*Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)*


# Comparing two- and three-dimensional finite element analysis of a cement stabilized trench with actual displacement monitored after excavation

## Comparaison des analyses bi-dimensionnelle et tri-dimensionnelle par éléments finis d'une tranchée stabilisée au ciment avec le déplacement réel surveillé après excavation

**Somaye Sadeghian**, Chris Grimes & Nicola Ridgley
*Beca Ltd, New Zealand, somaye.sadeghian@beca.com*

ABSTRACT: The paper is going to discuss analysis, design and instrumentation and monitoring data of Daldy Street Outfall Extension Project (DSOE) in Auckland, New Zealand. The DSOE was completed by the Wynyard Edge Alliance (WEA) for Auckland Council (Healthy Water) to enable improvements in water quality in the inner harbour basin. The scope of DSOE was to extend the 3 to 3.5m diameter High-Density Polyethylene (HDPE) pipe beneath Brigham Street from its current outfall position underneath North Wharf, to a new outfall position 510m to the north at the end of Wynyard Point. The installation of the pipeline requires 5.5m deep excavation adjacent to critical pipelines and tanks to the west and to the harbour to the east. Cutter Soil Mixing (CSM) has been adopted as the preferred ground improvement methodology to stabilise the 5.5m deep temporary trench. To assess the displacement relative to the adjacent tanks and pipelines, two- and three-dimensional Finite Element Analysis using PLAXIS2D and PLAXIS3D were carried out. Besides, during the excavation of the trench sections, lateral displacements of the CSM panels were monitored. The monitoring data shows good agreement with finite element analysis. This paper outlines the design and compares estimated design performance with the actual monitored results.

RÉSUMÉ : Cet article présente la conception et l'analyse des données d'instrumentation et de surveillance du projet d'extension de l'éxutoire de la rue Daldy à Auckland (Healthy Water) , en Nouvelle-Zélande. Le projet d'extension de l'éxutoire de la rue Daldy (DSOE) à Auckland, en Nouvelle-Zélande, a été réalisé par la Wynyard Edge Alliance (WEA) pour le Conseil d'Auckland afin de permettre l'amélioration de la qualité des eaux dans le bassin intérieur du port. L'objectif de la DSOE était de prolonger les canalisations de la rue Brugham - tuyaux de 3 à 3,5m de diamètre en polyéthylène haute densité (PEHD), de leur point de rejet actuel sous le North Wharf (Quai Nord) jusqu'à un nouvel éxutoire situé à 510m au nord, à l'extrémité de Wynyard Point (Pointe Wynyard). L'installation de cette section additionnelle nécessite une excavation de 5,5m de profondeur à proximité de canalisations et des réservoirs critiques existants. (à l'ouest et du port à l'est). La méthode de mélange de sol par fraise (Cutter Soil Mixing - CSM) a été privilégiée pour l'amélioration du sol afin de stabiliser la tranchée temporaire de 5,5 m de profondeur. Pour évaluer le déplacement induit par l'excavation au niveau des réservoirs et canalisations adjacents, une analyse par éléments finis en deux et trois dimensions utilisant PLAXIS2D et PLAXIS3D a été réalisée. En outre, pendant l'excavation des sections de tranchées, le déplacement latéral des panneaux de MSC a été surveillé. Les données de suivi montrent une bonne corrélation avec l'analyse par éléments finis. Cet article décrit la conception et compare les performances estimées de la conception avec les résultats réels de la surveillance de l'excavation. Le document présente également les principaux enseignements tirés de la modélisation numérique du MSC.

KEYWORDS: PLAXIS2D, PLAXIS3D, cutter soil mixing, ground improvement.

## 1 INTRODUCTION

### 1.1 *Project Background*

The project involved the relocation of the existing Daldy Street Outfall, from the south-western corner of Wynyard Basin to the northern end of Wynyard Point. HDPE pipe (3.0m-dia southern section, 3.5m-dia northern section) was laid beneath Brigham Street to form a connection between the existing outfall and the relocated outfall. The total length of the new pipeline was 510m. For access and constructability, the project was undertaken in two discrete sections, the Southern section of Brigham Street and the Northern section of Brigham Street, Figure 1.

CSM as the selected ground stabilization technique was carried out along the western boundary of Brigham Street, to a maximum depth of approximately 9m below the current road level. The purpose of the stabilization was to enable a 5m deep excavation of trench with self-supporting near-vertical sides for installation of the pipe. Once formed, the pipeline was laid in 15m lengths and the trench was then backfilled (refer Figure 2).

