# Diaphragm Wall Trench Stability in Recently Reclaimed Land: A Case Study Review

T.M.S. Sacadura*, L. Lee, O. Haye

Intrafor Hong Kong Limited, Hong Kong

*Corresponding author
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**Abstract**

This paper focuses on the design and review of diaphragm wall trench stability using bentonite as the stabilizing fluid on a reclaimed site in Hong Kong SAR, Hong Kong Boundary Crossing Facilities. The site geological conditions were challenging for construction of long diaphragm wall panels due to the presence of considerable thickness of soft Marine Deposit and Alluvium Clays. In addition, a special type of diaphragm wall panel (Y-panel) was required for a multi circular cell cofferdam. The applicability of three-dimensional finite element methods software, Plaxis 3D, to model the trench stability is discussed through a comparison with other analytical trench design methods. Two site trials were undertaken, one for a triple-bite panel, 6.8m long and 1.5m thick, and another for the 5-bite panel (Y-panel), 3.6m x 6.5m. The latter required ground improvement, Cutter Soil Mixing, works to ensure both a satisfactory factor of safety against failure and acceptable lateral movements of the trench. The ground treatment extended for 1.0m around the perimeter of the trench with two different degrees of improvement in terms of strength and deformability. The shallower deposits, Fill, disturbed and natural Marine Deposit (approximately 20.5m) were treated with nominal ground improvement (UCS = 0.5 MPa) while the underlying Marine Deposit and Alluvium (approximately 20.7m) were fully treated to achieve an UCS of 1.0 MPa. Both site trials were instrumented, and the results are compared against the design predictions from the Plaxis 3D model.

**Keywords:** Diaphgram wall, Trench stability, Reclamation

## 1 Introduction

The Tuen Mun – Chek Lap Kok Link (TMCLKL) project will provide a strategic link connecting the Northwest New Territories with the Hong Kong Zhuhai Macau Bridge Hong Kong Boundary Crossing Facilities (HKBCF), North Lantau and the Hong Kong International Airport. The overall project consists of a dual 2-lane 5 km long subsea tunnel, a northern and southern landfall with the respective reclamation and associated works. The southern landfall approach ramp is located on the reclaimed site of the HKBCF. The site was reclaimed between 2011 and 2015 and was subject to extensive ground investigation, which revealed a range of ground conditions at the site. This included Alluvium Clays and Sand overlain by soft Marine Deposit (MD) with significant thickness overlain by Fill materials. The Fill materials comprise man-made Fill and pockets of disturbed MD.

The lowest formation level for the approach ramp of TMCLKL was at -37.5mPD, which equated to an excavation of about 43m since the existing ground level was at +5.5mPD. Design development of the southern landfall approach ramp saw a typical multi-propped straight cofferdam transform into a temporary hybrid Excavation and Lateral Support (ELS) scheme with 15 consecutive cells. This ELS


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scheme allowed for a strut free construction within the cells, with cross walls and localized struts at the panels connecting adjacent cells, Y-Panels. It was also a stiffer ELS solution, which minimized lateral movements and associated settlements.

Due to the challenging ground conditions and to ensure proper trench stability, single bite, 2.8m long x 1.5m thick, diaphragm wall panels were adopted in the initial panel layout design of the ELS. Special 5-bite diaphragm wall panels, Y-Panel, 3.6m x 6.5m, were required adjacent to each cell to transfer the hoop stress loads through the struts and cross wall to the opposite side of the cofferdam. Ground improvement works using Cutter Soil Mixing (CSM) were required to ensure both a satisfactory factor of safety against failure as well as an acceptable lateral ground movement of the trench. Two site trials were undertaken to confirm the trench stability design and confirm lateral movements, one for a straight 6.8m long x 1.5m and another for the Y-Panel.

## 2 Site Description and Ground Conditions

### 2.1 Site Description

The southern landfall approach ramp of TMCLKL, which is approximately 20 hectares, is located on the recently reclaimed site of the HKBCF, see Figure 1. The 150 hectares artificial island was reclaimed between 2011 and 2015 using 31m diameter steel cells as the inner perimeter seawalls combined with sloping rockfill in the outer perimeter. The reclamation was non-dredged leaving in place soft MD, which was subject to a number of different methods of ground improvement, e.g. stone columns, prefabricated vertical drains with surcharge. The ground level prior to the cut-and-cover tunnel excavation was generally at +5.5mPD. The 550m long cut-and-cover tunnel adopted a temporary hybrid ELS scheme with 15 consecutive circular cells, see Figure 1. The circular cell radius varied from approximately 21.9m to 28.5m with the minimum and maximum cell radius occurring at cells 1 and 15, respectively. A total number of 424 diaphragm wall panels, including the arch and the cross walls, together with 30 nos. of Y-Panels were required. The maximum excavation depth within the cut-and-cover tunnel was approximately 43m, formation level of -37.5mPD.


**Figure 1:** a) Tuen Mun Chek Lap Kok Link Project Location, adapted from www.hzmb.hk/eng/about_tmclkl.html (Accessed on the 31/03/2021);
b) Hybrid ELS Scheme, courtesy of Dragages Bouygues Travaux Public JV

## 2.2 Ground Conditions

The site was subject to extensive ground investigation campaigns with one occurring pre-reclamation works and another four campaigns post-reclamation. The site investigation confirmed the highly variable geological conditions. The reclamation Fill is predominantly loose to dense silty fine to coarse sand with occasional gravel and cobbles. Possibly during fill placement some of the underlying natural soft to firm silty clay MD was displaced and formed localized pockets of disturbed MD within the fill. The variable Alluvium deposits, which range from silty clay to silty fine to coarse sand, underlie the MD and overlie completely decomposed rock, which in turn overlies engineering rockhead. Depending on location, bedrock can either be granite or meta-sedimentary rock (siltstone / sandstone). Soil parameters were derived from the ground investigation and subsequent *insitu* and laboratory testing. Moderately conservative geotechnical design parameters adopted in this study were based on the suggested parameters determined by the main contractor's ELS designer, Atkins, and are shown below in Table . Table 2 summarizes the range of stratum thickness.

