Excavation along the Atlantic Seaboard, Cape Town – Part 2: Modern-Day Development

The following article is the second in a three-part series that reviews the historical, current and future practice of excavation along the Atlantic Seaboard in Cape Town and is the full version of an abridged paper that will be presented at the 2nd Southern African Geotechnical Conference in Umhlanga from 28 to 30 May 2025. **The first article** was published in the March 2025 issue (Vol.33, No.2) of *Civil Engineering*.

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Bantry Bay, Clifton, and Camps Bay have seen significant development since the construction of Kloof Road, Victoria Road, and Camps Bay Drive. In 2025, the houses and apartments in Clifton and Bantry Bay are steeply nestled on the mountainside, creating a striking landscape akin to Hong Kong and Monaco.

For ease of reference, the locality plan provided in the first article, with all the sites under discussion, is provided in Figure 1.

NEED TO EXCAVATE

The incentive to undertake deep excavations on the Atlantic Seaboard is driven by several factors, including the high land values – among the most expensive in Africa – which create a strong need for densification. Additionally, zoning regulations that control building height and area, limited land availability, the demand for parking and service rooms (such as air conditioning and electrical systems), as well as the steep hillside topography and small property sizes all contribute to the push for deeper excavations in this region.

Required parking bays

According to Transport Research Record 816 (Mackay, van Zyl & Vorster), 1 to 1.5 parking spaces are required for units with one

Figure 1 Area of the Atlantic Seaboard under consideration

Figure 2 (a) Unsupported cut created in Barbara Road, Camps Bay, (b&c) damage to the adjacent house, and (d) damage to the house above the cut

to four bedrooms. Additionally, 0.5 parking bays per unit should be allocated for visitors, resulting in a total of 1.5 to 2 parking bays per unit. However, in Clifton, due to limited municipal street parking and the high demand from luxury developments, a minimum of 3 parking bays is typically provided per unit. This makes it extremely challenging to develop high-density projects on small footprint sites without incorporating solutions such deep basements and car lifts, which often requires obtaining approval from neighbouring properties to install grouted anchors into their property. The requirement of large open spaces and minimal shear walls in such parking areas makes buttressed retaining walls generally impossible.

Permanent support on a hillside

In excavations on level land, the infill structure can typically provide permanent support to earth pressures, as the forces are balanced from opposite sides. However, in hillside excavations, earth pressures are unbalanced, with forces acting in the downslope direction and often across the erf due to variations in topography, such as promontories and valleys. As a result, permanent support is required, either through grouted anchors within the property boundary or into neighbouring properties, or by incorporating structural shear elements into the building itself.

Often, reliance is placed on interface shear resistance between the lateral support from neighbouring properties, particularly in the downslope direction. However, this shear resistance could be compromised if excavation occurs on adjacent properties, making it difficult for the neighbour to restore the support if they choose to excavate in the future. Structural shear walls, however, are met with reluctance in luxury developments, where architectural freedom is highly valued.

The first principle of economic development is to maximise allowable building volume, and as a result, internal permanent anchorages are often avoided. Excavating right up to the property boundary is typically not ideal either, as it can result in a lack of natural light, particularly if the building must provide permanent lateral support. Therefore, it is often more prudent to explore a range of design options to achieve optimum investment returns and maintaining functional and aesthetic qualities.

Unsupported cuts

It is still common practice to create unsupported vertical cuts, often as a cost-saving measure or due to a lack of understanding of certain lateral support systems, which stabilise a cut face from the top down, as opposed to conventional retaining walls or segmental block walls that require a cut to be made before their construction.