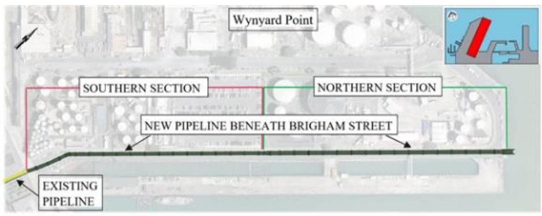


Figure 1. Plan View of the Brigham Street - Distinguishing Between Southern and Northern Sections

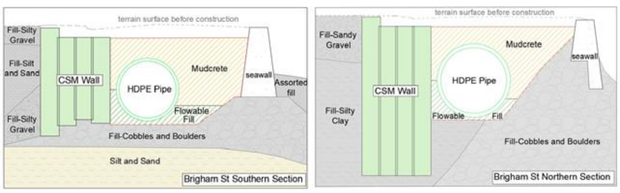


Figure 2. Cross-sections – Southern Section and Northern Section of Brigham Street

### 1.2 *Ground Condition*

The Wynyard Point reclaimed land, which includes the construction site and adjacent area, dates from the 1800s to the 1930s. The Wynyard reclamation was constructed across an old eroded river valley. The reclamations are bordered by seawalls and piled reinforced concrete wharves. The topography of the site is flat with ground levels varying from approximately +5.0 to +5.5m CD. The reclamation was developed by forming:

- A perimeter bund with basalt boulders and cobbles, placed on the seabed and constructed to a level of around 1.3m CD (to the base of the seawall), Figure 3.
- A rock-faced seawall, which was then constructed on top of the perimeter bund, before a haul road was constructed on the western side of the seawall.
- The haul road was constructed on the western side of the seawall, in the southern section, up to approximately chainage 265 (refer Figure 3). The exact width is not known, but it was likely used to provide access for hauling rockfill along parts of the reclamation edge prior to filling with dredging's (hydraulic fill). It is variable in character and typically comprises gravel and cobble sized material within a sandy silt matrix. The gravel and cobbles include basalt, concrete, bricks and other construction materials. Fragments of steel and pump housings and steel rebar were encountered in test pits and during the trench excavation.
- Hydraulic fill was encountered to the rear of the rock bund and haul road fill. It typically comprises sand, sandy silt, silty sand, silty clay and was encountered mainly within the northern section of CSM.

Groundwater level was at around 3.5mCD in the landside of the seawall and varies from low tide (+0.41mCD) to high tide (+3.8mCD) at the other side.

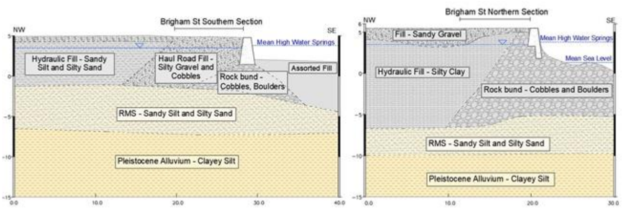


Figure 3. The Typical Geological Profile of Southern and Northern Section

The published geology of this area indicates that the reclaimed site is underlain by marine sediments then Pleistocene alluvium. Recent Marine Sediments (RMS) comprise fine sandy silt and silty sand with some clay and trace of shells. The Pleistocene alluvium comprises firm clayey silt and loose to medium dense sand. These sit on top of the East Coast Bay Formation alternating sandstone and mudstone (ECBF).

The area was initially utilized by the timber trade, and then in the 1930's, it started to be used for bulk petrochemical storage. Most of the contaminants in the site have originated from activities relating to the handling and storage of petroleum hydrocarbons and the historical disposal of gasworks waste during land reclamation. Contamination by hydrocarbons has been identified throughout the profile across the site, above adopted clean fill criteria.

### 1.3 *Cutter Soil Mixing*

Cutter soil mixing is a type of deep soil mixing method. For this method, overlapping panels form the continuous CSM wall and the ease of the process depends on the soil type.

For this project, cutter soil mixing was carried out by Wagstaff Australia. The CSM equipment used consisted of a two-wheel cutter head and a nozzle located between the wheels (See Figure 4). The wheels cut through in-situ soil while rotating around their horizontal axes. Simultaneously, cementitious material was pumped through the nozzle and mixed with the in-situ soil.

