Proceedings of 12th IOE Graduate Conference
Peer Reviewed
ISSN: 2350-8914 (Online), 2350-8906 (Print)
Year: 2022 Month: October Volume: 12

Investigation of Base Stability for an Underground Kathmandu Metro station

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Abstract
Excavation work essential for the development of underground space is heading in the directions of greater depths, larger scale, and difficult soils. The aim here is to analyze a deep excavation planned for the underground Kathmandu Metro Railway Station in the soil of Kathmandu valley in order to understand the basal heaving mechanism. The research relies on the use of classical analytical and recent numerical limit analysis solution from different literatures to verify the finite element analysis performed for the determination of Factor of safety.

Keywords
Deep Excavation, Numerical Analysis, FE Analysis, Total Stress Analysis, LEPP Mohr-Coulomb Model, Manual c-ϕ reduction, Intersection Method, Failure Mechanism study, Underground Kathmandu Metro Station, Braced Excavation

1. Introduction

1.1 Background

Due to various physical, economic, social and political factors, the necessity of development of underground space for use as transportation for railway tunnels, metro stations, service tunnels, basements for underground parking, etc. is therefore becoming more and more common.

Although a number of deep excavation works have been already carried out failures (stability) associated with deep excavation are commonly reported [1], [2] and [3]. Despite the needs of numerical modelling to fully capture the performance, the collapses of deep excavations have been reported and linked to inappropriate numerical analyses during design phase [1].

1.2 Statement of the problem

While the preliminary designs for major and intermediate Metro Stations have been proposed by several designers and researchers for the Underground Kathmandu Metro Rail, the geotechnical feasibility of such preliminary designs are unexplored in the context of soils of Kathmandu.

1.3 Objectives

The major objective of this study is to determine the Factor of safety of the Preliminary design for Underground Kathmandu Metro Station.

1.4 Scope of the research

The geotechnical investigative study of the short-term excavation base stability based on the preliminary design for Underground Kathmandu Intermediate Metro Station (with the geometric design parameters: such as excavation length 144.0 m, excavation width 26.0 m, excavation depth 36.0 m and diaphragm wall thickness 1.5 m) prepared by Department of Transport Management (DOTM) for New Baneshwor Intermediate Station has been studied in this research for the soil conditions typical to Kathmandu valley particular to Khullamunch area using the Soil Investigation Report [4]). Linearly Elastic Perfectly Plastic (LEPP) Mohr-Coulomb Model of soil and Perfectly Elastic Model of structure are scoped for the simplistic modelling of the stability problem.

2. Literature Review and Data Collection

2.1 Analytical solutions

The summary of analytical solutions is taken after [5] as shown in Table 1. The factor of safety has been defined as the Right Hand Side (RHS) (which is called as the stability factor of overall resistance to basal heave soil failure) of the equation divided by the Left Hand Side of the Equation (LHS) (which is the driving factor to basal heave soil failure) [6].

Table 1 is only for the homogenous clay layer. However, based on the failure surface assumed, the Equations can be used by averaging the shear strength over depths as mentioned in [7]. The s_{ub}, s_u is averaged at the depths from H to H+0.7071B. In other words the average depth for the s_{ub}, s_u is H + 0.3536B [7]. Likewise, the s_{uu} is averaged at the depths from 0 to H. In other words the average depth for the s_{uu} is 0.5H [7].The average depths are based upon the failure zone assumed in the limit analysis while deriving the solutions. With above mentioned averaging of shear strength, as well as some adjustments of unit-weights in the surcharge term; Method 5 has been investigated to be the most appropriate solution for predicting the stability of undrained stability of deep excavations based on three real case failure histories from the literature [8].

