

Design and Construction Considerations for Steel Struts in Excavation of KVMRT Project in Malaysia

Yee-Eng Loh, Shaw-Shong Liew, Yean-Chin Tan and Han-Seng Tong

G&P Geotechnics Sdn. Bhd.

E-mail: gnp-geo@gnpgroup.com.my

ABSTRACT

Steel strutted Temporary Earth Retaining System (TERS) is probably the most common shoring support method for deep excavation. Therefore, good understanding of fundamental structural steel design in association with imposed loadings from geotechnical analysis is a key to successful implementation in deep excavation projects. Yet, merely understanding the structural steel design is inadequate to design the steel strutting safely and economically in TERS projects unless appropriate mobilised geotechnical loadings at various construction stages are carefully assessed. In fact, safe and economical design can be achieved when the geotechnical engineer is able to bridge the gap in complicated soil-structure interaction between geotechnical analysis and structural design with practical design considerations. This paper will discuss the current common practices by Malaysia's geotechnical engineers/specialist strutting contractors and the design and construction considerations for steel strutting elements for Klang Valley Mass Rapid Transit (KVMRT) project in Malaysia. The design of steel strutting elements for KVMRT project is in accordance with the limit state design concept in BS5950 and recommendations of CIRIA Special Publication 95 (SP95).

1. INTRODUCTION

In Malaysia, strutting elements are normally designed in accordance with British Codes. No specific technical reference is available locally for the design of strutting elements associated with excavation works. In contrast to this, Singapore had first published the Technical Reference for Deep Excavation – TR26 in year 2010 to include some good engineering practices for deep excavation design including the design of temporary strutting members following the painful and expensive lessons learnt from Nicoll Highway failure in 2004. There are two major design methods for temporary steel strutting in Malaysia; namely traditional permissible stress design in accordance with BS449 in the past and the forthcoming limit state design in accordance with BS5950.

2. PERMISSIBLE STRESS DESIGN

Permissible stress design was done by using factors of safety to reduce the ultimate or failure stresses to permissible working stresses. The permissible stress design method requires that the stresses calculated from the most adverse combination of service loads must not exceed the specified permissible stresses. BS449 requires a factor of safety (FS) of 1.7 generally, with a 25% overstressing allowed for any combinations of stresses arising from bending and axial loading to a steel structural member. Thus

$$\text{Working Stress} \leq \text{Permissible Stress} \approx 1.25 \times \frac{\text{Ultimate Stress}}{\text{FS}} = \frac{\text{Ultimate Stress}}{1.36} \quad (1)$$

This allowance of overstressing in combined stresses has been traditionally adopted in the design of temporary works in the past, and will not be discussed further in this paper. Nonetheless, the overstress of 25% shall be strictly applicable to permissible stress design only and shall not be applied to limit state design.

3. ULTIMATE LIMIT STATE DESIGN

The basis of the limit state design approach is on statistical data and lies in assessing the probability of reaching a given limit state and deciding upon an acceptable level of that probability for design purposes. The ultimate limit state is a condition of a structure, or some parts of it, is unsafe for its intended purpose, which would potentially lead to partial or total collapse. The ultimate limit state design method requires the design load effects not exceeding its design resistance as below:

$$\text{Design Load Effects} \leq \text{Design Resistance} \quad (2)$$

The design load effect is the sum of the effects of the characteristic loads multiplied by partial load factors which allow for the variability of the loads and the structural behavior. The partial load factors recommended by BS5950:2000 and TR26 are summarised in Table 1.

Table 1. Partial load factors for strutting design

Load Case	Load Factor				
	Excavation load (Earth Pressure+ Groundwater), EL	Dead load, DL	Live load, LL	Temperature load, TL	Impact load, IL
Normal working condition (NWC)	1.4	1.4	1.6	1.2	-
Accidental impact (AI)	1.05	1.05	0.33 (BS5950) 0.5 (TR26)	-	1.05
One strut failure (OSF)	1.05	1.05	0.33 (BS5950) 0.5 (TR26)	-	-

References: Table 2, Clause 2.4.5.3 and Clause 2.4.5.4 of BS5950:2000 and Table 3 of TR26

The partial load factors recommended by TR26 are the same as those in BS5950:2000 except the load factors for accidental impact and one strut failure. For design robustness considerations, load factor of 0.5 for live load is specified in TR26 for both accidental impact and one strut failure checking instead of 0.33 in BS5950:2000 for avoidance of disproportionate collapse.

