

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Dealing with the challenges of underground construction

O. Sigl

Geoconsult Asia Singapore, Singapore

ABSTRACT: The paper is intended to highlight and discuss the solution to major challenges of planning underground projects in urban environments. These discussions are presented in the form of examples from the view-point of a practitioner, who is deeply involved in the actual design for the implementation of such projects. Infrastructure in large cities is getting denser over time. Therefore, interference with existing structures is becoming a very common feature during the implementation process of new projects. As consequence, actual geotechnical challenges often relate to the application of innovative methods of construction in order to minimize potential construction impact or disruption. The paper wants to direct the focus on the application of unusual construction methods and related design and construction challenges.

1 GENERAL

As infrastructure in the dense urban environment of today's large cities is getting even denser over time and interference with existing structures is becoming a very common feature during the implementation process of new projects. Although, on most metro transportation projects, sufficiently large work sites are provided for station excavation, this is most of the time not true for entrance structures, which often are located on the "other side" across major roads or junctions, requiring connection by underground pedestrian linkways or underpasses. Such linkway structures normally are Cut & Cover excavations, construction of which is often affected by numerous stages traffic diversions and existing services and utilities running across the intended alignment.

Therefore, actual geotechnical challenges often relate to the development of innovative construction methods featuring improved safety and productivity as well as reduction of potential construction impact and traffic disruption. This is demonstrated on a number of examples presenting, among other aspects, new or unusual methods of construction.

2 BOX JACKING

The subway station is located beneath a major road and features five entrances connecting the surface to the station's concourse level. As two of the entrances are located on the other side of a road junction, they

are connected through an about 156 m long pedestrian underpass crossing the road junction.

The structure is very shallow with about 6m of overburden and located in very soft ground with SPT-N₃₀₀ values in the range of 15 blows or less, which is overlaying highly weathered residual soils of granite rock.



Figure 1 Layout of pedestrian underpass with arrows marking location of knock-out panels for future connections

Although, the reference design considered a conventional Cut & Cover solution for the underpass and the entrances, it was found that due to the number of utilities and services running along the road a trenchless solution for the underpass would have numerous advantages for the construction program, by eliminating diversion of services as well as the related stages of traffic diversions.

Since conventional mined options were not feasible due to very shallow overburden and the presence

of very soft ground, an alternative construction method was developed using a Rectangular Box Jacking Machine (RBJM). The proposed alternative method resulted in productivity improvement, thus reducing the overall construction duration required for the completion of this underpass structure.

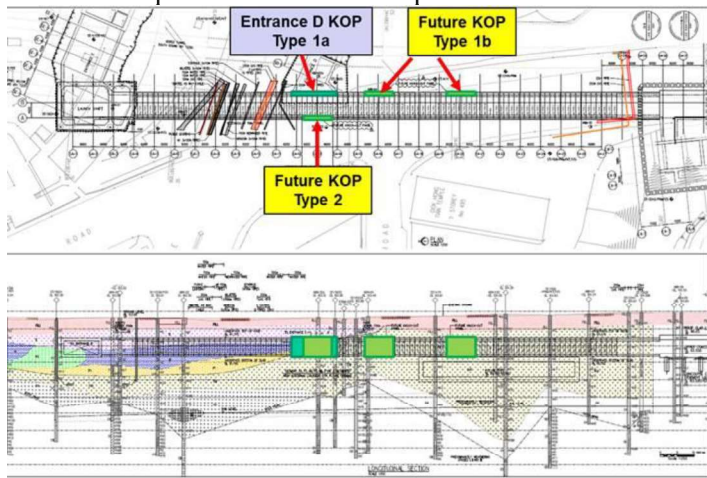


Figure 2 Plan and geotechnical longitudinal section (KOP=Knock out panel)

The earth pressure balanced RBJM was pushed through the ground from the jacking shaft at Entrance E. The pre-cast box segments with dimensions $H/W/d/L = 5.6/7.6/0.5/1.5$ m were cast on site.