2.2 FEM based solutions

2.2.1 Reason for selecting numerical solution such as FEM based solution

For a complete theoretical solution of geotechnical problems the following four conditions should be satisfied: [9]

1. Equilibrium
2. Compatibility or kinematic relationship
3. Material Constitutive behaviour
4. Boundary conditions
   (a) Boundary Conditions of displacements (Essential boundary conditions)
   (b) Boundary Conditions of force/stress (Natural boundary conditions)

Numerical methods are techniques to approximate the governing equations in the mathematical models. Unlike other solutions such as limit equilibrium, limit analysis, etc., a full numerical analysis satisfies all four conditions of a theoretical solution approximately, and the satisfaction of the condition can be improved with refining of the numerical grids, mesh, volume or corresponding calculation units [10]. Finite Element Method, commonly abbreviated as FEM is a very versatile type of numerical method.

2.2.2 Choice of constitutive model

The FEM analysis solely rests upon the choice of constitutive model. Because the fine-grained clayey silts of cohesive nature and low permeability are the predominant soils affecting the Underground Kathmandu Metro Station excavation, a Total stress based $\phi = 0$ Linearly Elastic Perfectly Plastic (LEPP) Mohr-Coulomb Model was adopted to predict the stability of the excavation. The above model was verified to make reasonable wall displacements for intermediate to final stages for excavations in soft clayey nature soil such as CL [11]. Nevertheless, the same model has been verified and validated by several researchers in carrying out the stability analysis to give quite accurate results in soft clay soils [12], [13].

2.2.3 Method of Stability Analysis

The FEM Intersection Approach has been verified as the best method for assessing basal heave stability in soft clays [12], [13], [14]. The FEM Intersection Method can be carried out with an updated mesh Finite Element Analysis, an advanced option in PLAXIS for the Intersection Method [13]. The Angle Method can also be used to check out for any failures [13]. The intersection method must be carried out by manual means of iteration, because the c-phi reduction in the Safety Analysis doesn't carry out the c-phi reduction from the first stage of the excavation, but only carries out the reduction for a single stage concerned [12]. Moreover, the c-phi reduction method will reduce the c-phi until a large number of plastic points are formed or the numerical solution diverges. It has been explained that the onset of failure doesn't start near the divergence of the solution, nor at the large number of plastic points [13]. It starts when the displacements of soil, structures, etc. start changing rapidly while reducing the shear strength by a same amount, which is the basis of the intersection method [13]. The relation of intersection method to a failure mechanism was also introduced in [14].

2.2.4 Verification and Validation of FEM model

Benchmarking is the process of testing, validating, confirming, or checking the performance or

Table 1: Summary of Analytical solutions used for the verification of the FEM models (taken after [5])

S.N.	Description	Equation	Remarks
1.	Method 1	$\frac{\gamma H}{s_u} = \mathrm{N}_{c1} + \sqrt{2}(\frac{H}{B})$	Base case-homogenous deep clay
	Terzaghi (1943)		$\mathrm{N}_{c1} = 5.7$, $d_b > 0.707B$
2.	Method 2	$\frac{\gamma H}{s_u} = \mathrm{N}_{c2}$	Base case-homogenous deep clay
	Bjerrum and Eide (1956)		$\mathrm{N}_{c2} = f\ (H/B) = 5.1 \rightarrow 7.5$
3.	Method 3	$\frac{\gamma H}{s_{uu}} = \mathrm{N}_{c1}(\frac{s_{ub}}{s_{uu}}) + \sqrt{2}(\frac{H}{B})$	Two clay layers:
	Terzaghi (1943)		s_{uu}, s_{ub} –strengths in retained soil and below excavated grade
4.	Method 4	$\frac{\gamma H}{s_u} = \mathrm{N}_{c2} + \mathrm{x}\frac{\pi^2 M_y}{D(D+h)s_u}$	Effect of embedment-elastic strain energy for bending of wall
	O'Rourke (1993)		M_y –yield moment of wall; h–excavation depth below lowest support; and x = 1/8, 9/32, 1/2 for free, sliding, and fixed-end conditions
5.	Method 5	$\frac{\gamma H}{s_u} = \frac{(2+\pi)(\frac{B}{H}) + b_2 + b_3\sqrt[4]{\frac{M_p}{s_u D^2}} + \left(\frac{M_p}{s_u D^2}\right)(\frac{D}{H})^2\{c_2(\frac{D}{H})(\frac{H}{B})^2 + c_3\sqrt{\frac{(D/H)}{(H/B)}}\}}{\frac{B}{H} + a_2 + \sqrt{\frac{M_p}{s_u D^2}}(\frac{D}{H})\{a_3(\frac{D}{H}) + a_4\sqrt{(\frac{D}{H})}\}}$	Numerical limit analysis: Design charts solution
	Ukritchon et. al. (2003)		$a_2 = 0.608$, $a_3 = 0.208$, $a_4 = -0.224$, $b_2 = 6.102$, $b_3 = 2.082$, $c_2 = 0.147$, $c_3 = 0.173$

