DESIGN AND CONSTRUCTION OF A LRT TUNNEL IN KUALA LUMPUR, MALAYSIA

by
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1) INTRODUCTION

The recently completed Light Rail Transit (LRT) System in the capital of Malaysia and its surrounding area has complimented the comprehensive rail based public transport need of the public in and around Kuala Lumpur (KL). The layout of rail based public transport system is shown in Fig. 1. Most of the LRT are at grade except for a 4.4km of bored twin tunnels of 5m diameter with five underground stations in the heart of Kuala Lumpur. 60% of the tunnel is below the Klang River. This paper describes some issues on design and construction of the tunnel highlighting the influence of geological and geotechnical aspects. Only the design and construction of 1.8km tunnel from Masjid Jamek Station to Kg. Baru Station are covered in this paper.

2) GEOLOGY

Fig. 1 also shows the three main geological formations over the rail based transport network. Kuala Lumpur Limestone is the second oldest rock formation of upper Silurian age. Most of the Kuala Lumpur Limestone has been metamorphosed into marble. Kuala Lumpur Limestone are notorious for its karstic features such as pinnacles, cavities and these features post tremendous challenges to the engineers (Gue, S. S. [1999], Neoh, C. A. [1997], Yeap E. B. [1985] and Ting, W.H. [1974]).

The Kenny Hill Formation consists of interbedded shales and sandstone of upper Silurian-Devonian age and lies uncomformibly over the Kuala Lumpur Limestone. It exists as broad synclinal belt generally 7km to 10km wide, running from Kuala Lumpur city southward through the suburbs of Petaling Jaya and further to the south for at least 30km. The Kenny Hill sedimentary rocks have been regionally metamorphosed to form metasediments. The degree of metamorphism of Kenny Hill Formation varies regionally. As such under low grade metamorphism Quartzites (from sandstone) and Phyllite (from the shales) have evolved. On the other hand, Schist (from shales) has evolved under higher grades of metamorphism.

The intrusion of Lower of Middle Mesozoic granite rock resulted in the deformation and metamorphism of the older rock.

Granite rock occupies a large hilly undulating area around Kuala Lumpur except in the south. The texture and composition of granitic rock range from coarse to very grained with some slightly porphiritic biotite granite.

Alluvial deposits mainly consisting of loose silty sand and gravels were deposited during the Quarternary period over the low lying areas forming river plains. Extensive tin mining activities in the past had also contributed to the vast but random distribution of these deposits.

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3) SUBSOIL PROFILE

The preliminary ground profiles for the section includes 120 boreholes and 50 seismic survey near the centre lines of the twin tunnels. Ground investigation on the alignment is difficult and has potential risk during tunnelling as 60% of the tunnel is below the Klang River. In addition, the presence of utilities with many of them laid during the pre-independence days with limited records. Thus, seismic survey was added extensively.
Additional ground investigation was carried out during the detailed design and construction. It consists of 70 number of boreholes, six seismic survey profiles and two down the hole seismic profiling.

These boreholes were undertaken using conventional split spoon sampler, NMLC core barrel and Mazier triple tube core barrel. Insitu field testing such as pressumeter and permeability tests were carried out at selected depths and laboratory tests were also performed on disturbed and undisturbed soil samples collected from the boreholes to determine their geotechnical properties.

Although the available geological map does not indicate the presence of limestone along the route, the detailed ground investigation have encountered limestone pinnacle 33m below the ground level at the West temporary shaft. Limestone was also encountered at the Kg. Baru Station below the tunnel level. A very strong rock; 'Skarn' (Cal-silicate) having a compressive strength of approximately 270 mPa was also encountered at two locations of the tunnel alignment

The results of the Standard Penetration Test (SPT) N-values near the interface between Kenny Hill and Limestone Formation exhibit low SPT N-values. This weak zone known as "Slump Zone" is a common feature in limestone formation.

The profile of the subsoil is shown in Fig. 2. It was anticipated that tunnelling would generally encountered two distinct zones of materials. These are the highly to completed weathered Kenny Hill Formation and the top layer of loose to dense alluvium deposits. A section of the tunnel has only 4m of the overburden soil below the riverbed and some areas the maximum depth of overburden is 14m. Isolated limestone pinnacles was also encountered during the tunnelling through this section.

3) DESIGN

This section highlights the typical geotechnical designs of a temporary shaft and the tunnel section.

<u>Temporary Shaft</u>

Typical internally strutted secant pile wall is shown in Fig. 3 with aluminium reinforcement at the tunnel section for ease of drilling. The soil parameters selected for the design of a temporary shaft is shown in Table 1. Two sets of soil were adopted in the analyses. One set is known as the moderately conservative soil parameters and the other is known as the worst credible soil parameters. A lower K_o value was adopted for the shaft design. This is because the allowable movement for the shaft is much larger than the tunnel as indicated by a number of case histories of similar soil in KL area.

The factors of safety adopted for the computed strut forces, bending moments and shear forces for various designs are as follows:-

Type of Soil Parameters	**Computed Strut Force, B Moment & Shear Force for Structural Design**
Moderately Conservative	1.4 / 1.2
Worst Credible	1.1

The partial safety factors adopted for the materials used are as follows:-

Concrete	**Reinforcement Bar**	**Strut Steel (Grade 43EE / 50EE)**
1.5	1.15	1.6 / 1.4

The normal water level which is about 3m below the ground level was used for the design and a check was also done for water level at ground level because of the previous incident of flash flood in the area. The Finite element analyses were also carried out for the design and to check for deformation of retaining ground. Well points at the base was also provided to relieve uplift pressure to ensure stability during flooding. The toe displacement was limited to 50mm and the distortion of the wall at 1:150.

<u>The Tunnel</u>

The general design criteria on clear zone as shown in Fig. 4 were used and the typical designs section are shown in Fig. 5 and the soil parameters derived from the boreholes and in-situ pressuremeter tests are shown in Table 2. The load diagram for the tunnel in use is shown in Fig 6 and the results of finite element analyses of the tunnel are shown in Fig. 7. Additional pressure during construction such as advance of shield tunnel machine as shown in Fig. 7b was included and some results of typical finite element analyses are shown in Fig. 7c. In areas near the adjacent buildings, the pressure induces by buildings were also considered as shown in Fig. 7d.

In addition, the tunnel was also checked against floatation particularly in an area with shallow overburden as indicated in Fig. 8 and it has a factor of safety of larger than 1.2 after omitting the surcharge load.

The enclosed type mechanical shield machine (Earth Pressure Balance Machine) was selected for its performance of the safe and accurate tunnelling work under the congested urban area and its soil conditions.

4) CONSTRUCTION ISSUES

Three construction issues are elaborated, namely i) the collapse of thin sandy alluvium over the weathered Kenny Hill during the driving of the tunnel ii) the ingress of high seepage of water through the pinnacles outcrop during the excavation of a temporary shaft and iii) the encountering of extremely strong rock; ‘Skarn’ having a compressive strength in excess of 270 mPa.

i) Collapse of thin sandy alluvium over the Weathered Kenny Hill during the driving of the tunnel.

Excessive settlement and sinkholes had occurred near Masjid Jamek Station during the driving of the tunnel. Fig. 9 shows the photos of the problem and Fig. 10 shows the illustration of the most probably causes of the occurrence. The settlement/sinkhole repeated even after extensive chemical grouting and patching from the surface.

The first occurrence of the settlement and sinkholes was due to the ingress of disturbed sandy soil with water into the tunnel. Pieces of rotten wood were also found in the overburden. The subsequent three occurrences were similar mainly due to the soil ingress through the ungrouted zone.