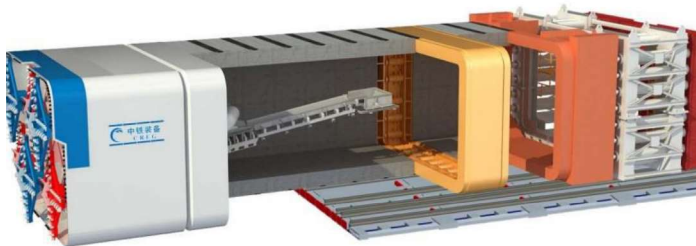


Figure 3 Rectangular Box Jacking Machine (RBJM)

As an additional complication, the design of the precast box segments had to accommodate knock-out panels for connection with Entrance D and three more future connections to entrances to be constructed at a later stage by conventional Cut & Cover methods with sheet piling. Since these connections are of the same width as the underpass itself, the respective openings cover typically 5 or 6 box segments, a structural span which is impossible to fully accommodate in the design of the underpass structure.



Figure 4 Jacking segment, lifting into the jacking shaft.

Therefore, such connections are designed such that the actual structure supporting the large opening is constructed together with the future structure, thus requiring only relatively small allowances in the structural design of the underpass' box elements.

The longitudinal continuity of the underpass structure tunnel was ensured by internal second stage cast in situ structures and post tensioning with high-strength rebars.

As a side note, the RBJM machine was later reused to successfully construct another pedestrian subway on another project in similar conditions featuring shallow overburden and very soft ground.

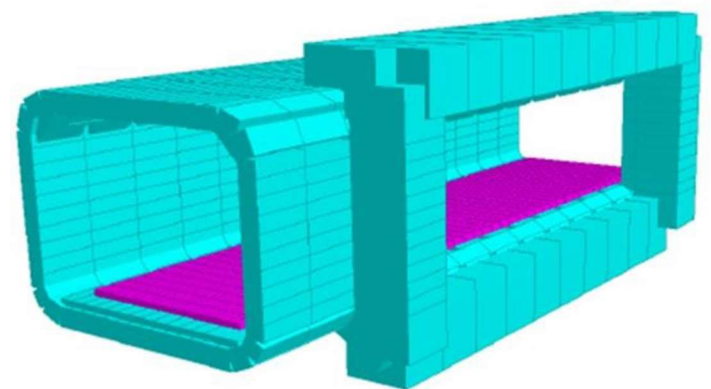


Figure 5 Structural arrangement at knock-out panel locations.

3 PIPE PILE BOX WITH STEEL FRAMES

A new pedestrian linkway will be built to connect a new entrance with an existing subway station.

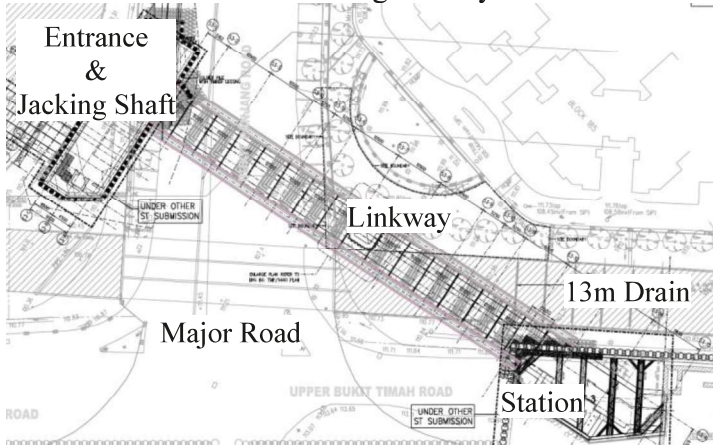


Figure 6 Layout plan of the pedestrian linkway

The 7.5m wide and 6m high linkway is passing underneath a major road and a major, 13m wide, drainage canal.

The ground condition generally comprises of Fill, soft clays, fluvial sand, residual soil, and granite rock.

Owing to the very shallow cover and the poor ground conditions, a robust excavation support system was required and interlocked jacked-in pipe pile box with heavy steel support frames was chosen in the reference design.

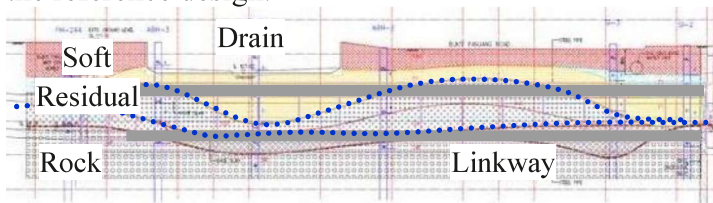


Figure 7 Geotechnical profile.

