

Comparison of Designing a Retaining Wall for bottom up and top down construction of a deep underground station

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INTRODUCTION

Phase one of the Copenhagen Metro, is being constructed by COMET, a consortium of six companies (Astaldi S.p.A., Bachy Soletanche Ltd., Ilbau Gesellschaft mbH, NCC Rasmussen & Schiotz Anlaeg A/S, SAE International and Tarmac Construction (Denmark) Ltd.). The work involves the construction of seven underground stations, 7.1km of twin bored tunnels, and 6km of elevated viaducts and stations. As part of COMET, Bachy Soletanche are carrying out the geotechnical work, i.e. retaining walls, dewatering and grouting. This paper considers the effect that changing the construction sequence of a deep station has on the retaining wall. Stability calculations are not considered in this paper, since it is only concerned with the effects on a given retaining wall (in terms of depth and stiffness).

STATION DESCRIPTION

The deep stations are generally located within the “town squares” of Copenhagen surrounded by four or five storey buildings very close to the proposed stations. These buildings are typically bearing on shallow foundations approximately 3m below the ground level. To ensure the stability of these structures, stiff, multi-propped retaining walls were chosen for the deep stations. These walls for the deep stations (approximately 65m long x 20m wide x 23m deep) consist of hard/soft secant piled walls. The hard piles being 1180mm diameter in the overburden materials, reducing to 1050mm diameter in the Copenhagen limestone, and generally spaced at 1500mm centres. Between these piles, to form a water cut-off, 750mm diameter soft piles extend to toe into the limestone.

GROUND CONDITIONS

The ground conditions in Copenhagen consist of a layer of superficial made ground, predominantly loose sand with some peat and clay. Underlying the made ground there is a sequence of interbedded sand and clay till of glacial origin. These glacial deposits are highly variable and over-consolidated, though are considered to have a firm to stiff consistency. Found at the base of the glacial deposits are the meltwater deposits generally consisting of fine, medium dense sand with a varying proportion of gravel. Beneath the meltwater deposits the Copenhagen limestone commences, becoming less weathered and stronger with depth. The upper portion of the limestone has generally been sheared by geological processes and for design purposes is considered to have no cohesion. Characteristic soil properties are given in table 1, these values apply to working conditions and under EC7 rules would be factored for an ultimate limit state calculation against stability failure.

Soil	Bulk unit weight (kN/m ³)	Angle of internal friction (°)	Wall friction (both sides)	Drained cohesion (kN/m ²)	Elastic modulus (kN/m ²)	Poisson's ratio	Coefficient of at-rest pressure
Made Ground	19	30	$\frac{2}{3} \phi (0)$	0	10000	0.20	0.70
Glacial Till	21	32.5	$\frac{2}{3} \phi (0)$	0	50000	0.15	1.00 (0.50)
Meltwater Deposits	20	38	$\frac{2}{3} \phi (0)$	0	30000	0.20	0.50
Sheared Limestone	21	40	$\frac{2}{3} \phi (0)$	0	100000	0.35	0.50
Competent Limestone	22	40	$\frac{2}{3} \phi (0)$	50	100000	0.35	0.50

Table 1 : Characteristic soil properties (long term values in brackets)

CONSTRUCTION METHODS

Given their urban location, all of the deep underground stations were originally intended to be constructed top down, i.e. installing the permanent works during bulk excavation, to provide stiff support to the walls as early as possible and with the minimum amount of excavation.

The top down sequence is shown in figure 1a by the solid arrow line, and consists of first installing the deep roof beams at 5.5m centres close to the top of the piles. A deep capping beam is provided over the piles to resist torsional effects and to tie the contiguous hard piles

together. Excavation then takes place to the underside of the intermediate slab which is then installed prior to excavation for the station base slab. During the construction of the station, the intermediate slab is hung from the roof beams thereby applying bending moments to the piles. The completion of the station, atrium walls, technical levels etc. then further increases the bending moment applied to the piles.

The timing of the arrival of the tunnel boring machine at one station, Christianshavn, meant that a bottom up construction sequence had to be considered in order to have the station fully excavated before the machine arrives. The bottom up sequence is shown in figure 1b by the dashed arrow line, and consists of using two levels of temporary props to reach the base slab. Part of the intermediate slab is constructed during excavation to create a partial waling beam supported by temporary props extending over the full width of the station, phase 1 of figure 2.

