REDEVELOPMENT OF HERITAGE BUILDINGS IN SYDNEY: CASE HISTORY OF COMPLEX UNDERPINNING AND SUPPORT WORKS

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ABSTRACT

The redevelopment of Sydney's Central Business District has prompted a trend toward updating heritage buildings to include modern functionality while preserving their historical integrity. The Lands Department 'Sandstone' project transforms a heritage sandstone building into a luxury hotel, requiring a deep shaft excavation within the heritage building footprint, followed by a mined tunnel to connect to the adjacent heritage which is also updated.

The project geotechnical challenges included numerous rock joints due to proximity to the G.P.O. Fault Zone, resulting in rock wedges and other mechanisms that required underpinning of the heritage and new foundations around the new shaft. The presence of sensitive heritage structures with limited deformation tolerance, coupled with restricted site access, further complicated design and construction. Site investigations demonstrated the variable geotechnical conditions, prompting downhole camera and trial trench inspections to confirm the rock jointing locations with precision for underpinning design.

A key structural challenge was to support an existing five-story sandstone block wall directly above the shaft. A precast concrete beam with integrated steel members was designed as a load-transfer structure, redistributing wall loads onto new underpinned foundations. The underpinning of the new transfer beam and the other existing footings alongside the proposed shaft excavation were supported by rock bolts. To address building settlement risks, flat jacks were incorporated into the transfer structure foundations with staged excavation and design verification. Through innovative engineering solutions, and meticulous planning construction and verification, this project successfully solved the geotechnical complexities associated with the redevelopment of the heritage building, and provides a valuable precedent for future similar developments.

1 INTRODUCTION

The redevelopment of Sydney's Central Business District has prompted a trend toward updating heritage buildings to include modern functionality while preserving their historical integrity. The Lands Department 'Sandstone' project transforms a heritage sandstone building (Figure 1:) into a luxury hotel, requiring a deep shaft excavation within the heritage building footprint, followed by a mined tunnel to connect to the adjacent heritage building which is also updated.

Figure 1: Photograph of existing heritage sandstone buildings being redeveloped

Figure 2: Site locality

The site is located in the northern part of Sydney's CBD close to Circular Quay (Figure 2:), on a block bounded by Bridge, Loftus, Bent, and Gresham Streets. The four-storey sandstone building was built around 1880 and historically was occupied by the NSW Department of Lands. Until as recently as 2016 it was occupied by the NSW Department of Planning.

Some notable historical features of Lands Department building comprise:

- The Victorian-style building façades, crafted from sandstone blocks and adorned with classical sculptures and arched windows
- A significant feature on the Bridge Street façade is the Lands Department Datum Benchmark Plug, which served as the origin for all survey levels in New South Wales
- The central clocktower rising to a height of 5 storeys
- Moat walls surrounding the perimeter of the building basement on Gresham, Bridge and Loftus Street to provide essential light and ventilation to the basement

2 SITE CONSTRAINTS

The redevelopment of the building into a luxury hotel required a 10 m deep shaft to be excavated within the existing building footprint. This shaft was to provide "back of house" functionality to the hotel, as well as access to the new tunnel underneath Loftus Street that connects into the adjacent redeveloped heritage building. A series of precast slabs are to be placed over top of the shaft to provide continuity of the hotel ground floor.

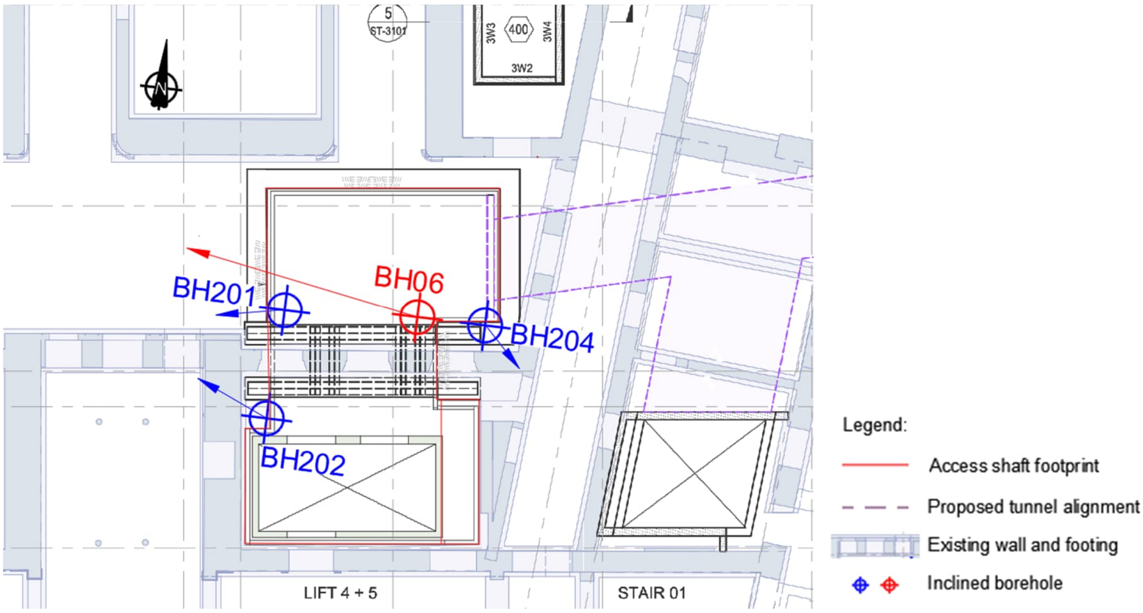


Figure 3: Site plan

Physical constraints to the shaft include:

- An existing four storey / five-level sandstone block wall with window openings, spanning directly over the shaft location.
- Other sandstone block walls surrounding all four sides of the shaft; the bearing pressures beneath these walls are in the order of 500 kPa (i.e. due to the weight of up to 25 m height of stacked sandstone blocks!).
- Adjacent to the western side of the shaft, a clocktower that extends above the top of the building (Figure 1:) and imparts a bearing pressure of over 1000 kPa.
- The above-mentioned walls and clocktower are all founded on one to two rows of unconnected roughly (hand) cut sandstone blocks in the order of 1 m long and 0.5 m wide and high, stacked on top of each other and founded on the underlying insitu bedrock, with nearly zero offset to the excavation face (Figure 3:).
- The new shaft ceiling slabs also impose loads around the edge of the shaft due to the self weight of the concrete and also the soil and trees of the planter boxes contained on the slabs.

Each of the heritage elements adjacent to the shaft had tight limits on total and differential movements, particularly given their construction from sandstone blocks.

Site constraints for construction meant that there was very limited access to machineries and most of the work had to be undertaken using small plant.

3 GEOTECHNICAL MODEL

3.1 GEOLOGICAL SETTING

Sydney CBD is underlain by Hawkesbury Sandstone (Pells et al, 2019), which is typically a medium to high strength horizontally bedded sedimentary rock unit with either massive or cross-bedded fabric. At this site, the main geological feature of relevance is the "G.P.O. Fault Zone" which is a zone where sub-vertical joints striking NE-SW are expected to be encountered more frequently (Figure 4:). It is important to note that this does not mean that the entire width of the G.P.O. Fault Zone as mapped (Och et al (2004), Pells et al (2004)) contains closely spaced joints. Likewise, it also does not mean that conditions return to "normal" immediately beyond the extent of the G.P.O. Fault Zone as mapped, simply that the prevalence of sub-vertical joints will gradually decrease.