The pipe piles are installed by pipe jacking method. The pipe elements are welded together and, after the completion of the permanent works, will be filled with concrete.

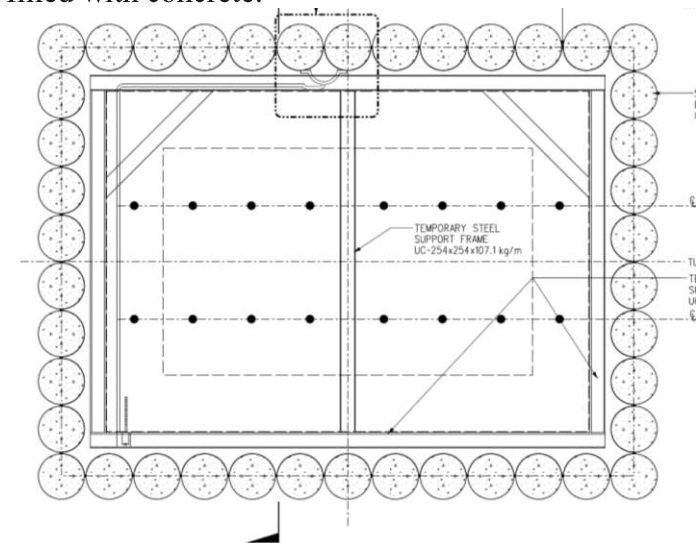


Figure 8 Original cross section with 813mm diameter pipe piles

After completion of excavation works, the permanent works will be cast in-situ in stages. The

temporary steel support frames are removed on-by-one during this process.

The original plan was to install a pipe pile box consisting of 42 nos. of 813mm diameter pipe piles placed around the circumference of the excavation. Subsequently, after award, the contractor proposed to change the arrangement to fewer, but much larger diameter pipes with 2.03m diameter.

This alternative proposal was the outcome of project risk assessments based on the following main considerations related to the main hazards and risks:

- Presence of mixed ground condition: With the presence of hard rock and soft ground within the same excavation face the pipe jacking installation of the pipe piles is an issue. Worn out cutting tools and blockages in the slurry system of small diameter machines are a well-known risk as small diameter pipe jacking machines will not allow cutterhead interventions. A pipe pile, which gets stuck prematurely, considerably increases risks further down the timeline, such as instabilities and volume loss during subsequent excavation and ground water inflows. Using large diameter pipe pile access to the cutterhead and the already installed string of pipes allows corrective actions, if needed.
- Overcoming a stuck pipe: In the case, a pipe is getting stuck because of misalignment of the adjacent interlocking pipes, a large diameter pipe allows retracting the pipe jacking TBM and reattempting the installation with a slightly smaller (undersized pipe).
- Presence of very soft ground: In this condition, the excavation face during the excavation is highly unstable, thus requiring ground treatment of the soil encompassed within the pipe pile box. When using large diameter pipes, the ground treatment can be carried out through the pipes instead of the ground surface.
- Ground treatment: Since the reference design also required ground treatment due to the soft nature of the soil material, all such ground treatment was planned to be carried out horizontally from the shafts, as lane-to-lane traffic diversion on the surface was practically impossible at this location. The quality of horizontal ground treatment over long distances was considered inferior when compared with vertical ground treatment. The alternative proposal included ground treatment installed from within the pipes instead of from the surface.
- Number of pipes to be installed: A large number of pipe piles will increase the risk and probability of one pipe pile getting stuck. Since the ground treatment in the alternative proposal could be tailored to actual conditions with the drilling of the ground treatment holes acting also as probe drillings, the

lower pipe row at the bottom of the box could be omitted.