**Table 1**. Summary of Design Soil Parameters (Moderately Conservative Values)

| Stratum | Bulk Unit Weight ($kN/m^3$) (unsaturated) | Poison's Ratio v | Cohesion, c' (kPa) | Angle of Internal Friction, f' (°) | Undrained Shear Strength, $c_u$ (kPa) (z' refers to mPD) | Drained Young's Modulus E' (MPa) (z' refers to mPD) | Undrained Young's Modulus Eu (kPa) | Typical Permeability k (m/s) |
|---|---|---|---|---|---|---|---|---|
| Reclamation Fill | 20 / (19) | 0.25 | 0 | 33 | - | 15 | - | $5x10^{-4}$ |
| Rock Fill (Working Platform) | 20 | 0.30 | 0 | 40 | - | 80 | - | $5x10^{-4}$ to $1x10^{-3}$ |
| Marine Deposit Clay (Natural) | 16 | 0.50 | 0 | 28 | 30+1.5(-z-10) | - | $300c_u$ | 1 $x10^{-8}$ to $1x10^{-9}$ |
| Marine Deposits Clay (Disturbed) | 16 | 0.50 | 0 | 28 | 0.22 s'v | - | $300c_u$ | 1 $x10^{-8}$ to $1x10^{-9}$ |
| Alluvium Silt/Clay | 19 | 0.50 | 0 | 28 | Cat1: z ≥-35: 80<br>z ≤-35: 80+2(-z-35)<br>Cat2: z ≥-35: 60<br>z ≤-35: 60+2(-z-35) | - | $400c_u$ | $1x10^{-8}$ to $1x10^{-9}$ |
| Alluvium Sand | 19 | 0.25 | 0 | 35 | - | z ≥-30: 30<br>-30>z≥-40:30+3(-z-30)<br>z ≤-40: 60 | - | $8x10^{-5}$ to $8x10^{-6}$ |
| Completely Decomposed Granite | 19 | 0.50 | 5 | 32 | - | z ≥-50: 60<br>z <-50: 60+11.2(-z-50) | - | $1x10^{-6}$ to $1x10^{-8}$ |

**Table 2.** Range of Stratum Thickness

| | **Fill (Including DMD Pockets)** | **Marine Deposit Clay** | **Alluvium (Silt / Clay / Sand)** | **Completely Decomposed Granite** |
|---|---|---|---|---|
| **Range of Stratum Thickness** | ~20m | 6m – 14m | 18m – 32m | 4m – 41m |

## 3 Trench Stability Calculation Methods

An excavated trench relies on the soil arching and the stabilizing fluid, e.g., bentonite, polymer, to maintain its stability. Bentonite was the stabilizing fluid used in this project. It has the advantage of effectively creating a filter cake on the excavation faces, which restricts fluid loss into the soil. The presence of the filter cake, combined with the fact that bentonite's bulk unit weight is larger than that of groundwater, allows for a positive hydrostatic head to be maintained within the excavation. At surface where the bentonite pressure is insufficient or the acting loads from plant / equipment are high, a guide wall protects the trench against failure. The presence of fine-grained soils such as silts and clays usually present in Marine and Alluvium strata combined with the short-term nature of diaphragm wall trench excavation benefit trench stability analysis methods that can consider the undrained behavior of soils. In addition, the applicability of these methods also needs to consider if complex geometries or specific loading conditions can be addressed in the analysis.

### 3.1 Soil Pressure Balance on Trench Walls

Huder (1972) and Schneebeli (1964) developed trench stability methods based on the theory of soil arching, Terzaghi's and Caquot's, respectively. Wong (1984) concluded that Schneebeli's method was the least conservative of the two methods and had the advantage of excluding from the input the earth pressure parameter. However, it also found the Schneebeli method to be conservative in Hong Kong historical case studies with presence of Fill, Marine Deposit, Colluvium and Completely Decomposed Granite. Wong (1984) states that a possible cause is Hong Kong's local practice of ignoring soil cohesion in the design and that when it's considered the results from the method are closer to the one's he observed in the test panels. Wong (1984) suggests calculating the horizontal earth pressure acting on the trench using formula (1), adapted to include the cohesion of the soil, which considers a surcharge load located at least half of the trench's length away.

$$\sigma'_3(z) = K_a \frac{\gamma L}{\sin 2\emptyset}\left[1 - e^{-sin2\emptyset\frac{z}{L}} + \frac{q \sin 2\emptyset}{\gamma L} e^{-sin2\emptyset\frac{z}{L}}\right] - 2c'\sqrt{K_a} \quad (1)$$

where s'3 (z)= effective horizontal earth pressure at depth z, Ka = active earth pressure, γ = soil's density, L = length of trench, F = soil's angle of friction, z = depth, q = surcharge load, when load is applied at a depth equal to z.

The Factor of Safety (FS) would then be calculated in accordance with equation (2) below and a value of 1.2 was quoted by Wong (1984) as adequate.

$$Fs = \frac{(p_s - p_w)}{p} \quad (2)$$

where FS = factor of safety, $p_s$ = bentonite pressure acting on the wall of the trench, $p_w$ = water pressure acting on the wall of the trench, p = s'$_3$ = effective horizontal earth pressure.

### 3.2 Limit Equilibrium

Several methods were developed to determine the stability of trenches, using bentonite as the stabilizing fluid, based on limit equilibrium analysis. Morgenstern (1965) proposed a method for cohesionless soils which considered the stability of a 2D wedge, see Figure 2. A 3D limit equilibrium method proposed by Aas (1976) applicable to soft clay was developed based on the experience gained from full scale field tests carried-out on Norwegian soft clays. The method assumed two separate blocks that could create the failure of the soil mass. The upper block was assumed to move vertically and at the same time the lower block would move horizontally towards the trench, see Figure 3.

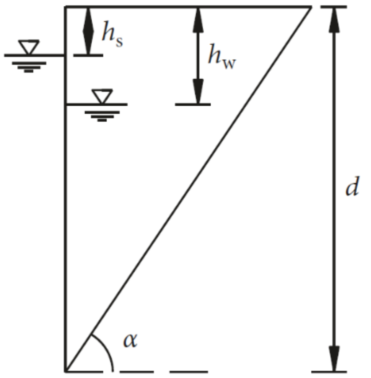


**Figure 2**: Morgenstern (1965) Stability of Sliding Wedge

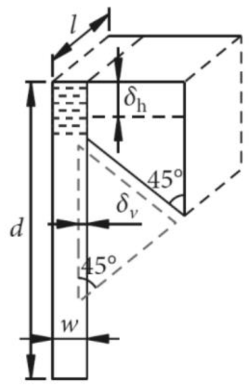


**Figure 3**: Aas (1976) Assumed Failure Condition in Trench

Aas (1976) proposed formula (3) below to calculate the factor of safety. Other methods considering different shapes of wedges / failure conditions were developed throughout the years and some of these methods consider local stability in addition to the overall stability of the trench. However, all limit equilibrium methods require an iterative process to identify the possible failure wedge / block that leads to the minimum Factor of Safety, which can be time consuming.

$$F = \frac{\tau_{VD}}{D\,(\gamma - \beta^2\,\gamma_f)}\left[2\,\frac{\tau_{TD}}{\tau_{VD}} + 0.86\,\frac{D}{L} + 0.6\right] \tag{3}$$

where D = depth to lowest point of failure surfaces, L = length of trench panel, β = slurry depth as a fraction of failure depth, γ = unit weight of clay, $\gamma_f$ = unit weight of trench slurry, $\tau_{TD}$ = triaxial compression strength at depth, D and $\tau_{VD}$ = vane shear strength at depth D.