functioning of software [9]. In geotechnical application, a FE analysis model approved once for an application needs to be re-validated w.r.t different other solutions because the ground conditions are changeable, and the nature of the geotechnical problem is highly diversified [15].

Verification ensures that the process of executing the software solves the mathematical model correctly. Verification errors can be caused by faults in program code, defects, or difficulties caused by software hardware interface or operating system interface, code changes, and so on. Thus, verification may be tested against analytical answers to simple scenarios. Verification is the developer's primary obligation and the user's secondary responsibility.

Validation, on the other hand, is the process of determining whether or not the outcome of executing the program accurately represents physical reality. Errors in the application of an incorrect mathematical model (constitutive model), errors in the user's data input, errors in obtaining the input geotechnical parameters, errors in modelling simplifications, and method misuse can all lead to validation errors. Thus, validity may be tested against empirical solutions of similar realistic physical conditions. Validation is the user's duty, and the ways for doing so are briefly addressed.

There are several ways to test the software. Based on multiple literature reviews, the following are some of the most prevalent methods:

- Initial check with respect to reasonability of the output (Deformed mesh and Deformations, Plastic points, etc.).
- Comparison with known solutions (closed form solutions, solutions from design charts and empirical methods of case histories).
- Determining sensitive parameters and verifying the model (to ensure that the main parameters that drive the key outputs are matching with the parameters that are based on the physics of the problem and they are not based on the mesh quality or interface values [1], [16].
- Comparison of other numerical analysis software (within 10% variation, would provide some sort of validation of the original FE analysis) [15].
- Comparison with site monitoring data.

3. Collection of data

3.1 Geological study

The Engineering and Environmental Geological Map of the Kathmandu Valley (1:50,000) published by DMG which is intended for planning at a regional scale and for the basis of site investigation was collected and studied for general idea and is compared with the maps published in JICA's reports for earthquake assessment in Kathmandu [14].

3.2 Project Requirements of Kathmandu Rail Project and Proposed Design

The design of Kathmandu Rail project of the Government of Nepal was thoroughly reviewed to understand the project requirements. The length of the intermediate metro station platform is 144.0 m in length, whereas the width is 25.1 m to accommodate two twin tunnels. Likewise, the intended depth of excavation is dependent upon the actual tunnel line alignment. The intended depth at the New Baneshwor Station is 35.9 m (which includes the over excavation tolerance).

The existing design proposed is as mentioned below. The design proposed in a rectangular diaphragm wall along with plunged columns in the middle of the zone of excavation. The total thickness of the R.C.C. diaphragm wall proposed in the excavation is 1.5 m. A base slab at the bottom of the intended depth of excavation is also proposed for long term stability as well as the functional requirement of the metro station platform. Likewise, the total length (in the direction of depth) of each individual diaphragm wall proposed is 45.0 m, i.e., the embedment depth of 9.1 m. The spacing of the last prop from the intended depth of excavation is 4.2 m. The depth of the first prop is near the top (half the thickness of prop and some clearance (say 0.5m)). The spacing of the five props w.r.t each other are 9.4m, 8.1m, 6.3m, 5m, 2.9m respectively. The average spacing of the spacing of first prop from the top, all five props w.r.t each other and the spacing of last prop from the intended depth of excavation (temporary and permanent but excluding base slab) is 5.2 m.

