SENSITIVITIES OF BRACED DEEP EXCAVATIONS TO JET GROUT PILE APPLICATIONS

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ABSTRACT

In poor ground, jet grout pile (JGP) slabs may be used as preinstalled struts in deep excavations, to bring significant benefits to the earth-retention system. Based on the author's experience, top-down construction projects may require JGP slab usage to enable serviceability design criteria to be met at all construction stages, and to reduce base heave, ground settlement and soil loads on retaining walls during construction. This paper discusses parametric finite element sensitivity studies performed to predict qualitatively the performance of JGP slabs in reducing retaining wall loads, deflections, and associated ground movements.

1 INTRODUCTION

Jet grouting is a partial replacement/mixing technology that uses a tool equipped with one or more high pressure jets to erode and hydraulically excavate soils, while mixing cement grout with the *in situ* soils, creating soil-cement columns or soil-cement panels (Bruce, 2005).

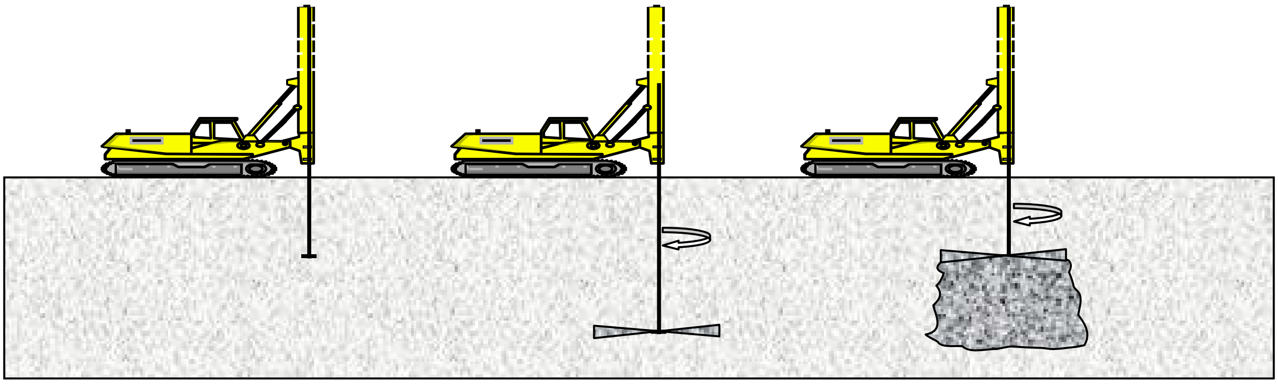


Figure 1: JGP construction stages (Hewitt *et al.,* 2006).

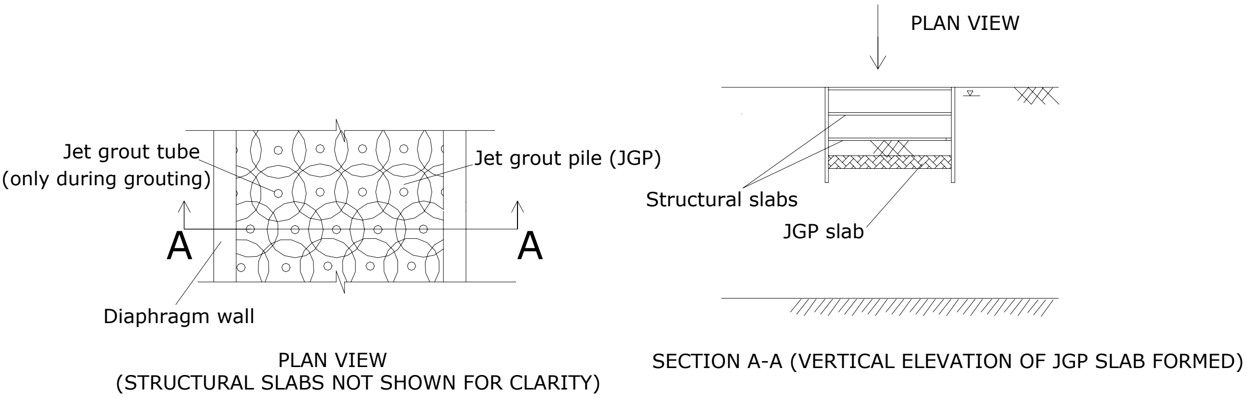


Figure 2: JGP grid, completed piles viewed in plan and elevation (slab).

When this grouting is done in a grid arrangement within structural walls (Figure 2), the end result is a slab composed of these piles. The principle, in this paper's context, is to introduce these slabs as lateral restraints, or struts, to walls at depth, prior to excavation commencement. Generally, this reduces the effective unbraced lengths of walls as excavation progresses and also counters base heave.

In addition, braced deep excavation walls often rely on some fixity in ground below formation level, for stability and deflection control. Where poor soils constitute deep layers (of the order of 60 m) below the proposed formation level, the wall depths required to attain this fixity can be economically prohibitive. JGP slabs can effectively fix such walls by acting as preinstalled struts, before excavation commences, thereby stabilising the deep excavation system.

2 THE JGP METHOD

2.1 CONSTRUCTION ASPECTS

From constructors' or specialist sub-contractors' perspectives, the following are the key issues in JGP slab applications:

- JGP diameter selection – concerns bore diameter and type of grout jets used;
- JGP single, double or triple tube drill bit usage - elaboration is beyond this paper’s scope;
- JGP consistency – concerns the mix proportions of pre-existing soil and injected grout in the resulting piles;
- Jetting pressures - up to around 400 bars (40, 000 kPa) in the author's experience;
- JGP diameter –can range up to about 3 m (Bowles, 1996);
- JGP location – the targeted depth band;
- Jetting damage to temporary works - many deep excavation systems in densely built-up areas utilise temporary structural steelwork. The effects of high-pressure grouting in their immediate vicinity must be considered with regard to their continued functionality, eventual removal for reuse and construction sequencing;
- Jetting damage to buried services - pressure relief trenches excavated between the services and the JGP area are often used to provide paths of reduced soil resistance. Significant proportions of the jetting force at upper soil depths are dissipated via ground heave in these trenches. Alternatively, the pressure relief trenches can be around the services themselves, which are temporarily supported to keep them clear of the base heave and
- Jetting damage or adverse effects on tunnels in the vicinity – requires in-tunnel monitoring and a performance-based approach to grouting.

2.2 DESIGN ASPECTS

For design, the following are the key parameters of the JGP slab:

- Its stress-strain modulus;
- Its cross-sectional area;
- Its cohesion - derived from the unconfined compressive strength of grout samples tested. This value is of interest when modelling the JGP strut as an improved soil layer, as opposed to an elastic strut (foreign to the soil);
- Its depth below final formation level;
- The flexural rigidity and axial stiffness of the braced excavation system comprising walls and slabs and
- Surrounding soil parameters.

3 TOP-DOWN CONSTRUCTION IN THIS PAPER'S CONTEXT

Top-down construction may be used for underground metro stations, rail tunnels, building basements and underpasses, among others. These structures may be multi-levelled below the ground, involving a roof slab, several floor slabs, and a base slab. Geotechnically, the walls and slabs perform the following functions:

- Temporary earth-retention during construction;
- Permanent earth-retention during the structure’s life-cycle and
- Resisting downward vertical loads and uplift due to floatation and heave.

With conventional top-down construction, the walls are typically installed first, followed by excavation, with slab casting at the requisite levels. Once each slab attains the requisite axial and bending strength, excavation proceeds below it, and the procedure repeats until base slab completion. The slabs generally act as struts throughout the structure’s life.

Due to the walls deflecting within the temporary construction stage, prior to its permanent in-service stage, there can be difficulties keeping within the respective serviceability limit state criteria (crack widths) - a significant design criterion for tanked structures in poor soils.