At this site and around the Sydney CBD there is also extensive prior land use resulting in variable depth and character of fill. At this site, there were historic excavations for pipes, footings, pits, etc that either contained fill or were simply voids full of water. However the natural soil profile had been essentially removed, meaning he fill directly overlay the sandstone bedrock.

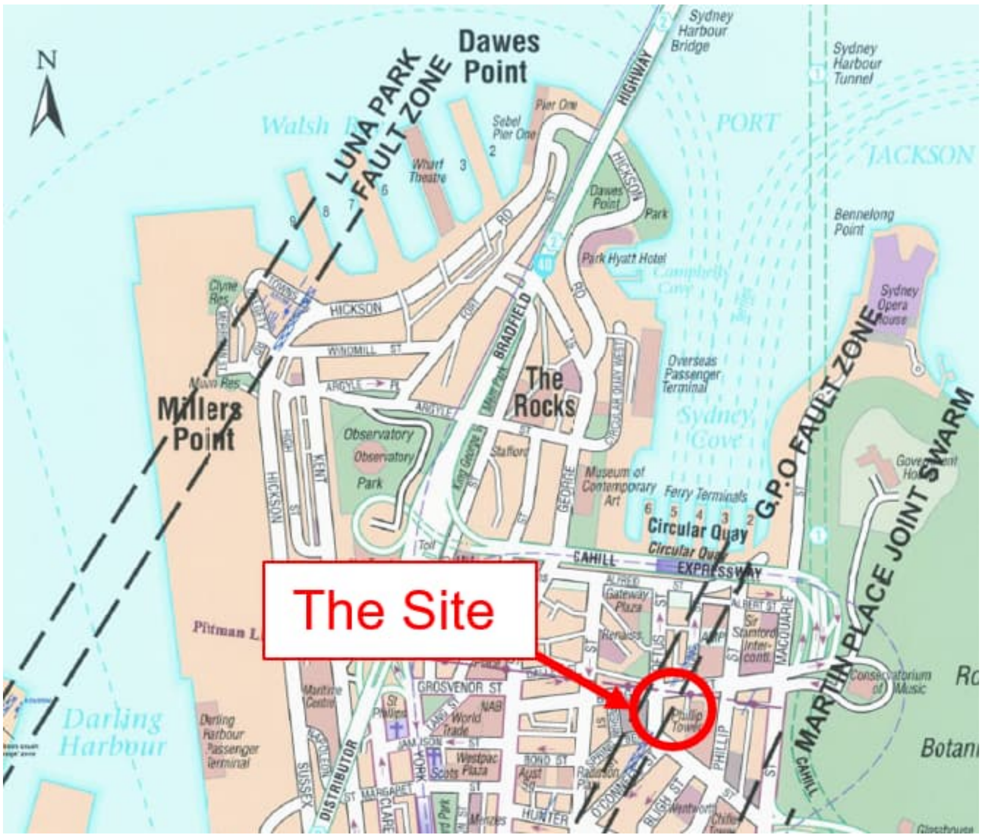


Figure 4: Geological setting (from Och et al 2004 and Pells et al 2004)

3.2 SITE INVESTIGATION

Based on the expectations of the geological setting, a series of inclined boreholes (Figure 3:), ranging from 5 to 7.2 m at 70° inclination angle, were undertaken, particularly to assess the presence of NE-SW striking defects in areas of most relevance to the design. Downhole camera inspections (Figure 5:) were undertaken within the boreholes to provide further clarity on the insitu condition of the defects.

The rock mass encountered consisted of typical medium strength sandstone with some localised low strength rock occasionally interbedded with the rock mass. A 20 to 40 mm thick, sub-horizontal clay seam was observed intersecting the deep boreholes. This clay seam separated two distinct lithologies. The upper sandstone unit was less weathered and of low to medium strength, while the underlying unit was moderately weathered with iron staining and was of medium to high strength.

Downhole camera inspections using an endoscope camera identified two joint sets within the cored boreholes. The first set was sub-vertical with a strike direction of NNE to NE, consistent with the orientation of the nearby GPO fault zone. These joints were closely spaced, with an average spacing of 0.1 to 0.2 m. The second joint set was also steeply dipping, striking NW-SE and widely spaced.

During the detailed design, it was collectively decided between the geotechnical engineer and contractor to undertake trial trenches within the shaft near its perimeter. The main purpose of the trial trenches was to further increase the certainty of the precise location of defects prior to construction, such that the design could be almost completely prescriptive and not have to respond to changes in conditions encountered during construction. The trial trenches, excavated approximately 1 m deep into sandstone bedrock (Figure 6:), confirmed the persistence and orientation of the NNE-NE striking joint set on both sides of the sandstone block wall to be underpinned. The projection of this joint set indicated a potential intersection with the eastern rock pillar designated to support the new transfer beam footing and extending behind the tunnel portal face. Every defect intersecting the top part of the shaft wall was encountered and precisely mapped, to inform the design.

The outcome was an essentially deterministic geotechnical model (Figure 7:) which was used to show the locations of all of the defects with respect to the proposed underpinning and ground support works. Most of the sub-vertical defects were concentrated within two distinct zones that traversed the shaft and underlay the proposed rock pillars for the underpinning.

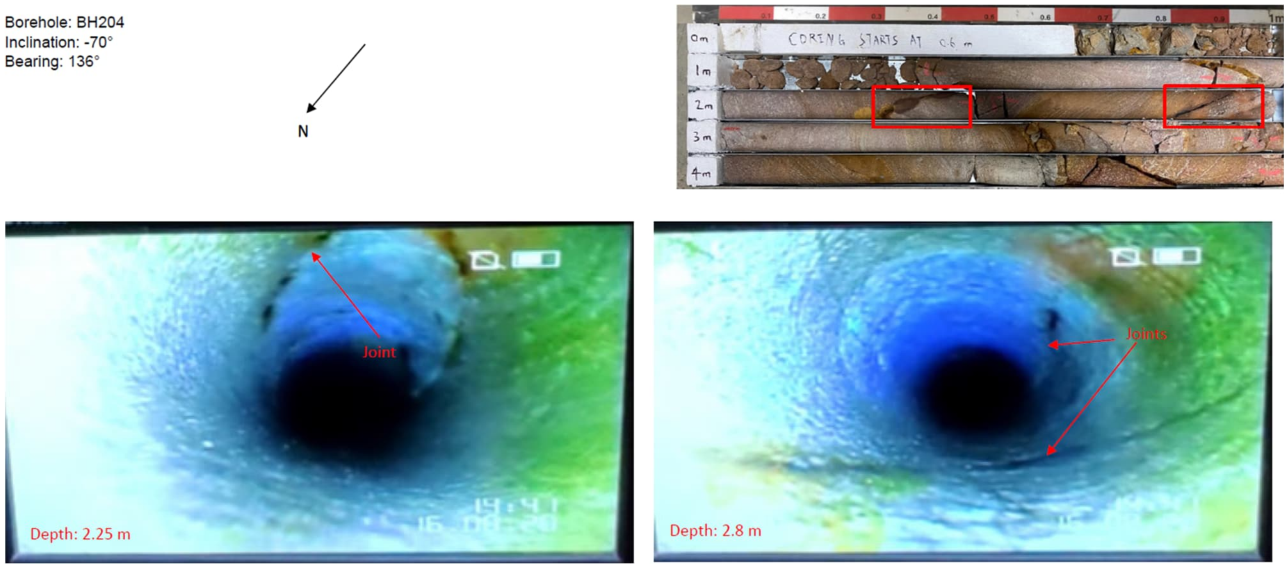


Figure 5: Typical borehole conditions – core photo (top) and endoscope inspection records (bottom)

Figure 6: Typical jointing observed in trial trenches, refer to the blue hatched zone in Figure 7

3.3 IMPLICATIONS FOR DESIGN AND CONSTRUCTION

With the site investigations having confirmed the geotechnical conditions, the design implications of the presence of the sub-vertical jointing within the Hawkesbury Sandstone included:

- Requirement to provide support to potential rock wedges, unconfined compressive mechanisms within the rock substance, and other mechanisms beneath the heritage walls around the shaft perimeter.
- Requirement to provide support to potential large-scale rock wedges and/or planar sliding and/or toppling mechanisms occurring within the narrow rock pillars underneath the underpinning pad footings.
- Requirement to provide support to potential rock wedges above the tunnel crown.
- Consideration of the effects of excavation-induced stress relief, although the shaft was highly three-dimensional which was expected to result in minimal ground deformations.

