**Proceedings of the ICBBG2025**
2025 International conference on Bio-mediated and Bio-inspired Geotechnics
18-20 May, 2025 | Tempe, Arizona, USA
*https://doi.org/10.53243/ICBBG2025-144*

ICBBG
18-20 May 2025

# Improving Railway embankment stability through Bio Inspired Soil Improvement

**Yvo Veenis[1], Eliam Vlijm[2], Agnes van Uitert[3], Edward Kavazanjian Jr.[4], Leon van Paassen[5], Caroline van de Steenoven[6], Paul de Groot[2]**

[1]Manager Innovation, Groundwater Technology BV, Rotterdam, The Netherlands, E-mail: yvo.veenis@gtbv.nl
[2]Cohere Consultants BV, Amersfoort, The Netherlands
[3]Prorail, Utrecht, The Netherlands
[4]Regents' Professor, Department of Civil, Environmental, and Sustainable Engineering, Arizona State University, Tempe, AZ 85287
[5]Design, Tender and Engineering Department, Boskalis, Rosmolenweg 4, 3356 Papendrecht, Netherlands, Email: leon.van.paassen@boskalis.com
[6]Groundwater Technology BV, Rotterdam, The Netherlands

## ABSTRACT

Bio-inspired soil improvement (BISI) is being considered as a cost-effective method to improve railway embankment stability and to mitigate the potential for liquefaction of loose sand under railway embankments subject to effects of intensifying demand for rail traffic as well as impacts of climate change. (BISI) employs processes that are based on natural soil biology or implement similar processes with the aim of changing the geotechnical (physical) properties of soils. Microbially Induced Carbonate Precipitation (MICP) causes precipitation of calcium carbonates (mostly as calcite) by in-situ biogeochemical processes. BISI can be applied in a minimally invasive manner and the precipitates have potential to improve sediment stiffness, strength, and dilatant behaviour through inter-particle bonding and sediment particle surface roughening, commonly referred to as bio-cementation.
In this paper, we present the results of laboratory tests of the BISI processes on actual embankment material under realistic density and combine these results with modelling to determine the anticipated effects on risk mitigation. We conclude that bio-cementation offers the potential to effectively mitigate the potential for liquefaction of loose sand beneath and adjacent to track beds.

## INTRODUCTION

ProRail manages and maintains the Dutch railway network of 7,000 km of track and is working on a future-proof infrastructure. The demand for rail transport is forecasted to grow by 30% for passengers and 50% for freight in comparison to 2018. To accommodate this growth, ProRail improves rail capacity and investigates innovative solutions. In doing so, special attention is given to the stability of railway embankments.
As part of the Embankment programme, Prorail commissioned a series of research project to assess measures aimed at improving railway embankment stability and bearing capacity, with the aim to safely facilitate the anticipated increase in rail transport. One of these research projects assesses how Bio Inspired Soil Improvement (BISI) can improve embankment stability and bearing capacity

Climate change induced increase in extreme groundwater levels cause additional concern about rail track stability. At extremely high groundwater levels, vibrations resulting in pressure changes caused by passing trains can lead to static liquefaction, which can lead to flow liquefaction and thereby track failure. Excessive pore water pressure in saturated sand can result in liquefaction as it reduces the effective stress between the sand grains, leading to reduction of the sand load bearing capacity. Relevant soil properties that affect static liquefaction susceptibility include grain size distribution (and sorting); angle of internal friction, cohesion, permeability, compaction, small strain stiffness, specific weight of the soil materials, and grain shape and uniformity.

Liquefaction is predominantly a critical property of loosely packed sand, as these sands tend to contract during (cyclic) loading, generating excess pore water pressures. Strongly compacted sand tends to dilate during (cyclic) loading, generating negative pore pressures, which in fact increase the shear strength. Dutch Ministry of Infrastructure, Rijkswaterstaat, publishes 'rules-of-thumb' to quantify liquefaction susceptibility based on vertical load and relative density ($R_e$, mathematically expressed as equation 1, where $e_{max}$ is the maximum void ratio in the loosest condition, e is the void ratio in the natural state, and $e_{min}$ is the minimum void ratio in the densest condition) for sand with an effective vertical load between 20 – 200 kPa (20 kPa < $\sigma_{vo}$ < 200 kPa): $R_e < 0.33$: highly

$$R_e = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad \textbf{Equation 1}$$

susceptible; $0.33 < R_e < 0.67$: susceptible; $R_e > 0.67$: not susceptible Rijkswaterstaat, 2022 ).

The effect of liquefaction on track bed stability depends on multiple factors, including the decrease of strength, the size and geometry of the liquefiable layer and surrounding materials. For soils with a low density, more calcium carbonate ($CaCO_3$) cementation is required to achieve the same effect compared to more compacted soils (El Mountassir et al., 2018). This is likely caused by greater average interparticle distance, requiring more $CaCO_3$ cementation to 'bridge the gaps'. An inverse relation between amount of fines (< 64 µm) and cementation effectiveness has also been observed; likely caused by $CaCO_3$ precipitation on fines (not 'bridging the gaps') and reduced permeability, leading to less uniform cementation.

