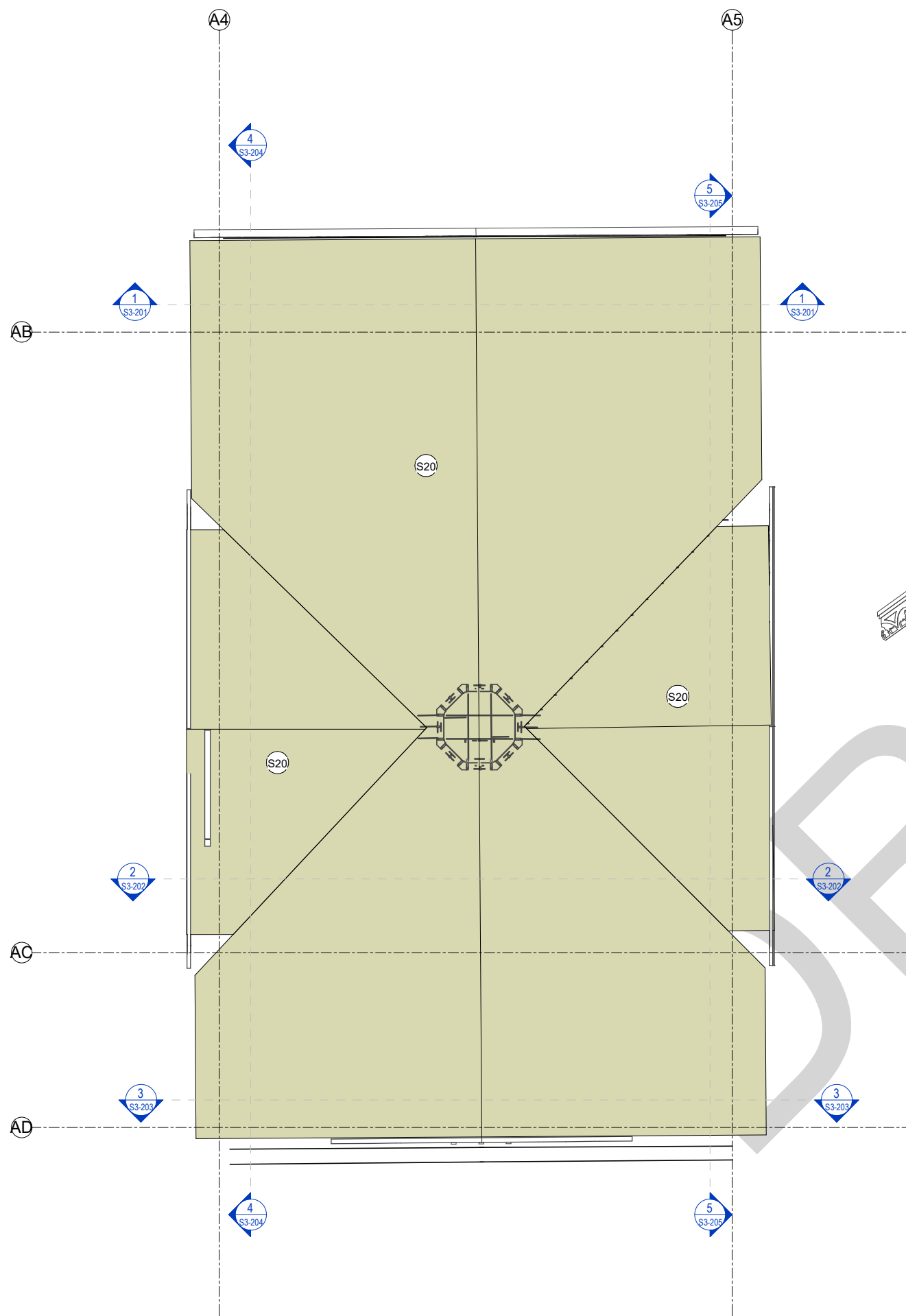
CEILING PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

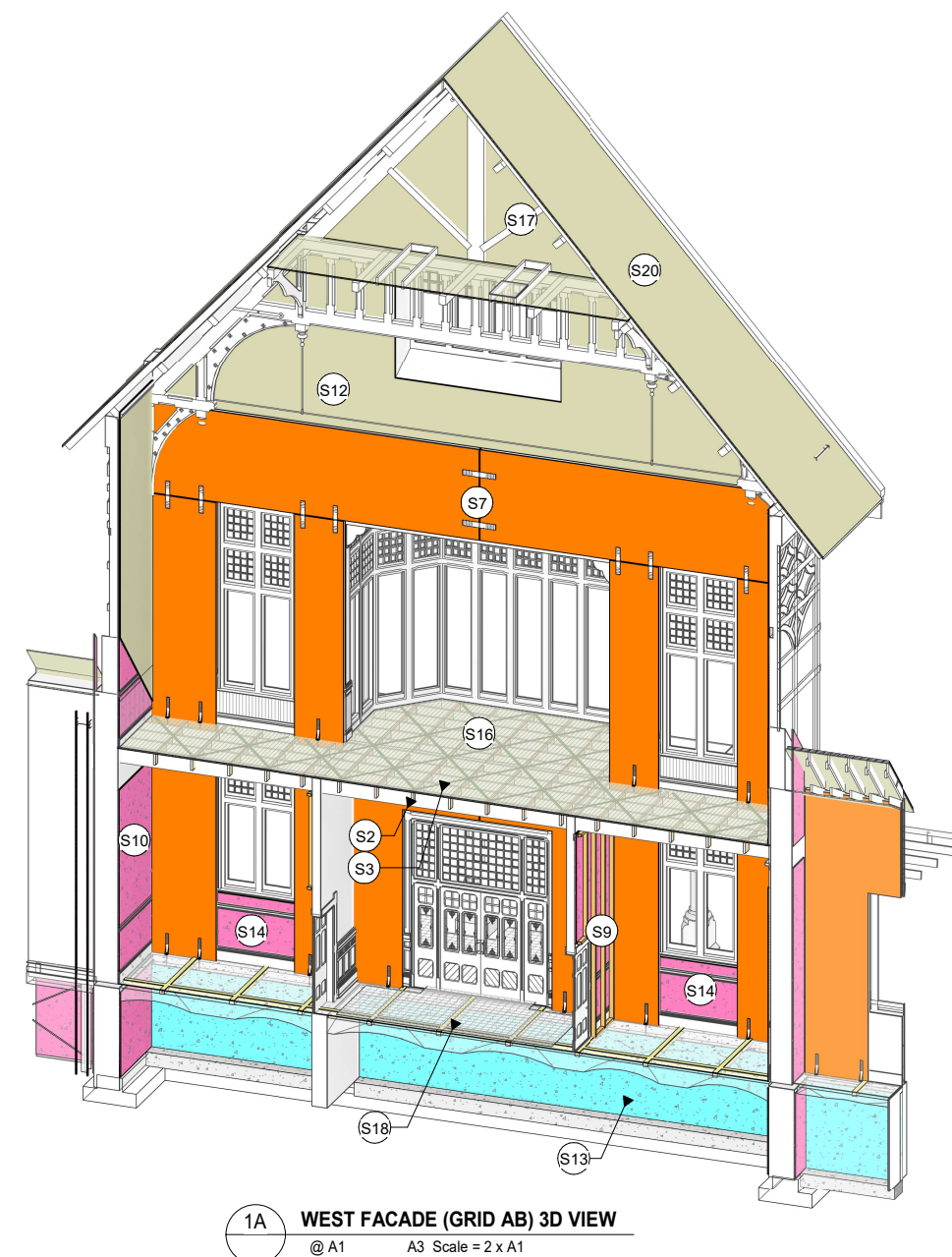
SCALE
1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S3-105	A



ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 Sg8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 Sg8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 Sg8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m ³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
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S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

JOB No.	DWG No.	REVISION
J2187	S3-106	A



ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE.
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
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S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
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S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
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S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

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Rev	Date	Description	App

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PROJECT

ROTORUA MUSEUM
DEVELOPMENT

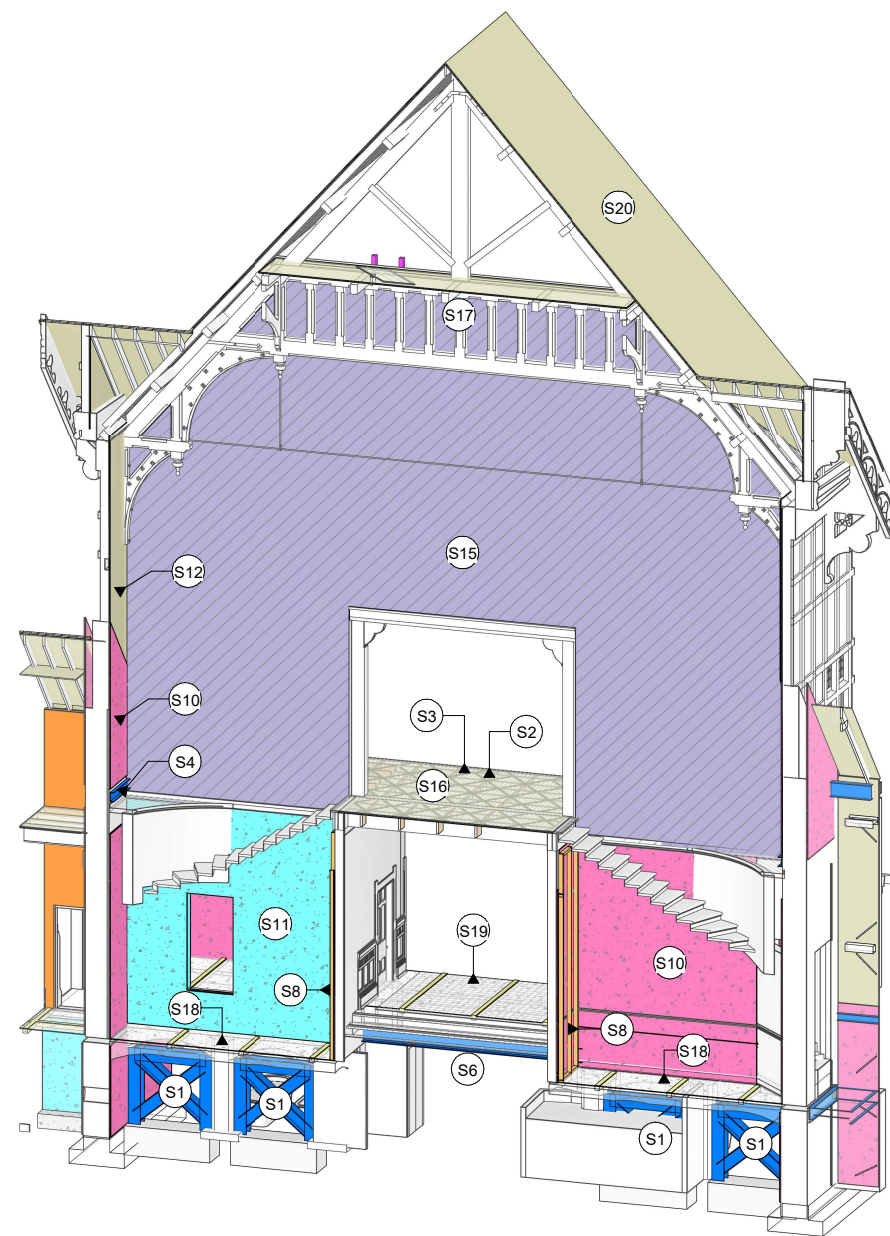
SHEET TITLE

ZONE 3

SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD
SCALE			
1:50 @ A1			
1:100 @ A3			
JOB No.	DWG No.	REVISION	
J2187	S3-201	A	



2A MID SHEAR WALL- 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE.
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 150 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS

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PROJECT

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DEVELOPMENT

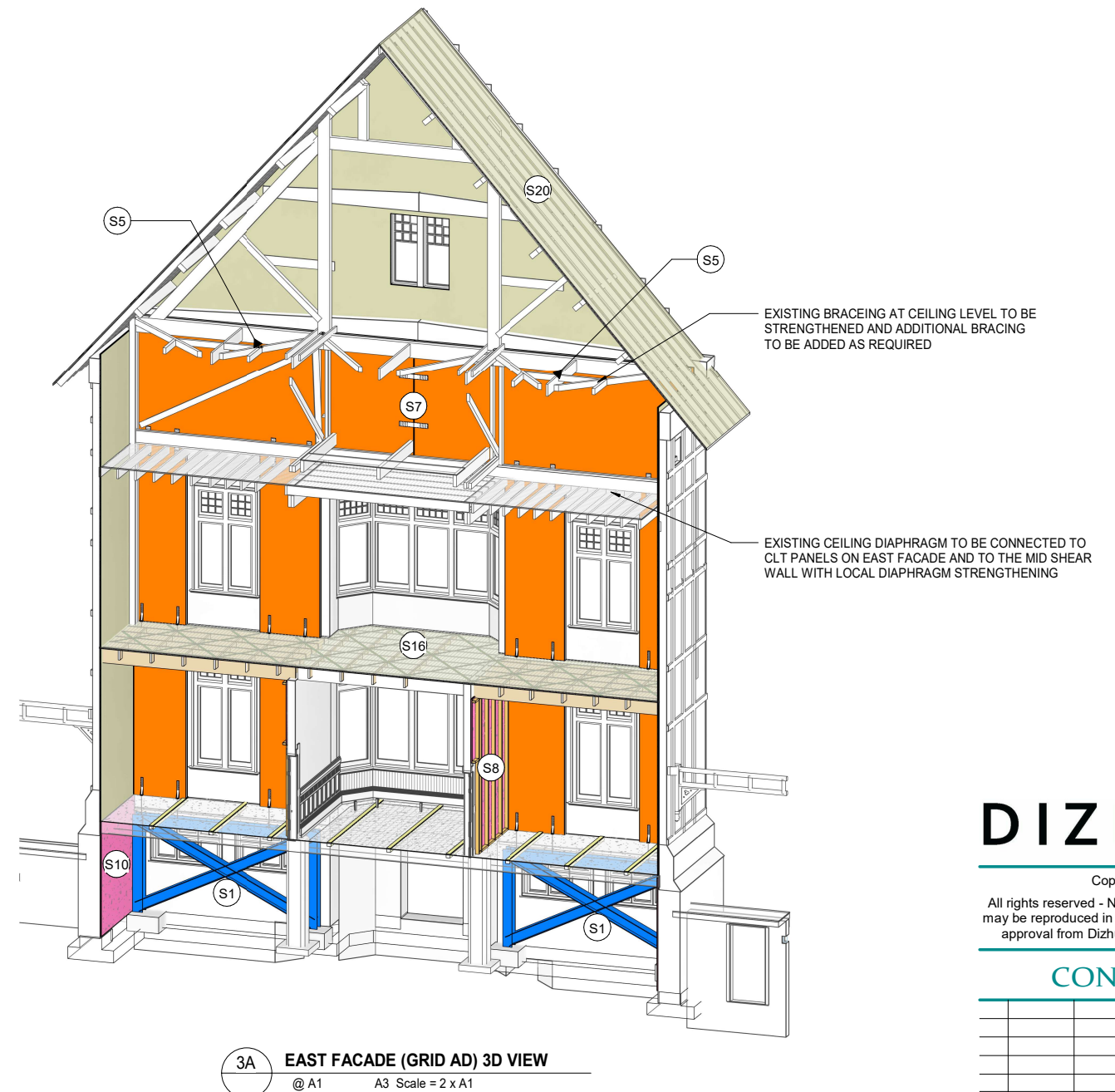
SHEET TITLE

ZONE 3

SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD
SCALE 1:50 @ A1 1:100 @ A3			
JOB No.	DWG No.	REVISION	
J2187	S3-202	A	



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A	23.11.21	CONCEPT DESIGN ISSUE	DD
Rev	Date	Description	App

PROJECT

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DEVELOPMENT

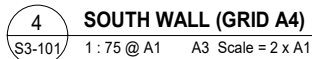
SECTIONS

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE 1:50 @ A1
 1:100 @ A3

JOB No.	DWG No.	REVISION
J2187	S3-203	A

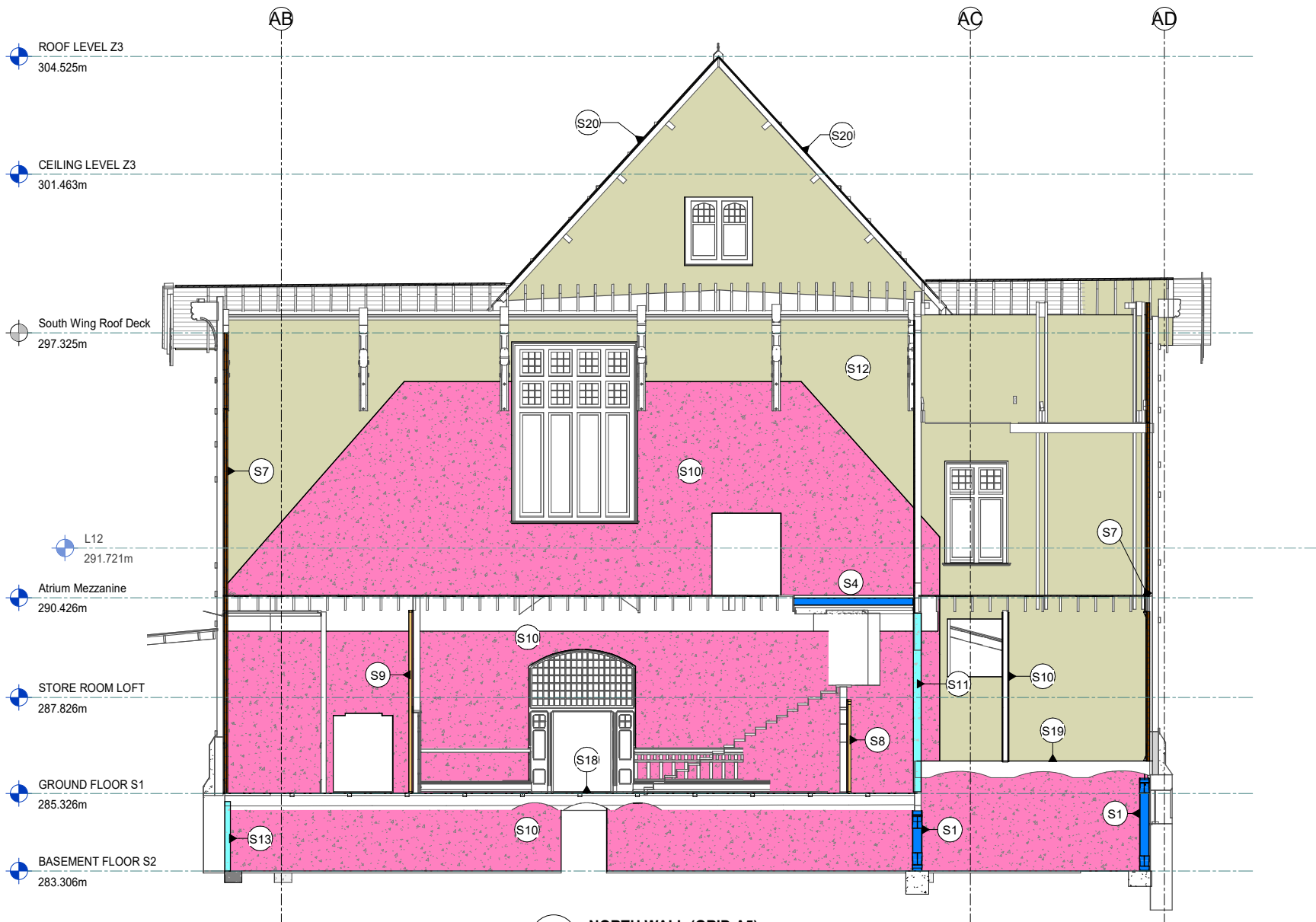
ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
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S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS



4A SOUTH WALL (GRID A4) 3D VIEW

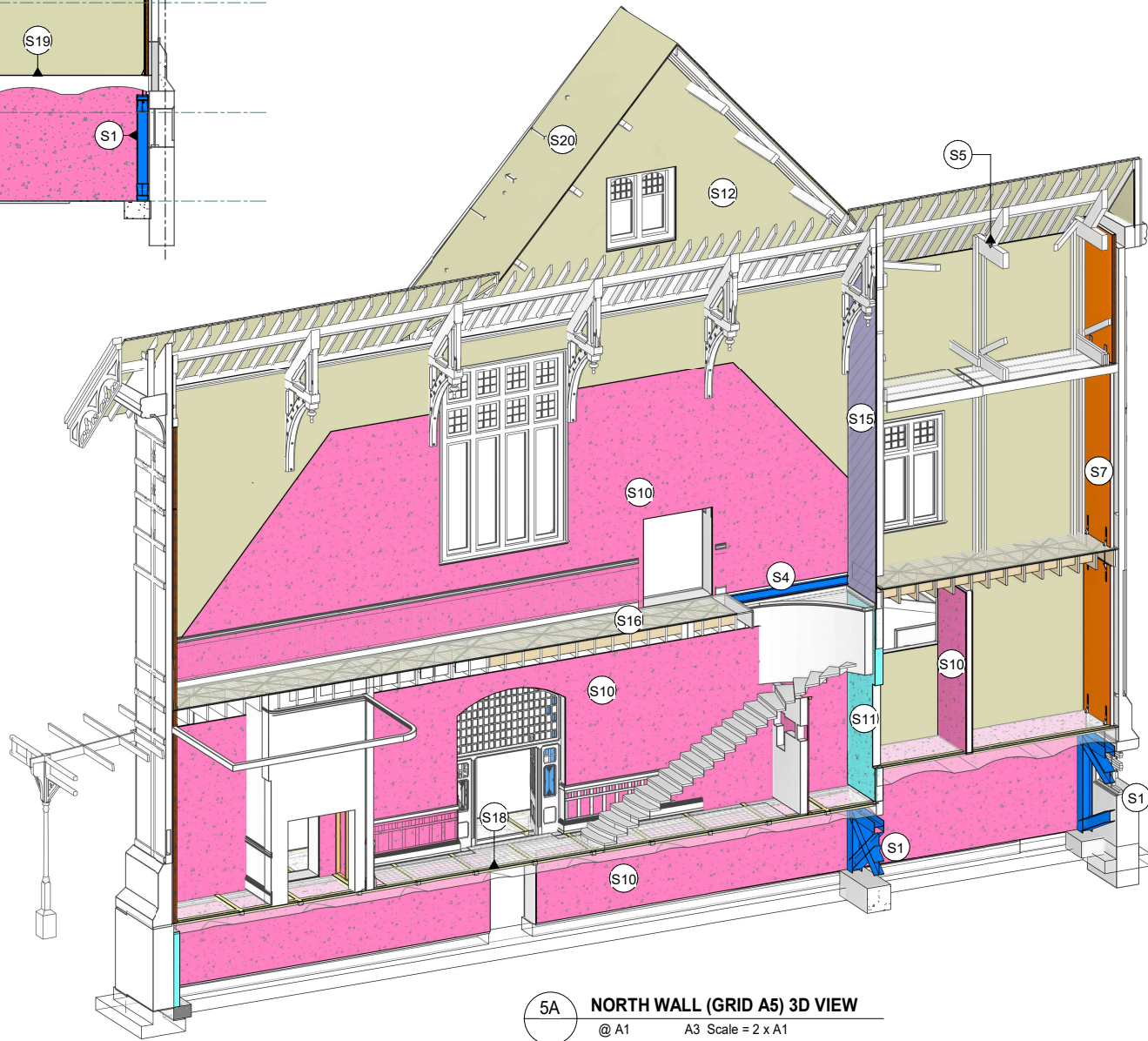
4A SOUTH WALL (GRID A4) 3D VIEW
@ A1 A3 Scale = 2 x A1

JOB No.	DWG No.	REVISION
J2187	S3-204	A



5 NORTH WALL (GRID A5)
S3-101 1:75 @ A1 A3 Scale = 2 x A1

ZONE 3 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S2	NEW 240x45 SG8 BLOCKINGS @ 800 CRS
S3	NEW STRAP BRACING OVER EXISTING FLOOR JOISTS
S4	NEW PFC STEEL SECTION AT STAIR OPENINGS
S5	ALL EXISTING TIMBER CONNECTIONS IN THIS AREA ARE TO BE STRNGTHENED WITH NAILON PLATES OR SIMILAR. SPECIFIC CONNECTION TO BE DESIGNED DURING DETAILED DESIGN PHASE.
S6	NEW 300PFC DRAG BEAM
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS
S8	SINGLE SIDED FRCM + TIMBER STRONGBACK - 1/90x45 SG8 STUDS @ 600 CRS
S9	SINGLE SIDED FRCM + TIMBER STRONGBACK - 2/90x45 SG8 STUDS @ 600 CRS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S11	200THK RC WALL (250kg/m³ OF REINFORCING)
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 2 LAYERS OF PLYWOOD SHEATHING ON BOTH SIDES OVER NEW STRUCTURAL FRAMING: 150 x 50 FRAMING AT 450 CRS
S16	NEW 2x 12mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING FLOOR JOISTS AND NEW STRAP BRACING. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S17	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOIST WITH BLOCK AS REQUIRED FOR EDGE SUPPORT. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S3-302
S19	EXISTING ONE-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DWG S3-302
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS



5A NORTH WALL (GRID A5) 3D VIEW
@ A1 A3 Scale = 2 x A1

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CONCEPT

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A	23.11.21	CONCEPT DESIGN ISSUE	DD

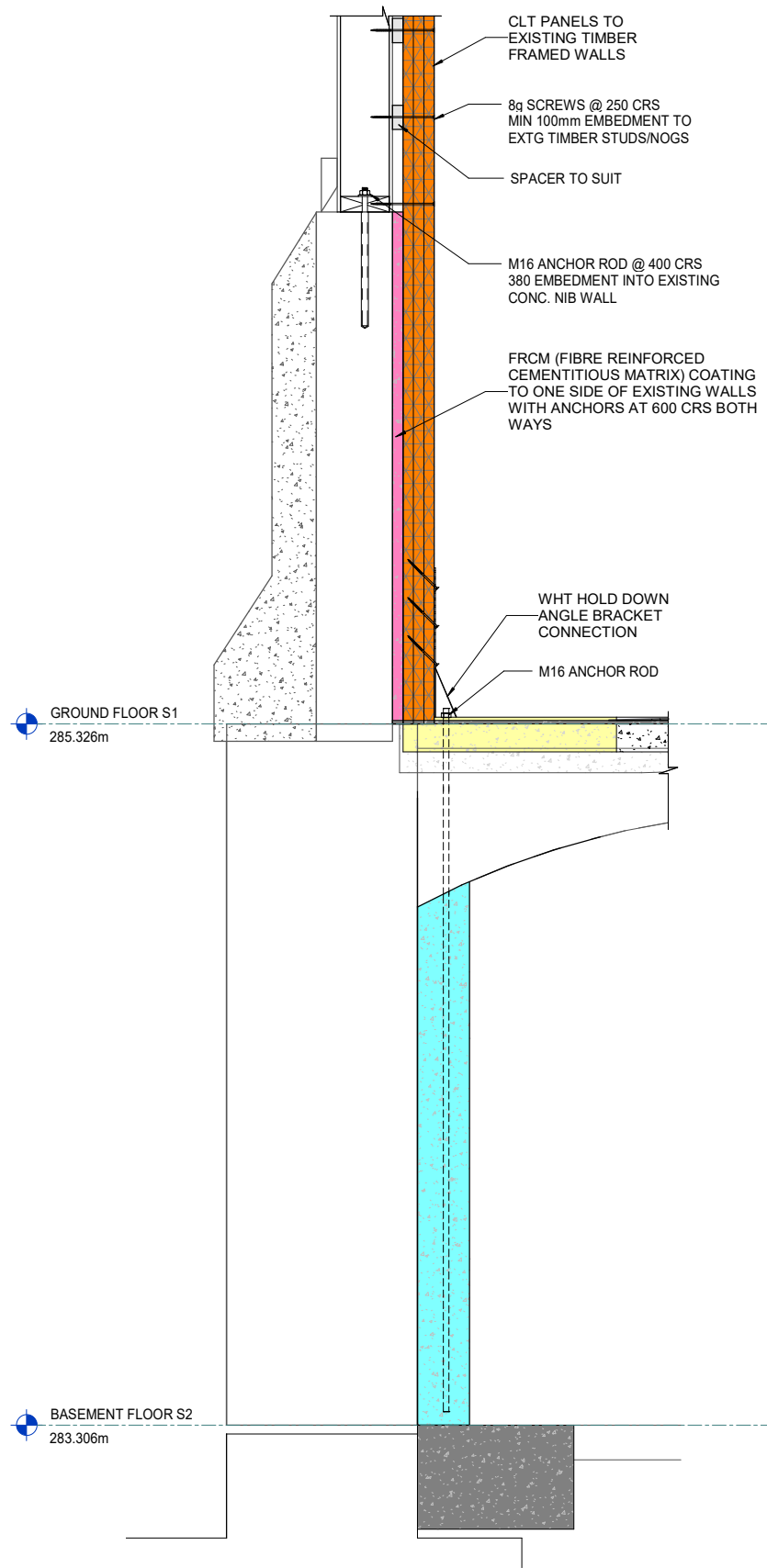
CLIENT
ROTORUA LAKES COUNCIL
Te Kaunihera o ngā Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

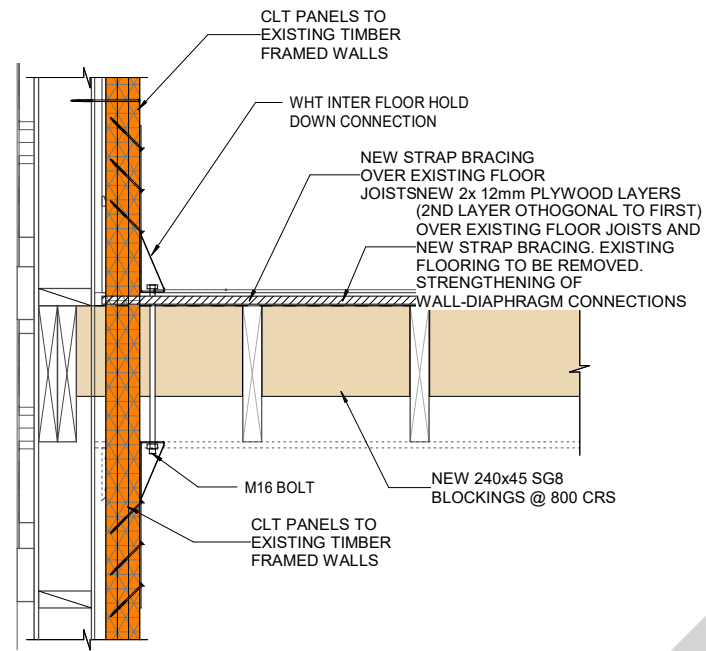
SHEET TITLE
ZONE 3

SECTIONS
**PRELIMINARY CONCEPT
80% NBS (IL3)**

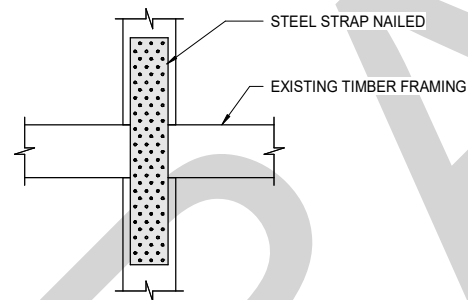
DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD
SCALE	1:50 @ A1 1:100 @ A3		
JOB No.	DWG No.	REVISION	
J2187	S3-205	A	



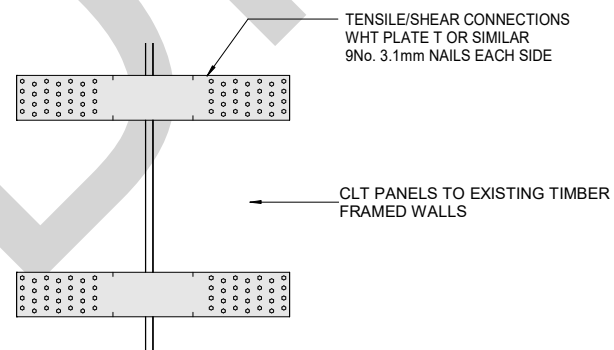
1 GRID AB+ (WEST WALL) BASE CONNECTION
S3-101 1: 10 @ A1 A3 Scale = 2 x A1



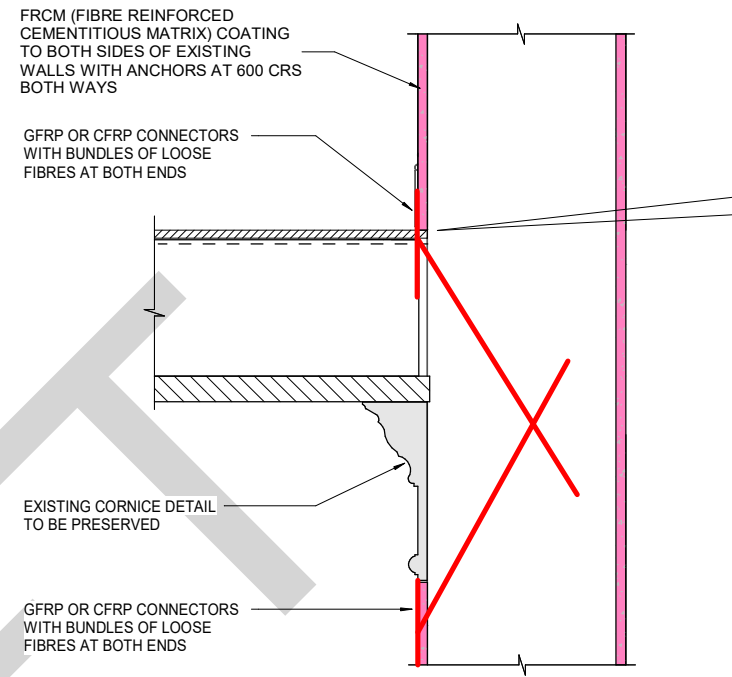
2 GRID AB+ (WEST WALL) INTER FLOOR CONNECTION
S2-103 1: 10 @ A1 A3 Scale = 2 x A1



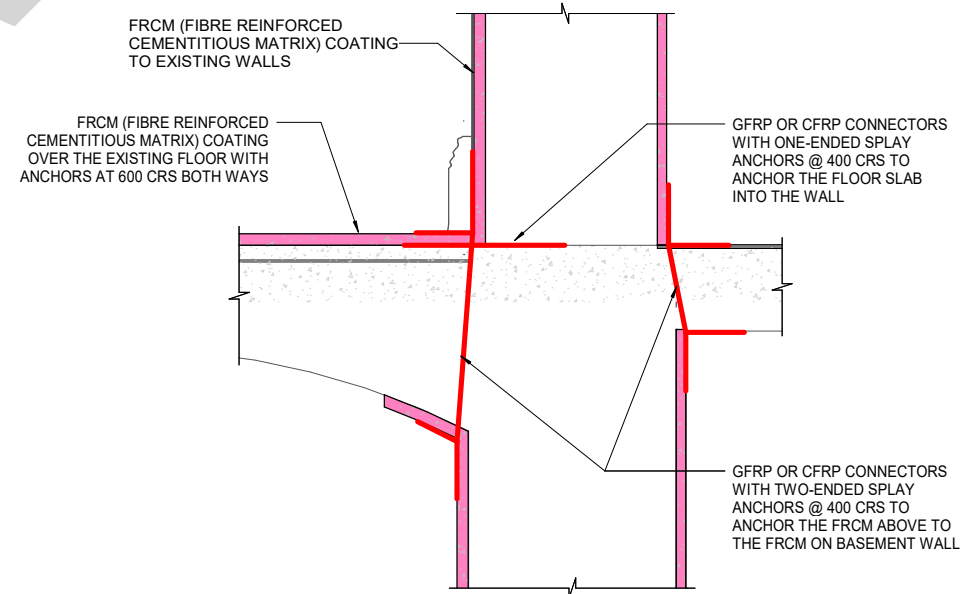
5 TYP. STRENGTHENING OF EXISTING TENON-MORTISE JOINTS
1: 10 @ A1 A3 Scale = 2 x A1



6 CLT PANEL TO PANEL CONNECTION
S3-201 1: 10 @ A1 A3 Scale = 2 x A1



3 FRCM CONTINUATION DETAIL AROUND THE EXISTING MEZZANINE JOISTS
S2-103 1: 10 @ A1 A3 Scale = 2 x A1



4 FRCM CONTINUATION DETAIL AROUND THE EXISTING CONCRETE VAULTED SLAB
S3-101 1: 10 @ A1 A3 Scale = 2 x A1



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CONCEPT

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A	23.11.21	CONCEPT DESIGN ISSUE	DD

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Te Kaunihera o ngā Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

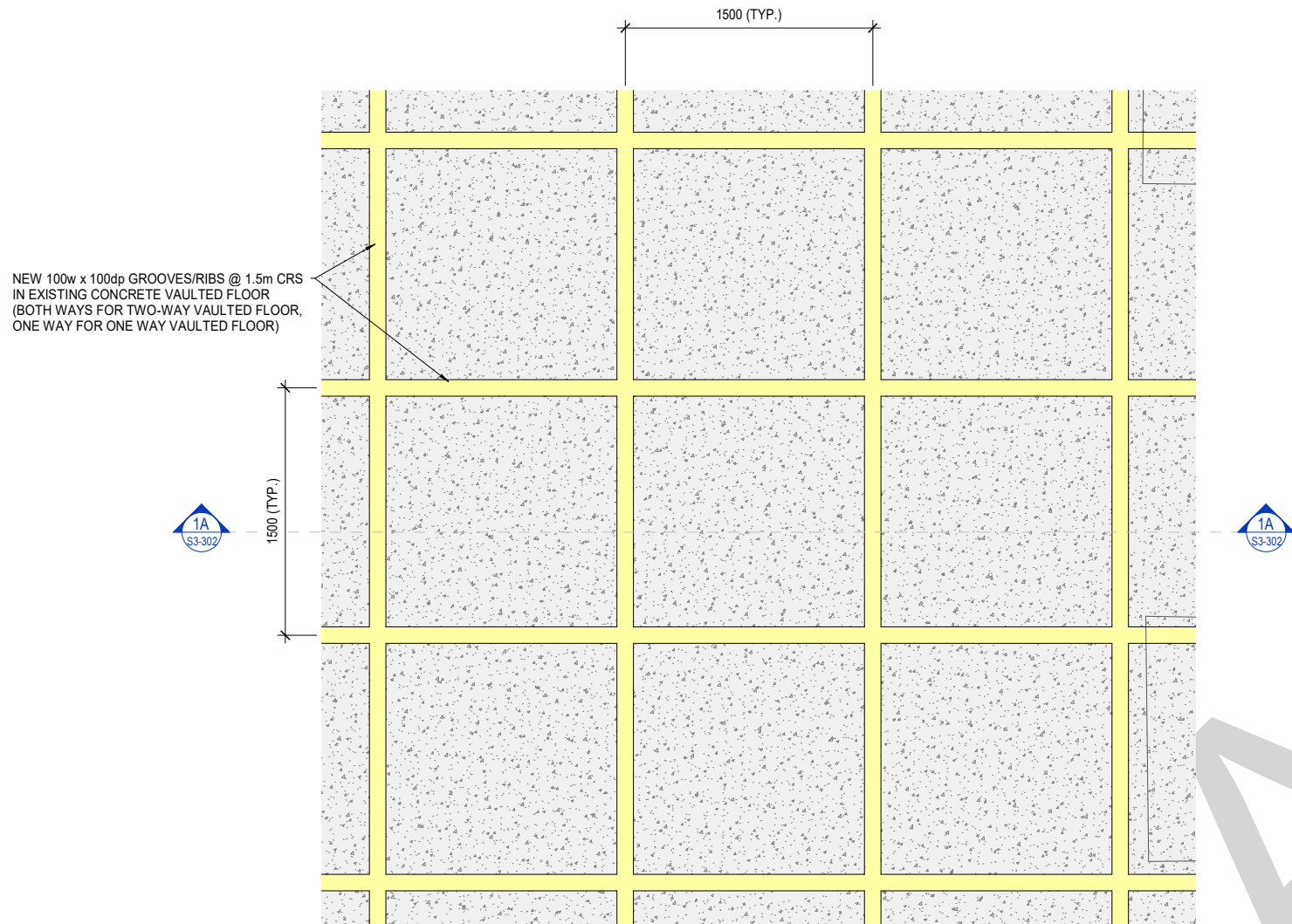
SHEET TITLE
ZONE 3

DETAILS

PRELIMINARY CONCEPT
80% NBS (IL3)

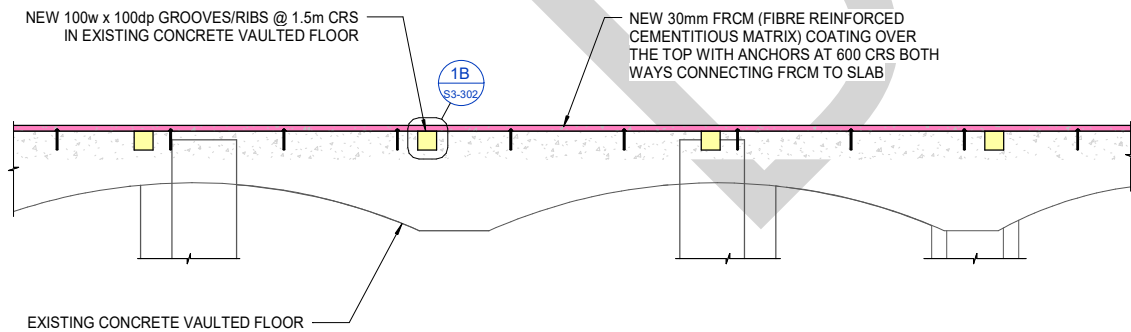
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DRAWN	KT	CHECKED	DD
SCALE	1:10 @ A1 1:20 @ A3	REVISION	
JOB No.	DWG No.	REVISION	
J2187	S3-301	A	

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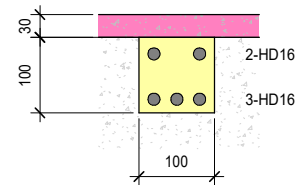