BS5950 vs TR26 vs EC3 The design practice recommended by BS5950 and TR26 is to use the worst combination of dead load with imposed load and allow lower load factor of 1.2 for increase in strut force due to temperature effect. This approach seems to be rational when all loads act together; the probability of all loads attaining their maximum values simultaneously is relatively low.

However, there is different method of combining loads used in the Eurocode (EC) limit states design method, which is based on statistical analyses of the loads and the structure capacities as discussed by Trahair et al (2008). Strength design in EC is usually carried out for the most severe combination of actions using

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (3)$$

where

Σ	=	the combined effect of actions
γ_G	=	partial factor for the permanent / persistent G actions (EL & DL)
γ_Q	=	partial factor for the variable Q actions (LL & TL)
Ψ_0	=	combination factor for reduce probability of simultaneous actions

The partial load factors recommended by EC for common situations are presented in Table 2.

Table 2. Partial load factors recommended by EC for common situations

Ultimate limit state	Permanent actions γ_G		Variable actions γ_Q	
	Unfavourable	Favourable	Unfavourable	Favourable
EQU	1.1	0.9	1.5	0
STR/GEO	1.35	1.0	1.5	0

Notes:

EQU = loss of static equilibrium of the structure on any part of it

STR = failure by excessive deformation, transformation of the structure or any part of it to a mechanism, rupture or loss of stability of the structure or of any part of it

GEO = failure or excessive deformation of the ground

Summary of partial load factors and load combinations for BS5950, TR26 and EC3 under normal working condition are presented in Tables 3 and 4, respectively. Table 4 clearly shows that the steel strutting design using EC3 could yield more economical design as compared to BS5950 without taking into consideration on the structural design of steelworks with the fact that relative lower partial load factors have been specified by EC3 for permanent and variable actions.

Table 3. Summary of partial load factors recommended by BS5950, TR26 and EC3 for steel strutting design under normal working condition

Codes / Reference	Load Factor			
	Excavation load (Earth Pressure+ Groundwater), EL (Permanent)	Dead load, DL (Permanent)	Live load, LL (Variable)	Temperature load, TL (Variable)
BS5950 and TR26	1.4	1.4	1.6	1.2
EC3	1.35	1.35	1.5 ($\psi = 1.0$)	1.5 ($\psi = 0.6$)

Notes:

1. Ψ = combination factor for reduce probability of simultaneous actions
2. References: Table 2 of BS5950:2000, Table 3 of TR26, Table 2.2 of BS EN 1993-1-1 and Table 1.1 of BS EN 1990.

Table 4. Summary of load combinations recommended by BS5950, TR26 and EC3 for steel strutting design under normal working condition

Ultimate limit state	BS 5950 and TR26	EC3
STR/GEO	1.4EL + 1.4DL + 1.6LL + 1.2TL	1.35EL + 1.35DL + 1.5LL + 0.9TL

Note: Strut force due to EL is the serviceability limit state value calculated using soil-structure interaction analysis with actual soil strengths

The load combination is only part of the steel strutting design and the other aspects to look into details for comparison of BS5950 and EC3 shall be the structural design of steelworks where similar yet different approaches have been adopted by both codes of practice. The comparison of strut capacity determined from BS5950 and EC3 is not covered in this paper.

4. DESIGN CONSIDERATIONS FOR KVMRT PROJECT

In view of the massive deep excavation works that are currently taking place in Klang Valley for the underground stations along Sungai Buloh-Kajang line (SBK line) of Klang Valley Mass Rapid Transit (KVMRT) which is first MRT line in Malaysia, there is a need to establish a set of standard design criteria for temporary strutting works design, in which the similar recommended load combinations by TR26 as detailed in Table 1 are adopted. Table 5 shows the design load considerations in the design of temporary strutting for underground stations of KVMRT project.

Considering the depth and width of excavation required for KVMRT project, laced struts system as shown in Figure 1 has been chosen as temporary strutting system for the excavation works of underground stations. Similar strutting elements arrangement is now commonly adopted for deep excavation projects in Malaysia.

The lateral accidental impact load acting normal to y-axis of the steel member will have remarkable adverse impact on the single strut condition unless lateral bracing or supports are introduced at appropriate spacing to avoid high induced moment on y-axis of the member affecting its structural stability. For laced struts on the other hand, the load combination for accidental impact is usually not the governing case as the lacing members in triangular truss arrangement will efficiently resolve the lateral impact load to axial load in the lacing members to the supports which can be designed accordingly.