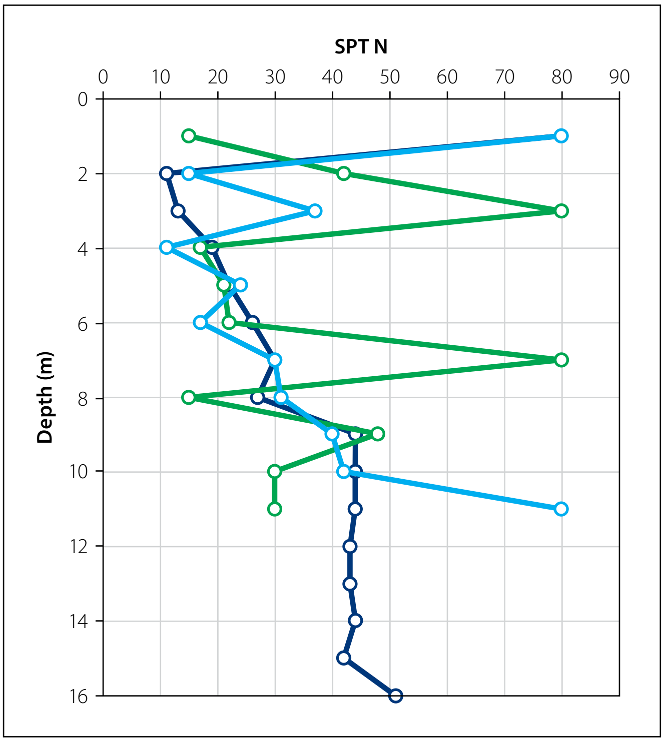

Figure 3 SPT-N plots for Boulder apartment

Although cuts for conventional retaining walls also require temporary lateral support, they are often created without support. Additionally, neighbouring properties may be unwilling to allow temporary anchorage. This practice is considered dangerous and poses a risk to the client, especially when the cut is located directly adjacent to a property boundary and there is no possibility of creating a safe batter angle.

There have been several instances where unsupported cuts have resulted in collapse or significant damage to neighbouring properties, with the failures at Dunmore Apartments and Barbara Road being the most notable. Photos of the Barbara Road slip (to the south in Figure 1) and the resulting damage to adjacent properties are shown in Figure 2. This unsupported cut led to damage to four adjacent properties and years of legal disputes.

Local government should ensure that, when such unsupported cuts are made, the owner has employed a competent professional to meet the requirements of SANS 10400A – Application of the Building Regulations Part A: General Principles and Requirements and SANS 10400G – Excavations. If such a professional has not been appointed, lateral support insurance cannot be obtained, and local authorities should halt work until the required professional has been appointed.

However, if an engineer or engineering geologist signs off on the stability of a high unsupported cut, they are assuming considerable risk, as they are not typically present on site every day to enforce the observational method. Construction often extends across both wet and dry seasons, with wet conditions frequently leading to failures.

In some instances, dense developments are still constructed with unsupported cuts permanently left in place. Many of these cuts show signs of distress over time and are difficult to stabilise at a later stage due to the space constraints imposed by the surrounding development. The risk of leaving a cut unsupported, especially when future access for stabilisation is not possible, could have highly detrimental effects on the development.

Access issues

Due to the steep nature of most of these sites, lateral support solutions are often restricted to the use of small rigs, which are typically hoisted onto site by cranes. The use of large-diameter soldier piles is not typically feasible, as large rigs cannot easily access the site, and constructing extensive working platforms would be required.

History of modern-day development

In the 1960s, four large apartment blocks were developed: Valhalla (Site 3), San Michele (Site 4), La Corniche (Site 5), and The Beaches (Site 6). To our knowledge, only The Beaches was originally constructed using anchors and shotcrete (Civil Engineering, 1994). Valhalla, San Michele, and La Corniche on the other hand were built using minor terraced cuts, typically around 1.5 m high, with a deeper cut cantilever or braced retaining wall on the lower portion of the site. Valhalla, San Michelle and La Corniche were, however, later retrofitted with geonails, rock bolts and anchors.

The construction of these glamorous and opulent structures was met with dismay from Clifton residents, who felt they detracted from the area's charm, which had been defined by its historic bungalows dating back to 1886. In 1984, the decision to designate zones around Clifton's 2nd, 3rd, and 4th Beaches, Glen Beach, and Bakoven as national monuments sparked significant criticism (De Beer, 1987). The aim of this move was to preserve the unique character and charm of the area, including its historic bungalows, and to prevent the development of large apartment blocks similar to those at Clifton's 1st and 2nd beaches from encroaching on Clifton's 3rd and 4th beaches.

Since 1987, some of the largest excavations in the area have included those at Bantry Point (to the north in Figure 1), Eventide (Site 8), Dunmore Apartments (Site 7), and Clifton Terraces (Site 9). From a foundation perspective, most large developments use raft or pad foundations on solid rock. For properties located above unsupported road cuts, the cut slopes are typically on council land, making it difficult for property owners to stabilise the slopes. As a result, these owners often rely on piled foundations and beams to ensure that the weight of the structure is transferred deeper into the soil profile, preventing excessive surcharge from acting on the unsupported slope.

CASE STUDIES

Three case studies will be presented, highlighting the challenges associated with creating excavations in this densely developed and steep area.