NOTE:
EXISTING CONCRETE TO BE CAREFULLY REMOVED UNDER EXISTING INTERNAL WALLS TO ENSURE RIB CONTINUITY

1 EXISTING CONCRETE VAULTED FLOOR STRENGTHENING - PLAN
1 : 20 @ A1 A3 Scale = 2 x A1



1A SECTION 1A
1 : 20 @ A1 A3 Scale = 2 x A1



1B RIB REINFORCING
1 : 5 @ A1 A3 Scale = 2 x A1

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SHEET TITLE
ZONE 3

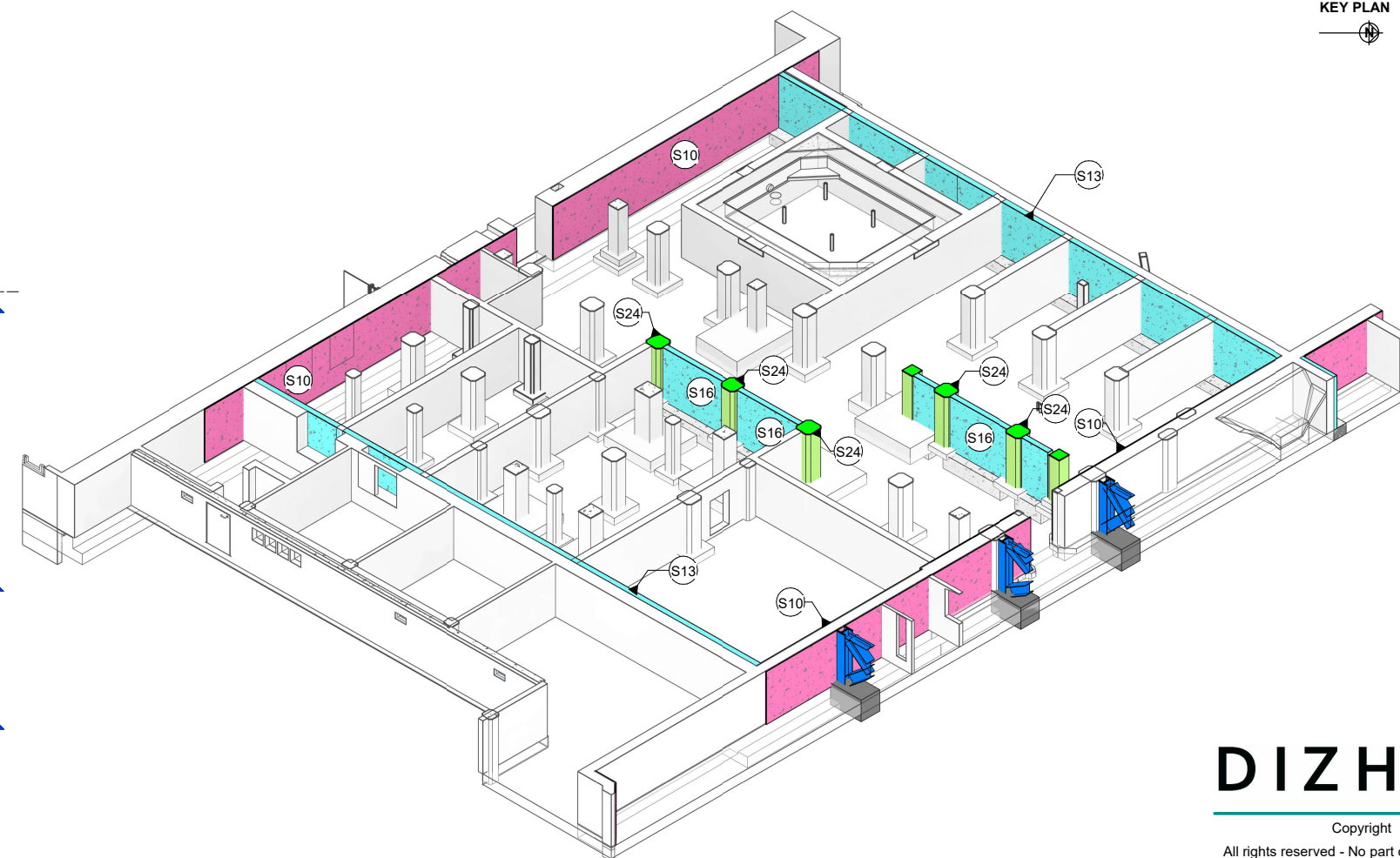
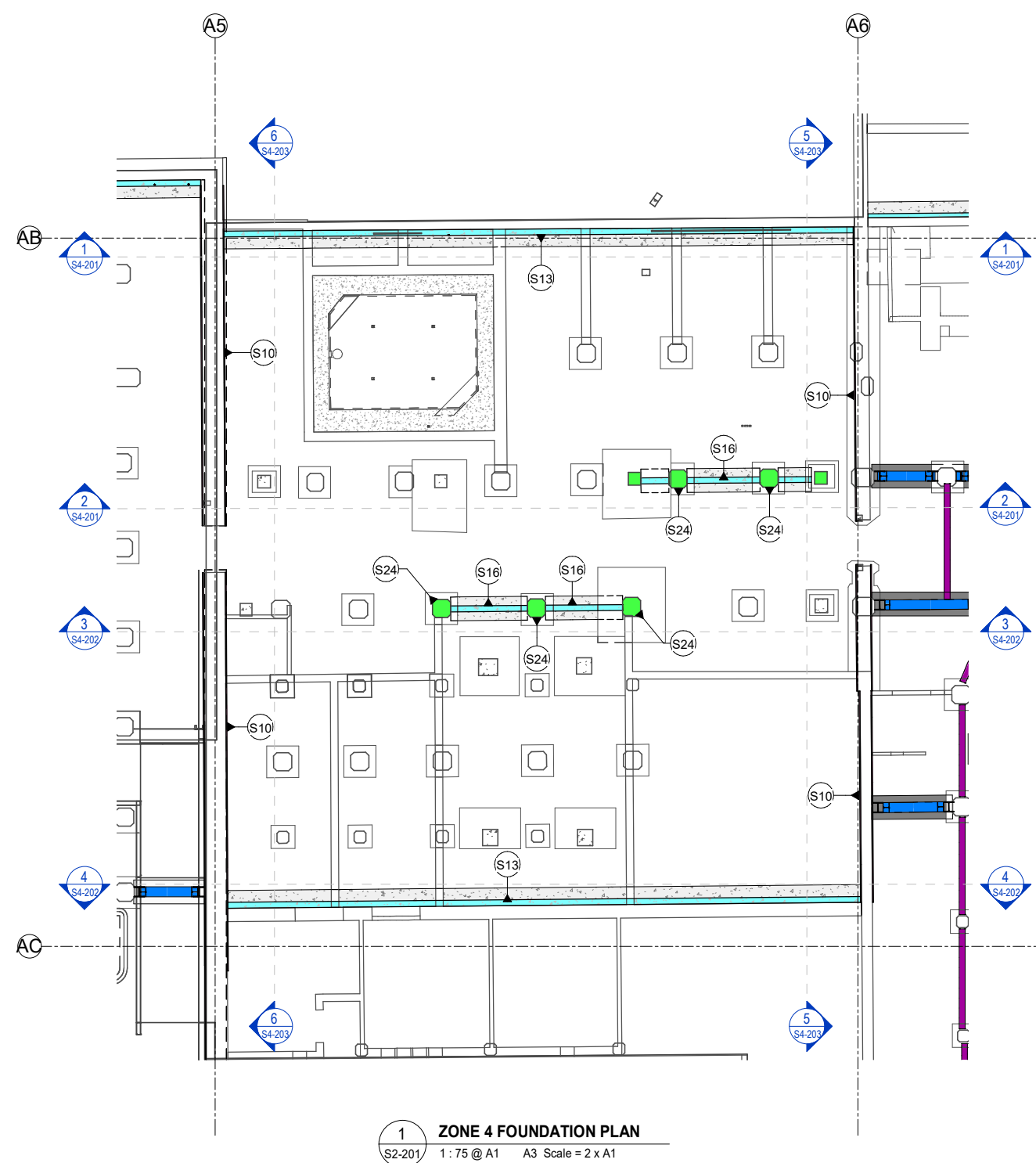
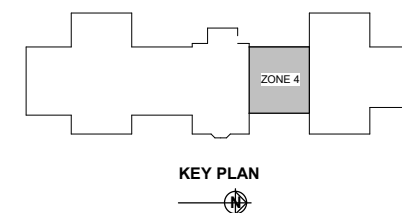
DETAILS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE
As Noted

JOB No.	DWG No.	REVISION
J2187	S3-302	A



2 FOUNDATION 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S16	NEW 150THK RC SHEAR WALL ON NEW 600 WIDE STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S4-301
S19	NEW 2x 17mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S21	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOISTS WITH NOGGING TO SUIT AS NEEDED
S22	12mm PLYWOOD SHEATHING OVER EXISTING SARKING, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN
S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED.

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PROJECT

ROTORUA MUSEUM
DEVELOPMENT

SHEET TITLE

ZONE 4

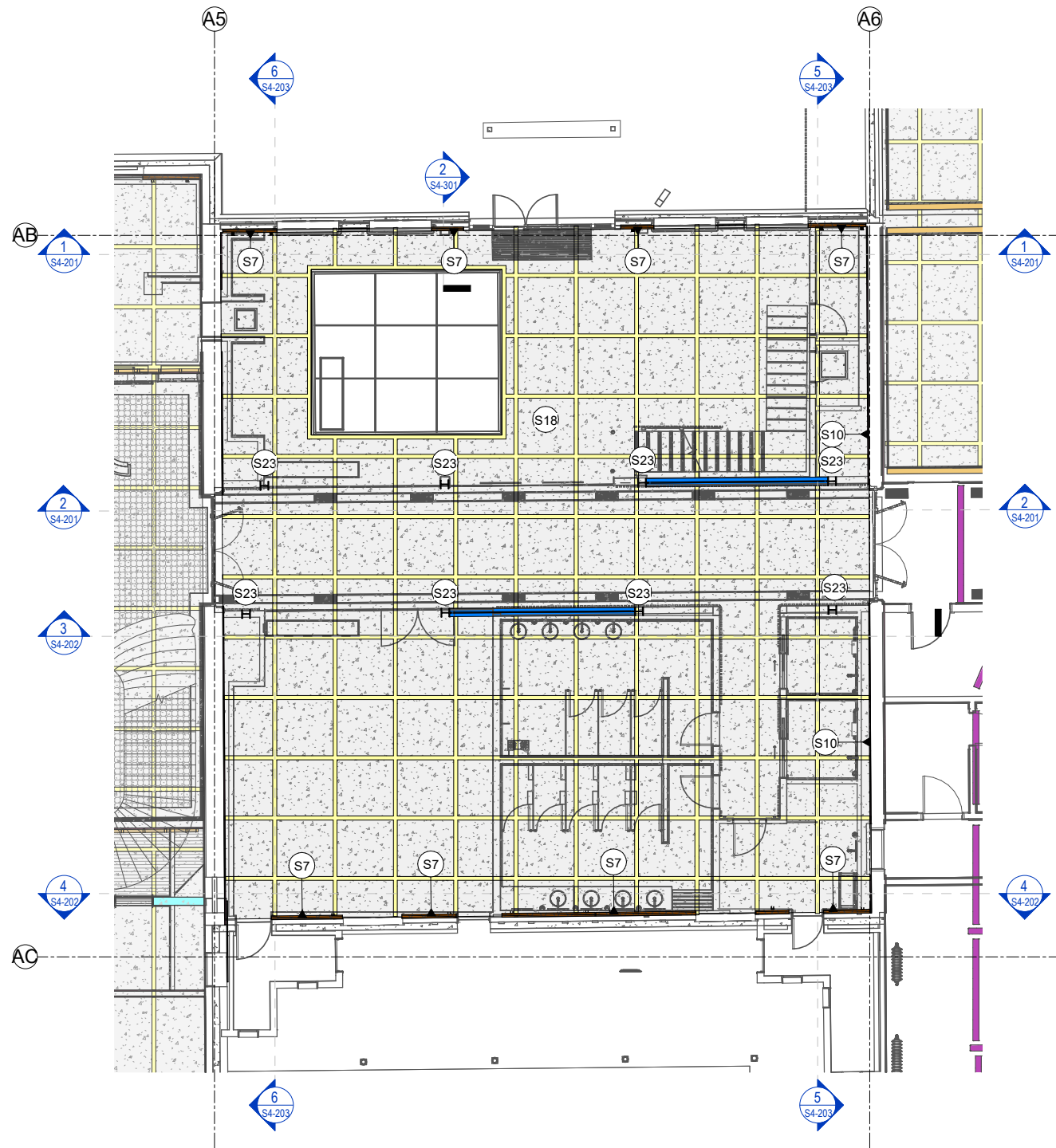
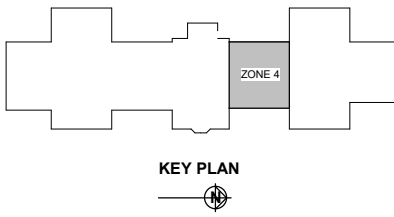
FOUNDATION PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

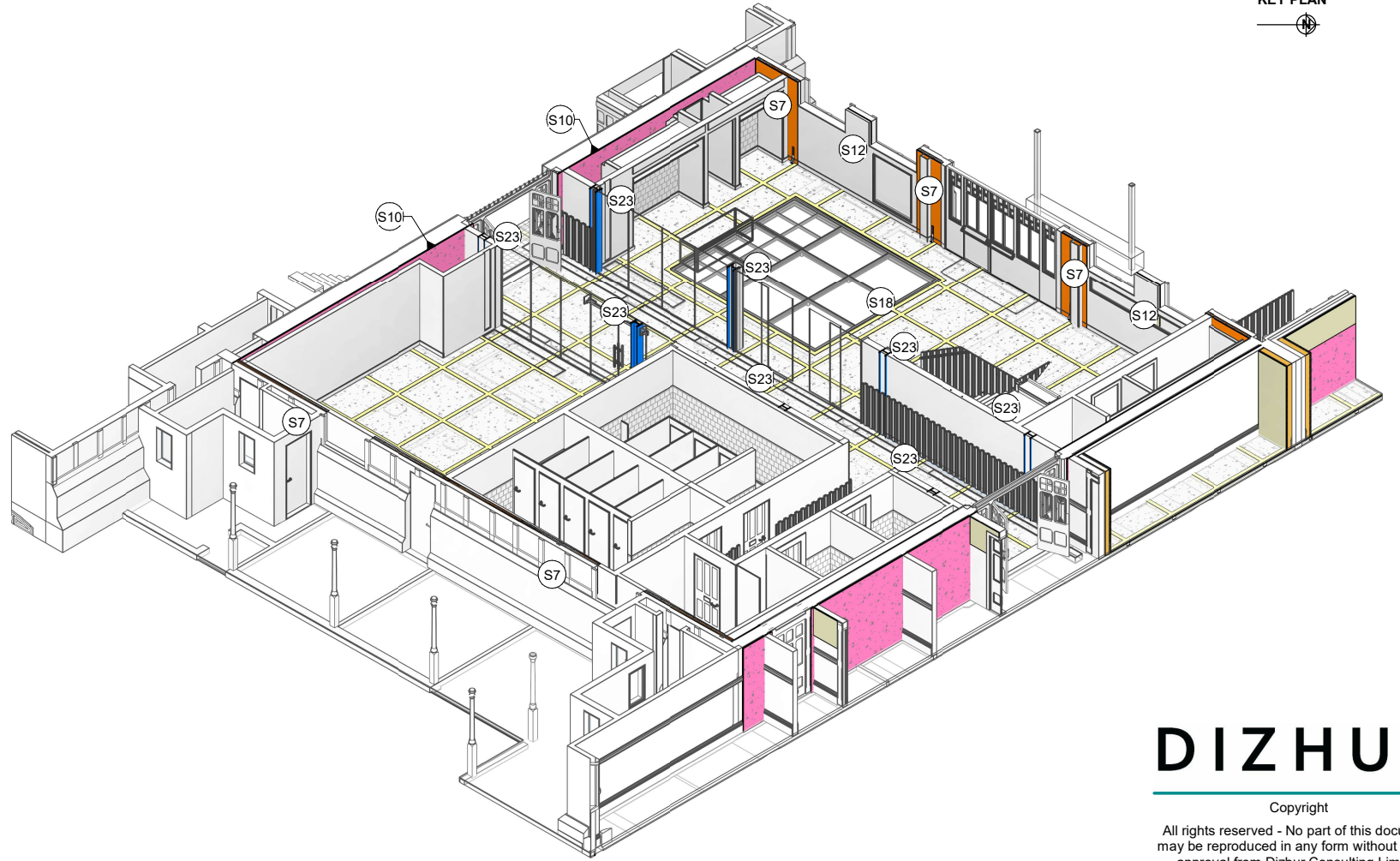
DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE 1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S4-101	A



1 ZONE 4 - GROUND FLOOR PLAN
1 : 75 @ A1 A3 Scale = 2 x A1



1A GROUND FLOOR 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S13	NEW 150THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S16	NEW 150THK RC SHEAR WALL ON NEW 600 WIDE STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S4-301
S19	NEW 2x 17mm PLYWOOD LAYERS (2ND LAYER OTHOGONAL TO FIRST) OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING FLOORING TO BE REMOVED. STRENGTHENING OF WALL-DIAPHRAGM CONNECTIONS
S21	NEW 21mm PLYWOOD SHEATHING OVER EXISTING CEILING JOISTS WITH NOGGING TO SUIT AS NEEDED
S22	12mm PLYWOOD SHEATHING OVER EXISTING SARKING, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN
S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED

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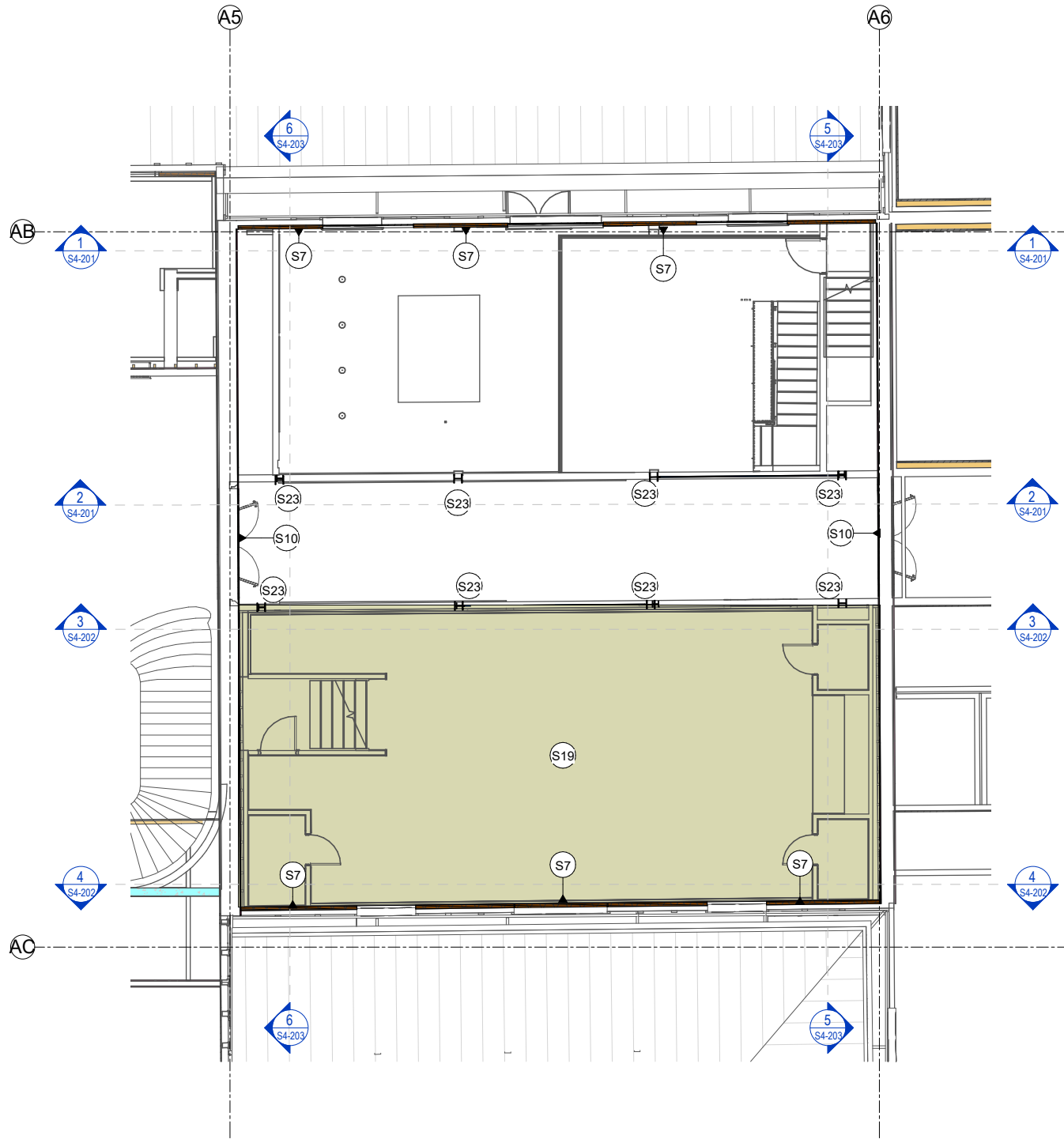
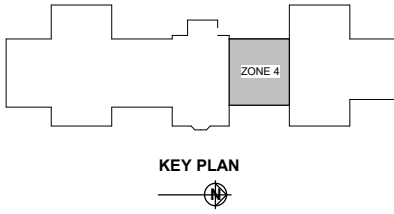
PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 4