Chemical grouting using a solution mixture of potassium carbinate and sodium bicarbonate together with solution of sodium silicate was injected from both the surface and through the inside face of the tunnel by double tube drilling method under a pressure of 8 to 10 bars to stop further settlement occurrence and formation of the sinkholes.

ii) Ingress of high seepage of water

The west temporary shaft constructed to launch the tunneling machine encountered a large limestone pinnacle at 33m below ground level. Although the pinnacle was crystalline, having a compressive strength of about 80 mPa, it did not exhibit any opened joints. However, recemented relict joints or fused joints by brown iron oxide were present. The ingress of high seepage during the excavation of a temporary shaft was encountered probably due to the higher permeability of the weathered limestone of Grade II/III at the pinnacle outcrop. Attempts to seal the seepage by chemical grouting using double tube drilling seepage method were not successful. Hence, three submersible pumps of 150mm and 100mm were used to drain the water in the shaft throughout the construction at a seepage rate of about 100 litre per second.

iii) Encountering extremely strong rock

Extremely hard rock was encountered at two locations of the tunnel alignment for lengths of 15m and 30m even though ground investigations did not detect their presence. Based on the results of the ground investigation, it was generally anticipated that the compressive strength of Quartzite and Phyllite Kenny Hill Rock (Grades I and II) would be in the region of 50 – 100 mPa.

The extremely strong rock has been indentified as ‘Skarn’, a cal-silicate having a compressive strength in excess of 270mPa. The consequent of this has necessitated the change of cutter bits as shown in Fig. 11 and has caused considerable delay. In addition, the material discharge system was changed from pumping system to conveyor belt system. The extra overbreak has also increased the intake of grout behind the tunnel concrete linings (Muhinder Singh, 1999).

The tunnel construction details are shown in Fig. 12. On the whole, the progress was good except for a few locations that encountered difficult geological features mentioned above. The average progress rate of tunnelling was 8m per day and a maximum of 15m per day for a 24-hour day with three shifts.

5) CONCLUSIONS

Tunnelling in this urban area had been a challenge and especially working close to limestone formation. Many surprises were encountered even with comprehensive ground investigation due to the geological features such as pinnacles, sinkholes, cavities and slump zones and etc. Additional feature such as the very strong rock; Skarn, which has an unconfined compressive strength of about 300mPa posed additional difficulties to the tunnelling. Working in limestone formation and its surrounding area, there is always a need to change equipment and as well as increase in the time of construction when some of the features mentioned above are encountered.

There are many problems and uncertainties related to design and construction of tunnels. Thus flexible attitude to the design and construction is always necessary. Sometimes, the construction requires a combination of tunnelling methods for ground of different geology and geotechnical properties. Hence, it is necessary to have contingency plans for the unforeseen. Proper planning and interpretation of ground investigations can only reduce the risk, cost and time of construction.