Table 5. Design load considerations for strutting

Design Load Combination	EL (kN/m)	LL (kN/m)	TL ΔT (°)	IL (kN)	
				Major Axis	Minor Axis
NWC	From FEA (Working Stress Condition)	1.50	10	0	0
AI		1.50	0	*50	*10 to 50
OSF	From FEA (OSF)	1.50	0	0	0

Notes: FEA – Finite Element Analysis, * recommended in Section 5.4 of CIRIA SP95.

Fig. 1. Schematic diagram of strutting elements

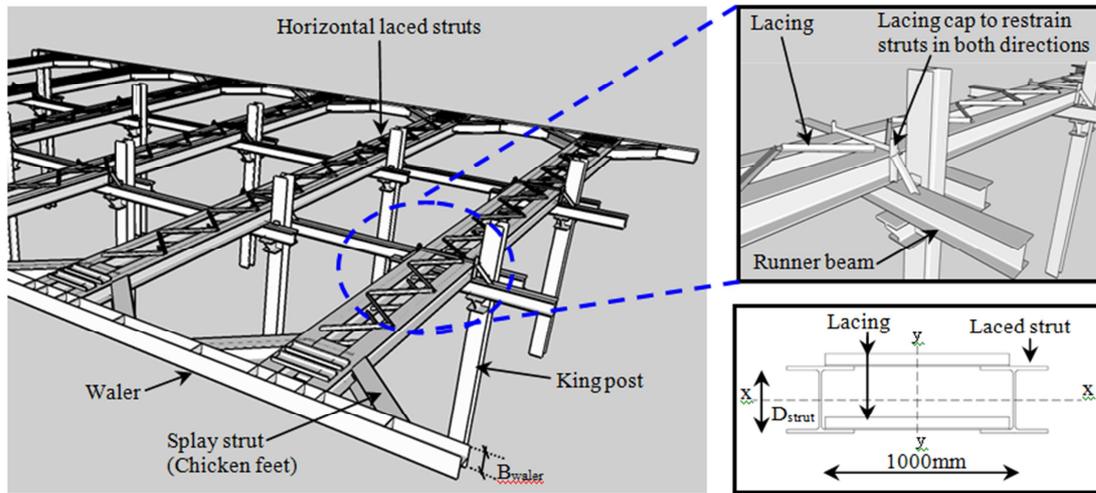


Figure 2 shows the sketch-up and photographs of typical strutting arrangement adopted for cut and cover tunnel at Maluri Portal while Figure 3 shows the king-post-free long strut for construction of TRX Station (Tun Razak Exchange Station, previously known as Pasar Rakyat Station).

The depth of excavation for TRX Station is about 45m deep where more than two third of the excavation is within rock mass of limestone. During the installation of secant pile wall, it was noticed that the rock head encountered at ventilation shaft of TRX Station is merely 5m to 8m below the existing ground. Hence, site team has proposed to use king-post-free long strut at this corner instead of the typical strutting arrangement adopted in Maluri Station that required king-post to be installed as intermediate support for the strut.

The arch strut supports shown in Figure 3 were installed after excavated to 6m below the lowest strut level in order to enhance the stability of the long strut before proceed to the final excavation level. By using king-post-free strut, expensive king-post with about 43m rock coring due to shallower rock head encountered at the ventilation shaft of TRX station is omitted to expedite the rock blasting works for station box excavation.