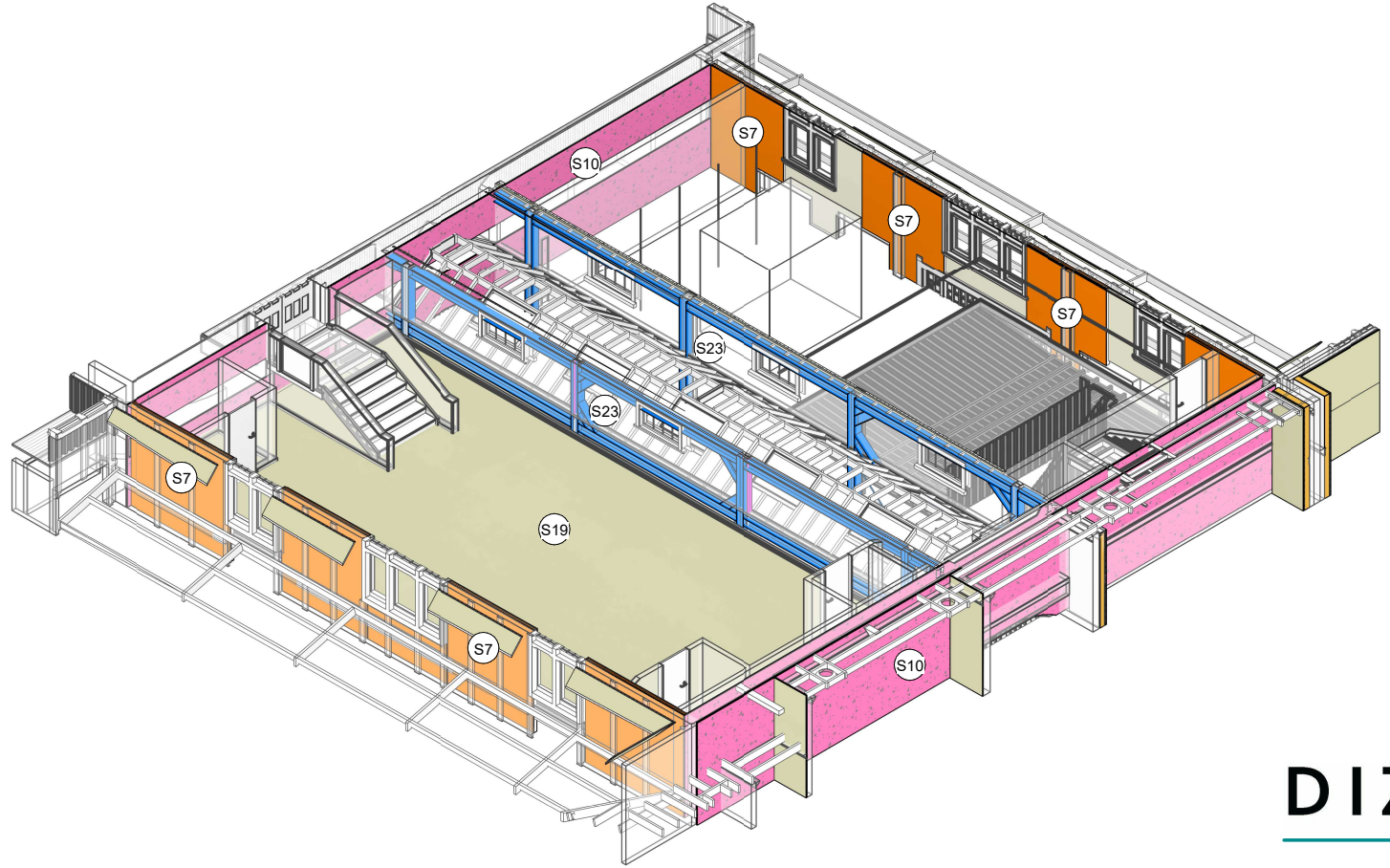
GROUND FLOOR PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD
SCALE		1:75 @ A1 1:150 @ A3	
JOB No.	DWG No.	REVISION	
J2187	S4-102	A	



1 ZONE 4 - MEZZANINE PLAN
1 : 75 @ A1 A3 Scale = 2 x A1



2 ZONE 4 MEZZANINE - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
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S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN
S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED

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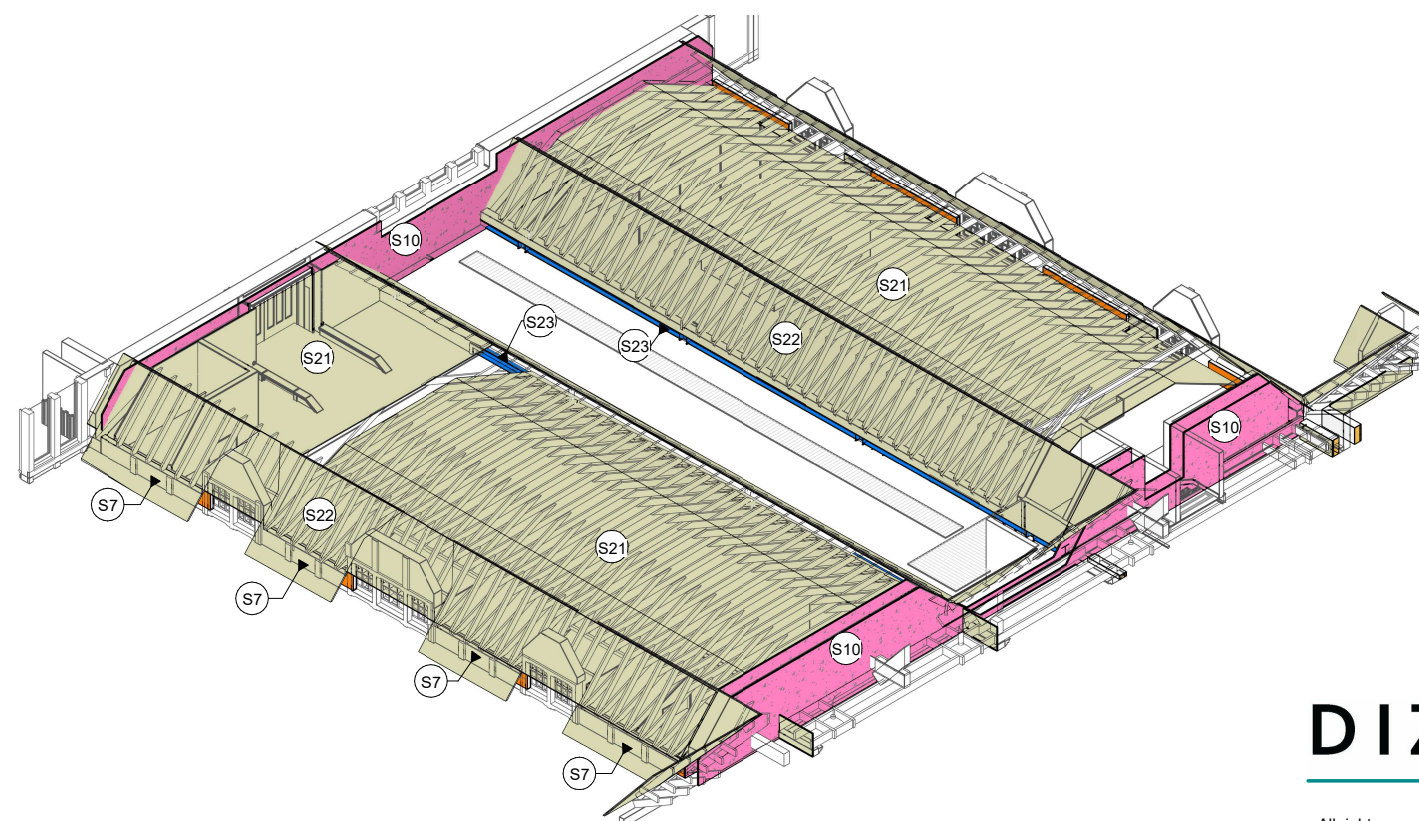
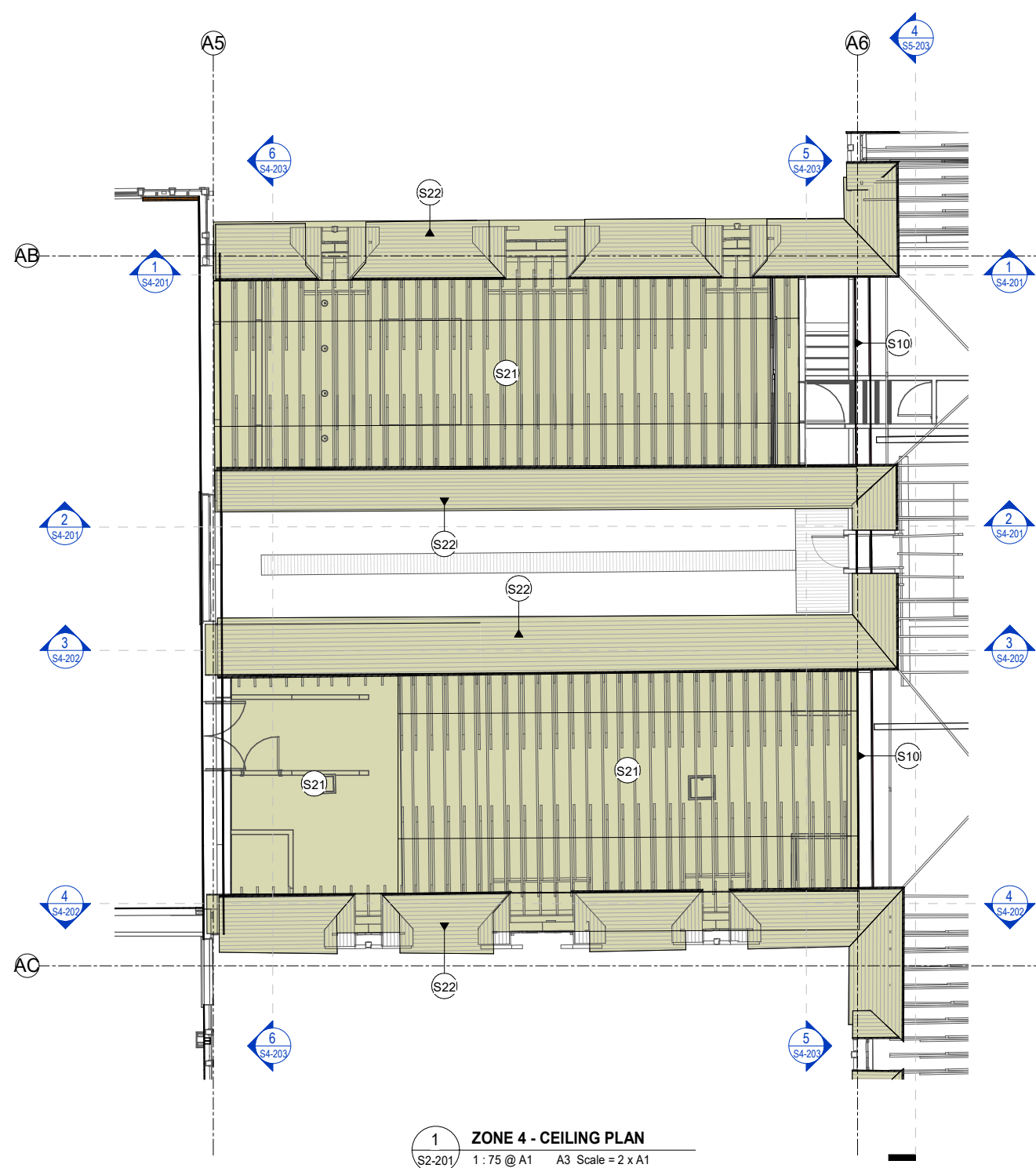
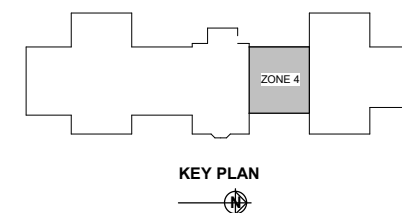
PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 4

MEZZANINE PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD
SCALE		1:75 @ A1 1:150 @ A3	
JOB No.	DWG No.	REVISION	
J2187	S4-103	A	



2 **CEILING / ROOF- 3D VIEW**
@ A1 A3 Scale = 2 x A1

ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
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S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED

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PROJECT

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SHEET TITLE

ZONE 4

CEILING PLAN

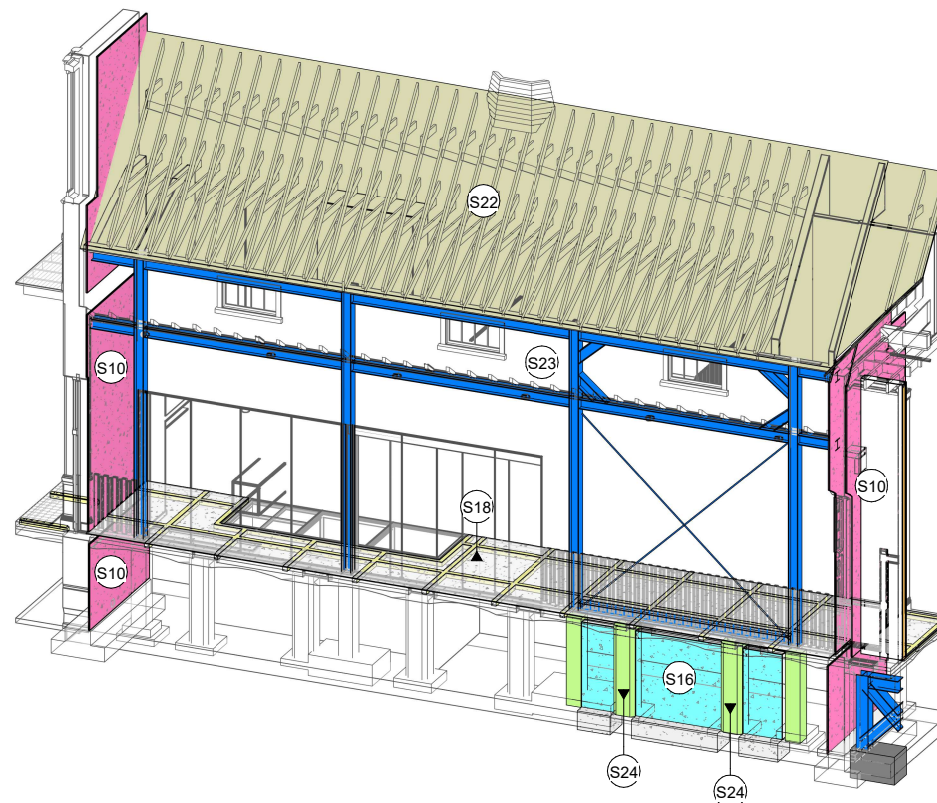
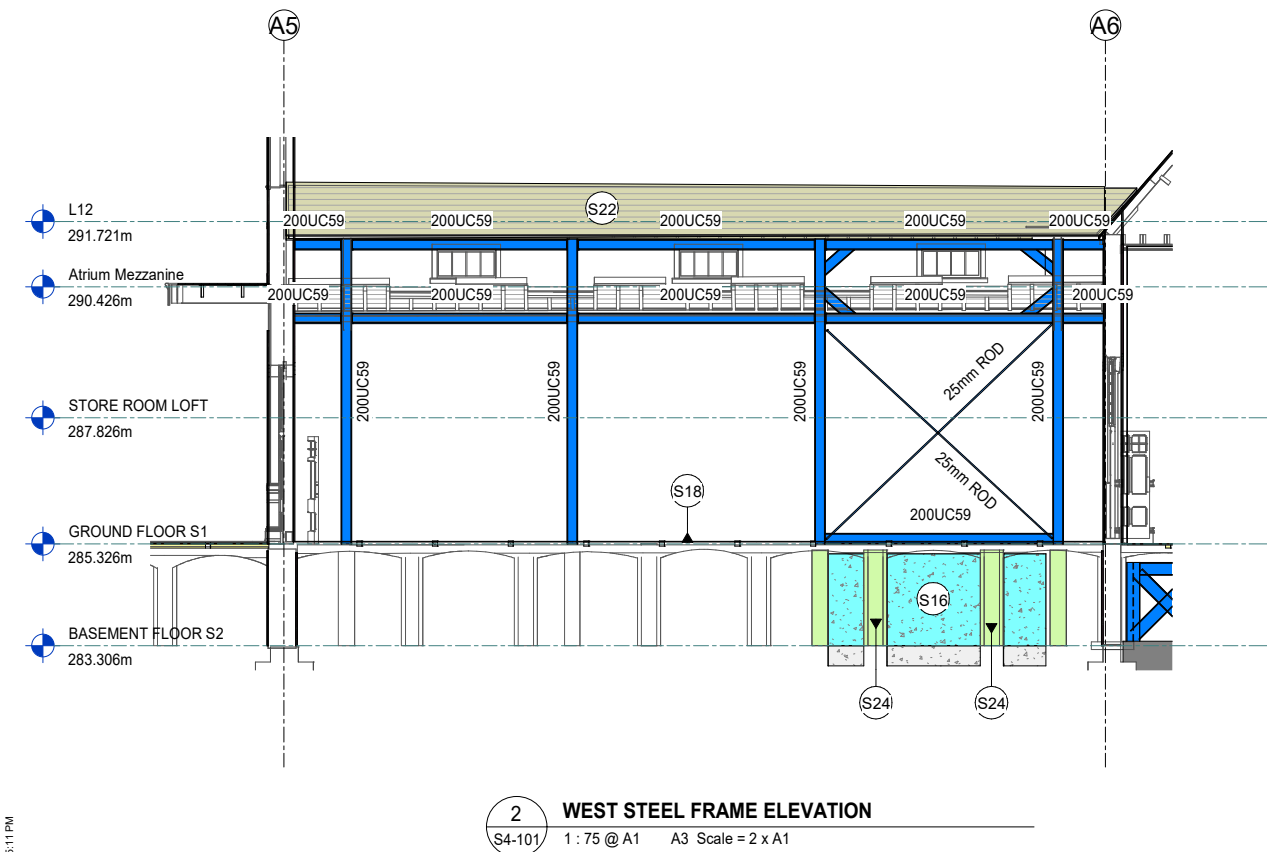
PRELIMINARY CONCEPT
80% NBS (IL3)

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SCALE 1:75 @ A1 1:150 @ A3			
JOB No.		DWG No.	REVISION
J2187		S4-104	A

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ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
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S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED



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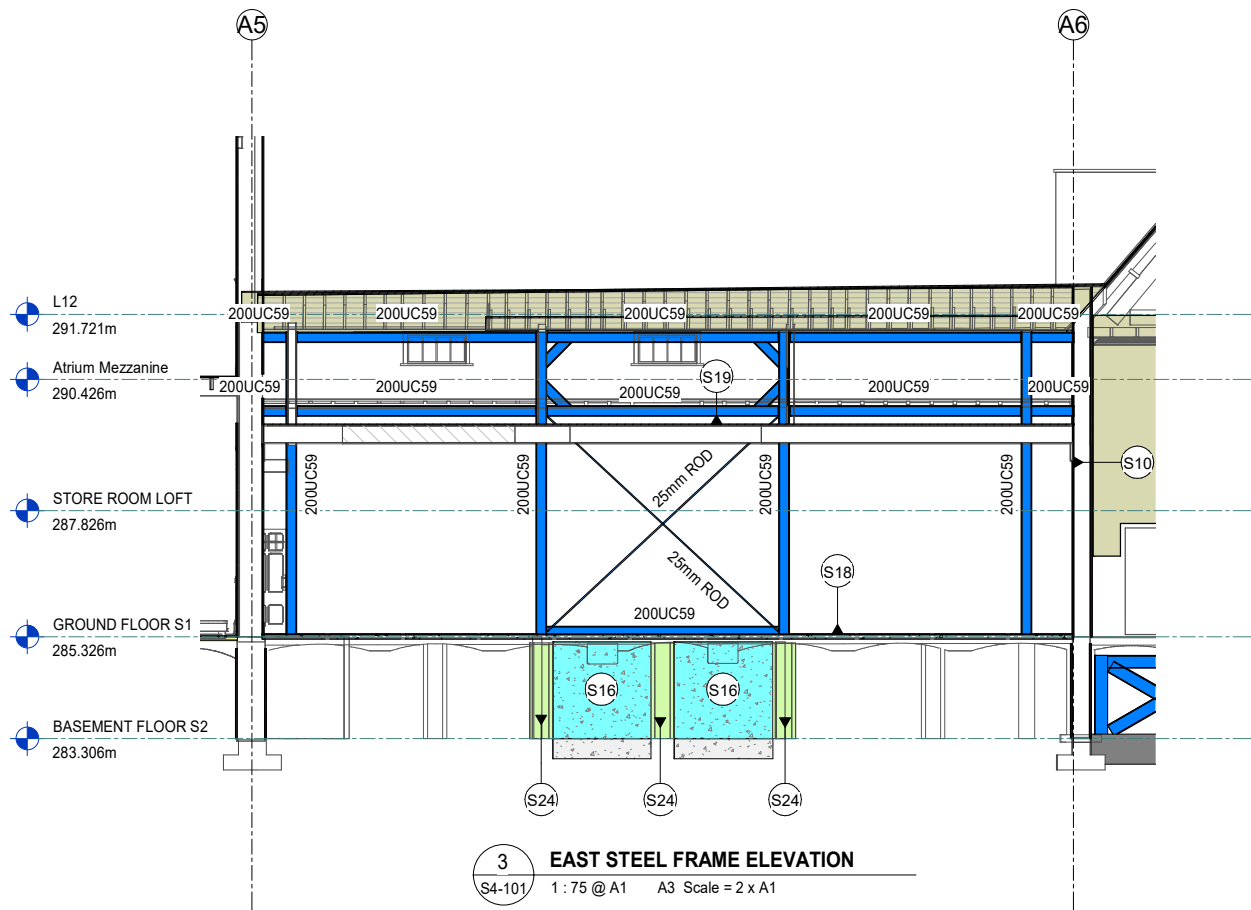
PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 4