6) ACKNOWLEDGEMENTS

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7) REFERENCE

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GEOTECHNICAL
PERUNDING

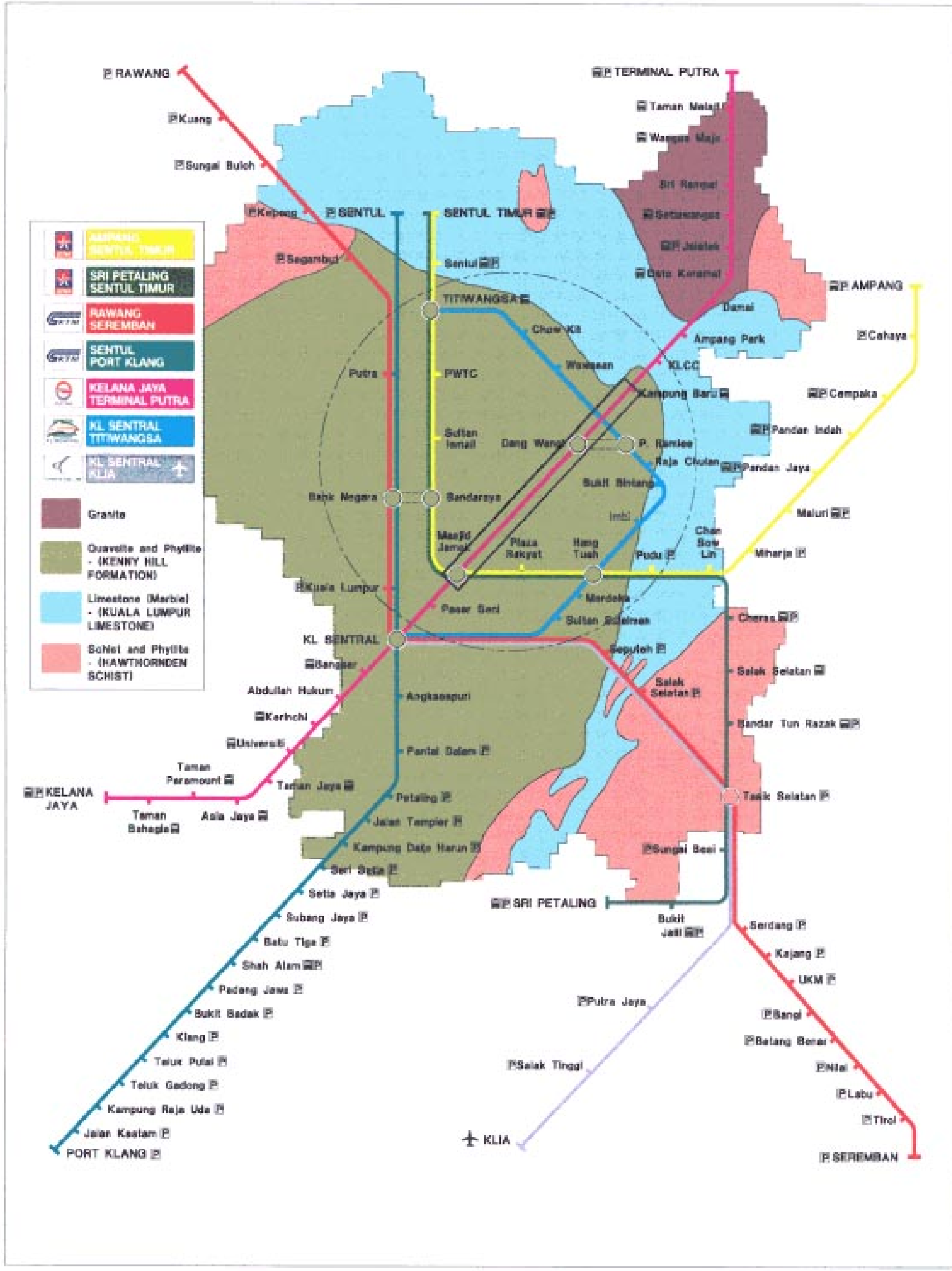