Fig. 2. Typical strutting arrangement for KVMRT project

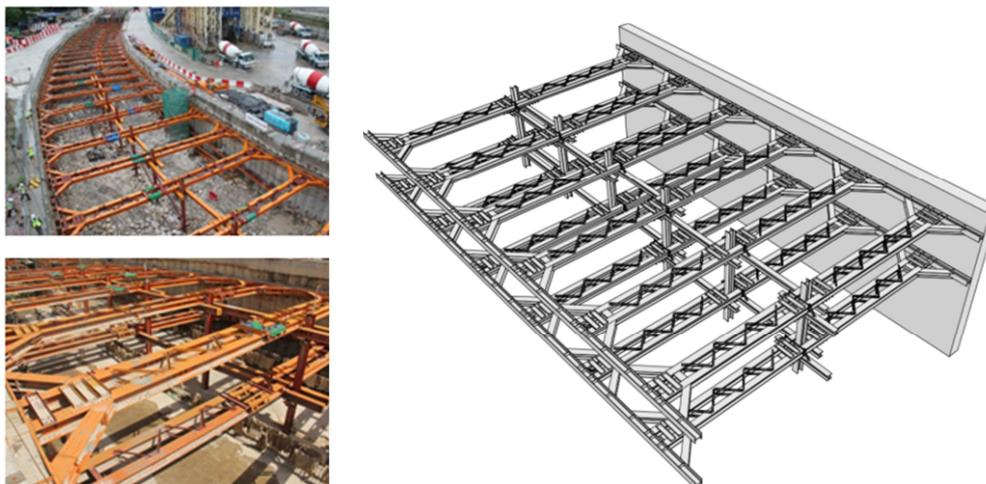
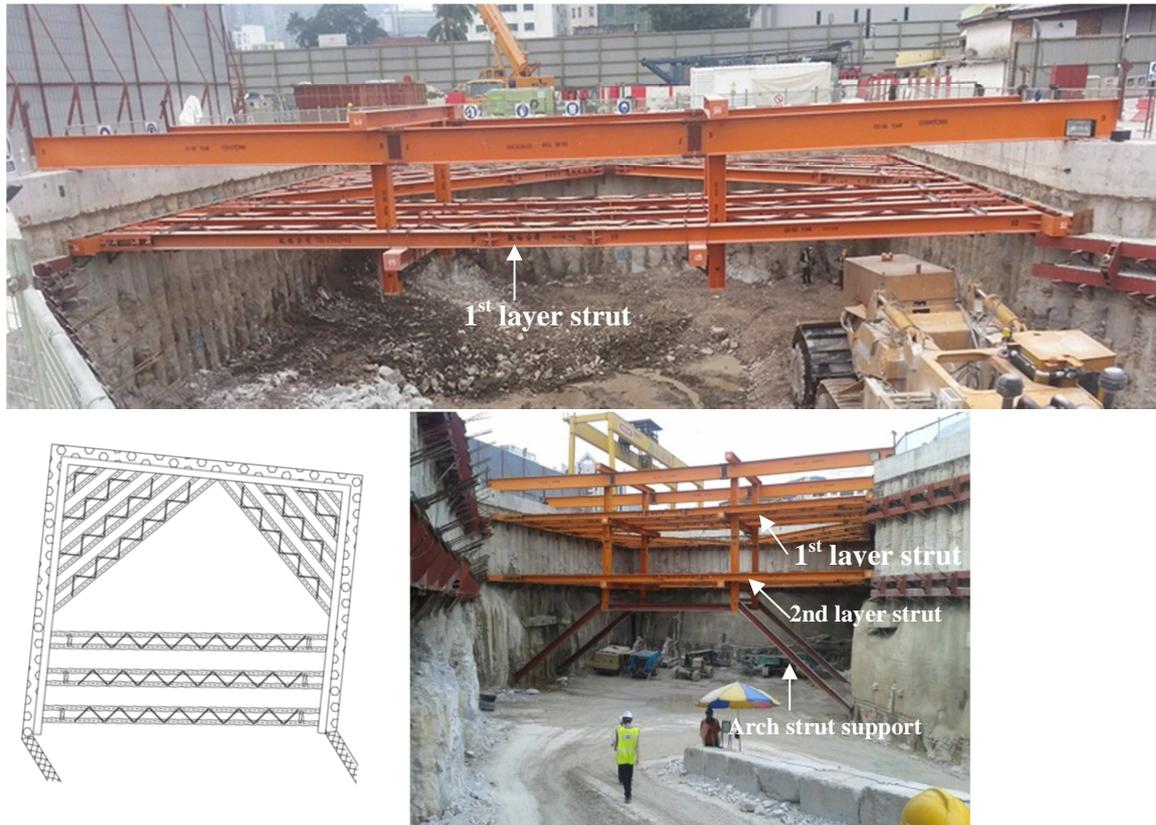


Fig. 3. King-post-free long strut for TRX Station excavation of KVMRT project



An elastic analysis is required to determine the deflection of the king-post-free long strut so that this can be compared with the desirable limiting values and forms a basis for determination of threshold limit for long strut deflection monitoring during excavation.

For this purpose, manual calculation assuming strut is simply supported at both ends as detailed in equation 4 is used to determine the deflection of long strut under its own self-weight (sw). It is imperative to include the additional moment ($P \times \delta$) due to deflection of strut in strut buckling check to ensure safe design that satisfies for both serviceability and ultimate limit states.