This paper describes testing and analyses demonstrating the potential for bio-cementation via MICP to mitigate the potential for liquefaction of loose sand under railway embankments in a setting typical for the Netherlands.

## BIO INSPIRED SOIL IMPROVEMENT

Bio Inspired Soil Improvement (BISI) is a series of techniques to improve the (geotechnical) properties of soils using methods applied *in situ*. BISI methods utilize natural processes that are inherent to natural soils and/or processes that are similar to such natural processes. The BISI technique of bio-cementation offers the promise of a cost-effective means for mitigation of static liquefaction potential in loose sand. In bio-cementation, mineral precipitation, generally of calcium carbonate in the form of calcite, is induced to enhance the strength of granular soil by cementing inter-particle contacts and roughening soil particle surfaces. BISI processes that induce bio-cementation include Microbially and Enzyme Induced Carbonate Precipitation (MICP / EICP). Figures 1 and 2 present the results of several laboratory studies of MICP and EICP in terms of unconfined compressive strength (USC) versus precipitated carbonate content.

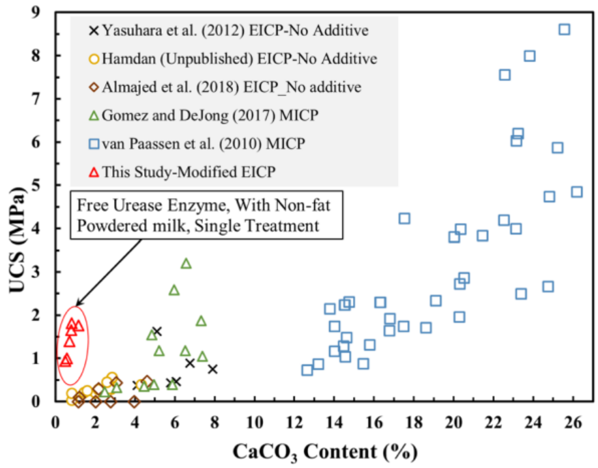


**Figure 1. UCS Strength versus Calcite content**

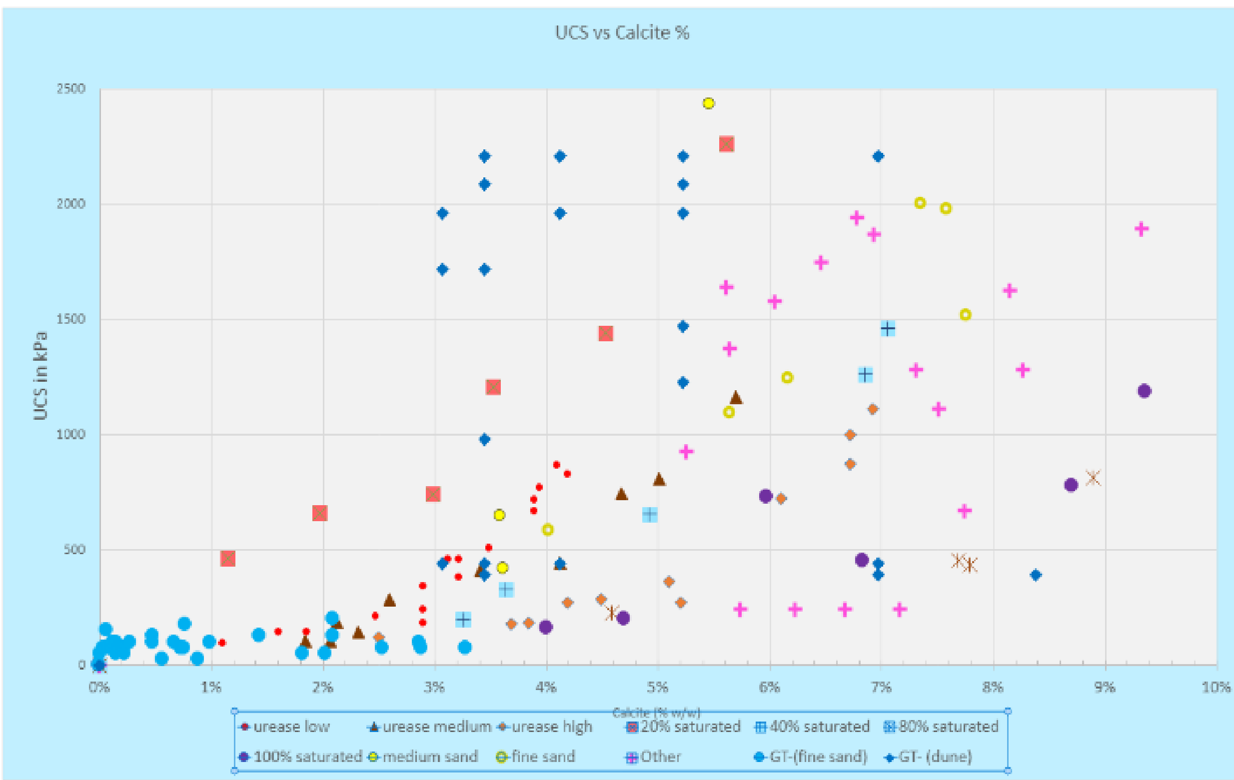


**Figure 2. UCS Strength versus Calcite content (combining published data & GT unpublished data)**

## Background

In consultation with ProRail, a test location was selected where the track is on a 9.5 m high embankment on an 8.4 m thick sand bed in 10.5 m of clay & peat. Natural groundwater is at elevation -0.5 m- -1.9 m below surrounding ground level, fluctuating with season and weather. The embankment consists of silty sand, with up to 20% of fines. Compaction varies, with very well compacted sand up to 2 m directly below the tracks. Below 3.0 m, compaction is minimal. Figure 3 shows the tip resistance from a CPT sounding at the site alongside a relative density profile interpreted from the CPT results using Baldi's equation (Baldi et al, 1986).

## Laboratory tests on soil samples

Soil samples were taken from the slope of the embankment in 12 litre buckets. Coarse materials (lumps of clay, stones, roots) were removed as such materials could severely impact the DS tests (which is done on 60 * 60 * 20 mm blocks); the retained material was homogenized and analysed (89.1% dry weight, 2.6% TOC, 0.5% calcite, pH 5.7 and grain size). Table 1 summarizes the properties of the sandy embankment soil as per sample grainsize analyses. Minimum and maximum density was determined according to (Dutch) standards on oven-dried material and found to range from 1260 kg/m3 to 1611 kg/m3.