Furthermore, deep excavations in cities are often constrained by physical and statutory limitations, precluding the use of ground anchors and soil nails. Examples include:

- Proximity of existing structures like basements, foundations or tunnels;
- Proximity of statutory 'reserves' surrounding them, within which works are either forbidden or permitted under very rigorous constraints and
- Land acts, building or environmental statutes forbidding the installation of temporary works under or at the perimeter of adjacent properties, when these cannot be reliably, safely and completely removed later. Where safe removal techniques exist, their cost and program implications may be prohibitive.

For these reasons, only braced top-down excavations are considered in the analysis here.

4 FINITE ELEMENT MODELLING OF JGP APPLICATIONS

The finite element program PLAXIS Version 8 was used in this analysis, with the following modelling considerations, and as shown in Figure 3:

- The structure is two-tiered with cell heights of 5.4 m, to examine the effects of JGP struts on generic multi-level structures like underground expressways, metro stations and basement car-parks;
- The construction method (Figure 4) is top-down with the priority to minimise existing road disturbance – the first excavation (Stage 3) is just to enable roof slab casting, so that traffic flows can be resumed quickly, while construction proceeds below the roof;
- Diaphragm walls (generally regarded as the stiffest walls available) with rectangular panels are assumed;
- From the structural serviceability perspective, the wall deflection profile is considered. This, together with the predicted base heave, are used by construction teams for monitoring purposes;
- From the environmental impact perspective, ground settlements immediately behind the wall and around 25 m away are captured to provide a basis of comparison for the effects of various JGP applications;
- A 50 m thick layer of firm clay is considered here as a generic representative of deep soil layers like alluviums estuarine and marine clays. The following parameters were assumed:
  - Unit weight - 16 kN/m^3
  - Permeability - 8.64E-05 m/day
  - Poisson's ratio - 0.34
  - Stress-strain modulus - 20 MPa
  - Cohesion - 50 kPa and
  - Internal angle of friction - 0°.
- The groundwater table was at 2 m depth;
- Construction surcharge of 20 kPa was applied everywhere behind the walls. It was applied at the same first stage as diaphragm wall installation, and deformations were reset to zero from the second stage onwards, to clarify relevant construction-related settlements;
- Modelling and parametric application were carried out in accordance with Brinkgreve *et al.* (2004);
- Key output criteria taken are wall bending moment profiles, slab-strut axial loads, wall deflection profiles, base heave and ground settlement, to evaluate the effects of the JGP strut arrangements. The effects on the ground water table are also qualitatively observed and
- Bending moment envelopes also provide indications of areas where serviceability criteria, like 0.2 mm crack widths for tanked structures, may govern structural design considerations.

The objectives of the modelling were to examine the effects of JGP slab usage, compared to conventional top-down construction, then to consider the optimal location, thickness, unconfined compressive strength and stiffness of JGP slabs, to minimise deflections, settlements and heave.

4.1 CONTROL MODEL IN FIRM CLAY (MODEL 1) - NO JGP

This basic model, without JGP slab application, was set up using the construction and dewatering sequences as shown in Figure 4.

Ground settlement immediately behind the wall, and 24.6 m away were 66 mm and 125 mm respectively. Maximum base heave was 177 mm, and the groundwater table dropped 1.5 m behind the wall.

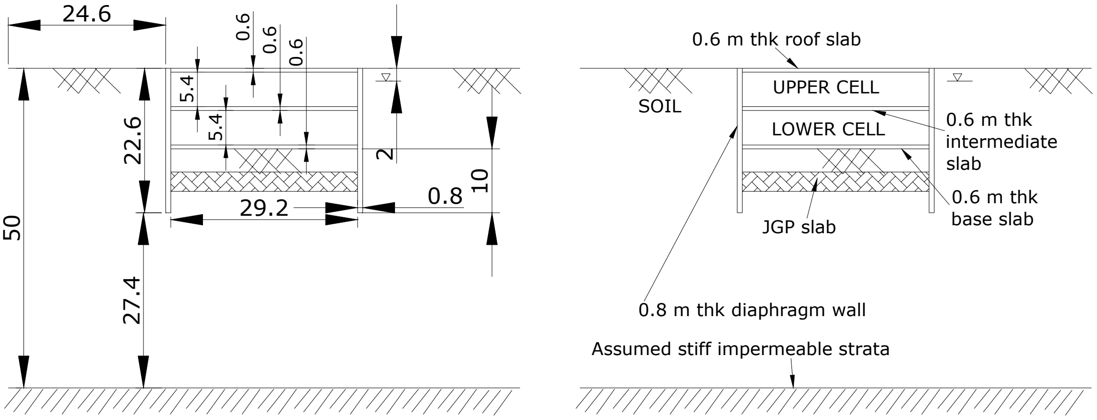
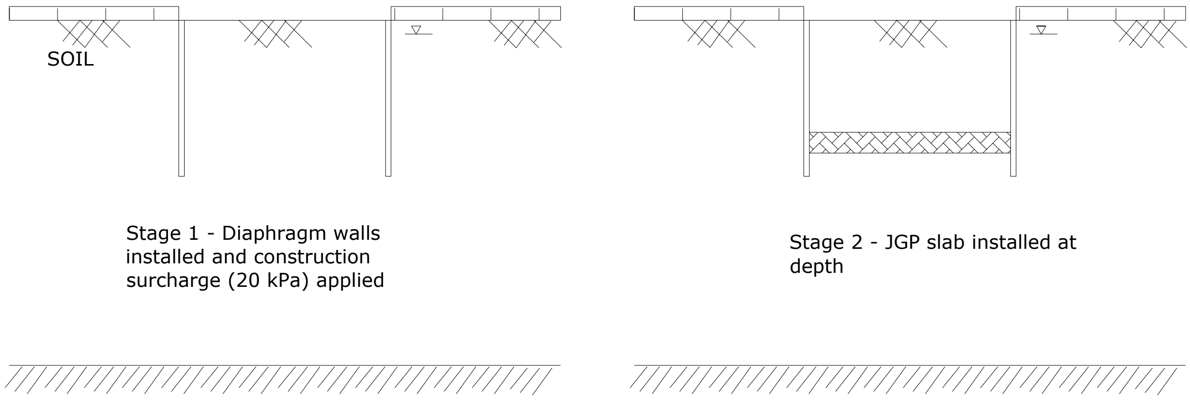
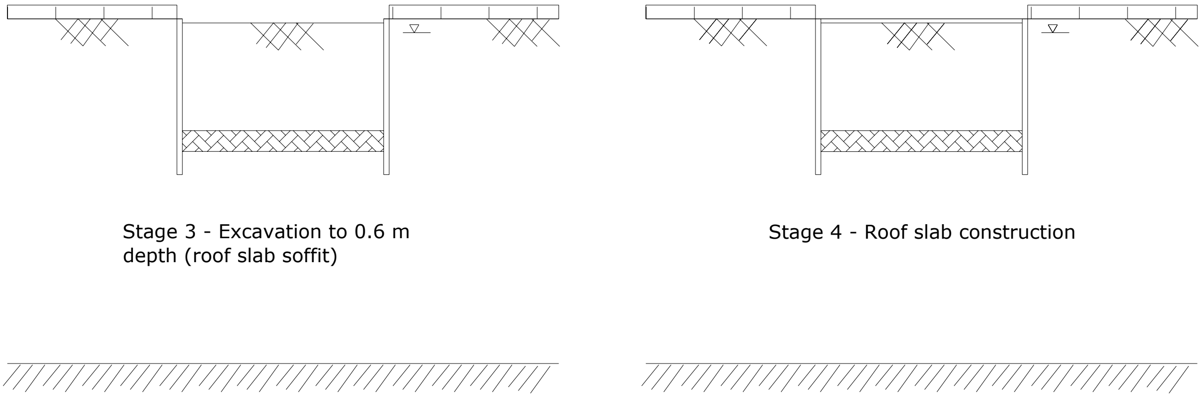


Figure 3: Geometrical and structural details of finite element top-down model.