### 3.3 Finite Element Analysis (Plaxis 3D)

The use of Finite Element Analysis (FEA) software in trench stability problems is not recent. Wong (1984), used a 2D FEA analysis to demonstrate the soil arching effect through the stress distribution adjacent to the trench. Brzakala (2008) used Plaxis 3D Foundations software to analyze the stability of trenches, Grandas-Tavera (2012) analyzed the stability of corner panels using ABAQUS while Wei Li (2019) used FLAC 3D to compare numerical simulations with analytical models.


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Plaxis is a commercial Finite Element Analysis software that allows users to accurately model the construction process and obtain realistic assessment of stresses and displacements for every node. The 3D package allows for problems that cannot be solved by the 2D software to be calculated. In addition, the software allows for undrained behavior to be modelled using pre-defined material models such as the Mohr-Coulomb, Hardening Soil, Soft Soil models. A realistic calculation of stress / strain allows the software to define a failure mechanism more accurately than assuming a certain shape of wedge or a soil arching extent. In addition, actual earth pressures are calculated rather than assuming limit equilibrium values.

The safety calculation (phi/c reduction) was adopted as a calculation stage in Plaxis 3D to compute global safety factors. In this calculation stage the shear strength parameters (tan φ' and c (either c' or cu)) are successively reduced until failure occurs. The total multiplier ∑Msf is used to define the value of the soil strength parameters at a given stage in the analysis as per formula (4) below.

$$\sum Msf = \frac{\tan\phi_{input}}{\tan\phi_{reduced}} = \frac{c_{input}}{c_{reduced}} = \frac{s_{u,input}}{s_{u,reduced}} = \frac{Tensile\ strength_{input}}{Tensile\ strength_{reduced}} = \frac{available\ strength}{strength\ at\ failure} \tag{4}$$

where the strength parameters with the subscript "input" refer to the properties entered in the material sets and parameters with the subscript "reduced" refer to the reduced values used in the analysis.

The ∑Msf is set to 1.0 at the start of a calculation to set all material strengths to their input values. It must always be checked whether the final step has resulted in a fully developed failure mechanism. If that is the case, the ∑Msf represents the factor of safety as calculated by equation (4) above.

## 4 Trench Stability Design and Site Implementation

The initial panel layout for the 15 consecutive cell cofferdam was based on single bite diaphragm wall panels, 2.8m long, because of trench stability concerns associated with the adverse geological / geotechnical conditions discussed in Section 2 above. An alternative panel layout with triple bite diaphragm wall panels, maximum 6.8m long, was studied concurrently with the physical works on site. Figure 4 below shows two adjacent cells, one with short panels, primary / closing arrangement, and another already incorporating long panels, primary / successive / closing arrangement. Since the works were ongoing at the time of the trial the long panel layout was adopted in 8 circular cells out of the 15 and to the final straight portion of the ELS. The alternative reduced the number of diaphragm wall panels by about half, which meant reducing the associated time consuming "milled joints" between primary / closing panels. The construction programme was reduced by approximately 15% with obvious beneficial impacts to cost, environment, and quality. The final total number of diaphragm wall panels constructed in the hybrid 15 circular cell ELS scheme was 424 nos.

Y-Panels are located between circular cells and are required to transfer the hoop stresses from the arch panels of each cell through the cross walls and reinforced concrete struts to the cofferdam's opposite side, see Figure 4, Figure 5 and Plate . Trench stability and lateral ground movement were of concern for these 5 bite-panels due to their size, 3.6m x 6.5m, and therefore, it was envisaged that ground improvement using Cutter Soil Mixing was required, see Plate 2. A total of 30 nos. of Y-Panels were required for the successful execution of the hybrid ELS Scheme.

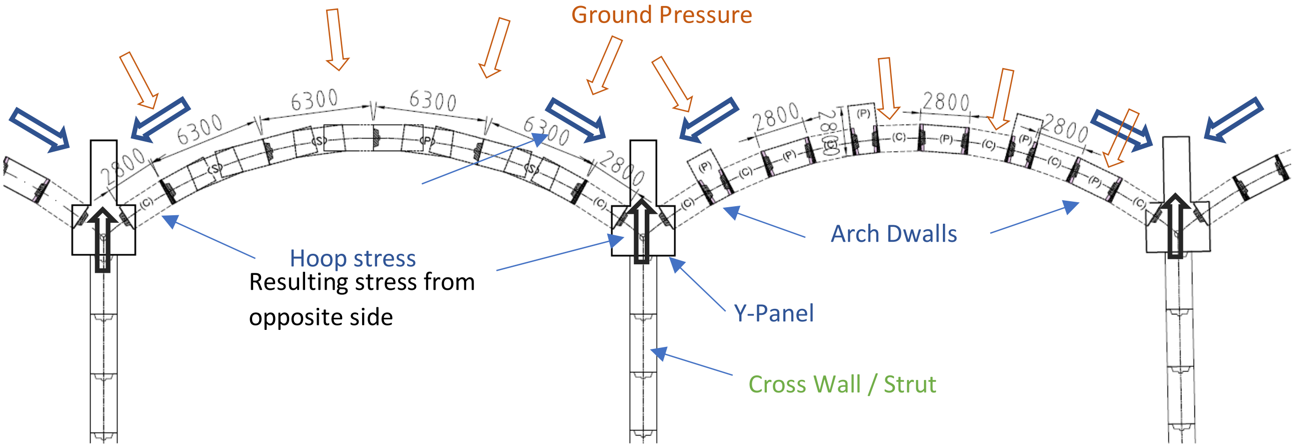


**Figure 4**: Adjacent cells, left-hand side with the long panels alternative incorporated and right-hand side with short panels