3.3 Geotechnical study

The geotechnical study was carried out in the Khullamunch area with the help of boring up to 50.0 m depth, sampling, Standard Penetration Test and Ground Water Monitoring [4]. A 100 mm diameter sized borehole was drilled with the help of wash boring, and logged continuously in the field [4]. The details of the testing and results of the lab tests are in the Kathmandu Metro Rail Project- Geotechnical Investigation Report [4], which was collected for the geotechnical study of the Khullamunch area.

4. Methodology

4.1 Flowchart of overall works

The flowchart is shown in Figure 1 and 2.

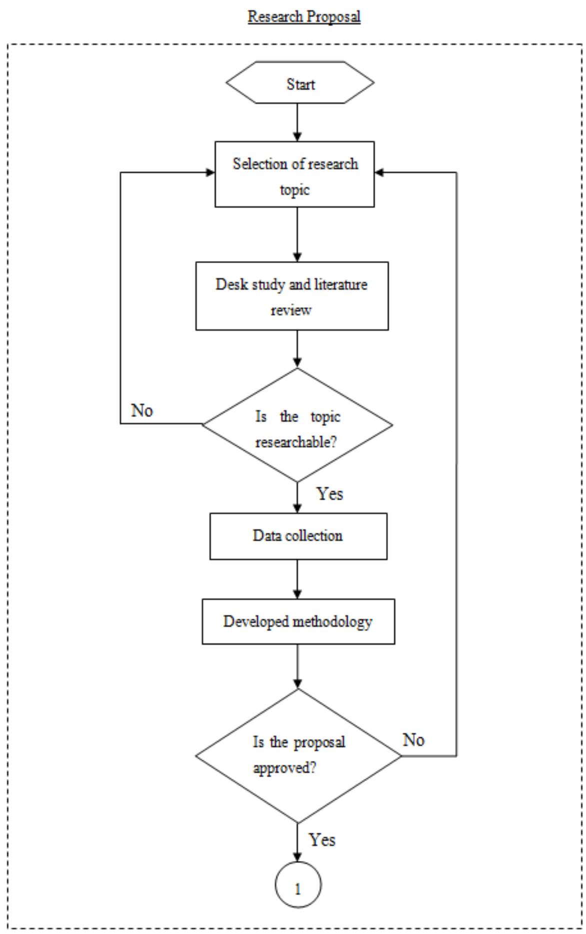

Figure 1: Flowchart showing overall methodology (Part I)

4.2 Benchmarking of FEM software

4.2.1 Geometry Adopted for Benchmarking of FEM Software

The entire geometry adopted was taken as same as that in the research paper [17].