The soil stabilization in this project consisted of four rows of Cutter Soil Mixing (CSM) panels installed west of the proposed trench excavation, as shown in Figure 2. In each row, primary panels were installed first and once partial curing took place the secondary panels were installed (i.e hit and miss) with an overlap of 100mm. Row 1 was constructed first and continued with rows 3, 2 and 4, Figure 5.

Figure 4. Cutter Soil Mixing Head and Operating Rig

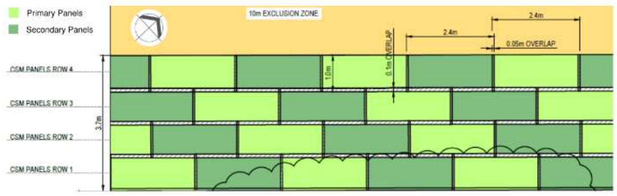


Figure 5. Construction Sequences of Continuous Walls with Primary and Secondary Panels

The panels were required to extend about 8.5m below the ground surface unless they encountered the perimeter rock bund and were designed to retain reclamation and hydraulic fill to a depth of 5m. The panels were 2.4m long and 1.0m wide, installed parallel to the road and excavation. Construction debris and some boulders were removed manually by grabs and excavator where the CSM refused at depths higher than the rockbund.

#### 1.3.1 *CSM Strength Properties*

Cutter soil mixing improves ground stiffness and strength parameters by mixing the in-situ soil with cementitious material.

Shear strength of the soil-cement (wet mixing) is typically taken to be half of the design UCS strength (Porbaha 2000, Kitazumi & Terashi, 2013). The typical tensile strength of CSM material is low, typically 10 – 20% of UCS. 15% is considered a reasonable value to adopt for design (Kitazumi & Terashi, 2013). CSM strength properties also depend on the in-situ soil properties.

In this project, the strength criteria were adopted as (a) the minimum specified UCS was 750 kPa and (b) the minimum specified average UCS was 1MPa. During construction, samples were taken: (a) during stabilization of the wet mix and (b) after stabilization of the cured mix.

Figure 6 and Figure 7 show the location of the samples, sampling method, Water/Cement Ratio (W/C ratio), and Unconfined Compressive Strength (UCS) of the sample. With W/C ratios of 0.6 to 0.9 applied during mixing, UCS fluctuated from ~800kPa to ~15 MPa. (Djoric, A., et. al., 2021)


Figure 6. Unconfined Compressive Strength (UCS) versus Water/Cement Ratio (Djoric, A., et. al., 2021)

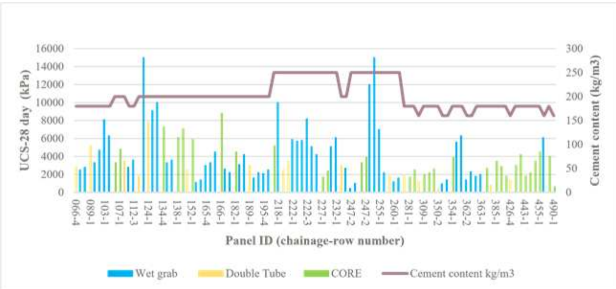


Figure 7. Unconfined Compressive Strength (UCS) versus Cement Content (Djoric, A., et. al., 2021)

The northern section (Chainage 260 to Chainage 490) comprised finer sandy/silty and clayey in-situ materials obtaining an average UCS of approximately 2.0MPa. Higher average strengths were obtained in more gravelly in-situ soils encountered in the southern section, (Chainage 066 to Chainage 255), with average strength approximately 4.0MPa. (Djoric, A., et. al., 2021)

## 2 NUMERICAL MODELING

Two- and three-dimensional numerical modelling was adopted to calculate the excavation-induced lateral displacement. Plaxis2D and Plaxis3D were used to perform Finite Element numerical modeling. Figure 8, Figure 9, and Figure 10 show the geometry of the numerical models.

In the southern section (Chainage 066 to Chainage 255) the majority of the panels terminated at the rockbund at depths 5m to 6m below ground level, however to the north where the fill was deeper, the majority of panels reached depths 7m-8.5m below ground level, except from Chainage 429 to Chainage 433 where the rock bund was shallow and panels were terminated at depth ~5m.