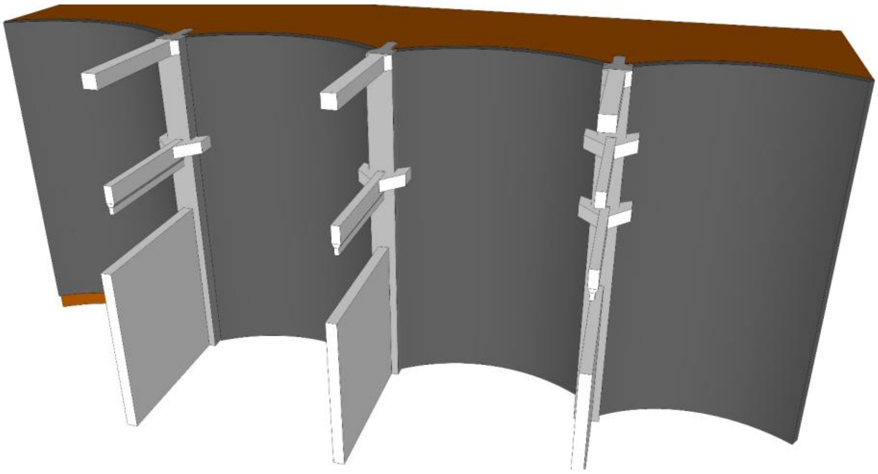

**Figure 5**: Simplified 3D View of ELS System

**Plate 1:** Exposed Y-Panel with Struts and Arch Cells

**Plate 2:** Cutter Soil Mixing for Y-Panel

The most common methods for trench stability design were reviewed in terms of their applicability to this project. The methods highlighted in Sections 3.1 and 3.2 above, rely on empirical assumptions to define the extent of soil arching and the failure wedge / block slip failures. In addition, the limit equilibrium methods require an iterative process to ensure that all possible failure mechanisms are studied, which can be time consuming if not automated. The presence of fine-grained soil, i.e., MD and Alluvium Silt / Clay, at TMCLKL site and the fact that diaphragm wall panel trench excavation is a short-term loading, benefit a method that can consider the undrained behavior of these soils. The soil pressure balance methods ignore the undrained behavior of cohesive materials, which is one of the known limitations. This is even more critical in soft cohesive materials like the ones present on site due to the inherent difficulty to determine accurately the drained parameters. Moreover, these parameters generally govern design when compared to the undrained parameters. The analytical methods were devised based on a straight panel and therefore, could not accurately consider the complex geometry of the Y-Panel. Plaxis 3D was the chosen method for the design of trench stability; however, trench stability was also calculated using the pressure balance method for the long panel for comparison and due to its historical significance in Hong Kong.

To validate the trench stability design and access expected lateral ground movements, fully instrumented site trials, one for each kind, long panel and Y-panel, were undertaken. The site trials were done outside of the footprint of the future ELS but at a location where the geological / geotechnical conditions were most adverse. This way the trials are considered conservatively representative for the whole site. For both the long and Y-Panel trials, bentonite properties were checked as per normal operational procedures, that were also followed during the construction works. A fresh mixture was checked twice per shift while during excavation the bentonite properties were checked three times per panel every 24h. When recycled bentonite was being used its properties were checked once per bentonite poll per 24h with a minimum of two tests. The monitoring plan specified that the inclinometers were checked every 8h during the whole trial period. Koden tests were done once per shift and once a day during excavation and the extended monitoring period, respectively. The inclinometer readings, and to a lesser degree the Koden test results, allowed determining if excessive ground lateral movement or trench necking was occurring. The trench's depth was measured every shift during the extended monitoring period to ensure that no collapse was occurring, which would cause material to accumulate at the bottom of the trench.

### 4.1 Loading Conditions

Different plant / equipment was used to excavate the diaphragm wall trench. Five loading conditions were studied and detailed as follows, general 20kPa, mechanical grab, hydraulic grab, BC40 cutter mounted both on a MC96 and a MC64 base machines. The plant properties and loading conditions are summarized in Table 3.

For the method detailed in Section 3.1 the load from the plant was distributed through the plant's footprint (AxB), and a load spread of 45 degrees was considered until the load reached the face of the panel (A' x B'), see Figure 6. The surcharge from the plant was then added at the determined depth while at the surface a nominal 5kPa load was considered. In Plaxis 3D, the load case assuming that the load is spread through the whole plant's footprint (AxB) was analyzed; however, additional load cases were considered by applying the plant's load directly on its tracks. In addition, another load case considered there was an unbalanced load, which was added to the track load of the track closest to the trench, $q_1$, and subtracted from the other, $q_2$. These originated different loading conditions, both in terms of load and distance to the trench, see Table 3, Figure 7 and Figure 8.

**Table 3**. Summary of Plant Properties and Loading Conditions

| Plant | Track Width (m) | Track Length, A (m) | Length, B (m) | Distance to face of trench, D (m) | Plant Load (kN) | Unbalanced Load on Track (kN) |
|---|---|---|---|---|---|---|
| B: Mechanical Grab | 0.8 | 5.36 | 4.90 | 3.50 | 1400 | +/- 344 |
| C: Hydraulic Grab | 0.8 | 4.70 | 4.47 | 2.50 | 1150 | +/- 399 |
| D: BC40 & MC96 | 1.0 | 6.10 | 5.31 | 2.90 | 2070 | +/- 586 |
| E: BC40 & MC64 | 0.9 | 5.75 | 5.05 | 2.90 | 1700 | +/- 593 |

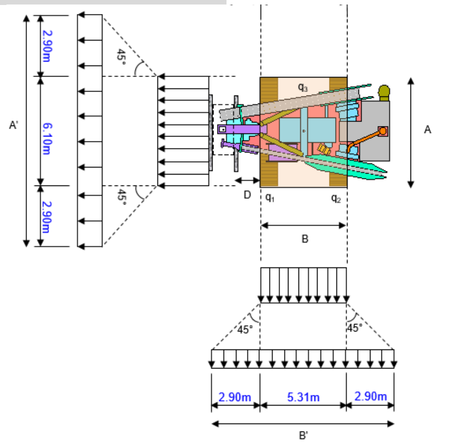


**Figure 6:** Load Case D for Pressure Balance Method

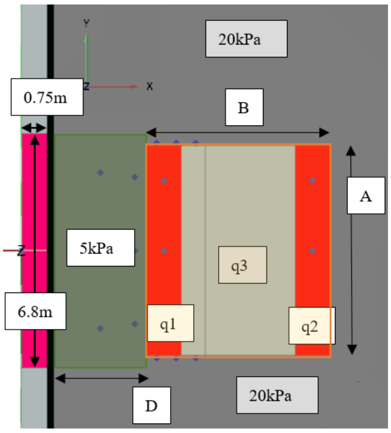


**Figure 7:** Typical scheme for Plant Load Cases (Long Panel), Plaxis 3D

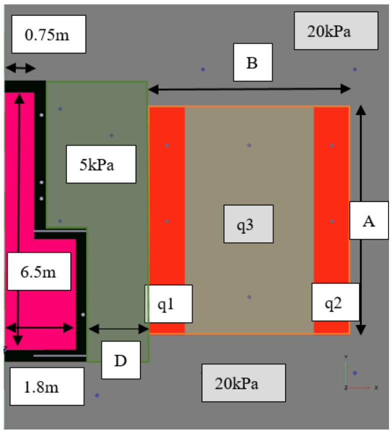


**Figure 8:** Typical scheme for Plant Load Cases (Y-Panel), Plaxis 3D

## 4.2 Long Panel

Trench stability calculations were carried out using the soil pressure balance method, Section 3.1, as well as the finite element analysis method, Section 3.3, for a 6.8m long, 1.5m thick and 56m deep panel. The calculations assumed that the existing ground level was at +5.5mPD and that the ground water table was 2.0m below ground (+3.50mPD). The 0.2m thick guide wall protrudes 0.2m from the ground and is 1.2m below ground, which means it has a total length of 1.4m. The bentonite slurry level was maintained 0.2m below the guide wall level (+5.5mPD) with a bulk density of 10.6 kN/m$^3$. The trench stability factor of safety using the soil pressure balance method for the several plant load cases and the overall 20kPa load are summarized in Table 4. The method considered the drained propertied of fine-grained soil (DMD, MD and Alluvium Clay). Figure 9 shows the effective horizontal stress and the bentonite pressure versus depth. From Table 4 and Figure 9 it is possible to determine that the pressure balance method estimates failure to occur, FS < 1.2, at two different locations. All load cases show a possible failure mechanism developing at the interface between the Fill and the Disturbed Marine Deposit while load case no. 1 (20kPa) also indicates a possible failure just below the guide wall.