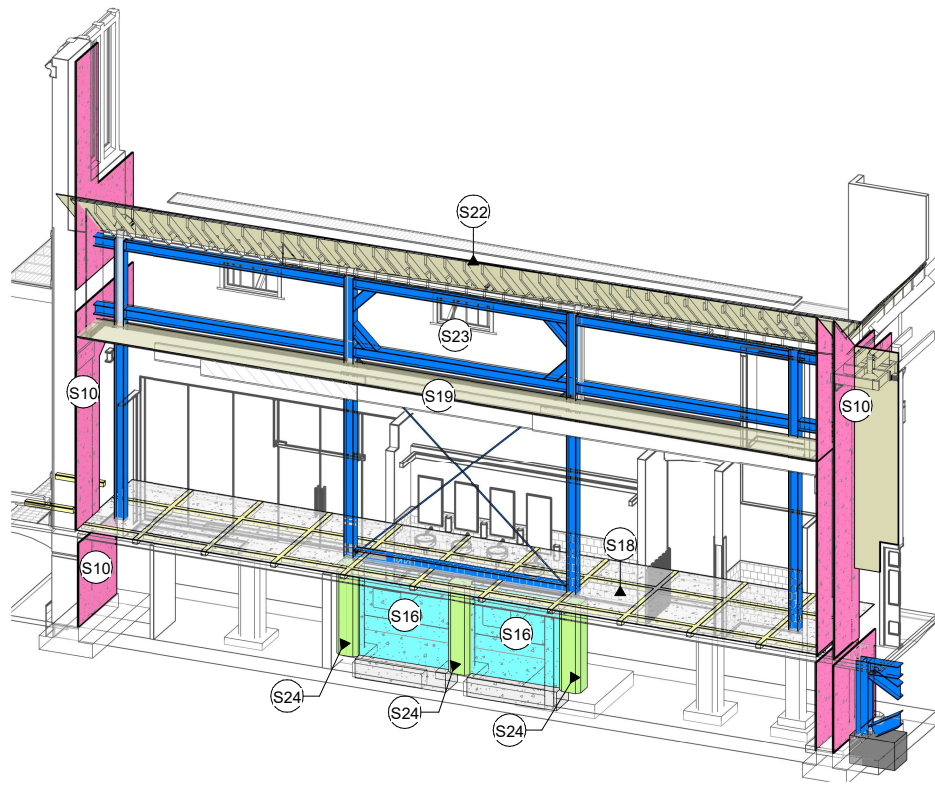
SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

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DRAWN	KT	CHECKED	DD
SCALE		1:75 @ A1 1:150 @ A3	
JOB No.	DWG No.	REVISION	
J2187	S4-201	A	



3 EAST STEEL FRAME ELEVATION
S4-101 1 : 75 @ A1 A3 Scale = 2 x A1



3A EAST STEEL FRAME ELEVATION - 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
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S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN
S24	EXISTING CONCRETE COLUMNS UNDERNEATH STEEL BRACED FRAMES TO BE STRENGTHENED



4 EAST WALL ELEVATION (GRID AC)
S4-101 1 : 75 @ A1 A3 Scale = 2 x A1



4A EAST WALL ELEVATION (GRID AC) - 3D VIEW
@ A1 A3 Scale = 2 x A1

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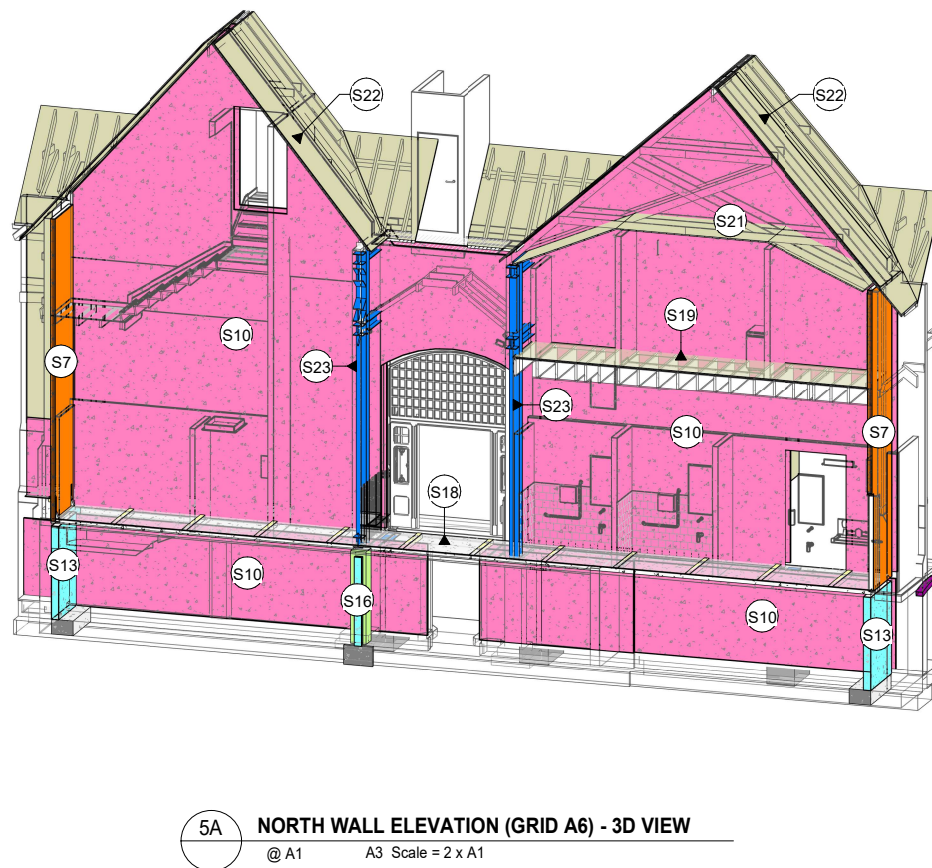
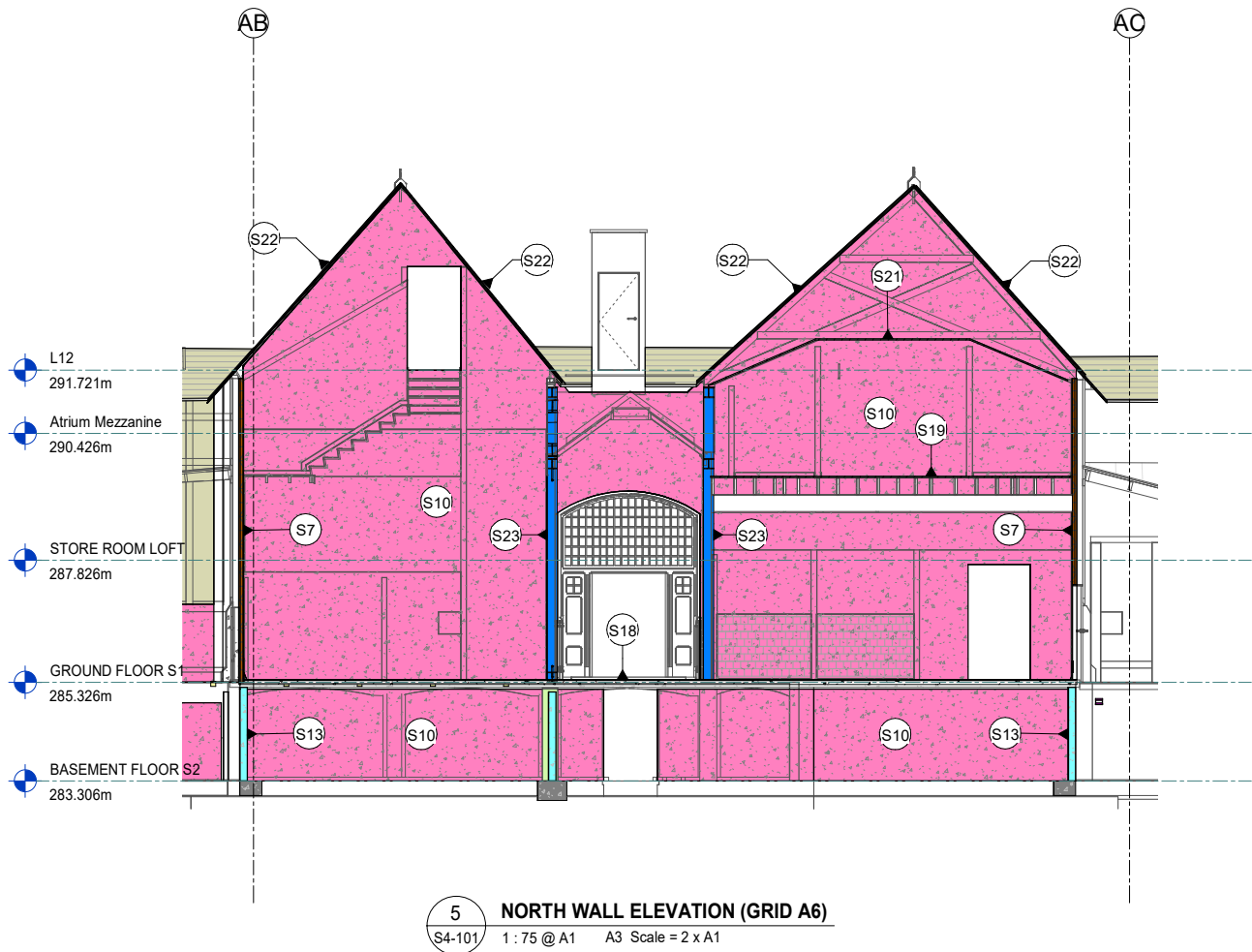
PROJECT
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SHEET TITLE
ZONE 4

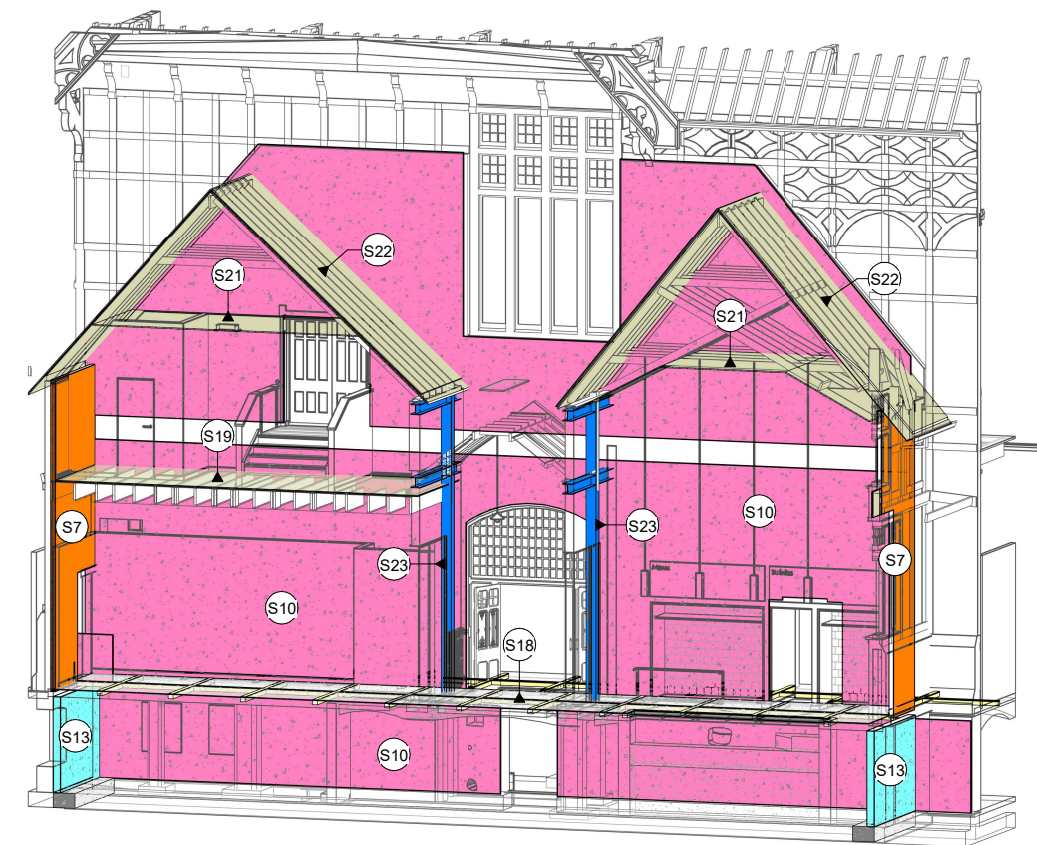
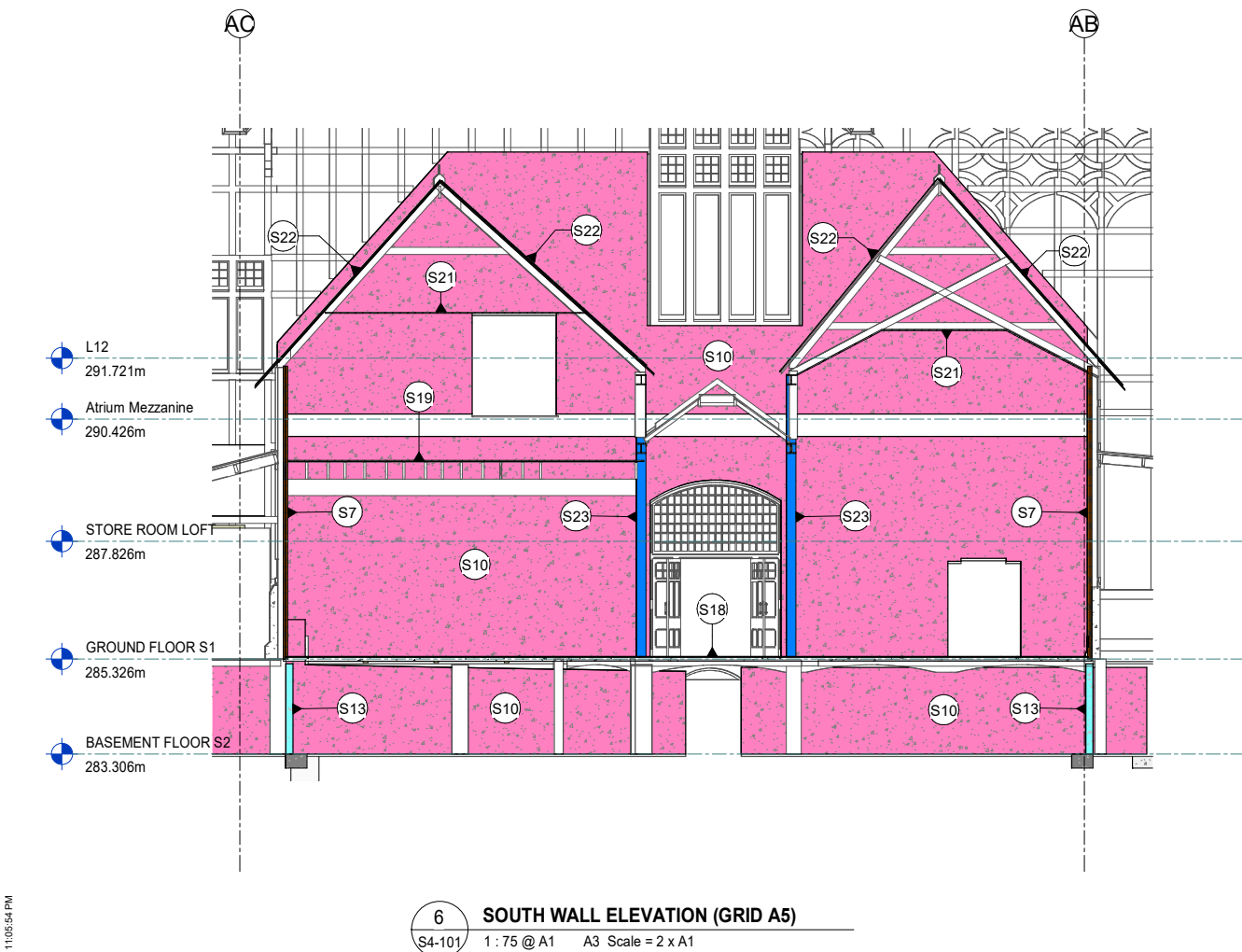
SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
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SCALE		1:75 @ A1 1:150 @ A3	
JOB No.	DWG No.	REVISION	
J2187	S4-202	A	



ZONE 4 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S7	CLT PANELS TO EXISTING TIMBER FRAMED WALLS WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS
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PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE

ZONE 4

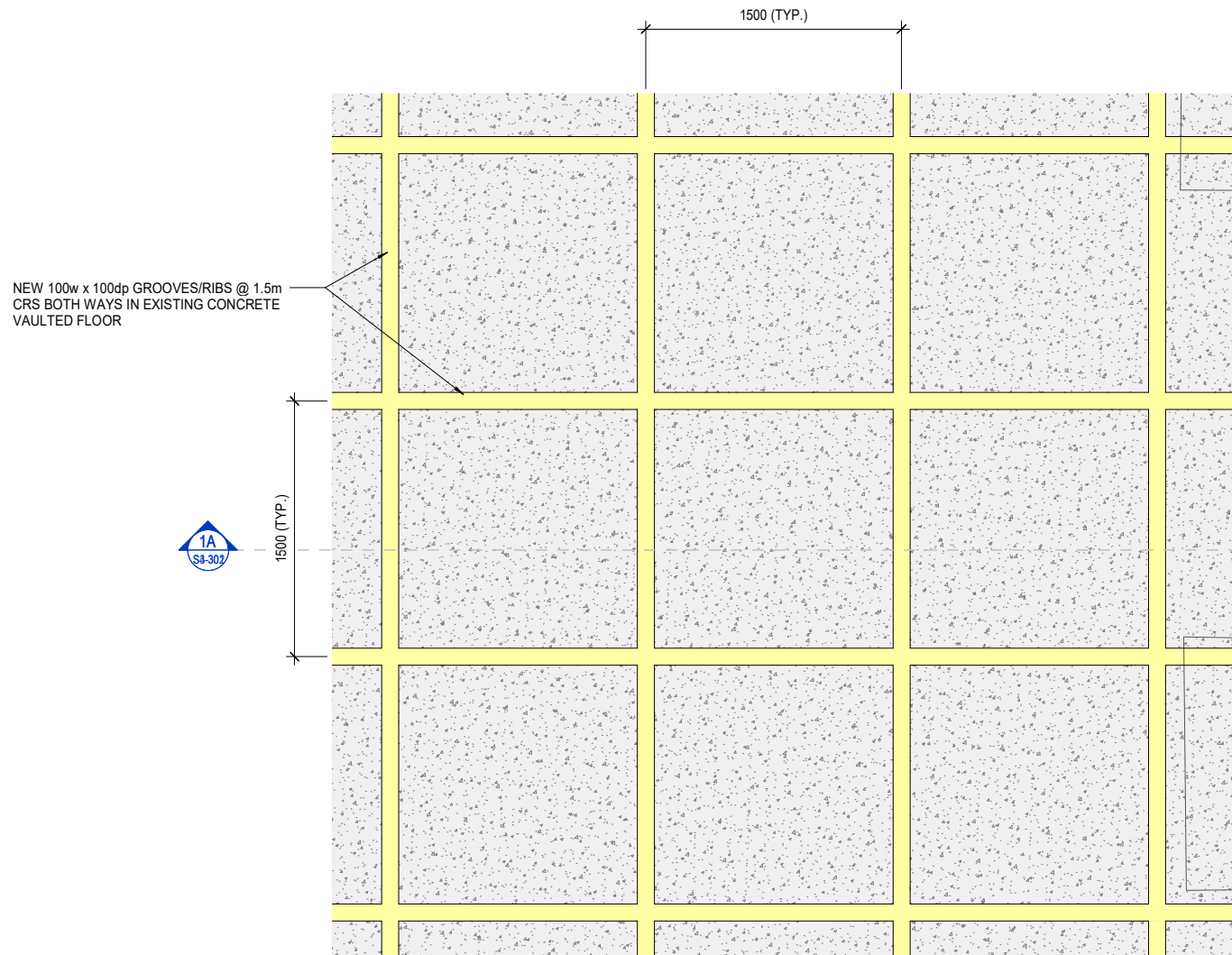
SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