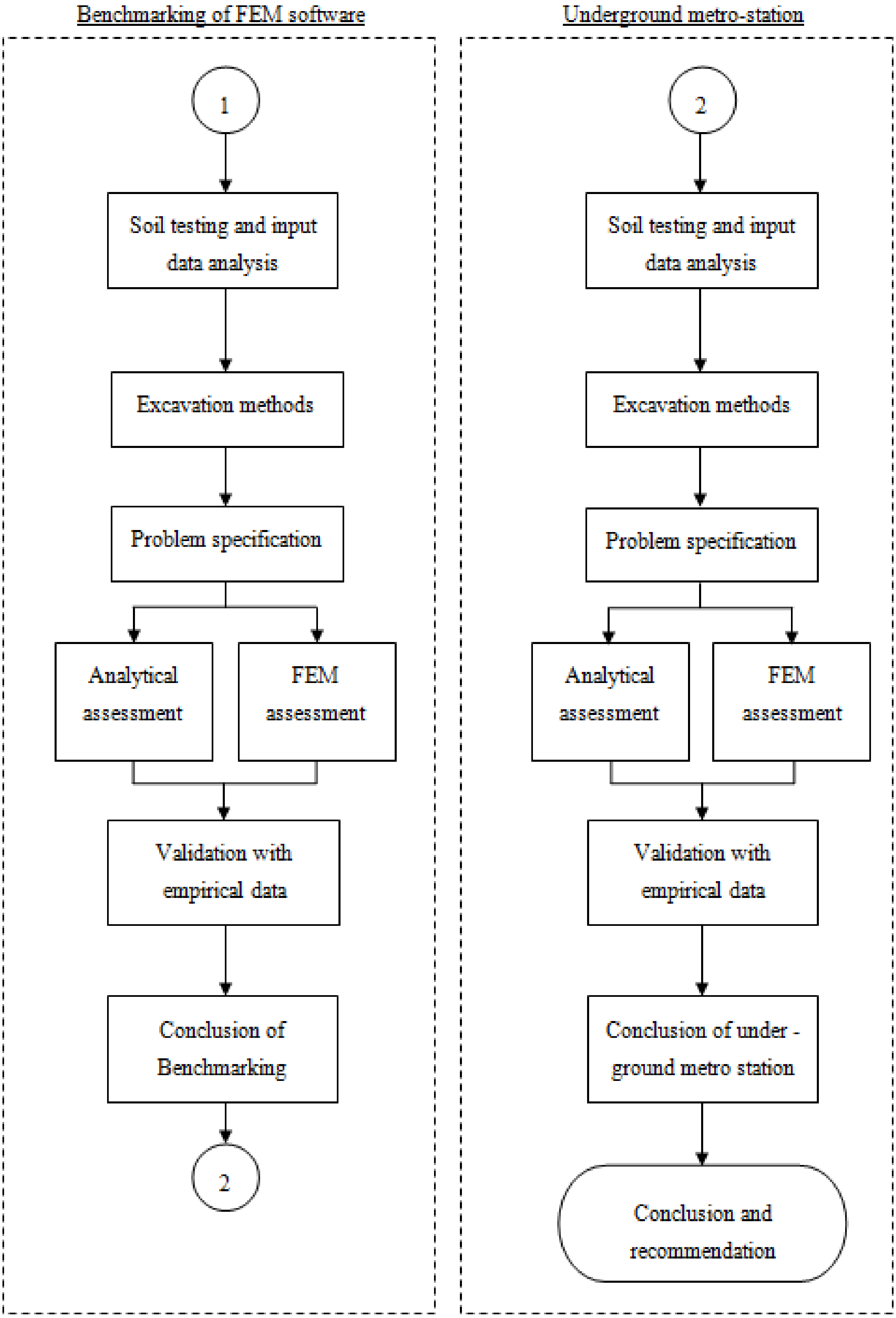

Figure 2: Flowchart showing overall methodology (Part II)

4.3 FEM Model Development for Benchmarking and Analysis of FEM

The entire FEM Model Development for Benchmarking of FEM Software was taken as same as that in the research paper [17].

4.3.1 Manual c-phi reduction Method / Intersection Method

The shear strength profile of the original model was reduced by a strength reduction (SF) and the model was run corresponding to each shear strength profile turn by turn.

Different models were prepared for the underground metro station and run with variable SF and the displacements of key structures and soils were noted down in systematic order, after the initial check of the output of the FEM for reasonability and nature of deformations. Firstly, $k_o = 0.5$, was performed. Similarly, $k_o = 1$, was also performed. A total of 22 different models were analysed for the two k_o cases.

Each of the cases was plotted with SF in the x-axis vs. Displacement of key structures and soil in the y-axis. The intersection point corresponding to the formation of a slip-line mechanism checked w.r.t plastic points in both of the k_o values, beyond which there is a rapid increase in the displacements; was noted down. The SF corresponding to that particular intersection gave the Factor of Safety correspondingly.