To the west of the northern section, there were exiting structures (tanks, pipelines and buildings) in the close vicinity to the trench. Some of these structures had small tolerances with respect to the lateral movement (~10mm). In the vicinity of the existing structures with low tolerable lateral movement, additional support rows were installed. Figure 8 and Figure 9 show the typical cross-sections along the southern and the northern sections, respectively. The sections in Figure 8 and Figure 9 were used in the numerical finite element model.

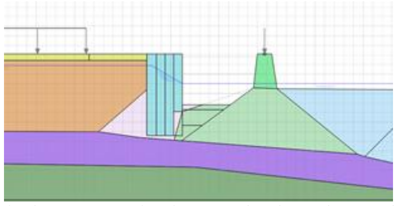

Figure 8. Typical CSM Termination Depth Along the Southern Section – first row was terminated at 5&6 mbgl; second, third, and fourth rows were terminated at 8mbgl

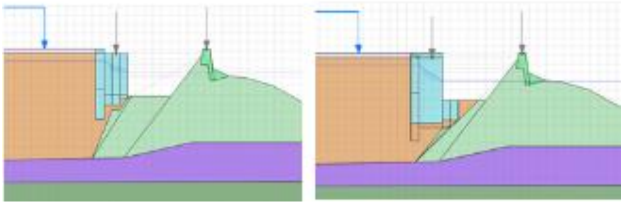

Figure 9. Typical CSM Termination Depth Along the Northern Section – majority of the CSM blocks were terminated at 8mbgl as shown in the right figure; additional support rows installed in limited areas with below 20mm allowable lateral movement; the left figure shows the typical cross section from Chainage 429 to Chainage 433 where the rock bund was shallow – first row was terminated at 5 m below ground level; second and third rows were terminated at 6.5mbgl, and fourth rows were terminated at 8mbgl

### 2.1 *Material Properties*

Material properties adopted in the design have been shown in Tables 1 and 2. Hardening Soil (HS) Constitutive model is a Plaxis built-in constitutive model which accounts for the confining stress dependency of the elastic modulus. HS model also allows for different elastic modulus in loading and unloading/reloading.

Haul road fill, Hydraulic fill, and GAP65 materials were modeled using HS model to account for unloading due to excavation as well as stress dependency of elastic modulus. The rest of the soil layers were modeled using Mohr-Columb (MC) Model.

Table 1. Material Properties (Mohr-Coulomb Model)

| Item | Unit | CSM | Rock Bund | Infill | RMS | UTG |
|---|---|---|---|---|---|---|
| **Material model** | - | MC | MC | MC | MC | MC |
| **Drainage type** | - | UB | UB | D | D | D |
| **Unsaturated Unit Weight** | kN/m³ | 18 | 15.5 | 19 | 19 | 19 |
| **Saturated Unit Weight** | kN/m³ | 18 | 15.5 | 19 | 19 | 19 |
| **Undrained Strength** | kN/m² | 250 | 39 | - | - | - |
| **Effective Cohesion** | kN/m² | - | - | 0 | 0 | 0 |
| **Effective Friction Angle** | ° | - | - | 40 | 40 | 40 |
| **Dilation Angle** | ° | - | - | 0 | 0 | 0 |
| **Elastic Modulus** | MN/m² | 60 | 12 | 80 | 80 | 80 |
| **Poison Ratio** | | 0.3 | 0.3 | 0.3 | 0.3 | 0.3 |

Table 2. Material Properties (Hardening Soil Model)

| Item | Unit | Hydraulic Fill | Haul Road Fill | GAP65 |
|---|---|---|---|---|
| **Material model** | - | HS | HS | HS |
| **Drainage type** | - | D | D | D |
| **Unsaturated Unit Weight** | kN/m³ | 16.5 | 16.5 | 21 |
| **Saturated Unit Weight** | kN/m³ | 16.5 | 16.5 | 21 |
| **Undrained Strength** | kN/m² | - | - | - |
| **Effective Cohesion** | kN/m² | 0 | 0 | 0 |
| **Effective Friction Angle** | ° | 28 | 30 | 38 |
| **Dilation Angle** | ° | 0 | 0 | 12 |
| **E50 [1]** | MN/m² | 9 | 9 | 35 |
| **Eur [2]** | MN/m² | 9 | 9 | 35 |
| **Eoed [3]** | MN/m² | 27 | 27 | 100 |
| **m [4]** | - | 0.45 | 0.45 | 0.66 |