Following construction of the base slab, the full width of the intermediate slab is constructed with box outs around the temporary props, phase 2 of figure 2. Short temporary props are then installed between the intermediate slabs whilst the full width temporary props are removed. The intermediate slabs are then completed by casting infills, the permanent props installed and the short temporary props removed, phase 3 of figure 2. Once the roof beams are constructed the upper level of temporary props is removed. This sequence applies all of the fixed end moments to the piles at roof level after the station is completed. The two sequences are summarised in table 2.

Top down construction sequence		Bottom up construction sequence	
Stage no.	Activity	Stage no.	Activity
1-5	Application of surcharges due to existing buildings	1-5	Application of surcharges due to existing buildings
6	Dewater to -13.77mKN	6	Dewater to -13.77mKN
7	Excavate to -1.25mKN	7	Excavate to -1.25mKN
8-9	Install roof beam at +1.10mKN and apply moment due to it's self weight (315kNm/m)	8	Install temporary prop at 0.00mKN
10	Excavate to -13.77mKN	9	Excavate to -13.77mKN
11-12	Install intermediate slab at -12.27mKN	10	Install temporary prop / partial waling beam at -12.37mKN
13	Apply moment at roof level due to intermediate slab self weight (445kNm/m)	11	Dewater to -21.25mKN
14	Dewater to -21.25mKN	12	Excavate to -20.45mKN
15	Excavate to -20.45mKN	13-14	Install base slab
16-17	Install base slab	15-16	Install complete intermediate slab at -12.27mKN
18	Apply moment at roof level due to station construction (2400kNm/m)	17	Remove temporary prop at -12.37mKN
19-25	Apply long term soil and water conditions	18-19	Install roof beam at +1.10mKN and apply moment due to it's self weight (315kNm/m)
NOTE : Where two supports are actually at the same level, they have been installed 100mm apart to satisfy WALLAP requirements		20	Remove temporary prop at -1.90mKN
		21	Apply moment at roof level due to station construction (2400kNm/m)
		22-28	Apply long term soil and water conditions

Table 2: Analyses of construction sequences

For both sequences the retaining wall was analysed using version 4.08 of the computer program WALLAP. WALLAP uses a series of springs to model the soil, whilst dividing the retaining wall into elements to compute bending moments and shear forces. Clearly, more information regarding ground movements would be forthcoming if a finite element analysis were performed, however the aim of this paper is to compare the effects that two different construction sequences have on the wall. The construction stages analysed for the two sequences are listed in table 2.