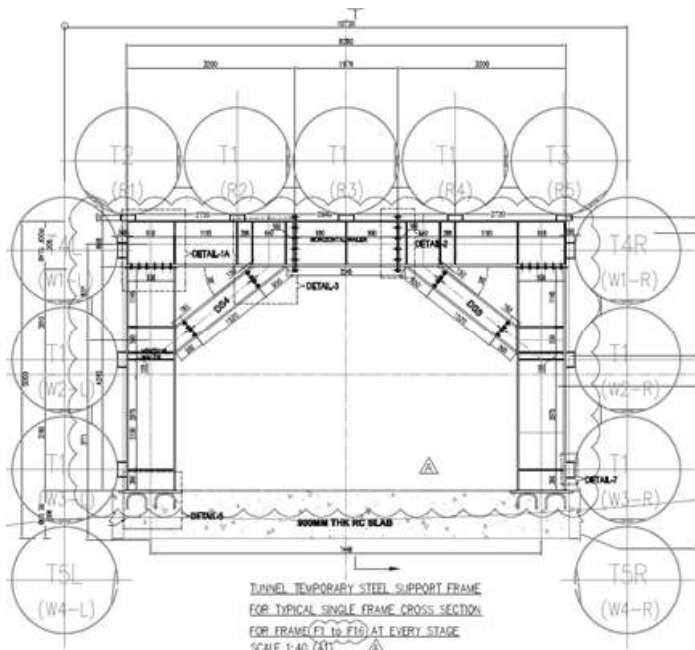


Figure 9 Cross section with 2.03m diameter pipe piles

Therefore, when compared with the original reference design, the alternative proposal with fewer but much larger pipe piles, significantly reduced the overall project risk and had definitive advantages with respect to productivity and the overall project schedule.

As mentioned above, when using the large diameter pipe piles, it became possible to carry out jet grouting (JGP) from within the pipe piles down to the rock head. This required only relatively short JGP columns of 2 to 4m length instead of almost 12m, when carried out from the surface.

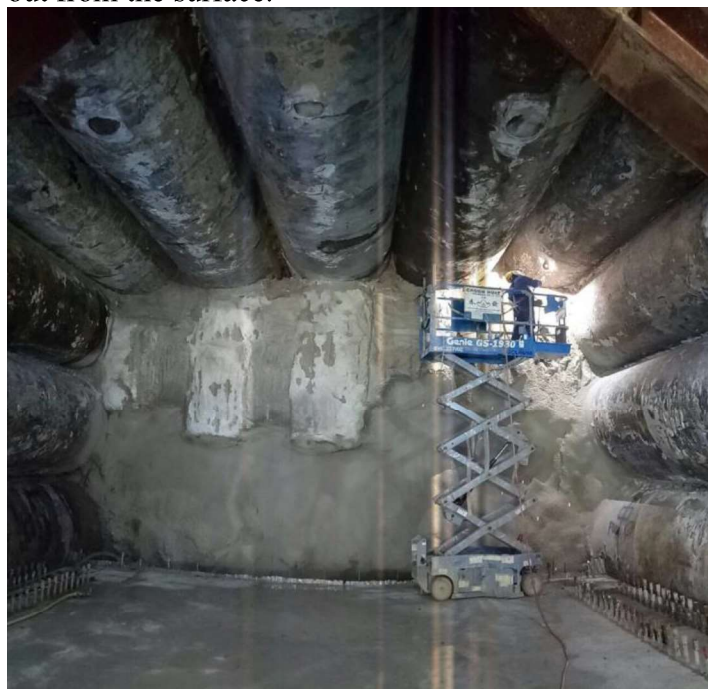


Figure 10 Pipe piles and large diameter jet grouting columns

The installation of JGP ground treatment from within the large diameter pipes proved to be an outright success. This was despite significant initial concerns with respect to the proposed achievable JGP

column diameter of at least 1.2m, which was considered unreasonably large.

However, actual ground treatment results indicated the achievable JGP column diameter was closer to 1.8m rather than 1.2m.



Figure 11 Finished pedestrian underground linkway

4 MINED TUNNEL WITH SHALLOW COVER

A 11 m wide and 5.5 m high pedestrian linkway underpass had to be constructed to connect an underground subway station with an entrance structure on the other side of a major road. The ground conditions comprised of only 6m of overburden consisting of very soft ground materials, including soft marine and estuarine clays.

The excavation took place under a main road with significant day & night traffic load as well as major utilities and services, such as a 1.2 m water main, located at less than 500 mm above the excavation.

The linkway was originally planned to be excavated using a pre-installed box of pipe-jacked 800mm diameter steel pipe piles, very heavy temporary steel frame propping installed during excavation as well as extensive ground treatment. Pipe pile installation was anticipated to occur from the entrance side (right to left in figure below), whereas the linkway excavation for programming reasons had to occur from the station side (left to right in figure below).