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**Table 4**. Summary of Trench Stability FS Using the Soil Pressure Balance

| Load Case \ Plant | | A: 20kPa | B: Mechanical Grab | C: Hydraulic Grab | D: BC40 & MC96 | E: BC40 & MC64 |
|---|---|---|---|---|---|---|
| 1 | 20kPa | 1.13 | - | - | - | - |
| 2 | Load Distributed by Plant's Footprint (A'xB') | - | 1.14 | 1.14 | 1.12 | 1.13 |

Based on symmetry conditions the panel was modelled in Plaxis 3D as a 0.75m thick and 6.8m long trench, see Figure 7 and Figure 12. The model was extended in all three directions for at least 20m beyond the trench to ensure that deformations and stresses could develop. The load cases and plant considered in this analysis as well as the calculated FS is summarized in Table 5. Safety calculations were undertaken for all loading cases and Figure 10 shows the determined failure mechanism per load case. The minimum FS to prevent failure was maintained at 1.2, similarly to what was assumed for the balance pressure method. From Table 5 only the load case no. 4, unbalanced load towards the front track is under the 1.2 threshold. However, after inspection of the safety calculation output, see Figure 10, and the FS plot vs displacement, see Figure 11, it can be determined that the failure mechanism that drives the FS occurs under the track. Since the objective of the study is the trench stability a new load case no. 5 assumes that a typical 6.0m long, 2.4m wide and 30mm thick steel plate is placed centered to the front track to spread the load. Other methods could have been considered, e.g. replacing the top of the Fill with a more competent working / piling platform. The calculated FS for the new load case 5 is in line with the FS determined for load cases no. 1 to 3, which indicated a deeper failure within the MD and Alluvium Clay, see Figure 10. However, due to the increased pressure at the surface, the safety stage shows increased movement at the trench located at the underside of the guide wall. Although Plaxis 3D confirmed the stability of the trench the estimated soil lateral movements in the plastic stages, approximately 200mm, would be unacceptable. Therefore, a site trial was deemed required to confirm if trench squeezing could occur. The findings of the site trial and comparison with Plaxis 3D results are further discussed in Section 5 below.

**Table 5**. Summary of Long Panel's Trench Stability FS Using Plaxis 3D

| Load Case \ Plant | | A: 20kPa | B: Mechanical Grab | C: Hydraulic Grab | D: BC40 & MC96 | E: BC40 & MC64 |
|---|---|---|---|---|---|---|
| 1 | 20kPa | 1.45 | - | - | - | - |
| 2 | Load Distributed by Plant's Footprint (AxB) | - | 1.44 | 1.45 | 1.44 | 1.45 |
| 3 | Equal Load on Each Track | - | 1.44 | 1.45 | 1.43 | 1.44 |
| 4 | Unbalanced Load on Tracks | - | 1.44 | 1.12 | 1.16 | 1.17 |
| 5 | Unbalanced Load on Tracks with Plate | - | - | 1.45 | 1.43 | 1.44 |