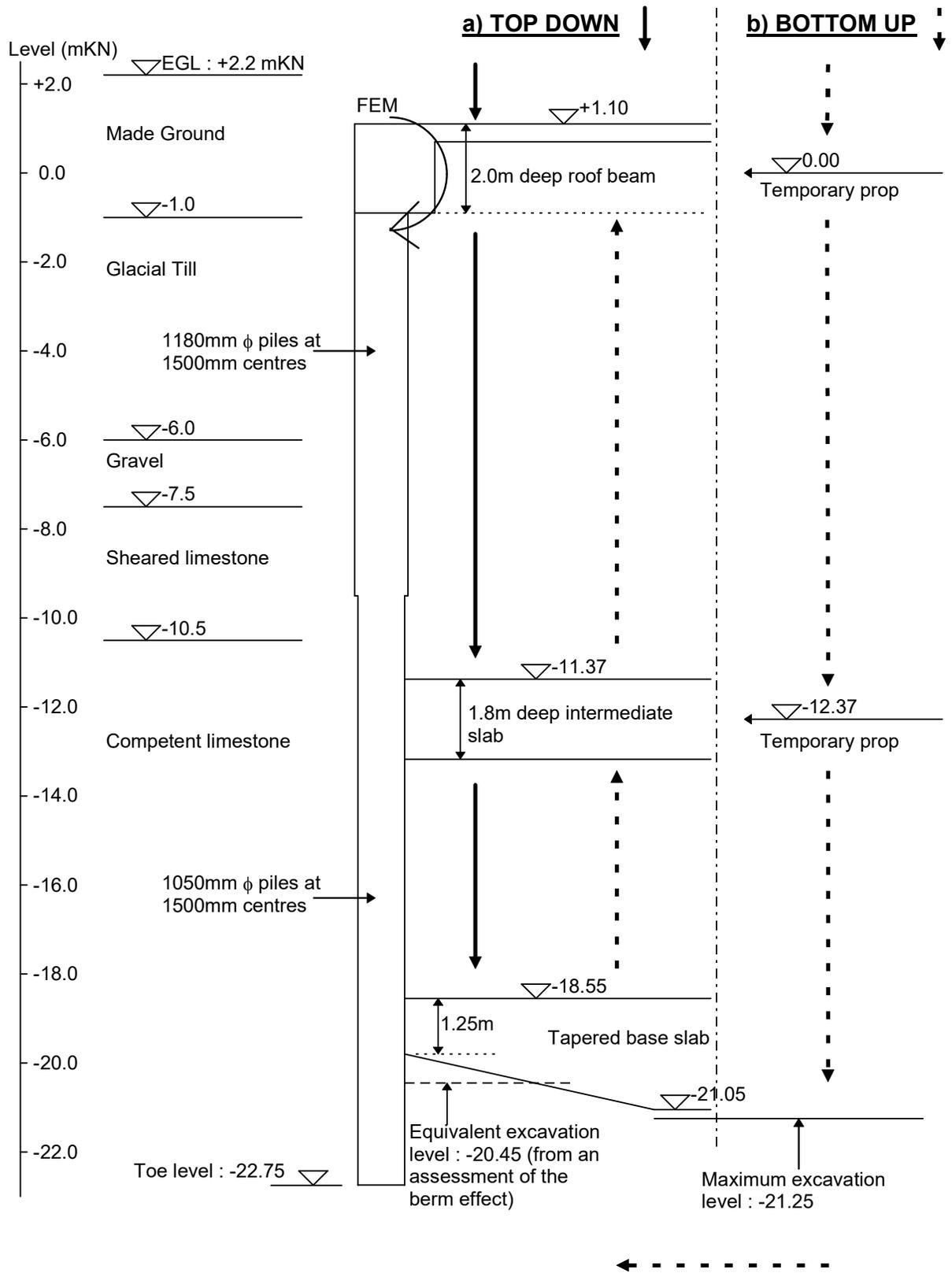
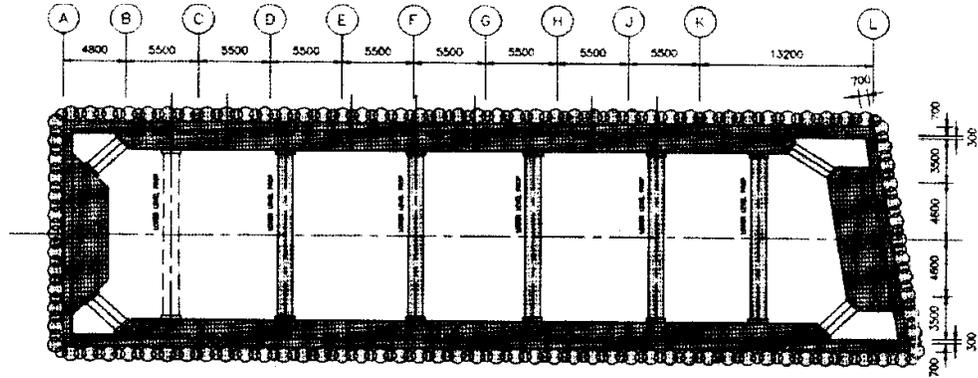