FIG. 1 RAIL BASED PUBLIC TRANSPORT SYSTEM

GUE & PARTNERS SDN BHD

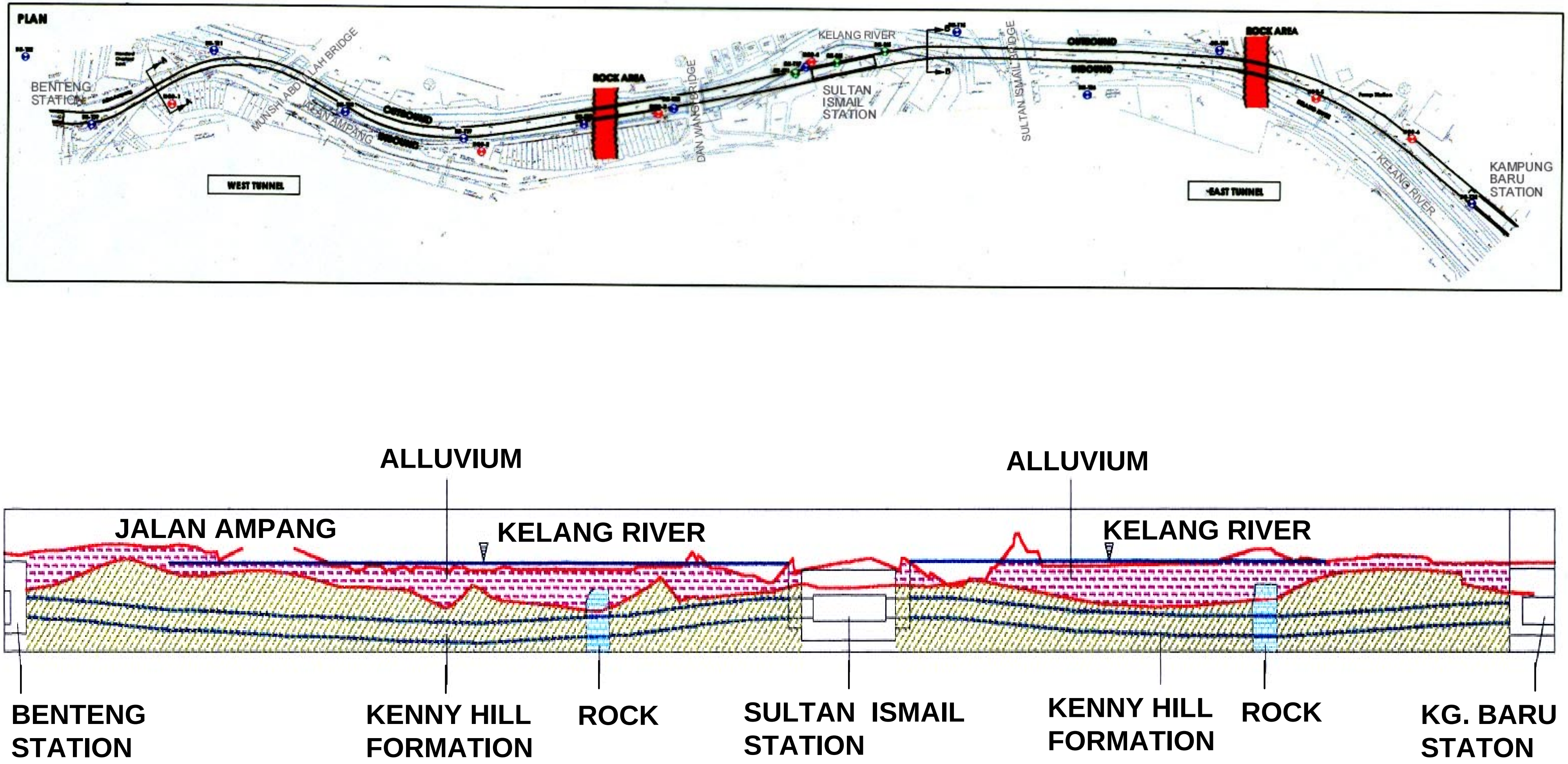


FIG. 2 SUBSOIL PROFILE

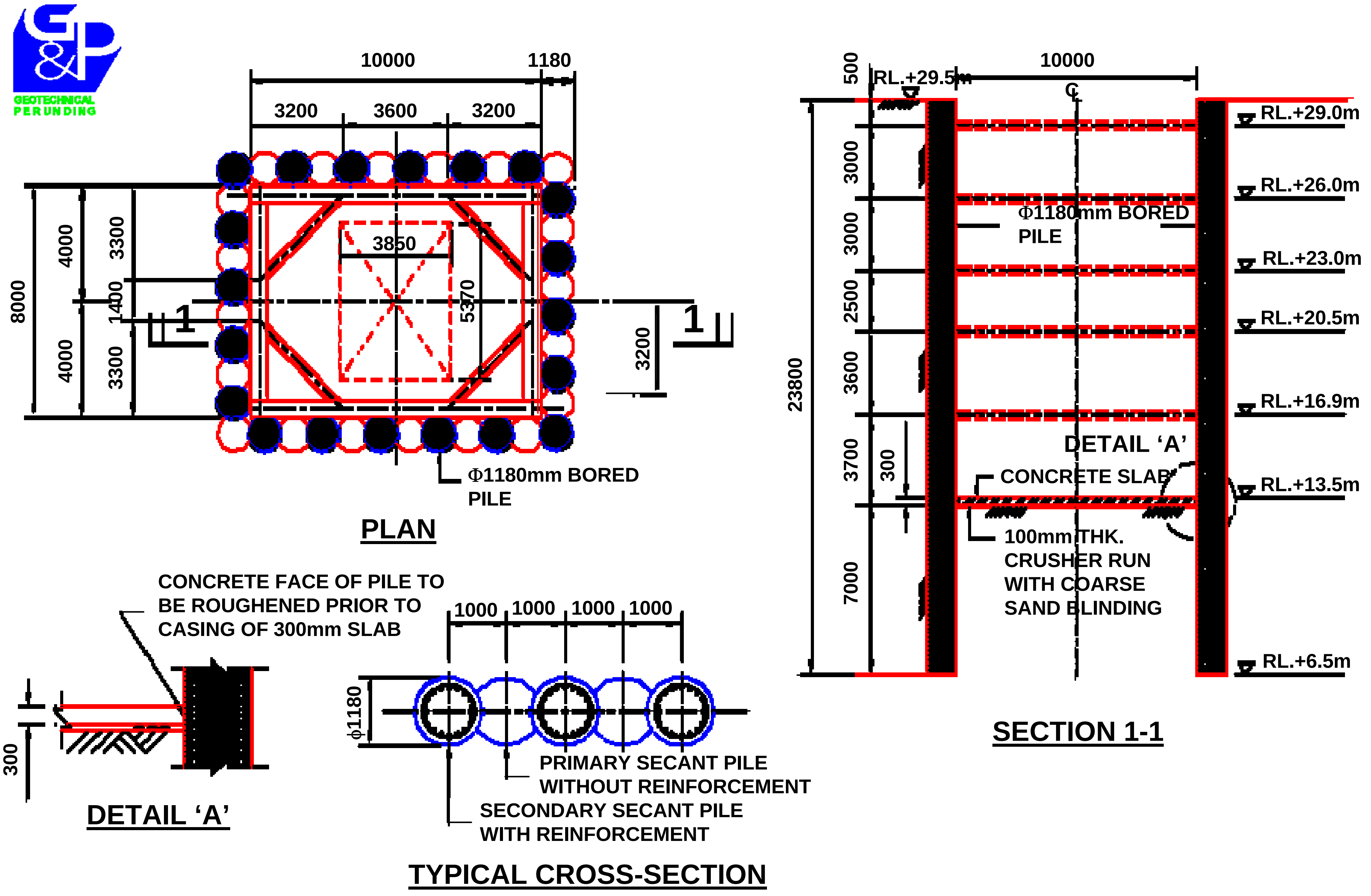
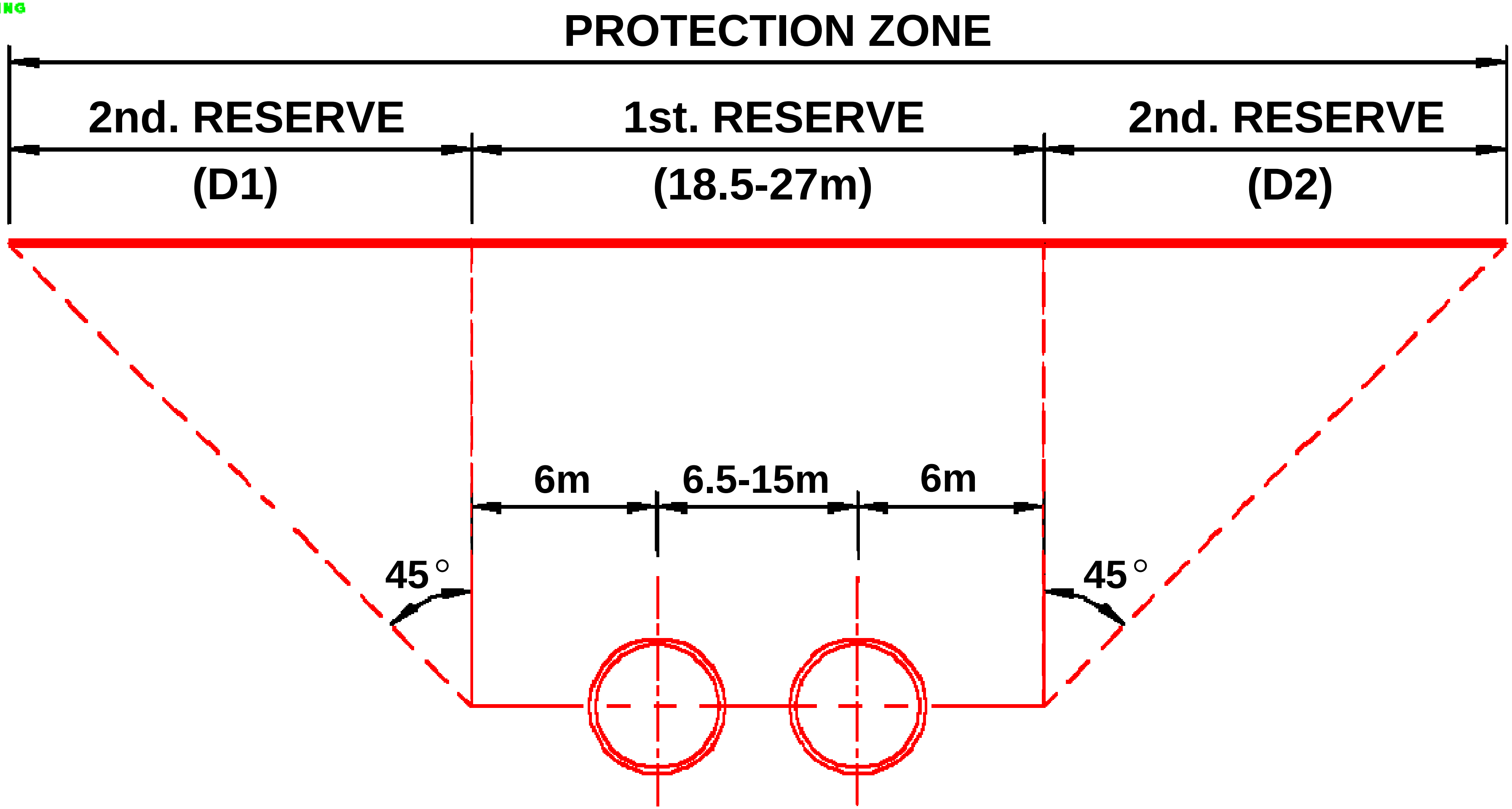