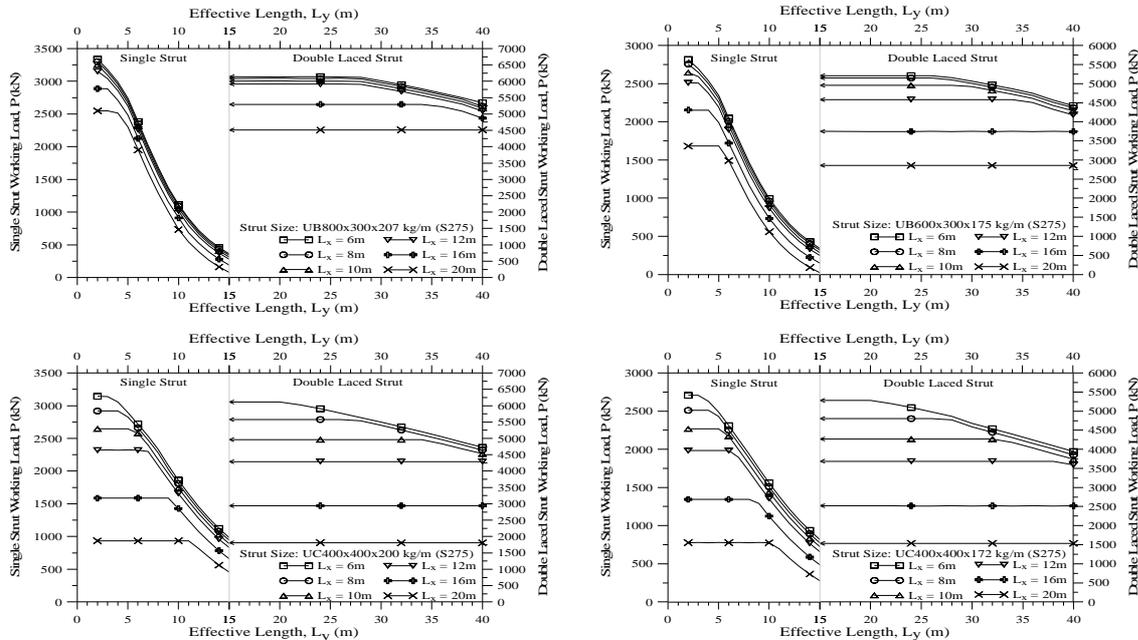
$$\text{Deflection of strut due to self-weight, } \delta = \frac{5 \times sw \times L^4}{384 EI} \quad (4)$$

The allowable strut deflection for long strut at TRX Station is 80mm (Length/275) which includes the allowable eccentricity at strut-waler connection and limit the deflection to be less than Length/200 as recommended by BS5950 for serviceability limit state. The monitoring results of the optical survey prisms installed along the struts show that the measured deflections of struts fall well within the allowable values.

5. P-L CURVE

Example of strut working load versus effective length (P-L) curves for Universal Beam (UB) and Universal Column (UC) based on the design criteria for load combination under normal working condition (NWC) are presented in Figure 4.

Fig. 4. P-L curves for selected single and laced UB/UC sections (load combination under NWC)



Notes: Separate checking is required for AI and OSF load cases to ensure the satisfactory requirements of the selected steel sizes and spans

It is worth noting that the unrestrained length of the struts will determine the efficiency of the strut. From the plotted P-L curves, the following general observations can be made:

- 1) Strut capacity of single UB is very sensitive to variation of effective unrestrained length in y-axis (L_y).
- 2) Strut capacity of UC is very sensitive to variation of effective unrestrained length in x-axis (L_x) for L_y not more than 8m.
- 3) UC of equivalent sectional area performs better than UB when used as single strut with L_y longer than 8m and L_x less than 16m as evidenced from the sections explored.
- 4) UB of equivalent sectional area performs better than UC when used as double laced struts. However due consideration shall be taken to ensure the breadth of waler is adequate for proper strut/waler connection ($B_{waler} > D_{strut}$).
- 5) Optimum strut capacity can be achieved with typical strut support (eg. King-post) spacing of not more than 10m ($L_x \leq 10m$).
- 6) For excavation width more than 15m wide, use laced strut to avoid massive intermediate lateral restraints and supports for economic design and better performance.

6. RESTRAINTS AND BRACING

The purpose of the restraint is to reduce the effective length of the strut to a desired length in which the overall axial strut capacity can be effectively utilised to its structural capacity without buckling or structural instability. The restraining force induced in steel struts given by BS5950:2000 shall not be less than 1% of the strut axial load for intermediate restraint, and 2.5% for lacing forming the laced struts. If there is missing or inadequate bracing system to properly transfer and balance the restraint

force with necessary overall static equilibrium, the strut may be under-designed and can contribute to the strutting failure of excavation as reported by Liew and Loh (2011) as presented in Figures 5 to 7.