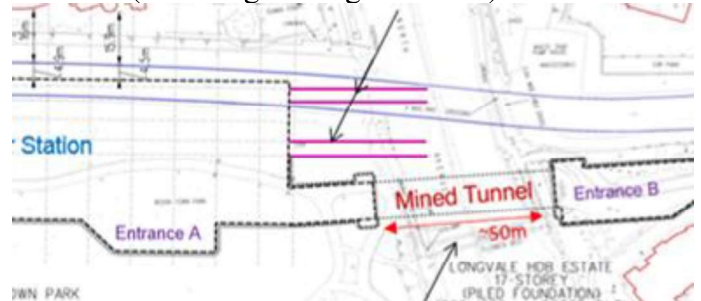


Figure 12 Layout Plan of the Mined Tunnel.

After award, however, it was realized that the construction method had to be changed, as the anticipated construction directions could not be maintained as planned due to the fact that the installation of the pipe piles from the entrance side finally proved to be significantly more time consuming than originally anticipated.

As a consequence the completion date of the entrance was pushed onto the critical path of the entire project, thus delaying the completion of the entrance beyond the completion date for the station.

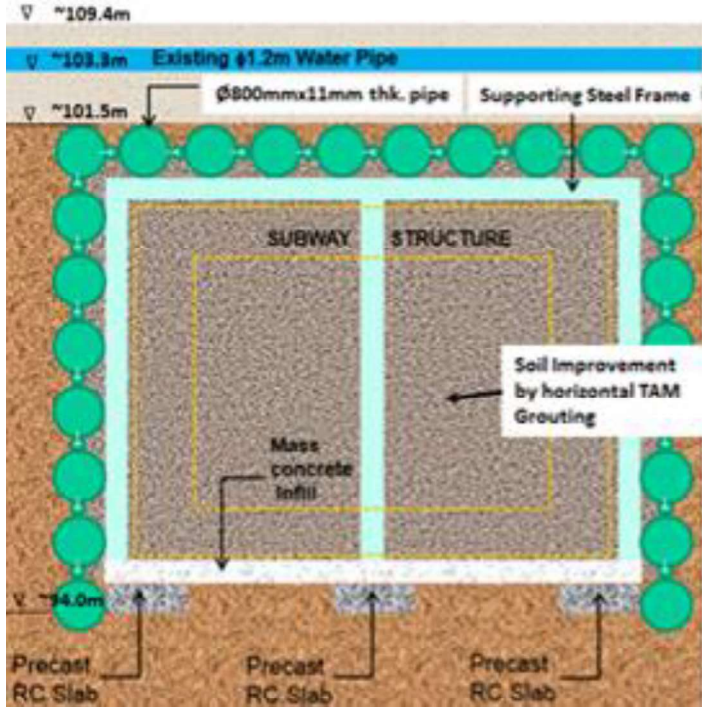


Figure 13 Original Construction Method.

Therefore, all construction activities for the linkway underpass now had to occur faster and all from the station side, resulting in the need for an alternative construction method, which included a mined solution based on the omission of the steel pipe piles and replacing them by a mined tunnel with heavy drilled pipe roof, shotcrete support.

The original construction and ground support method using a rectangular pipe pile box with support frames was abandoned and instead changed to a mined tunnel with sequential excavation and temporary sprayed concrete lining (SCL). The excavation was staged into first and second side drift and, within a side drift, into top heading and bench/invert excavation staging. The ground support lining consisted of sprayed concrete with 2 layers of wire mesh reinforcement and a double layer heavy drilled pipe roof (139mm diameter) for advance crown and face support.

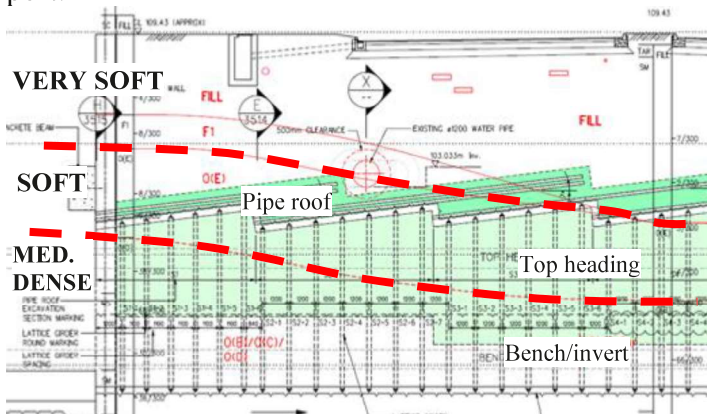


Figure 14 Alternative Construction Method – Mined SCL Tunnel.