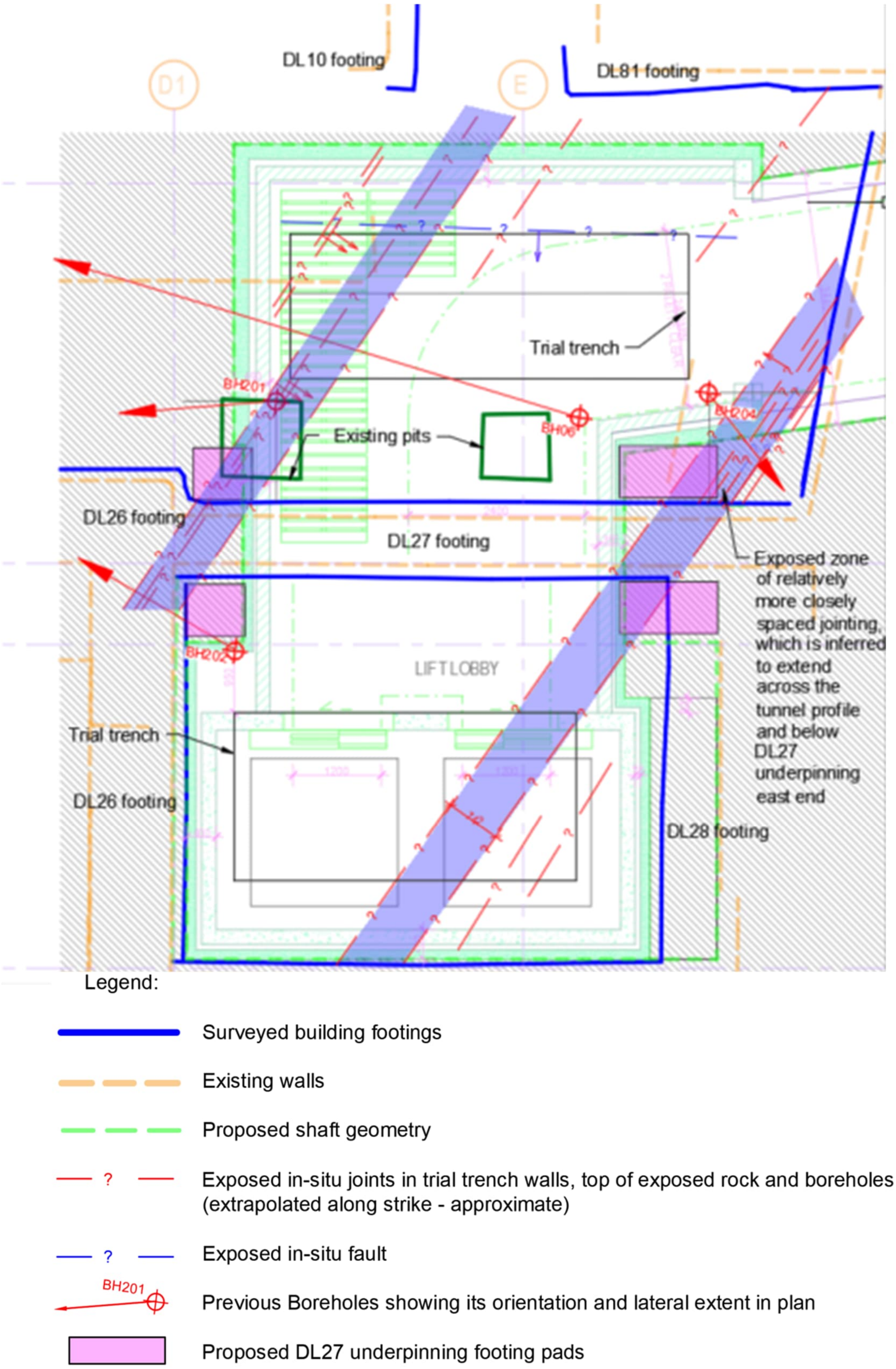


Figure 7: Geotechnical model plan

4 DESIGN SOLUTION

4.1 TRANSFER STRUCTURE

The structural solution for transferring the load of a five-storey building wall arching over the proposed shaft excavation consisted of a pair of prefabricated steel underpinning beams (7 m long 940x400 custom steel I beams encased in cast in situ concrete), positioned longitudinally on either side of the existing wall toe, integrated with the transverse mini-niche beams (310 UC 158) through the bedrock immediately beneath the sandstone block footings. These elements were designed to span across and be found on rock beyond the shaft footprint as a load-transfer structure, ultimately redistributing wall loads onto new underpinned foundations. The structural details were designed by the structural engineer, refer to Figure 8: for the typical cross-section of the mini niche beam to connect between longitudinal steel members on each side of the existing wall.

The transfer structure was required to control ground deflection of under span/300 to reduce the cracks to the heritage wall, along with the subsequent excavation. To address building settlement risks, flat jacks were incorporated into the transfer structure foundations to offset the anticipated building settlement associated with the shaft excavation and transfer of loads via the underpinning beams onto the new footings and rock pillars. The integrated jacking system was designed to accommodate up to 25mm vertical and 15mm lateral convergence as a result of the relief of stress concentrations in the rock mass, known as rock mass relaxation. In Sydney CBD, the lateral movements in rock at the surface around the excavation perimeter have usually ranged between 0.5mm and 1mm per meter depth of rock excavation (Pells (1990)). It was anticipated that the stress relief movements from the shaft excavation would be at the lower end of precedent (i.e. no greater than 0.5 mm per metre), given its proximity to the ground surface and its relatively small excavation footprint in plan, creating a three-dimensional effect that helps resist rock relaxation.

The construction sequence and support strategy are crucial to the success of load transfer under the strict ground deformation limit. The geotechnical and structural engineers worked collaboratively with the contractor and the structural designer to integrate the constructability, structural and geotechnical capacity. Refer to Section 5 – Construction Sequence for further details.