**Table 1. Properties of the soil at the test site**

| Parameter | Value |
|---|---|
| Clay (< 2 µm) | 8.4% |
| Silt (> 2 µm & < 64 µm) | 8% |
| Sand (> 64 µm & < 2 mm) | 83.6% |
| Gravel (> 2 mm) | 0% |
| $D_{10}$ | 0.004 mm |
| D50 | 0.32 mm |
| D90 | 0.814 mm |
| uniformity coefficient | 108 [-] |
| coefficient of gradation | 25 [-] |

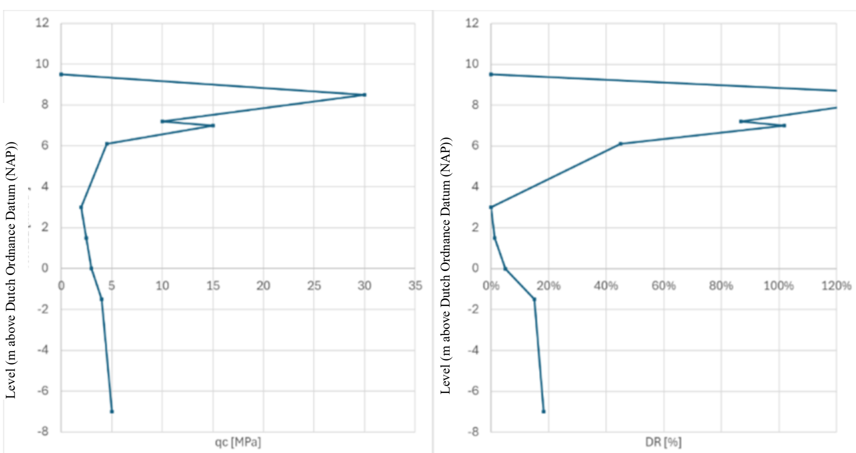


**Figure 3. CPT sounding and interpreted relative density based upon Baldi (C0= 157; C1= 2.41; C2=0,55) at the test site**

DeJong and Gomez state that at D10 < 50 µm (as in this case) filtration may limit the transport of microbes (DeJong and Gomez, 2022) It follows that MICP has successfully been implemented at soil samples with 120 µm <D50 < 1000 µm. We conclude that MICP should well be feasible in the soil sampled, despite the relatively high content of fines. Based on data in the literature (e.g., National Academies of Sciences, Engineering, and Medicine, 2016), we expect that considering the low relative density of the deeper part of the embankment and underlying sand layers this material is susceptible to liquefaction.

## Geotechnical testing

We find that in most published results of geotechnical tests, samples tested were MICP treated in the apparatus in which subsequently the geotechnical tests were implemented. This was not feasible in this case. Attempts to transfer MICP treated soil samples into a triaxial test apparatus failed; the MICP bonds are brittle and were broken by the sample preparation and handling. Part of the issue is the standard support pressure in the tests, which is greater than the formation pressure on the samples at the typical treatment depths. We therefore elected to use Direct Shear (DS) tests to evaluate the strength of the bio-cemented soil. This requires smaller sized samples which can be produced in the correct sizes during our regular BISI tests. These samples can be transferred to a standard DS testing apparatus in a commercial geotechnical lab and yield results.
As CPT data indicate loosely packed material at the test site, we elected to conduct all MICP tests at a relative density (Dr) equal to 40% (=1380 kg/m3). Table 2 summarizes the geotechnical density data for the sand at the test site.