[1] E50 = Secant stiffness in standard drained triaxial test in the HS model
[2] Eoed = Tangent stiffness for primary oedometer loading in the HS model
[3] Eur = Triaxial unloading/reloading stiffness in the HS Model
[4] m = Power for the stress-level dependency of stiffness in the HS Model

### 2.2 *Construction Stages and Boundary Condition*

The following construction methodology was envisaged and was incorporated into Plaxis 2D and Plaxis 3D:

1. Initiate the stress state and reset the displacement (Ground Water Table was modelled to be at 1.7m bgl and 2.7m below ground level, outside and inside the trench, respectively. During construction the actual excavation remained full of water, and could not be pumped out due to the contamination – See Figure 8 and Figure 9)
2. Construct the CSM block.
3. Carry out a staged excavation to the base of the trench (in the finite element model, excavation was broken down into three stages)
4. In the three-dimensional finite element model, stages 2 and 3 were executed for 30m-wide excavation and repeated for 45m, and 60m excavation advancement.

### 2.3 *Three Dimensional Model*

The length of the excavation is one of the driving factors which governs the actual excavation-induced lateral displacement. Two-dimensional analyses assume plane strain condition. This assumption results in a higher calculated lateral displacement as the opening length is assumed to be infinite.

In this project, the effect of the opening length on the maximum lateral displacement was studied by building three-dimensional models in Plaxis3D using the typical southern cross-section as shown in Figure 8.

Figure 10 shows the geometry of the model built in Plaxis3D. In the model the first row of the CSM panels was terminated at 5m below ground level and the second, the third, and the fourth rows were terminated at 8m below ground level.

Figure 11 shows the lateral displacement resulted from plaxis3D model with the opening length of 30m. The 30-m length of the opening allowed for the required working space, including: (a) the space required for the first segment of the pipe (15m) to be installed in the trench; and (b) the enough space for a stable slope (1m rise in 3m run) at the front face of the excavation.

The excavation continued for an additional 30m in two stages (45m-long and 60m-long openings) as it has been shown in Figure 12 and Figure 13. As agreed with the construction team, no further excavation proceeded until the first segment was backfield with cement stabilized excavated spoil. As such, the maximum opening length was 60m at all times during construction.

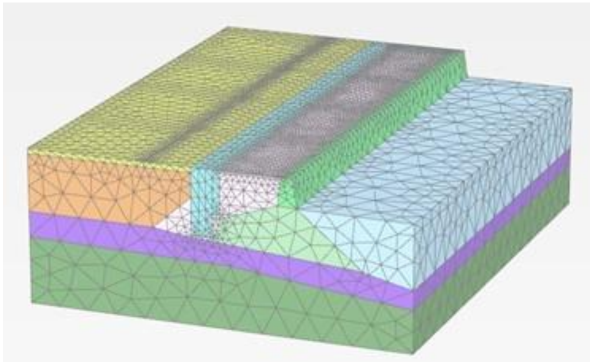

Figure 10. Geometry of PLAXIS 3D Model - first row was terminated at 5&6 mbgl; second, third, and fourth rows were terminated at 8mbgl