Boulder apartments, Bali/Bakoven Bay (Site 10)

Location and problem statement

An overall cut of 17 m was required for the construction of a seven-level apartment block in Bali Bay, Camps Bay (Site 10). The excavation included a 12 m vertical face, along with two shallower vertical cuts above. The site is located next to an old, infilled drainage gully to the south. No grouted anchors were allowed to be installed into the neighbouring properties and shear walls had to be minimised to preserve architectural freedom. As a result, the rear lateral support line was shifted 22 m into the site to ensure that permanent grouted anchors could be installed within the property boundary (see Figure 4)

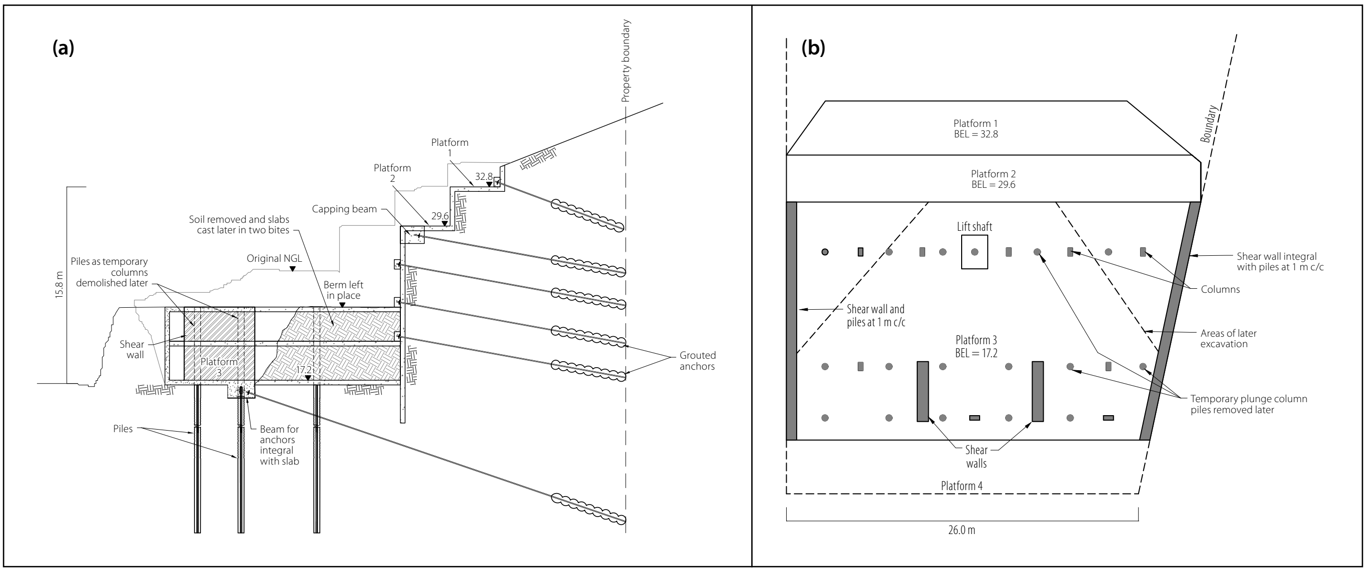

Figure 4 (left) Cross section through lateral support system at Boulder apartments and (right) plan view showing location of shear walls and plunge columns

Geology

The site is situated in colluvium overlying residual granite, with the depth of the colluvium increasing and larger boulders present towards the south, within the drainage gully. The SPT-N plots are shown in Figure 3. The water table was located just above the residual granite.

Solution

Back analyses were conducted using slope stability software on the existing cut, slopes, and historic structures present on the site and surroundings to estimate the minimum shear parameters. Both partially drained (above the groundwater table), undrained (below the groundwater table), and completely drained analyses were performed and compared.

These parameters were then used in analyses that considered the proposed cut. Concerns arose regarding the length of the top anchors when excavating to the full required depth. Consequently, a decision was made to adopt a top-down, partial-width construction method for the bottom two levels of the excavation. The solution involved the use of bored soldier piles for lateral and vertical capacity, with shotcrete infill panels and permanent grouted anchors connected to the soldier piles using concrete waler beams. A concrete reinforced frame was constructed, incorporating two central shear walls as well as shear walls along the northern and southern boundaries of the site (Figure 4). These walls were positioned at the front of the site (adjacent to Victoria Road) before all soil was removed and slabs constructed to prop up against the soldier pile wall before mining below the slabs. Below these shear walls, a row of anchors was connected to the foundation to provide the required lateral restraint within the property boundary. 2D finite element analyses were conducted to predict displacements, anchor forces, and slab reaction forces.

