

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**DESIGN OF MSE/SOIL NAIL HYBRID WALLS
UNDER HIGHSPEED RAILWAY LOADS**



M.Sc. THESIS

Ehsan SEYEDTEHRANI

Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Programme

Thesis Advisor: Assoc. Prof. Dr. Aykut ŞENOL

OCTOBER 2016

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**DESIGN OF MSE/SOIL NAIL HYBRID WALLS
UNDER HIGHSPEED RAILWAY LOADS**



M.Sc. THESIS

**Ehsan SEYEDTEHRANI
(501121327)**

Department of Civil Engineering

Soil Mechanics and Geotechnical Engineering Programme

Thesis Advisor: Assoc. Prof. Dr. Aykut ŞENOL

OCTOBER 2016

İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**YÜKSEK HIZLI TREN YÜKLERİ ALTINDA ESNEK İSTİNAT YAPILARI VE
ZEMİN ÇİVİLİ TOPRAKARME DUVAR TASARIMI**

YÜKSEK LİSANS TEZİ

**Ehsan SEYEDTEHRANI
(501121327)**

İnşaat Mühendisliği Anabilim Dalı

Zemin Mekaniği ve Geoteknik Mühendisliği Programı

Tez Danışmanı: Doç. Dr. Aykut ŞENOL

EKİM 2016

Ehsan-Seyedtehrani, a M.Sc. student of İTÜ Graduate School of Science Engineering and Technology student ID 501121327, successfully defended the thesis entitled “DESIGN OF MSE/SOIL NAIL HYBRID WALLS UNDER HIGH SPEED RAILWAY LOADS”, which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor : **Assoc. Prof. Dr. Aykut ŞENOL**
İstanbul Technical University

Jury Members : **Assist. Prof. Dr. Esra Ece BAYAT**
İstanbul Technical University

Assoc. Prof. Dr . Kubilay KELEŞOĞLU
İstanbul University

Date of Submission : 30 September 2016
Date of Defense : 6 October 2016



To my family,



FOREWORD

This master of thesis has been prepared for submission to Istanbul Technical University, Civil Engineering Department, Soil Mechanics and Geotechnical Engineering Programme.

I would like to envice my sincere thanks to my advisor Assoc. Prof. Dr. Aykut ŞENOL, for sharing his knowledge, for his direction and efforts to help me complete this thesis on time.

I would like to present my special thanks to Prof. Dr. Turan DURGUNOĞLU who has always been sensitive and instructive for the preparation of this thesis.

Special thanks are due to Mr. Eric LUCAS and to Mr. Murat ÖZBATIR, General Manager of Reinforced Earth A.Ş. Turkey, for their support through out this thesis.

October 2016

Ehsan SEYEDTEHRANI
(Civil Engineer)

TABLE OF CONTENTS

	<u>Page</u>
FOREWORD	ix
TABLE OF CONTENTS	xi
ABBREVIATIONS	xv
SYMBOLS	xvii
LIST OF TABLES	xix
LIST OF FIGURES	xxi
SUMMARY	xxv
ÖZET	xxvii
1. INTRODUCTION TO SOIL NAILS SYSTEM	1
1.1 Mechanism of Soil-Nailed Structures	1
1.2 Nail Tension	2
1.3 Shear Stress and Bending Stiffness in the Nails	4
2. MODES OF FAILURE OF SOIL - NAILED STRUCTURES	9
2.1 Internal Failure	10
2.1.1 Failure by nail breakage	11
2.1.2 Failure during excavation	11
2.1.3 Failure by pullout	11
2.1.4 Failure of facing	11
2.2 External Failure Modes	12
2.2.1 Sliding failure along the base	12
2.2.2 Bearing capacity failure	12
2.2.3 Overall slope failure	12
2.3 Deformation Analysis of Soil Nail Walls	12
3. DESIGN METHODS FOR SOIL NAIL WALLS	15
3.1 Limit-Equilibrium Method	15
3.2 Working-Stress Design Methods	15
3.3 German Gravity Wall Method	16
3.4 French Multicriteria Analysis	18
3.4.1 Shear resistance of the soil	18
3.4.2 Skin friction of the nails	19
3.4.3 Normal interaction between the soil and the nails	19
3.4.4 Strength of the nail	20
3.4.5 Strength of the nail	22
3.5 Kinematical Limit Analysis Design Method	22
3.6 FHWA 1996 Design Method	24
3.7 FHWA 2003 Design Method	26
4. SELECTION OF DESIGN PARAMETERS	29
4.1 Unit Weight	29
4.2 Shear Strength	30
4.3 Cohesive Soils – Drained Conditions	31
4.4 Cohesive Soils – Undrained Conditions	32
5. INTRODUCTION TO MSE WALL SYSTEM	33

5.1 Mechanism of Reinforced Earth Wall Structures.....	33
5.1.1 Advantages and potential disadvantages.....	35
Advantages of MSE walls:.....	35
Potential disadvantages;.....	37
5.2 Description of MSE System.....	37
5.2.1 Reinforcement geometry.....	38
5.3 Facing Systems.....	38
5.3.1 Welded wire grids.....	39
5.3.2 Post-construction facing.....	39
5.4 Reinforced Fill Materials.....	39
5.5 Reinforced Soil Design Criteria.....	41
5.5.1 Mode of reinforcement action and external stability.....	42
5.5.2 Sliding stability.....	43
5.5.3 Eccentricity limit check.....	48
5.5.4 Eccentricity criteria.....	49
5.5.5 Bearing capacity check.....	50
5.5.6 Settlement assessment.....	54
5.6 Internal Stability.....	55
5.6.1 Defining critical slip surface.....	55
5.6.2 Establish vertical layout of soil reinforcements.....	60
5.6.3 Calculation of factored tensile forces in the reinforcement layers.....	61
5.6.4 Maximum Tension Calculation.....	66
5.6.5 Calculation of soil reinforcement's resistance.....	67
5.6.6 Select grade of soil reinforcement elements at each level.....	68
5.6.7 Internal stability with respect to pullout failure.....	69
5.6.8 Checking connection strength.....	71
5.6.9 Connections to concrete panels.....	72
5.6.10 Lateral movements.....	72
6. INTRODUCTION TO THE SOFTWARE.....	75
6.1 Limit State Design.....	75
6.1.1 Ultimate limit state.....	75
6.1.2 Serviceability limit state.....	76
6.1.3 Assumptions in limit state calculations.....	76
6.2 Introduction to Valdez Software.....	77
6.3 Finite Element Method (FEM).....	77
6.4 Introduction to Plaxis Software.....	78
6.5 Introduction to Slide Software.....	78
6.6 Introduction to RocLab Software.....	79
6.7 Case Study Ankara – Sivas High Speed Railway Project-Reinforced Earth Walls.....	79
6.8 Design Criteria and Requirements for Valdez Anlaysia.....	81
6.8.1 Subsoil investigations.....	81
6.8.2 Selected backfill material.....	85
6.8.3 Loads and vibrations.....	88
6.8.4 Appurtenant materials of construction (EPDM pads).....	92
6.8.5 Adhesion failure.....	92
6.8.6 Governing load.....	93
6.9 Valdez Calculation of MSE Walls.....	94
6.9.1 Case One: Design of the wall with rectangular cross section.....	94
6.9.2 Case Two : Design of the wall with trapezoidal cross section.....	95

6.10 Case Three : Design of The Hybrid MSE-Soil Nail Wall.....	98
6.10.1 Step one: MSE wall design	100
6.10.2 Step Two: MSE nail design	103
6.10.3 Soil nail spacing	103
6.10.4 Soil nail inclination	103
6.10.5 Soil nail length	104
6.10.6 Bond strength	104
6.10.7 Pullout resistance per unit length	105
6.10.8 Tensile resistance of the tendon	105
6.11 Finite Element Analysis of MSE/ Hybrid Wall	106
6.11.1 Facing panel and shotcrete elements	106
6.11.2 Steel strips	107
6.11.3 Interface element	110
6.12 Pseudostatic Slope Analysis	111
6.13 Earthquake Potential Evaluation of the Region	112
6.14 Soil Properties and Soil Models Method.....	113
6.14.1 Linear elastic model (LE)	113
6.14.2 The Hardening Soil Model(HS).....	113
6.14.3 Hook Brown method for rock	114
6.14.4 Modulus of deformation.....	116
6.14.5 Mohr-Coulomb criterion	116
6.15 Parametric Study	117
6.15.1 Mobile loads.....	119
6.16 Safety Factors for MSE Wall Structure.....	119
6.16.1 Plaxis safety factor software	119
6.17 Analysis Methods of Slide Software.....	120
6.18 Analysis and Evaluations	121
6.18.1 Deformation analysis for MSE wall with rectangular cross section	121
6.18.2 Deformation analysis for MSE wall with trapezoidal cross section	124
6.18.3 Deformation analysis for hybrid MSE-soil nail wall	126
6.18.4 Safety factors for MSE walls	131
6.19 Parametric Study for MSE Wall with Rectangular Cross Section	132
6.19.1 Young's modulus and poisson's ratio effects in deformation of wall with rectangular cross section	134
Selected Backfill	135
6.19.2 Cohesion and friction angel effects in deformation of wall with rectangular cross section	136
7. RESULTS AND DISCUSSION	139
REFERENCES.....	145



ABBREVIATIONS

A_t	: Cross-sectional area of tendon
C_f	: Cohesion of the foundation soil
D_{DH}	: Diameter of the drill hole
D_g	: Diameter of grouted drill hole
F^*	: Pullout resistance factor which depends on depth
F_i	: Skin friction
I_0	: Transfer length
K_h	: Modulus of lateral subgrade reaction
L_a	: Embedment length in the resistance zone
L_e	: Length of embedment in the resisting zone
M_p	: Limit bending capacity of the nail
P_{max}	: Maximum passive resistance that can be mobilized in the soil
R_C	: Shear strength of the nail
R_C	: Ratio of the width of strip to the distance of center of the horizontal strip.
R_N	: Tensile strength of the nail
SF_1	: Factor of safety with respect to pullout resistance
S_H	: Horizontal spacing of the nail
S_V	: Vertical spacing of the nail
T_C	: Shear force acting on soil nail
T_{MAX}	: Maximum tension of reinforcements
T_N	: Normalized axial force of soil nail.
T_P	: Pullout resistance of the nail
T_S	: Axial tensile force acting on soil nail
W_n	: In-situ moisture content
f_Y	: Nominal yield resistance of tendon
I_0	: Transfer length
q_u	: Bond strength of the nail-grout-soil interface
f_b	: Bond coefficient of the soil (skin coefficient)
σ_V	: Unfactored vertical stress at the reinforcement level in the resistant zone
c	: Soil cohesion
D	: Diameter of drilled hole
E	: Young's Modulus
H	: Height of the soil nail wall
I	: Moment of Inertia
L'	: Effective foundation width
La	: Embedment nail length in the resistant zone



SYMBOLS

PGA	: Peak Ground Acceleration
RD	: Relative Density
SPT	: Standard Penetration Test
MSE	: Mechanically Stabilized Earth
NATM	: New Austrian Tunneling Method
FSP	: Factor of safety with respect to pullout
SLD	: Service Load Design
LRFD	: Load and Resistance Factor Design
FEA	: Finite Element Analysis
SLS	: Service Limit State
ULS	: The ultimate limit state
ASD	: Allowable Stress Design
EV	: Vertical earth pressure
LS	: Live load surcharge
ES	: Earth surcharge
LE	: Linear elastic model
HS	: Hardin soil Model
LS	: Live load surcharge
ES	: Earth surcharge



LIST OF TABLES

	<u>Page</u>
Table 4.1 : Correlations between SPT N-values, CPT, and friction angle of cohesion less soils – SPT (FHWA, 2015)	31
Table 4.2 : Correlations between SPT N-values, CPT, and Friction Angle of Cohesion less Soils – CPT (FHWA, 2015).	31
Table 5.1 : MSE selected granular reinforced fill requirement (AASHTO, 2007)...	40
Table 5.2 : Resistance factors, for tensile and pullout Resistance for MSE walls (AASHTO, 2007).	68
Table 6.1 : Rock properties of Basalt type 1 (Pazarcik Volkaniteleri).	83
Table 6.2 : Rock properties of Basalt type 2 (Pazarcik Volkaniteleri).	83
Table 6.3 : Soil properties of foundation output from Rock lab for Basalt type 1....	85
Table 6.4 : Soil properties of foundation output from Rock lab for Basalt type 2....	85
Table 6.5 : Selected backfill's grain size distribution according to percent of passing from sieve (KGM, 2013).	86
Table 6.6 : Maximum accepted soil parameters (Selected backfill 1) used in modeling the MSE wall in this project.	87
Table 6.7 : Minimum accepted soil parameters (Selected backfill 2) used in modeling the MSE wall in this project.	87
Table 6.8 : Soil Parameters used in the model (Hossain et al, 2012).	88
Table 6.9 : Soil Parameters Used in the Model (Damians et al, 2013)	88
Table 6.10 : Soil parameters used in the model (Hou, 2009).	89
Table 6.11 : Safety factor for internal and external stability of the rectangular MSE wall	96
Table 6.12 : Safety factor for internal and external stability of the Trapezoidal MSE wall.	99
Table 6.13 : Safety factor for internal and external stability of TerraLink™ MSE wall.	102
Table 6.14 : Calculation of soil-nail properties as geogrid.	103
Table 6.15 : Estimated Bond strength for Nails in rock-Drilling method, Rotary Drilled (Elias and Juran, 1991).	105
Table 6.16 : Elastic properties of reinforced concrete facing panels.	109
Table 6.17 : Elastic properties of shotcrete materials.	109
Table 6.18 : Equivalent layer rigidity (KN/ml) for various type and number of strips	110
Table 6.19 : Recommended horizontal seismic coefficients (Melo and Sharma, 2004).	112
Table 6.20 : Input parameters of HS model (Plaxis Material Models Manual, 2012)	114
Table 6.21 : Variable properties of soil in parametric study.	118
Table 6.22 : Minimum selected backfill properties of selected backfill in this project.	118
Table 6.23 : Total displacements in soil mass and the total displacement of facing panels in -X direction.	123

Table 6.24 : Guide to the effects of settlements (BS 8006-1, 2010).	123
Table 6.25 : Total displacements in soil mass and the total displacement of facing panels in -X direction.	126
Table 6.26 : Total displacements in soil mass and the total displacement of facing panels in -X direction with direct connection.	129
Table 6.27 : Total displacements in soil mass and the total displacement of facing panels in -X direction.	131
Table 6.28 : Safety factors for three geometry and two type of loads for mentioned selected backfills.	135



LIST OF FIGURES

	<u>Page</u>
Figure 1.1 : Location of maximum tensile forces in soil nails (FHWA, 2015).	3
Figure 1.2 : Elastic analysis for soil-nail interaction (Schlosser, 1983).	5
Figure 1.3 : Constant Modulus of lateral subgrade reaction (Alhabshi, 2006).	6
Figure 2.1 : Principal failure modes in soil nail walls (Byrne et al, 1998).	9
Figure 2.2 : Mode of failures in soil nail walls (Lazarte et al, 2003).	10
Figure 2.3 : Deformation of soil nail walls (Clouterre, 1991).	14
Figure 3.1 : German gravity wall methods (Gassler and Gudehus, 1981).	17
Figure 3.2 : German method: Design chart for stability calculations (Gassler and Gudehus, 1981).	17
Figure 3.3 : Long nail subjected to combined loading (Elias and Juran, 1991).	20
Figure 3.4 : Multi criteria slope stability analysis method (Elias and Juran, 1991 and Clouterre, 1991).	21
Figure 3.5 : Kinematical limit analysis method: design assumptions (Elias and Juran, 1991).	23
Figure 3.6 : Kinematical method: Design charts for perfectly flexible nails, $N = 0$ (Juran et al, 1990).	24
Figure 3.7 : Nail tension distribution diagram (Byrne et al, 1998).	26
Figure 3.8 : Construction stability of soil nail wall (Byrne et al, 1998)	27
Figure 3.9 : Preliminary design chart for soil nail walls (Byrne et al, 1998).	27
Figure 3.10 : Preliminary design chart for soil nail walls (Lazarte et al, 2003).	28
Figure 4.1 : Estimation of soil unit weight (Kulhawy and Mayne, 1990).	30
Figure 4.2 : Friction angle of cohesion less soils as a function of normalized overburden (Schmertmann, 1975).	32
Figure 5.1 : Connection parts in steel strips (Ekli, 2006).	34
Figure 5.2 : Soil and reinforcement interaction (FHWA, 2009).	34
Figure 5.3 : Sea-Tac Airport - 3rd Runway, WA-reinforced earth company (Reinforced Earth company, 2009).	36
Figure 5.4 : The 1999 kocaeli earthquake in turkey, Reinforced earth MSE wall (Reinforced earth company, 1999).	37
Figure 5.5 : Stress transfer mechanisms for soil reinforcement (FHWA, 2009)	42
Figure 5.6 : Potential external failure mechanisms for a MSE wall (FHWA, 2009).	43
Figure 5.7 : External analysis: nominal earth pressures; horizontal back slope with traffic surcharge (AASHTO, 2007).	44
Figure 5.8 : External analysis: earth pressure; sloping backfill case (AASHTO, 2007).	44
Figure 5.9 : External analysis: earth pressure; broken back slope case (AASHTO, 2007).	45
Figure 5.10 : Calculation of eccentricity and vertical stress for bearing check, for horizontal back slope with traffic surcharge condition (FHWA, 2009). ..	49
Figure 5.11 : Calculation of eccentricity and vertical stress for bearing check, for sloping back slope condition (FHWA, 2009).	50

Figure 5.12 : Location of potential failure surface for internal stability design of MSE Walls inextensible reinforcements and (b) extensible reinforcements (FHWA, 2009).....	56
Figure 5.13 : Variation of the coefficient of lateral stress ratio (K_r/K_a) with depth in a MSE wall (Elias and Christopher, 1997; AASHTO, 2002; AASHTO, 2007).....	58
Figure 5.14: Notation for Coulomb active earth pressures used with wall batters, θ , greater than 100 degree (AASHTO, 2007).	58
Figure 5.15 : Calculation of vertical stress for sloping backfill conditions for internal stability (FHWA, 2009).....	59
Figure 5.16 : Distribution of stress from concentrated vertical load for internal and external stability calculations (FHWA, 2009).....	64
Figure 5.17 : Distribution of stresses from concentrated horizontal loads (FHWA, 2009).....	65
Figure 5.18 : Reinforcement load contributory height (FHWA, 2009).	66
Figure 5.19 : Nominal vertical stress at the reinforcement level in the resistant zone, Beneath a sloping backfill (FHWA, 2009).....	70
Figure 5.20 : Empirical curve for estimating lateral displacement during construction for MSE walls (Christopher et al, 1990).	73
Figure 6.1 : Active and Resisting Zones in Reinforced Earth Structures (Liang, 2004).....	76
Figure 6.2 : Typical-Cross section of MSE wall for high-speed railways done by Reinforced earth company.....	80
Figure 6.3 : Plan view of the project part 6- KM: 289 (Reinforced earth company, Turkey project 2016).	81
Figure 6.4 : Geological profile of cross section wall KM: 0+289.	82
Figure 6.5 : Major principle stress plot minor principle stress for rock lab for Basalt type1.	84
Figure 6.6 : Normal stress plot shear stress from rock lab for Basalt type 1.	84
Figure 6.7 : Cross-section of the instrumented structure at CER in Rouen (Soyez, 2009).....	91
Figure 6.8 : Typical EPDM pad is used in MSE walls.	92
Figure 6.9 : Global stability of wall by considering 22.5 KN for each line.....	93
Figure 6.10 : Cooper E 80 axle load diagram (AREMA Manual, 2007)	94
Figure 6.11 : Screen shot of the wall with rectangular cross section from Valdez software	95
Figure 6.12 : Trapezoidal cross section type of reinforcement geometry (FHWA, 2009).....	97
Figure 6.13 : Screen shot of the wall with Trapezoidal cross section from Valdez software.	98
Figure 6.14 : Screen shot of the wall with Terralink TM cross section from Valdez software.	101
Figure 6.15 : Total displacement of Soil nail wall by Plaxis.	104
Figure 6.16 : Incremental displacement of wall with rectangular cross section.	107
Figure 6.17 : Incremental displacement of wall with trapezoidal cross section.	108
Figure 6.18 : Incremental displacement of wall with Terralink TM cross section.	108
Figure 6.19 : Pseudostatic analysis approaches (Melo and Sharma, 2004).	112
Figure 6.20 : Turkey Seismic Risk Map (2006).....	113
Figure 6.21 : Hyperbolic stress strain response curve of hardening soil model.	114

Figure 6.22 : Minor principal stress plot major principal stress (Hoek et al, 2002).	117
Figure 6.23 : Plasticisation (crosses) in the earth retained by the reinforced soil structure with inextensible reinforcing strips (Freitag et al, 2011).	119
Figure 6.24 : Total displacements in -X direction for 100KN (Plaxis 2016).	122
Figure 6.25 : Total displacements of facing panels in -X direction for 100 KN, (Plaxis 2016).	122
Figure 6.26 : Total facing panel displacements in -Y direction for 100 KN (Plaxis , 2016).	124
Figure 6.27 : Total displacements in -X direction for 100KN, (plaxis 2016).	125
Figure 6.28 : Total displacements of facing panels in -X direction for 100KN, (plaxis 2016).	125
Figure 6.29 : Total facing panel displacements in -Y direction for 50 KN. (Plaxis ,2016).	127
Figure 6.30 : Total displacements of facing panels in -X direction load, Hybrid MSE wall with direct connection (Plaxis 2016).	128
Figure 6.31 : Total displacements in -X direction for 100 KN load, Hybrid MSE wall with direct connection (Plaxis 2016).	128
Figure 6.32 : Total displacements in -X direction for 100KN load, Hybrid MSE wall without direct connection (Plaxis 2016).	130
Figure 6.33 : Total displacements of facing panels in -X direction load, Hybrid MSE wall without direct connection, (Plaxis 2016).	130
Figure 6.34 : Total shotcrete displacements in -X direction for Hybrid MSE wall with direct connection and 100 KN load.	132
Figure 6.35 : Axial forces in the soil nail before the friction element and strips are connected.	133
Figure 6.36 : Soil nail under 100 KN load and with connection to friction elements and strips.	133
Figure 6.37 : Screen shot of safety factor calculation by slide 5.0 software	134
Figure 6.38 : Effect of modulus of elasticity in MSE wall displacements.	136
Figure 6.39 : Effect of Poisson's ratio in MSE wall displacements.	137
Figure 6.40 : Effect of cohesion in MSE wall displacements.	137
Figure 6.41 : Effect of friction angel (degree) -Dilatancy angel in MSE wall displacements.	138



DESIGN OF MSE/SOIL NAIL HYBRID WALLS UNDER HIGHSPEED RAILWAY LOADS

SUMMARY

This research is focused on numerical modeling of Ankara-Sivas High speed railway project – The high speed trains will pass above the wall with the maximum height of wall 14 meters. The sensitivity of the project leads to forecast the wall deformations, by numerical modeling of reinforced earth wall for two types of train.