Stage 1 - Diaphragm walls installed and construction surcharge (20 kPa) applied

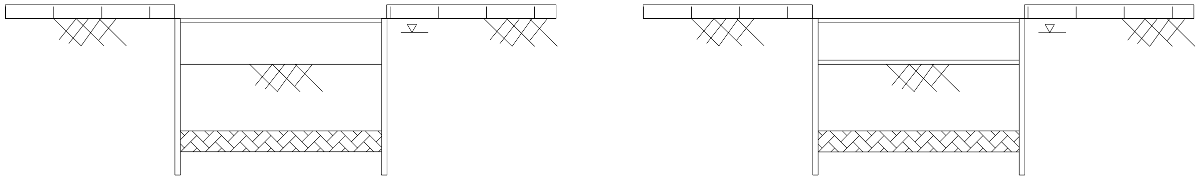
Stage 2 - JGP slab installed at depth



Stage 3 - Excavation to 0.6 m depth (roof slab soffit)

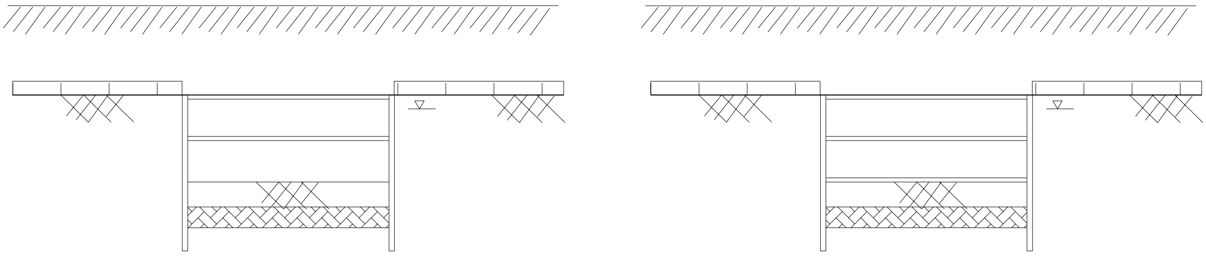
Stage 4 - Roof slab construction

Figure 4: Finite element top-down model construction sequence Stages 1 to 4.



Stage 5 - Excavation and dewatering (groundwater flow calculation) to 6.6 m depth (intermediate slab soffit)

Stage 6 - Intermediate slab construction



Stage 7 - Excavation and dewatering (groundwater flow calculation) to 12.6 m depth (base slab soffit)

Stage 8 - Base slab construction

Figure 4: Finite element top-down model construction sequence Stages 5 to 8 (continued from previous page).

Note the following pertinent wall depths, for subsequent graph interpretation, and clarified in Figure 5:

- 3.3 m – mid-height of wall in upper cell above intermediate slab;
- 6.3 m – mid-height of intermediate slab;
- 9.3 m – mid-height of wall in lower cell above base slab;
- 12.3 m – mid-height of base slab;
- 15.1 m – wall depth at 2.5 m below base slab soffit;
- 18.1 m – wall depth at 5.5 m below base slab soffit;
- 22.6 m – wall toe depth.

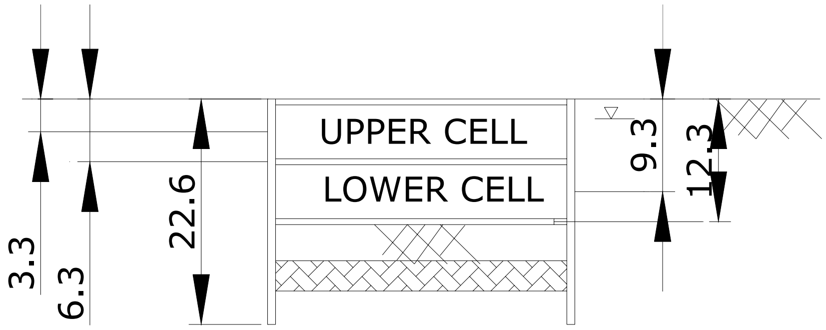


Figure 5: Pertinent wall depths as referenced by subsequent graphs.

Figure 6 gives the bending moment envelope (positive and negative) through all construction stages, for the left wall.

The following are salient points:

- Peak moments are at the slab support points, with relatively high bending at mid-height of the upper cell;
- The maximum moments at upper cell mid-height are significantly higher than at mid-height of the lower cell;
- Below the mid-height of the lower cell, there is no moment reversal and

- Moments reduce almost linearly with depth between base slab and toe – reasonable fixed cantilever behaviour.

As expected, the intermediate slab takes the maximum axial load of 1271 kN/m in compression. The roof slab takes 115 kN/m in tension, while the base slab effectively takes nothing (1.1 kN/m in compression), because of its late installation stage.

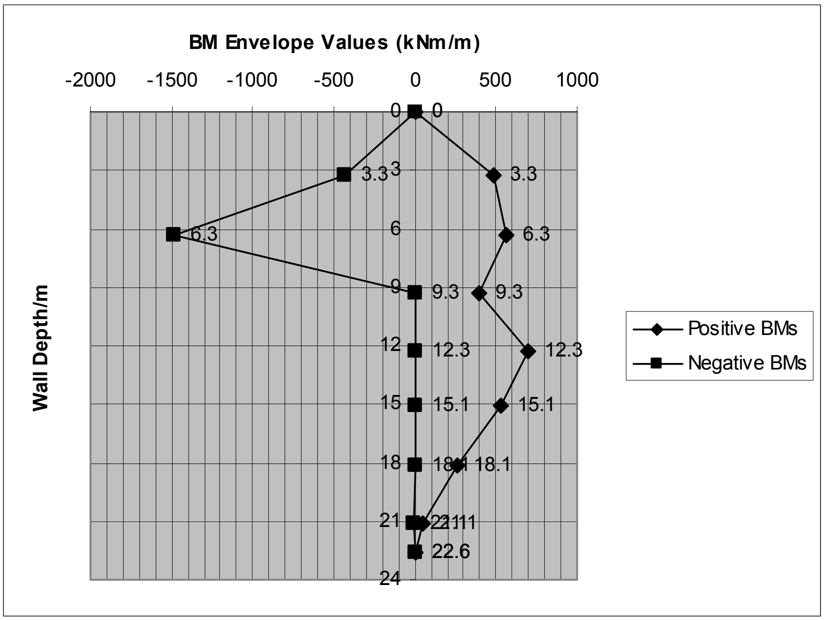


Figure 6: Bending moment (BM) envelope for left wall with wall depth (Model 1 - no JGP).

4.2 MODELS PERFORMED

The following is a summary of the models that were set-up and run:

Table 1: Summary of PLAXIS models run.