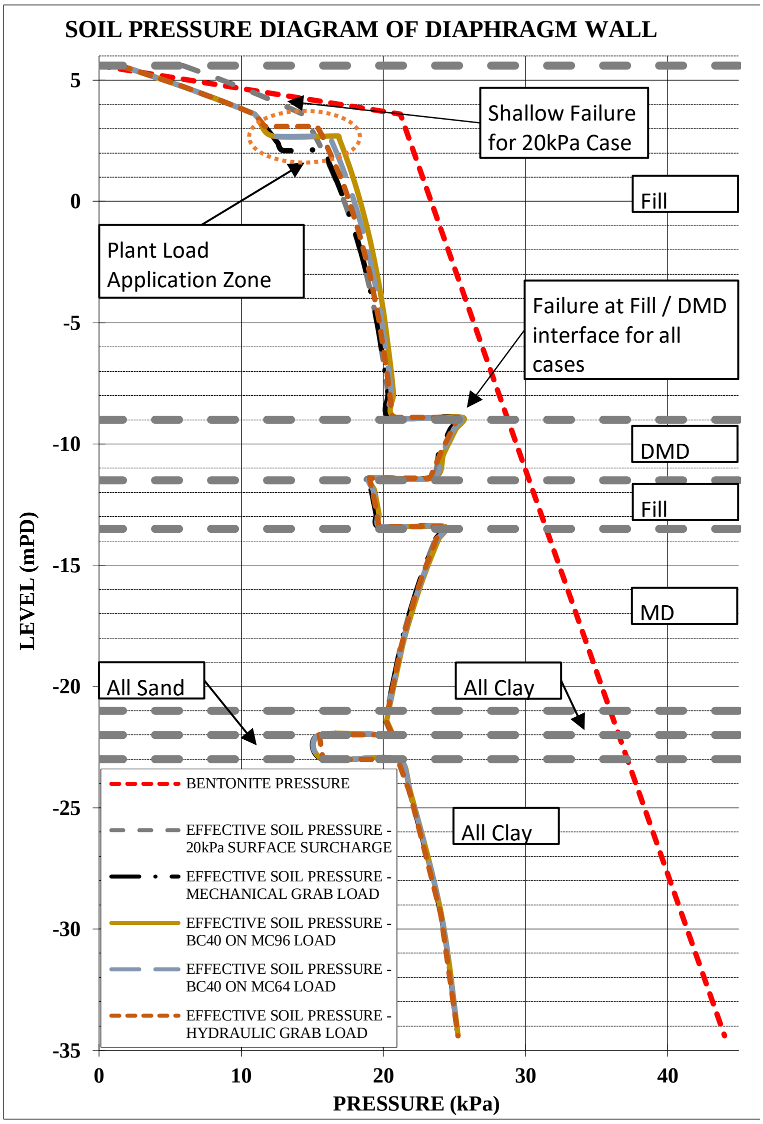



**Figure 9:** Pressure Diagram of Diaphragm Wall Trench

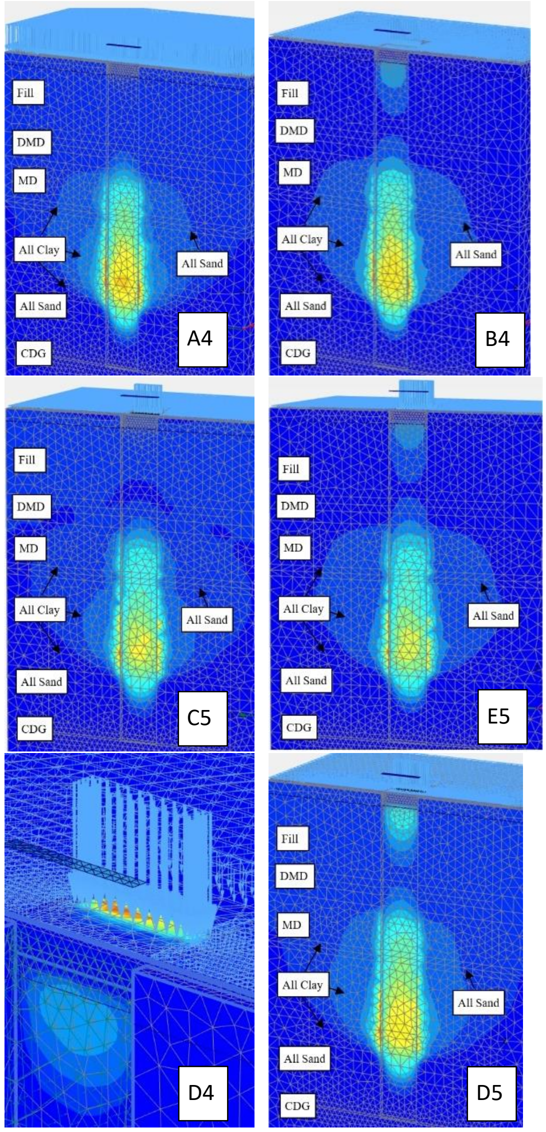



**Figure 10:** Failure Mechanism for all Load Cases

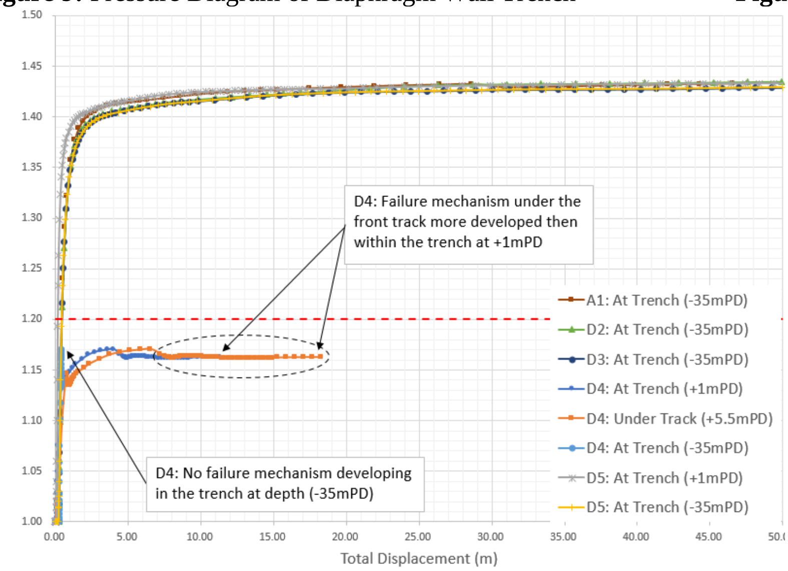



**Figure 11:** Plaxis 3D Safety Stages: Calculated FS vs Displacement

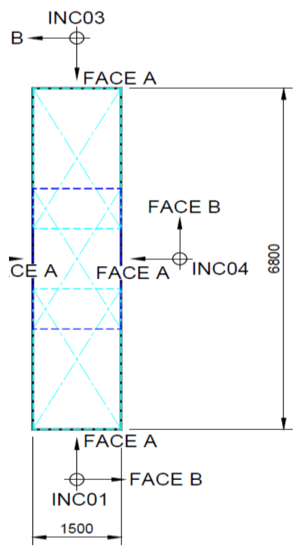



**Figure 12:** Inclinometer Location at Long Panel

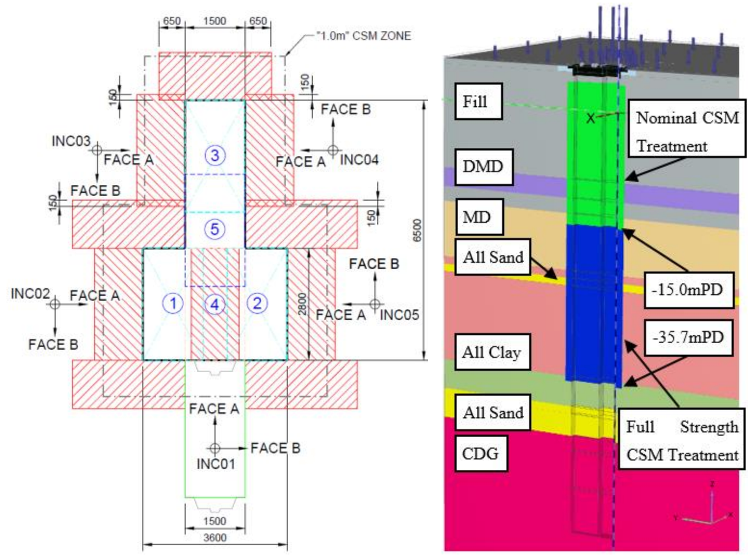
### 4.3 Y-Panel

The Y-Panel is a 56m deep, 5-bite panel, 3.6m x 6.5m, see Figure 13, and due to its geometry the pressure balance method and the limit equilibrium methods are not suitable to accurately calculate the trench stability. Therefore, the Y-Panel's trench stability was only calculated using finite element analysis, Plaxis 3D. The guide wall's allowable bearing capacity had to be increased due to the heavy weight of the steel reinforcement cages, up to 140 tons. Its thickness increased to 0.3m and horizontal sections of reinforced concrete around the whole perimeter of the Y-Panel were added, 0.7m to 1.0m long. To increase the stability at the top of the trench and ensure a better load distribution from the plant the top 0.7m of Fill was replaced by compacted working platform, soil properties as shown in Table 1. Remaining assumptions, i.e. bentonite level and density, guide wall depth below ground, remained the same as per what was described for the long panel.

Four load cases were considered in this calculation, A1, E2, E3 and E4 as shown in Table 6. The calculated factor of safety for the Y-Panel was initially unsatisfactory for load case E4 with no ground improvement, FS inferior to 1.2, see Table 6. Similarly, to the long panel the model showed that a failure mechanism was developing under the track; however, for the Y-panel case the failure mechanism developed through the track to the underside of the guide wall as the model showed similar ground movement at both locations. In addition, this failure mechanism occurred even when a more competent piling / working platform was provided instead of the Fill. The plastic calculation stages were also showing unacceptable ground lateral movement (in excess of 250mm) for all the load cases, see Figure 14. Therefore, a solution to ensure both adequate trench stability and limit lateral movements was required.