This is the first time the Reinforced earth wall has been used for High-speed railway support in Turkey. All the design technology and data which has been used and expressed here is exclusively belongs to Reinforced Earth Company and copying without permission is prohibited.

The preliminary designs were prepared by the Reinforced Earth Company using the Valdez program, The Valdez program is specifically prepared to analyze the MSE wall for the design of Reinforced Earth walls. This program is user friendly and provides two types of design cross section rectangular and trapezoidal. Furthermore you can design tiered wall with various setback for wall steps. The method of design of Valdez program is based on the limit state design procedures, which has just given the length of strips by applying defined safety factors. The program is not able to estimate the displacements in the walls because as previously mentioned, the designs were based on “Limit State” methods. Which is using the limits of the materials in design but does not take into account the strains formed by stresses. Therefore a new method to calculate the displacements is needed.

In this project because of the sharp cuts which are formed by Basalt, the preliminary designs were done in three alternative and three individual methods. Because of the importance of the project and heavy loads, this project was prepared by three alternative geometry and a numerical analysis of the reinforced walls system. Finite element method of analysis, are capable of calculating the displacements in any part of the defined system. This is done by the finite element program Plaxis 2016.

The reinforced earth walls design, will design according to limit state method, and then will check by the numerical analysis to control the deformation. In the evaluating of the design parameters the Rock lab program is used, in addition the slide 5.0 is used to calculate the stability and safety of the project.

The first case is the MSE wall with rectangular shape geometry. For the better understanding of the working system, the mentioned geometry is analyzed according to maximum and minimum material properties which is defined from company as a selected back fill material for this case, in addition for effect of other backfill properties from other cases and parametric comparison with them, three selected backfills are taken from three articles which is the properties of selected backfill is so close to this case.



YÜKSEK HIZLI TREN YÜKLERİ ALTINDA ESNEK İSTİNAT YAPILARI VE ZEMİN ÇİVİLİ TOPRAKARME DUVAR TASARIMI

ÖZET

Bu tez çalışması, “Ankara-Sivas Hızlı Tren Projesi” kapsamında inşaa edilen, maksimum yüksekliği 14 metre olan bir donatılı toprak duvar (toprakarme) yapısının karşılaştırmalı nümerik analiz hesaplarını içermektedir. Donatılı toprak duvar hızlı tren hattını taşıyacak şekilde tasarlanmıştır. Tasarım aşamasında iki farklı tipte trene göre deformasyon analizleri numeric analizler vasıtasıyla hesaplanmıştır. Söz konusu yapı, Türkiye’de hızlı tren hattı gövdesini tutan ilk donatılı toprak duvar olması açısından önemlidir. Çalışma kapsamında detaylı bir şekilde anlatılan tasarıma ait bütün veriler Reinforced Earth İnşaat Proje ve Ticaret A.Ş. ‘ye aittir. Bu şirketin izni dışında verilerin kopyalanarak kullanılması yasaktır.

Çalışma kapsamında ön tasarım, donatılı toprak duvarların projelendirilmesinde kullanılan Reinforced Earth şirketine ait “Valdez” isimli yazılım yardımıyla yapılmıştır. Valdez program, hesap kesiti boyunca sabit (dikdörtgen kesit) veya değişken boylarda (trapez kesit) donatıların kullanımına izin veren kullanışlı bir analiz programıdır. Program, kademeli duvar tasarımında da kullanılabilir. Hesap yöntemi olarak “Limit Durum Metodu” esasını kullanan program, tanımlanan güvenlik faktörlerini dikkate alarak, tanımlanan geometriye ve zemin koşullarına göre donatı boyları önermektedir. Program limit durum yöntemini esas aldığından dolayı, mevcut gerilmelerden kaynaklanan deformasyonları hesaplayamamaktadır. Bu sebepten dolayı, deformasyonların hesabı için, limit durum yöntemine ek olarak yeni bir metot esas alınmalıdır.

Bu çalışma kapsamında, yapının demiryolu yüklerine maruz kalacağından ötürü üç farklı yöntem ve geometri ile dizayn çalışmaları yürütülmüştür. Deformasyonların hesabında sonlu elemanlar metodu ile analiz yapılmasına imkan veren “Plaxis” yazılımı kullanılmıştır.

Tasarım aşamasında ilk olarak limit durum yöntemi esas alınarak stabilite analizleri yapılmış, nümerik analizle yapılan hesaplamalar sürdürülmüştür. Tasarım parametreleri Rocklab yazılımı vasıtasıyla Slide 5 programına tanımlanarak, yapının güvenliği ve stabilitesi kontrol edilmiştir. Tasarım kesiti olarak, donatılı toprak duvar yapısının projede yer alan en yüksek kesimi seçilmiştir.

İlk olarak dikdörtgen kesitli geometri analiz edilmiştir. Analizlerde dolgu malzemesinin tasarıma etkisini gözlemek amacıyla, farklı geoteknik parametrelerle tasarım tekrarlanmıştır.

İkinci durum için, trapez kesitli geometri analiz edilmiştir. Donatılı toprak duvar yapısında üst dolgu tabakalarından aşağı tabakalara inildikçe donatıların kopma dayanımı arttığından, üst tabakalarda uzun, alt tabakalarda kısa boylu donatılar kullanılmıştır. Duvar yapısının sağlam zemine oturduğu durumlarda bu geometri avantaj sağlamaktadır.

Üçüncü geometri, donatılı toprak duvar yapısının gerisinde zemin çivisiyle birlikte teşkil edildiği durumu incelemektedir. Arazinin kazılabilirlik açısından sorunlu olduğu bölgelerde, taban şeritlerinin uzatılması mümkün olmadığından bu sistem dikkate alınmıştır. Bu yolla duvar taban bölgelerindeki kazı kübajında tasarruf sağlanabilmektedir. İhtiyaç duyulan mukavemet, taban bölgelerinde zemin çivileri vasıtasıyla sağlanmaktadır.

Son geometride, iki farklı tipte taban zemini için hesap yapılmıştır. İlk zemin tipi, patlatma ile kazı yapılabilen, sert Bazalt olarak tanımlanmıştır. Bu zemin geometride taban seviyesinden itibaren 2. Şerit seviyesine kadar seyretmektedir. İkinci zemin tipi, tabandaki ilk tabakaya nazaran ayrılmış, ancak kazılabilirliği düşük basalt malzemedir. Şeerit boyları dikkate alınarak geleneksel bir çözüm sistemi izlendiğinde, yüksek miktarda kazı kübajı ortaya çıkmaktadır. İmalatın hızlandırılması ve donatı boylarından tasarruf seçenekleri göz önüne alındığında, zemin çivileri ve duvar dolgusu içerisinde yer alacak merdiven tipi ek donatılarla desteklenmiş bir çözüm yöntemi tercih edilerek analizleri yapılmıştır. Zemin çivili toprakarme istinat duvarı olarak tanımlanabilecek bu sisteme TerraLink™ adı verilmektedir.

TerraLink™ sistemine ait elemanlar, geleneksel donatılı toprak duvar sistemine ait elemanlarla benzerlik göstermektedir. Bu sisteme ait donatı elemanları, serme sıkıştırma yapılarak elde edilen geri dolgu tabakaları arasına teşkil edilmektedir. Bu elemanların temel özelliği, seçilmiş dolgu arkasındaki zemin çivisi yüzeyi ile mevcut donatılı toprak duvar gövdesi arasında süreklilik sağlayarak, donatı boyunun geleneksel yöntemle göre kısa geldiği kesimlerde dayanımı sağlamasıdır.

Bu çözüm yönteminin tercih edilmesi, topoğrafik koşullara, mevcut zeminin geoteknik parametrelerine, inşaa edilecek donatılı toprak duvarın geometrisine bağlıdır. Bu koşullar göz önüne alındığında, teknoloji iki farklı bağlantı yöntemi kullanılarak uygulanabilmektedir:

İlk yöntem sürtünme kuvveti esasına dayanan bağlantı yöntemidir. Bu sistemde, yeterli kazı mesafesinin elde edilememesinden ötürü, yardımcı bağlantı elemanları ile kısa şeritler mesafe olarak birbirlerine bindirilerek dayanım sağlanmaktadır. Bu sistem donatılı toprak duvarın esnek yapısının korunmasının yanında, ek donatılar ile şeritlerin bindirilmesine imkan verildiği sürece ekonomi de sağlamaktadır.

İkinci yöntemde donatı şeritlerinin kazı yüzeyine direk bağlantısı sağlanır. Bu sistem kazı mesafesinin çok dar olduğu kesitlerde tercih edilir. İlk çözüme nazaran daha rijit bir yapı oluşturur. Çalışma kapsamında her iki tip bağlantı yöntemi için de deformasyon analizleri yapılmış, sonuçlar karşılaştırılmıştır.

Yukarıda çalışma prensibi özetlenen Valdez yazılımı, hesap kesitlerinde zemin çivisinin tanımlanmasına imkan vermemektedir. Bunun yanında, valdez program yardımıyla elde edilen iç ve dış stabilite analizlerinin sonuçlarından hareketle, zemin çivisi ve ek donatıların tasarımında faydalanılabilecek veriler elde edilmiştir.

Bahsi geçen tüm modeller tasarlandıktan sonra, deformasyon analizlerinde dikkate alınan elastik ve plastik parametrelerin, yapının davranışına olan etkisi irdelenmiştir. Tez çalışması kapsamında klasik donatılı zemin duvar elemanlarının davranışının, zemin çivisi gibi destekleyici yapılarla birlikte çalışma prensipleri irdelenmiştir. Yüksek hızlı tren yükleri gibi tasarımı kompleks hale getiren şartlar ile, kazı probleminin ortaya çıktığı topografik koşullar altında sistem tasarımı ele alınmış, geleneksel tasarım yöntemleri ile mukayese edilmiştir.

Yukarıda bahsedilen farklı geometrik koşullarda ve sürşarj yükleri altında sürdürülen analizler, farklı geoteknik parametreler dikkate alınarak, nümerik analiz programları yardımıyla karşılaştırmalı olarak tekrarlanmıştır. Çalışma kapsamında özet olarak, donatılı toprak duvar sistemlerinde kullanılan donatıların farklı zemin koşulları altında tasarıma etkisi, deformasyonların durumu irdelenmiştir.

Bu çalışmadan hareketle, klasik donatılı zemin duvarların tasarım konseptinin; farklı yükleme koşulları, geometrik kriterler, farklı donatı tipleri ve zemin koşulları altında farklılaşması incelenmiştir. Özellikle kompleks geometrili ve ağır yükler altındaki yapıların performansının analiz edilmesi açısından yapılan çalışma önemli bulgular ortaya koymaktadır. Tasarım literatüründe geleneksel yaklaşımların yanında, sistemin klasik yaklaşımlarla çözülemediği projelerin tasarımında bu karşılaştırmalı tez çalışmasının faydalı olabileceği düşünülmektedir.





1. INTRODUCTION TO SOIL NAILS SYSTEM

The concept of in soil reinforcement dates is back to the ancient times and it is very old. People in ancient cultures for better resistance of their houses and other religious structures against the natural disasters and war used sticks and branches for reinforcing mud that they used.

In recent years, several techniques and method of soil stabilization have been developed such as soil nailing and Mechanically Stabilized Earth (MSE). These two systems are well suited to the needs of highway construction and reconstruction.

Soil nailing method was used for a first time in Europe in the 1960s with the introduction of the New Austrian Tunneling Method (NATM). This method come forth from rock bolting that applied bonded steel bars and shotcrete to support tunnels.

The first construction of soil nailing was done in France as a permanent support of retaining walls in a cut in soft rock in 1961. In 1972, the French contractor Bouygues and the specialist contractor Soletanche constructed the first soil nail wall in France. The wall height was 18 meters and slope was built in Fontainebleau sand, it was a part of project of a railway widening near Versailles.

1.1 Mechanism of Soil-Nailed Structures

The principal concept of soil nailing is based on improving the stability of soil mass which relies upon two possible systems.

The first mechanism is the transfer of tensile forces which is generated in the nails through frictional interaction between the ground and the soil nail.

The second main notion is the deformation of soil mass, which is caused by the shear stress and bending stiffness in the nails beside the tensile forces and shear stress and bending stiffness in the nails, In addition, the interaction between soil structure and facing. The soil helps to repel the excessive displacement, limit decompression

during and after excavation, and produce nail head load between the nail and the facing and is necessary to develop the force along the nails (Byrne et al, 1998).

1.2 Nail Tension

Soil nailing technique and reinforced soil systems function in a composite soil mass is similar in function. Tensile forces in passive zone resist the deformation of active zone. The line of the maximum tensile forces in the nails separates the reinforced soil mass into two zones, an active zone and a passive or resistance zone, where the maximum tensile force always coincides with the potential slip surface.

The active zone at the reinforced soil mass is located near the facing. In the active zone, the main tension forces increase because of the pullout action which is caused by the mobilized lateral shear stress close to the facing and the directed outward.

The passive zone is a stable zone which opposite the active zone the shear stresses are inwards and tend to keep the nails and reinforcement being pullout. Therefore, this zone has a vital function in stability.

The nail act is like a tie to keep the active zone and resistant zone together. On the one hand we have the mobilized tensile force (T_{max}) which must be balanced by the effective friction in the passive zone. On the other hand, an adequate nail length that is embedded in soil must be provided in order to increase the effective friction in the soil-nail interface in the resistant zone and prevent the pullout failure. Moreover, the pullout resistance (T_p) along with the nail head strength must be adequate to provide the required nail tension at the slip surface.

The (T_{max}) can be evaluated as follows (Figure 1.1):

$$T_{MAX} \leq \frac{T_P}{SF_1} = \frac{D\pi L_a F_1}{SF_1} \quad (1.1)$$

Where:

T_P : Pullout resistance of the nail

SF_1 : Factor of safety with respect to pullout resistance (1.75 ~ 2.0)

D : Diameter of drilled hole

L_a : Embedment length in the resistance zone

F_1 : The limit interface lateral shear stress obtained from pullout tests

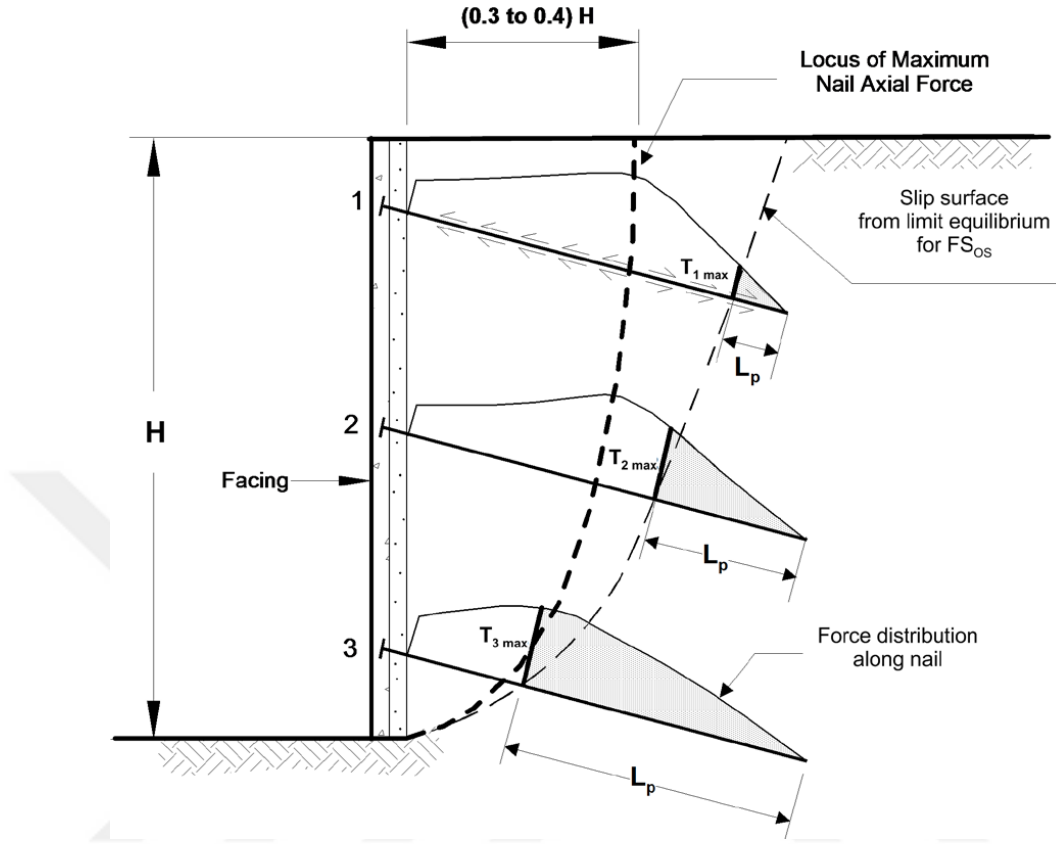


Figure 1.1 : Location of maximum tensile forces in soil nails (FHWA, 2015).

The limit interface lateral shear stress is obtained by considering the Jewell and Pedley (1992) suggestion, he mentioned that the limit lateral shear stress or bond stress can be mobilized at the contact between nail surface and soil mass. The bond stress and shear resistance of the soil is assumed to be directly proportional, thus the relation below is given by Jewel:

$$F_1 = \sigma' f_b \tan \phi' \quad (1.2)$$

$$0.7 \leq \frac{\sigma'_r}{\sigma'_v} \leq 1 \quad (1.3)$$

Where:

f_b : Bond coefficient of the soil (Skin Coefficient)

f_b : 1 for fully rough soil-grout interface

f_b : (0.2 – 0.4) for smooth interface

σ_r' : Normal effective stress acting on the circumference of the nail

σ_v' : Effective normal stress

1.3 Shear Stress and Bending Stiffness in the Nails

By the movement of the active zone adjacent to the stable resistance mass, a shear zone is developed within the soil nail where deformation is centralized. The movement of the soil mass relative to the resistance mass, subjects the soil nails to bending moment and shear forces in addition to axial tensile forces. The bond strength is affected by:

- Ground condition around the nail.
- Size of the grouted zone.
- The construction of soil nail, which is consisted of drilling method, grouting procedure, injection and pressure of injection, affects the bending strength.

The relation between the nail in soil mass, when the movement of the soil mass loads in the nails, are perpendicular to its long axis which is similar to the laterally loaded piles system.

For laterally loaded piles, Poulos and Davis (1980), develop a relation to find the effect of bending stiffness by using the simplified method of the coefficient of subgrade reaction. According to the FHWA manual for design of soil nail wall, shear and bending in the nails are not considered because their benefits in comparison with the nail grout ultimate adhesion and minimum axial force are negligible and small.

By considering the theory of Elasticity, the bending moment in the nail is equal to zero at the surface of failure when the tension and shear forces are maximum (Figure 1.2).

$$N = \frac{K_H D I_0^2}{\gamma H S_H S_V} \quad (1.4)$$

$$I_0 = \sqrt[4]{\frac{4EI}{K_h D}} \quad (1.5)$$

Where:

I_0 : Transfer length

E : Young's modulus

I : Moment of inertia

D : Diameter of soil nail

H : Height of soil nail wall

S_V And S_H : Vertical and horizontal spacing of the nail

K_h : Modulus of lateral subgrade reaction (Figure 1.3).

γ : Unit weight of soil

The bending stiffness number, N value for practical structures varies from 0.1 to 1.5. The maximum bending is generated at the relative rigidity of the nail at the distance of the $(\frac{\pi}{4}I_0)$ from the shear surface; this relative stiffness is defined as the transfer length.

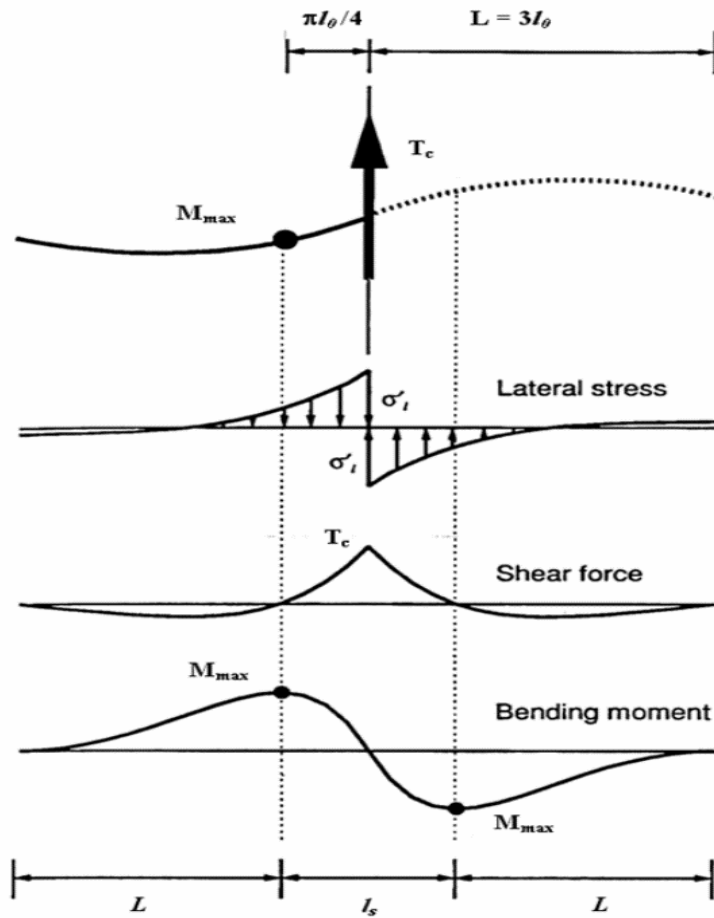


Figure 1.2 : Elastic analysis for soil-nail interaction (Schlosser, 1983).