Construction and ground movement

Construction commenced in 2013, and ground movement measurements were taken using accurate survey readings. The maximum final movement recorded was 39 mm at the top of the uttermost cut face and 26 mm at the capping beam. Several piles were exposed where parts of the shaft lacked concrete, and these were remedied up to the bulk excavation level (BEL). Since reliance was placed on both vertical shaft capacity and lateral restraint below the BEL, additional piles were centrally cored below bulk excavation level to check for any voids. Piles with voids were grouted, but 80% of the piles were not tested. As a result, an extended guarantee had to be sourced.

Figure 5 provides a photo of the near completed lateral support system.

Figure 5 Lateral support at Boulder apartments

Clifton Terraces (Site 9)

Location and problem statement

An overall maximum cut of 43 m was required for the construction of a 12-level apartment block in Clifton (Site 9). This was achieved through a four-tier system, with the maximum single vertical cut reaching 26 m in height. This remains the deepest single excavation created in Clifton to date for for a single development. The site is located next to Clifton Steps to the north, a servitude to the south, public open space to the east, and the 27 m deep cut for Eventide to the west across Victoria Drive. Clifton Steps and the servitude are only 3 m to 4 m wide, with privately owned land adjacent to both. Grouted anchors were not permitted to be installed in the privately-owned land to the north and south; however, permissions were obtained from the council to install permanent anchorages within these servitudes and the public open space to the east.

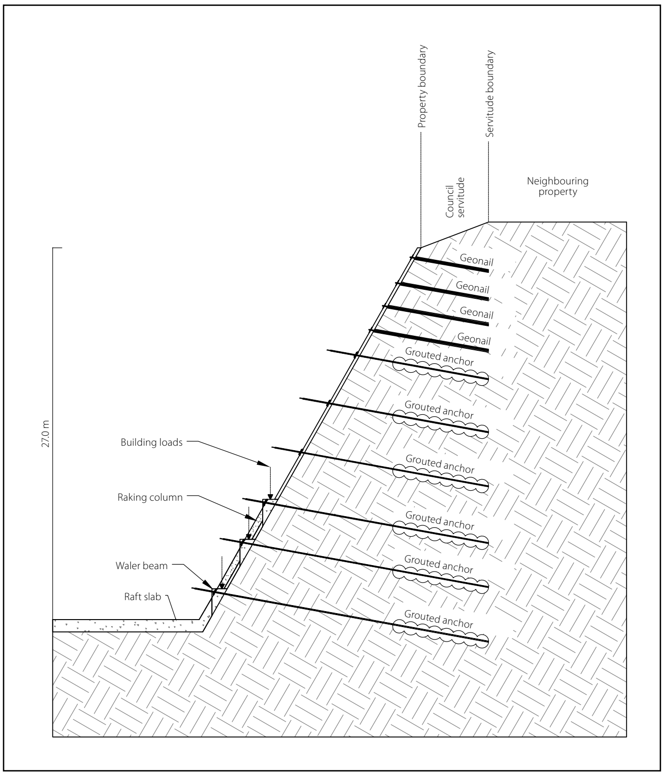

Figure 6 Cross section through 60° lateral support

Geology

The site is typically covered by a 1 m thick layer of sandy colluvium, which rests on a 1 m thick residual granitic soil layer that transitions into very soft rock and then hard rock at greater depths. Contours of the bedrock hardness were determined using data from surrounding outcrops, joint orientations, and borehole investigations.

Solution

Back analyses were conducted using slope stability software to evaluate the cut, slopes, and structures on the site and its surroundings to estimate the minimum shear strength parameters. These analyses employed Mohr-Coulomb, Hoek-Brown, and Barton-Bandis strength models. The strength values were adjusted and rationalised to achieve similar results in terms of factors of safety for all analysis methods used. Subsequently, wedge analyses were performed to determine the required horizontal force to stabilise the excavation, considering varying friction angles and cohesion values. This was done in 3 m increments to assess the increase in horizontal force with depth for each segment.

The design incorporated a nailed shotcrete facing interspersed with walers with permanent grouted anchors. On the eastern and northern faces, the lateral support was battered at a 60° angle from the horizontal, starting from the property boundary (Figure 6). This resulted in the lateral support toe line stepping into the site at an angle, nonparallel to the boundary. The reinforced concrete frame that partially rested on the southern and northern faces was supported by waler beams connected to raking columns and a concrete raft foundation.