**Table 2. Geotechnical density data**

| $G_s$ | $\gamma_{dmin}$ (sample density) [g cm$^{-3}$] | $\gamma_{dmax}$ sample density) [g cm$^{-3}$] | $n_{max}$ (porosity) | $n_{min}$ (porosity) | $e_{max}$ (void ratio) | $e_{min}$ (void ratio) |
|---|---|---|---|---|---|---|
| 2.6 | 1.26 | 1.61 | 0.51 | 0.38 | 1.06 | 0.61 |

| DR [%] | e | n | $\gamma d$ [g cm$^{-3}$] |
|---|---|---|---|
| 100 | 0.61 | 0.38 | 1.61 |
| 40 | 0.88 | 0.47 | 1.38 |

The following geotechnical tests were implemented on untreated specimen at a density of 13.35 kNm$^{-3}$ to provide a baseline for evaluating the degree of soil improvement:

- DS (direct shear) at 50 kPa, 100 kPa & 300 kPa confining pressure
- Isotopically consolidated drained (CiD) Triaxial tests, including bender testing for shear wave velocity, consolidated 50 kPa, 100 kPa, and 300 kPa confining pressure
- Isotopically consolidated undrained (CiU) Triaxial tests, including bender testing for shear wave velocity.

Combining the CiU and CiD data led to a peak effective stress strength described by:

$$\phi'_{peak} = 32{,}3° \; ; \; c' = 1.0 \text{ kPa}$$

Table 3 presents the results of the bender element tests on four (duplicate) isotopically consolidated untreated specimens.

**Table 3. Results Bender tests**

| | σ3' [kPa] | Vs [m/s] |
|---|---|---|
| M001A-x5 | 50 | 157 |
| M001A-x6 | 100 | 197 |
| M001A-x7 | 50 | 162 |
| M001A-x8 | 100 | 200 |

## BISI laboratory testing

We developed a column set-up in which we treat soil samples of 100 mm length and 200 mm diameter. Within these samples, we place steel rims of 60 mm x 60 mm x 20 mm from which the treated specimen can be transferred into the DS testing apparatus. For tests K1 through K4 columns were very carefully filled with 10 cm dry (not oven dried) sand (in multiple lifts) to achieve the desired density of 1380 kg m$^{-3}$(oven-dried). Test K5 was done on a column filled by wet pluviation, which resulted in a density of 1758 kg m$^{-3}$ (saturated). For treatment, columns were subjected to vertical upwards flow by peristaltic pumps. Amendments were added aiming for 4% (by weight) calcite formation. After treating the specimens for tests K1 and K2, the treated sand could easily be removed from the columns and light cementation was noted. The containers were carefully removed, packed, and shipped to the geotechnical lab. Test K3 was implemented with additional stimulation of bioactivity through injection of cultured bacteria. Again, light cementation was noted.

The samples were analysed for calcite content using an Eijkelkamp Calcimeter according to Schneibler's principle; the analytical procedure follows NEN-ISO 10693. The calcite content values are presented in Table 4.

**Table 4. Calcite content of treated DS specimens**

| Column test | Calcite content after test (w/w%) | Density before treatment kg m$^{-3}$, @field moist / @ dried |
|---|---|---|
| K1 | 4.55 % | 1549 / 1380 |
| K2 | 5.40 % | 1549 / 1380 |
| K3 | 5.03 % | 1549 / 1380 |
| K4 | 3.50 % | 1549 / 1380 |
| K5 | 2.76 % | 1973 / 1758 |

Direct Shear tests were implemented on water saturated BISI treated specimens at 50 kPa, 100 kPa and 300 kPa confining pressure. The results are shown in Table 5.

**Table 5. Results DS test**

| Test | Description | γ dry | Normal stress p' | Peak τ | Peak strength τ | Residual τ | Residual strength |
|---|---|---|---|---|---|---|---|
| | | [kN m$^{-3}$ | [kPa] | [kPa] | [kPa]2 | [kPa] | |
| B002-M001A-1 | Untreated | 13.4 | 50 | 31 | ϕ=31.9° C'=3.6 kPa | 29 | ϕ=30.3° C'= 4.0kPa |
| B002-M001A-2 | Untreated | 13.5 | 100 | 70 | | 68 | |
| B002-M001A-3 | Untreated | 13.2 | 300 | 190 | | 178 | |
| | | | | | | | |
| K5-M001 | MICP treated | 16.7 | 50 | 52 | ϕ=38.2 C'= 18kPa | 39 | ϕ=37.4 C'= 0kPa |
| K5-M002 | MICP treated | 17.1 | 100 | 104 | | 75 | |
| K5-M004 | MICP treated | 16.4 | 300 | 252 | | 230 | |
| | | | | | | | |
| K2-1 | MICP treated | 13.4 | 50 | 31 | ϕ=23.1 C'=13.1kPa | 24 | ϕ=23.8 C'=7.7kPa |
| K2-3 | MICP treated | 13.5 | 100 | 60 | | 59 | |
| K2-4 | MICP treated | 13.2 | 2814 | 140 | | 139 | |

Figure 4 presents shear stress versus horizontal displacement for the three confining pressure levels for untreated and treated specimens. The untreated samples show typical loose sand behaviour: no