I_s : Distance between points of maximum moment

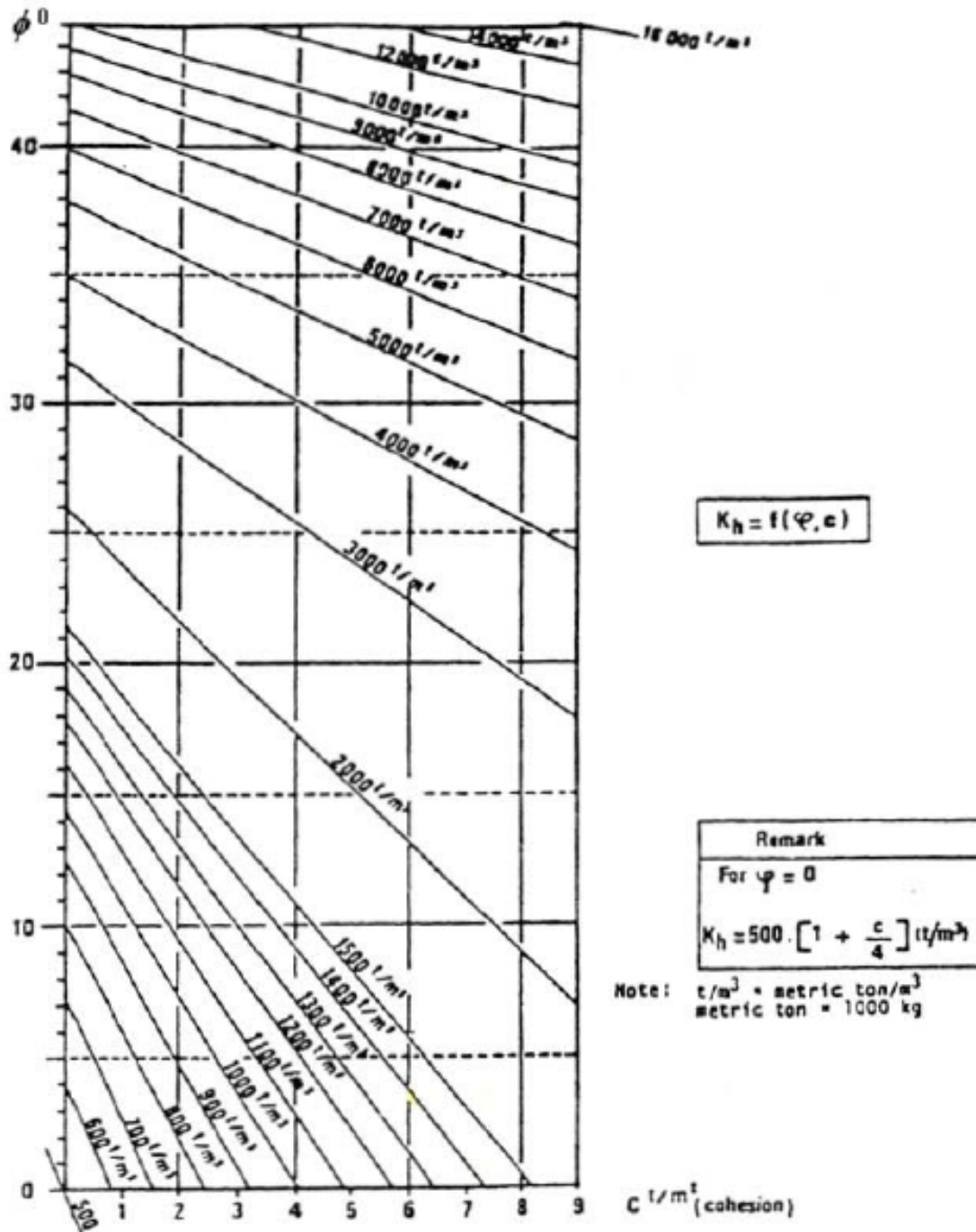


Figure 1.3 : Constant Modulus of lateral subgrade reaction (Alhabshi, 2006).

The shear forces mobilized in the nail can be evaluated using the following non-dimensional parameters.

$$T_N = \frac{T_{MAX}}{\gamma H S_H S_V} \quad (1.6)$$

$$T_S = \frac{T_C}{\gamma H S_H S_V} \quad (1.7)$$

And we can obtain shear force acting on soil nail by the following relationship:

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 \leq 1 \quad (1.8)$$

Where:

T_N : Normalized axial force of soil nail

T_S : Normalized shear force of soil nail

T_N : Axial tensile force acting on soil nail

T_C : Shear force acting on soil nail

R_N : Tensile strength of the nail

R_C : Shear strength of the nail

By considering the elastic solution described by Schlosser (1983) and Michelle and villet the estimation of the maximum bending is possible by knowing the value of shear force which is acting on soil nail.

$$M_{max} = \frac{T_c I_s}{4.9} \quad (1.9)$$

I_s : Distance between points of maximum moment



2. MODES OF FAILURE OF SOIL - NAILED STRUCTURES

Studies and experiment based on several building sites and instrumented full-scaled CLOUTERRE project, soil nail walls show three types of failure modes. As shown in Figures 2.1 and 2.2, these failure modes include:

I) External modes with potential failure surface passing completely behind the reinforced zone.

II) Internal modes, which is the failure surface, passing either the nail bars or the facing or both.

III) Mixed modes of internal and external failure that involves internal failure of the reinforced zone beside the extended failure surface beyond the reinforced zone of the soil mass.

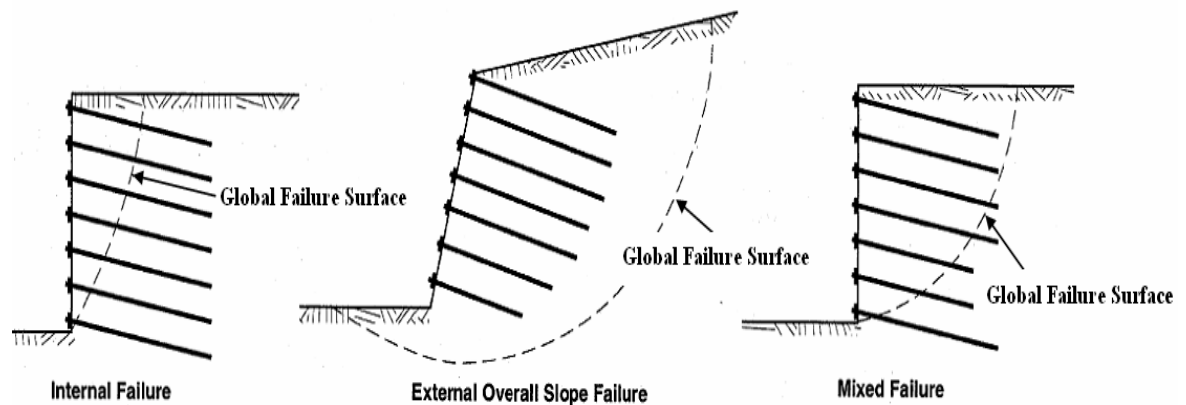


Figure 2.1 : Principal failure modes in soil nail walls (Byrne et al, 1998).

Failure modes will depend primarily on the condition of the wall. For instance, during construction, the main problem will be the stability of excavation part, problems may include excessive surcharge load which may be accrue by local water and ground water table, although in design case ,the engineer must consider drainage system but during construction sudden rise of water may cause stability problems. After construction, types of failure will be mobilized bending stiffness and shear forces as illustrated by internal failure in Figure 2.2. After construction, the long term

concerns may include functions of drainage system, grout creep and corrosion of the nail bars. Here there would likely be some settlement with mobilization of bending stiffness and shear forces along with the development of shearing zone within or behind the reinforced mass (FHWA, 2009).

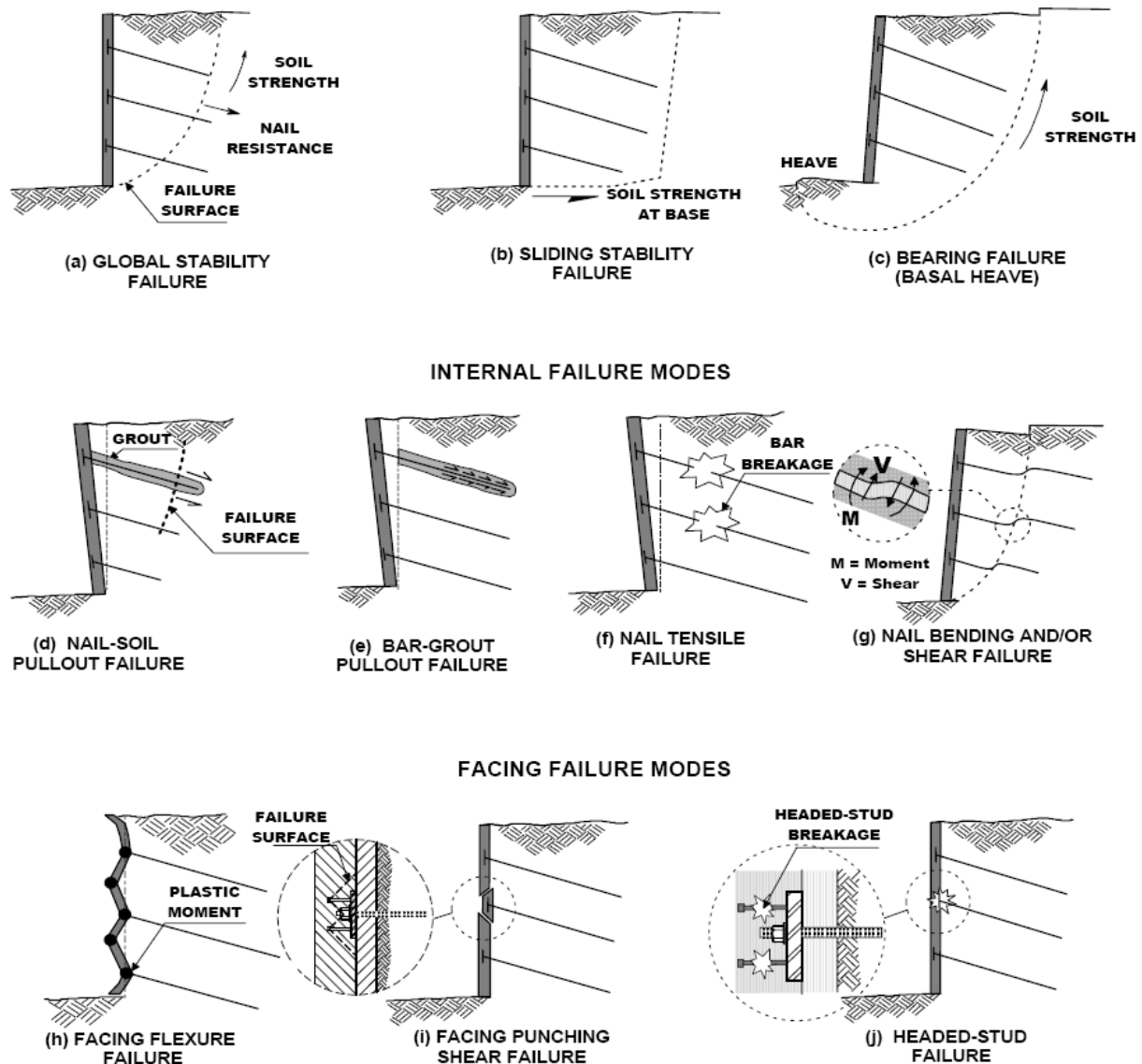


Figure 2.2 : Mode of failures in soil nail walls (Lazarte et al, 2003).

2.1 Internal Failure

Internal failure mechanisms related to soil nails, with which we commonly confronted, is nail pullout failure. This failure occurs through the soil-nail interface due to deficient intrinsic bond strength or insufficient nail length in the stable zone .Inadequate tensile strength of the soil nail bar causes tensile failure and slippage of the bar-grout interface.

2.1.1 Failure by nail breakage

In this case, when the stress in soil nail reaches its maximum capacity, deformation occurs. A slip surface may be developed because of full mobilization of shear strength of the soil. Consequently, the nails are subjected to tensile forces and bending moments and shear forces; thus, insufficient nail cross section or corrosion in steel bar or any excessive surcharge load, leads to soil nails failure. Usually, this type of failure happens suddenly and with no warning.

2.1.2 Failure during excavation

In this case, we normally have two types of failure; both are similar with different causes. The first failure will occur when the height of the soil cut is comparatively large. It happens suddenly due to local instability and propagation to the top. The general backfill properties like friction angle, ground water table and cohesion have the main role in estimating stability problem in this case. This instability is caused by continuous flow of soil behind the wall facing due to consecutive elimination of arching effect. In order to prevent such a failure, the height of the excavation must be done between the ranges 1.5 for each stage.

The second failure may occur due to presence of water pocket in soil. During excavation, water seepage and pore water pressure weaken the soil and cause failure in soil due to the sudden flow of the soil.

2.1.3 Failure by pullout

This failure is more common and happens because of inadequate shear resistance between soil and soil around the nail. The first reason is underestimating the bond stress or pullout capacity of nail .The second reason is a defect in shear resistance caused by insufficient embedded length in the resistance zone. The third failure is the consequence of saturation of cohesive soil because this way, the shear effective strength of soil diminishes.

2.1.4 Failure of facing

Failure of the facing usually occurs due to inadequate structural design or low quality of used materials. The most frequent failure mechanisms at the facing-nail head connection are:

- Failure due to excessive bending moment beyond the facing flexural capacity of the facing.
- Failure due to punching shear of facing around the nails.
- Failure of headed-studs in tension particularly in permanent facing.

2.2 External Failure Modes

The external stability refers to the overall stability of the soil nail wall. This failure is advert to mechanisms in which relatively large failure surfaces are developed in the soil. When soil nails contribute to stability, the main issue is global stability failure. If the failure surface passes under and behind the soil nails and bearing failure in soft soils does not cross the soil nails, they do not contribute to stability. The evaluation of the external stability is an important aspect in the design of soil nail walls because the magnitude of the failure can be significant and may cause major collapse. Three main reasons of external failure are listed below (Alhabshi, 2006).

2.2.1 Sliding failure along the base

Sliding along the base or a failure surface, under the influence of the lateral earth pressure applied by the ground retained behind the reinforced zone.

2.2.2 Bearing capacity failure

This failure is usually associated with overturning and global failure. Due to poor quality of foundation soils or deficient soil nail length, bearing capacity failure will take place under the compound effect of structure (self-weight) and lateral earth pressure.

2.2.3 Overall slope failure

Overall slope failure will occur in the ground of soil nail wall or in deep layer of soil nail foundation.

2.3 Deformation Analysis of Soil Nail Walls

A soil nail wall is a passive reinforcement system, and some deformation of the wall should be expected during its service life but limitation of deformation is a vital part

of the design. For instance, in some cases, a railway or a life line passes over the wall or on top of the wall so excessive deformation may disrupt the function of these facilities. Soil nail wall and the soil behind tend to deform outwards during and after construction. Most of the movement of soil nail walls will occur shortly after the excavation. Post-construction deformation is generally small unless there is a problem in stability of the whole wall. Creep in the fine-grained retained soils may take place with the consequent increase of tensile forces in the soil nails and deflection of the wall after construction.

Lateral deflections are generally larger for taller walls and increase by increasing of nail distance from each other; the recommendation is the nailing must done between 1-1.5 m for each layer, nail inclination, and surcharge magnitude has the same effect on lateral deformation. Vertical displacements are also affected by similar factors. They are generally largest near the facing and commonly smaller than lateral deflections measured on the top of the wall.

The maximum long-term horizontal and vertical displacements on top of the wall, δ_h and δ_v , can be estimated as follows (Clouterre, 1991):

$$\delta_v \approx \delta_h = \left(\frac{\delta_h}{H} \right)_i \times H \quad (2.1)$$

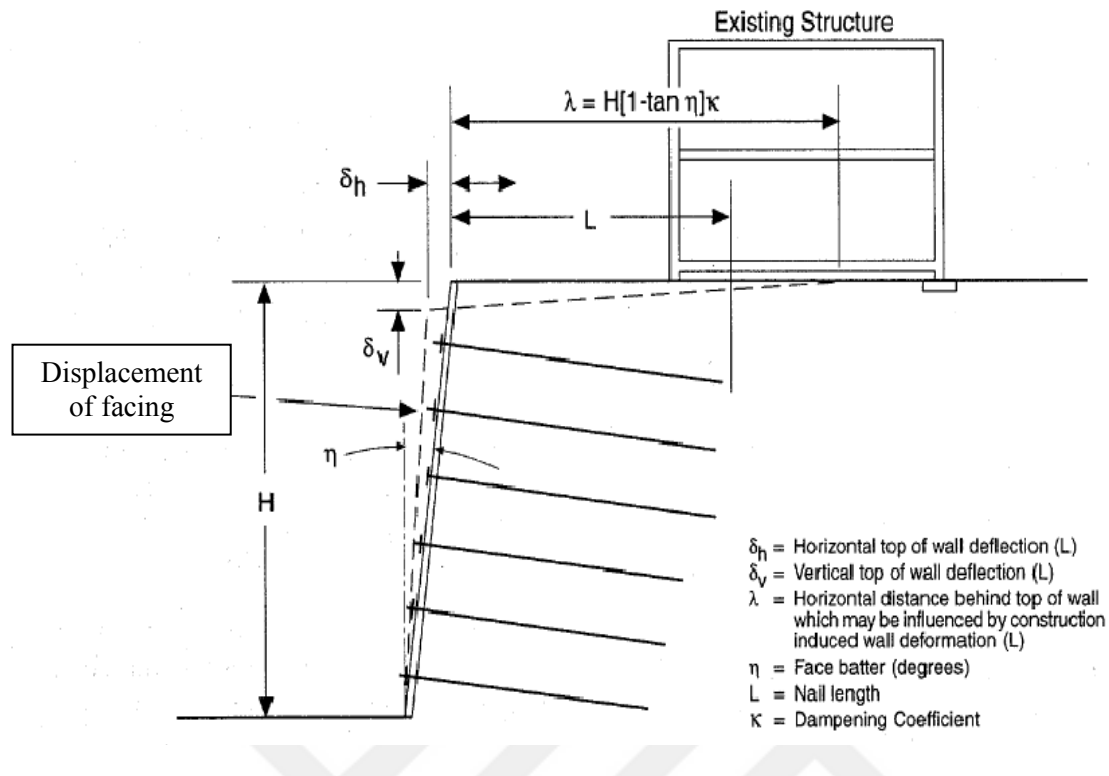
Equation above is used to compute the maximum long-term horizontal and vertical displacement top of the wall.

Where:

$\left(\frac{\delta_h}{H} \right)$ Is a ratio that depends on soil conditions, as indicated in Table from Figure 2.3.

This equation is valid as long as:

- The ratio $L/H \geq 0.7$, where L = soil nail length, H = wall height;
- The surcharge is negligible.
- $FOS \geq 1.5$ for overall stability.



Type of Soil	Weathered Rocks Stiff Soils	Sandy Soils	Clayey Soils
δ_h and δ_v coefficient κ	$H / 1000$ 0.8	$2 H / 1000$ 1.25	$3 H / 1000$ 1.5

Figure 2.3 : Deformation of soil nail walls (Clouterre, 1991).

These ways are considered in order to decrease deformations:

- Using a battered wall
- Installing longer nails on the top portion of the wall
- Increasing the overall steel section of the nails per unit area of the wall
- In some cases, the use of ground anchors in conjunction with the soil nails is recommended

3. DESIGN METHODS FOR SOIL NAIL WALLS

Commonly designs were done by considering two available methods for soil nail walls; Limit-equilibrium, and Working-stress design methods. Limit equilibrium methods investigate the equilibrium of a soil mass and working-stress design methods try to estimate the nail forces, they briefly explained in sections 3.1 and 3.2.

3.1 Limit-Equilibrium Method

Limit equilibrium methods investigate the equilibrium of a soil mass tending to slide down under the impact of the gravity or other force or loads. Computer programs which we use for slope stability analysis are based on the limit equilibrium concept for a two- or three-dimensional model. Two-dimensional (2D) models are analyzed assuming plane strain considerations. Stability analyses of two-dimensional slope geometries using simple analytical approaches can provide important insights into the initial design and risk assessment of slopes. These approaches design the reinforced walls under failure conditions. Transitional or rotational movement is considered on an assumed or known potential slip surface below the soil or rock mass. They involve valuation of a global safety factor while taking into account the shearing, tension, or pullout resistance of the nails crossing the potential failure surface.

3.2 Working-Stress Design Methods

Working-stress design methods try to estimate the nail forces such as tension, shear forces, and bending generated during construction by applying service loading conditions. After taking all of these forces and loads into service condition, the check and evaluation of local stability at each level of nails should be measured and assessed.

Conventional methods of slope stability analysis can be classified into three individual groups: kinematic analysis, limit equilibrium analysis, and rock fall simulators. All of these methods have evolved from limit equilibrium methods (LEM) but use different definitions of safety factors and different assumptions with regard to the shape of the failure surface, mode of soil-reinforcement interaction, and the resisting forces in the nails. In all methods, the slip surface is assumed to separate the reinforced mass into a moving block (active zone) and a stationary mass (resistance zone).

The failure surface is assumed to be bi-linear, parabolic, circular or logarithmic spiral. The methods incorporate, to some extent, the tension resistance and pullout capacity of the nails (Juran et al, 1990).

The current limit-equilibrium design methods provide only a global factor of safety with respect to soil shear strength but do not evaluate the maximum tension and shear forces generated in the nails.

The limitations of these methods do not allow their use for evaluating local stability at each nail level or for predicting wall deformation.

3.3 German Gravity Wall Method

This method was presented by Stocker et al. (1979). This design methodological analysis is based on limited number of model tests subjected to significant surcharge loads. The method uses a force-equilibrium approach with a bilinear slip surface with consideration of tension forces in the nails.