Fig. 5. Site condition after CBP wall failure

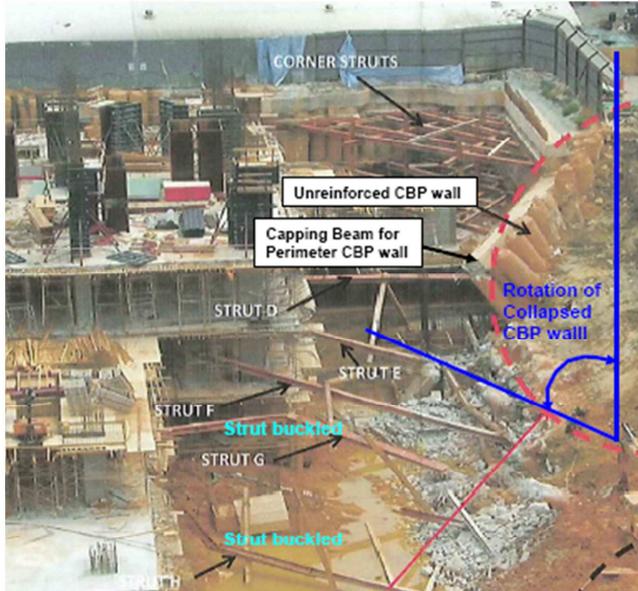


Fig.6. As-built strutting layout without proper bracing for overall static equilibrium

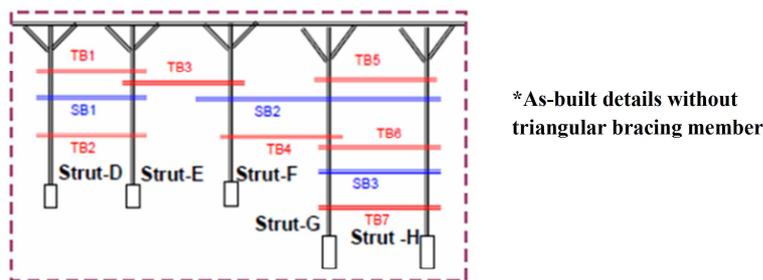
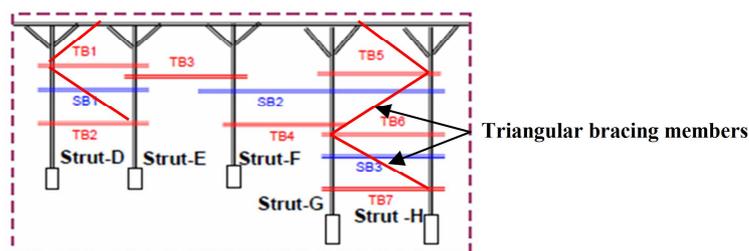


Fig. 7. Strutting layout with proper bracing for overall static equilibrium



7. FINITE ELEMENT ANALYSIS

In today practice, finite element (FE) analysis is often used to evaluate the potential geotechnical loads acting on excavation support systems from the surrounding soil mass at every stage of works. Without appropriate consideration of end-restraint conditions in the FE model, design loads predicted by computer analyses may not be representative of actual loading conditions. For instance, the connection between the strutting and retaining wall normally is not designed to take tension. However this is often ignored where the strut member is not modelled as compression only element in FE analysis. It is important to specify zero tension capacity for the modelled strut so that the strut does not restrain the

retaining wall when the retaining wall is moving away from the excavation area which is often observed for the upper layer strut of multi-propped wall in deep excavation as illustrated in Figure 8.

The calculation of required capacity and stiffness of the strut for member sizing is essentially an iterative process. For design purposes it is suggested to estimate the distributed prop loads (DPL) using DPL diagrams recommended by TR26 or appropriate apparent earth pressure diagrams (AEPD) summarised by JSSMFE (1993) or other relevant references and then evaluate the member sizing with P-L curves before proceeding with sophisticated FE modelling. Reviewing the member sizing with P-L curve shall be performed again after the first determination of approximate prop loads from FE analyses. It is imperative to investigate the sensitivity of the prop loads to variations in the soil parameters and other input assumptions such as fluctuation of groundwater table, construction sequences, etc in finite element analysis. The process by which this may be achieved is summarized in Figure 9.