2D finite element (FE) analyses were performed to predict the deformation using both Mohr-Coulomb and Hoek-Brown constitutive models. The total displacement predicted by the Mohr-Coulomb model after excavation was 32 mm, and 75 mm after construction of the building on the upper tier. In comparison, the Hoek-Brown model predicted 32 mm and 37 mm of displacement, respectively.

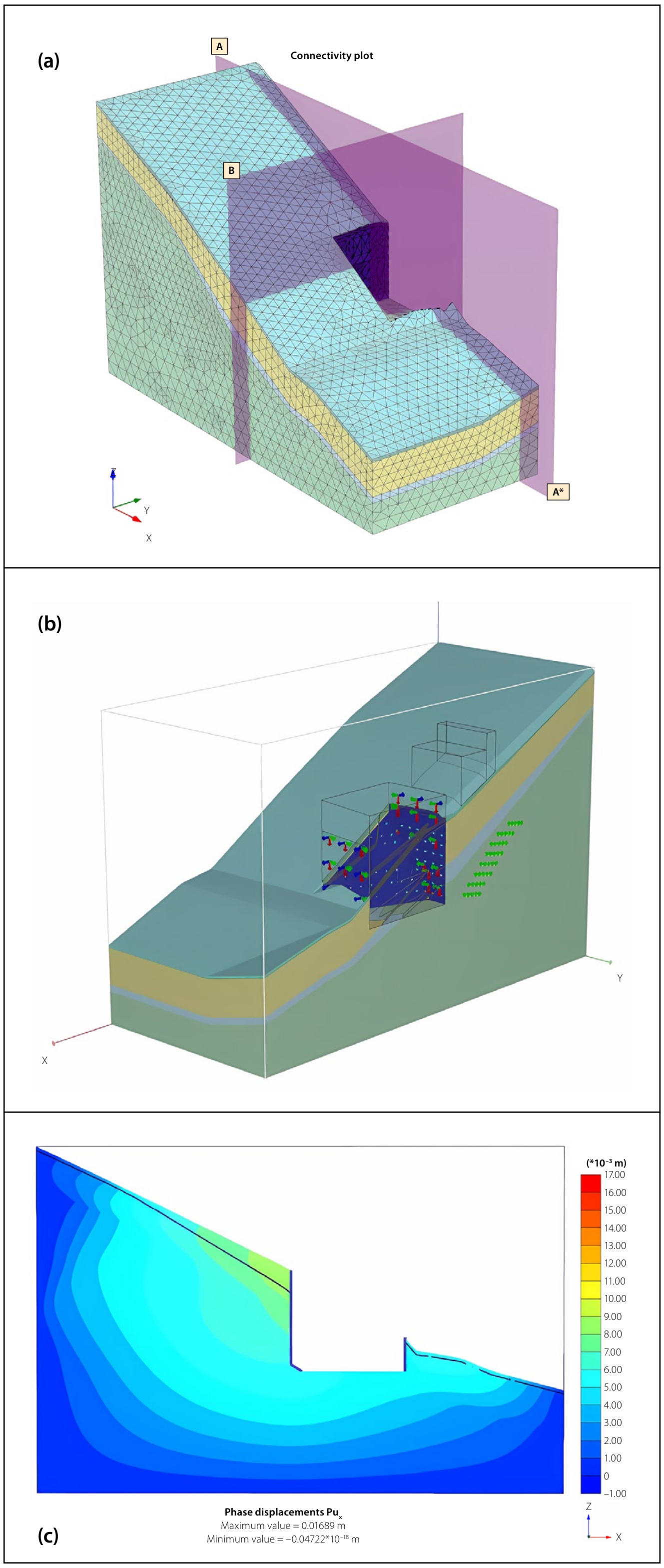

Figure 7 (a&b) 3D finite element model and (c) displacement plot through an east-west section

Additionally, a 3D FE analysis was conducted (Figure 7), which predicted a total displacement of 20 mm after excavation. This displacement is lower than the 32 mm predicted by the 2D analysis. This difference can be attributed to the more accurate representation of the failure mechanism in the 3D model, where additional strength from the sides of the slip surface is considered. Furthermore, in some cases, if the soil was able to arch between the two lateral support faces, the pressure could be further reduced, as the failure mechanism would resemble that of a semi-silo.