FIG. 3 TYPICAL DESIGN OF THE TEMPORARY SHAFT



NOTES :

1st. RESERVE : RESTRICTIONS ON PILING, EXCAVATION, BLASTING, GROUND ANCHORS AND GROUTING

2nd. RESERVE : CONSTRUCTION TO BE AGREED BUT GENERALLY PERMITTED

FIG. 4 TYPICAL DESIGN CRITERIA OF “CLEAR ZONE”

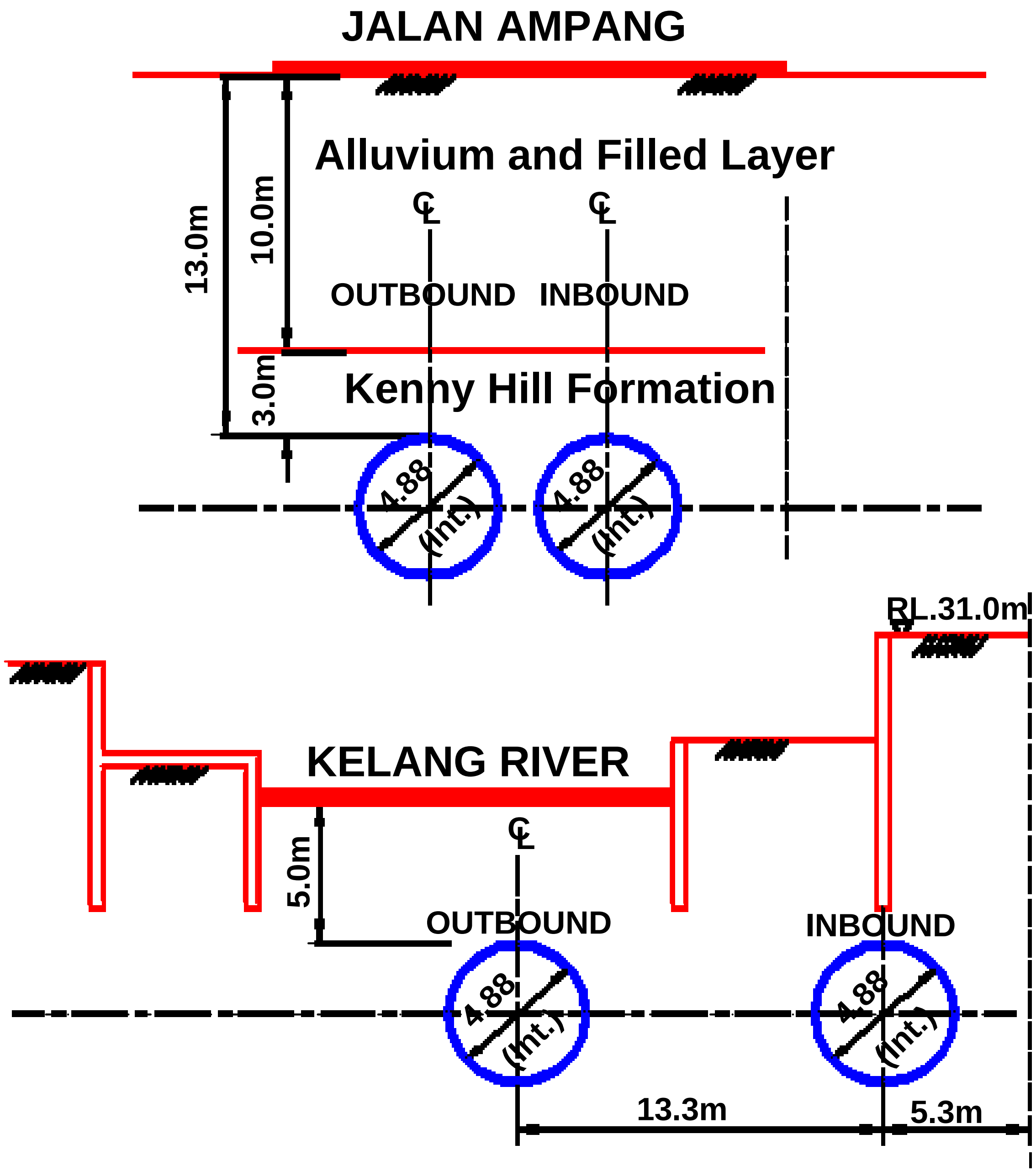
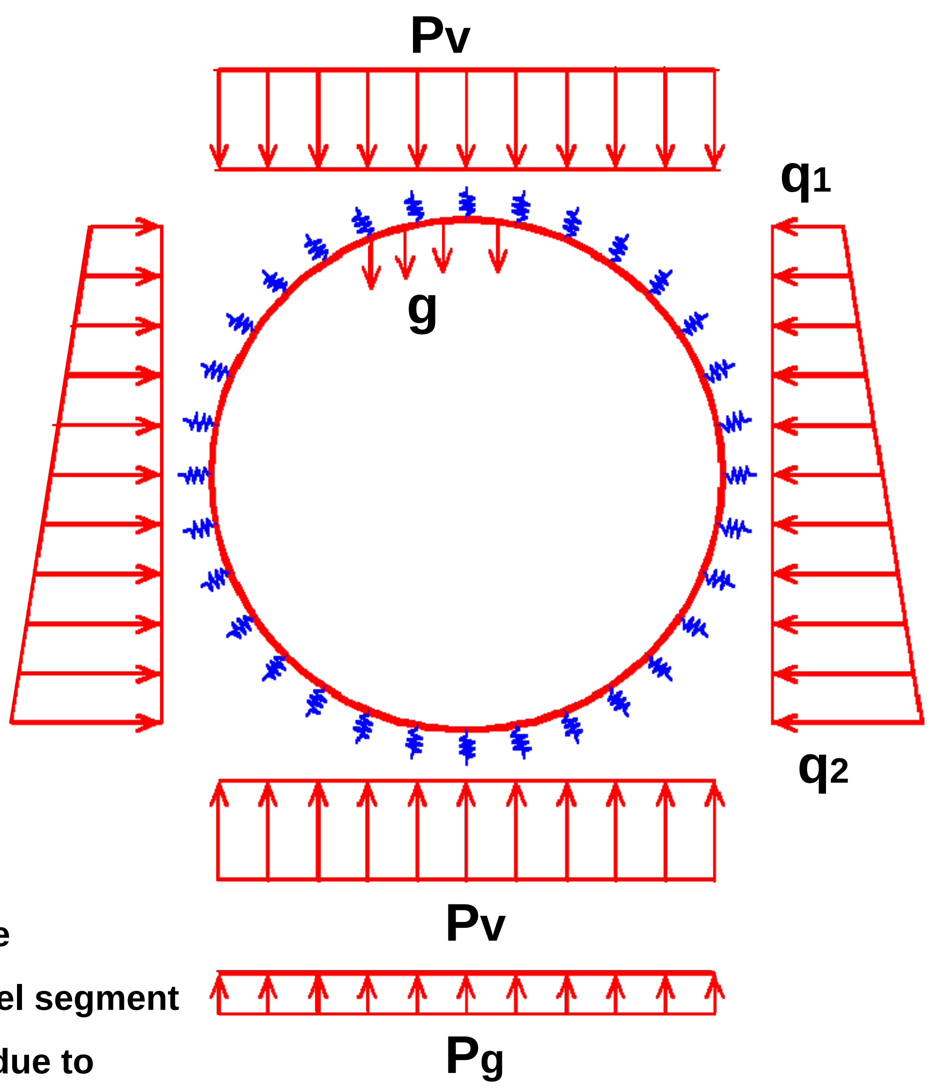