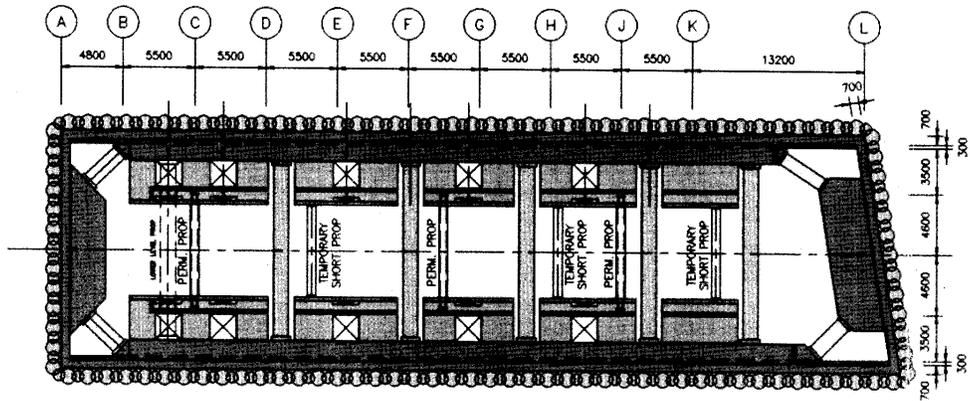