**Table 6**. Summary of Trench Stability FS Using Plaxis 3D

| Load Case \ Plant (E: BC40 & MC64) | | Unimproved Ground | | Ground Improvement with CSM | |
|---|---|---|---|---|---|
| | | A: 20kPa | E: BC40 & MC64 | A: 20kPa | E: BC40 & MC64 |
| 1 | 20kPa | 1.37 | - | 1.70 | |
| 2 | Load Distributed by Plant's Footprint (AxB) | - | 1.37 | - | 1.70 |
| 3 | Equal Load on Each Track | - | 1.37 | - | 1.70 |
| 4 | Unbalanced Load on Tracks | - | 1.14 | - | 1.70 |



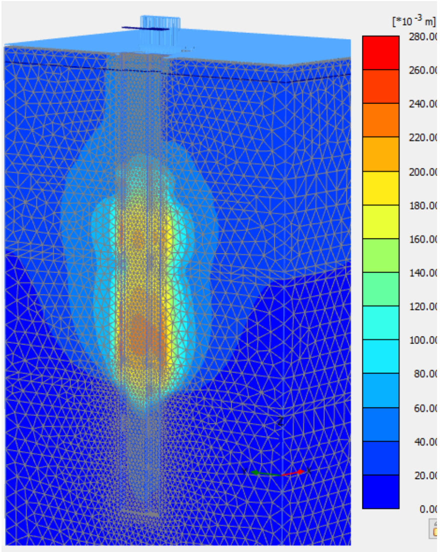



**Figure 13:** Y-Panel diaphragm wall excavation sequence and CSM ground treatment on plan and section views

**Figure 14:** Ground Lateral Movement for Y-Panel Excavation without CSM

Cutter Soil Mixing (CSM) was chosen as the ground improvement method to enhance trench stability and limit lateral ground movements, due to both the applicability and availability of equipment on site. Through the design development stage, it was determined that a 1.0m perimeter ground treatment was required to ensure trench stability and limit lateral movements, see Figure 13. To achieve this treatment envelope 9 nos. of 1.2m x 2.8m CSM bites were required with an extra CSM bite in the middle of the panel so that the required verticality limit, 1/400, could be achieved. The ground improvement had two target strength requirements, the shallower Fill, Disturbed MD and upper part of the natural Marine Deposit (approximately 20.5m) were treated with a nominal ground improvement (UCS = 0.5 MPa). The underlaying Marine Deposit and Alluvium (approximately 20.7m) were fully treated to achieve an UCS of 1.0 MPa. The ground improvement was modelled using the Mohr-Coulomb material model with undrained behavior and conservatively it neglected any tension cut-off resistance. Figure 15 and Figure 16 show the failure mechanisms (safety calculation) without ground improvement and with the CSM treatment, respectively. Table 6 summarizes the Y-Panel's trench stability factor of safety for both treated and untreated ground conditions for all load cases.

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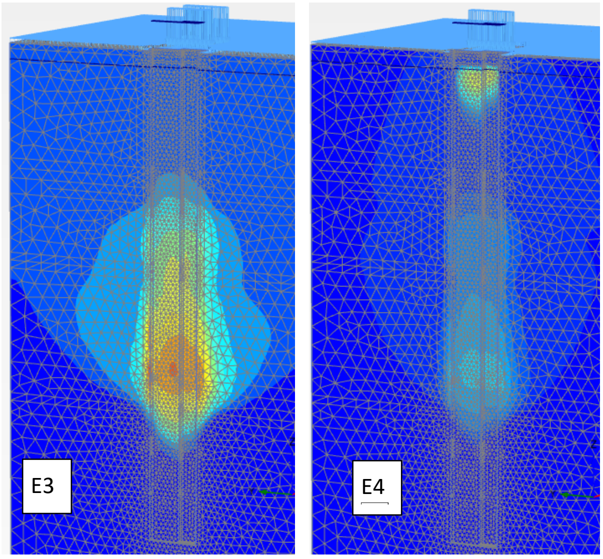


**Figure 15:** Y-Panel Failure Mechanisms without CSM

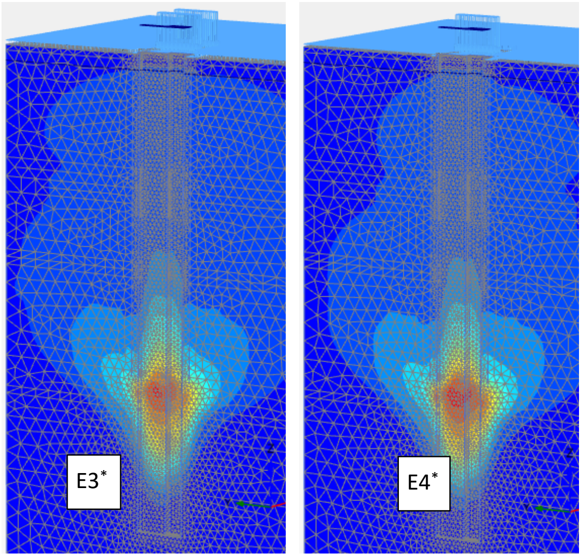


**Figure 16:** Y-Panel Failure Mechanisms with CSM*

## 5 Trench Stability Lateral Ground Movement Review

A total of four inclinometers, one at each side of the panel, see Figure 12, were installed to monitor the ground's lateral response of the long / triple bite panel's trench excavation. The panel excavation ended on the 24 of January 2018 with a recorded maximum ground movement of less than 10mm towards the excavation, INC02 Face A. Considering the same plan location as the inclinometer, Plaxis 3D estimated a maximum lateral movement of about 200mm towards the excavation. Figure 17 below shows a comparison between the two and while Plaxis 3D gave a reasonably accurate lateral ground movement profile with the maximum occurring within the Alluvium Clay the magnitude of this movement was greatly overestimated, Face A direction. Plaxis 3D calculated negligible movement along the perpendicular direction, Face B. Four days after the excavation, timeframe considered for preparation works to be done prior to casting the diaphragm wall panel, the recorded displacement had increased to about 22mm. The panel was left open for an additional two weeks to study its behavior and the maximum ground lateral movement recorded by the inclinometers was less than 35mm. Recorded displacement in the perpendicular direction (Face B) was about 21mm. The inclinometer in the opposite side recorded less movement towards the excavation, 24mm, but similar movement in the perpendicular direction. The most likely scenario is that a combination of reasons is responsible for the over-estimation of lateral ground movement calculated by Plaxis 3D rather than a particular one, e.g. the use of the Mohr-Coulomb material model rather than more advanced models, excessively conservative deformability parameters particularly when used for trench excavationThis latest reason might have been by os, which might have been exacerbated by ongoing ground improvement effects.