They believed that soil-nailed walls should be considered as gravity retaining walls. The reinforced zone with soil nails was modeled as single composite mass acting to the soil behind it. A soil-nailed wall was modeled with a bilinear wedge, and the location of the intersection of the two-angled failure surface was to be behind the reinforced zone. Several improvements have been suggested to refine the original method since then. Experimental work by Gassler (1988) showed that two plane translational failures were the only failure mode that warranted the consideration. Their research illustrated that all other models of failure, like steep circular surface and deep slip surfaces and also the combined tilting failure produce a higher factor for safety. Gassler is defined over all safety factor as the ratio of dissipative forces

along the slip surface combined with the nail effect to external forces applied to the system. The general description of the force equilibrium method for global stability is shown in Figure 3.1 and 3.2. The minimum factor of safety is evaluated by reiterative procedure in which the planar angle θ_A at the toe, computed from the horizontal, is varied while keeping the second planar angle θ_B at $(45^\circ + \frac{\phi}{2})$. The design chart developed based on this bi-linear wedge is shown below:

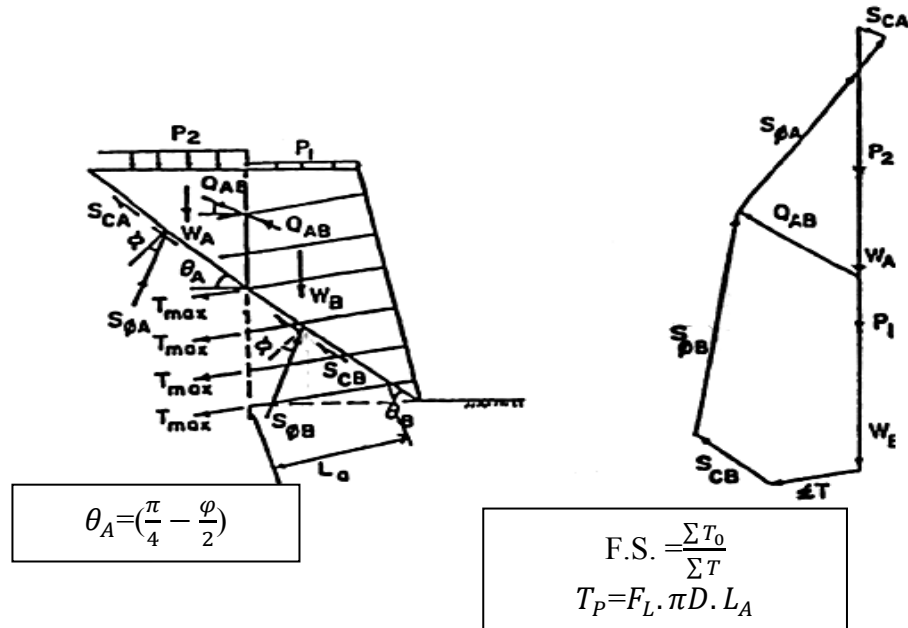


Figure 3.1 : German gravity wall methods (Gassler and Gudehus, 1981).

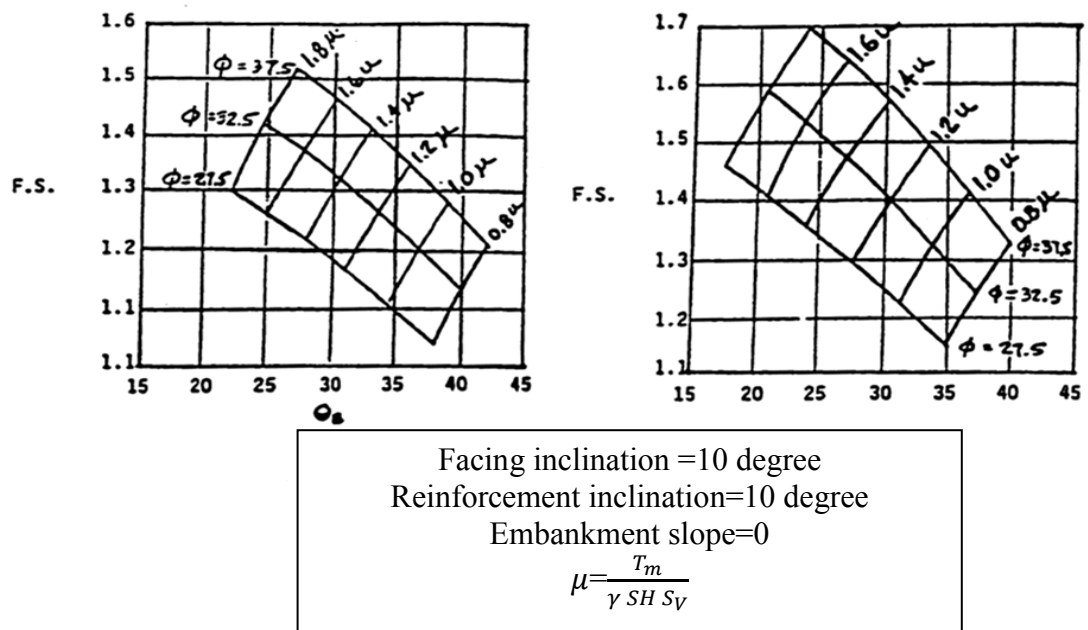


Figure 3.2 : German method: Design chart for stability calculations (Gassler and Gudehus, 1981).

3.4 French Multicriteria Analysis

The four-year French national project “CLOUTERRE” presented a Multicriteria analysis process for soil nail wall design. This method considers both tension and shearing resistance of the nails with respect to nail bending stiffness. The overall stability is evaluated assuming a circular slip surface using the simplified Bishop’s method. The method considers the mobilized lateral earth pressure on the nails and the corresponding shearing resistance developed in the nails. It also mentions that the nail shear and bending capacity increase the global FOS by less than 10%. The method is based on the assumption that the nails behave similarly to long elastic piles subjected to lateral.

The Multicriteria analysis is conducted to evaluate the factors of safety by considering Figure 3.2 based on four main aspects:

- I) Shear resistance of the soil
- II) Skin friction of the nails
- III) Normal interaction between the soil and the nails
- IV) Strength of the nail against tension, shear forces, and bending moment

3.4.1 Shear resistance of the soil

The soil is characterized by the Mohr-Coulomb criteria. The shear resistance of the soil is calculated from relation below:

$$\tau = c + \sigma_v \tan \phi \quad (2.2)$$

Where:

c : Soil cohesion

ϕ : Internal friction angle of the soil

The normal vertical stress is equal to:

$$\sigma_v = \gamma \cdot z \quad (2.3)$$

3.4.2 Skin friction of the nails

The skin friction of the nail (q_s) can be obtained by pullout test performed in both laboratory and field. Observations of soil nail walls by Schlosser (1983) presented independence of the soil from the nail.

$$\mu^* = \tau \sigma_v \quad (2.4)$$

Due to a decrease in dilatancy, it is offset by an increase in the normal stress σ_v ;

$$q_s = \mu^* (z) \gamma z = \text{constant} \quad (2.5)$$

Therefore, the maximum resistance of the nails is dependent on the soil-nail interaction criteria. Assuming the skin friction constant along the embedment length, the nail tensile strength T_n is evaluated using the following relationship (Clouterre, 1991):

$$T_s \leq q_s \pi D L_a \quad (2.6)$$

Where:

L_a : Embedment nail length in the resistant zone

D : Diameter of the soil nail

3.4.3 Normal interaction between the soil and the nails

The progressive mobilization of the passive lateral earth pressure on the nails leads to the development of the shear zone in reinforced mass results. This development is like the interaction between the laterally loaded piles with surrounding soil. The conventional p-y analysis may be used to estimate the ultimate shear forces and bending moments mobilized in the nails. This analysis models the soil response as a series of elastoplastic springs where the lateral reaction modulus K_h is figured from pressure meter data or Figure 3.3 (Elias and Juran, 1991).

The maximum shear force (T_c) mobilized at the point of intersection between the failure surfaces as shown in Figure 3.4 is given by:

$$\frac{pD I_o}{2} T_c ; P < P_{\max} \quad (2.7)$$

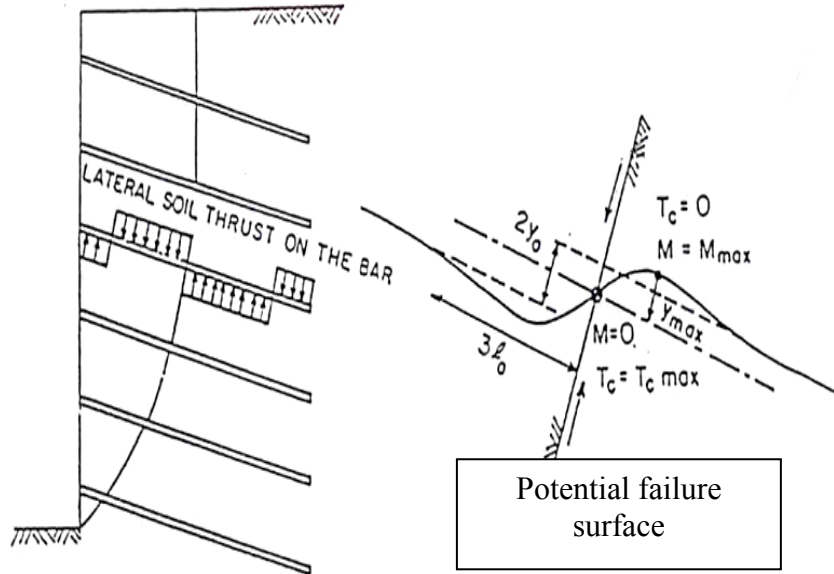


Figure 3.3 : Long nail subjected to combined loading (Elias and Juran, 1991).

The maximum bending moment mobilized at the distance $(\pi/4) l_0$ from point O is given by:

$$P M_{\max} = 0.16pDI_0^2 < M_p \quad (2.8)$$

Where:

p : Passive earth pressure on the nail

$P_{\max}$: Maximum passive resistance that can be mobilized in the soil

l_0 : Transfer length

M_p : Limit bending capacity of the nail

3.4.4 Strength of the nail

The nails system must tolerate both tension (T) and shear force (V). Accepting the nail element follows Tresca's failure criterion (Elias and Juran, 1991):

$$\frac{T^2}{F_y^2} + \frac{V^2}{R_c^2} < 1 \quad (2.9)$$

Where:

F_y : Tensile strength of the nail

R_c : Shear strength of the nail ($R_c = F_y/2$)

Using the principle of maximum plastic work, the mobilized tensile and shear forces T_f and V_f at failure can be evaluated as:

$$V_f = \frac{R_c}{\sqrt{(1+4\tan^2(45-\phi))}} \quad (2.10)$$

$$T_f = 4V_f \tan(45-\phi) \quad (2.11)$$

Where:

α : Nail inclination

It can be noted that for $\alpha = 0^\circ$, only the tensile force can be developed in the nail
for $\alpha = 45^\circ$ only shear forces are mobilized.

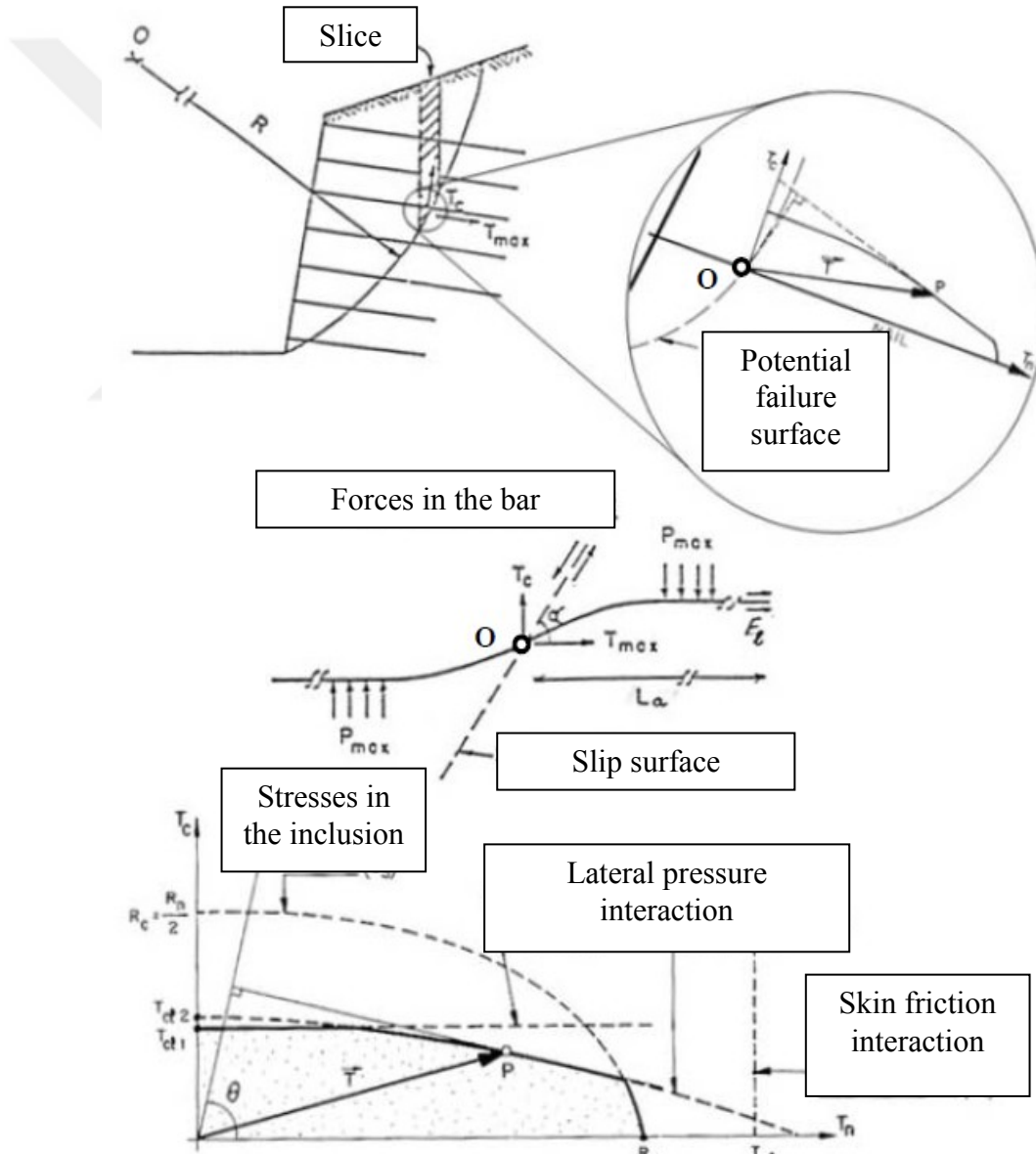


Figure 3.4 : Multi criteria slope stability analysis method (Elias and Juran, 1991 and Clouterre, 1991).

3.4.5 Strength of the nail

The nails system must tolerate both tension (T) and shear force (V). Accepting the nail element follows Tresca's failure criterion (Elias and Juran, 1991):

$$\frac{T^2}{F_y^2} + \frac{V^2}{R_c^2} < 1 \quad (2.9)$$

Where:

F_y : Tensile strength of the nail

R_c : Shear strength of the nail ($R_c = F_y/2$)

Using the principle of maximum plastic work, the mobilized tensile and shear forces T_f and V_f at failure can be evaluated as:

$$V_f = \frac{R_c}{\sqrt{(1+4\tan^2(45-\alpha))}} \quad (2.10)$$

$$T_f = 4V_f \tan(45-\alpha) \quad (2.11)$$

Where:

α : Nail inclination

It can be noted that for $\alpha = 0^\circ$, only the tensile force can be developed in the nail for $\alpha = 45^\circ$ only shear forces are mobilized

3.5 Kinematical Limit Analysis Design Method

Most of the limit-equilibrium design methods base on two important factors; the first one is global safety factor with respect to the shear strength characteristics of the soil, and the second one is the pull-out capacity of the nails. They do not provide any estimation about the maximum tension and shear forces generated in the nails and therefore cannot be used to evaluate the local stability of the reinforced soil mass at each reinforcement level. Kinematical limit analysis design method was developed by Juran et al. (1990). The design approach is based on solution obtained through limit analysis on observation displacement/failure mode, as observed on model walls. The main design assumptions are shown in Figure 3.5, (Juran et al, 1990).

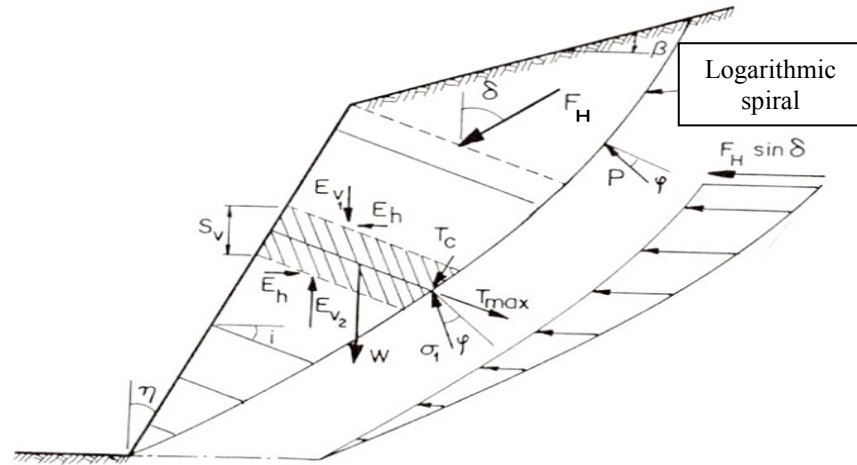


Figure 3.5 : Kinematical limit analysis method: design assumptions (Elias and Juran, 1991).

- Failure phenomena happen by a quasi-rigid body rotation of the active zone that is limited by a log-spiral failure surface.
- At the failure moment, the location of maximum tension and shear forces correspond with the failure surface developed in the soil.
- A thin layer of soil at a limit state of rigid plastic flow divided the quasi-rigid active and resistant zones to two separate parts.
- The reinforced mass is divided into slices parallel to the nails.
- The shearing resistance of the soil defined by considering Coulomb's failure criterion is completely mobilized all along the failure surface.
- The horizontal components (E_h) of the interslice forces acting on both sides of a slice comprising a nail are equal.
- The affect of the slope or horizontal surcharge at the top elevation of the reinforced zone on the forces in the nails is linearly diminishing with depth along the failure surface.

The maximum nail tensile force (T_n) and the maximum shear force (T_c) are ascertained in conformity with two non-dimensional parameters, (T_n) and (T_c).

N is a non-dimensional bending stiffness parameter, which depends on both the relative rigidity of the reinforcement of the soil and the structure height.

For preliminary design, simplified design charts have been developed to estimate the nail length, these charts were prepared with regard to tensile and shear forces at the maximum value of S/H , T_N and T_S for 15° nail inclination and uniform nails length.

These charts are prepared by considering S is the length of the nail in the active zone. The preliminary design charts are demonstrated in a dimensionless format as illustrated in Figure 3.6. The total length of the nail can be obtained by satisfying the following design criterion:

$$\frac{L}{H} \geq \frac{S}{H} + FSP \left(\frac{T_N}{\pi \mu} \right) \quad (2.12)$$

$$\mu = \frac{F_i D_g}{\gamma S_h S_v} \quad (2.13)$$

Where:

FSP : Factor of safety with respect to pullout

F_i : Skin friction

D_g : Diameter of grouted drill hole

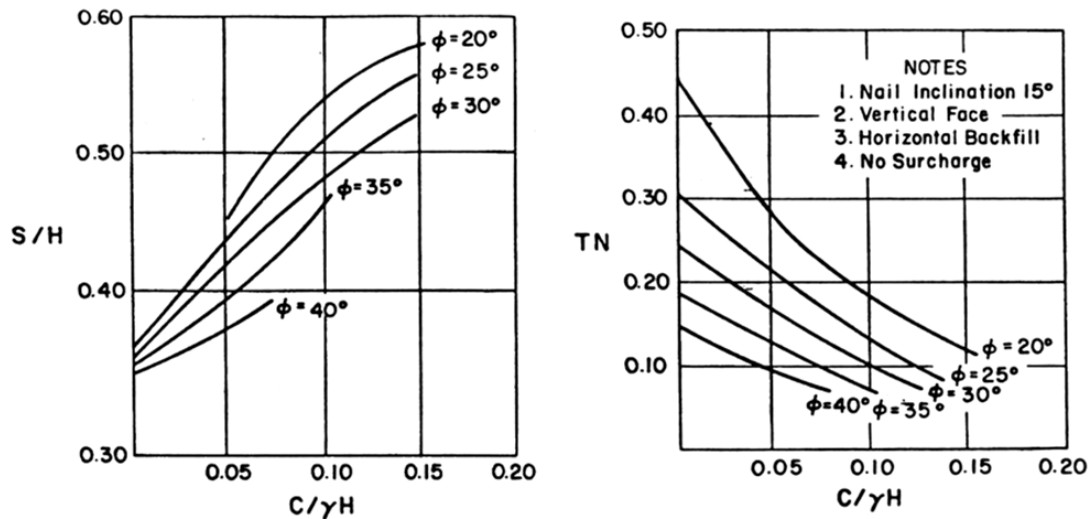


Figure 3.6 : Kinematical method: Design charts for perfectly flexible nails, $N = 0$ (Juran et al, 1990).

3.6 FHWA 1996 Design Method

The U.S. Federal Highway Administration (FHWA) published the "Manual for Design and Construction of Soil Nail Walls" (FHWA-SA-96-069) in 1996. This

manual is the collection of the work in Germany, France and currently U.S. practiced, and published as a form of guideline for soil nail design for highway works.

The limiting equilibrium method has been followed in this approach. The method considers only tension in the nails as the main load transfer mechanism in the nails to stabilize the soil mass (FHWA, 1996).

In the cases that the nails cross the slip surface, this design method accepted that the nails have a huge role in improving stability due to a major increase in the normal force within the reinforced soil mass, which causes the increment in the soil's shear resistance along the slip surface in soils with friction.

The suggested tension distribution of tensile forces in the nail is shown in Figure 3.7. The tension distribution diagram is based on two main points, the nail head strength and the bond strength between grout and soil.

The design path implements both the Service Load Design (SLD) and the Load and Resistance Factor Design (LRFD). This method is required to check both the internal and external stability. The internal stability is checked in two stages.