4.4 Investigation of Basal Heave Stability for Kathmandu Metro Station

4.4.1 Geometry Adopted for Underground Metro Station

Adopting the information in the section 3.2 of the collection of data, the total width of the excavation has been adopted as 26 m. Finally, the total depth of the excavation for the Khulla munch section is adopted as 36.0 m.

4.4.2 Shear Strength Profile from Geotechnical Study

Based on the report mentioned in the section 3.3 of the collection of data, the undrained shear strength profile was derived from the SPT values based on the latest method tested for the soils of Kathmandu from the data of 69 boreholes [18] and then was also validated w.r.t. measured UCS strength values.

4.4.3 FEM Model Development for Underground Metro Station

Geometry of FEM model and Meshing
The total length of the excavation has been assumed to be sufficiently long to model the stability using 2D, and to yield slightly conservative stability analysis.

Due to the symmetrical geometrical as well as loading conditions, only half of the geometry was modelled. The meshing was done gradually and the outputs of the FEM were analysed. The mesh was adopted as very fine mesh with refinements near diaphragm walls and soil stratums.

Boundary Conditions of FEM
A left boundary was set at a distance greater than four times the excavation depth. Likewise, a right boundary was set at the centre of the excavation considering the symmetry. The bottom was set at a distance sufficiently below the toe of the diaphragm wall.

4.4.4 Analysis of FEM

The initial analysis was carried out using k_o procedure. The all other analyses were carried out using staged construction procedure. A total of 25 Stages were analysed to obtain the FEM solution.

4.4.5 Manual c-phi reduction Method / Intersection Method

The shear strength profile of the original model was reduced by a strength reduction (SF) and the model was run corresponding to each shear strength profile turn by turn.

Different models were prepared for the underground metro station and run with variable SF and the displacements of key structures and soils were noted down in systematic order, after the initial check of the output of the FEM for reasonability and nature of deformations. Firstly, $k_o = 1$, was performed. Similarly, $k_o = 0.5$, was performed. A total of 17 different models were analysed for the two k_o cases.

The plotting for the FOS determination was done in the same way as in Section 4.3.1.

5. Result and discussion

5.1 Result

5.1.1 Benchmarking of FEM Software

The results of the literature based analytical solutions calculated for the N2 Excavation for the Benchmarking is listed in the Table 2.

Table 2: Summary of Factor of Safety (FOS) from analytical calculation for Benchmarking of FEM software and Underground Metro Station

Description	Benchmarking	Underground Metro
Method 1	1.094	0.675
Method 2	1.088	0.622
Method 3	1.034	0.595
Method 4	1.069	0.630
Method 5	1.054	0.658

Similarly, the results of the FEM intersection method for the same N2 Excavation are also shown in the Figure 3 and 4.

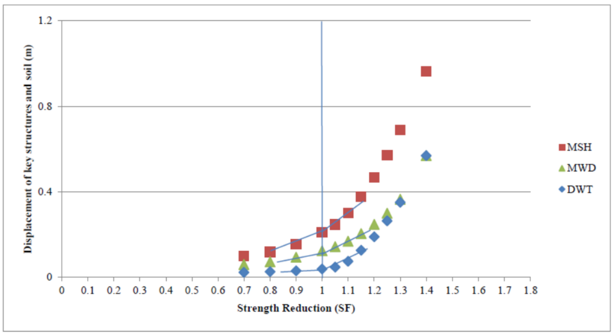

Figure 3: Assessment of Factor of Safety (FOS) for Benchmarking of FEM Software (k_o =0.5), FOS = 1.0 based on mechanism and Intersection

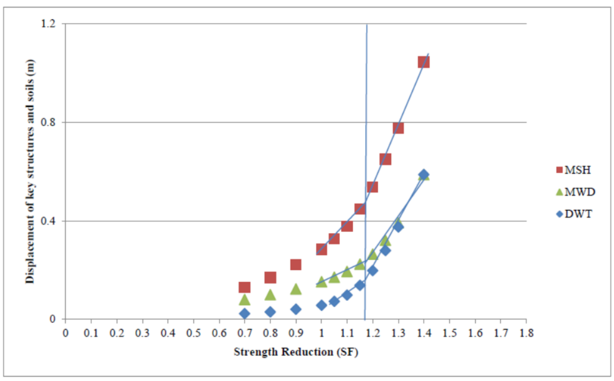

Figure 4: Assessment of FOS for the Benchmarking of FEM Software (k_o = 1), FOS = 1.16 based on mechanism and Intersection