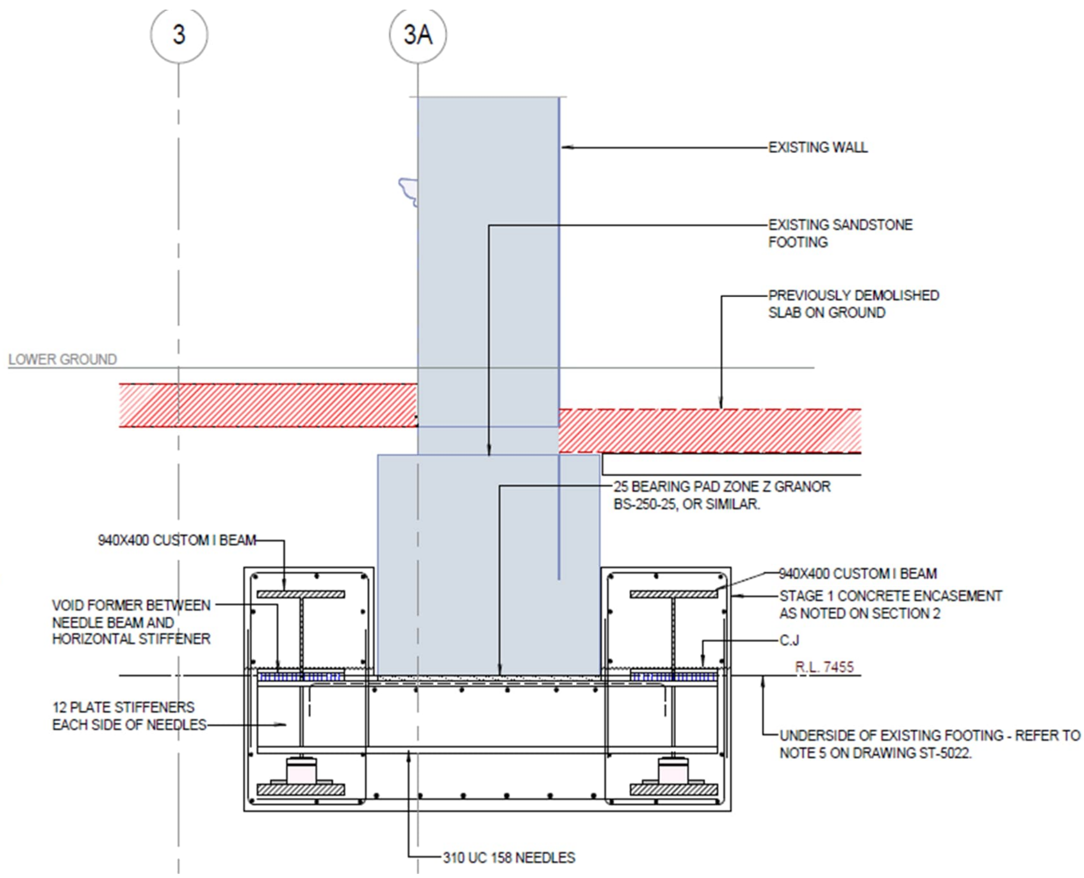


Figure 8: Typical cross section showing the connection between the mini niche beam and longitudinal steel members

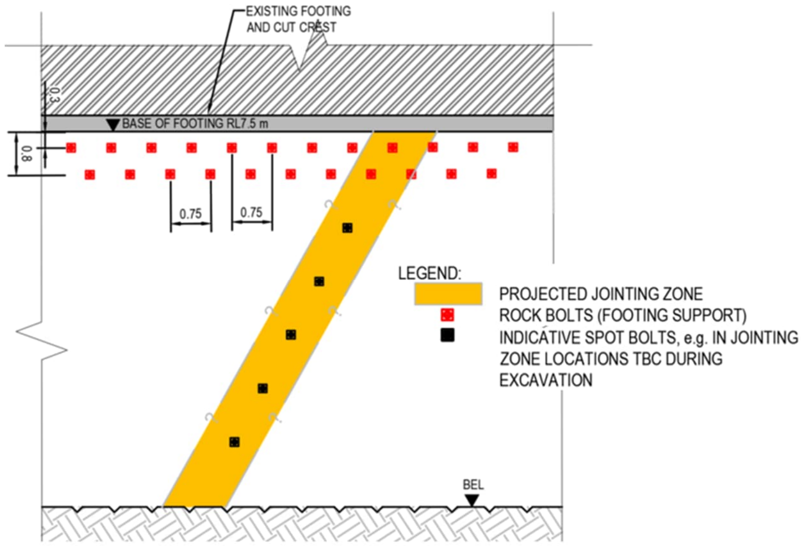
4.2 UNDERPINNING

Due to the potential for various failure mechanisms of the rock face as surcharged by the footings of the existing walls, including planar, wedge, toppling, and rock mass mechanisms, underpinning was required for both the existing walls and of the more highly loaded new transfer structure footings. Typical loads of the existing walls ranged from 380 to 1670 kN/m, whereas the loading on the new transfer structure footings was 720 kPa. The underpinning system consisted of multiple rows of Glass Fibre Reinforced Plastic (GRP) bolts, 40 mm diameter solid bars with bolt lengths ranging from 3.5 to 6 m. The bolts were designed to secure against potential instability under the applied footing loads, including:

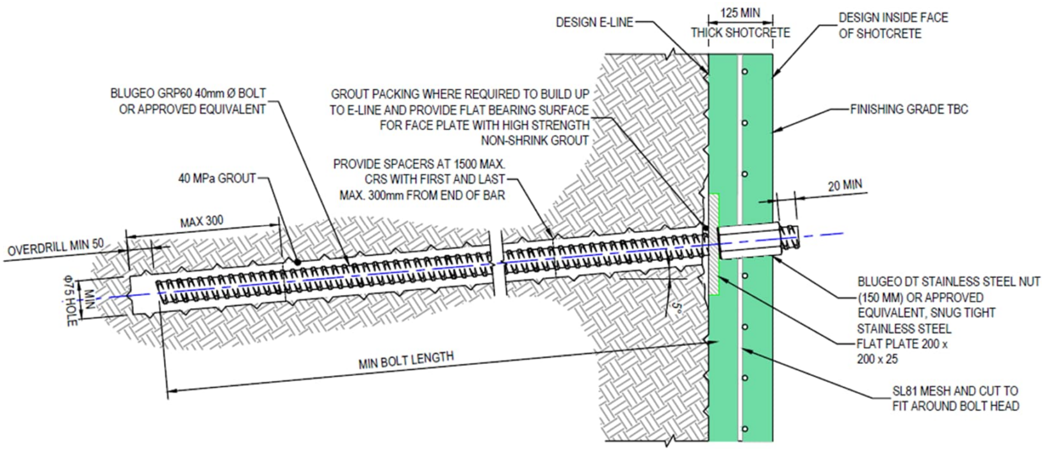
- Planar sliding along the moderately to steeply dipping NE-SW striking joints as a basal plane daylighting above the wall toe,
- Two-sided wedges formed by the intersection of two distinct and oblique joint sets daylighting at the cut face (e.g. NE-SW and NW-SE striking joints), and
- Toppling failures occurred at the crest of the cut face induced by the sub-vertical joints dipping into the wall.

The design was governed by the rock bolt pull-out capacity, defined as the minimum of three values: the grout-to-rock bond strength, the bolt-to-grout bond strength, and the tensile capacity of the GRP bar. See Figure 9: for the typical pattern bolt layout and typical shotcrete and GRP bolt details to underpin the existing wall footings adjacent to the shaft footprint.