FIG. 5 TYPICAL DESIGN SECTIONS



Notes

Pv = vertical pressure

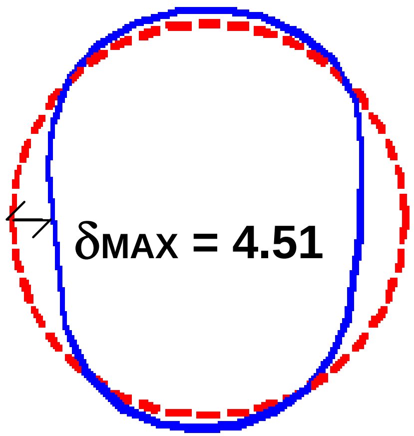
q = horizontal pressure

g = self weight of tunnel segment

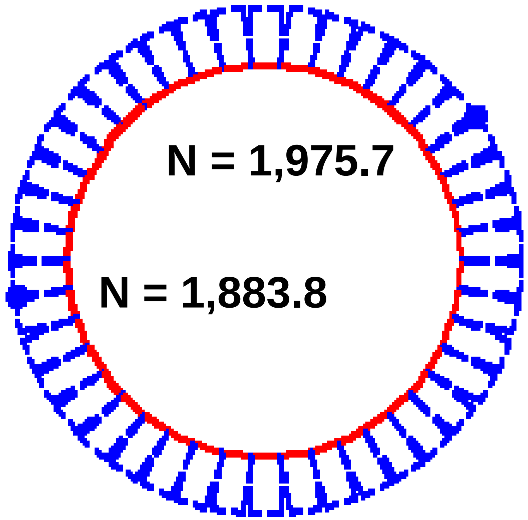
Pg = reaction pressure due to self weight of tunnel segment

FIG. 6 SUMMARY OF LOADING (For Tunnel In Use)

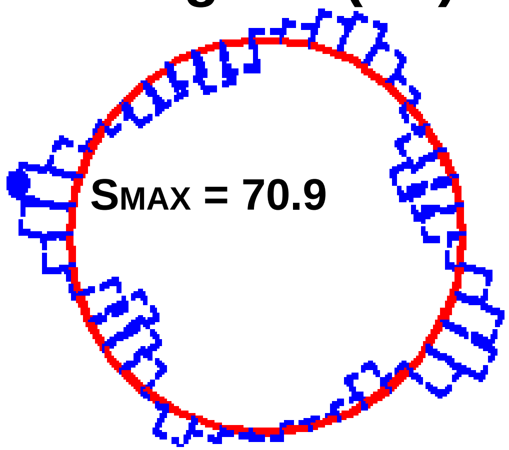
Displacement diagram (mm)



Axial force diagram (kN)



Shear force diagram (kN)



Bending moment diagram (kN-m)

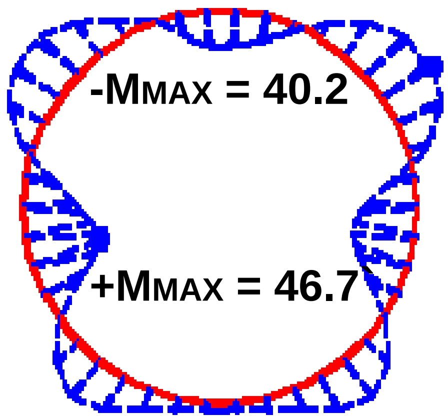


FIG. 7a RESULTS OF ANALYSIS FOR A SECTION THE TUNNEL (In Use)