The first step is insuring about the nail itself with respect to endorsement of tensile strength of the nail tendon, ground-grout bond (friction bond), and grout-tendon bond.

The second step is controlling the stability of the nail head and the wall facing by considering resistance to soil earth pressure, flexure of the wall face, and punching shear of the facing and connection system.

In the external stability, the important factors which have to be controlled are listed below:

- Provision of a satisfactory factor of safety against horizontal sliding along the base (Figure 3.7).
- Bearing capacity failure of the foundation soil.
- Overall stability of the ground (Figure 3.8).

A set of charts were developed in order to design soil nail walls with geometries regarding to highway practice. The charts are demonstrated at (Figure 3.9). These

charts were prepared for equal length nails by inclination of 15° , and a global factor of safety of 1.35 for SLD factor.

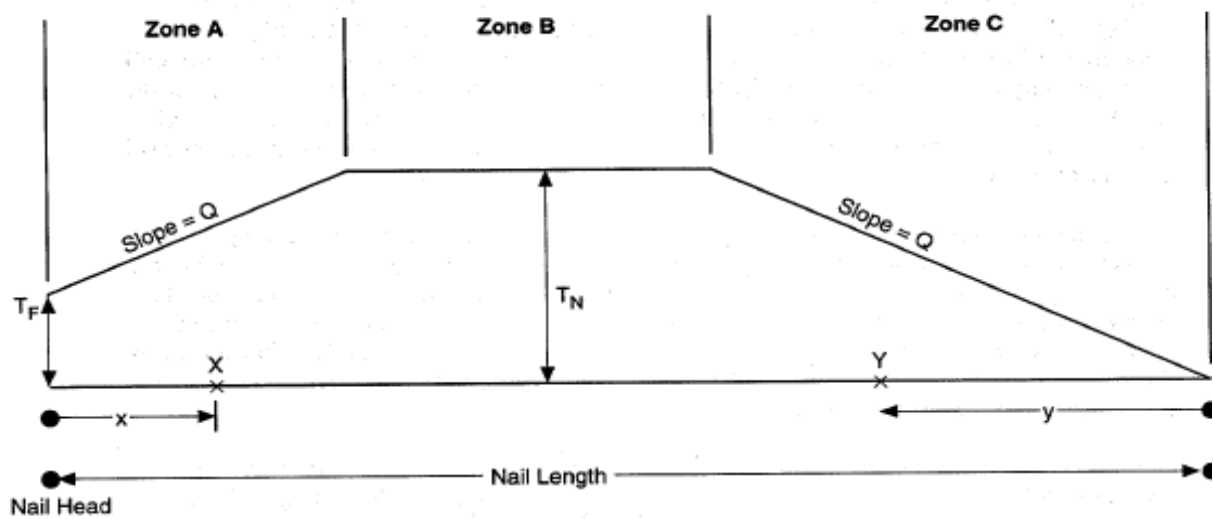


Figure 3.7 : Nail tension distribution diagram (Byrne et al, 1998).

Nail support to slip surfaces intersecting the nail in zone A at point $x = T_F + Q_x$

Nail support to slip surfaces intersecting the nail in zone B $= T_N$

Nail support to slip surfaces intersecting the nail in zone C at point $Y = Q_Y$

T_F : Strength of nail head-facing connector

T_N : Nail tendon tensile strength

Q : Nail-ground pullout resistance

3.7 FHWA 2003 Design Method

In 2003, the U.S. Federal Highway Administration published the new version of soil nail wall design method at “Geotechnical Engineering Circular No.7”. This manual is based on Allowable Stress Design (ASD) method. A series of preliminary design charts are also introduced by this manual (Figure 3.10). The mentioned charts have been developed using computer program SNAIL by considering face batter and back slope of the soil nail walls, and effective friction angle of the soils as important design parameters for a factor of safety of 1.35. The limit equilibrium computer program SNAIL or GOLDNAIL are used to calculate the factor of safety.

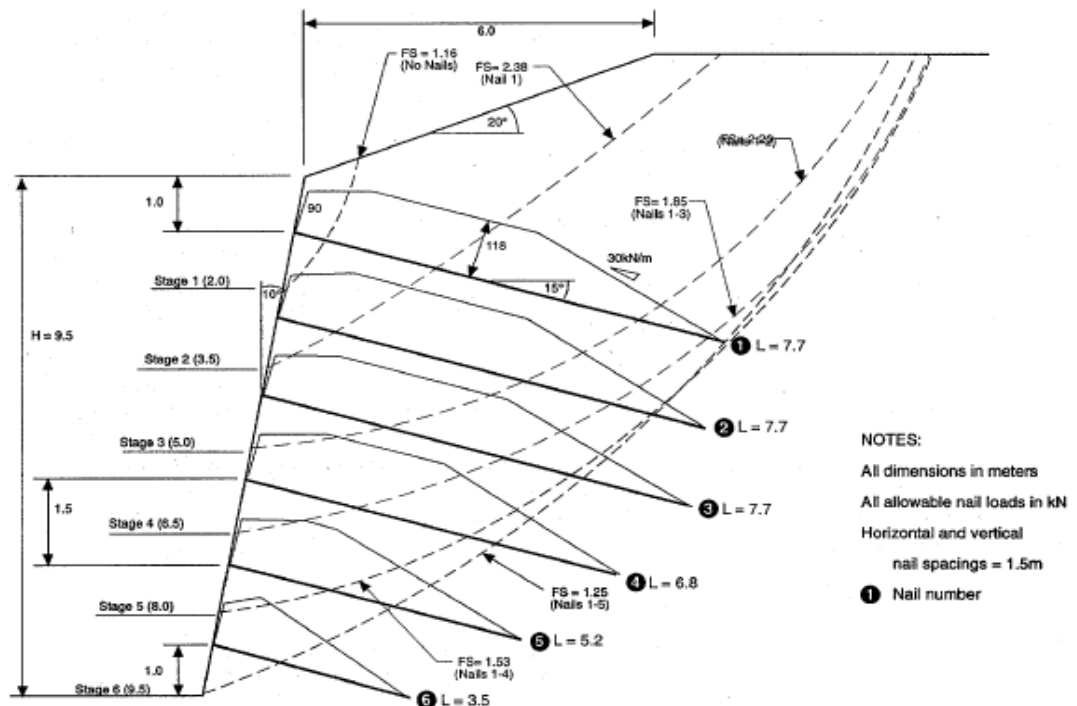


Figure 3.8 : Construction stability of soil nail wall (Byrne et al, 1998).

These charts were prepared for equal length nails by inclination of 15°, and a global factor of safety of 1.35 for SLD factor.

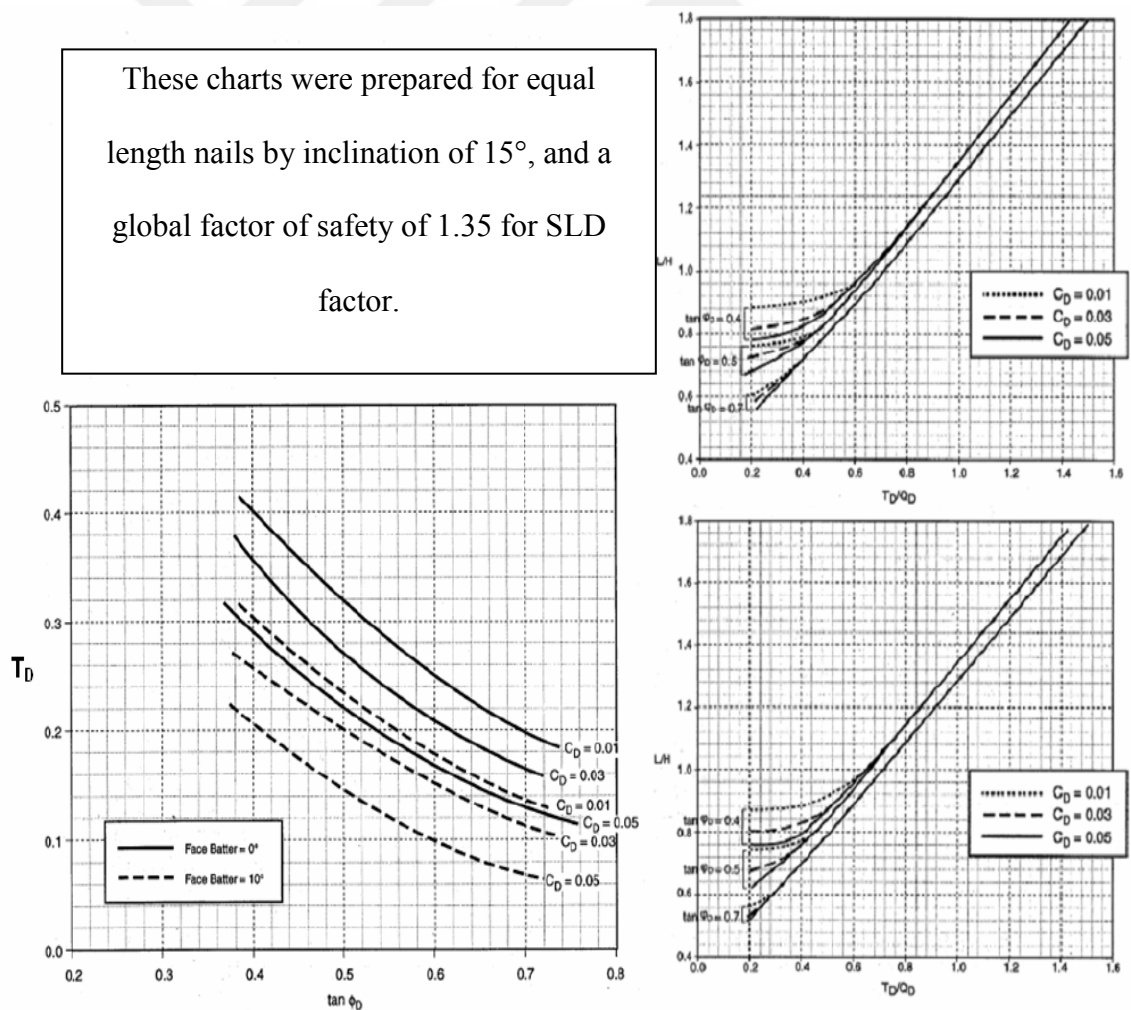


Figure 3.9 : Preliminary design chart for soil nail walls (Byrne et al, 1998).

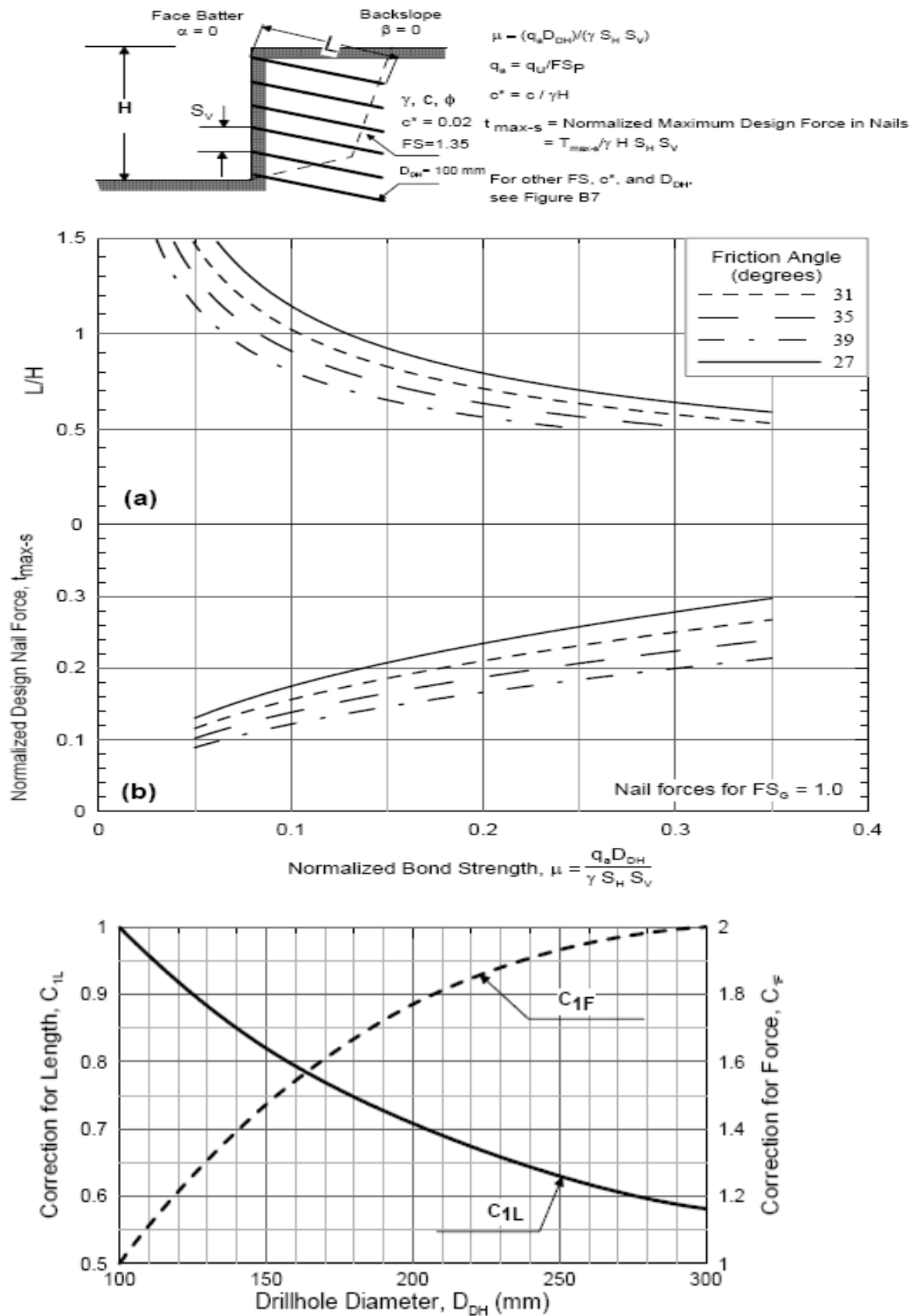


Figure 3.10 : Preliminary design chart for soil nail walls (Lazarte et al, 2003).

4. SELECTION OF DESIGN PARAMETERS

The main source of estimating design parameters is the data we get from borings, field tests, and laboratory. The designer must develop a test plan and the site stratigraphy in order to choose the most relative design parameters for each stage of the design.

Developing the site stratigraphy must include the exact location of boreholes in the plan in order to prepare geological profile of site and the exact locus of layers in the design part. And the contact between soil, weathered rock, and possibly bedrock must be defined and determined for better perspective. In some cases, design should be done in order to reach harder layers; the locations of these contacts are not just important for the global stability analyses, but also to demonstrate whether the soil nails need to extend near or into the underlying harder materials. The contrasting conditions mentioned above beside the engineering judgment and considering the financial analysis lead to choose the best method of stability for the whole system.

Site stratigraphy is critical for the design because the nature, extent, and distribution of the soil units affect the selection of the drilling methods, control the shape and dimensions of the potential sliding soil mass behind the soil nail wall, and affect the number and length of nails.

4.1 Unit Weight

The unit weight is the weight per unit volume of a material. The unit weight is the primary parameter in stability analysis and design calculations for soil nail wall and MSE wall design. The unit weight of the selected back fill soils, which is recommended in selected backfill of MSE wall and also in modeling the soil nail wall, can be measured from soil descriptions in conjunction with approximates of the relative density or via correlations (Kulhawy and Mayne, 1990). Figure 4.1 shows the dry unit weight (γ_d) which is expressed as a value normalized by the unit weight

of water (γ_{wr}) for various USCS-classified cohesion less soils without plastic fines that are favorable for soil nailing and MSE wall and as a function of the relative density .

We can evaluate the relative density by using Figure 4.1, or the friction angle.

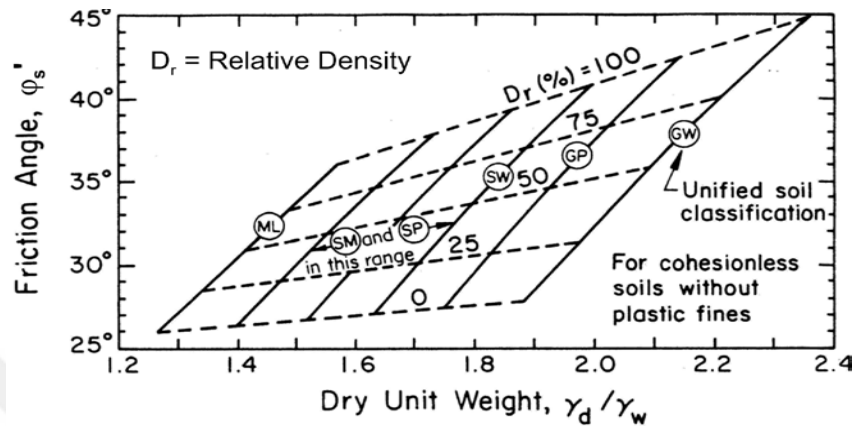


Figure 4.1 : Estimation of soil unit weight (Kulhawy and Mayne, 1990).

4.2 Shear Strength

Shear strength in soil describes the magnitude of the shear stress that a soil can sustain. The shear resistance of soil is a result of friction and interlocking of particles, and possibly cementation or bonding at particle contacts. Due to interlocking, particulate material may expand or contract in volume as it is subject to shear strains. In most of the projects with drain condition, the effective angle of internal friction (ϕ_s), which illustrates the main component of the shear strength of cohesion less soils, can be gained by using correlated values of SPT and CPT.

In Cohesion less Soils the effective cohesion, c' , is evaluated separately. For instance, cemented sands exhibit cohesion that should be considered for the design of the soil nail wall, but depending on the project characteristics, there may be a potential for significant disturbance of the soil matrix during excavation and there may be a huge loss of cohesion.

Preliminary estimates of the effective friction angle can be estimated from Table 4.1 using SPT N-values, OR from Table 4.2 using normalized CPT values.

Figure 4.2 shows the increase of the effective friction angle based on SPT N60 as the function of increasing normalized, in-situ effective overburden, $\sigma'V_0/Pa$, Where $\sigma'V_0$ is the in-situ effective overburden pressure, and Pa is the atmospheric pressure.

Table 4.1 : Correlations between SPT N-values, CPT, and friction angle of cohesion less soils – SPT (FHWA, 2015)

SPT N60 (blows/ft)	Soil Density Description	Soil Relative Density	ϕ_s' (deg)
0 to 4	Very Loose	< 20	< 30
4 to 10	Loose	20 – 40	30 to 35
10 to 30	Medium	40 – 60	35 to 40
30 to 50	Dense	60 – 80	40 to 45
> 50	Very Dense	> 80	> 45 (3)

Notes:

- Uncorrected values are considered equivalent to N60.
- From Kulhawy and Mayne (1990), based on Meyerhof (1956).
- Sands should be selected to have a friction angle ≤ 40 degrees. Gravelly soils should be selected to have a friction angle ≤ 45 degrees.

Table 4.2 : Correlations between SPT N-values, CPT, and Friction Angle of Cohesion less Soils – CPT (FHWA, 2015).

Normalized CPT Cone Bearing Resistance, q_c/Pa	Soil Density Description	Soil Relative Density	ϕ_s' (deg)
< 20	Very Loose	< 20	< 30
20 to 40	Loose	20 – 40	30 to 35
40 to 120	Medium	40 – 60	35 to 40
120 to 200	Dense	60 – 80	40 to 45
> 200	Very Dense	> 80	> 45 (3)

4.3 Cohesive Soils – Drained Conditions

The shear strength of cohesive (fine-grained) soils is generally a function of the rate of loading to drain excess pore pressures. Cohesive soils are often considered as undrained in short-term excavation-support applications. For long-term stability of soil nail wall, the drained strength should be considered. Results from consolidated undrained triaxial (CU) tests with pore-water pressure measurements must be used to precisely appraise the long-term drained strength of fine-grained soils.

4.4 Cohesive Soils – Undrained Conditions

The undrained shear strength of fine-grained soils must be taken to calculation for short-term stability of a wall during construction if these soils are disturbed by the excavation. Undrained stability analyses should be performed to model the wall stability directly after excavation and before the nail installation for that lift. The undrained shear strength (S_u) of fine-grained soils should be obtained using field tests like CPT or VST, although SPT may be used for preliminary estimations, and specific laboratory testing techniques that take into account the undrained response of these soils. In over consolidated clays, triaxial testing on undisturbed specimens can be used to determine the value of undrained shear strength for design.

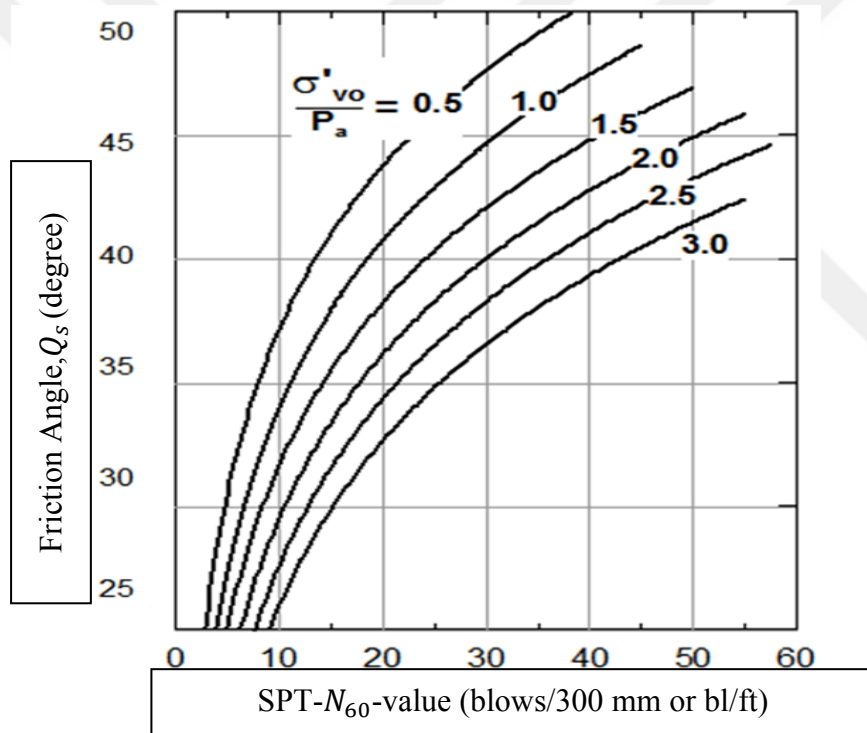


Figure 4.2 : Friction angle of cohesion less soils as a function of normalized overburden (Schmertmann, 1975).