MODEL	SOIL	JGP SLAB LOCATION DEPTH (m)	JGP SLAB STRENGTH	JGP SLAB THICKNESS/M
1	Firm Clay	NONE	-	-
2	Firm Clay	12.6 -14.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	2
3	Firm Clay	14.6 - 16.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	2
4	Firm Clay	20.6 – 22.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	2
5	Firm Clay	12.6 -14.6	c_{ref} = 300 kPa; E_{ref} = 150 MPa	2
6	Firm Clay	14.6 - 16.6	c_{ref} = 300 kPa; E_{ref} = 150 MPa	2
7	Firm Clay	20.6 – 22.6	c_{ref} = 300 kPa; E_{ref} = 150 MPa	2
8	Firm Clay	12.6 -14.6	c_{ref} = 600 kPa; E_{ref} = 300 MPa	2
9	Firm Clay	14.6 - 16.6	c_{ref} = 600 kPa; E_{ref} = 300 MPa	2
10	Firm Clay	20.6 – 22.6	c_{ref} = 600 kPa; E_{ref} = 300 MPa	2
MODEL	**SOIL**	**JGP SLAB LOCATION/ DEPTH IN M**	**JGP SLAB STRENGTH**	**JGP SLAB THICKNESS/M**
11	Firm Clay	12.6 – 16.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	4
12	Firm Clay	12.6 – 18.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	6
13	Firm Clay	14.6 – 20.6	c_{ref} = 100 kPa; E_{ref} = 50 MPa	6
14	Firm Clay	12.6 – 18.6	c_{ref} = 300 kPa; E_{ref} = 150 MPa	6
15	Loose to Medium-Dense Sand	NONE	-	-

Note : c_{ref} and E_{ref} denote cohesion and Young's Modulus respectively (Brinkgreve *et al.*, 2004).

For brevity, JGP slabs with c_{ref} = 100, 300 and 600 kPa will be referred to as low-strength (LS), medium-strength (MS) and high-strength (HS) JGP slabs respectively henceforth.

4.3 MODEL OUTPUT

4.3.1 The effect of the presence of a 2 m thick LS JGP slab (comparison between Models 1 and 2)

The LS JGP slab clearly has a beneficial effect on the bending moment envelopes (Figure 7). At intermediate slab level, the maximum negative moment falls from 1488.5 to 1180.3 kNm/m – a reduction in excess of 20 percent. Similarly, at base slab level, the maximum positive moment falls by 29 percent, from 703.1 to 501 kNm/m.

Figure 8 shows that the toe deflection is reduced by 22.4 mm from 107.9 to 85.5 mm – an improvement of 21 percent. As expected, the deflection reduction within the cells is lower than at the toe, due to the fixity provided by the slabs.

Both models have similar roof slab tensile forces of 115 (Model 1) and 113 (Model 2) kN/m, while base slab loads in both are insignificant compressions of 1.1 (Model 1) and 2.2 (Model 2) kN/m. However, the intermediate slab, which bears the greatest force, enjoys a reduction of 14 percent, from 1271 (Model 1) to 1094 kN/m (Model 2).

Maximum ground settlements immediately behind the wall and 24.6 m away drop from 125 mm and 66 mm respectively in Model 1 to 108 mm and 39 mm respectively in Model 2. Base heave falls from 177 to 161 mm, and the groundwater table is not significantly affected in Model 2, while it falls 1.5 m behind the walls in Model 1.

It is suggested, therefore, that, even a low-strength JGP slab of relatively small thickness (2 m) placed immediately below final formation level, is likely to be significantly beneficial to a top-down braced excavation in firm clay with a high groundwater table.

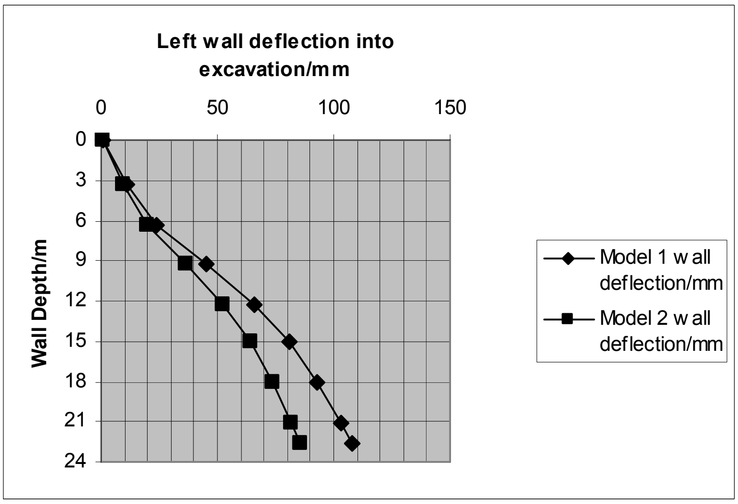


Figure 7: Bending moment (BM) envelope comparison between Models 1 and 2 (left walls).

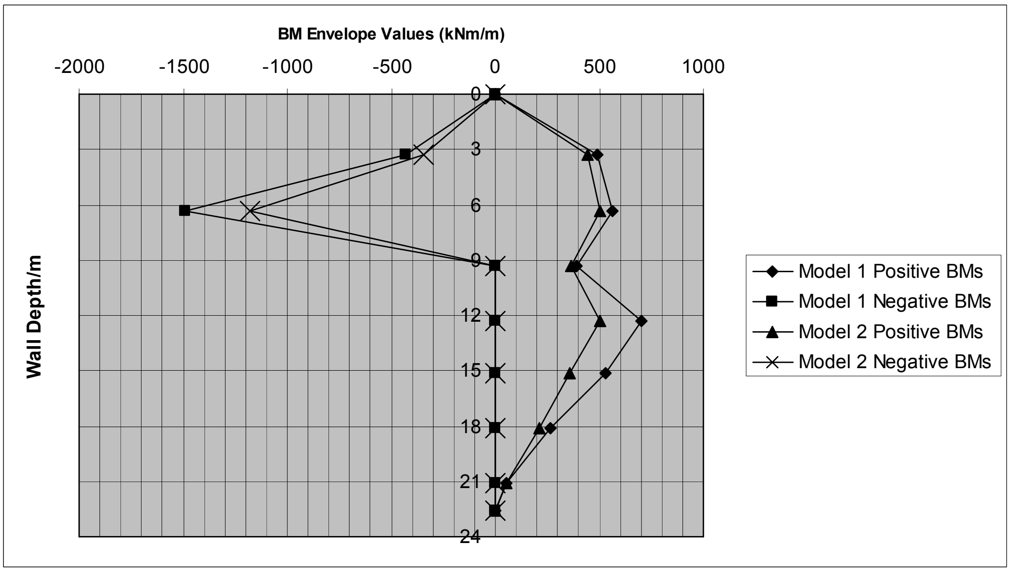


Figure 8: Left wall deflection profile comparison between Models 1 and 2.

4.3.2 The positional effects of a 2 m thick LS JGP slab (comparison between Models 1, 2, 3 and 4)

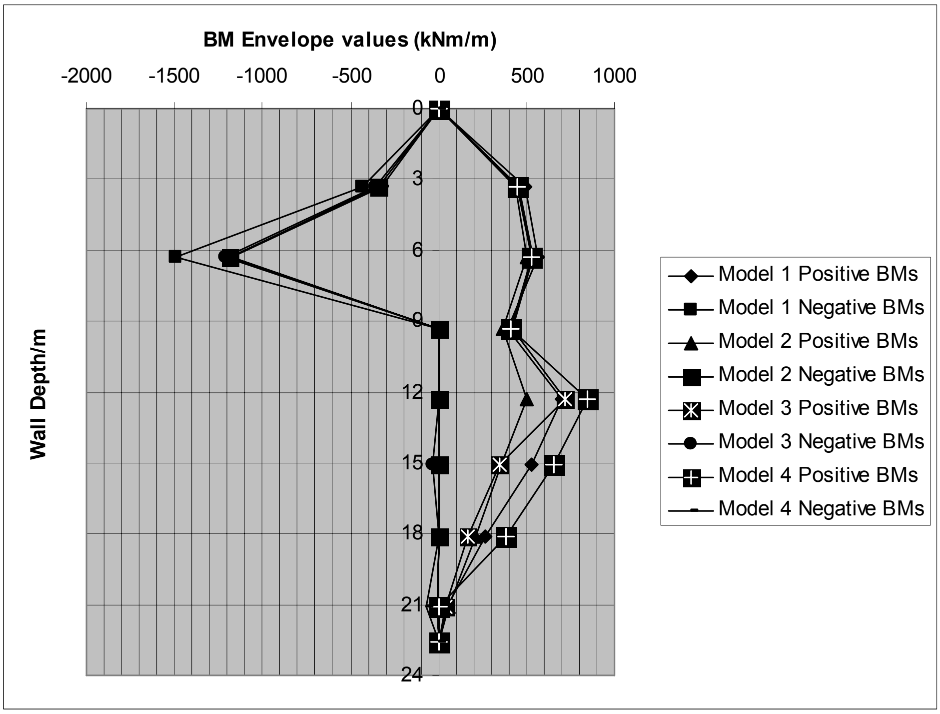


Figure 9: Bending moment (BM) envelope comparison between Models 1, 2, 3 and 4 (left walls).