Figure 8 Photo of Clifton Terraces excavation

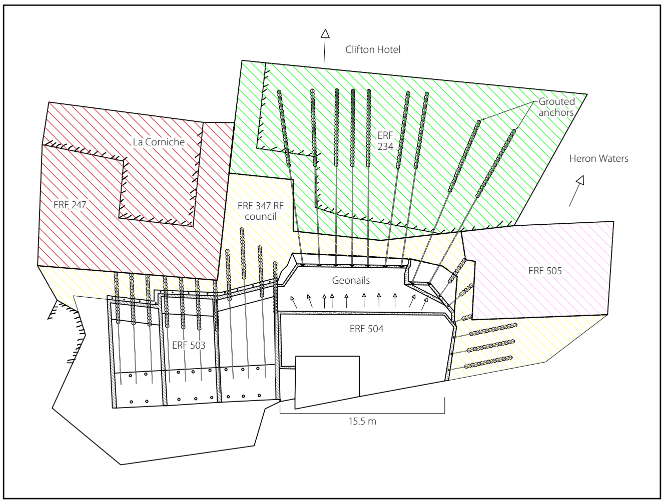

Figure 10 Plan view of anchor layout required for ERF 503 and 504

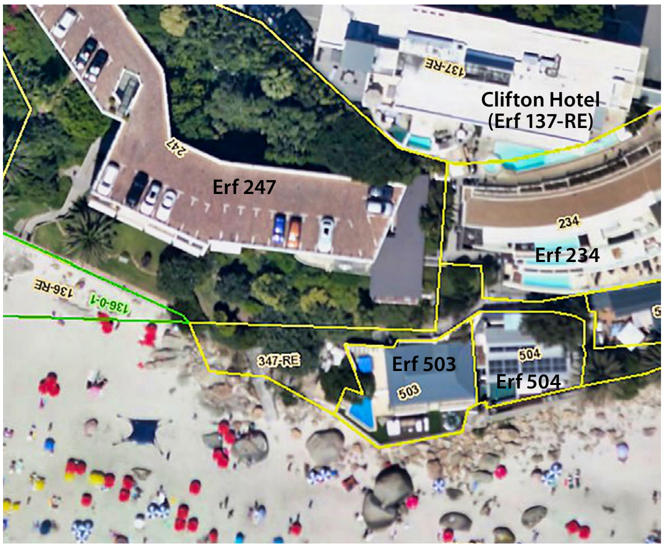

Figure 9 Aerial photograph showing ERF 234, 503 and 504 (source: CCT Citymap viewer)

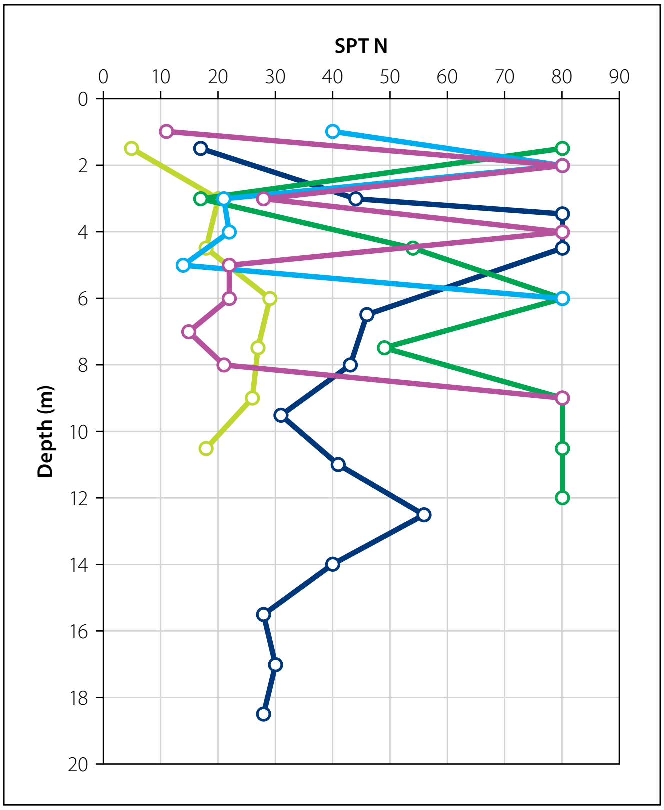

Figure 11 SPT-N plots at ERF 234 and 503

Construction and ground movement

As the anchors are permanent, thorough quality assurance and control processes were enforced to ensure double corrosion protection was achieved at the grouted anchor heads, where the anchors are most often prone to corrosion, especially if a void is left in the drilled hole just behind the face. Some grouted anchors were installed with removable covers for future inspection and lift-off tests as part of the maintenance plan. Ground movement measurements were undertaken along with accurate survey readings. The final total displacement measured, after construction of the building on the lower 26 m high eastern face, was 41 mm, which aligns well with that predicted from the Hoek-Brown 2D analysis. A photo of the lower tier excavation is provided in Figure 8.