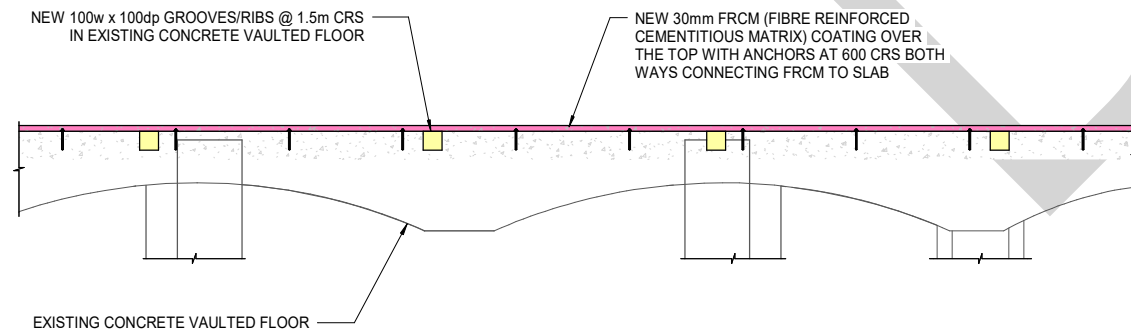
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JOB No.	DWG No.	REVISION
J2187	S4-203	A

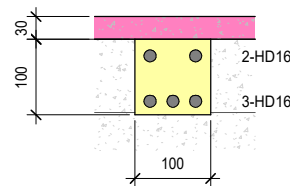


NOTE:
EXISTING CONCRETE TO BE CAREFULLY REMOVED UNDER EXISTING INTERNAL WALLS TO ENSURE RIB CONTINUITY

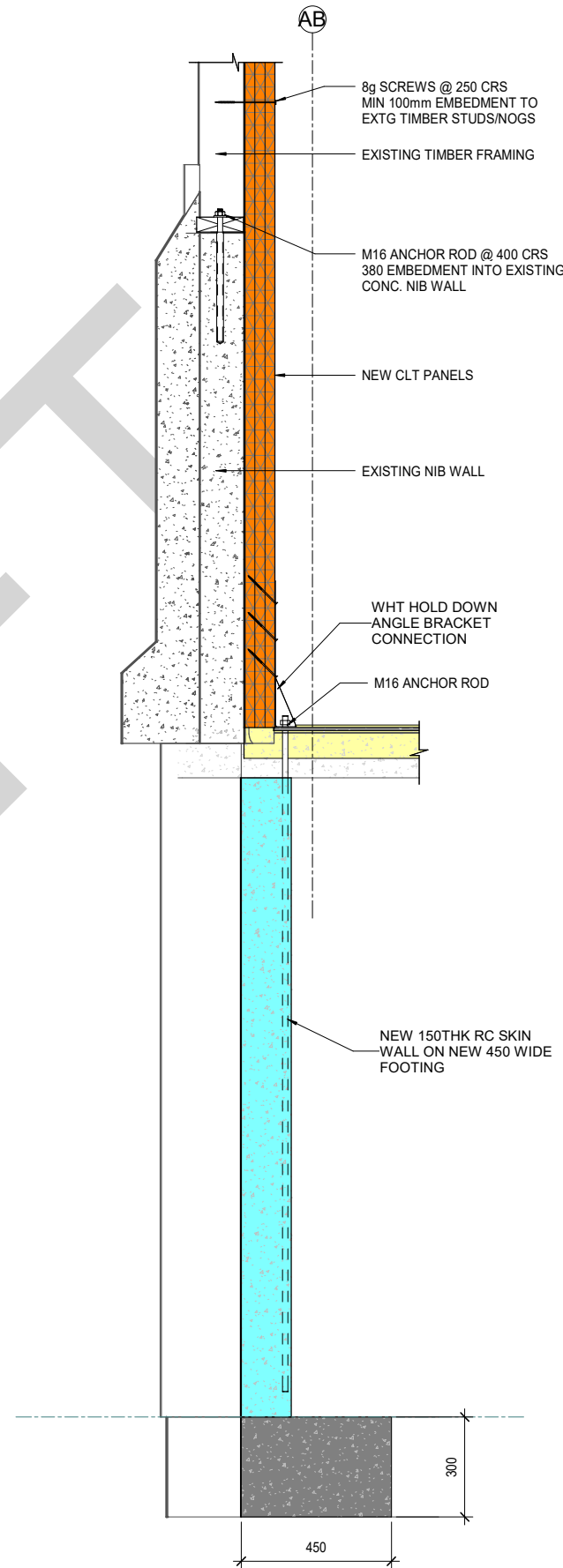
1 EXISTING CONCRETE VAULTED FLOOR STRENGTHENING - PLAN
1:20 @ A1 A3 Scale = 2 x A1



1A SECTION 1A
S4-301 1:20 @ A1 A3 Scale = 2 x A1



1B RIB REINFORCING
1:5 @ A1 A3 Scale = 2 x A1



2 CLT PANEL CONNECTION DETAIL
S4-102 1:10 @ A1 A3 Scale = 2 x A1

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CONCEPT

A	29.11.21	CONCEPT DESIGN ISSUE	DD
Rev	Date	Description	App

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Te Kaitiaki o ngā Roto o Rotorua

PROJECT

ROTORUA MUSEUM
DEVELOPMENT

SHEET TITLE

ZONE 4

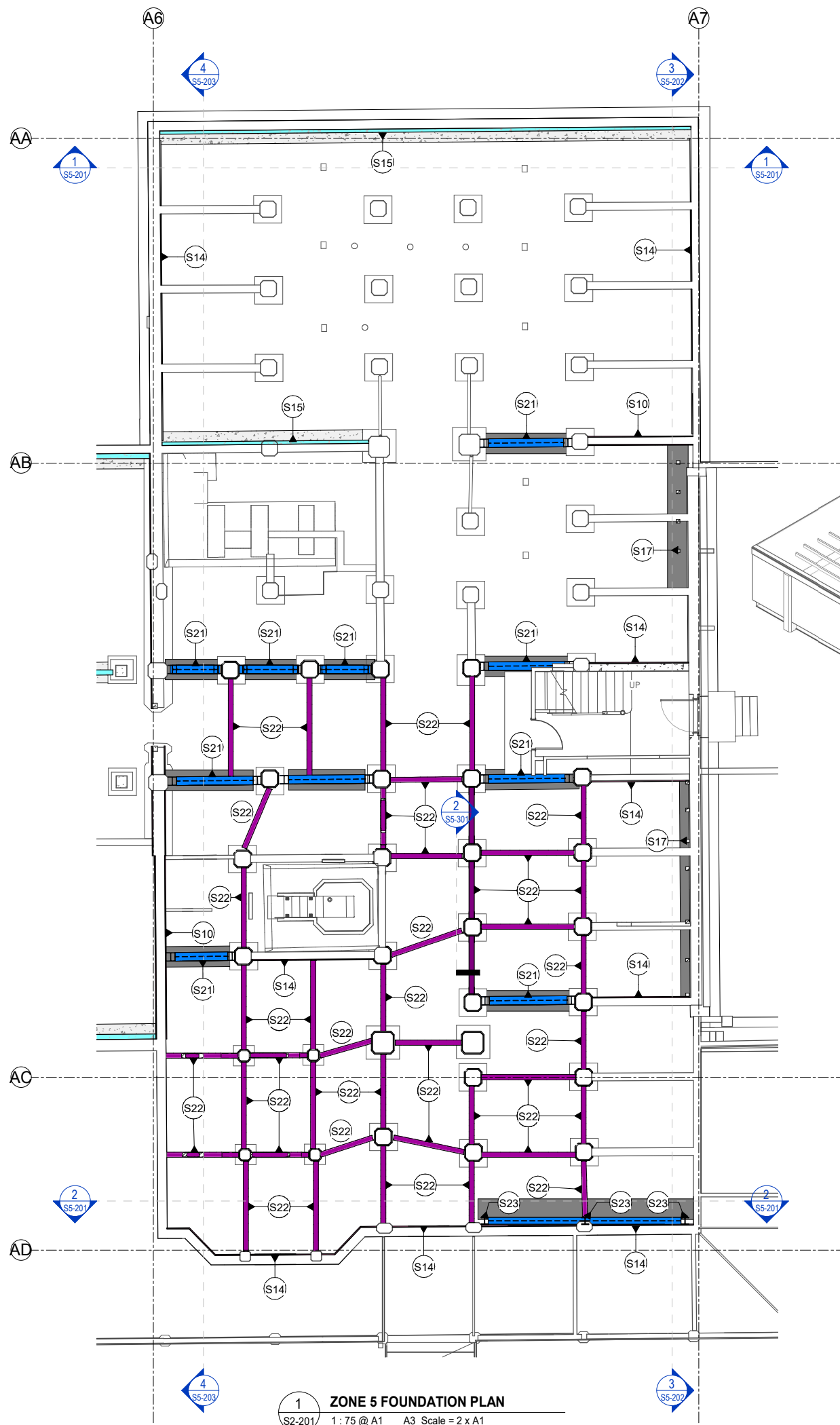
DETAILS

PRELIMINARY CONCEPT
80% NBS (IL3)

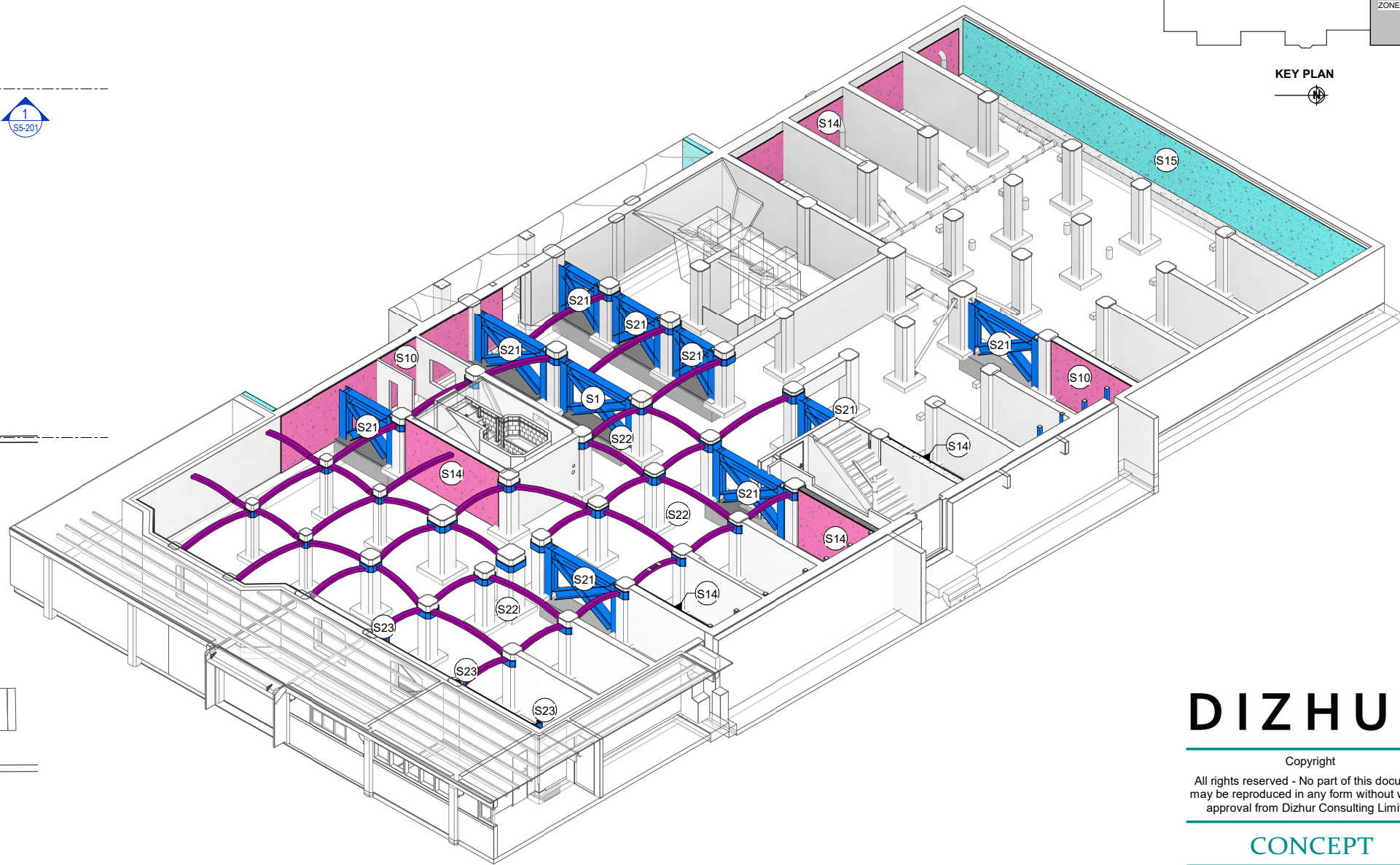
DESIGNED	Designer	DATE	OCT 2021
DRAWN	KT	CHECKED	DD

SCALE 1:10 @ A1
1:20 @ A3

JOB No.	DWG No.	REVISION
J2187	S4-301	A

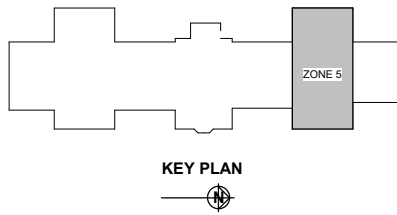


1 ZONE 5 FOUNDATION PLAN
1: 75 @ A1 A3 Scale = 2 x A1



2 FOUNDATION 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 5 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S6	NEW 150x50 TIMBER STUD WALL DIRECTLY IN FRONT OF EXISTING WALL, WITH 2 LAYERS OF 21mm PLYWOOD (1ST LAYER ORTHOGONAL TO 2ND LAYER)
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS, AND STRENGTHENING OF TIMBER FRAME TO NIB WALL CONNECTIONS AND WALL TO CEILING DIAPHRAGM CONNECTIONS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 100THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S17	NEW STEEL STRONGBACKS AT 1200 CRS, FROM BASEMENT TO ROOF LINE WITH NEW R.C STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED, REFER DRAWING S5-301
S19	NEW 21mm PLYWOOD SHEATHING 1.8m WIDE OVER CEILING OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING CEILING RAFTERS, WITH NEW BLOCKING TO SUIT AND NEW CONNECTIONS TO EXISTING WALL
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S21	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S22	NEW TWO-WAY BENT RHS150x100x4 BEAMS SPANNING BETWEEN EXISTING COLUMNS WITH STEEL COLLARS. REFER DWG S5-301 FOR DETAIL
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN



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PROJECT
ROTORUA MUSEUM
DEVELOPMENT

SHEET TITLE
ZONE 5

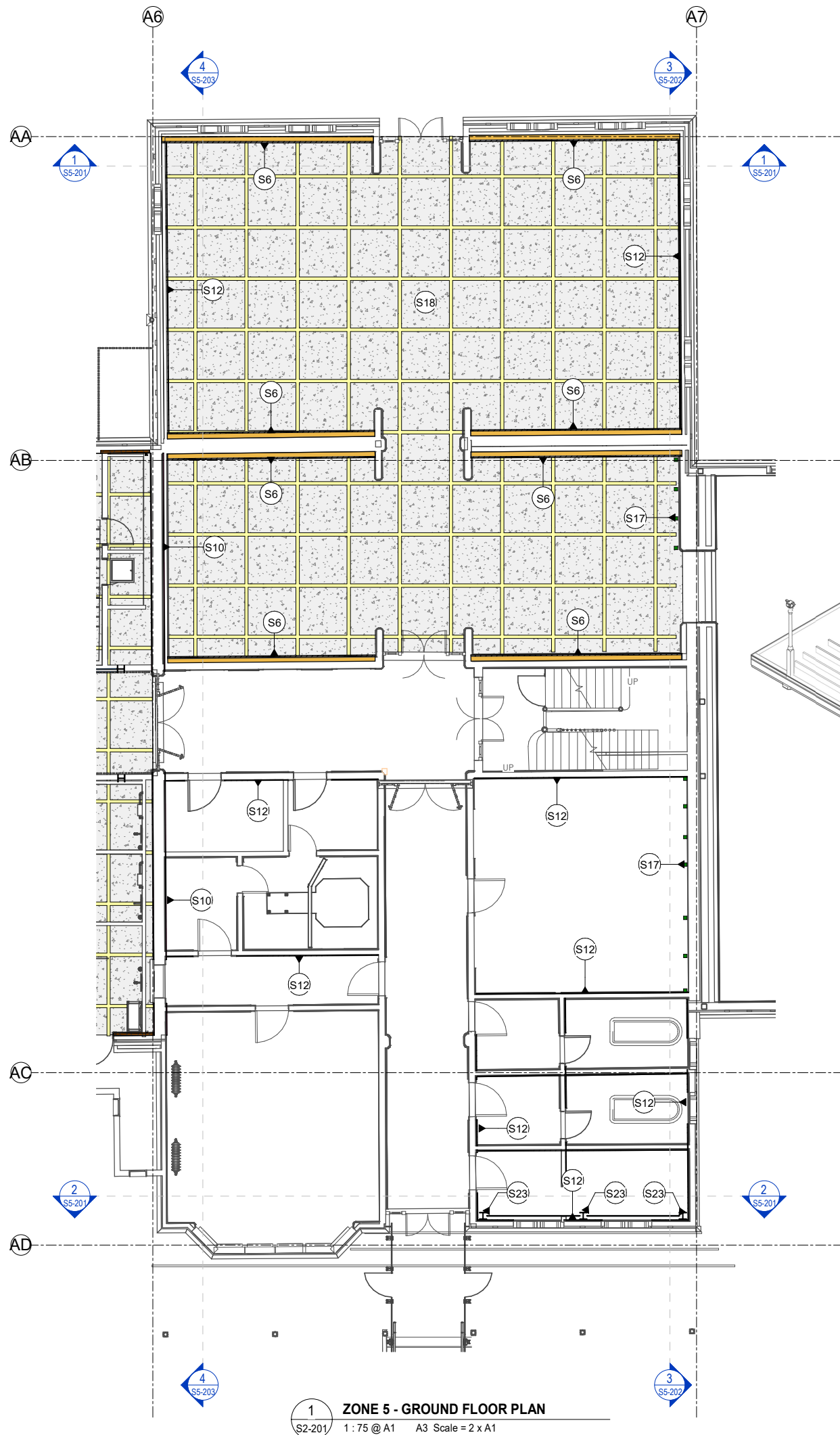
FOUNDATION PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

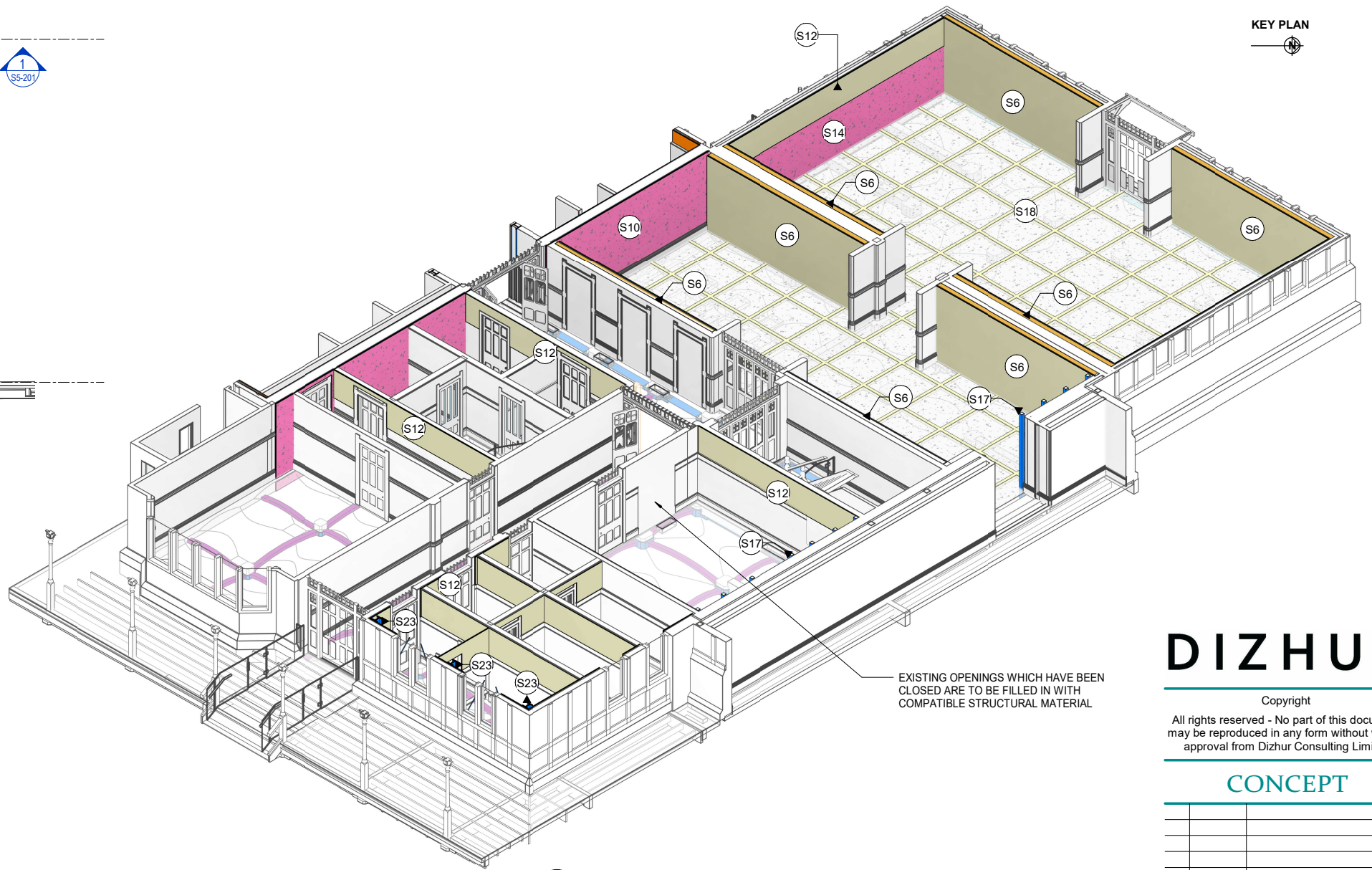
DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD

SCALE
1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S5-101	A

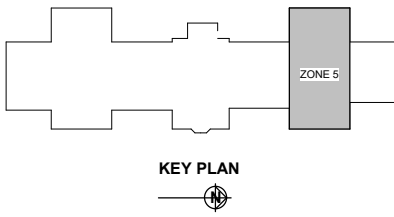


1 ZONE 5 - GROUND FLOOR PLAN
S2-201 1 : 75 @ A1 A3 Scale = 2 x A1



2 GROUND FLOOR 3D VIEW
@ A1 A3 Scale = 2 x A1

ZONE 5 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S6	NEW 150x50 TIMBER STUD WALL DIRECTLY IN FRONT OF EXISTING WALL, WITH 2 LAYERS OF 21mm PLYWOOD (1ST LAYER ORTHOGONAL TO 2ND LAYER)
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS, AND STRENGTHENING OF TIMBER FRAME TO NIB WALL CONNECTIONS AND WALL TO CEILING DIAPHRAGM CONNECTIONS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 100THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S17	NEW STEEL STRONGBACKS AT 1200 CRS, FROM BASEMENT TO ROOF LINE WITH NEW R.C STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S5-301
S19	NEW 21mm PLYWOOD SHEATHING 1.8m WIDE OVER CEILING OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING CEILING RAFTERS, WITH NEW BLOCKING TO SUIT AND NEW CONNECTIONS TO EXISTING WALL
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S21	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S22	NEW TWO-WAY BENT RHS150x100x4 BEAMS SPANNING BETWEEN EXISTING COLUMNS WITH STEEL COLLARS. REFER DWG S5-301 FOR DETAIL
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN



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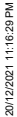
PROJECT
ROTORUA MUSEUM
DEVELOPMENT

SHEET TITLE
ZONE 5

GROUND FLOOR PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD
SCALE	1:75 @ A1 1:150 @ A3		
JOB No.	DWG No.	REVISION	
J2187	S5-102	A	



@ A1 A3 Scale = 2 x A1

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SHEET TITLE

ZONE 5

ROOF PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

SCALE 1:75 @ A1

1:150 @ A3

JOB No.