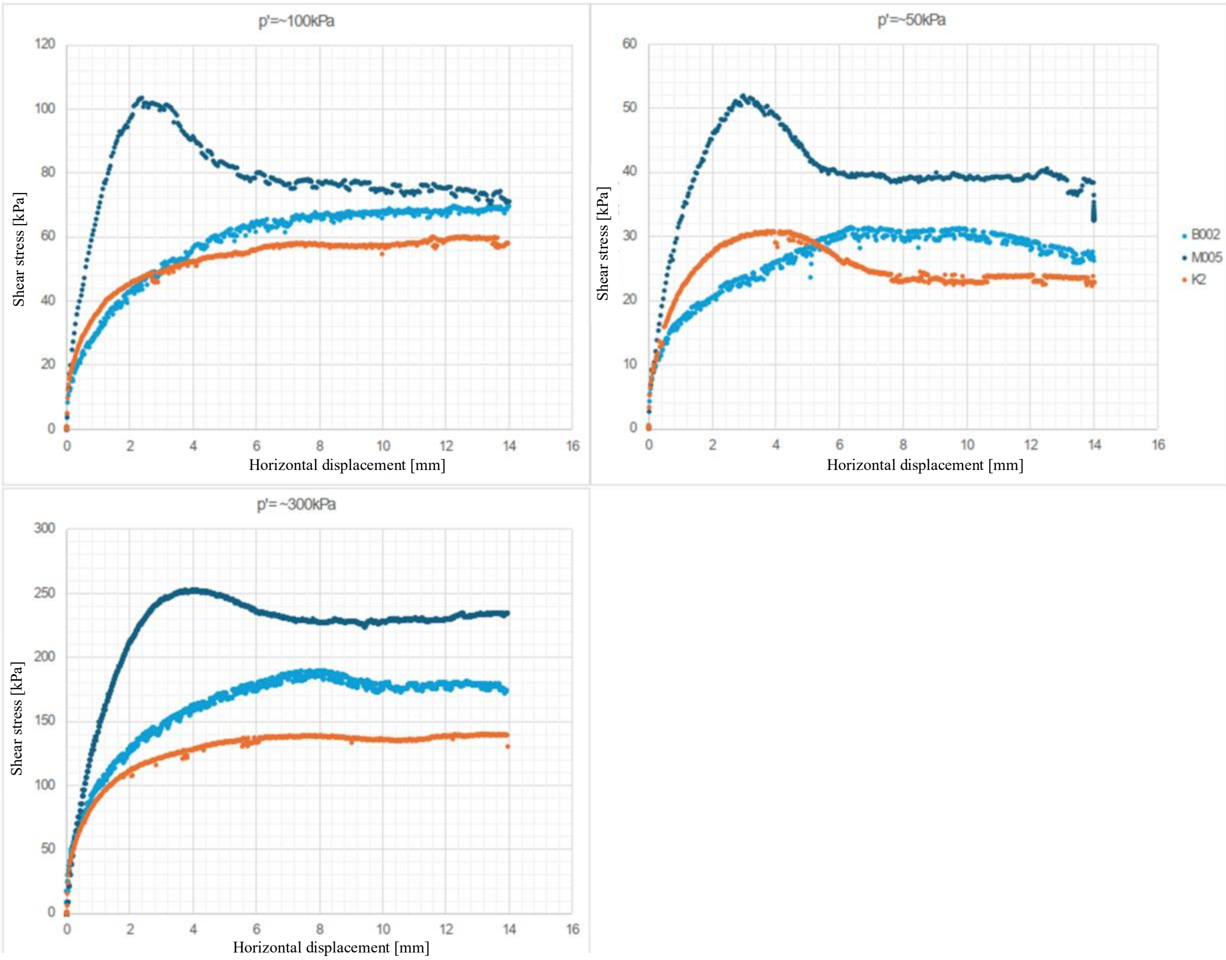


**Figure 4. Shear stress [kPa] (y-axis) versus horizontal displacement [mm] (x-axis) obtained from Direct Shear test. Light blue = untreated specimen (B002); dark blue= MICP treated (K5); orange=MICP treated (K2)**

clear peak strength and relatively weak behaviour. In tests K5 & K2, in particular at the 50 kPa confining stress, the specimen show much stiffer behaviour; there is a clear peak strength and softening beyond the peak strength. At higher confining pressure, K2 resembles untreated sand; K5 displays stiffer & stronger behaviour at all confining pressure levels.

Figure 5 shows that sample K5 displays dilatant behaviour, which yields the high peak strength. K2 displays contractant behaviour, implying that the shear strength peak is not affected by compaction, but rather by cohesion or internal friction.