Erven 503, 504 and 234, Clifton (Sites 11 and 12)

Location and problem statement

An overall cut of 27 m was required for the construction of a five-level apartment block and later a bungalow below it, in Clifton, immediately above 2nd Beach on erven 234 and 504 (Figure 9) respectively. The site is situated just below the site of the original Clifton Hotel (Erf137-RE). A further 6 m cut was also later required for a bungalow on the adjacent ERF 503 below La

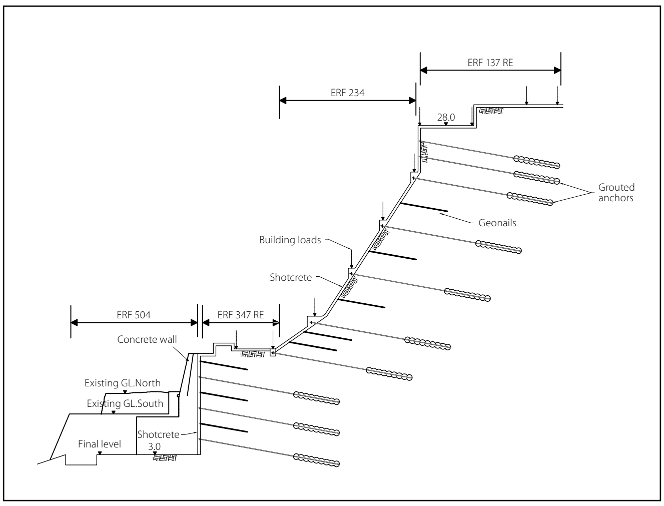

Figure 12 Section through ERF504 and 234

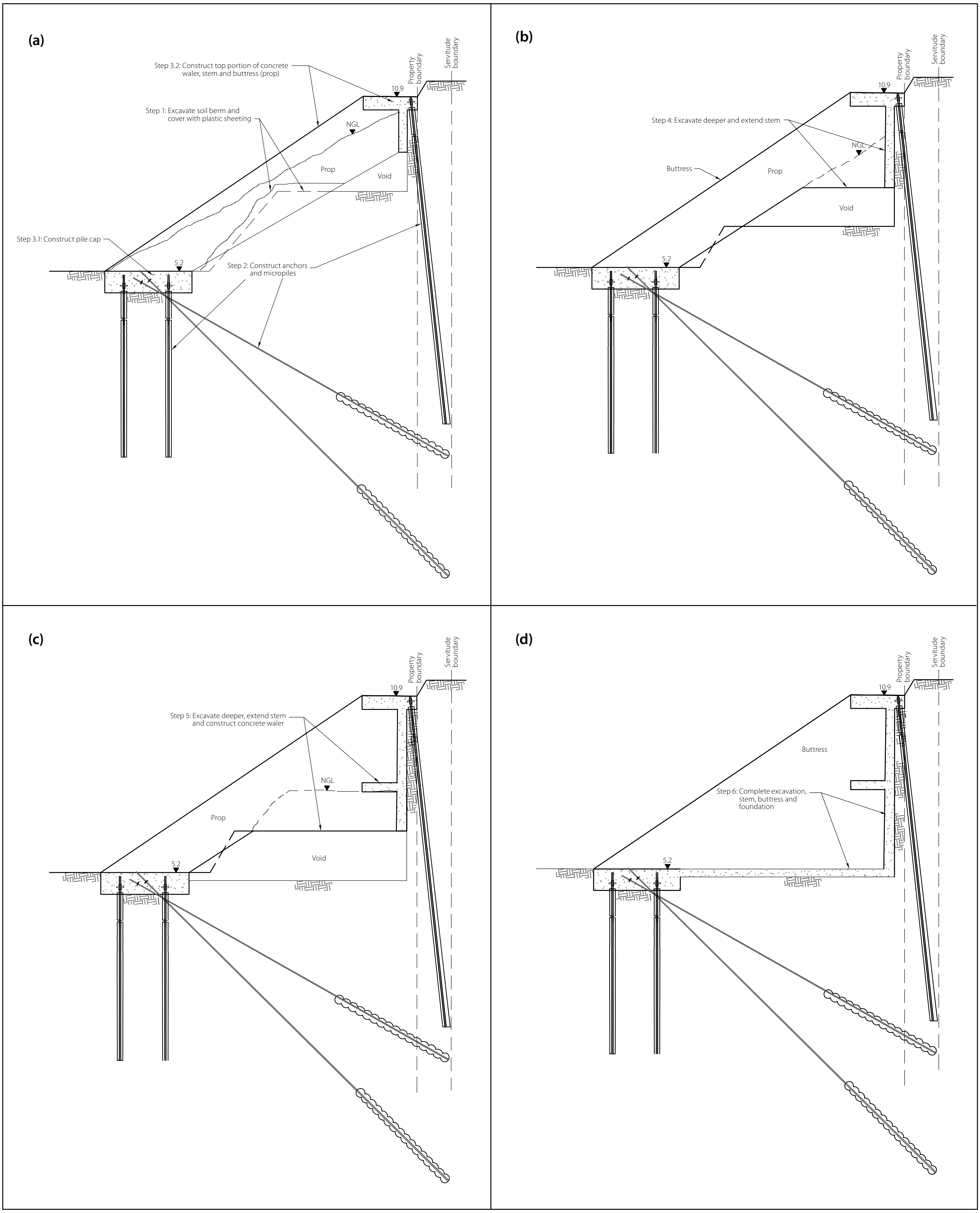

Figure 13 Construction sequence of buttress lateral support on ERF503

Corniche apartment block. Sole access was by way of a 1.5 m to 2 m walkway between the sites.

Permission was obtained for permanent anchors beneath the Clifton Hotel and public open space but not beneath La Corniche and Heron Waters (erven 247 and 237). This required innovative solutions. The complexity of these is illustrated by Figure 10 and 12, showing the spread of anchors in the area.

Geology

The site is predominately situated on a promontory of mixed landslide debris and colluvium underlain by granite bedrock at depths in excess of 19 m. Evidence of the landslide is provided by aerial photography extending to the top of Lion's Head and the mixture of very large granite and sandstone boulders exposed on the beach. The SPT-N plot is provided in Figure 11.

Figure 14 Photo of buttressed lateral support on ERF 503

Solutions

The initial design for lateral support on ERF234 involved a vertical cut. However, finite difference modelling predicted that this excavation would result in a total displacement of approximately 50 mm beneath the front of the old Clifton Hotel, which was deemed unacceptably high. Such a cut would also require anchors to be installed into La Corniche and Heron Water. Given the access constraints and the presence of boulders, a stiffer system using soldier piles was not considered a feasible alternative.

As a result, a battered lateral support system was chosen, with a face angle ranging from 30° to 50°. This solution, however, presented another challenge: the building needed to be founded on foundation pads or walers resting on the lateral support system. While the in-situ colluvium offered sufficient bearing capacity, the vertical load's downslope driving component had to be counteracted by stressed grouted anchors, which provided a resulting additional frictional force to balance the load (see Figure 12 and 15).