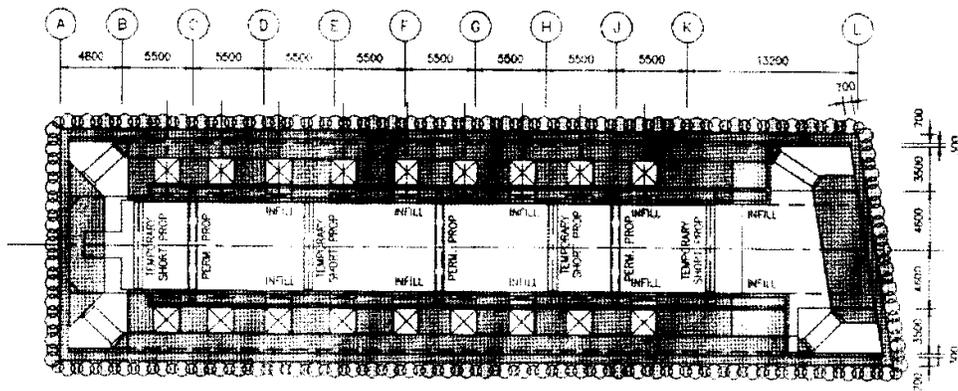
Figure 1 : Construction sequences



Phase 1: Construct partial waling and install main temporary props



Phase 2: Construct full width of intermediate slab and install short temporary props



Phase 3: Remove main temporary props, cast slab infills and remove short temporary props

Figure 2: Construction phases for the intermediate slab in the bottom up sequence

RESULTS OF ANALYSES

The results have been presented as a series of graphs to show wall displacements (figure 3), bending moments (figure 4) and shear forces (figure 5).

a) Displacements

Figures 3a and 3b show the development of wall displacements as each of the construction sequences proceeds, while figures 3c-3f compare displacements at different construction stages. Shown in figure 3d is the reduced displacement, approximately 12mm less at -8.0mKN, for the top down sequence as excavation proceeds to the intermediate level. The application of the fixed end moment, towards the excavation side, at roof level causes the wall to rotate at roof level reducing the displacements below.

Excavation to the underside of the base slab, figure 3e, causes additional movement only in the lower part of the wall. In the long term, figure 3f, the application of the final bending moment at roof level causes increased movement at the top of the wall. The rotation caused is considerable greater in the top down sequence compared to the bottom up sequence. This rotation though generally serves to reduce slightly the displacements below the roof beams. For the existing foundations at approximately 3m below the ground level, it is possible that either of the construction sequences may cause some damage. It would be difficult to suggest which of the displacement profiles and rotations are more favourable to the foundations, though overall displacements are greater for the bottom up sequence.

b) Bending moments and Shear forces

Figures 4 and 5 show the wall loading effects resulting from the two construction sequences. The two sequences only cause a significant difference in bending moments and shear forces in the upper half of the wall (above approximately -10.0mKN). Indeed below this level the loading effects are very nearly the same for both sequences. In general, changing from the top down to the bottom up sequence causes the bending moment diagram to be shifted from positive (i.e. soil side of wall in tension) to negative (i.e. excavation side of wall in tension) bending moments. Remembering that this is a piled wall, it is only the magnitude of the bending moment that affects the reinforcement distribution, assuming that the reinforcement is evenly distributed around the pile section.

In the top down sequence bending moments are applied at the roof level during the bulk excavation, i.e. before the retaining wall has had a chance to rotate due to the removal of soil. Even at the first stage of excavation, figure 4d, the rotational restraint associated with the roof beams cause large bending moments in the wall close to roof level. In the bottom up sequence the absence of the applied bending moments and rotational restraint means very little bending moment at the upper prop level and a much greater moment at the centre of the span. The increase in bending moment at roof level during excavation for the base slab in the top down sequence, figure 4e, is only due to the application of the fixed end moment. The bottom up sequence demonstrates that the excavation itself causes no change to the moments at the top of the wall.

In the long term, figure 4f, the full station completion bending moment has been applied for both construction sequences causing the wall bending moment to increase further at roof level. Generally, the long term service bending moment controls the reinforcement quantity because of the requirement to design for a 0.2mm crack width limit using the rules given in EC2. The approximately 50% larger bending moments resulting from the top down construction sequence would cause a similar increase in reinforcement quantity, compared to the bottom up sequence. Heavier main reinforcement at the top of the wall also causes serious congestion problems with the reinforcement provided in the capping beam and roof beam.