Owing to the significant width of the linkway (11m wide) in relation to the relatively shallow overburden of only about 6m, the excavation had to be carried out in one side drift first, followed by part-installation of the permanent lining structure with temporary support steel props within the first side drift. Only then, excavation of the second side drift could follow. After completion of the second side drift, the permanent RC structure was closed and the temporary steel props removed.

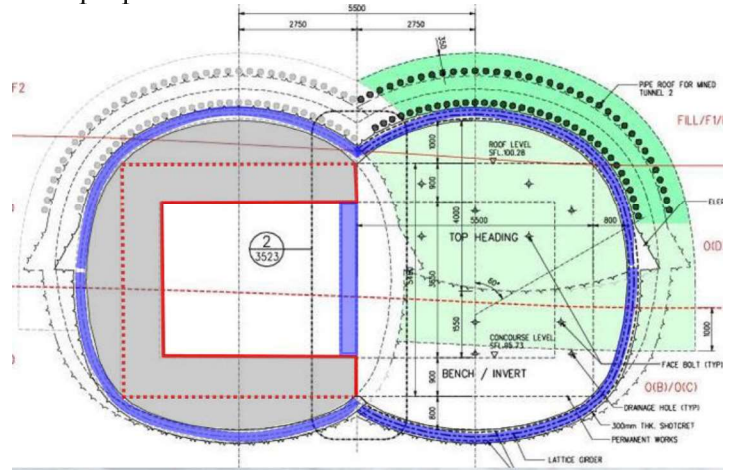


Figure 15 Side drift arrangement.

Sub-divided into 7 excavation sections, a total of more than 600 pipe roof pipes were installed with total length of more than 5000 m.

Drilling of pipe roof pipes and the GFRP face grouting pipes was carried out with a Pacchiosi drill rig, where the entire drilling boom can be rotated around its longitudinal axis.

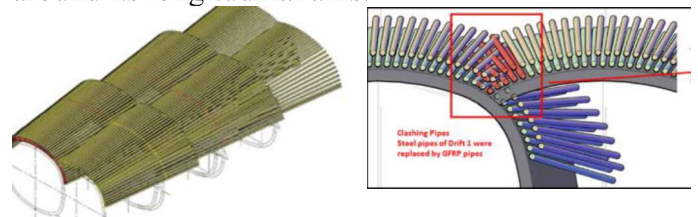


Figure 16 Drilled pipe roof with details.

Before drilling of pipe roof in each section, horizontal TAM grouting was carried out.

The 1.2m water main located at only 0.5m proximity to the excavation was underpassed successfully, causing only very small deformations to the steel pipe. The recorded deformations of the water main were in the range of 10mm and less.

Although, the experienced settlements on the road surface generally stayed within the expected levels, when the excavation face of the first drift reached to the center of the road, the settlements started to exceed expectations by about 15%. This was due to the fact that, from that location on, the interface between the soft soil and the harder and cemented material was lower than expected. This resulted in the soft soil materials taking up a large proportion of the excavation face.



Figure 17 Excavation completed. Permanent works completed on the left side.

The construction of the linkway was completed successfully and achieving the objective of removing the entrance completion from the critical path in the project's overall construction program.

5 EVALUATION OF MONITORING RESULTS

In urban environments, underground infrastructure projects are frequently located in close proximity to sensitive structures such as roads, viaducts & piers, utilities & services for which the construction impact is assessed and controlled based on monitoring results. Although this is principally the correct way of dealing with the control of inherently risky construction activities, in many cases the process is executed "blindly" and very "rigidly", where the raw monitoring results, such as time histories are compared, "blindly" and "rigidly", to respective control values, disregarding how the deformations actually have occurred over time and continue to develop.

An important tool in this process is to derive and evaluate influence lines and trendlines and not only time histories. Therefore, evaluation procedures also considering trendlines, rather than time histories only, are considered much more appropriate, allowing a better and more accurate assessment of the ongoing conditions.

In the following, monitoring data is used taken from a tunnel excavation in difficult ground conditions involving mixed ground (rock and soft residual soil). The tunnel excavation started from a deep shaft and connected to a large diameter sewer tunnel about 85m away.