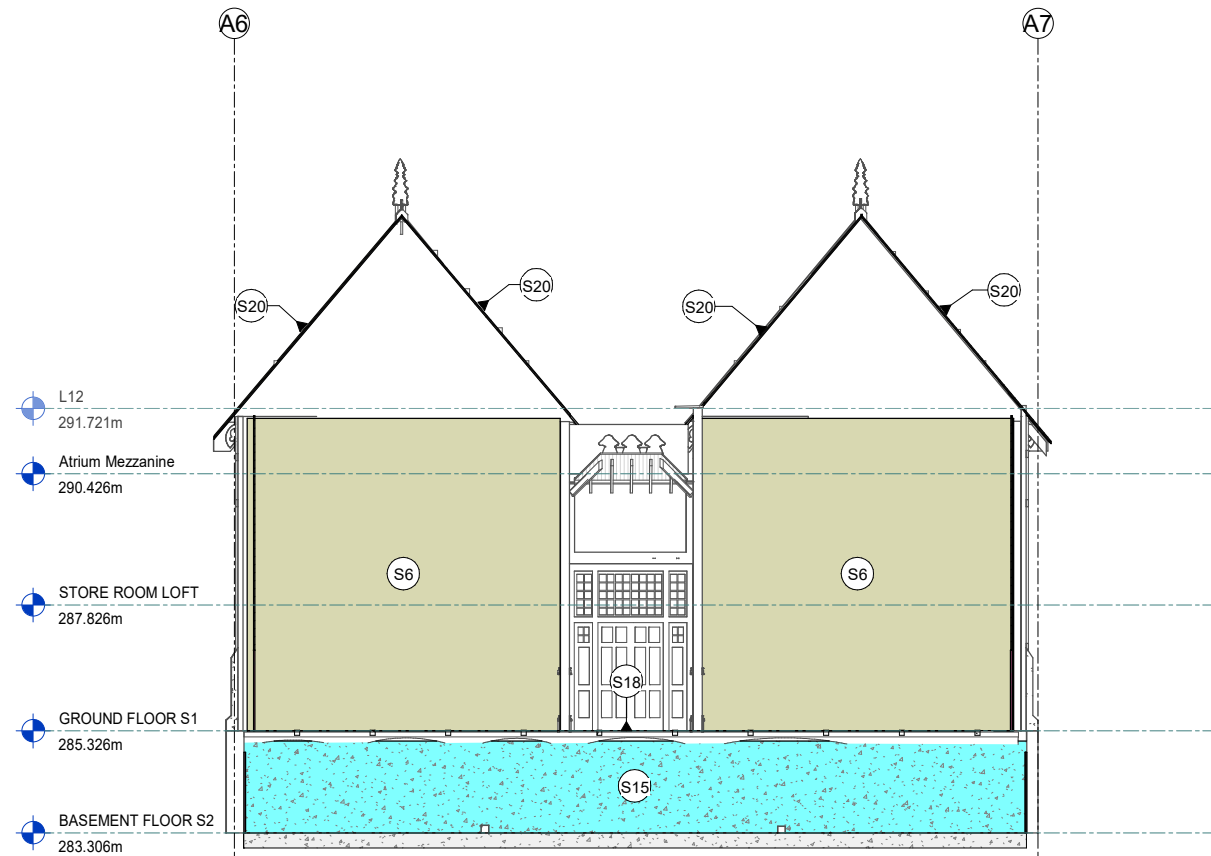
J2187

DWG No

S5-103

REVISION

A

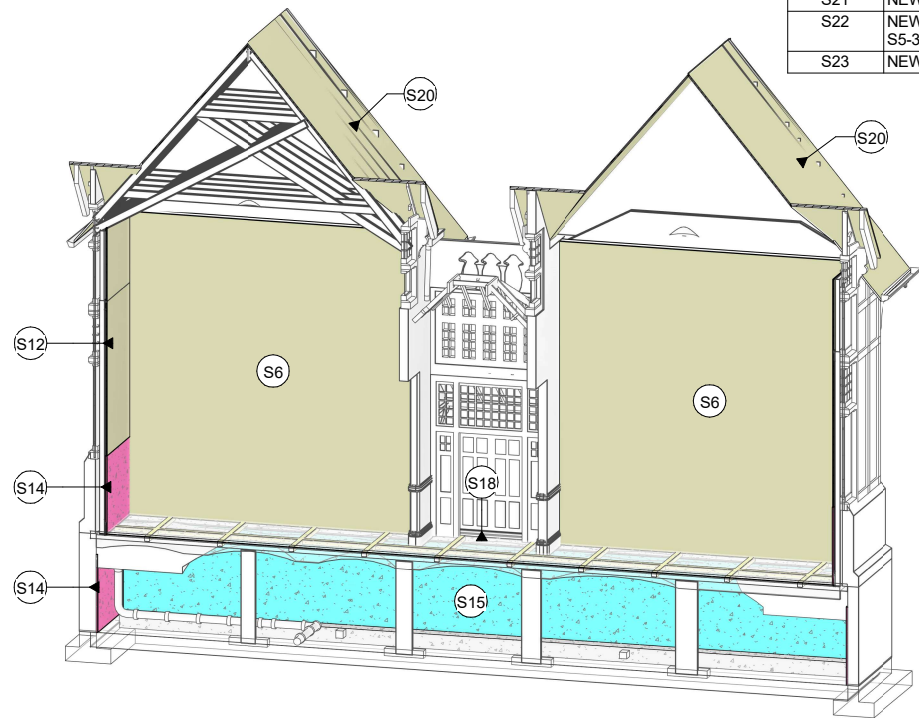


1 WEST WALL ELEVATION (GRID AA)
S5-101 1 : 75 @ A1 A3 Scale = 2 x A1



2 EAST WALL ELEVATION (GRID AD)
S5-101 1 : 75 @ A1 A3 Scale = 2 x A1

ZONE 5 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S6	NEW 150x50 TIMBER STUD WALL DIRECTLY IN FRONT OF EXISTING WALL, WITH 2 LAYERS OF 21mm PLYWOOD (1ST LAYER ORTHOGONAL TO 2ND LAYER)
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS, AND STRENGTHENING OF TIMBER FRAME TO NIB WALL CONNECTIONS AND WALL TO CEILING DIAPHRAGM CONNECTIONS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 100THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S17	NEW STEEL STRONGBACKS AT 1200 CRS, FROM BASEMENT TO ROOF LINE WITH NEW R.C STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S5-301
S19	NEW 21mm PLYWOOD SHEATHING 1.8m WIDE OVER CEILING OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING CEILING RAFTERS, WITH NEW BLOCKING TO SUIT AND NEW CONNECTIONS TO EXISTING WALL
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S21	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S22	NEW TWO-WAY BENT RHS150x100x4 BEAMS SPANNING BETWEEN EXISTING COLUMNS WITH STEEL COLLARS. REFER DWG S5-301 FOR DETAIL
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN



1A WEST WALL - 3D VIEW
@ A1 A3 Scale = 2 x A1



2A EAST WALL - 3D VIEW
@ A1 A3 Scale = 2 x A1

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PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 5

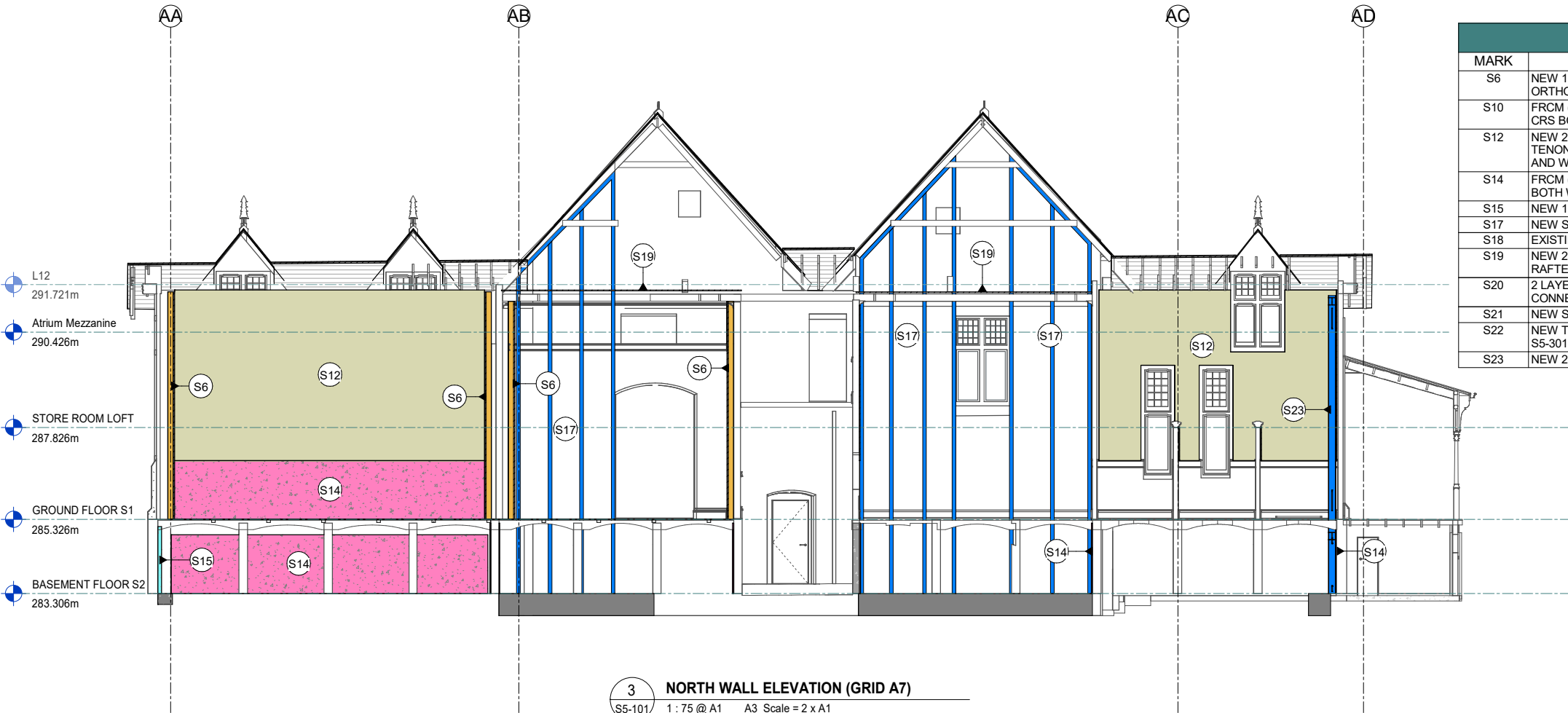
SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD

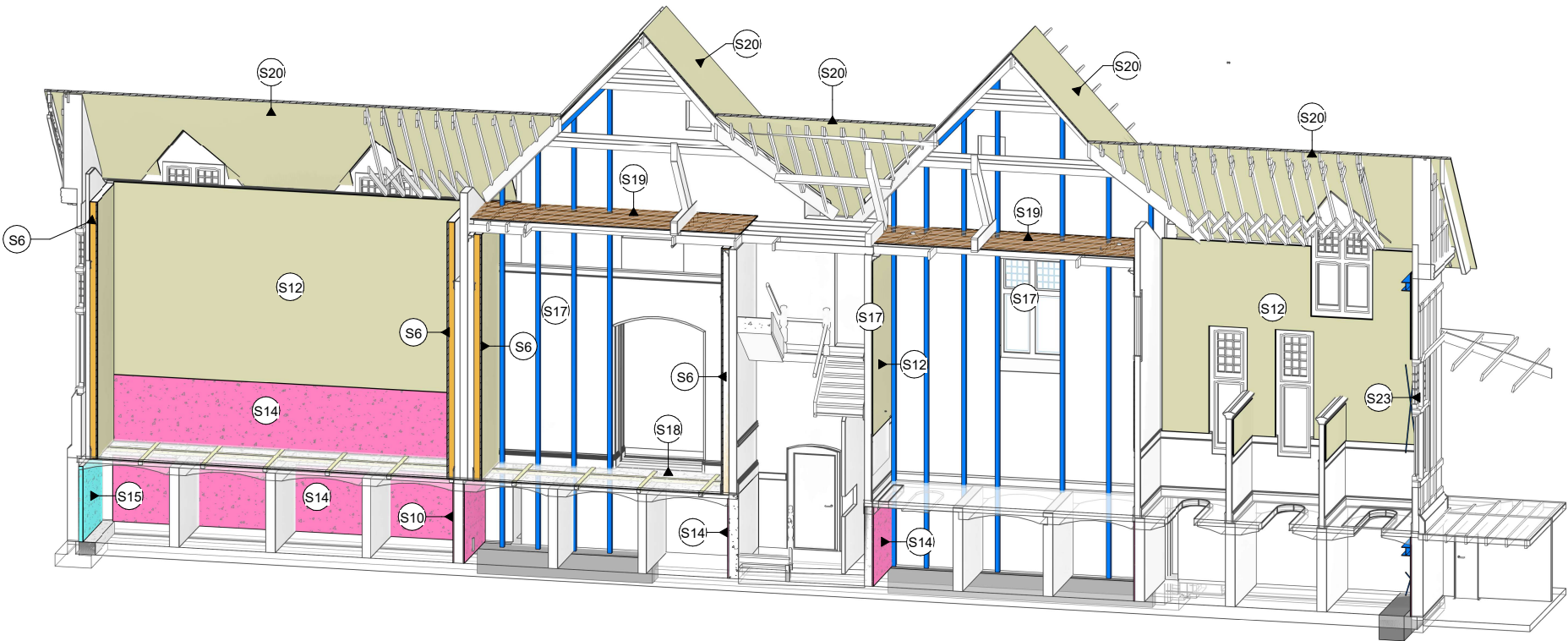
SCALE 1:75 @ A1
1:150 @ A3

JOB No.	DWG No.	REVISION
J2187	S5-201	A



3 NORTH WALL ELEVATION (GRID A7)
S5-101 1 : 75 @ A1 A3 Scale = 2 x A1

ZONE 5 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S6	NEW 150x50 TIMBER STUD WALL DIRECTLY IN FRONT OF EXISTING WALL, WITH 2 LAYERS OF 21mm PLYWOOD (1ST LAYER ORTHOGONAL TO 2ND LAYER)
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS, AND STRENGTHENING OF TIMBER FRAME TO NIB WALL CONNECTIONS AND WALL TO CEILING DIAPHRAGM CONNECTIONS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 100THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S17	NEW STEEL STRONGBACKS AT 1200 CRS, FROM BASEMENT TO ROOF LINE WITH NEW R.C STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S5-301
S19	NEW 21mm PLYWOOD SHEATHING 1.8m WIDE OVER CEILING OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING CEILING RAFTERS, WITH NEW BLOCKING TO SUIT AND NEW CONNECTIONS TO EXISTING WALL
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S21	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S22	NEW TWO-WAY BENT RHS150x100x4 BEAMS SPANNING BETWEEN EXISTING COLUMNS WITH STEEL COLLARS. REFER DWG S5-301 FOR DETAIL
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN



3A NORTH WALL - 3D VIEW
@ A1 A3 Scale = 2 x A1

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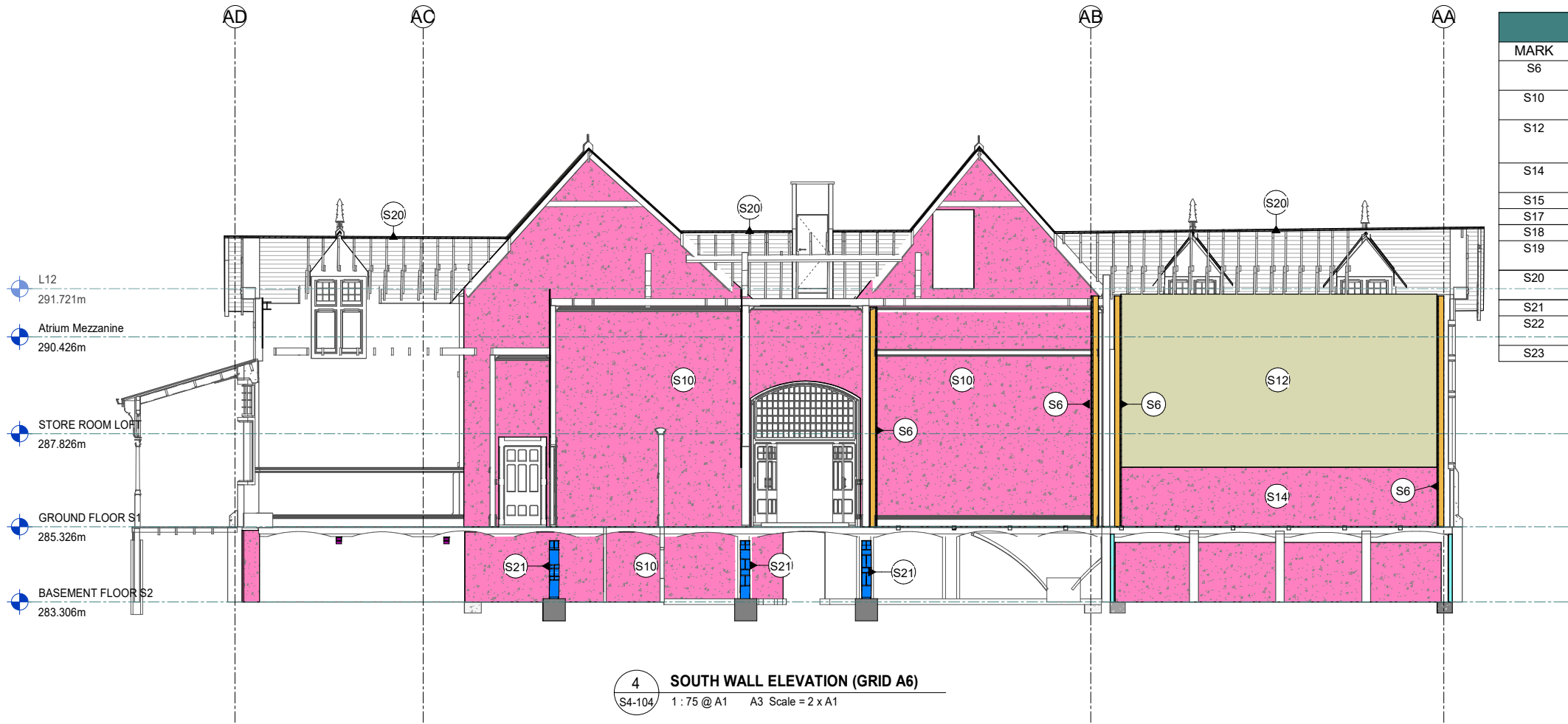
CLIENT
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Te Kaunihera o ngāi Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 5