The differences in shear forces are also concentrated in the top half of the wall, figure 5, with the top down sequence resulting in slightly higher shear forces at the top of the wall. These differences, though are of less significance when compared to the differences in bending moments between the two construction sequences.

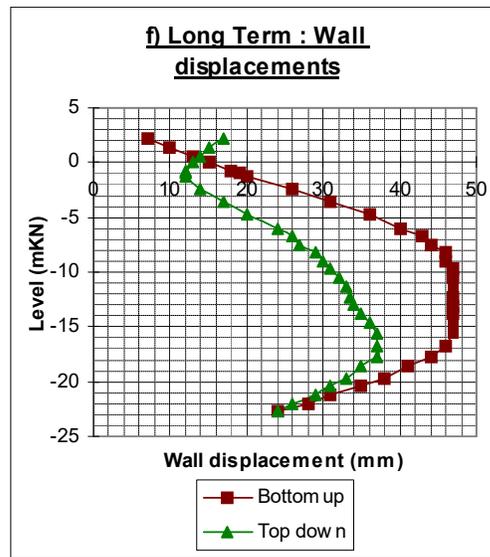
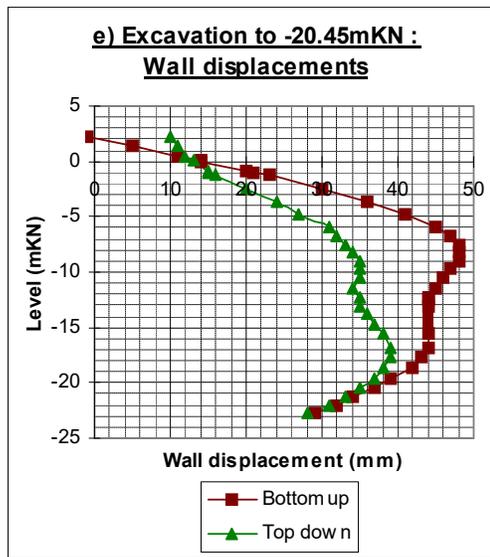
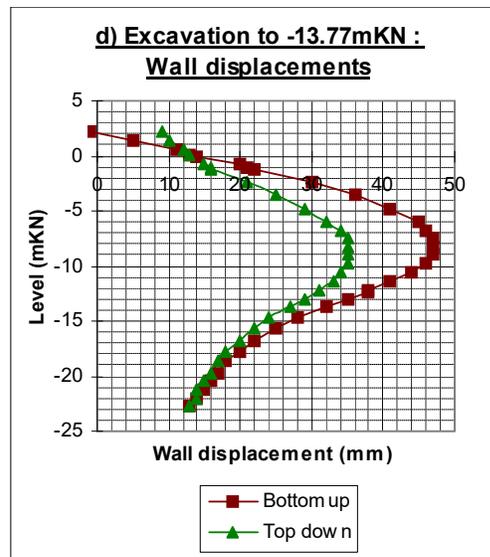
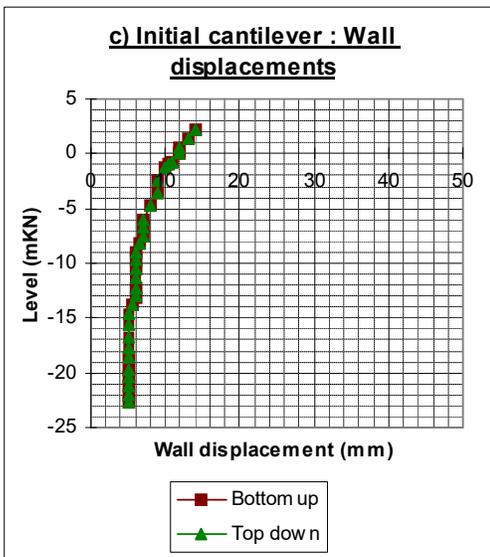
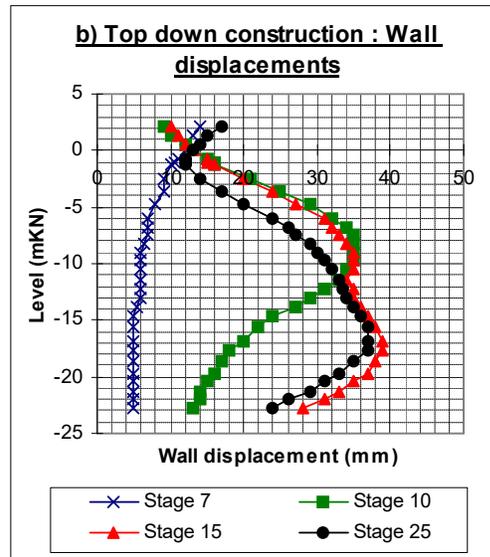
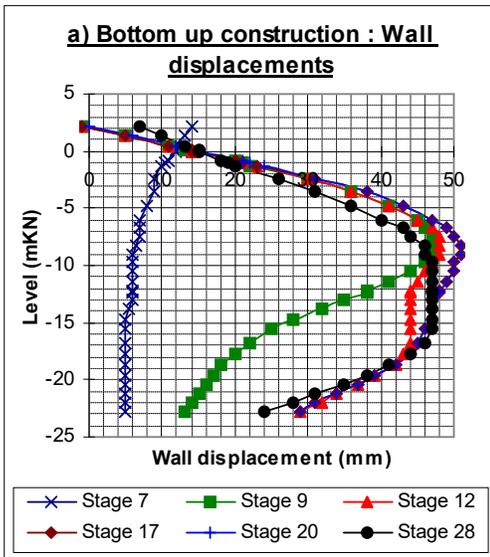