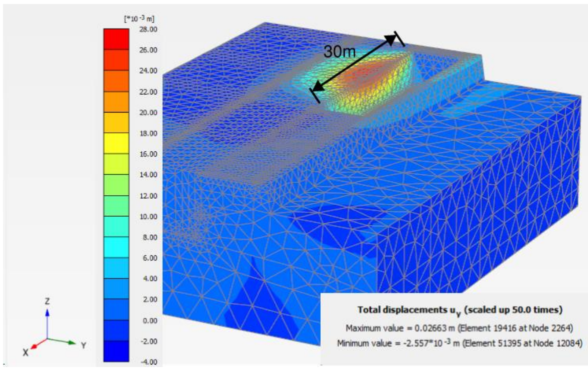


Figure 11. Lateral Displacement assuming 30m-long opening

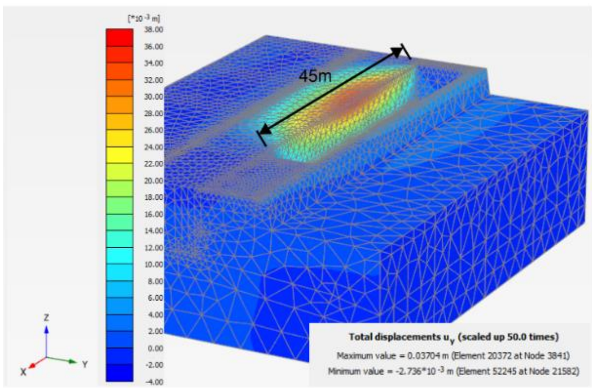


Figure 12. Lateral Displacement assuming 45m-long opening

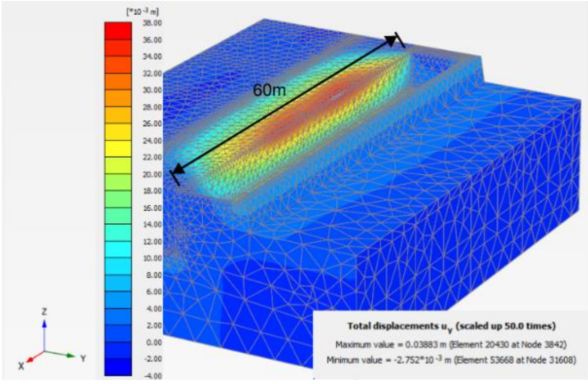

Figure 13. Lateral Displacement Assuming 60m-long opening

Figure 14 compares the maximum horizontal displacement calculated by assuming 30m, 45m, and 60m openings' length. The comparison showed by increasing the openings' length from 30m to 45m then to 60m the maximum lateral displacement increased from 14mm to 28mm to 38mm.

Figure 15 compares lateral displacement profiles along the edge of the CSM panels after 30m, 45m, 60m advancement of the excavation. The curve fitting showed that a polynomial equation with the order of two can be used to interpolate the lateral displacement.

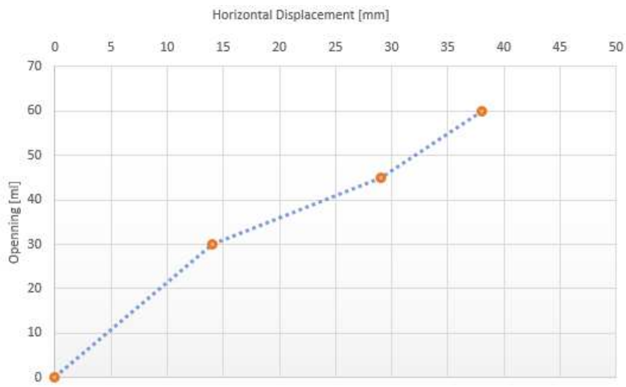

Figure 14. Opening's Length Versus Maximum Horizontal Displacement

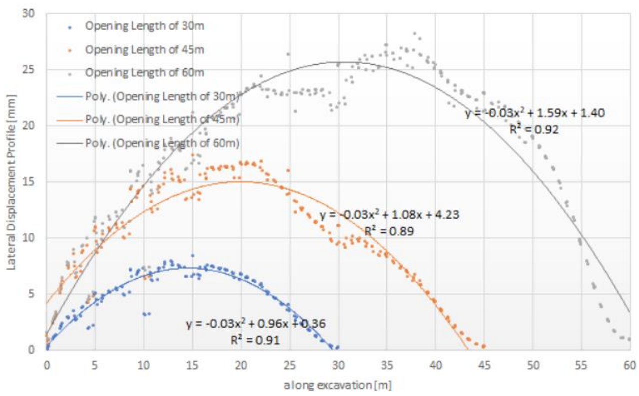

Figure 15. Horizontal Displacement Profile

## 2.4 *Two-Dimensional Finite Element Model*

Along the southern section the majority of the panels terminated at the rockbund at depths 5m to 6m below ground level. Figure 8 shows the typical cross-section along the southern section. Material properties which were adopted in the finite element analysis were the same as the 3D model as shown in Tables 1 and 2.

Figure 16 shows the lateral displacement of the CSM block estimated from Plaxis2D and Plaxis3D analysis. The Plaxis2D analysis estimated the maximum lateral displacement to be 30mm and 45mm for the CSM blocks with the depth of 5m and 6m in plane strain condition, respectively.

The typical cross-sections along the northern section were shown in Figure 9. Figure 16 shows the estimated lateral displacement from Plaxis2D model shown in Figure 9. In areas with 8m-deep rockbond, the finite element analysis showed without the support rows the maximum lateral displacement of the CSM panels are in the order of 40mm. However, by introducing support rows the maximum lateral displacement decreased from 40mm to 10mm. The finites element analysis also estimated the lateral movement of 7mm in areas with the shallower rock bund.