For the rock pillars beneath the new transfer beam, a specific pattern of rock bolts was designed to be installed from both sides of the pillar. This bolt arrangement was designed to restrain the largest potential planar mechanisms and wedges identified as being formed by the persistent NE-SW striking joint swarm (see Figure 10: for configuration details), as well as smaller wedges and toppling mechanisms.



a)



b)

Figure 9: (a) Typical underpinning bolt layout and (b) Typical shotcrete and GRP bolt details

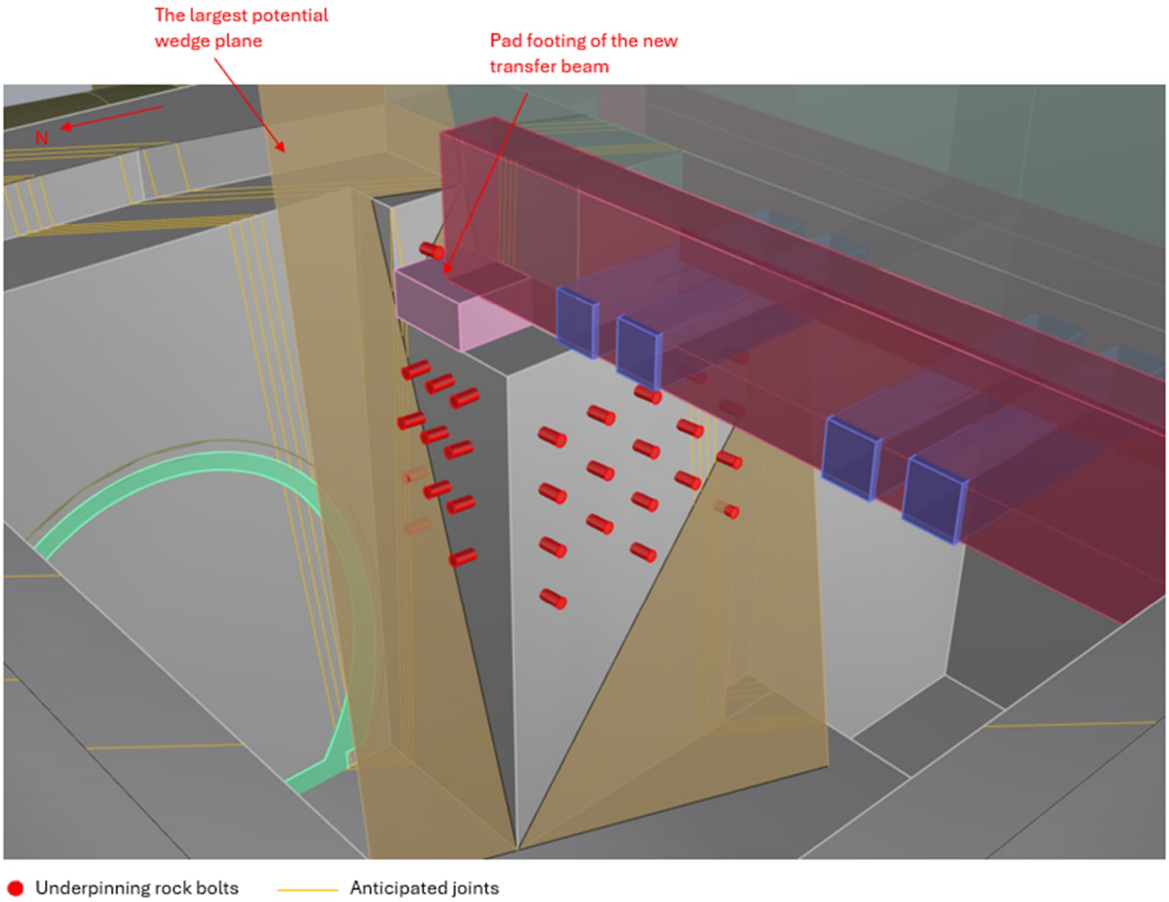


Figure 10: Underpinning design model (NB – 3D image of the pillars, jointing, and bolts)

The shaft walls were finished with a shotcrete facing for architectural purposes, and this also met the 100-year design life requirement. This high-performance finish was also specified to meet the durability requirements for the hotel basement. Although no significant groundwater ingress was anticipated, regularly spaced strip drains were installed behind the shotcrete, which was designed to capture any trapped water and divert it to a cavity drain at the toe of the wall, preventing hydrostatic pressure buildup behind the shotcrete wall.

4.3 TUNNEL PORTAL

The integration of the shaft and tunnel portal designs and construction sequence was critical for maintaining face stability during the tunnel excavation beneath existing building footings. The design approach was to provide permanent face support for the shaft wall as well as temporary support for the potential planar wedges anticipated to develop above the tunnel crown prior to installing the permanent tunnel lining.

Site investigation indicated ground conditions at the north-south striking tunnel portal comprised Sandstone Class III overlying Sandstone Class I/II with closely spaced sub-vertical joints striking NE-SW identified behind the tunnel portal. These adversely oriented joints were considered kinematically critical to the tunnel portal, potentially forming the basal plane of large scale planar wedges underneath the existing building footings, especially upon the tunnel breakthrough, which would undermine the wedge toe (see Figure 11:). The underpinning system consisted of multiple rows of double corrosion protection (DCP) steel bolts, 24 mm diameter solid bars with bolt length of 4 m, installed in a 75 mm diameter borehole. The bolts were installed beyond the basal planes and at a slightly inclined bolt angle to increase the resisting force component along the basal planes.

The design philosophy considered both a best guess and a worst-case scenario, aiming to support the largest kinematically feasible wedge defined by geometrical constraints. The design solution featured four rows of inclined double corrosion protection rock bolts installed at the portal face and embedded through the anticipated back release plane, designed as a cantilever system. The group of rock bolts were designed to provide adequate net pull-out capacity and shear strength to support the weight of the planar wedge and its associated footing loads. In addition, given the limited deformation tolerance underneath the existing footing, a three-dimensional (3D) numerical analysis was performed to assess the potential deformation of the wedge and overlying footings during staged tunnel excavation. The calculated footing movement was within the design tolerance.