Figure 3: Wall Displacements

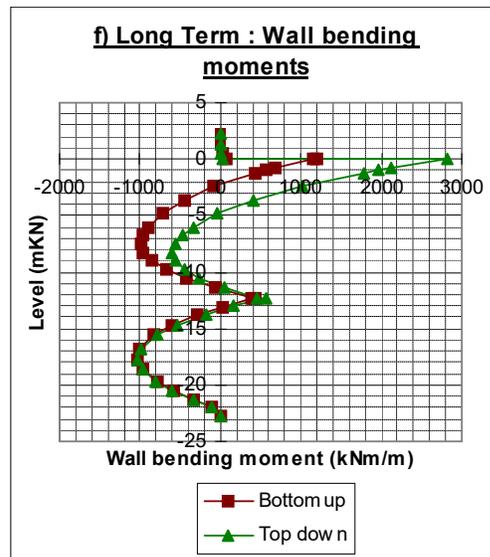
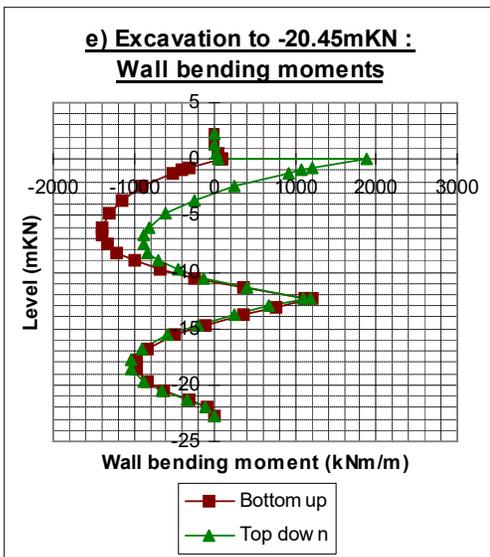
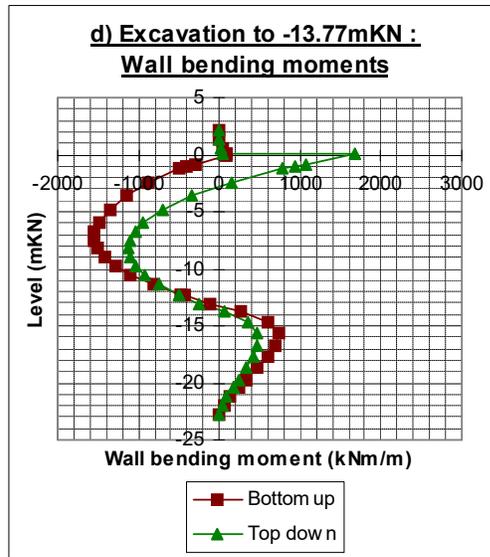
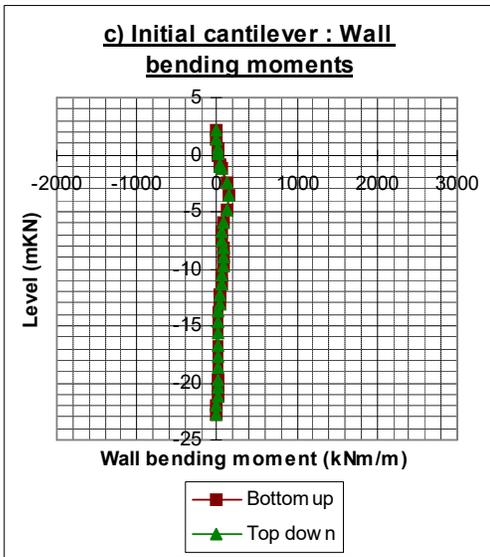
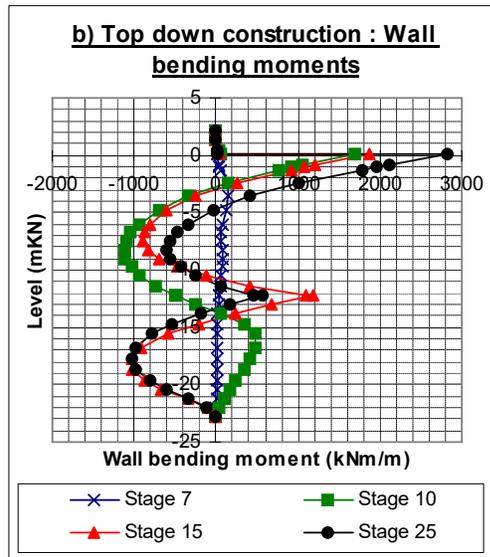
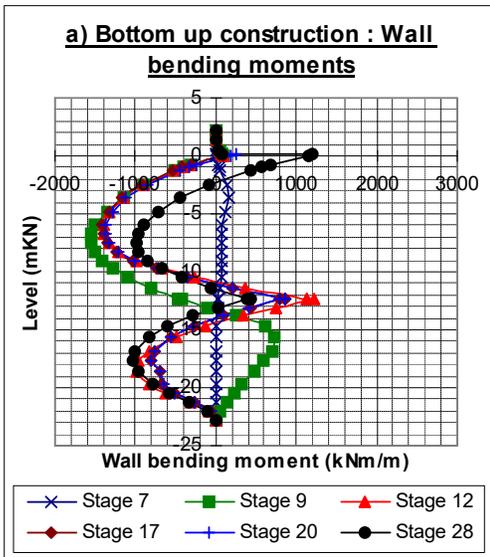