On ERF 503, a buttressed approach was implemented, with the excavation, buttresses, and waler constructed in a stepped sequence from the top down. Before the retaining structure was built, a narrow raft foundation, reinforced with micropiles and anchors, was installed at the toe of the buttresses. Openings in the buttresses were later created to facilitate access between rooms (see Figures 13 and 14).

Figure 15 Photo of lateral support on ERF234

Construction

The construction of the apartment on ERF234 began around 2006, followed by the commencement of construction on ERF503 around 2010 and thereafter ERF504. A photograph of the nearly completed buttressed lateral support is shown in Figure 13, while the lateral support on ERF234 is shown in Figure 14.

CONCLUSION

The three case studies highlight the complexity of developing in this area, predominately due to difficulty in obtaining permissions to install grouted anchors and access constraints while maintaining public access. The designs required innovative optioneering to ensure that feasible solutions could be developed.

Previously, excavation and anchors across boundaries and blasting and piling vibrations were not a significant issue. Now, they are paramount and critical.

ACKNOWLEDGEMENTS

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COMING UP

Parts 3 of this three-part series will be published in the next issue of *Civil Engineering.*

REFERENCES

References have been excluded due to space limitation. A complete list of references can be obtained from the authors.

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- **Frans van der Merwe** Pr Eng, holds a BEng in civil engineering from the University of Stellenbosch (2009) as well as an MEng (2017). His primary areas of expertise include the design of lateral support systems and pile design. He currently chairs the technical committee revising the SAICE Lateral Support Code of Practice, which was last updated in 1989.
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