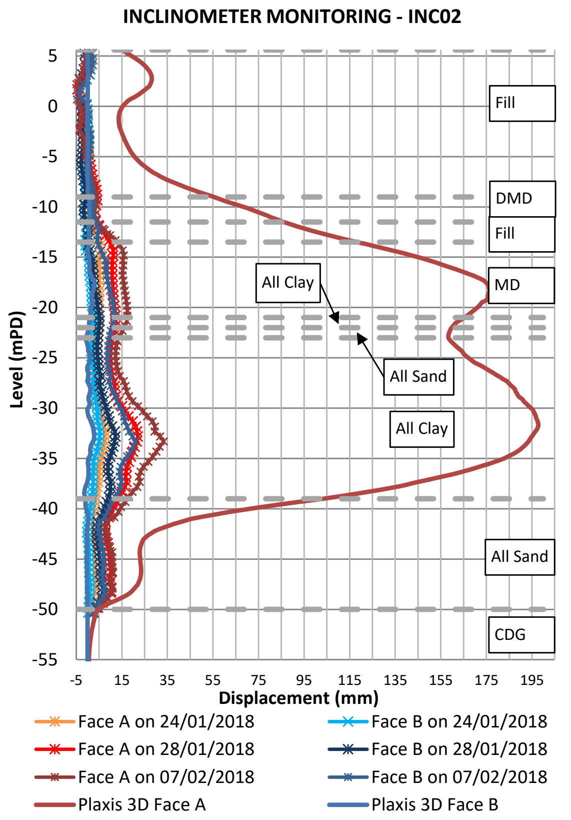


**Figure 17:** Long Panel Inclinometer Monitoring Results

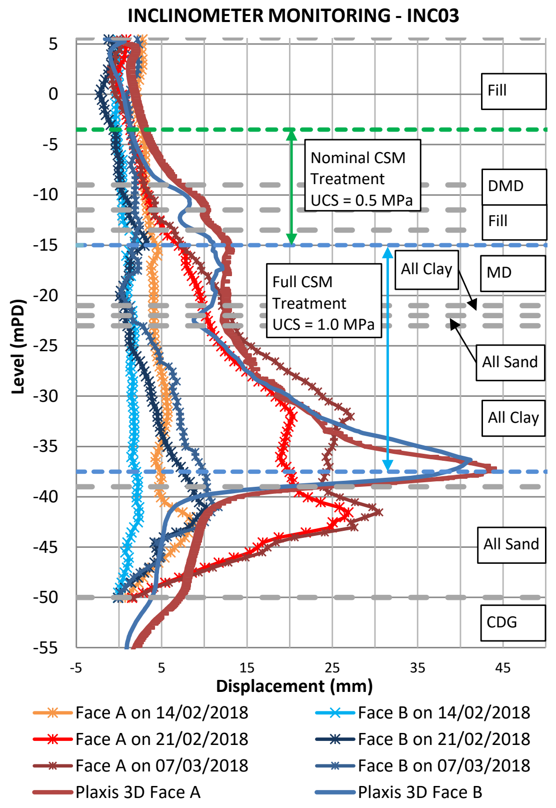


**Figure 18:** Y-Panel Inclinometer Monitoring Results

Similarly, the Y-Panel was instrumented with five inclinometers to register the ground's lateral response to the trench excavation, see Figure 13. The Y-Panel excavation ended on the 14 of February 2018 with a recorded maximum ground movement of less than 10mm towards the excavation, INC03 Face A, approximately 5m below the CSM treatment. The recorded ground movement towards Face B was well under 5mm. The ground lateral movement prediction from Plaxis 3D towards the excavation (Face A) and at the same plan location as the inclinometer was just under the 45mm mark. This movement was estimated to occur at the interface between the toe of the CSM treatment and the natural Alluvium Clay. Plaxis 3D estimated a similar lateral ground movement in both and profile when comparing the movement occurring in the direction towards the excavation (Face A) with its perpendicular direction (Face B). Assembling the Y-Panel cages is a more complex procedure and therefore, the timeframe from excavation to casting was increased to 7 days. The recorded displacement by the inclinometers in this time step increased to just under 30mm. Similarly, to the long panel, the Y-Panel was left open for another two weeks at which stage the maximum ground lateral movement recorded by the inclinometers was approximately 32mm, Face A. The final recorded movement by towards the perpendicular direction (Face B) was about 12mm. The Plaxis 3D overestimated the ground movements, especially in the perpendicular direction to the closest excavation face, Face B.

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## 6 Conclusions

The trench stability design for the long / triple bite panel using the pressure balance method proved to be too conservative for the TMCLKL ground conditions, when compared to the results obtained from both the site trials and Plaxis 3D. It identified two failure mechanisms, a shallow failure for the 20kPa general surcharge and a deeper failure at the interface of the Fill and Disturbed Marine Deposit for the other load cases. Plaxis 3D requires the user to have experience in identifying the governing failure mechanism and to ensure that these are both fully developed and relevant to the analysis. The Plaxis 3D trench stability design was validated by the site trials as no failure mechanism was identified in either. Despite this the FEA greatly overestimated the predicted lateral movement of the trench when compared to the monitored results from the trial. Based on the findings of the long panel trial the panel layout was updated in 8 circular cells, which lead to a saving of half the number of panels for those cells. This change represented a construction programme saving of about 15%.

The FEA allowed for the trench stability of the Y-Panel to be calculated since the other referenced methods could not accommodate its complex geometry. Initial analysis determined that the estimated lateral movement and the trench stability were not satisfactory. Therefore, a ground improvement scheme using CSM was analyzed. Plaxis 3D was used to refine the required strength of the CSM that was divided into two zones, a nominal treatment and full treatment with UCS values of 0.5 MPa and 1.0 MPa, respectively. The extent of treatment was also determined, 1.0m perimeter along the trench and about 41m deep. The site trial of the 5-bite panel confirmed the stability of the trench as per the design; however, the calculated lateral ground movement was overestimated, especially in the direction perpendicular to the closest excavation face. The Y-Panel was essential for the successful execution of the hybrid ELS scheme with 15 consecutive circular cells.

## References

[1] Huder, J. “Stability of Bentonite Slurry Trenches with some Experiences in Swiss Practice,” in Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, Vol. 1, Madrid, Spain, 1972

[2] Schneebeli, G, “La Stabilite Des Tranchees Profondes Forees en Presence de Bour,” La Houille Blanche, N. 7, 1964

[3] G. C. Y. Wong, “Stability analysis of slurry trenches,” Journal of Geotechnical Engineering, vol 110, no. 11, pp 1577-1590, 1984

[4] N. Morgenstern and I. Amir-Tahmasseb, “The Stability of a slurry trench in cohesionless soils,” Geotechnique, vol. 15, no. 4, pp 387-395, 1965

[5] G. Aas, “Stability of slurry trench excavations in soft clay,” in Proceedings of the 6th European Conference on Soil Mechanics and Foundation Engineering, pp 103-110, International Society of Soil Mechanics and Foundation Engineering, Vienna, Austria, March 1976

[6] W. Brzakala, K. Gorska, “On Stability Analysis of Slurry-Wall Trenches”, Institute of Geotechnical and Hydroengineering, Faculty of Civil Engineering Wroclaw University of Technology Wybrzeze Wyspianskiego 27, 50-370 Wroclaw, Poland, 2008

[7] Grands-Tavera C E, Triantafyllidis T. “Simulation of a corner slurry trench failure in clay” [J]. Computers and Geotechnics, 2012, 45: 107–117

[8] Li Wei, Zhang Chengping “Stability Analysis of a Slurry Trench in Cohesive-Frictional Soils” Open Geosciences, Vol 11, Iss 1, Pp 888-900, 2019