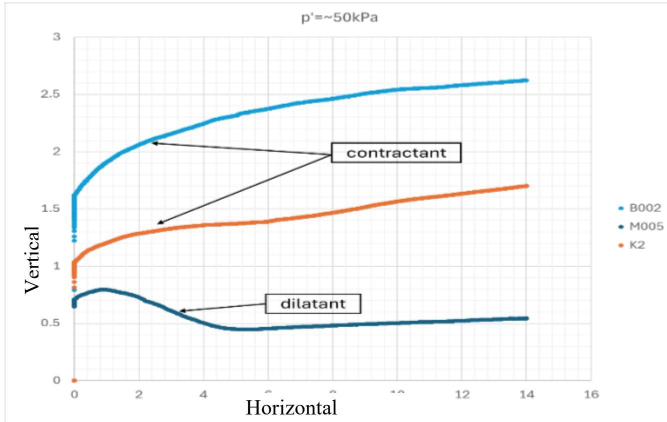


**Figure 5. Stress strain and volume change behaviour of DS specimens**

From results of the DS tests we conclude that:

- Untreated samples behave as expected for loosely packed sand; little or no dilatancy and the peak strength and residual strength are nearly equal;
- Sample K5 was more compact than other specimens and showed a significant increase in peak and residual strength, typical for compacted sand; masking BISI effects;
- Sample K2, at 50 kPa confining stress is stiffer and more brittle than the untreated sample. Peak strength is more or less comparable to the peak strength found for the untreated specimen. The peak in strength is followed by a decrease in strength that is not caused by compaction but by increasing cohesion or internal friction between the grains
- At higher confining stresses (100 & 300 kPa), behaviour of K2 closely resembles behaviour of untreated material, It is expected that in such loosely packed sand calcite cementing bonds were formed which, at the level of calcite formation, are brittle and will be easily crushed or broken during the application of the confining stresses of 100 & 300 kPa, thus losing the increased strength and stiffness. This may not be the case if the specimens were treated under the applied confining pressure. However, it has been demonstrated in literature that the effect of cementation at higher confinement stresses is limited (Van Paassen et al (2009).

These lab tests give insight into the effects of BISI treatment on sand behaviour. We conclude:

- The sand in the embankment is very loosely packed and contains a significant amount of fines;
- Calcite production was successfully implemented through MICP;
- BISI effects on the treated soil are found to increase the small strain stiffness and brittle behaviour at low confining stress.
- The effect of biocementation is limited at higher confining pressure (>100 kPa), which results in a limited increase in strength (c') or ($\phi$'), when fitting a Mohr-Coulomb failure criterion that includes the strengths at higher confining stresses.

## MODELLING APPROACH

To assess the potential to stabilize the embankment and mitigate liquefaction of the sand layers below active railways using BISI without implementing a pilot project, a numerical modelling approach was used at an exemplar track location. The undrained behaviour of the soil layers below the tracks with elevated groundwater was modelled in Plaxis 2D™ to assess static liquefaction susceptibility. To establish the initial conditions, drained behaviour was modelled in Plaxis based on the Hardening Soil Model with the Mohr Coulomb failure criteria.

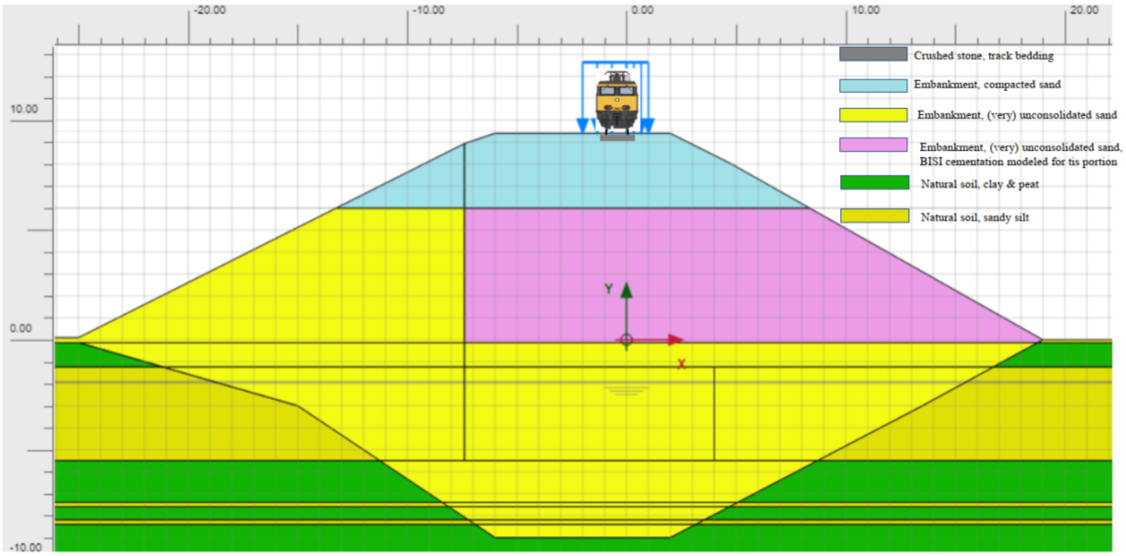


**Figure 6. Exemplar Track Cross Section**

The exemplar track section we used to model static liquefaction susceptibility is shown in Figure 6. We placed the groundwater level in the model at the top of the sand body to model the worst case for static liquefaction. The relevant parameters for the liquefiable loose sand are presented in Table 6. Geometry and loading were tested for stability in both undrained and drained conditions. Under drained conditions, increased loading leads to higher grain pressure in the sand, increasing stability. Under undrained conditions, the load is carried by the groundwater, thereby leading to a reduction in strength.
The 'Soil Test' module in Plaxis 2D was used to fit the model parameters to the data from the geotechnical laboratory tests. The NORSAND model is not suited to model BISI effects as it calculates soil behaviour primarily from the soil density. Therefore, we selected the HS model using modified Su (undrained shear strength) values, assuming a linear relation between increase in Su and increase in cohesion as cementation primarily causes an increase in cohesion. We took