Based on Figure 9, all modelled locations of the 2 m thick LS JGP slab provide similar reductions in the maximum negative moment at intermediate slab level (6.3 m depth) and mid-height of upper cell (3.3 m depth). As per Figure 7, there is no other area of significant negative bending moment envelope reductions.

Overall, Model 2 is the most effective at positive moment reduction, particularly above final formation level. However, below about 15 m depth, Model 3 tends to provide the greatest moment reduction.

Below the mid-height of the lower cell, Models 3 and 4 actually tend to increase the positive bending envelope values therein, greater than Model 1 values, with Model 4's envelope values being the most severe.

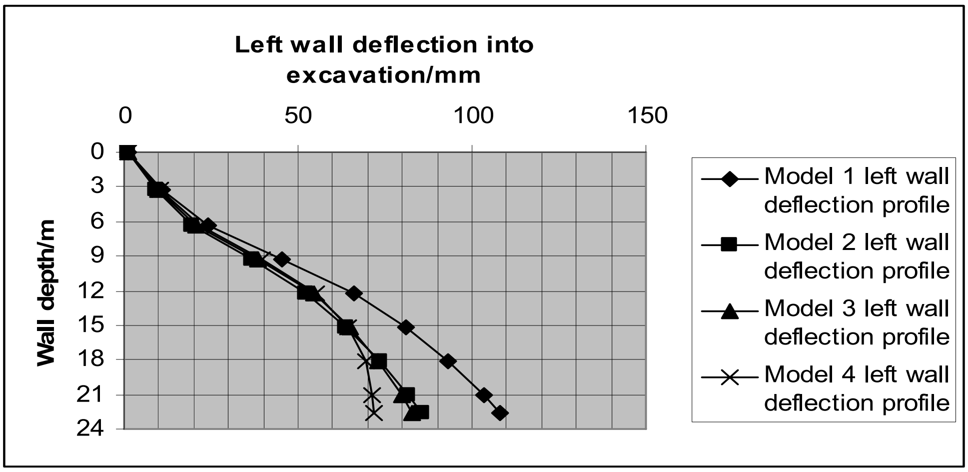


Figure 10: Left wall lateral deflection profile comparison between Models 1, 2, 3 and 4.

Based on Figure 10, all modelled JGP slab locations would reduce lateral deflection by approximately the same quantity above 18.1 m depth.

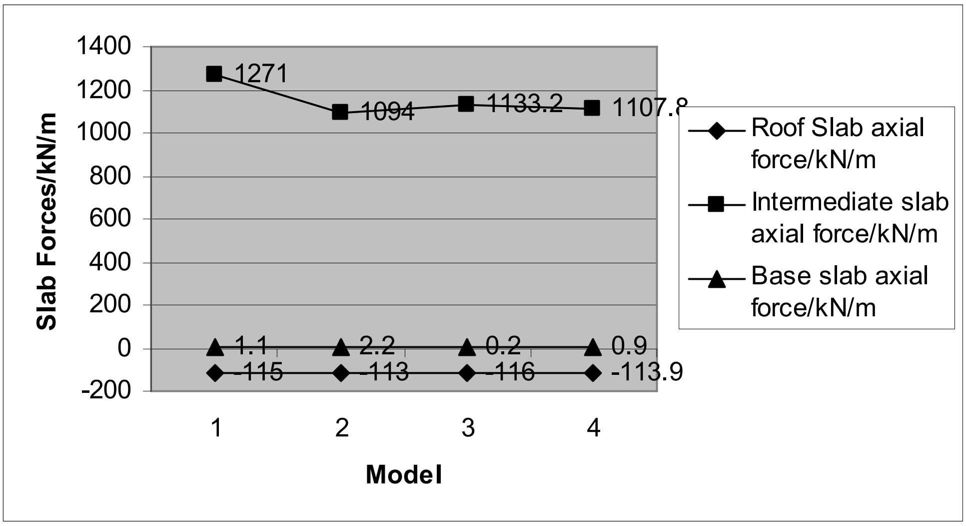


Figure 11: Slab axial force comparison between Models 1, 2, 3 and 4.

Base and roof slab forces appear largely unaffected by the presence of any JGP slab, let alone its position (Figure 11). However, the JGP slabs do reduce the intermediate slab axial forces somewhat.

Ground settlement is apparently unaffected by JGP slab location (Figure 12). However, Model 4's result suggests that the lower the JGP slab between walls, the lower the base heave.

The data suggests that, overall, the most effective JGP slab location is immediately below final formation level.

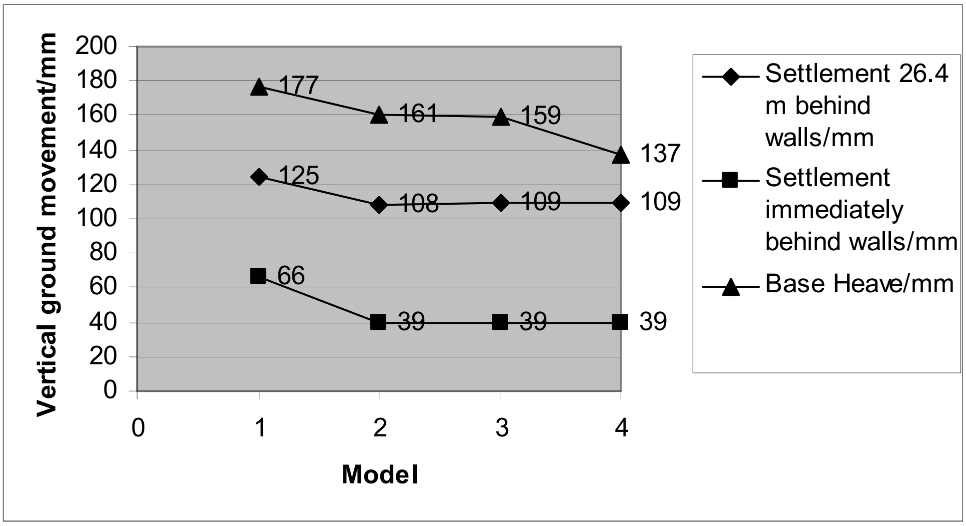


Figure 12: Vertical ground movement comparison between Models 1, 2, 3 and 4.

4.3.3 The strength effects of a 2 m thick JGP slab (comparison between Models 1 to 10)

Interestingly, Figure 13 suggests that increasing JGP slab material strength in factors of 3 and 6 does clearly reduce maximum positive moments above the mid-height of the lower cell (9.3 m depth), but not dramatically so. The dramatic reductions only occur between depths 9.3 m and 18.1 m (5.5 m below base slab soffit).

The opposite is true for the negative bending moment envelope - reductions are dramatic above 9.3 m depth. However, below that, the stronger the grout, the greater the magnitude of the negative bending moment.