Figure 4: Wall Bending Moments

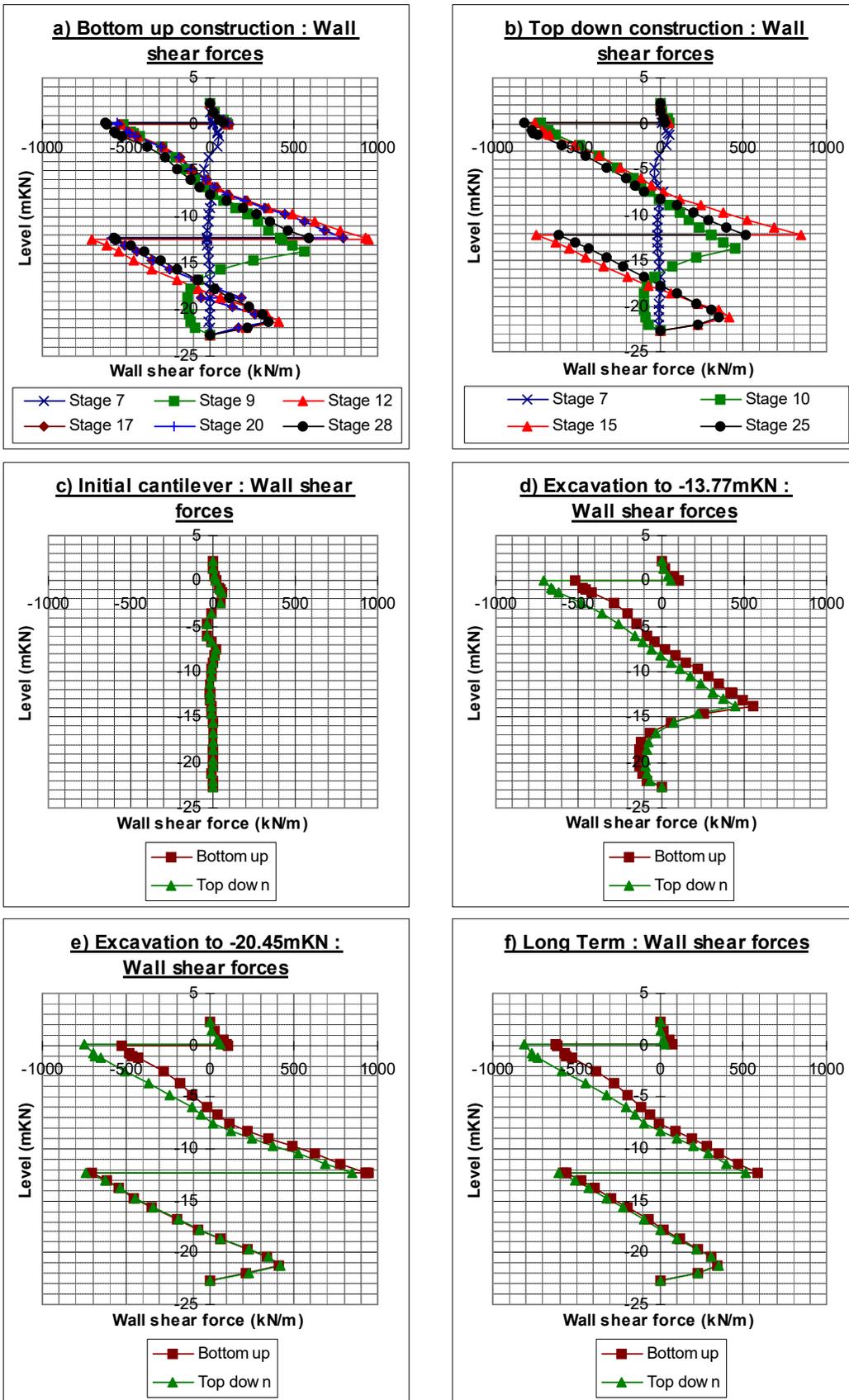


Figure 5: Wall Shear Forces

c) Support Loads

Given in table 3 are the support loads resulting from the two analyses. It can be seen that the distribution of compressive loads due to the excavation remain very similar regardless of the support stiffness. In the long term the bottom up sequence transfers less bending moment to the permanent structure because of the later installation of the permanent works.

Stage	Support Level (mKN)	Top Down Construction		Bottom Up Construction	
		Load (kN/m)	Moment (kNm/m)	Load (kN/m)	Moment (kNm/m)
Excavate to - 13.77mKN	+0.10 / 0.00	775	1636	628	N/A
Excavate to - 20.45mKN	+0.10 / 0.00	805	503	639	N/A
	-12.27 / -12.37	1583	94	1649	N/A
Long term	+0.10	841	1738	701	1303
	-12.27	1119	114	1161	27

Table 3: Support Loads

CONCLUSION

This paper has compared the effects that two different construction sequences of a deep underground station has on the retaining walls. The original top down construction sequence is contrasted with an alternative bottom up sequence.

Larger wall displacements, approximately 40% to 50% greater, are caused by the bottom up construction sequence. The installation of the roof beams, and the associated rotational restraint, in the top down sequence before the main excavation causes much larger wall bending moments compared to the bottom up sequence. These larger moments would cause a similar larger quantity of reinforcement.

It is therefore apparent that in this case the choice of the construction sequence (providing no other requirement overrules) is a trade off between displacements and reinforcement quantities. The stiffer support provided by the top down sequence reduces displacements but will increase reinforcement provision. The reinforcement increase is close to the top of the wall and therefore adds to the congestion at the capping beam level. A good design and construction sequence would lead to a balance between wall displacements and reinforcement requirements that satisfies all the product specifications.

These specifications arise from the often conflicting requirements of a number of affected parties. The client specifies the size of the station, both in terms of dimensions and passenger capacity. Neighbouring property owners are concerned about damage to their buildings caused by the station construction. The engineer specifies the design requirements in terms of strength (ultimate limit state) and durability (service limit state), for example the required crack width limit. The contractor meanwhile specifies construction requirements to ensure a quality product, for example the spacing of reinforcement bars. All of these specifications must be balanced with the need to arrive at an acceptable but efficient and economic design.

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