Key variations that had effect on the modelling included panel termination and rock bund depth, presence of the Haul Road and difference in water table within and outside of the trench.

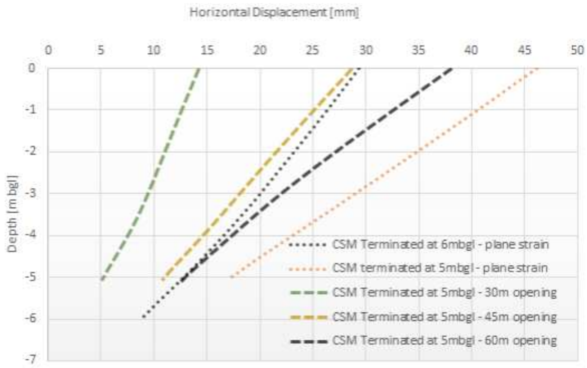

Figure 16. Lateral Displacement of the CSM Block Resulting from PLAXIS2D Analysis – Along Southern Section

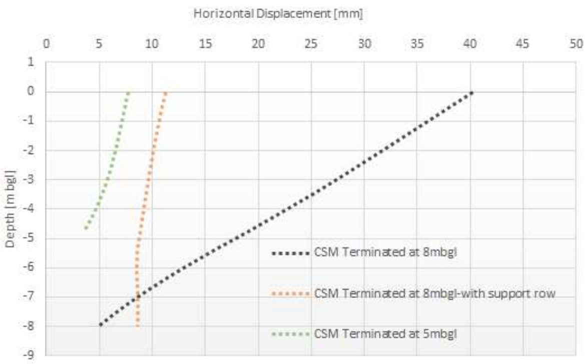

Figure 17. Lateral Displacement of the CSM Block Resulting from PLAXIS2D Analysis – Along Northern Section

## 3 INSTRUMENTATION AND MONITORING

As part of the instrumentation and monitoring program boreholes were drilled in the first row of the CSM blocks at around 15m centers Inclinometer casings were installed in the boreholes. Figure 18 shows the location of the inclinometers along the length of the works. This figure also shows the termination depth of the first CSM row.

During the excavation of the trench sections, lateral displacements of the CSM panels were monitored daily. Figure 18 shows the maximum deviation or lateral displacement measured at the top of each inclinometer. The measured deviations were smaller than 20mm. Figure 20 also shows the the estimated lateral displacement (from 2D model) at the top of the panel.

Figure 19 and Figure 20 show the deviation measured by inclinometer at sections shown in Figure 8 and Figure 9, respectively. To compare the actual values with the estimated deviation from 2D and 3D models, the analysis results were overlaid on Figure 19 and Figure 20.

The results shows 3D models predicts the displacement more precisely comparing with the 2D models.

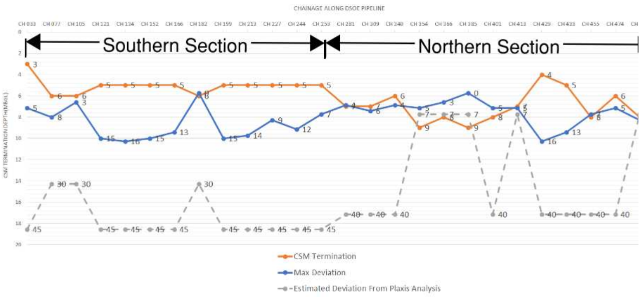


Figure 18. Summary of the measured versus calculated maximum lateral displacement