This suggests that increasing grout strength is not a straightforward solution to reducing wall bending loads.

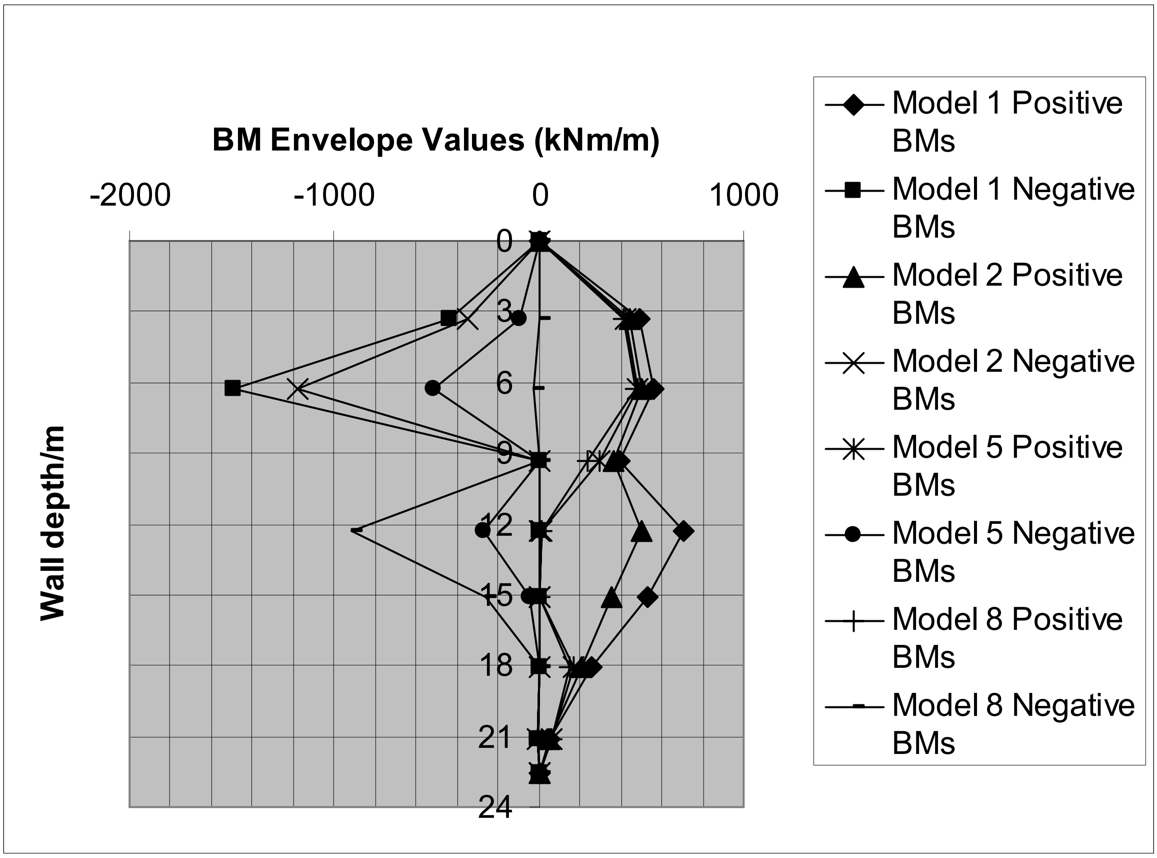


Figure 13: Bending moment (BM) envelope comparison between Models 1, 2, 5 and 8 (left walls).

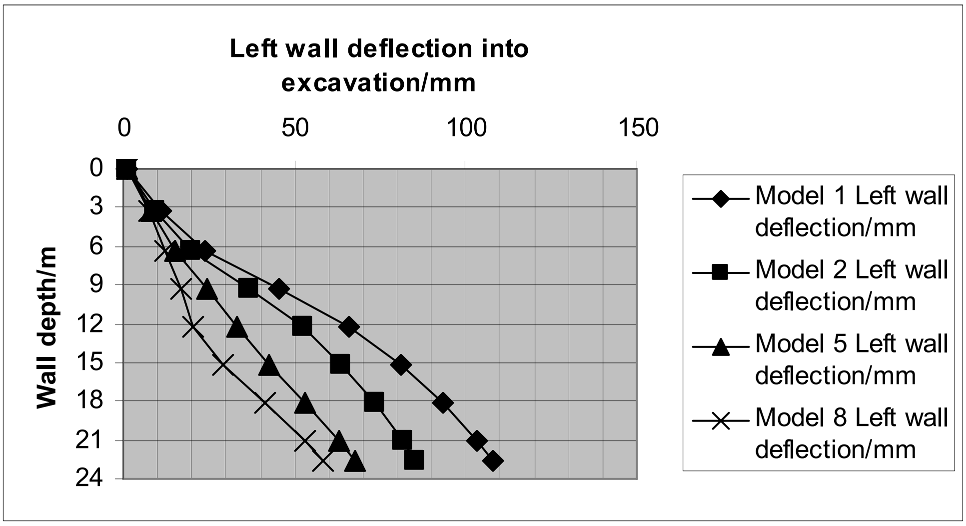


Figure 14: Left wall lateral deflection profile comparison between Models 1, 2, 5 and 8.

Figure 14 shows that as grout strength increases, wall lateral deflection clearly reduces, from intermediate slab depth (6.3 m) downwards. Figure 15 suggests that the magnitudes of roof and intermediate slab forces reduce with increasing grout strength. In addition, the roof slab axial force changes from tensile to compressive.

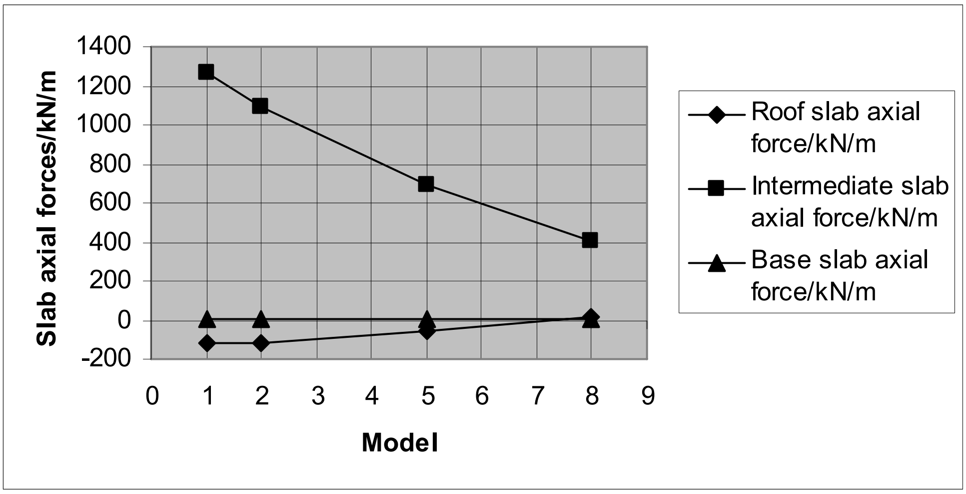


Figure 15: Slab axial force comparison between Models 1, 2, 5 and 8.