**Table 6. Properties of the exemplar section soil**

| | $kNm^{-3}$ | | c' (kPa) | Plaxis model |
|---|---|---|---|---|
| Compacted sand | 18/20 | 38.5 | 1 | HS |
| Loosely compacted sand | 17/18 | 32.5 | NA | NORSAND |
| Loosely compacted sand | 17/18 | 32.5 | 1.0 | HS |

the over-estimation of the undrained strength in the HS model into account. Figure 7 compares the model behaviour to the results of CIU triaxial compression tests (untreated specimens) and Figure 8 compares model prediction to DS test results (also untreated specimens).

## MODELLING RESULTS

As shown in Figure 9a, the Norsand model predicts static liquefaction and an unstable slope (FoS=0.56). Modelling BISI improvements up to 3.4 m below grade yields a stable slope at 5 kPa increase in undrained strength (corresponding to 5 kPa cohesion) as shown in Figure 9b.

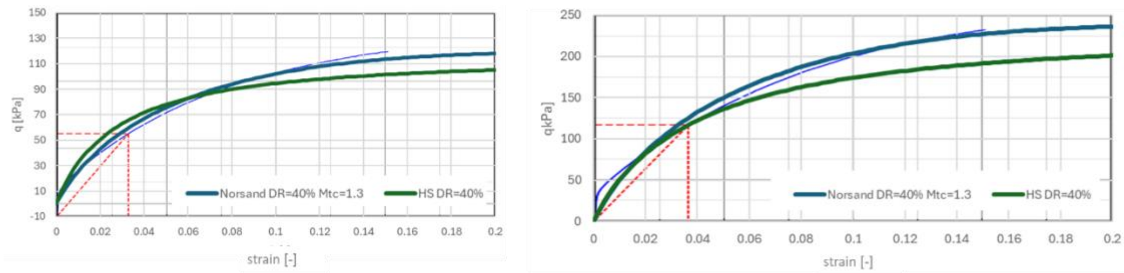


**Figure 7. HS & NORSAND model fitted to triaxial compression test results**

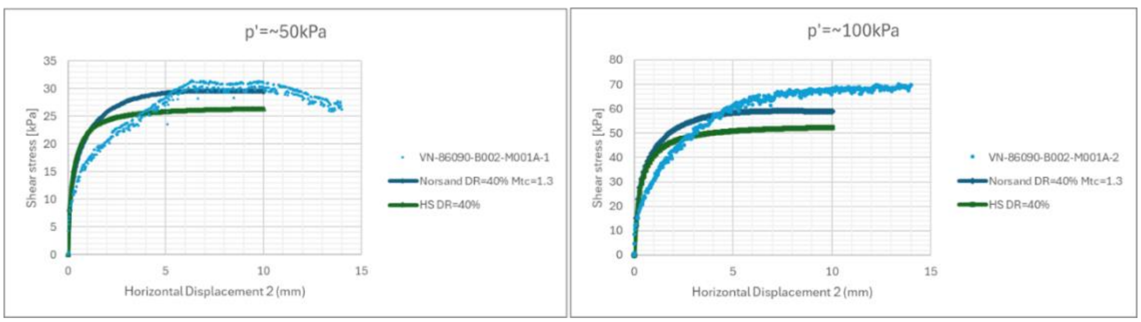


**Figure 8. HS & NORSAND model fitted to direct shear test results**

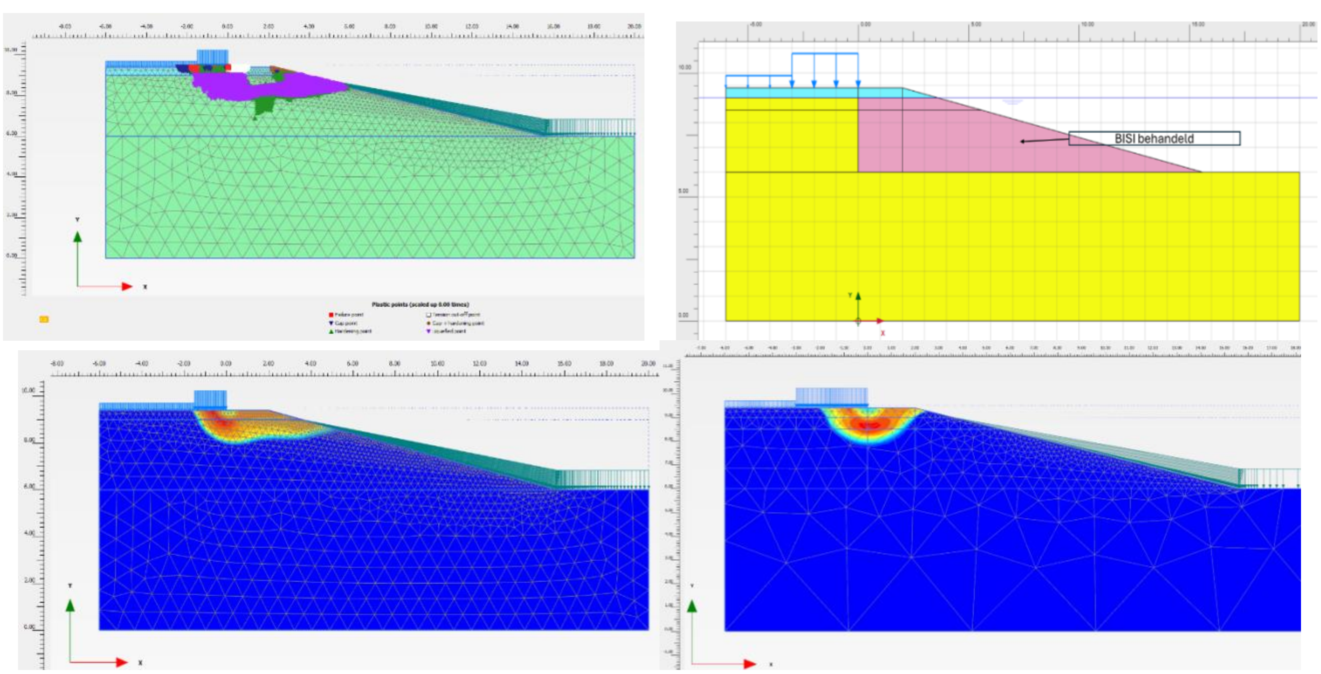


**Figure 9. Results of the Plaxis Analysis: a) without BISI (left); b) with 20 kPa improvement through BISI**

## CONCLUSIONS ON LIQUEFACTION MITIGATION USING BISI:

- The exemplar case presented here predicts liquefaction with a very low safety factor (FoS = 0.56) if no ground improvement is assumed. Increasing cohesion in the upper 3 m of loosely packed sand with 5 kPa yields a stable slope.
- Increasing cohesion by 5 kPa should be quite feasible using MICP (Van Paassen et al, 2010; Dejong and Gomez, 2022); however, this was not demonstrated by DS tests in this project.
- The NORSAND model is suited to model liquefaction; the HS model is not suited for this without adaptations.
- Using the HS model, results achieved when calculating with Sµ fitted to the NORSAND model are similar to results based on undrained strength tests;
- BISI effects are difficult to model using NORSAND; we therefore used the adapted HS model;