5.1.2 Investigation of Basal Heave Stability for Kathmandu Metro Station

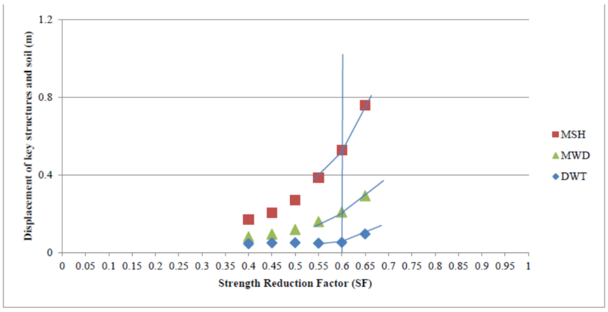

Figure 5: Assessment of FOS for the Investigation of Basal Stability of Underground Kathmandu Metro Station (k_o =0.5), FOS = 0.60 based on mechanism and Intersection

The results of the literature based analytical solutions calculated for the Kathmandu Metro Station Excavation investigation is listed in the Table 2. Similarly, the results of the FEM intersection method for the same Kathmandu Metro Station Excavation investigation are also shown in the Figure 5 and 6.

5.2 Discussion

The nature of the intersection points is similar at the SF values for different values of k_o and some judgement w.r.t plastic points from heaving mechanism could only distinguish between the FOS of the two cases. Therefore, FEM based intersection method should be applied together with the insight on the failure mechanism of the analysed problem.

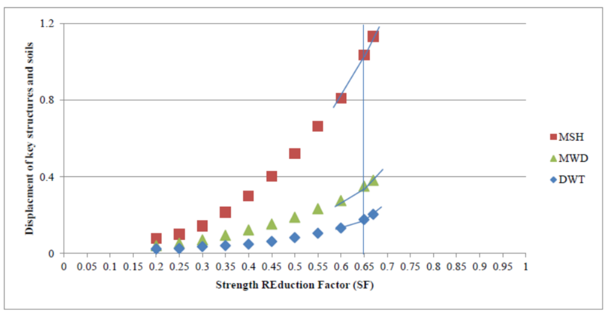

Figure 6: Assessment of FOS for the Investigation of Basal Stability of Underground Kathmandu Metro Station (k_o = 1), FOS = 0.65 based on mechanism and Intersection

6. Conclusion

The Preliminary design for Underground Kathmandu Metro Station is concluded to be highly unstable for the geotechnical conditions of Khullamunch area (based on the FOS of only 0.658 from latest analytical design chart equation i.e., Method 5 and FOS from FEM using ko adjustment also being 0.65).

The at-rest earth pressure coefficient, k_o has no significant effect on the Factor of Safety (FOS). The at-rest earth pressure coefficient has only mild effect on FOS; lower the k_o, slightly lower the FOS. Furthermore, the unadjusted values of k_o resulted in a modest underestimation of FOS compared to the analytical design chart equation.

7. Recommendation

The failure mechanism considered in this study is based on LEPP Mohr-Coulomb Model of soil and perfectly elastic behavior of the soil. However, in order to further study the detailed mechanism, a better analysis can be performed by considering the structure as elasto-plastic in the soil-structure interaction analysis. This kind of analysis would be more realistic in studying the failure mechanism of the basal heave.

Acknowledgments

The authors are grateful towards Mr. Pitamber Wagle and Mr. Pawan Babu Bastola for their important assistance during the research paper submission and for the data collection via official reports from the associated Departments of the Government of Nepal (GoN) respectively.

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