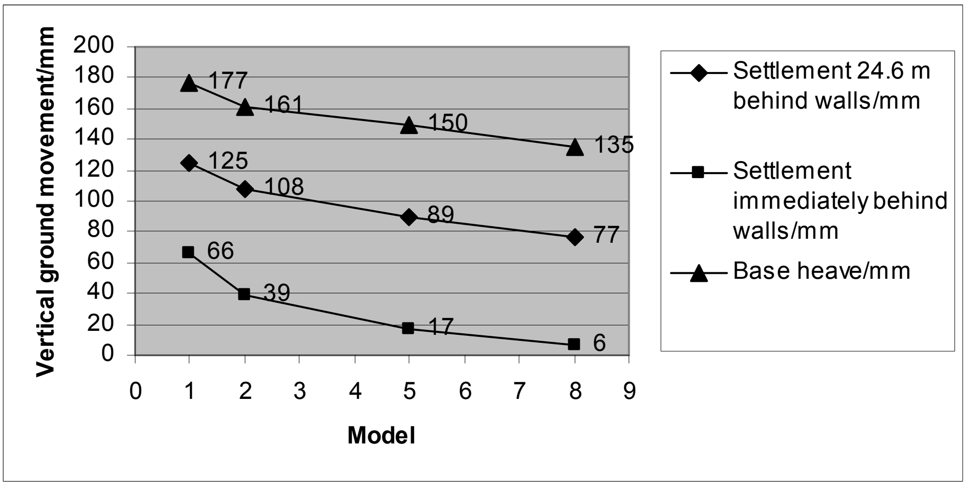


Figure 16: Vertical ground movement comparison between Models 1, 2, 5 and 8.

Figure 16 suggests that increasing grout strength may be a reliable method of ground settlement control around excavations.

Comparative inspection of the numerical output of Models 6, 7, 9 and 10 confirms the trends mentioned in Section 4.3.2 above - that, even for higher-strength slabs, ground settlement is not significantly affected by lowering the JGP slab, though base heave is significantly lowered. The slab force relationship with depth is also verified to match Figure 11 above.

The inspection also suggests that lowering the position of high-strength JGP slabs increases the bending moments, both positive and negative, especially above mid-height of the lower cell. The same is observed between JGP slabs at the same depth - increasing the grout strength significantly increases bending moments both positive and negative.

The conclusion here is that increasing JGP slab strength may achieve ground settlement, lateral wall deflection and slab axial load reductions, but significantly increases the required bending moment capacity of the wall, at almost all depths.

4.3.4 The effects of thickness of LS/MS JGP slab (comparison between Models 2, 11, 12 and 14)

Clearly, increasing the thickness of the LS JGP slab from 2, to 4, to 6 m reduces the negative bending moments in the upper cell, but nowhere else (Figure 17). In fact, below the base slab, negative moments actually increase around 15 m depth – the approximate centre of the JGP slabs in Models 11 and 12.

On the other side, increasing the JGP slab thickness significantly reduces the positive bending moment envelope only from around base slab soffit (12.6 m) downwards to around 19 m depth. In fact, above lower cell mid-height (9.3 m), moments actually increase marginally with increasing JGP slab thickness. Overall, only significant thickness increases from 2 m to 6 m result in significant moment reductions.

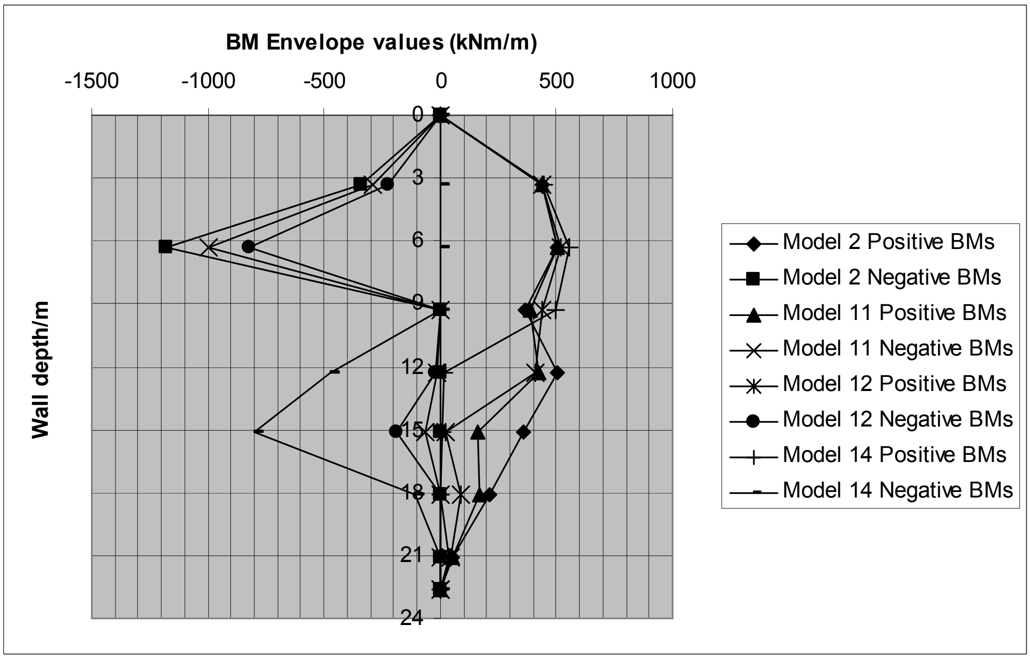


Figure 17: Bending moment (BM) envelope comparison between Models 2, 11, 12 and 14 (left walls).

Interestingly, by tripling the strength and stiffness of the 6 m thick JGP slab, the maximum positive bending moments marginally reach their maximums above 9.3 m depth, and then dramatically drop to zero all the way down to toe level. It is basically the reverse with the corresponding negative moment profile.

The conclusion here is that increasing JGP slab thickness does generally reduce moments everywhere, with low-magnitude local exceptions. Optimising the combination of grout strength and thickness increases can be effective in significantly reducing moments in critical areas.

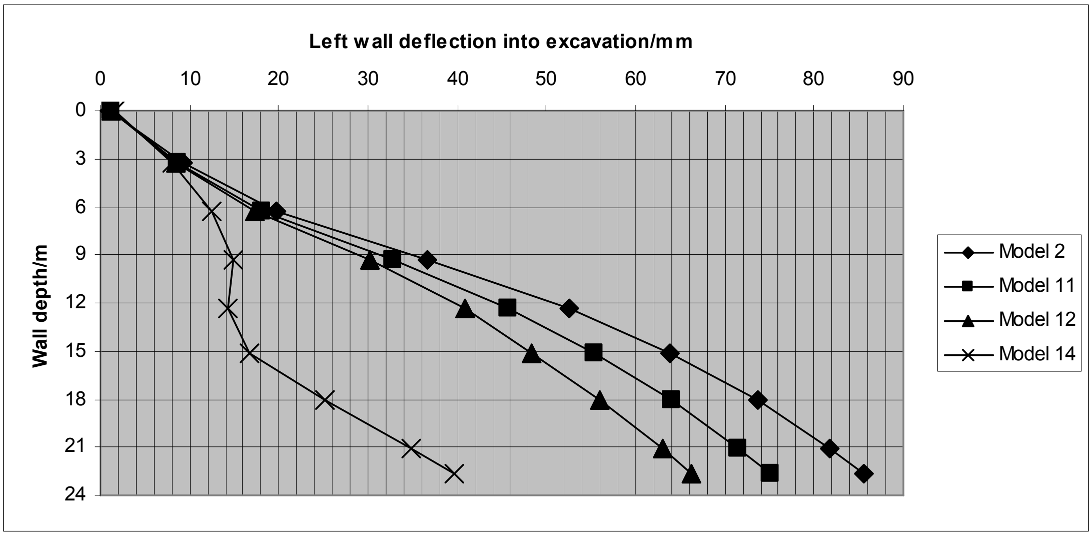


Figure 18: Left wall lateral wall deflection profile comparison between Models 2, 11, 12 and 14.

Figure 18 shows a clear trend of smaller lateral deflections with increasing JGP slab thickness. The reduction effects increase with depth, starting from mid-height upper cell (6.3 m depth).

Furthermore, as concluded earlier, grout strength increases do reduce deflection, and this effect may be magnified by thickness increases – note the large difference between Models 12 and 14, from upper cell (3.3 m depth) downwards.