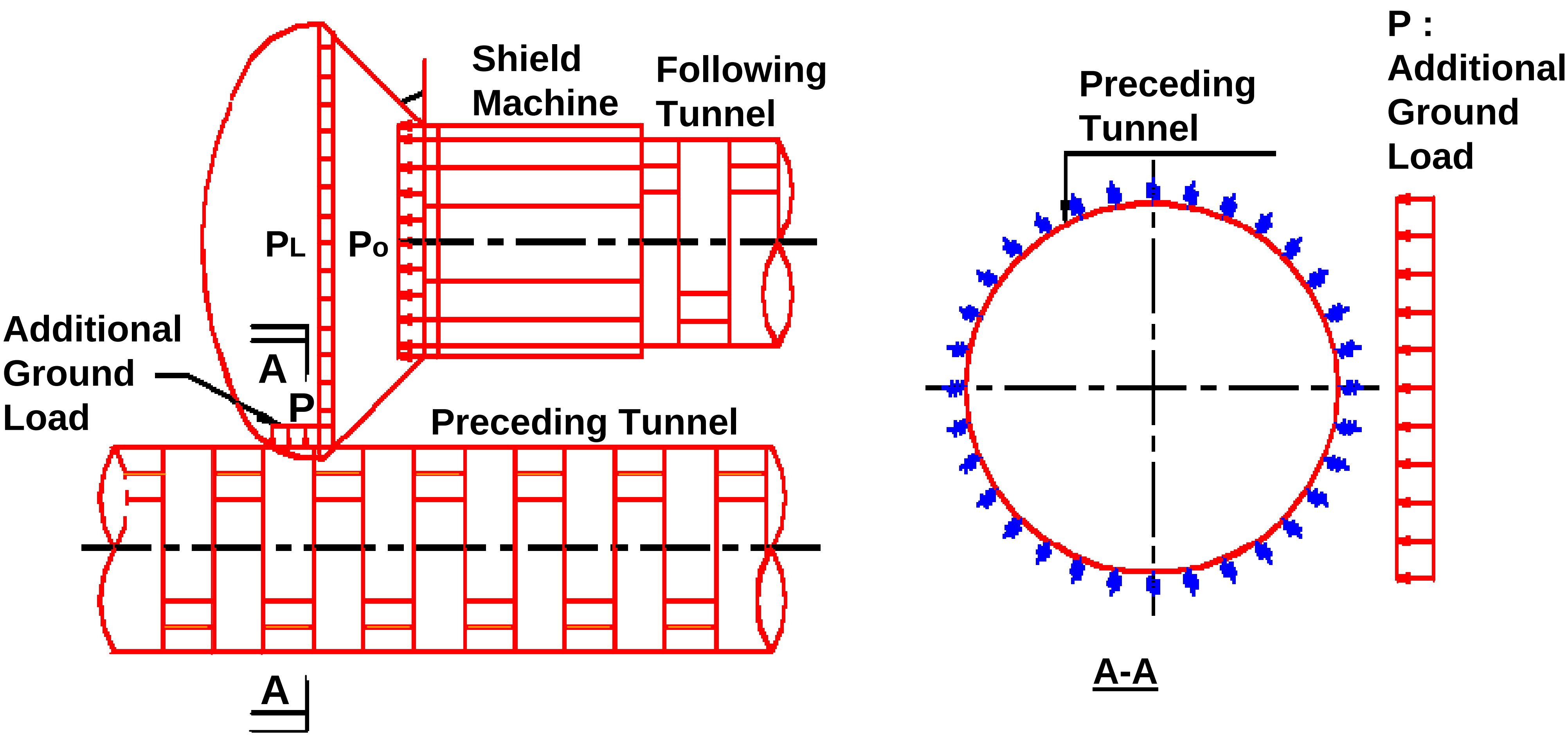


FIG. 7b ADDITIONAL GROUND LOAD DURING CONSTRUCTION

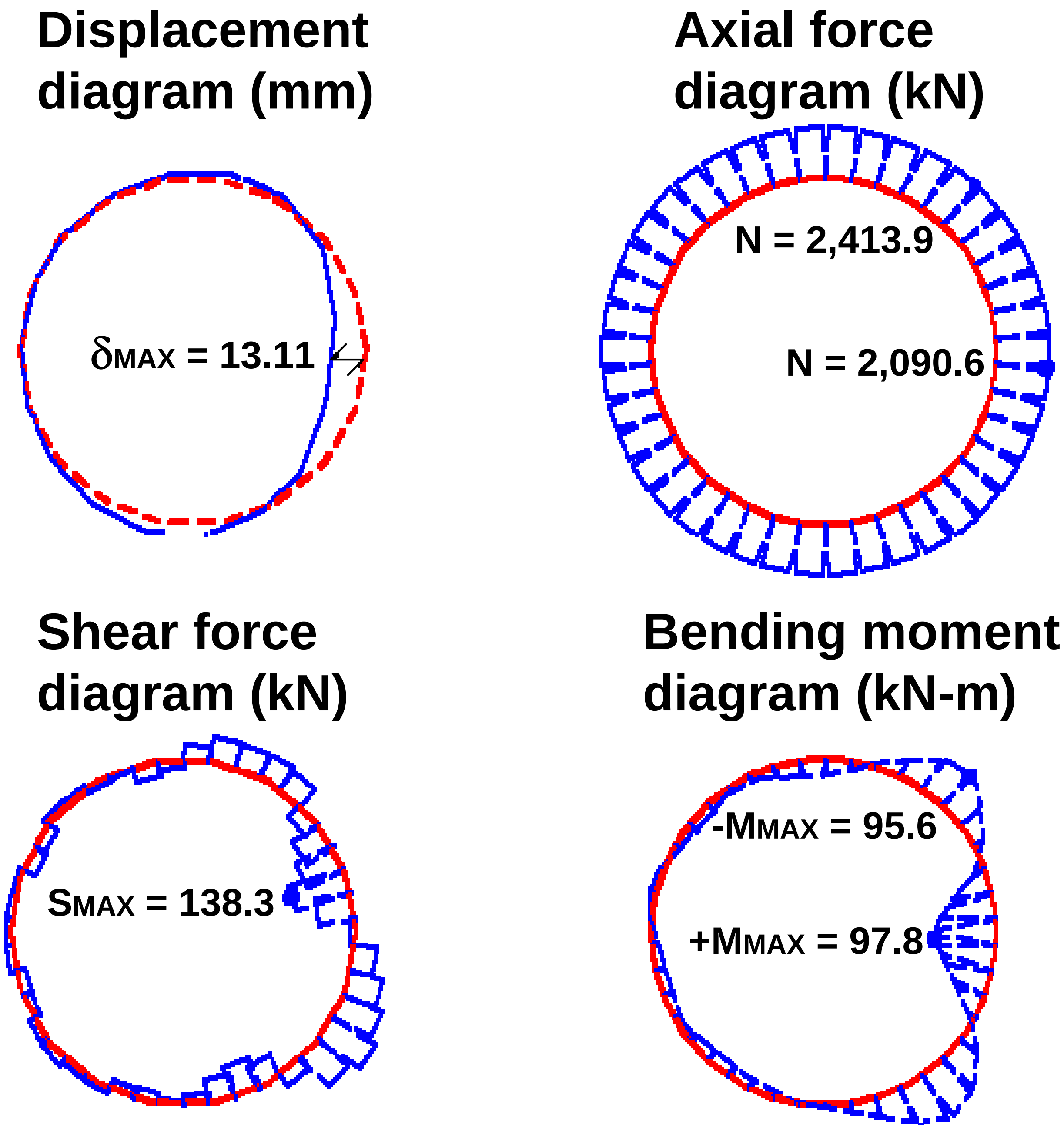


FIG. 7c RESULTS OF AN ANALYSIS DURING CONSTRUCTION FOR A SECTION OF A TUNNEL

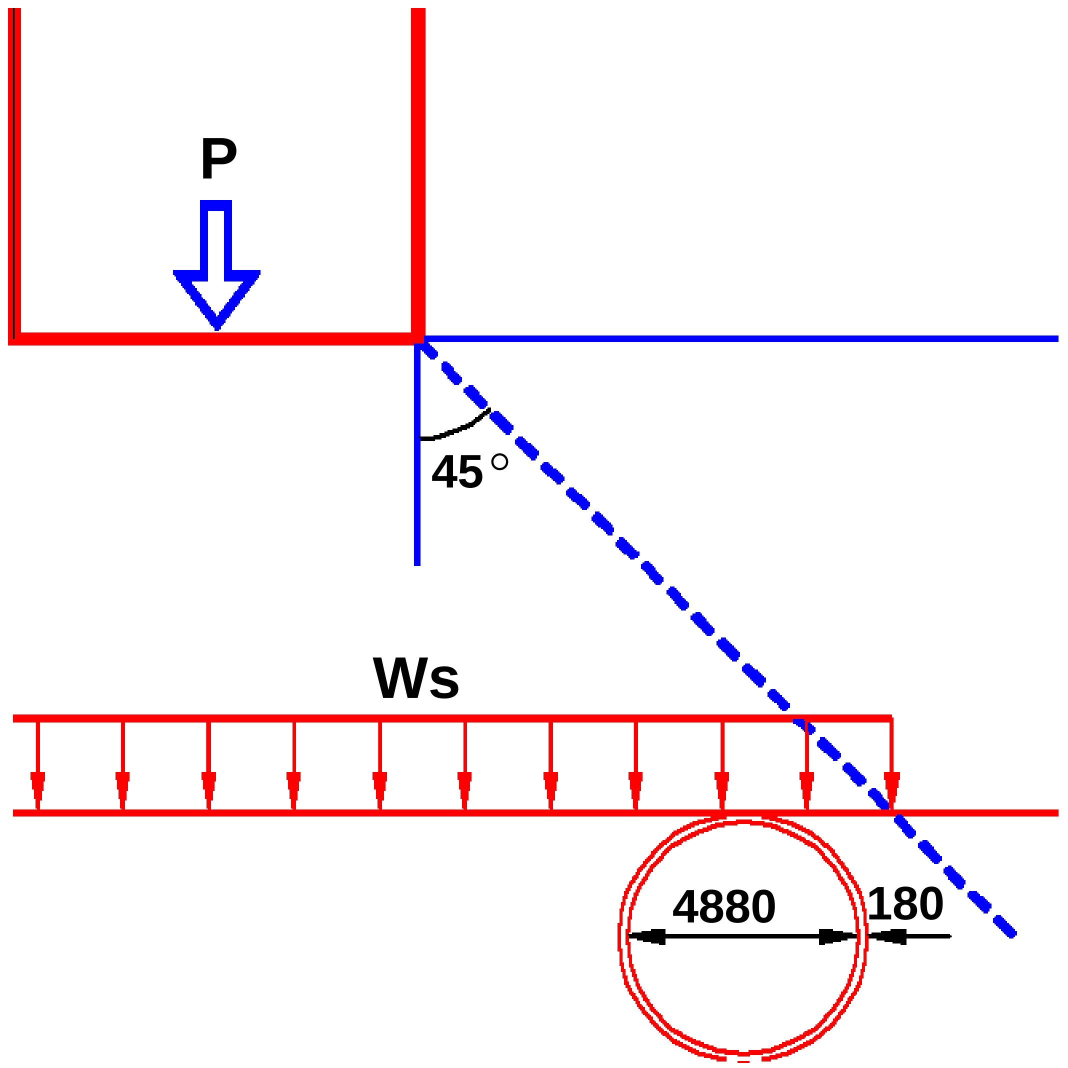


FIG. 7d ADDITIONAL PRESSURE DUE TO BUILDING LOAD

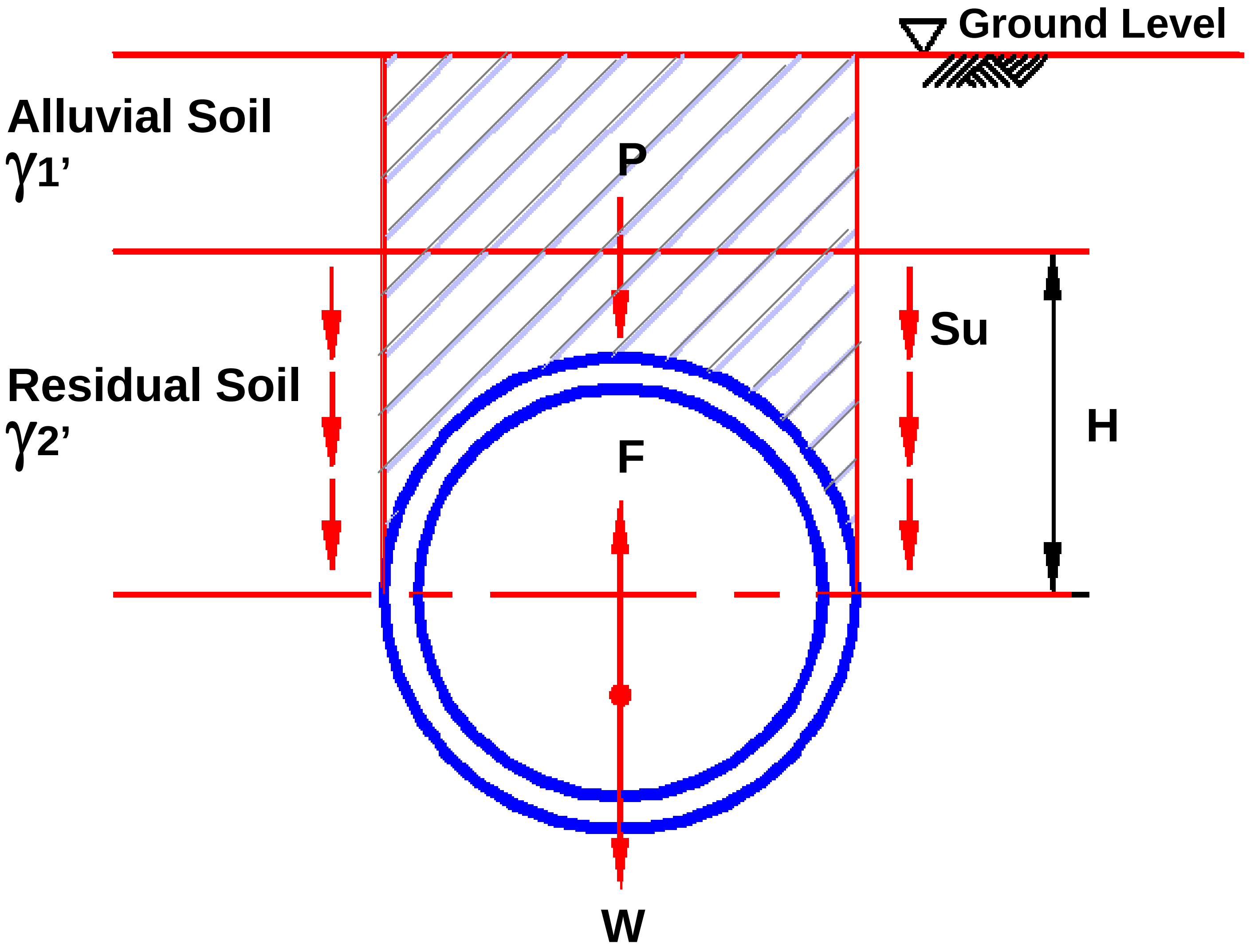


FIG. 8 A CHECK AGAINST FLOATATION (TYPICAL)