5. INTRODUCTION TO MSE WALL SYSTEM

Reinforced Earth company is a leader in soil mechanic innovation and laboratory scale models which have been proven useful in gaining an understanding of the behavior of full-scale structures by Henri Vidal, the inventor of Reinforced Earth technology. He constructed and studied sand and paper models in the early 1960s. From 1962 to 1982 with the technical and financial support of the Reinforced Earth Group, independent laboratories around the world conducted more than a dozen scale-model tests which lead the start of the reinforced earth wall technology.

MSEW structures are cost-effective and more secure alternatives for most applications where rigid retaining walls, like reinforced concrete or gravity type walls, have traditionally been used to retain soil. These include bridge abutments, wing walls, and railway overpasses. They are particularly suited to economical construction in steep-sided fields, in grounds subject to slope instability, or in cases where foundation soils are poor.

5.1 Mechanism of Reinforced Earth Wall Structures

The system is based on primary principles of soil mechanic as explained below:

The vertical stress for cohesion-less soil at depth “h” for semi-infinite mass is equal to the density of the soil times h, ($\sigma_v = \gamma h$), and the lateral stress when the soil is at rest is equal to vertical stress times:

$$K_0 \text{ (} \sigma_H = \sigma_v K_0 = \gamma h K_0 \text{), where } K_0 \approx 1 - \sin \phi, \quad (5.1)$$

ϕ : Angle of friction of the soil

As soon as soil expands laterally, the K_0 value reduces to a smaller value which is called K_a ; therefore, Lateral stress ($\gamma h K_0$) reduces to ($\gamma h K_a$), where:

$$K_a = \tan^2(45^\circ - \phi/2) \quad (5.2)$$

By applying vertical load to a soil mass, the soil sample deforms laterally with a strain of δ_h and vertically with a strain of δ_v by installing the reinforcements in soil layers horizontally. It generates adhesion between the soil and reinforcement by friction. Since the reinforcement is stiff (like metal strips), the particles of soil rigidly stick between reinforcements, and it is generates adhesion by friction. If the reinforcements are ribbed, the friction between soil and reinforcement (Figure 5.1) are increase gradually.



Figure 5.1 : Connection parts in steel strips (Ekli, 2006).

These kind of the strips as we discussed are called high adhesion strips or HA. Thus, the gain in the strength of a reinforced soil mass can be credited to a rising in confining pressure that is applied to the soil (Figure 5.2).

It is explicit that as the vertical stress (σ_v) increases, the lateral stress (σ_H) also increases. We can elicit that the stress circle for the soil with reinforcement condition is always below the rupture curve. As a result, when the reinforcement ruptures, there is no adherence between soil and reinforcement, therefore soil fails.

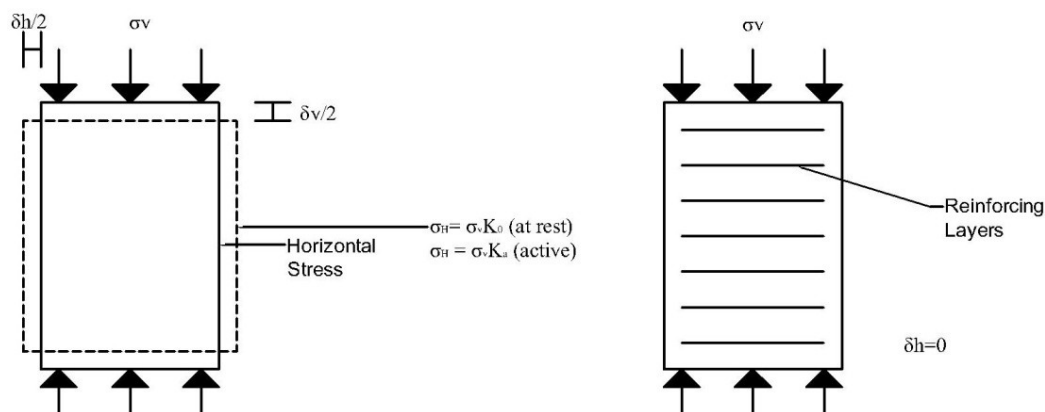


Figure 5.2 : Soil and reinforcement interaction (FHWA, 2009).

The force transferred from the unit of soil into the reinforcement is called lateral stress and it is equal to $(\sigma_H = \sigma_v K_0)$. Therefore, we can obtain tensile stress for each of the reinforcements by dividing lateral stress to cross-sectional area of the reinforcement $(T = \sigma_v K_0 / a_r)$. The strain in the reinforcement can be computed by dividing tensile stress by elastic modulus of the reinforcement, when the stiffness of the reinforcement is increase, the lateral strain is decrease; as a result, the lateral strain can reach zero by increasing the stiffness of the reinforcement. The lateral strain increases, and K_0 is equal to K_a .

5.1.1 Advantages and potential disadvantages

MSE walls offer important technical and cost advantages over conventional reinforced concrete rigid retaining structures at sites with poor foundation conditions. In cases with poor foundations, by applying this method we can reduce the costs of foundation improvements such as piles and pile caps. By considering economical requirements to support conventional structures, MSE walls have resulted in cost savings more than half of completed projects.

Advantages of MSE walls:

MSE walls have many advantages compared with typical rigid reinforced concrete and concrete gravity retaining walls.

MSE walls:

- Require simple construction steps and rapid construction procedures and do not require large and special equipment.
- Do not require special skills for construction.
- Do not need professional or skilled workers. Training the labors does not take time.
- Require less site preparation compared with other alternatives.
- Need less space in front of the structure for construction. Therefore, the operation could be continued just with small construction area and no road closing in highways.
- Do not need rigid, unyielding foundation support because MSE structures are flexible structures so they are tolerant to deformations.

- Are technically feasible to heights in excess of (40 m), for example, in the expansion of the SeaTac International Airport in the State of Washington (USA) total exposed height of approximately 43 meters (45 meters with wall base embedment (Figure 5.3).



Figure 5.3 : Sea-Tac Airport - 3rd Runway, WA-reinforced earth company (Reinforced earth company, 2009).

One of the greatest advantages of MSE walls is their resistance in earthquakes and their flexibility and capability to tolerate deformations due to poor subsoil foundation conditions. For instance, based on observations in seismically active zones, like 1999 Kocaeli earthquake, Turkey earthquake with such a high 0,42 maximum horizontal acceleration and forty-second earthquake, the MSE wall structures have demonstrated a higher resistance to seismic loading and just confronted with small deformation; therefore, MSE walls are vital for life road in critical conditions (Figure 5.4).

Precast concrete facing elements for MSE walls can be made with various shapes and textures, with little extra cost for aesthetic considerations for facing panels.



Figure 5.4 : The 1999 kocaeli earthquake in turkey, Reinforced earth MSE wall (Reinforced earth company, 1999).

Potential disadvantages;

In general, most of the potential disadvantages in structures with MSE walls are associated with local and project conditions. Those are listed below:

- Require a relatively large space behind the wall or slope face to install necessary reinforcement. If there is a cut, we confront with a large amount of excavation mass.
- Require the use of select granular as backfill materials. At some sites, the cost and preparation of selected fill materials are expensive and difficult. Therefore, they are uneconomical compared with Reinforced fill requirements for RSS.

5.2 Description of MSE System

The engineering community has accepted a general term Mechanically Stabilized Earth to characterize this type of retaining wall system.

Soil Reinforcement is a technique that is ordinarily used to stabilize soil. Soil reinforcement is made up of two important components, the tension element and the soil. Soil is the most abundant and least expensive construction material which has adequate strength to carry compressive stresses; we should consider that soil has

almost no tensile strength. By attending the soil dilatancy reports we can evaluate the volumetric change which associates dense soil during shear processes.

By adding linear or planer reinforcements that are strong in tension to the soil, and a merged material for backfill can be made in order to reach the specification granular size. The soil mass in backfill and reinforcement among it, is similar to reinforced concrete and has enhanced strength characteristics than the time when it is unreinforced. The soil can be constructed at angles much steeper than the internal angle of friction of the soil.

5.2.1 Reinforcement geometry

Three types of reinforcement geometry can be considered:

- I) Linear unidirectional : Strips, including smooth or ribbed steel strips, or coated geosynthetic strips over a load carrying fiber.
- II) Composite unidirectional : Grids or bar mats characterized by grid spacing greater than 150 mm .
- III) Planar bidirectional, Continuous sheets of Geosynthetics : Welded wire mesh, and woven wire mesh.

Reinforcement material differentiation can be made between the characteristics of metallic and nonmetallic reinforcements.

Metallic reinforcements are typically made of mild steel. The steel is usually galvanized or may be epoxy coated. In some countries, specifications just allow the use of steel with galvanized coating.

Nonmetallic reinforcements are generally polymeric materials consisting of polypropylene, polyethylene, or polyester. It depends on companies and materials they use; some have special materials with microfibers.

5.3 Facing Systems

Multifarious facing types are used in MSE wall construction because they are the only visible parts of the completed structure and it is important in terms of providing

protection against backfill sloughing and erosion and also have a minor role in drainage function.

Original metallic facings reinforced earth systems had facing elements composed of galvanized steel sheets formed into half cylinders. But nowadays, precast concrete panels are preferred and commonly used in reinforced earth walls; however, metallic facings may be appropriate for the cases where the design criteria require lighter facing elements.

5.3.1 Welded wire grids

Wire grid can be bent up in each step of facing at the front of the wall to form the wall face. This type of facing is used in the Hilfiker, Tensar, wire retaining wall systems.

5.3.2 Post-construction facing

For wrapped faced walls, it does not matter which material has been chosen. It can be attached after construction of the wall by shotcreting, or cast-in-place concrete or attaching prefabricated facing panels which are made from concrete, steel, or other materials. These Multi-staged facing approaches increase the cost of construction but it is kind of advantageous where considerable settlement is expected.

Precast elements can be cast in several shapes in order to have compatible textures that match with environmental requirements and blend aesthetically into the environment. Retaining structures with metal facing, in comparison with concrete facing, have the disadvantage of shorter life because of corrosion; unless provisions are made to compensate for it, one of most common method is special galvanized coating. The other facings like welded wire or gabions have the disadvantages of uneven surface, exposed backfill materials, more tendency for erosion of the selected backfill soil and stripes, possible shorter life from corrosion of the facing wires, and more susceptibility to vandalism in the absence of security.

5.4 Reinforced Fill Materials

MSE walls require high quality wall backfill for durability, good drainage, constructability, high resistance, and good soil reinforcement interaction which can be obtained from well-graded, granular materials. In some cases when there is no

appropriate material is available, the backfill materials are obtained by mixing the material at site and other materials which are obtained from other sources to reach the specifications criteria that are the first condition of many MSE systems based on friction between the reinforcing elements and the soil. Some systems rely on passive pressure on reinforcing elements, in those cases, the quality of reinforced wall fill is still critical. We can elicit that the soils with high clay contents are not suitable for this system and it disrupts the performance of the wall.

Selected backfill materials which are used in the structure for MSE wall structures should be reasonably free from organic or other waste materials and should follow the gradation, PI and other useful criteria listed in Table 5.1. The mentioned illustrates a large gradation range that is applicable across the United States.

Table 5.1 : MSE selected granular reinforced fill requirement (AASHTO, 2007).

<u>Gradation:</u> (AASHTO T-27)	<u>U.S. Sieve Size</u>	<u>Percent Passing^(a)</u>
	4 in. (102 mm) ^(a,b)	100
	No. 40 (0.425 mm)	0-60
	No. 200 (0.075 mm)	0-15
<u>Plasticity Index, PI</u> (AASHTO T-90)	$PI \leq 6$	
<u>Soundness:</u> (AASHTO T-104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles (or a sodium sulfate value less than 15 percent after five cycles).	

Notes:

- (a) To apply default F* values, C_u should be greater than or equal to 4.
- (b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to 3/4-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.

The soil which contains mica, gypsum, smectite, montomorrilonite or other soft durability particles should be carefully evaluated when the system is confronted with large strains. It is typically required to reach peak strength and pullout capacity. Therefore, in such a material, the lateral and vertical deformations are larger than the wall with higher quality granular fills.

Use of salvaged and waste materials such as asphaltic concrete or Portland cement concrete is not recommended.

Recycled asphalt is amenable to creep which has an adverse impact in both wall deformation and reinforcement pullout. Recycled concrete has a potential to bring forth tufa precipitate from anhydrate cement, which can block drains and emerge a white pasty substance onto the wall face. In this case, we face with aesthetic problems, and in some cases, it leads to erosion.

The compaction specifications should be consider, lift thickness and allowable range of moisture content with reference to optimum. Compaction moisture control should be $\pm 2\%$ of optimum moisture.

5.5 Reinforced Soil Design Criteria

A reinforced soil mass, in some aspect, is similar to reinforced concrete. In MSE wall system, the mechanical properties of the soil mass are improved by reinforcement placed parallel to the principal strain direction to increase the mass resistance. The tensile in soil mass increase in result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

Reinforcements in soil mass are distributed throughout the soil zone with a regular design; therefore, stress transfer between the soil and reinforcement takes place continuously along the reinforcement. Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and the reinforcement surface and there is a significant increase by using ribbed reinforcement.

Reinforcement which is used, such as steel strips, longitudinal bars in grids, geotextile, geosynthetic straps, that are dependent on friction, should be used and adjusted parallel to the direction of soil reinforcement relative movement.

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is commonly considered to be the first interaction for strips. The transverse ridges on "ribbed" strip reinforcement(Figure 5.5-part B) also provide some passive resistance. The magnitude of passive resistance is based on soil and strips. The shape of surface and reinforcement's

roughness (which is skin friction beside the grid opening dimensions), thickness of the transverse members, and elongation characteristics of the reinforcement are the important factors that affect the passive resistance. The other main factor that is significant for interaction development is the soil characteristics. The important effective parts are grain size distribution, particle shape, density, water content, cohesion, and stiffness.

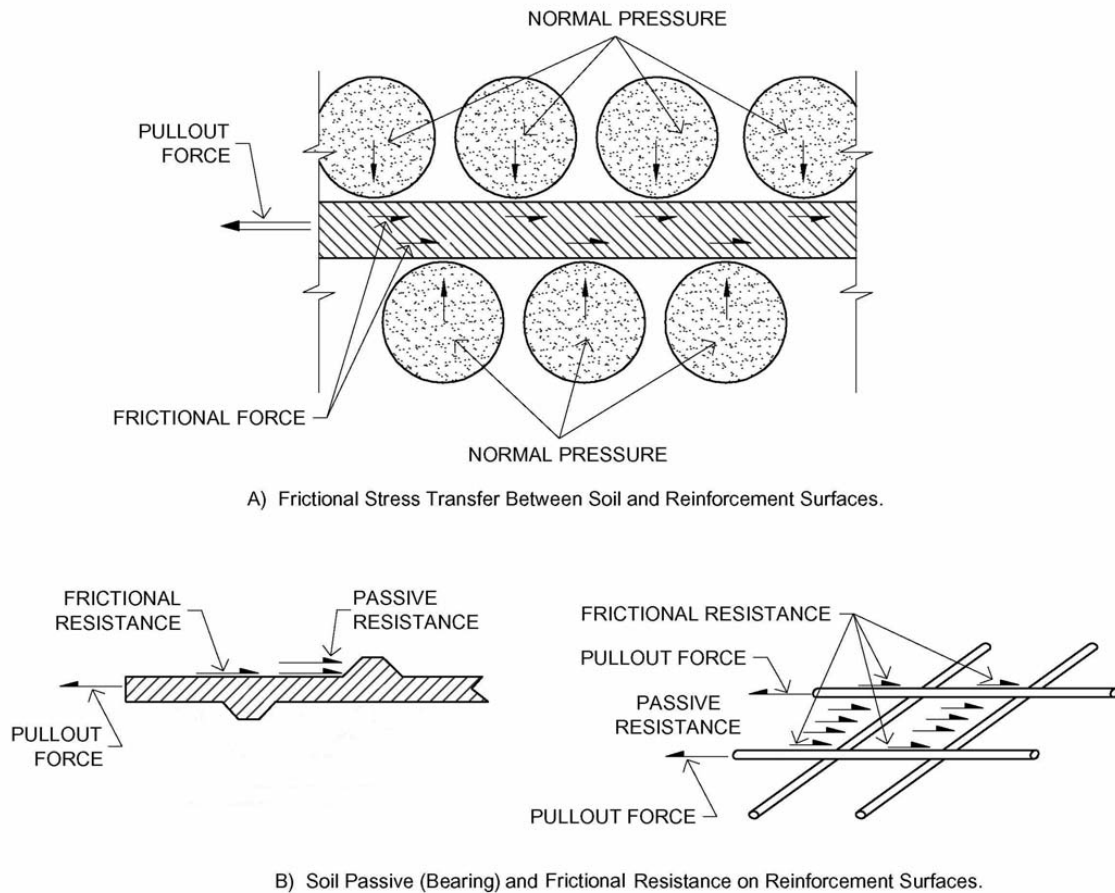


Figure 5.5 : Stress transfer mechanisms for soil reinforcement (FHWA, 2009).

5.5.1 Mode of reinforcement action and external stability

The most important function of reinforcements is to restrain soil deformations. In reinforced soil system, stresses are transferred and distributed from the soil to the reinforcement. These stresses are resisted by the reinforcement in three major parts: tension and shear and bending (Figure 5.5).

Tension is one of the primary action modes of tensile reinforcements. Reinforcing elements are aligned in the direction of soil extension. In fact, all longitudinal

reinforcements are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.

Shear and Bending are two major factors that "Transverse" reinforcing elements with some rigidity should withstand the shear stress and bending moments.

Classical gravity and semi-gravity retaining structures are commonly considered with four potential of external failure mechanisms in MSE walls, as shown in Figure 5.6.

They include Sliding on the base and Limiting eccentricity (formerly known as overturning), beside the bearing resistance Overall/global stability.

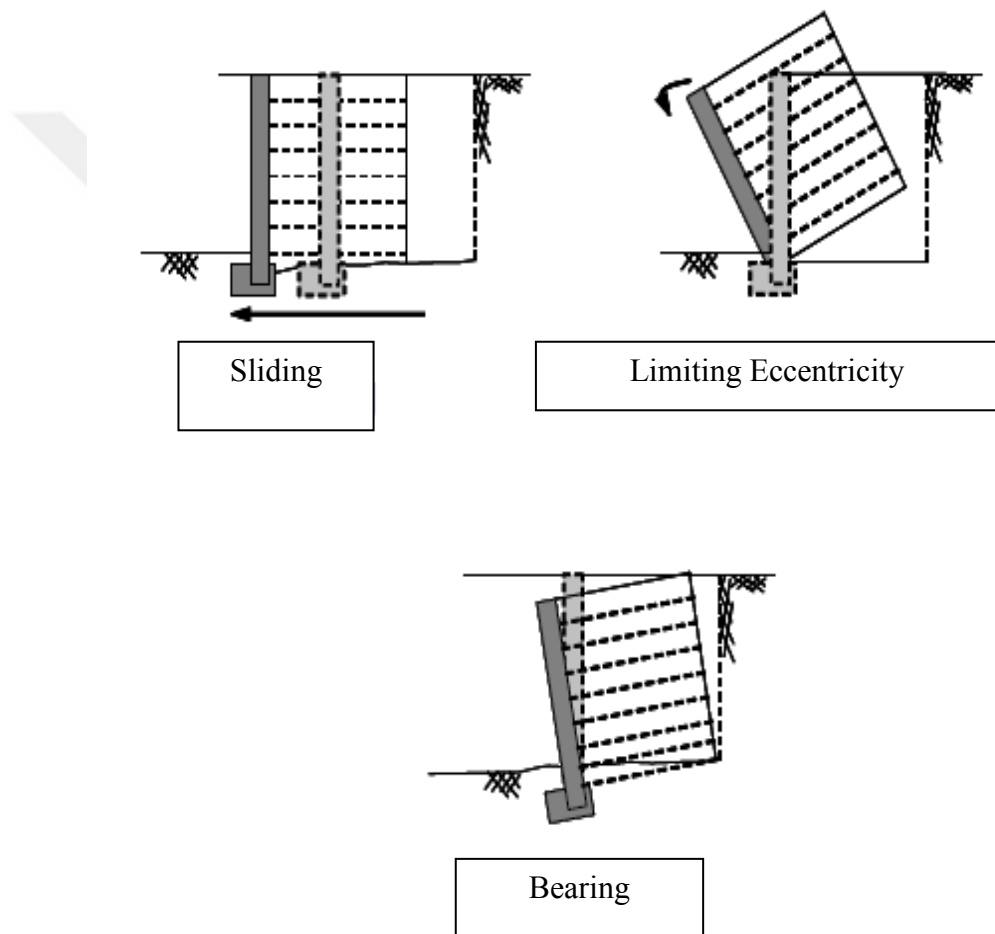


Figure 5.6 : Potential external failure mechanisms for a MSE wall (FHWA, 2009).

5.5.2 Sliding stability

In computing the reinforcement dimension and length, we assume the primary size and check the preliminary sizing with respect to sliding of the reinforced zone where the Resisting force is less than the shear resistance along the base of the wall or of a

weak layer near the base of the MSE wall, and the sliding force is the horizontal part of the thrust on the vertical plane at the back of the wall (Figures 5.7, 5.8 and 5.9).