Although, the ground conditions consisted of rock and weathered rock at tunnel level, at some distance from the shaft, residual soil reached down into the tunnel crown and almost the entire overburden consisted of soft and completely decomposed residual soil material with SPT- N_{300} values of 15 and less.

As an additional complication the excavation was carried out towards a large diameter high-pressure gas pipe line. Therefore, full control over the deformation

generated during advancing the tunnel excavation was adamant.

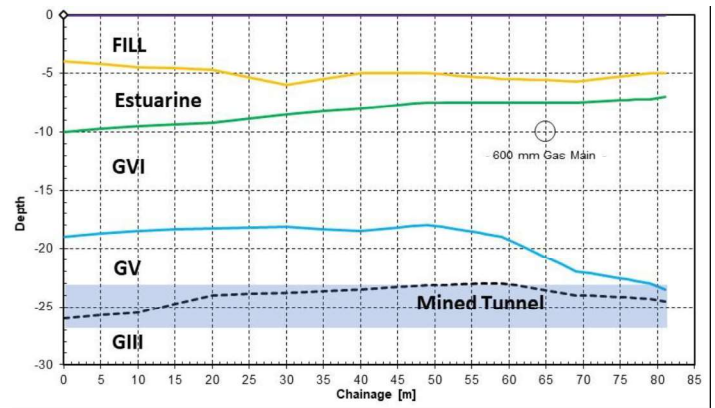


Figure 18 Ground conditions.

The excavation started at the shaft in stiff residual soil and then entered into almost full-face rock conditions. However, just before the position of the gas pipeline, the rock head dropped, and soft residual soil materials appeared in the excavation face.



Figure 19 Tunnel excavation.

During tunnel excavation, settlement points placed along the tunnel axis will start to show settlements when the excavation face reaches a certain proximity to the specific location. After this point in time, settlements will gradually increase with the face approaching further and passing the particular monitoring point. The maximum settlement will be reached when the tunnel's excavation zone has left the influence zone for this particular monitoring location.

It is common knowledge that in normal ground and excavation conditions, about 50% of soil deformations have already occurred, before the excavation face actually reaches to this location.

When approaching a change in ground conditions, where the excavation is subjected to larger deformation, 50% of the total final deformation also amount to a larger absolute value. This fact can be used to visualize monitoring data appropriately, such that any changes in behavior can be identified much earlier as it would be by just evaluating time history

plots only or convergence monitoring results from within the excavation.

Monitoring data can be processed by plotting the settlements measured along the tunnel alignment at regular points in time, typically daily. This yields a set of lines, which spread out only in the area of influence. Otherwise, they are sitting above each other. They will have a value of zero, when sufficiently far ahead of the face or do not change anymore showing the maximum settlement value, when already sufficiently far behind the face. Specifically identifying the settlement measured at the time, when the excavation face is passing underneath this location and connecting these points with a line yields a line, which is in the following referred to as the trendline.

In a situation with steady ground response as the tunnel advances, the trendline will appear in the diagram as a horizontal line. If deformation are getting worse with increasing advance, the trendline will dive down. On the other hand, if deformations improve, the trendline will show an upwards trend.

Taking monitoring point SS-6108 from the data set of the mentioned tunnel example, the corresponding point on the trendline will appear on 27 June, the day when the excavation face passed this location (ch20m). Point SS-6122 (ch45m) will plot on the trendline on 19 July, respectively.