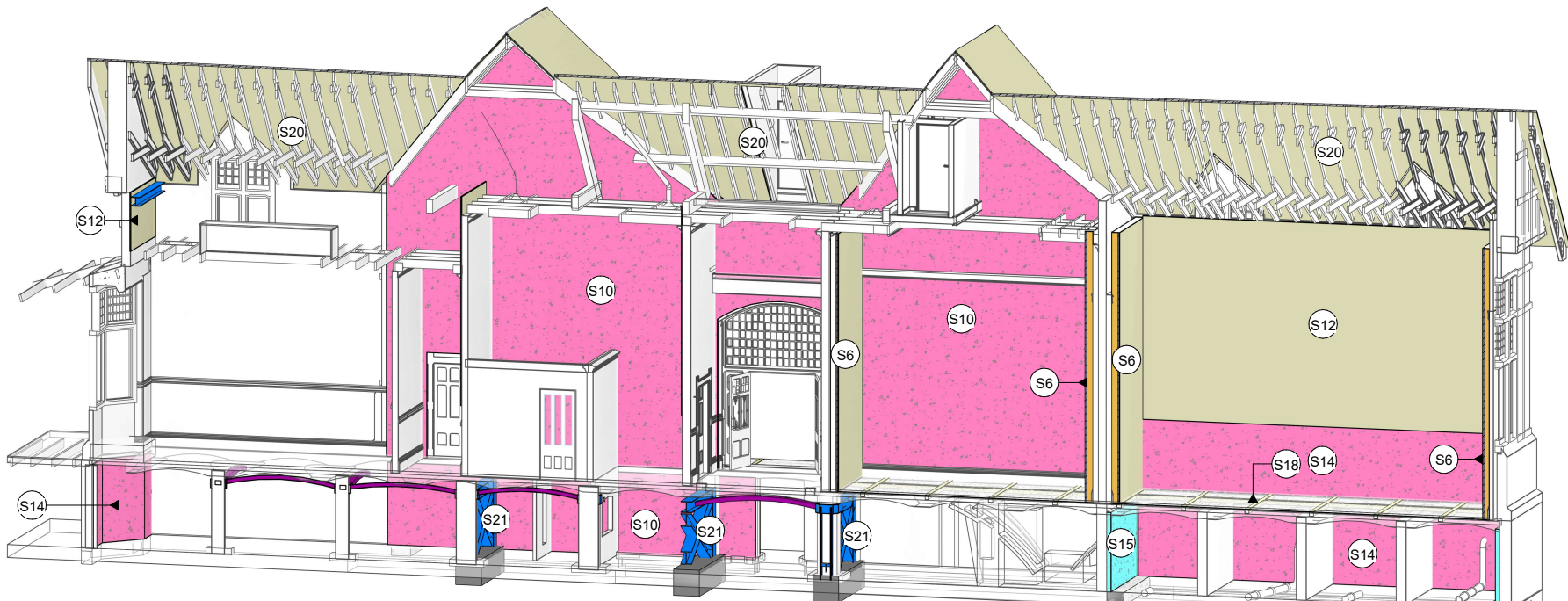
SECTIONS
PRELIMINARY CONCEPT 80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD
SCALE	1:75 @ A1 1:150 @ A3		
JOB No.	DWG No.	REVISION	
J2187	S5-202	A	



4 SOUTH WALL ELEVATION (GRID A6)
S4-104 1:75 @ A1 A3 Scale = 2 x A1

ZONE 5 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S6	NEW 150x50 TIMBER STUD WALL DIRECTLY IN FRONT OF EXISTING WALL, WITH 2 LAYERS OF 21mm PLYWOOD (1ST LAYER ORTHOGONAL TO 2ND LAYER)
S10	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO BOTH SIDES OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S12	NEW 21mm PLYWOOD SHEATHING TO EXISTING TIMBER FRAMED WALL, WITH STRENGTHENING OF EXISTING TENON-MORTISE JOINTS USING STEEL BRACKETS, AND STRENGTHENING OF TIMBER FRAME TO NIB WALL CONNECTIONS AND WALL TO CEILING DIAPHRAGM CONNECTIONS
S14	FRCM (FIBRE REINFORCED CEMENTITIOUS MATRIX) COATING TO ONE SIDE OF EXISTING WALLS WITH ANCHORS AT 600 CRS BOTH WAYS
S15	NEW 100THK RC SKIN WALL ON NEW 450 WIDE FOOTING
S17	NEW STEEL STRONGBACKS AT 1200 CRS, FROM BASEMENT TO ROOF LINE WITH NEW R.C STRIP FOOTING
S18	EXISTING TWO-WAY CONCRETE VAULTED FLOOR TO BE STRENGTHENED. REFER DRAWING S5-301
S19	NEW 21mm PLYWOOD SHEATHING 1.8m WIDE OVER CEILING OVER EXISTING MEZZANINE FLOOR JOISTS. EXISTING CEILING RAFTERS, WITH NEW BLOCKING TO SUIT AND NEW CONNECTIONS TO EXISTING WALL
S20	2 LAYERS OF 17mm PLYWOOD SHEATHING OVER EXISTING ROOF RAFTERS, AND IMPROVEMENT OF ROOF TO WALL CONNECTIONS
S21	NEW STEEL CROSS-BRACED FRAME WITH R.C FOUNDATION BEAM
S22	NEW TWO-WAY BENT RHS150x100x4 BEAMS SPANNING BETWEEN EXISTING COLUMNS WITH STEEL COLLARS. REFER DWG S5-301 FOR DETAIL
S23	NEW 200UC STEEL FRAME, WITH 25mm STEEL RODS CROSS BRACING AT LOCATIONS SHOWN



4A SOUTH WALL - 3D VIEW
@ A1 A3 Scale = 2 x A1

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Te Kaunihara o ngāi Roto o Rotorua

PROJECT
ROTORUA MUSEUM DEVELOPMENT

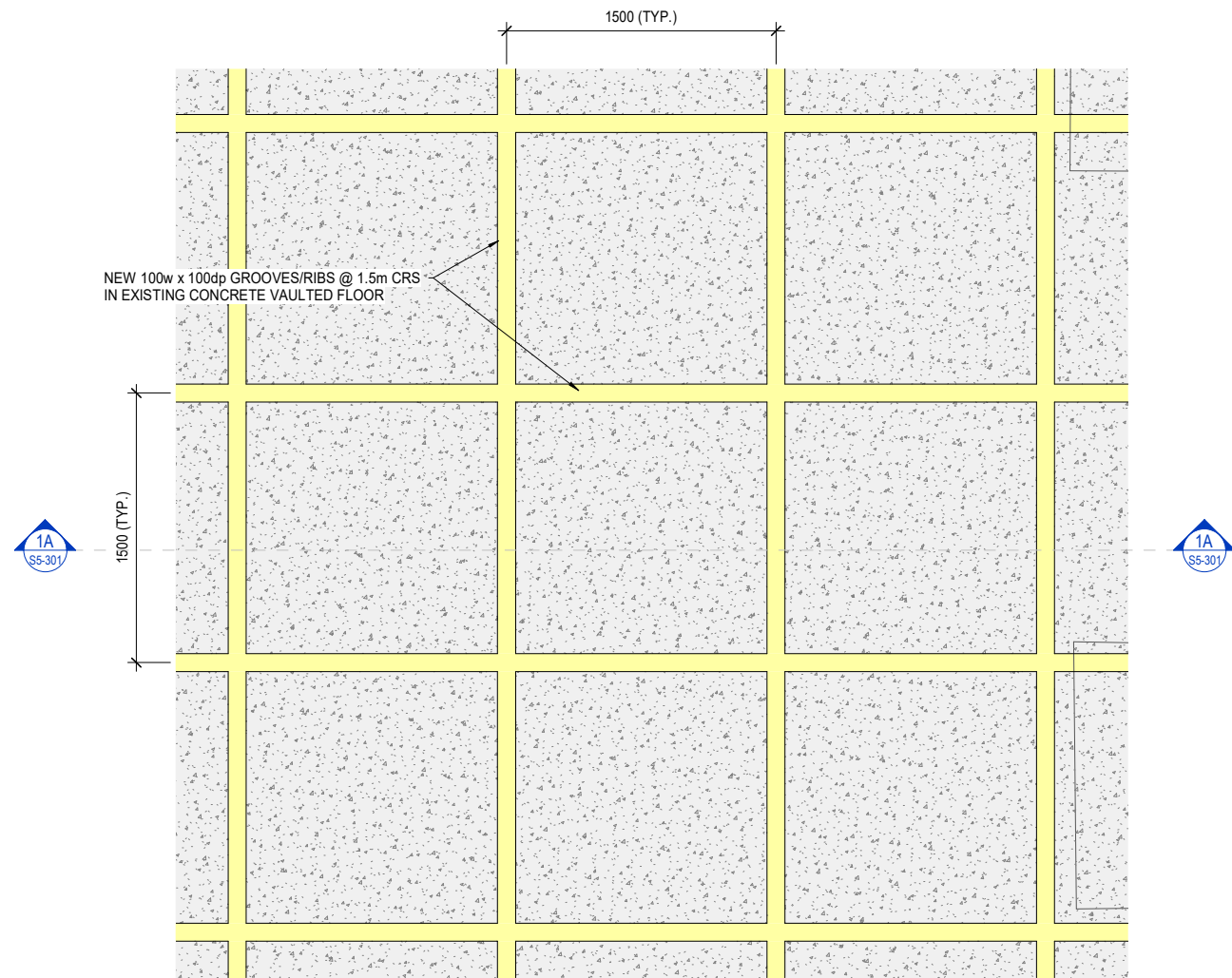
SHEET TITLE
ZONE 5

SECTIONS

PRELIMINARY CONCEPT
80% NBS (IL3)

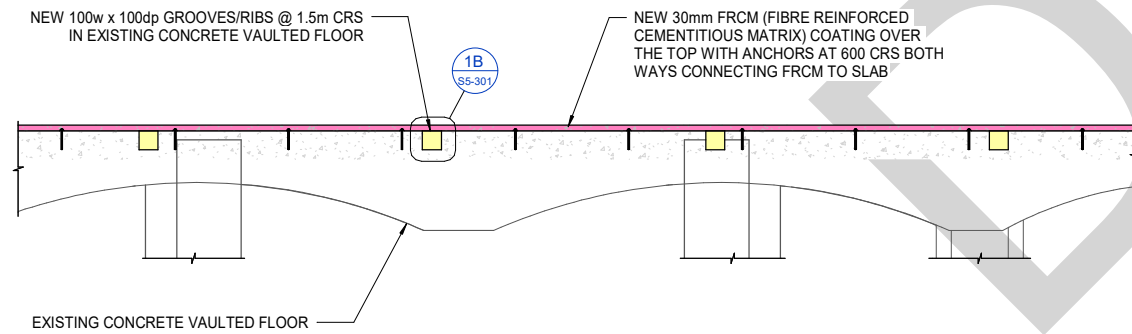
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DRAWN	KT	CHECKED	DD
SCALE	1:75 @ A1 1:150 @ A3		
JOB No.	DWG No.	REVISION	
J2187	S5-203	A	

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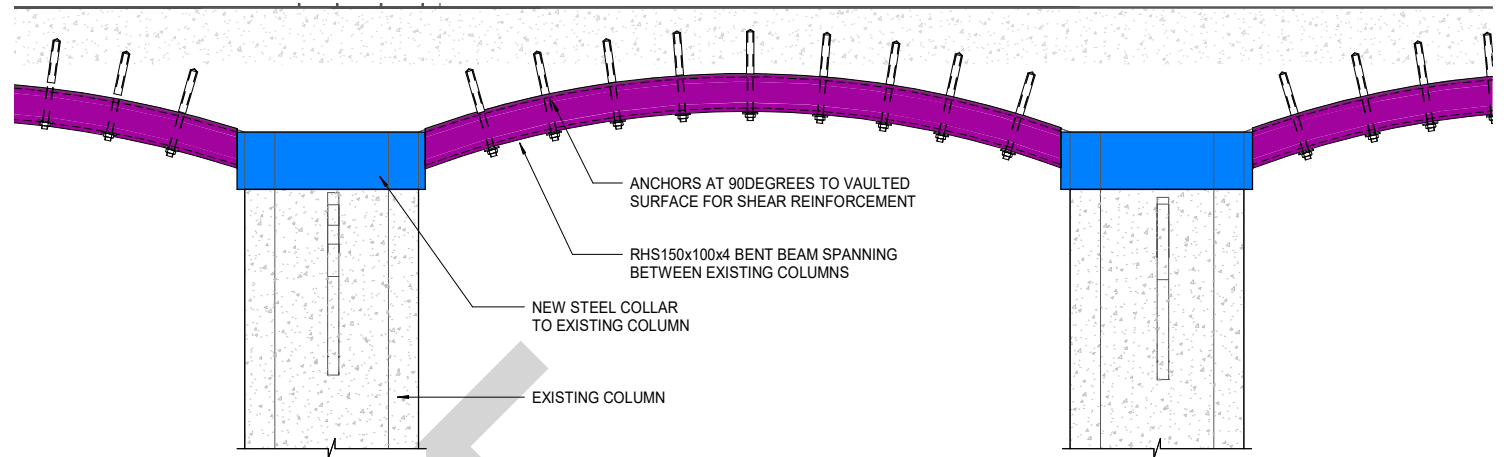


NOTE:
EXISTING CONCRETE TO BE CAREFULLY REMOVED UNDER EXISTING INTERNAL WALLS TO ENSURE RIB CONTINUITY

1 **EXISTING CONCRETE VAULTED FLOOR STRENGTHENING (TO WEST SIDE) - PLAN**
1 : 20 @ A1 A3 Scale = 2 x A1

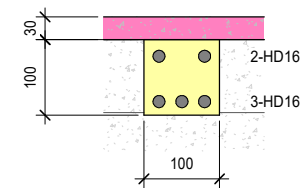


1A **SECTION 1A**
S5-301 1 : 20 @ A1 A3 Scale = 2 x A1



NOTE:
ARCHED STEEL PROFILE AND SOFFIT OF VAULTED SLAB TO BE PACKED WITH HIGH STRENGTH NON-SHRINK COMPATIBLE GROUT

2 **EXISTING CONCRETE VAULTED FLOOR STRENGTHENING (TO EAST SIDE)**
S5-101 1 : 10 @ A1 A3 Scale = 2 x A1



1B **RIB REINFORCING**
S5-301 1 : 5 @ A1 A3 Scale = 2 x A1

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PROJECT

ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE

ZONE 5

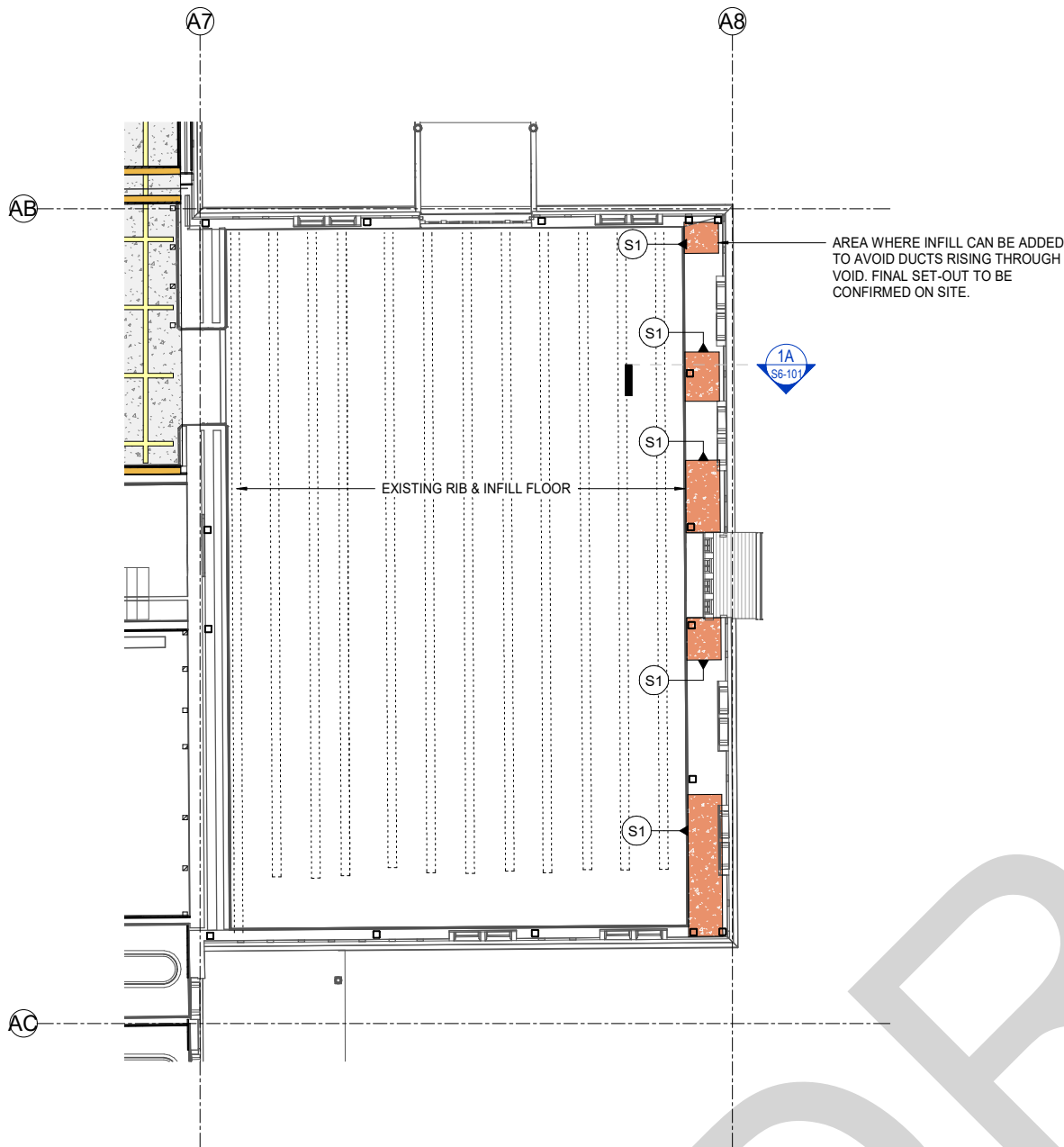
DETAILS

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DD

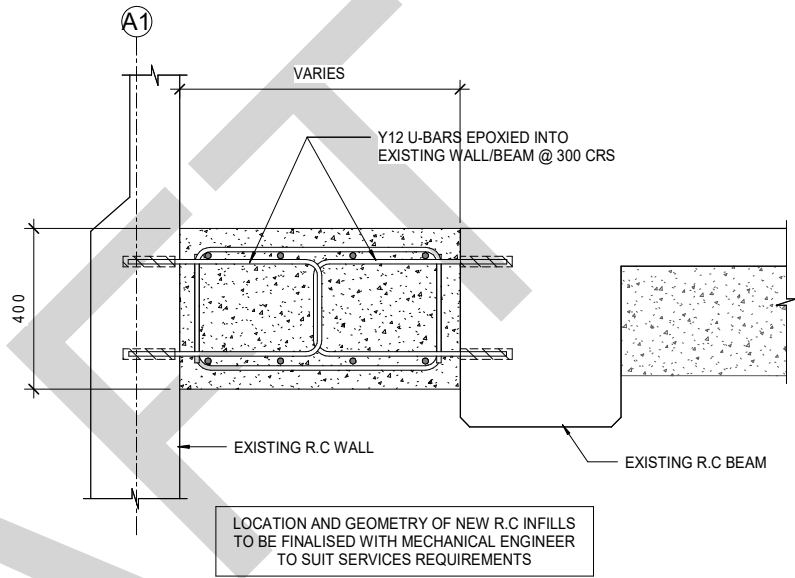
SCALE **As Noted**

JOB No.	DWG No.	REVISION
J2187	S5-301	A

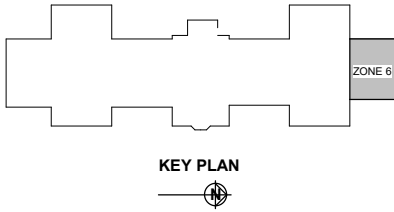


1 ZONE 6 GROUND FLOOR PLAN
1 : 75 @ A1 A3 Scale = 2 x A1

ZONE 6 - CONCEPT STRENGTHENING SCHEDULE	
MARK	DESCRIPTION
S1	EXISTING VOIDS BETWEEN THE SLAB EDGE AND WALL TO BE INFILLED WITH NEW 400 DEEP R.C INFILL



1A TYP. SECTION THROUGH NEW R.C INFILLS
1 : 10 @ A1 A3 Scale = 2 x A1



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PROJECT
ROTORUA MUSEUM DEVELOPMENT

SHEET TITLE
ZONE 6

GROUND FLOOR PLAN

PRELIMINARY CONCEPT
80% NBS (IL3)

DESIGNED	Designer	DATE	NOV 2021
DRAWN	KT	CHECKED	DS

SCALE As Noted

JOB No.	DWG No.	REVISION
J2187	S6-101	A