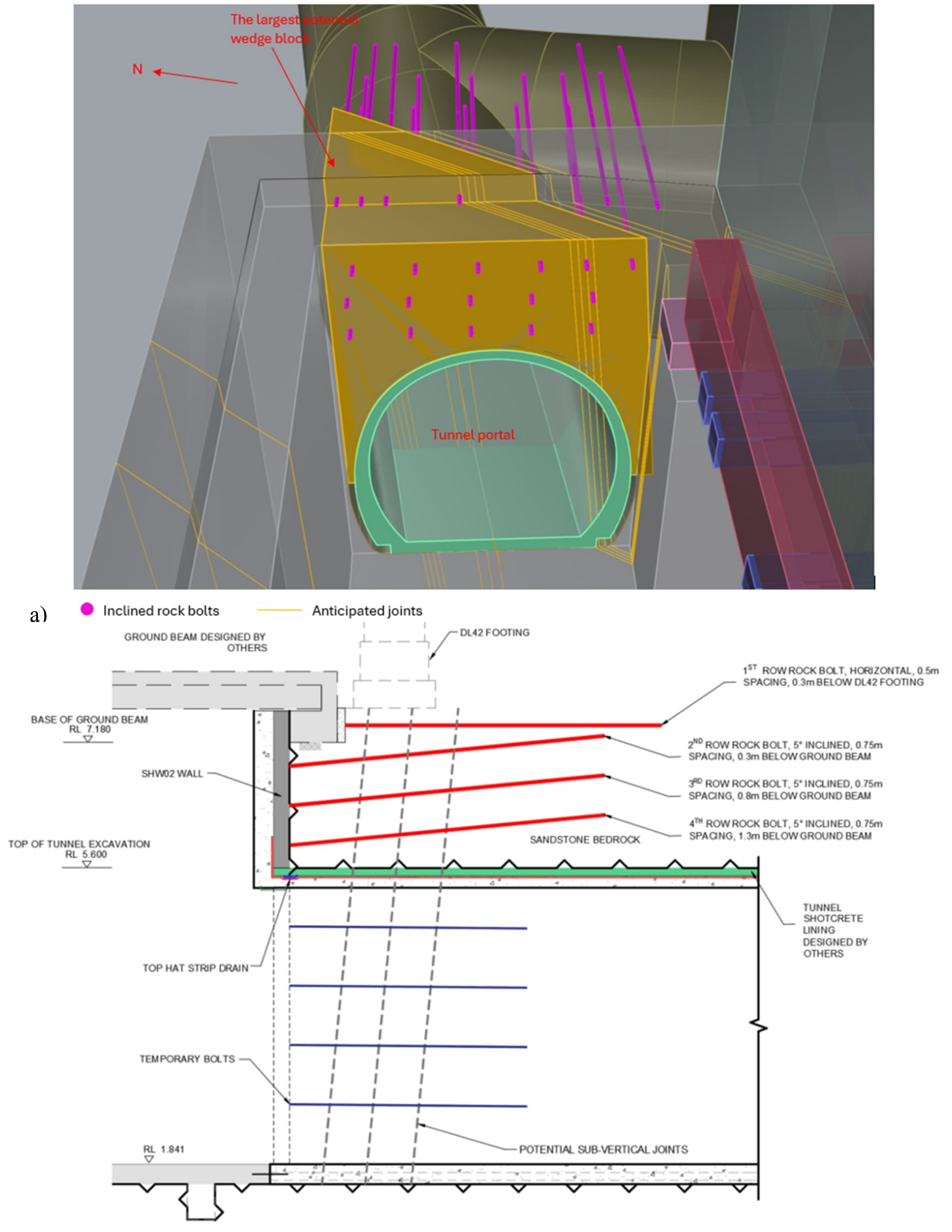


Figure 11: (a) Tunnel portal design model (NB – 3D image of the portal, jointing, and bolts) and (b) bolt layout along the tunnel profile

During shaft excavation, further verification of the tunnel portal ground model was undertaken to reduce the uncertainty regarding potential planar wedges. Borehole camera inspections within selected rock bolt drillholes were conducted to observe the presence of in-situ joints behind the shaft walls and confirm the extent of the potential wedge. It was confirmed that the wedge basal plane striking NE-SW behind the tunnel portal was present and consistent with the geological models adopted in the design, however, the anticipated east-west striking defect set, crucial for acting as a side release plane, was not positively identified. The absence of this side release plane implied that breakout through the sandstone rock mass between the tunnel portal and the basal plane would be necessary to form a planar wedge. This significantly reduced the assessed likelihood of wedge instability and consequently, reduced the minimum support required on the shaft wall.

5 CONSTRUCTION SEQUENCE

The construction sequence for the access shaft, particularly the installation of a transfer beam to support existing building structures while minimising ground deformation during subsequent bulk excavation, demanded multidisciplinary integration (constructability, structural and geotechnical). The sequence adopted for the access shaft excavation was:

1. Early Works
   a. Localised exposure and inspection of existing wall footings adjacent to the proposed shaft excavation to verify the as-built geometry and condition of the sandstone block footings and the nature of the underlying foundation materials.
   b. Excavation of trial pits within the shaft footprint to further investigate the extent and characteristics of the identified sub-vertical jointing prior to bulk excavation.
   c. Assessment of the temporary bearing capacity of existing footings that would be exposed during underpinning, informing the staged excavation and support strategy.
   d. Installation of monitoring prisms on existing structures and acquisition of baseline readings prior to commencing bulk excavation.

2. Transfer Structure Installation and Underpinning
   a. Temporary bracing of existing walls and stitching of the sandstone block footings to maintain their integrity during staged excavation.
   b. Localised box cuts at both ends of the existing wall to facilitate the installation of reinforced concrete pad footings for the proposed transfer beam.
   c. Controlled excavation of two parallel mini trenches within the sandstone bedrock immediately adjacent to and extending beneath the sandstone block footings, see Figure 12:.
   d. Installation of paired prefabricated steel members, positioned longitudinally alongside either side of the existing wall toe, followed by the insertion of transverse mini-niche beams through the openings created in the mini trenches to structurally connect these longitudinal steel members beneath the wall.
   e. Installation of flat jacks at the base of the mini niche beams and the transfer beam pad footings to actively control ground movement and pre-load the system (i.e. achieve a new load path for the wall into the ground via the new footings), thereby mitigating movement of the existing wall during subsequent excavation, see Figure 13:.
   f. Installation of specific underpinning (e.g. rock bolts, shotcrete) for the sandstone rock pillars supporting the transfer beam pad footing, undertaken prior to breakthrough excavation beneath the completed transfer beam.
   g. Staged breakthrough excavation beneath the completed transfer beam with real-time ground deformation monitoring and adjustment of flat jacks as required to control the ground movement
   h. Concrete encasement for the transfer structure upon verification of negligible ground deformation from the ongoing monitoring data following the shaft excavation to half of its planned depth, see Figure 14:.

3. Main Shaft Bulk Excavation (Post Transfer Beam Installation)
   a. Staged excavation of the remaining shaft on both sides of the completed transfer beam. Perimeter saw-cutting was employed along excavation boundaries adjacent to sensitive footing structures to minimise ground vibrations and disturbance to existing footings.
   b. Strict control of excavation lift heights (e.g. typical 1.0m lifts in the upper portions of the shaft), followed by immediate geotechnical inspection and mapping of the exposed cut face after each lift to verify the encountered ground conditions and adjust ground support if necessary.
   c. Systematic installation of ground support (e.g. rock bolts and shotcrete) to ensure the stability of the exposed sandstone rock face.
   d. The sequence of staged excavation, face inspection and support installation was repeated iteratively until the finished bulk excavation level was reached, see Figure 16:.

4. Tunnel Portal Works
   a. Implementation of a similar sequence to the main shaft bulk excavation.
   b. In addition, targeted borehole camera inspections were conducted within pre-drilled rock bolt holes at the designated portal location to further investigate in-situ jointing and confirm the geometry of potential planar wedges, supplementing earlier site investigation data.

c. The designed pattern bolting of four rows of inclined rock bolts was installed to stabilise anticipated planar wedges at the tunnel portal. Field adjustments to rock bolt locations were made where necessary to avoid clashes with pre-existing in-ground service trenches.
d. Upon completion of shaft excavation, careful staged excavation and installation of the primary tunnel support for the initial tunnel section commenced beneath the existing footings.
e. Subsequent works included the installation of permanent secondary lining and integration of structural connections and drainage systems between the shaft walls and the tunnel lining.