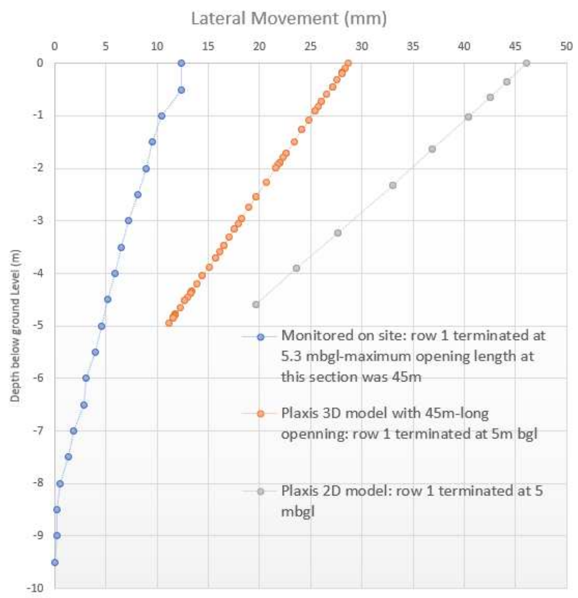


Figure 19. Deviation Measured by Inclinometers at the Southern Section

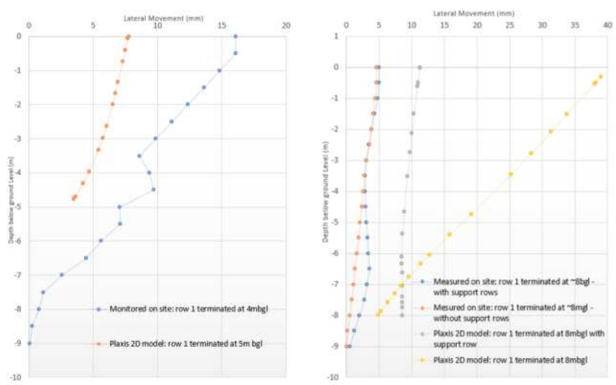


Figure 20. Deviation Measured by Inclinometers at the Northern Section

## 4 SUMMARY AND CONCLUSION

This paper discussed analysis, design and instrumentation and monitoring data of Daldy Street Outfall Extension Project in Auckland, New Zealand.

The scope of the project was to install the 510m long and 3 to 3.5m diameter HDPE pipe beneath Brigham Street. The installation of the pipeline required 5.5m deep excavation adjacent to the existing critical pipelines and tanks. CSM was adopted as the preferred ground improvement methodology to stabilize the landside of the 5.5m deep temporary trench. An existing seawall was at the seaside of the trench.

To assess the excavation-induced displacement, two- and three-dimensional finite element analysis using Plaxis2D and Plaxis3D were carried out. Plaxis 2D predicted the displacement to be approximately from 10mm to 45mm, depending on the geology and CSM termination depth. Plaxis 3D showed by limiting the opening length to 60m, 45m, and 30m the maximum lateral displacement decreased from 45mm (assessed from plane strain condition in Plaxis2D) to 38mm, 28mm, and 14mm, respectively.

During the excavation of the trench sections, lateral displacements of the CSM panels were monitored.

Instrumentation monitoring showed good agreement with the finite element analysis. And 3D model estimated the displacement more accurately compared with the 2D model.

Although 3D models estimate the displacement more accurately, it might not be feasible to model the whole site with 3D software. In this case, it is suggested to investigate 3D effects (e.g. similar to Figure 15) and apply the outcome to the 2D modeling results.

## 5 ACKNOWLEDGEMENTS

We thank Charlene Joubert for translating the abstract from English to French.

## 6 REFERENCES

Arnold, M. & Beckhaus, K. & Wiedenmann, U. 2011. Cut-off wall construction using Cutter Soil Mixing: a case study. *Geotechnik, Nr.*1, 34.

Djoric, A., et. Al. 2021. Cutter Soil Mix to retain a trench excavated in reclaimed land: a case study. Accepted for publication in *NZGS Symposium*, Dunedin, New Zealand.

Kermode, L.O. 1992. Geology of the Auckland urban area. Scale 1:50 000. Institute of Geological & Nuclear Sciences geological map 2. 1 sheet + 63 p. *Institute of Geological & Nuclear Sciences Ltd*. Lower Hutt, New Zealand.

Kitazume, M., and Terashi M. 2013. The Deep Mixing Method. *CRC Press/ Balkema, Leiden*, The Netherlands.

Kivelo M. 1997. Undrained shear strength of lime/ cement columns. *Proc. 14th International Conference on Soil Mechanics and Foundation Engineering*. 1173 – 1179

Nevarez-Garibaldi R. 2017. Influences of Test Conditions and Mixture Proportions on Property Values of Soil Treated with Cement to Represent the Wet Method of Deep Mixing. *MS Thesis, Virginia Polytechnic and State University*, Blacksburg, VA.

Porbaha, A. 2000. State of the art in deep mixing technology. Part IV: design considerations. *Ground Improvement 3*: 111 – 125. Thomas Telford, London.