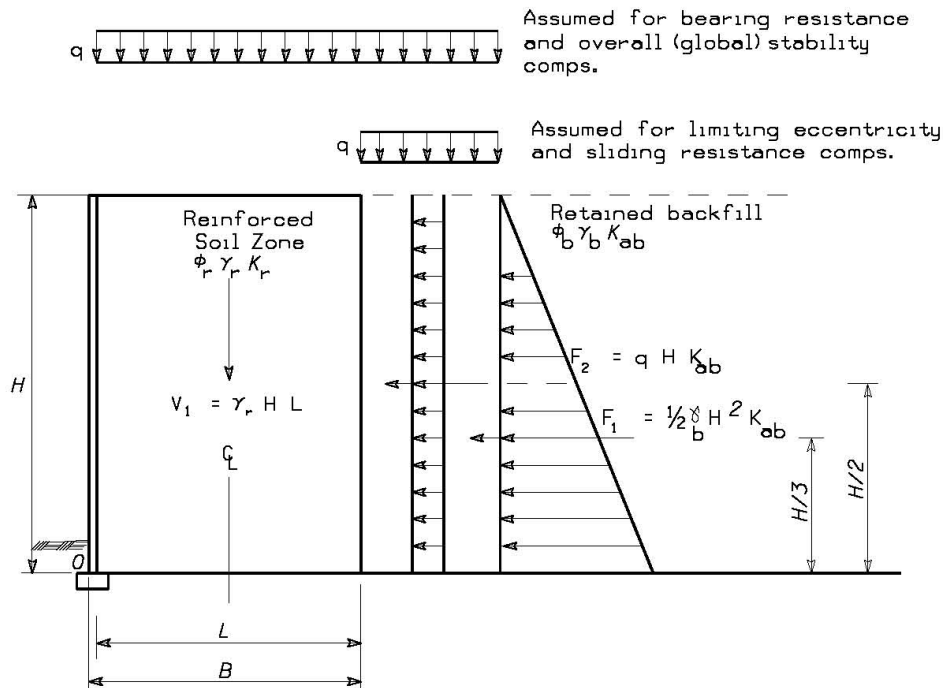


Figure 5.7 : External analysis: nominal earth pressures; horizontal back slope with traffic surcharge (AASHTO, 2007).

The live load surcharge is not taken into account as a stabilizing force when checking sliding; however, the sliding stability check should be applied when the live load is above the retained backfill, as shown at Figure 5.7.

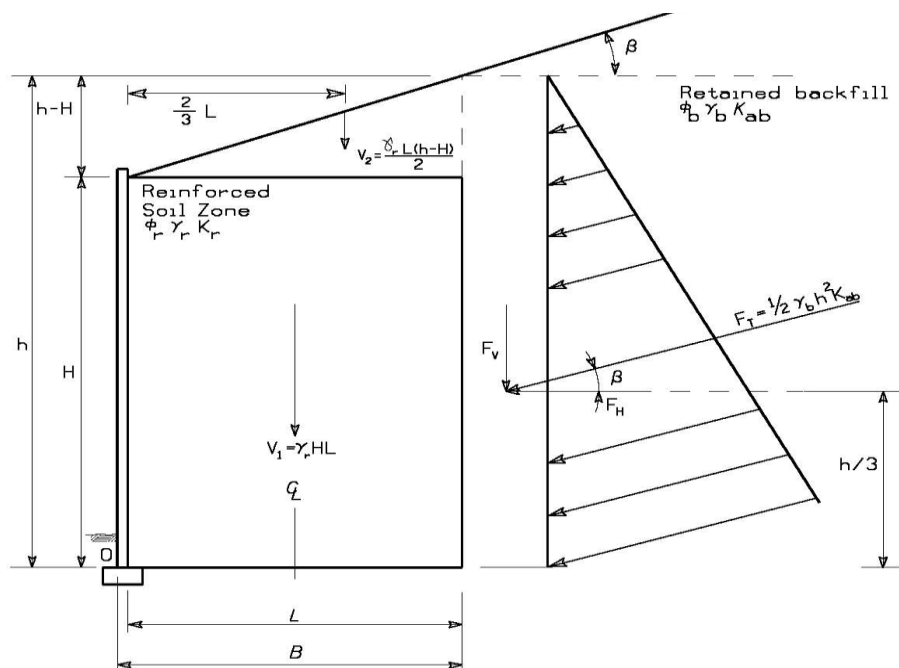


Figure 5.8 : External analysis: earth pressure; sloping backfill case (AASHTO, 2007).

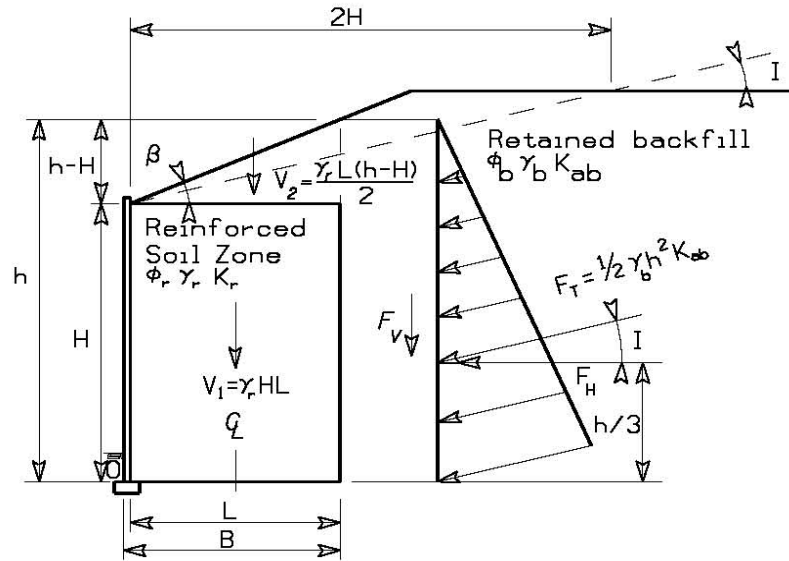


Figure 5.9 : External analysis: earth pressure; broken back slope case (AASHTO, 2007).

Sliding resistance along the base of the wall is assessed using the same processes as to spread footings on soil according to article 10.6.3.4 (AASHTO, 2007).

The factored resistance against failure by sliding (RR) can be calculated by:

$$RR = \phi_{\tau} R_{\tau} \quad (5.3)$$

Where:

ϕ_{τ} : Resistance factor for shear resistance between soil and foundation

R_{τ} : Nominal sliding resistance between reinforced fill and foundation soil

Passive resistance is usually not available during construction. In the design procedure, the soil passive resistance at the toe due to embedment is discounted due to the possibility for the embedded soil removal from the toe because of natural or manmade processes during its service life. The shear strength of the facing system is also conservatively neglected.

Calculation steps and equations to obtain sliding for two following cases are listed below. These equations should be extended to include other loads and geometries in special cases, such as additional live and dead load surcharge loads and significance of project.

Calculation of nominal thrust per unit width which is acting on the back of the reinforced zone,

I) Wall with Horizontal Back slope (Figure 5.7).

The retained backfill outcome, F_1 , is:

$$F_1 = \left(\frac{1}{2}\right) K_{ab} \gamma_B H^2 \quad (5.4)$$

For a uniform surcharge, the resultant is:

$$F_{12} = K_{ab} q H \quad (5.5)$$

Where:

K_{ab} : Active earth pressure coefficient for the retained backfill

γ_b : Moist unit weight of the retained backfill soil

H : Total height of the retaining wall

q : Uniform live load surcharge (it depends on type of the live load; it may be train, traffic, etc...).

ii) Wall with Sloping Backfill (Figure 5.8):

I) Calculation of nominal retained backfill force outcome per unit width, F_T

$$F_{T1} = \left(\frac{1}{2}\right) K_{ab} \gamma_B h^2 \quad (5.6)$$

Where:

K_{ab} : Active earth pressure coefficient for the sloping backfill.

H : Total height of wall, H , and slope at the back of the reinforced zone = $H + L \tan \beta$.

For a Triangular slope (Figure 5.9.), H (Total height of wall, H) should not exceed the height of the upper crest.

II) Calculate the nominal and the factored horizontal acting forces. For a horizontal

Back slope and uniform live load surcharge:

$$\sum F = F_1 + F_2 \quad (5.7)$$

$$P_d = \gamma_{EH} F_1 + \gamma_{LS} F_2 \quad (5.8)$$

For a sloping backfill condition:

$$F_H = F_T \cos \beta \quad (5.9)$$

$$P_d = \gamma_{EH} F_H = \gamma_{EH} F_T \cos \beta \quad (5.10)$$

In these equations, because it creates the maximum force effect for the sliding in limits state design method, therefore, the use of the maximum EH load factor equal to 1.5 is logical.

III) In the determination of friction angle in base to be on the safe side, we should consider the most critical frictional properties at the base. It depends on engineering judgment, but it is recommended to choose the minimum soil friction angle, ϕ for three possibilities:

- i) Sliding along the foundation soil, it would be happen, when shear strength is smaller than that of the reinforced fill material shear strength ($\tan \phi'_f$).
- ii) Sliding along the reinforced fill ($\tan \phi'_f$).
- iii) The soil-reinforcement friction angle, ρ , should directly be measured by means of interface direct shear tests in absence of the complete soil testing; it may be taken as $\frac{2}{3}$ of $\tan \phi'_f$.

IV) Compute the nominal parts of resisting force and the factored resisting force per unit length of the wall. For the wall with a horizontal back slope and uniform live load surcharge, the live load is neglected since it improves the sliding stability:

$$R_r = \gamma_{EV} V_1 * \mu \quad (5.11)$$

For a sloping backfill condition:

$$R_r = [\gamma_{EV} (V_1 + V_2) + \gamma_{EH} (F_T \sin \beta)] * \mu \quad (5.12)$$

μ : Minimum soil friction angle ϕ [$\tan \phi'_f \tan \phi'_s$] for continuous reinforcement.

External loads that cause the incensement in sliding resistance should only be let in count, if those loads are permanent, but we should consider that it depends on the project. The service life of MSE wall may be more than 50 years old, and it may have been changed by human and nature, so we shouldn't consider it in critical situations.

Therefore, the use of minimum EV load factor that is equal to 1 is recommended, it results in minimum resistance for the sliding limit state.

V) Compare factored sliding resistance, R_r , to the factored driving force, P_d , The division of R_r/P_d must greater than 1.

VI) check the capacity demand ratio (CDR) for sliding:

$CDR = R_r/P_d$. If the $CDR < 1.0$

The design engineer must increase the reinforcement length or type of reinforcement and dimension in order to get desirable conditions, and repeat the calculations.

5.5.3 Eccentricity limit check

The system of forces for checking the eccentricity at the base of the wall is demonstrated on scheme 5.10. It should be considered that the weight and width of the wall facing are generally ignored in the calculations.

Limiting eccentricity is a strength limit state check. The eccentricity limit check only applies the live load above the retained backfill.

The eccentricity, e , is the distance between the resultant foundation load and the center of the reinforced zone (i. e., $L/2$), shown in Figure 5.10. The exact quantity of eccentricity e is computed by adding together the overturning and the resisting moments at the bottom and center of the base length and dividing by the vertical load.

$$e = (\sum M_D + \sum M_R) / \sum R \quad (5.13)$$

Equations to calculate eccentricity for two general cases are as follow. These equations should be extended to include other loads and geometries or other supreme conditions.

Wall with horizontal back slope: Calculation steps for the finding of the eccentricity above the wall with a horizontal back slope and a uniform live load surcharge are as follows in Figure 5.10.

For a vertical wall with horizontal back slope and uniform live load surcharge, calculate the eccentricity e as follows:

$$e = \frac{(\gamma_{EH-MAX} F_1 (H/3) + \gamma_{LS} F_{q-Ls} (H/2))}{\gamma_{EV-MIN} V_1} \quad (5.14)$$

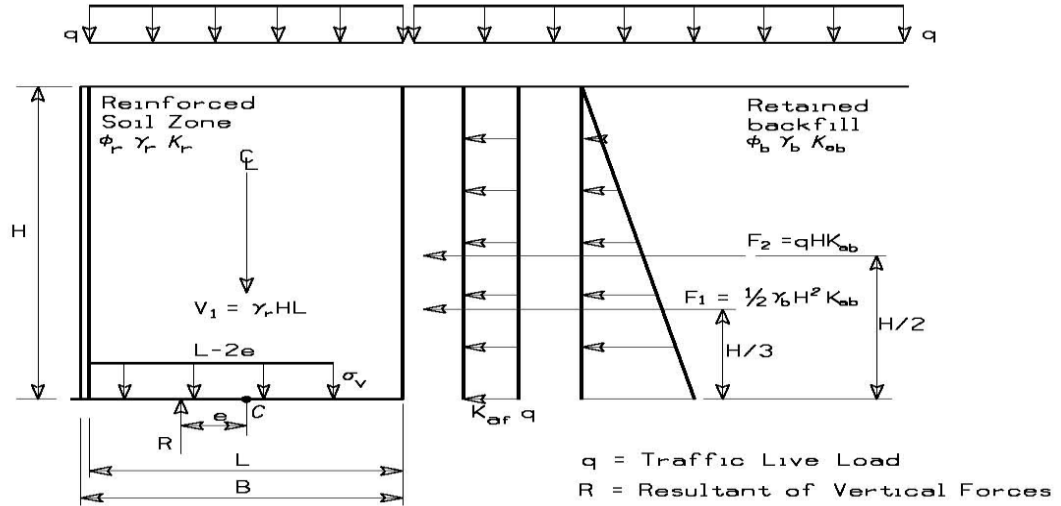


Figure 5.10 : Calculation of eccentricity and vertical stress for bearing check, for horizontal back slope with traffic surcharge condition (FHWA, 2009).

Wall with sloping backfill: The eccentricity at the bottom of the wall with a sloping backfill and without surcharges is calculated as follows; it is shown in Figure 5.11. Calculation the eccentricity (e) with factored loads in the wall with a sloping backfill, The eccentricity is equal to:

$$e = \frac{[(\gamma_{EH-MAX} F_T \cos \beta \left(\frac{h}{3}\right) - \gamma_{EH-MAX} F_T \sin \beta \left(\frac{L}{2}\right) - \gamma_{EH-MIN} V_2 \left(\frac{L}{6}\right)]}{[\gamma_{EV-MIN} V_1 + \gamma_{EV-MIN} V_2 + \gamma_{EH-MAX} F_T \sin \beta]} \quad (5.15)$$

5.5.4 Eccentricity criteria

The eccentricity, e , is considered stable and acceptable if the calculated location of the resultant vertical force which is based on factored loads within the middle one-half of the base width for soil foundations is equal to ($e_{max} = L / 4$) and at the middle three-fourths of the base width for rock foundations is equal to ($e_{max} = 3/8 L$). Therefore, for each strength limit load group, eccentricity must be less than e_{max} . If e is greater, then the length of the reinforcement must be changed, and a longer length is required. In special cases or significant projects which require special safety, maximum permanent loads and total extremes should be checked for such complex walls (regarding geometry and/or loadings) to identify the critical loading.

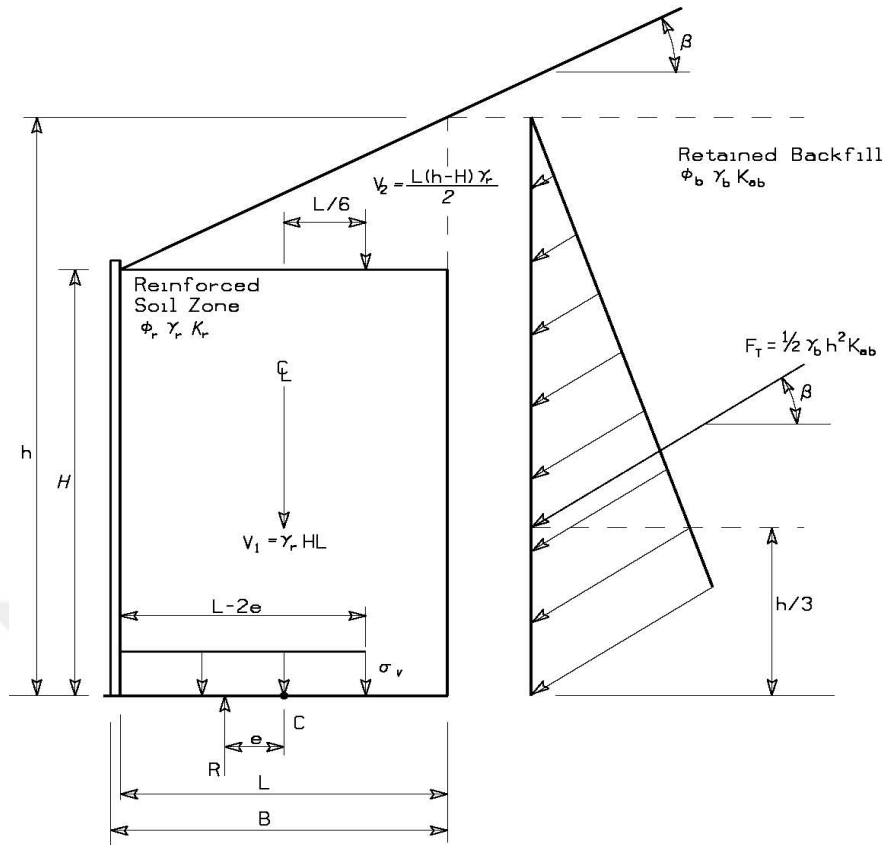


Figure 5.11 : Calculation of eccentricity and vertical stress for bearing check, for sloping back slope condition (FHWA, 2009).

5.5.5 Bearing capacity check

Two possibilities of bearing capacity failure exist. The first one is general shear failure, and the second mode is local shear failure.

Local shear occurs in foundation soil when the soft or loose soils exist below the wall. Bearing calculations should be done in order to consider both a strength limit state and a service limit state calculation.

Strength limit computations are done in order to check whether the factored bearing pressure is less than the factored bearing resistance.

Service limit calculations are used to find the bearing pressure for use in settlement calculations. The bearing check applies both for live load above the reinforced zone and the retained backfill, as shown in Figure 5.11. It is suggested that the factored vertical pressure at the base of the wall be calculated with the uniform Meyerhof type distribution to prevent bearing failure on the foundation soil, and it should be checked in order not to exceed the factored bearing resistance of the foundation soil:

$$q_R \geq q_{uniform} \quad (5.16)$$

The uniform vertical pressure is calculated as:

$$\sigma_v = \frac{\sum v}{L - 2e_B} \quad (5.17)$$

Where:

$\sum v$: Summation of vertical forces

L : Width of foundation or reinforcement length

e_B : Eccentricity for bearing calculation

In this step, each different case requires a distinct computation of the eccentricity value. Also, the engineer must consider in his/her calculation that the bearing check applies the live load above both the reinforced zone and the retained backfill. Furthermore, walls constructed on rocks must be checked due to the flexibility of MSE walls and their limited ability to transmit moment. (Article C11.10.5.4 {AASHTO, 2007}).

As the further explanations below, the calculation steps for MSE walls with a horizontal back slope or uniform live load surcharge and for sloping are listed.

As previously mentioned, these equations should be extended to be used in other loads and geometries for supreme cases.

I) Computing the eccentricity, e_B of the resulting force at the bottom of the wall. The e value from the eccentricity check cannot be used to calculate eccentricity with factored loads.

For a MSE wall with horizontal back slope and uniform live load surcharge located above the reinforced zone, the eccentricity is obtained as follow:

$$e_B = \frac{\gamma_{EH-MAX} F_1 \left(\frac{H}{3} \right) + \gamma_{LS} F_{q-L} \left(\frac{H}{2} \right)}{\gamma_{EH-MAX} V_1 + \gamma_{LS} Q_L} \quad (5.18)$$

Maximum load factors γ_{EH} and γ_{EV} are defined and take into computation, where the vertical stress σ_v is in its maximum amount.

The formula for walls with sloping backfill is demonstrated at equation above. Again, these equations should be extended by considering over plus loads and geometries.

It is important to know that the value of eccentricity, e_B , will vary when checking the various load factors and load combinations; furthermore, in calculation of eccentricity, if e_B , is negative, a value of 0 should be carried forward in the design stress equation (AASHTO, 2002).

II) Calculating the factored vertical stress σ_{V-F} is done by assuming Meyerhof-type distribution. For a horizontal back slope and uniform live load surcharge, the factored bearing pressure is gained from equation below:

$$\sigma_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{LS} q_L}{L - 2e_B} \quad (5.19)$$

This approach was first suggested by Meyerhof. He assumes that a stress distribution due to eccentric burden can be estimated by a uniform stress distribution over a reduced area at the base of the wall. As shown in Figures 5.10 and 5.11, this area is defined by a width equal to the wall width minus twice the eccentricity.

Use of the effective width, $L - 2e_B$ instead of the full width for obtaining the effect of eccentricity and load inclination is accepted. For walls with sloping backfill, the factored bearing stress is compute as below:

$$\sigma_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{EV-MAX} V_2 + \gamma_{EH-MAX} F_T \sin \beta}{L - 2e_B} \quad (5.20)$$

It is significant to consider that $(L - 2e_B)$ is equal to L when the value of eccentricity is below zero. A negative value of eccentricity may be confronted in some extreme geometries, for instance for wall section with very long reinforcements and a steep infinite back slope.

Where applicable, the influence of factored surcharge and factored concentrated loads must be included in the computation of bearing stress, (σ_{V-F}) , consistency with loads and load factors used in the eccentricity calculation and corresponding bearing stress calculation.

III) The bearing resistance q_n is determined from Equation 10.6.3.1.2a-1 of AASHTO (2007). For MSE wall and no groundwater:

$$q_n = C_f N_c + 0,5L\gamma_f N_\gamma \quad (5.21)$$

Where:

C_f : Cohesion of the foundation soil

γ_f : Unit weight of the foundation soil

N_c and N_γ : Dimensionless bearing capacity coefficients

L' : Effective foundation width

Partial embedment may be considered in the determination of q_n which is provided that the fill in front of the wall is placed and compacted as the reinforced fill is placed and all possible failure modes are examined.

IV) check whether the factored bearing resistance is exceeding the factored bearing stress,

$$q_R > q_{v-f'} \quad (5.22)$$

The factored bearing resistance (q_R) is given as:

$$q_R = \varphi q_n \quad (5.23)$$

Where:

φ : Resistance factor for MSE walls is 0.65

V) As showed in steps 2 and 3, the $q_{v-f'}$ can be decreased and q_R can be increased by rising the length of the reinforcements. The bearing resistance can often be obtained more accurately by applying more subsurface investigation and better definition of the foundation soil properties. If the foundation conditions cannot be achieved in order to use the minimum reinforcement or if by increasing the length of the reinforcements the cost of the project significantly increases, the improvement of the foundation soil may be considered like dynamic compaction, soil replacement, stone columns or pre-compression, etc.