Fig. 8. Illustration of possible multi-propped wall deflection

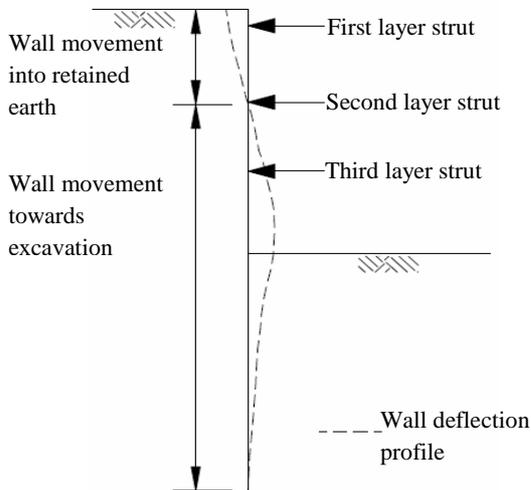
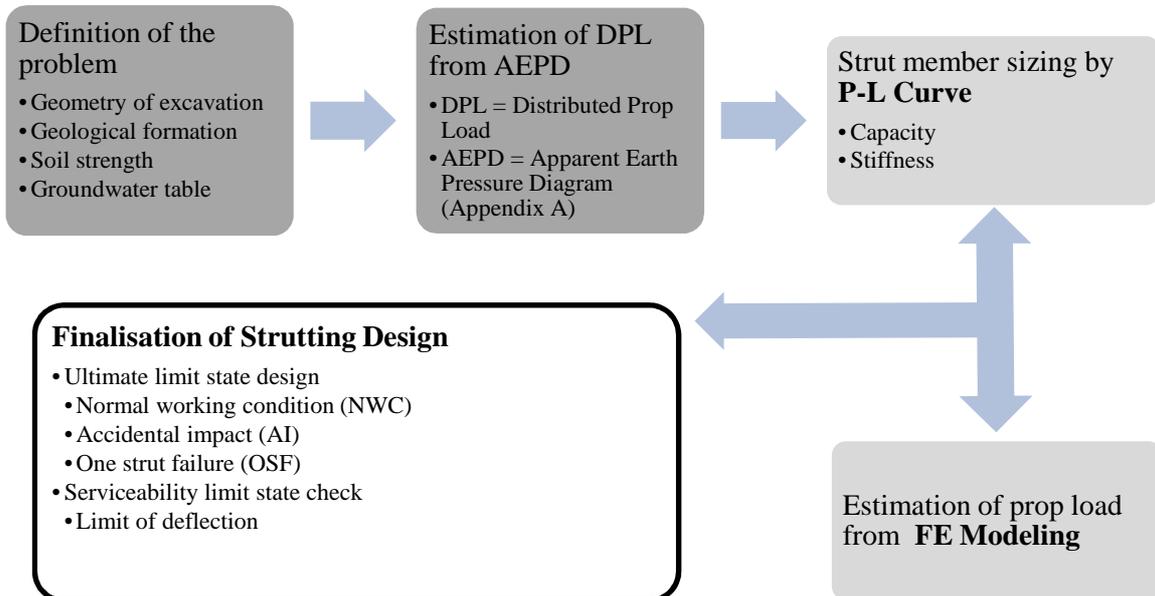


Fig. 9. The iterative design process



Is Preloading Required? There is a good practice to specify a nominal preload force of not more than 50kN/m to the strutting system if preloading is not specifically required to control the wall deflection, but it is necessary as precaution step to close up the contact gap between the waler and wall, if any.

In normal circumstance, the strut preload force shall be not more than 50% of the design working load of strut, considering primarily allowing incremental increase in strut loads with further excavation. In exceptional case, higher preload forces may be considered for stringent ground movements control but the removal of strut shall be modelled in FE analysis to study the possibility of overstressing of adjacent struts during strut removal stage. The arrangement and sequence of the jacking position for preloading shall be carefully evaluated to avoid uneven loading within the overall strutting system, particularly for strutting scheme with lacing and restraints, which makes the structural system statically indeterminate.

In most of the cases, the preloading of struts will be carried out before the complete connections done on the bracing or lacing for laced members in which the designer shall make sure the specified preloading forces do not exceed the permissible threshold of prestress of the struts under such condition or else adequate restraints would need to be installed before preloading. The P-L curves can be useful in assessing the suitable preload forces for struts without proper bracing or lacing during preloading and caution the designer on the usage of very high preload force for long struts in design analysis.