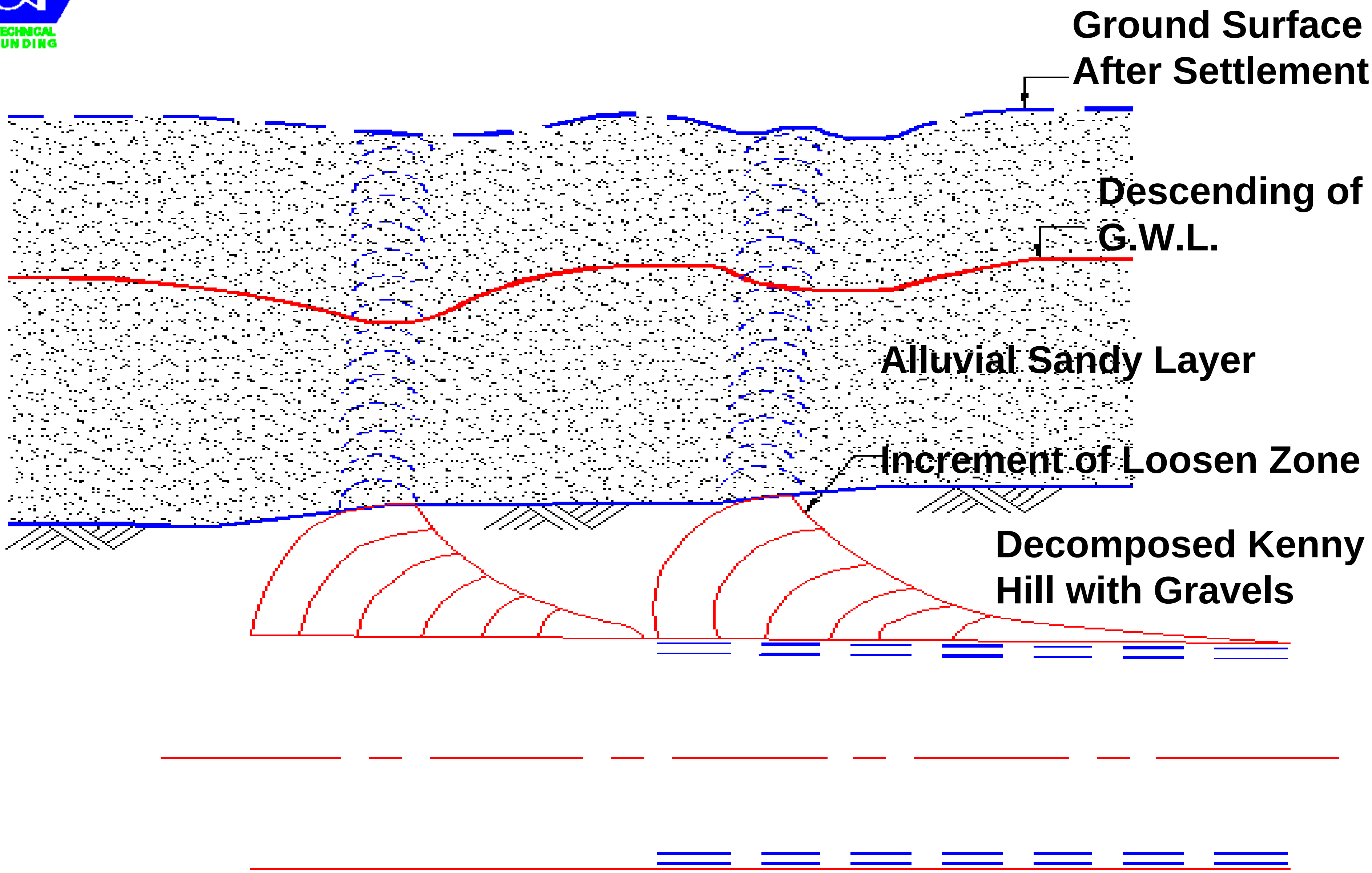


FIG. 10 ILLUSTRATION ON SETTLEMENT AND SINKHOLE OCCURRENCES

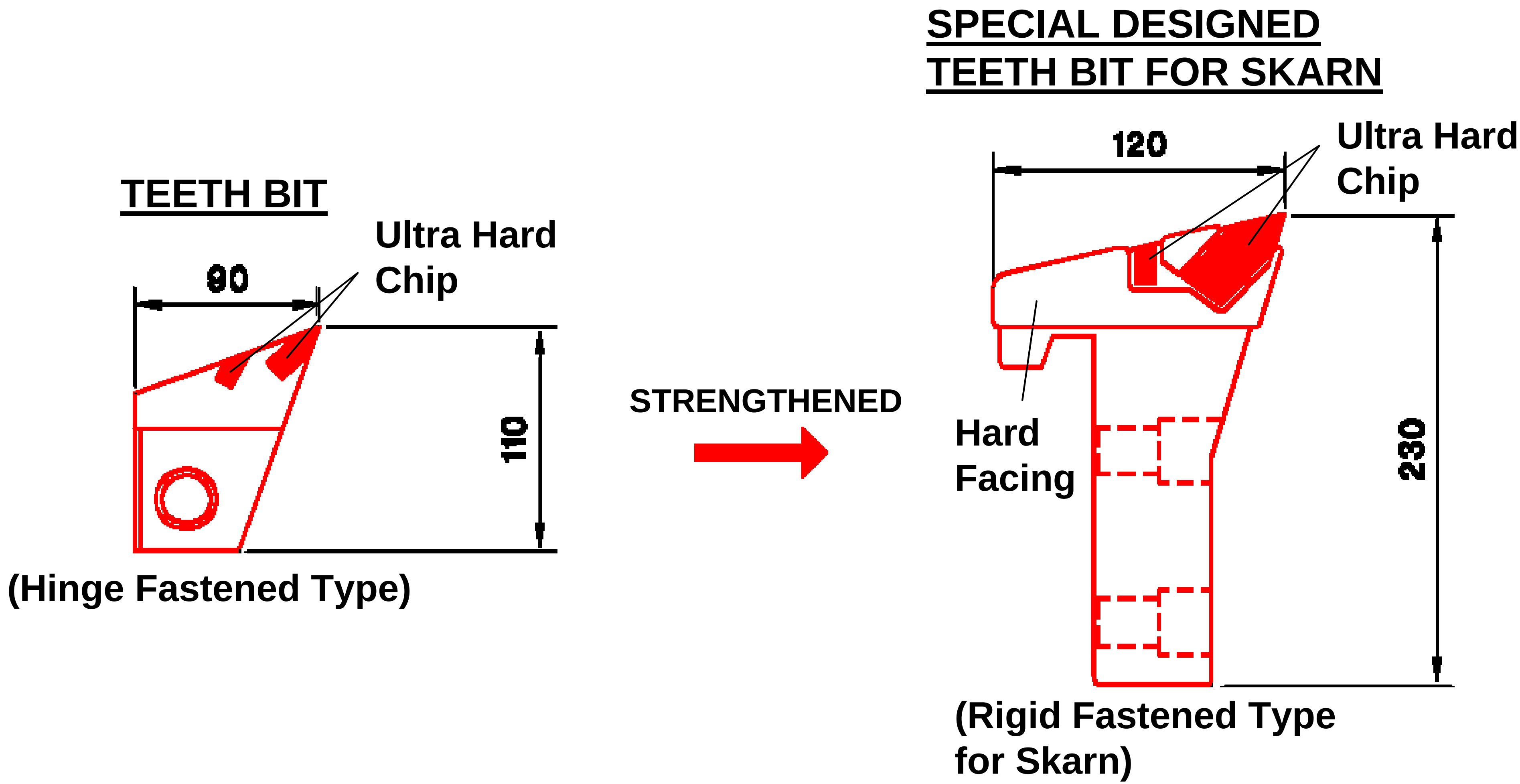


FIG. 11 CUTTER BITS

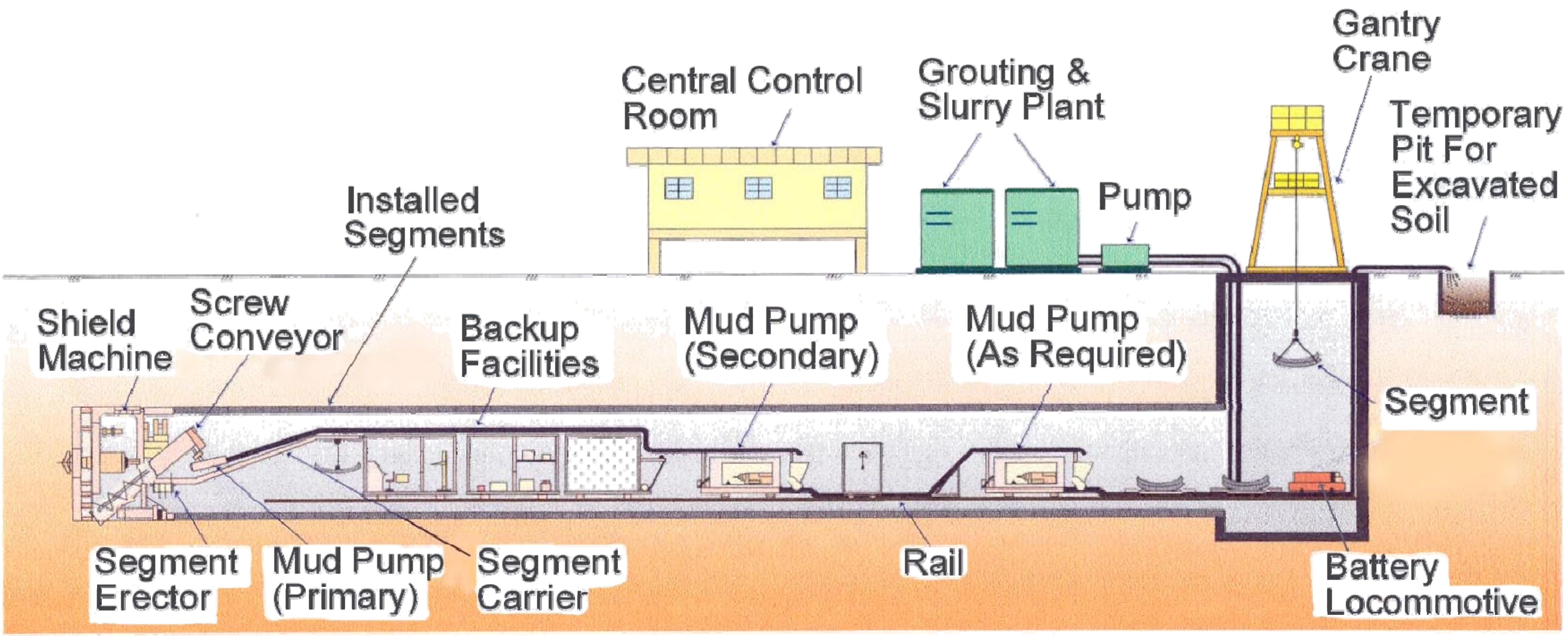


FIG. 12 DIAGRAM OF SHIELD TUNNELLING CONSTRUCTION

Layer	Depth (Below Existing Ground)	Soil Description	Soil Parameters Adopted	
			Moderately Conservative	**Worst Credible**
1.	0 – 6 m	Alluvium	C' = 0 ϕ' = 30° Ko = 0.5 Ka = 0.28 Kp = 4.50 É = 17.5MPa γ = 18kN/m3 √' = 0.3	C' = 0 ϕ' = 28° Ko = 0.53 Ka = 0.34 Kp = 3.30 É = 8MPa γ = 18kN/m3 √' = 0.3
2.	> 6m	Residual Soil	C' = kPa ϕ' = 36° Ko = 0.8 Ka = 0.22 Kp = 6.70 É = 200MPa γ = 21kN/m3 √' = 0.3	C' = 2 ϕ' = 33° Ko = 0.8 Ka = 0.28 Kp = 4.30 É = 75MPa γ = 21kN/m3 √' = 0.3

TABLE 1 THE SOIL PARAMETERS ADOPTED FOR THE DESIGN OF THE SHAFT

Layer	Description	The Recommended Soil Parameters
1	Alluvium	$C' = 0kPa$ $\phi' = 30°$ $E' = 17.5MPa$ $V' = 0.30$ $K_o = 1.0$ $\gamma = 18kN/m^3$
2	Residual Kenny Hill Soil	$C' = 5kPa$ $\phi' = 33°$ $E' = 75MPa$ $V' = 0.30$ $K_o = 1.4$ $\gamma = 21kN/m^3$
3	Kenny Hill Bedrock	Not required

TABLE 2 THE RECOMMENDED SOIL PARAMETERS FOR THE TUNNEL DESIGN