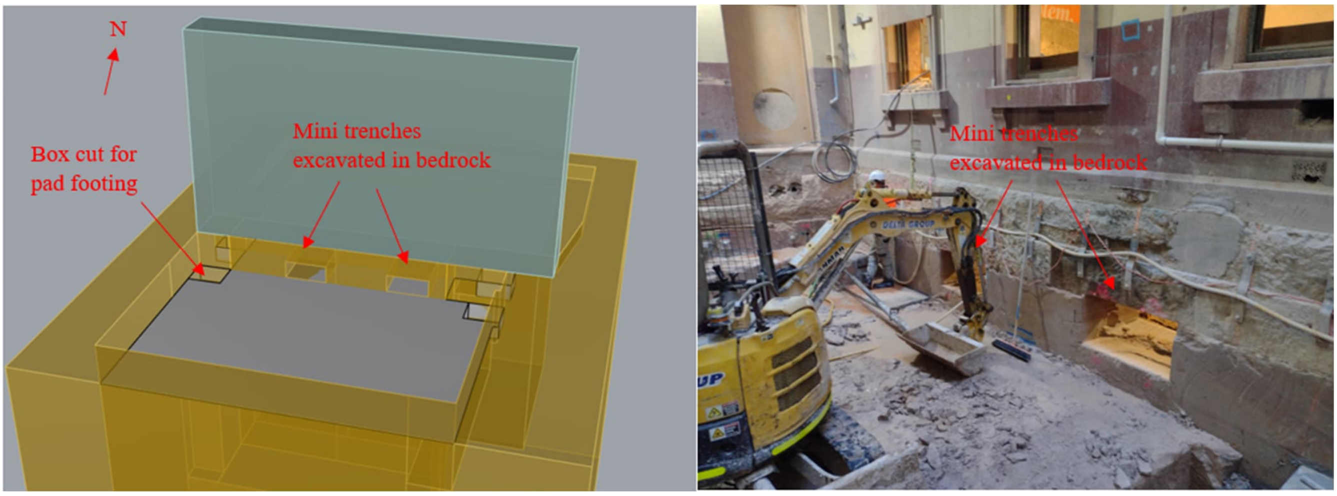


Figure 12: Pad footing and mini niche beam installation – 3D model isometric view (left) and site photography (right)

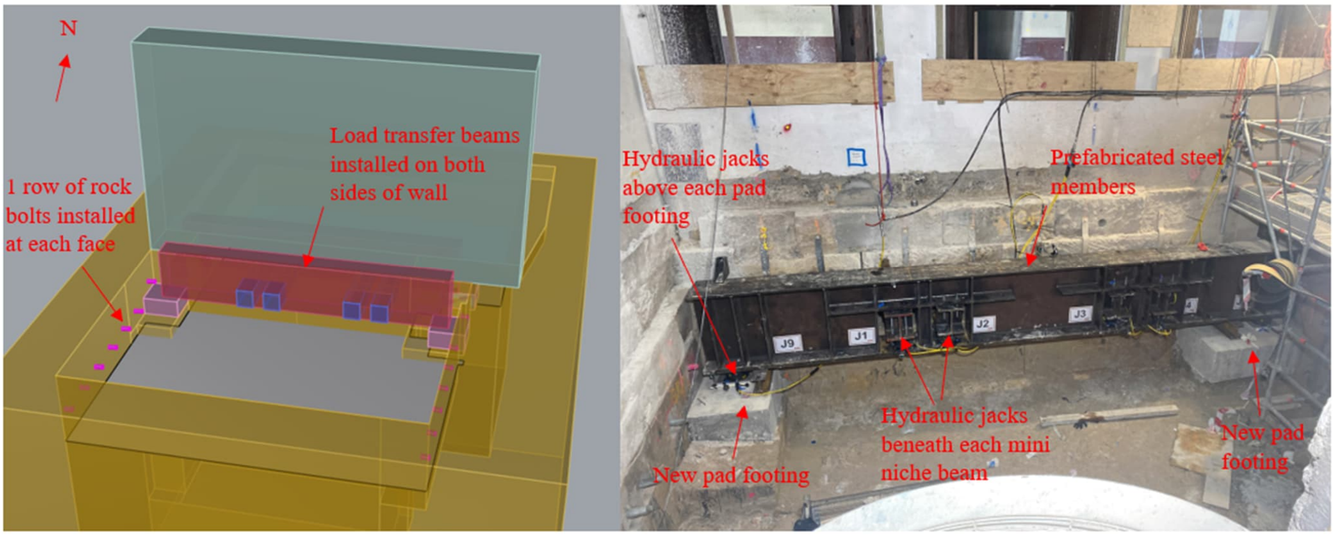


Figure 13: Load transfer beam and jacking system installation – 3D model isometric view (left) and site photography (right)

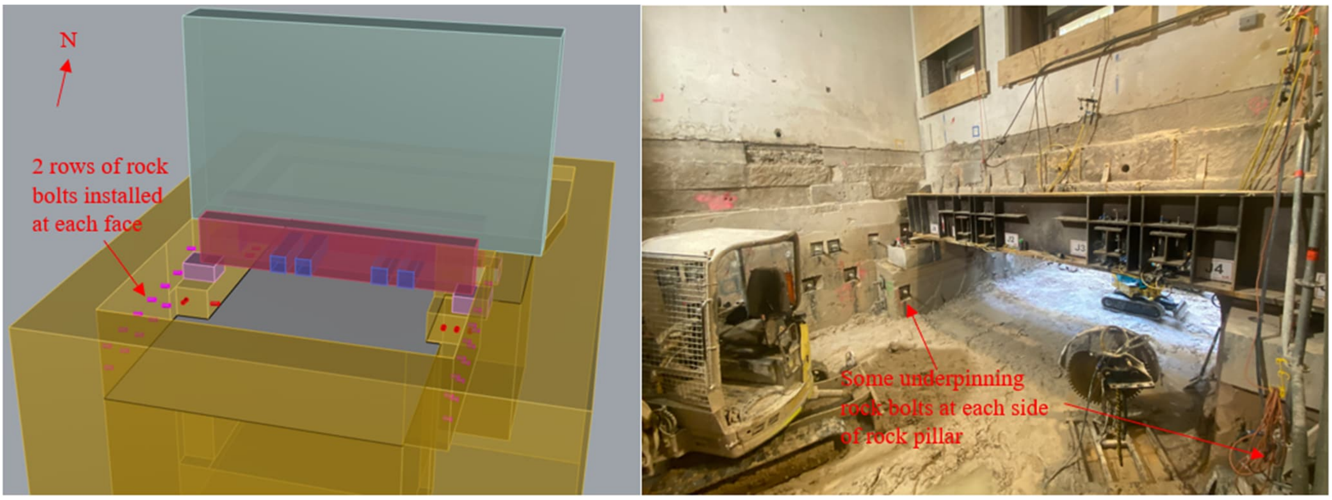


Figure 14: Wall breakthrough – 3D model isometric view (left) and site photography (right)