8. SUMMARY AND RECOMMENDATIONS

The current common practices by Malaysia's geotechnical engineers and specialist strutting contractors and the recommendations of design and construction considerations for strutting elements for TERS have been discussed. The recommendations of design and construction considerations for steel strutting elements for TERS are as follows:

- a) The design of strutting elements should follow the appropriate standards, technical references, guidelines, and good practices for safe and economical design.
- b) The design of strut shall include the additional moment ($P \times \delta$) due to any foreseeable deflection of strut in buckling check to ensure safe design for both serviceability and ultimate limit state.
- c) Creation of simple P-L curves for all strut member size explored will help the designer to expedite the design process with the selection of suitable strut sizes, spacing of restraint and preloading forces for subsequent analysis.
- d) Extra care to be taken on the overall restraining system when intermediate restraints are required for higher utilisation of strut axial capacity close to the structural capacity of the member section.
- e) Detailing of connections should be followed strictly at site to ensure the completeness of design from planning to construction executions.
- f) Engineering judgements are required for necessary preloading adjustments during design and execution. Use of high preloading strut forces are not recommended unless careful assessments on effects of high preload have been carried out with consideration of practical construction method.

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APPENDIX A

Lateral pressure distribution based on field measurement		Strut load (Stain gauge settled up strut)					
		Japanese proposal A	Japanese proposal B	Terzaghi & Peck	Tschebirtaroff	Japanese proposal C	Japanese proposal D
Lateral pressure distribution based on the earth pressure theory	Sandy soil	Japanese proposal A (Lateral pressure) $P_a = K \gamma H$ K ; Coefficient of lateral pressure Soft clay 0.5 ~ 0.8 Stiff clay 0.2 ~ 0.5	Japanese proposal B (Lateral pressure) $P_a = K \gamma H$ K ; Coefficient of lateral pressure Soft clay 0.4 ~ 0.5 Stiff clay 0.2 ~ 0.4 N > 4 N: N value of standard penetration test	Terzaghi & Peck (Lateral pressure) $P_a = 0.65 K_a \gamma H$ γ : wet unit weight of soil $K_a = \tan^2(45^\circ - \phi/2)$ ϕ : angle of internal friction	Tschebirtaroff (Lateral pressure) $P_a = 0.27 \gamma H$ γ : wet unit weight of soil	Japanese proposal C (Lateral earth pressure and water pressure) $P_a = a \gamma H$ a ; Coefficient of excavation depth $5 > H \ge 3m$ $a = (H-1)/4$ $5m \le H$ $a = 1$ C: Coefficient of geology N <= 5 C = 6 N > 5 C = 4	Japanese proposal D (Lateral pressure) $P_a = 0.27 \gamma H$ γ : wet unit weight of soil
		Cohesive soil	Japanese proposal A (Lateral pressure) $P_a = K_a \gamma H - 2C_a / K_a$ $K_a = \tan^2(45^\circ - \phi/2)$ γ : wet unit weight of soil γ' : submerged unit weight of soil γ_w : unit weight of water ϕ : angle of internal friction c : cohesion	Japanese proposal B (Lateral pressure) $P_a = K \gamma H$ K ; Coefficient of lateral pressure Soft clay 0.4 ~ 0.5 Stiff clay 0.2 ~ 0.4 N > 4 N: N value of standard penetration test	Terzaghi & Peck (Lateral pressure) $P_a = 1.0 K_a \gamma H$ γ : wet unit weight of soil $K_a = \tan^2(45^\circ - \phi/2)$ ϕ : angle of internal friction Diagram for soft to medium clays stiff- fissured clays $K_a = 1 - m \frac{4c}{\gamma H}$ m : taken as 1.0 except for truly normally loaded clays when $N = \gamma H/c$ exceeds about 4, under which condition, $m < 1.0$ Lower pressure to be used only when movement can be kept to a minimum and construction time is short	Tschebirtaroff (Lateral pressure) $P_a = 0.57 \gamma H$ γ : wet unit weight of soil Medium clays Stiff clays	Japanese proposal C (Lateral earth pressure and water pressure) $P_a = K \gamma H$ K ; Coefficient of excavation depth $5 > H \ge 3m$ $a = (H-1)/4$ $5m \le H$ $a = 1$ C: Coefficient of geology N <= 5 C = 6 N > 5 C = 4

Reference: JSSMFE (1993). "Underground construction in soft ground in Japan", Japanese Society of Soil Mechanics and Foundation Engineering, Japan, pp 26