## REFERENCES


Almajed, A., Tirkolaei, H.K., Kavazanjian, E., Jr., and Hamdan, N. (2019). Enzyme Induced Biocementated Sand with High Strength at Low Carbonate Content. Scientific Reports. https://doi.org/10.1038/s41598-018-38361-1

Azarafza, M., and Nyambayo, V. (2023). Biogeotechnics: A Review of Challenges, Opportunities and Perspectives in Geotechnical Applications. 9th International Congress on Environmental Geotechnics. Chania, Greece.

Baldi, G., Bellotti, R., Ghionna, V., Jamiolkowski, M., & Pasqualini, E. (1986). Interpretation of CPT's and CPTU's: drained penetration of sands. Proceeding. In 4 th International Geotechnical Seminar Field Instrumentation and In-Situ Measurement, Nanyang Technological Institute, Singapore (pp. 143-156).

Bentley. (2023). User Defined Soil Models - NorSand: An elasto-plastic model for soil behaviour with static liquefaction.

DeJong, J., and Gomez, M. (2022). State of the Art: MICP soil improvement and its application to liquefaction hazard mitigation. Sydney, Australia: Australian Geomechanics Society.

El Mountassir, G. E., Minto, J. M., van Paassen, L. A., Salifu, E., and Lunn, R. J. (2018). Applications of Microbial Processes in Geotechnical Engineering. Advances in Applied Microbiology, Volume 104.

Fugro NL Land B.V.. (2023). Baanhoek Sliedrecht. Rapportage geotechnisch onderzoek | Sliedrecht. 2423-232212-21-R01-v1.0-20230530.

Groundwater Technology B.V. (2024). 240301 ProRail TN374883. Werkplan laboratorium testfase werkpakket 5 Baanstabiliteit. Proeftuin fysieke maatregelen stabiliteit en draagvermogen baanlichaam.

National Academies of Sciences, Engineering, and Medicine. (2016). State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences.

Paassen, L. v. (2022). Determination of geotechnical parameters for bio-cemented sands for numerical simulations. Arizona State University.

Rijkswaterstaat. (2012). Handreiking Toetsen Voorland Zettingsvloeiing t.b.v. het opstellen van het beheerdersoordeel (BO) in de verlengde derde toetsronde.