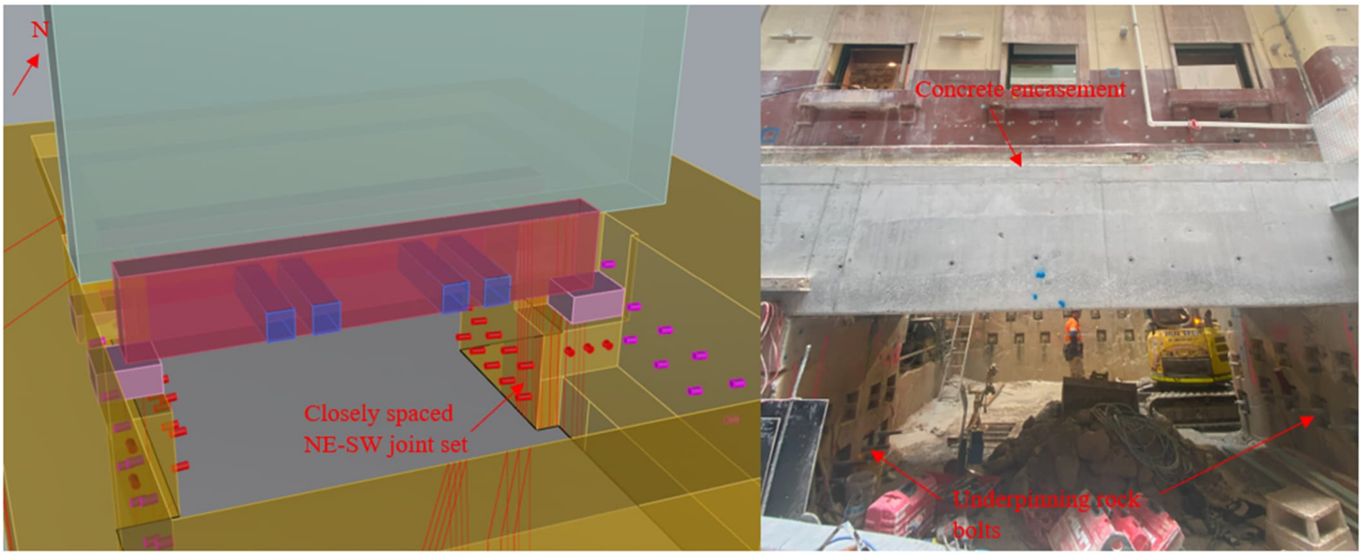


Figure 15: Underpinning the rock pillars - 3D model isometric view (left) and site photography (right)

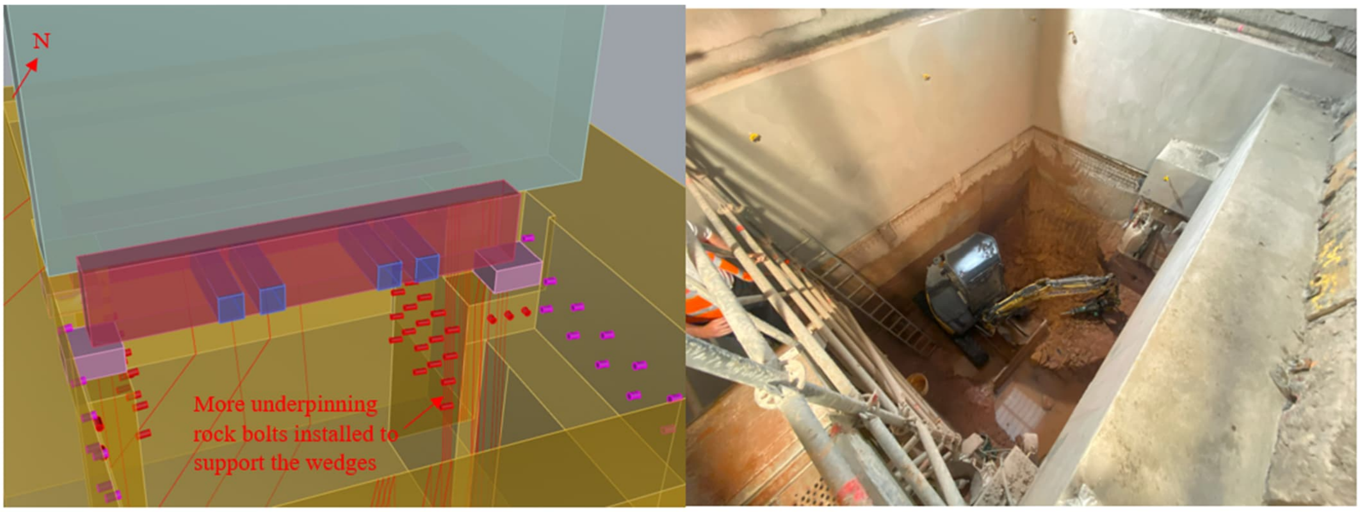


Figure 16: Bulk excavation of the remainder of access shaft - 3D model isometric view (left) and site photography (right)

6 CONSTRUCTION AND MONITORING

The successful execution of the design required a highly controlled construction approach, integrating specialised excavation techniques with a rigorous monitoring and design verification program. This was essential to manage the significant geotechnical risks and adhere to the strict deformation limits of the heritage structure. Key elements of this approach included:

- Close geotechnical involvement during construction including design verification such as hold points and witness points, and working collaboratively with the contractor to address issues and optimise the sequence.
- Saw cutting and line drilling, as opposed to rock hammering, to avoid overbreak and minimise vibrations.
- Rock bolt pull testing conducted to verify the design bond strength
- Maximum excavation lift heights specified, being 1 to 1.5 m at the top of the shaft and increasing to 2.5 m at the bottom of the shaft were rock mass conditions improved and the works were further from the heritage walls.
- Real-time wall monitoring.
- Adjustment of flat jacks as required to control the ground movement beneath the underpinned wall.

The implementation of these controlled construction and monitoring strategies proved highly effective. The measured movements of the existing walls and footings were negligible and remained well within the strict design tolerances throughout the construction. As a result, the impact of the access shaft excavation on the integrity of the existing heritage building was minimal. Figure 17: and Figure 18: show some photographs taken during the construction works.

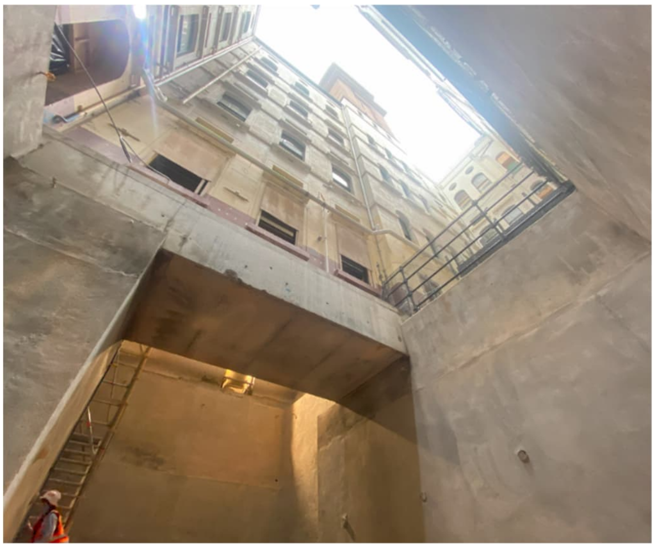


Figure 17: Photograph of the completed transfer structure, taken from the bottom of the shaft

Figure 18: Photograph of the completed tunnel portal before tunnel breakthrough (left) and after tunnel breakthrough (right)

7 CONCLUSIONS

The project case study presented in this paper is one example of an increasing trend for redevelopment of heritage buildings. Such redevelopments are often highly constrained by requirements to preserve and protect heritage elements. This particular project was additionally constrained by the geotechnical conditions namely the G.P.O. Fault Zone in Sydney's CBD.

Key project challenges included support of an existing five-story sandstone block wall directly above one of two new shafts, as well as support of similar walls and a higher clock tower around the perimeter of the shafts. Addressing these challenges required close integration between construction, structural, and geotechnical engineering disciplines.

Through innovative engineering solutions, and meticulous planning construction and verification, this project successfully solved the geotechnical and construction complexities associated with the redevelopment of the heritage building, and provides a valuable precedent for future development.

8 ACKNOWLEDGEMENTS

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This project was first presented to the AGS during an oral presentation by Mr Liang at the 2024 Sydney Chapter Young Geotechnical Professionals event, where it received the first prize award. This paper documents the presentation and provides further details. Thanks are extended to the organising committee for the opportunity to publish the paper in *Australian Geomechanics*.

CRediT authorship contribution statement

Juno Liang: Writing - original draft. **Jeremy Toh:** Writing – original draft.

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