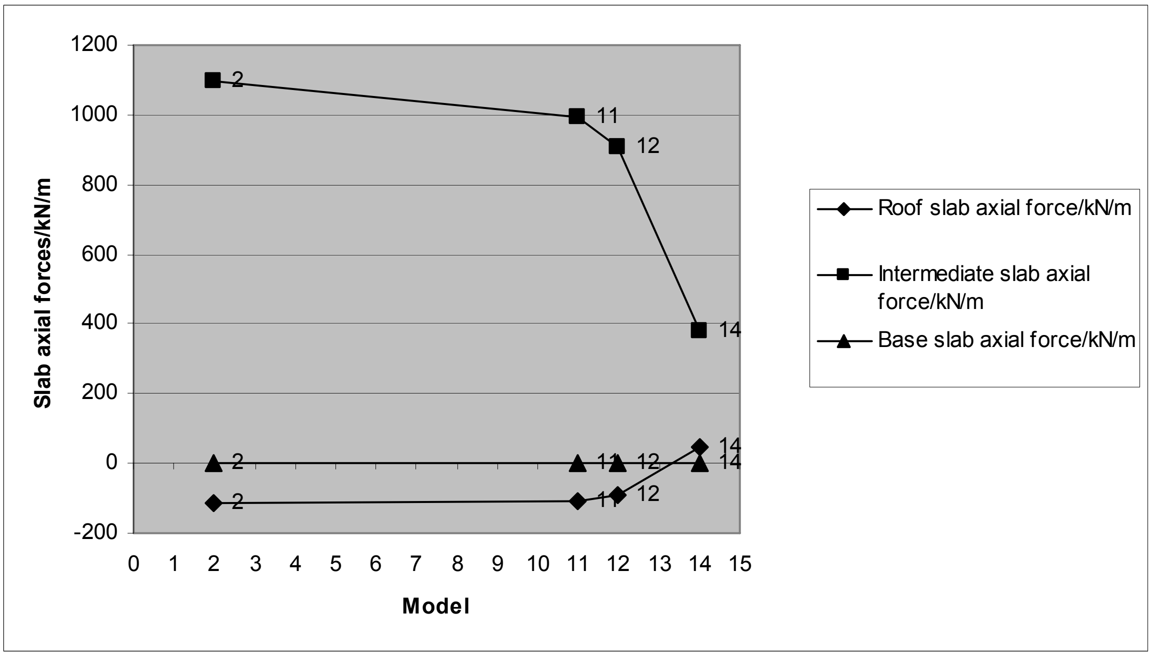


Figure 19: Slab axial force comparison between Models 2, 11, 12 and 14.

Figure 19 shows that intermediate slab axial loads reduce with increasing JGP slab thicknesses. Furthermore, increased JGP slab thickness, coupled with a strength increase (Model 14), results in a dramatic drop in intermediate slab loads, as well as reverses roof slab loads from tensile to compressive.

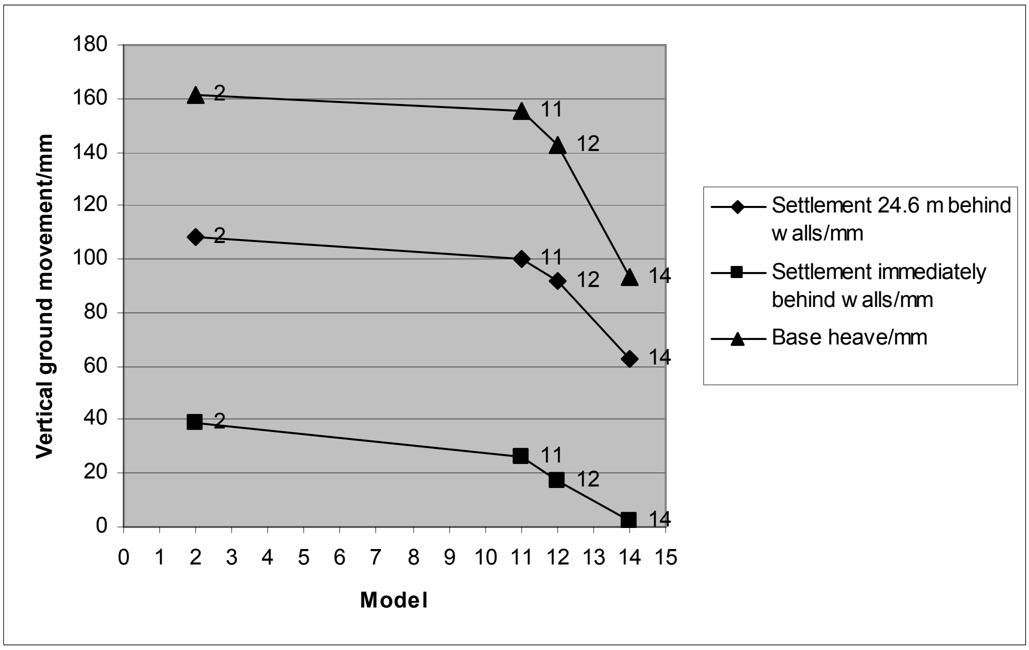


Figure 20: Vertical ground movement comparison between Models 2, 11, 12 and 14.

Increasing JGP slab thickness does reduce ground movements generally, but substantial reductions require substantial slab thickness increases (Figure 20). Coupling and increase in JGP slab strength and thickness dramatically reduces ground movements, with the strength increase being the driver.

Concluding this section, it is suggested that increasing JGP slab thickness generally reduces bending moments, wall lateral deflections, slab axial loads and ground movements. However, except for bending moments, it is not as effective as increasing JGP grout strength.

4.3.5 Comparing soils (comparison between Models 1 and 15)

The loose to medium-dense sand modelled had the following parameters:

- Unit weight - 17 kN/m³
- Permeability - 0.0086 m/day
- Poisson's ratio - 0.30
- Stress-strain modulus - 25 MPa
- Cohesion - 1 kPa and
- Internal angle of friction - 32°.

By inspection of the numerical output, the following observations were made regarding the same top-down excavation without JGP slabs, in loose to medium-dense sand and firm clay:

- In sand, both positive and negative moments are significantly less than their corresponding values in clay above intermediate slab (6.3 m depth), while the reverse is true below;
- In sand, lateral wall deflections into the excavation are about half those in clay, at all depths;
- In sand, roof and intermediate slab axial forces are significantly lower than in clay, though the reverse is true for the base slab and
- In sand, ground settlement is dramatically lower than in clay, although base heave is significantly greater.

5 CONCLUSIONS

The following general conclusions may be drawn, regarding top-down braced excavations in firm clay with high groundwater tables:

- Even a low-strength JGP slab of relatively small thickness (2 m) placed immediately below final formation level is likely to be significantly beneficial in terms of reducing wall/slab loads and vertical ground movements;
- The optimum JGP slab location is likely to be immediately below final formation level, though the lower it is (away from final formation level), the greater the base heave reduction;
- Increasing the strength and stiffness of JGP slabs is advantageous in reducing structural slab axial loads, wall deflections and vertical ground movements. However, it significantly increases wall bending moments;
- Only significant JGP slab thickness increases produce significant wall bending moment and vertical ground movement reductions. Wall lateral deflections and slab axial loads reduce with increasing JGP slab thicknesses, though the deflection reductions become more significant with increased proximity to the JGP slab itself. Increasing a JGP slab's thickness is not as effective as increasing its strength and stiffness and
- For qualitative purposes, based on the trends noted, simple approximate correlations between various parameters may be observable, but further work would be necessary to incorporate other factors like wall toe depth, wall stiffness and excavation width to confirm them.

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