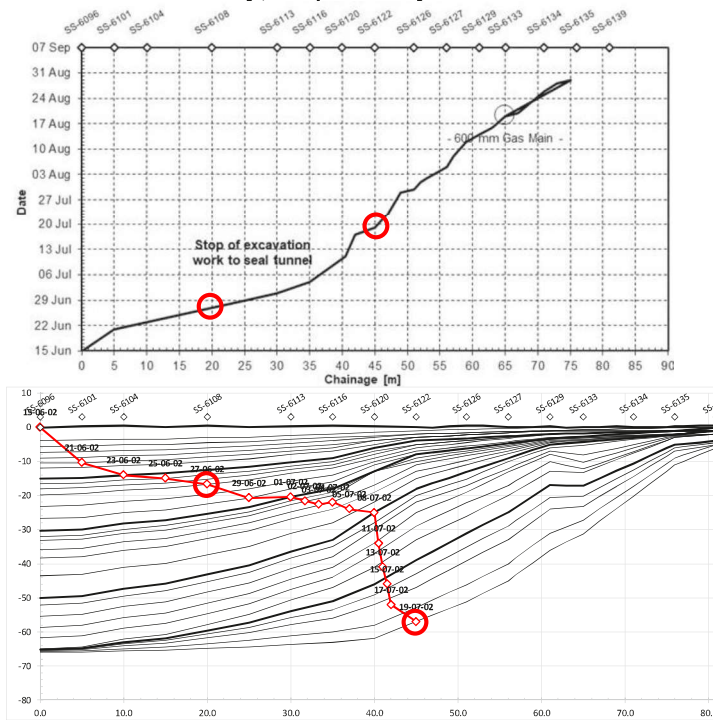


Figure 20 Construction progress (top) and Trendline plot produced on 19 July.

The following figure shows the recorded time histories of a number of monitoring points installed along the tunnel center line. In this diagram, time is plotting along the x-axis and settlement along the y-axis, respectively.

In the trendline plot, however, the x-axis shows chainage and the y-axis shows again settlement. As explained above, the trendline connects the settlement

points, when the tunnel's excavation face passed right below the respective point.

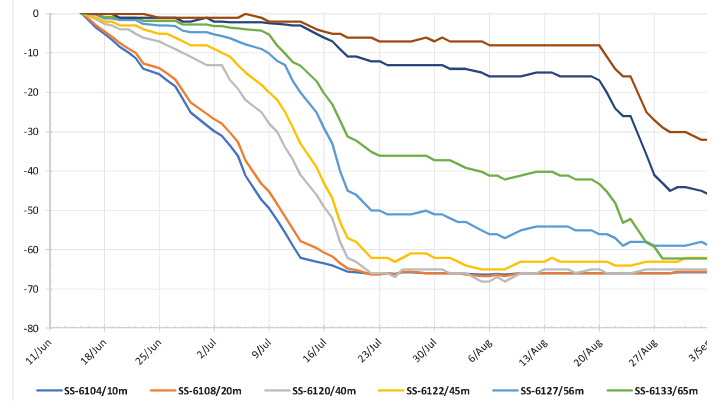


Figure 21 Settlement time history plots.

When comparing certain points in time and marking them on both the time history and the trend line it becomes evident that the trendline plot is able to provide a clearer picture for judgement. Time history plots at that the particular points in time did not show any confirmed indication of behavioral changes, which in the trendline plot was already clearly visible.

The following figure presents the monitoring results after completion of the tunnel excavation in both formats, time history as well as trendline.

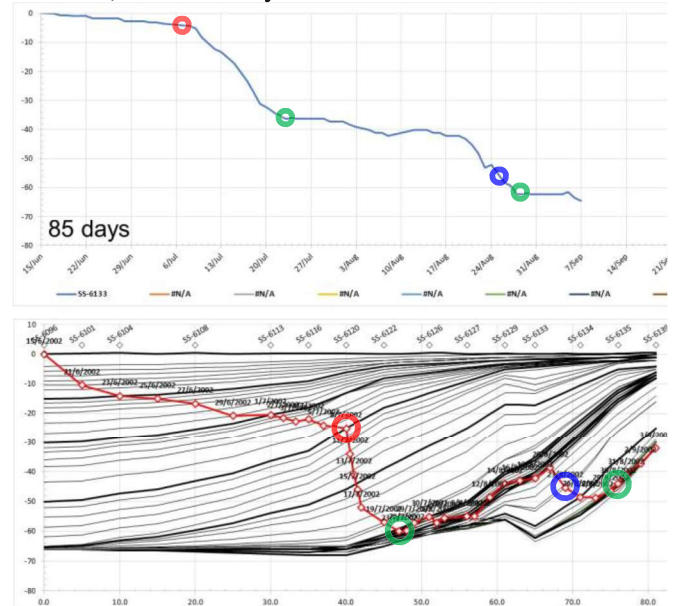


Figure 22 Time history versus Trendline plot.

The markings relate to the same points in time. The trend line diagram already shows more than one point confirming a particular trend change, whereas the time history, at best, just indicates a change in one monitoring result.

Despite the fact that, what was proposed above, is nothing new and has been practice in other parts of the world for quite some time, it is the authors experience that such assessments are getting disregarded or neglected quite often. Therefore, the basic schematic and processes involved in such a procedure are explained and presented below and practitioners are encouraged to use and develop them further.