In the process of choosing parameters for design, the engineer may use reduction factors in accepting parameters in order to be on the safe side and avoid being confronted with unpredictable situations. Conditions like Local Shear, Punching shear and Lateral Squeeze are the situations logical to reduce the value of mentioned parameters. Local shear is a transition between general shear and punching shear,

which can occur in loose or compressible soils, and in weak soils under slow (drained) loading.

When the local shear or punching shear failure is possible, according to AASHTO (2007), it is recommended to use reduced shear strength parameters for computing the bearing resistance. The reduced effective stress cohesion C^* is obtained from equal $0.67c'$. The reduced effective stress soil friction angle, ϕ^* is equal to $\tan^{-1}(0.67 \tan \phi')$.

When the MSE wall constructed on a weak cohesive soil layer overlying a firm soil layer, lateral squeeze occurs. Lateral squeeze failure is a consequence of significant horizontal movement of the soil under the structure.

To avoid the local shear of structures bearing on weak cohesive soils it is necessary to control the equation below:

$$\gamma_r H \leq 3C_u \quad (5.24)$$

Where:

γ_r : Unit weight of reinforced selected fill

H : Total height of the wall

C_u : Total stress cohesion of foundation soil

In lack of adequate support conditions, soft soils should be removed or ground improvement of the foundation soils is necessary.

5.5.6 Settlement assessment

Settlement analyses should be done in order to ensure that the immediate, consolidation, and secondary settlement of the wall are acceptable and also to ensure about the less than the requirements of the project. It varies according to the service life and importance of the project. Settlement is evaluated under bearing pressure computed by considering a service limit state.

Accurate estimation of post-construction foundation settlements demonstrate that the planned top of a wall line needs to be adjusted. Considerable differential settlements greater than 1/100 of total height of the wall need slip joints, which allow the independent vertical movement that affects the precast panels and leads to crack on it. Where the predicted settlements and their duration cannot be achieved and cannot

satisfy the specification, the ground improvement techniques such as stone columns, dynamic compaction, should be taken into consideration.

5.6 Internal Stability

For internal stability calculations using the simplified method, which contains internal coefficient of earth pressure in order to check the function of the type of reinforcement is taken into account, where the minimum coefficient (Ka) is used for walls constructed with continuous sheets of geotextiles and geogrids. In the internal stability calculation which is done with regard to the Rankine, failure surface is not suitable because the extensible reinforcements can be extended more than the soil before the failure; therefore, they do not significantly affect the shape of the soil failure surface.

5.6.1 Defining critical slip surface

The critical slip surface in a reinforced soil wall with simple geometry is accepted to occur as same as the maximum tensile force, T_{MAX} , in each reinforcement layer. The critical failure surface is accepted to be almost bilinear in the cases with inextensible reinforcements (Figure 5.12-A), and critical failure surface is assumed approximately linear in the case of extensible reinforcements (Figure 5.12-B). By considering that it isn't different in both cases it passes through the toe of the wall.

When failure occurs, the reinforcement may stretch and the deformation will happen at its intersection with the failure surface. Consequently, the tensile force in the reinforcement will increase and rotate. As a result, the reinforcements in the direction of the failure surface would increase. In stiff inextensible reinforcements, like steel strips, elongation and rotation of the reinforcements may be negligible. But it would be important in extensible reinforcement like geosynthetics. In the simplified method mentioned earlier, the rotation of reinforcement is ignored for internal wall stability calculations.

The primary noticeable matter for internal loading which must be considered in MSE wall is the earth pressure from the reinforced fill and surcharge loadings on top of the reinforced zone. The second step is taking vertical earth pressure (EV), live load surcharge (LS), and earth surcharge (ES), water, seismic, and vehicle impact loads into calculating the unfactored loads for MSE walls must be included.

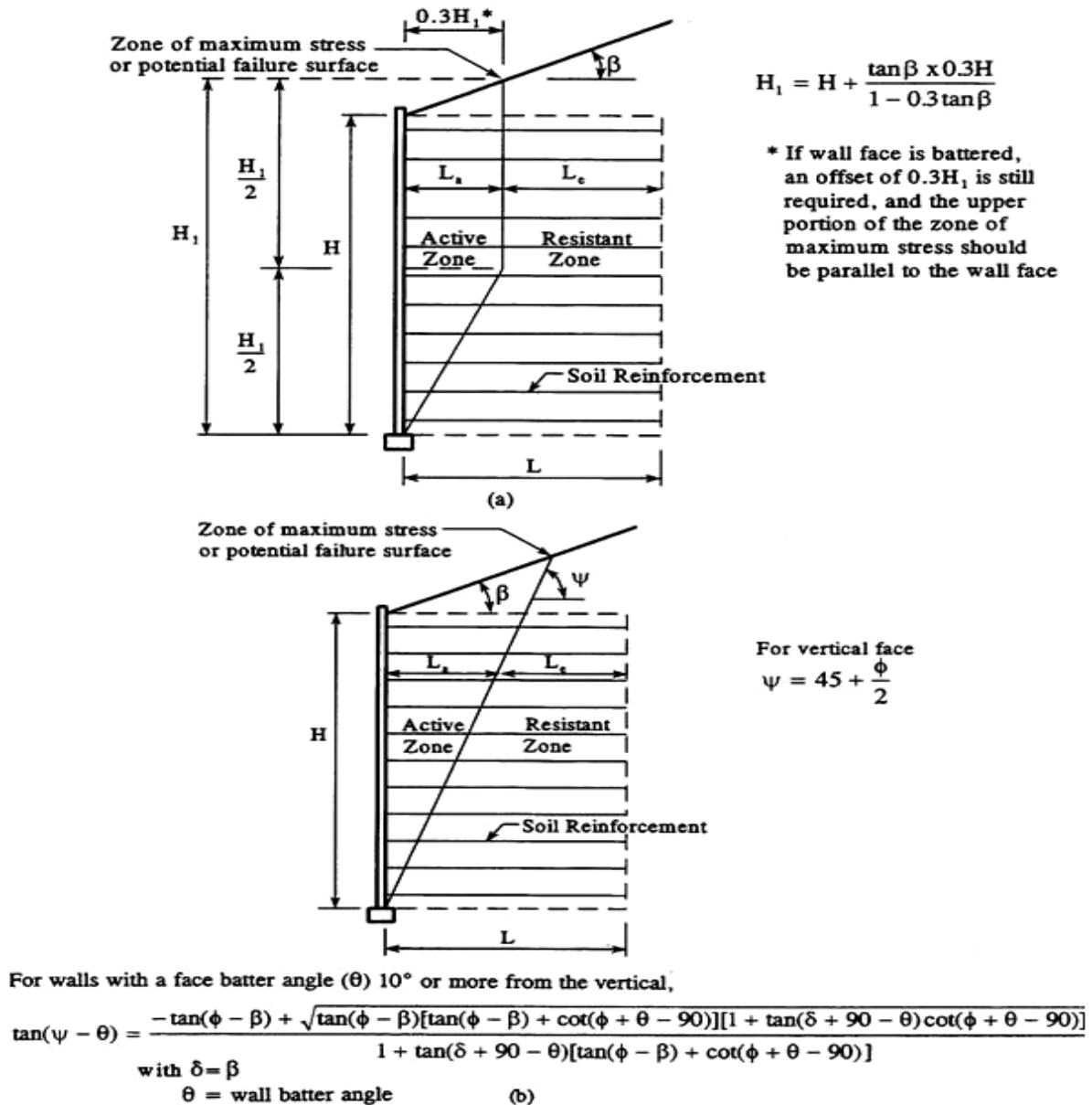


Figure 5.12 : Location of potential failure surface for internal stability design of MSE Walls inextensible reinforcements and (b) extensible reinforcements (FHWA, 2009).

Research studies have shown that the maximum tensile force is mainly related to the type of reinforcement in the MSE wall, which, in turn, is a function of the modulus, extensibility and density of reinforcement. (Collin, 1986; Christopher et al, 1990; Allen et al, 2004). This research indicates that the relationship between the type of the reinforcement and the overburden stress (Figure 5.13).

In various reinforcements the value of Kr/Ka is different from each other. The value of Kr/Ka for this type of reinforcement like geosynthetic is a constant. The Kr/Ka

ratio for inextensible reinforcements like steel strips decreases from the top of the reinforced wall fill to a constant value from six meters below this height. It is important to know that the resulting Kr/Ka ratio is addressed to the top of the wall at the face, excluding any coping units for both walls with level and with sloping backfills.

Figure 5.13 was developed by back analysis of the lateral stress ratio Kr which is collected from field data and available case study researches, where stresses in the strips were evaluated and normalized as a function of the Rankine active earth pressure coefficient, Ka .

In the calculation according to Rankine active earth pressure theory, the lateral pressure is assumed to be independent from backfill slope and interface friction.

In order to provide a comprehensive evaluation method for all cohesion less reinforced fill walls, as in the ratio shown in Figure 5.13, the values are represented by assuming that the results of the vertical stress are equal to the weight of the overburden (γH).

This research , does not cover newly developed reinforcement types, like special polymer fibers or nanotechnology fiber strips. In complex cases, for extension of the figure 5.13, the effect of full height panels and the behavior of the wall and strips in earthquakes must be investigated by instrumenting the structures and using numerical models to make corrections to the Kr/Ka ratio for routine and complex walls. Notation for Coulomb active earth pressures used with wall batters, θ , greater than 100 degree (AASHTO, 2007).

In the calculation of active earth pressure regarding the Coulomb earth pressure method, the assumptions are accepted as listed at Figure 5.14 , Do not consider any fraction in the wall calculation, and β angle is equal to zero.

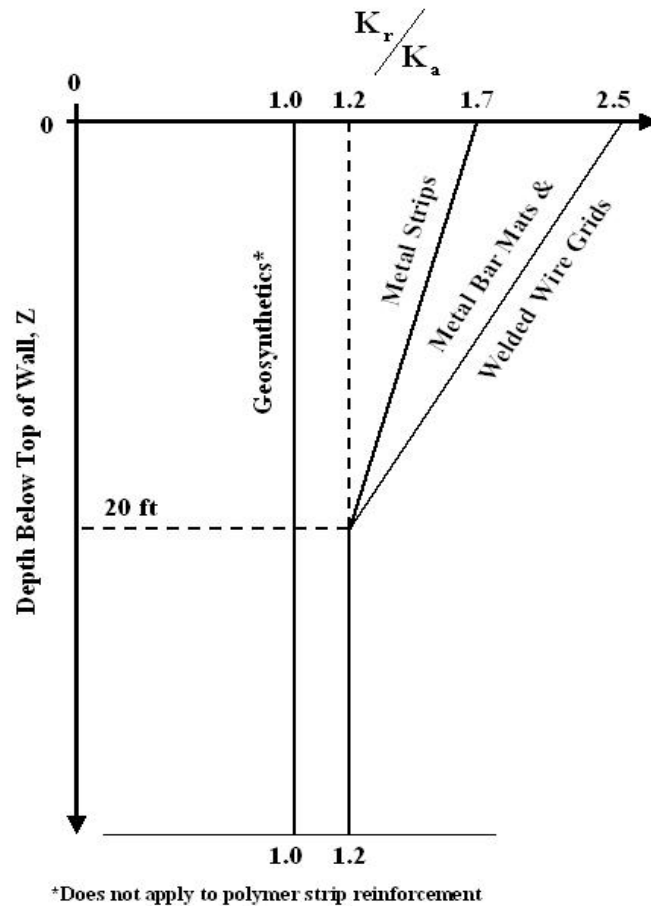


Figure 5.13 : Variation of the coefficient of lateral stress ratio (K_r/K_a) with depth in a MSE wall (Elias and Christopher, 1997; AASHTO, 2002; AASHTO, 2007).

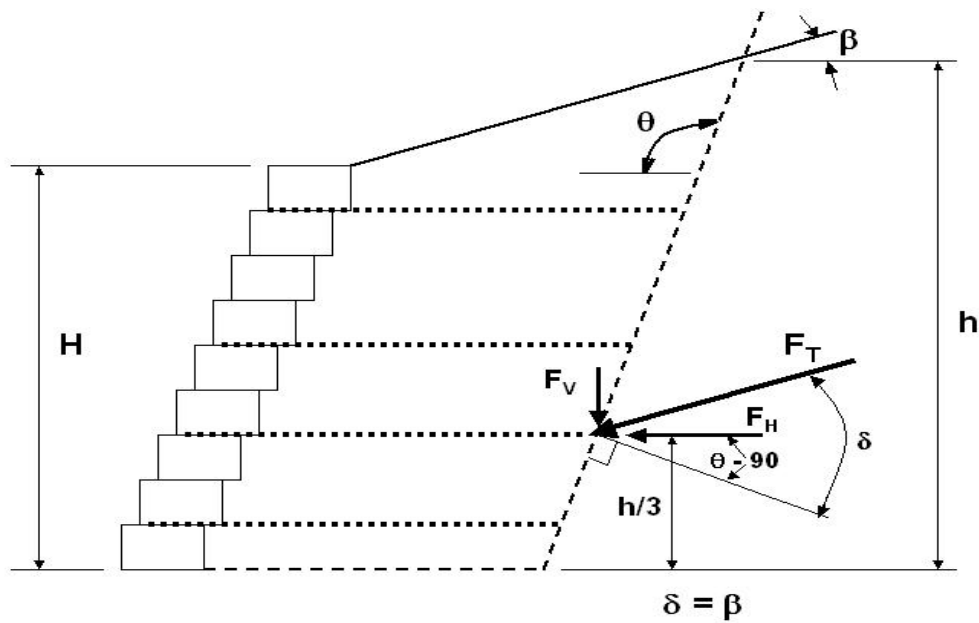


Figure 5.14 : Notation for Coulomb active earth pressures used with wall batters, θ , greater than 100 degree (AASHTO, 2007).

Therefore, the earth pressure reduces to the Rankine equation for a vertical wall:

$$K_A = \tan^2(45 - \frac{\phi_r}{2}) \quad (5.25)$$

For wall face batters equal or greater than 10° from the vertical, the following simplified form of the Coulomb equation can be used:

$$K_A = \frac{\sin^2(\phi + \phi_r)}{\sin^3\theta \left[1 + \frac{\sin\phi_r}{\sin\theta} \right]} \quad (5.26)$$

Where:

θ : Is the inclination of the back of the facing as measured from the horizontal which starts in front of the wall, as shown in Figure 5.14. Backfill on top of an MSE wall started stress.

σ_2 : Is shown in Figure 5.15

The parameters representing the total height are computed based upon the slope geometry. It is important to check the value of S_{eq} to not exceed the slope height for the broken back sloping fills. The vertical stress is equal to the product of equivalent soil height and the reinforced fill unit weight, and is uniformly applied across the top of the MSE zone.

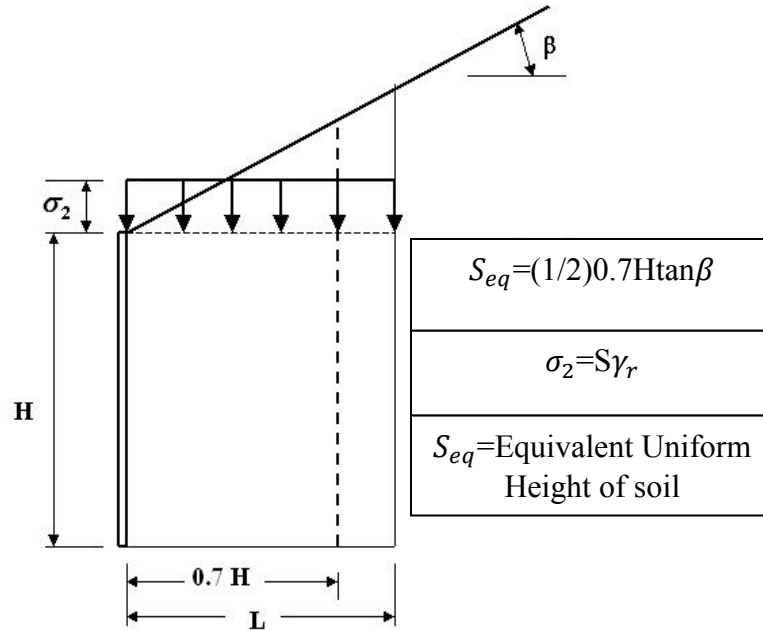


Figure 5.15 : Calculation of vertical stress for sloping backfill conditions for internal stability (FHWA, 2009).

5.6.2 Establish vertical layout of soil reinforcements

The design of the reinforcement length and spacing between them is done in order to reach the most economical engineering design, the use of a constant reinforcement unit in section and spacing for the full height of the wall is not recommended because this method gives more reinforcement in the upper section of the wall compared with required stripes for stability. Therefore, for more economical design may be it is possible to change the reinforcement density by increasing the depth.

For example in the top elevations of the wall the use of the thin strips 50 x4 or 50 x4 are more proper than in the bottom; with depth we can use the stripes with more thickness but fewer ties and less spacing than the top. However, to provide a coherent reinforced soil zone, vertical spacing of reinforcement should not exceed 800 mm.

There are generally two feasible methods to carry out this for MSE walls:

The constant reinforcement in identified spacing is constant. In these cases for reinforcements consisting of strips and grids used with precast concrete facings, and the reinforcement size, which means the size and tie or spacing numbers of stripes are constant with depth, but the type on reinforcement varies. For instance, the typical horizontal spacing of 40 x 4 mm in one panel by increasing the strips is changed to 50 x 4 mm in to attain stability.

The second and common method in designing MSE walls is changing the type and length of reinforcement besides the spacing by increasing the depth in order to reach an economical solution. For instance, in continuous sheet reinforcements, which are made of geotextiles or geogrids, a usual method is changing the reinforcement density T_{al}/S_v by changing the vertical spacing S_v , particularly if wrapped facing is used; especially it simply accommodates spacing variations.

The acceptable ambit of spacing is taken into account by considering of placement and compaction of the backfill. The reinforcement density T_{al}/S_v could be variable by changing the strength (T_{al}), particularly if wrapped facing techniques requiring a constant wrap height are used.

5.6.3 Calculation of factored tensile forces in the reinforcement layers

In the calculation of internal stability, evaluation of horizontal stress for internal stability analysis and the distribution of horizontal stress (σ_H) is the first step and it can be obtained at any depth within the reinforced soil zone as below:

$$e_H = K_r[\sigma_V] + \Delta\sigma_H \quad (5.27)$$

Where:

K_r : Defined as the coefficient of lateral earth pressure in the reinforced soil zone. It is obtained from Figure 5.13.

σ_V : Vertical pressure at the specified depth.

σ_H : Supplemental factored horizontal stress as a result of external surcharges.

In the internal stability, some assumptions are made to compute the factored vertical pressures, which are listed below:

I) Weight of the reinforced soil zone causes vertical stress, and this type of load is defined as a load type “EV” corresponding to the maximum load factor. The maximum load factor $\gamma_P - EV = 1.35$ is always considered to evaluate the critical stress.

II) All of the vertical surcharges on top of the wall which are originated from soil above the reinforced soil mass are taken into account as a load type “EV.” Furthermore, live loads like traffic surcharge and railway are represented by an equivalent that the uniform soil surcharge of height h_{eq} assumed as load type “EV” .

In external stability analysis, the live load traffic surcharge is considered as load type “LS” because in external stability analysis, the reinforced soil mass is accepted to be a rigid block. In the load type, in contrast with internal stability analysis, the load type “EV” is used so check the amount of soil reinforcement within the reinforced soil zone.

III) Unit weight of the reinforced soil zone γ_r is usually greater than that of backfill, but in calculation the unit weight of surcharge is accepted equal to the unit weight of reinforced mass.

IV) Another vertical surcharge which is based on project and is not related to soil source is considered as a load type “ES” For instance, the bearing pressure load under a spread footing on top of reinforced soil zone belongs to load type “ES”.

If the vertical pressures are based on unfactored loads, it is obtained as following:

$$\gamma_{P-ES} = 1.50$$

If the vertical pressures are based on factored loads (it means the loads are once factored), they should not be factored again; therefore, it is assessed as below:

$$\gamma_{P-ES} = 1.00$$

The factored horizontal pressure, $\Delta\sigma_h$, could be developed by the effect of several sources. Two main instances of horizontal pressures are mentioned below:

In the first example, the horizontal pressures are formed due to the horizontal shear stresses at the bottom of a spread footing on top of the reinforced soil zone.

The second example is about deep foundation elements extending through the reinforced soil zone which leads to horizontal pressures.

The horizontal pressures are classified in load type “ES” categories. In the application of the maximum load factor of $\gamma_{P-ES} = 1.50$ is allocated to the load type “ES” and the function of horizontal pressures is obtained as follows:

When the horizontal pressures are considered as unfactored loads, the use of $\gamma_{ESMAX} = 1.50$ is recommended, but when the horizontal pressures are based on factored loads, that means the load was previously factored, so the use of $\gamma_{P-ES} = 1.00$ is urged.

The examples and calculation methods which are illustrated below are for three main types of mechanically stabilized earth walls. The wall design method is ranging from simple to complex geometries. In some cases, the loads are not as simple as below. In these cases, the engineering logic is used to develop special equations which could be extended to the complex system of surcharges.

In case one, MSE wall with backfill above it without any surcharge is considered. This case represents the simplest MSE wall geometry.

In this case, the horizontal stress at any given depth Z below the top elevation of the wall is given as follows:

$$\sigma_H = K_r[(\gamma_r Z)\gamma_{EV-MAX}] \quad (5.28)$$

Where:

γ_r : Unit weight of soil in the reinforced soil zone

γ_{EV-MAX} : Maximum load factor, and it is equal to 1.35 for load type “EV”

In the computation of value K_r , two following assumptions are made:

(I) The value of Kr/Ka ratio Kr/Ka is illustrated in Figure 5.13 which begins from the top of the reinforced soil zone and it depends on type of reinforcement.

(II) K_a is obtained by using the Rankine method.

The second case is usually confronted in side-hill retaining wall applications. In this case, MSE wall has a sloping backfill. Distribution of stress from concentrated vertical load for internal and external stability is shown at Figure 5.16. An exemplified sketch of this case is demonstrated in Figure 5.11; the sloping surcharge is approximated by an equivalent uniform soil surcharge of height, S_{eq} .

The horizontal stress at any depth Z below the top of the wall can be calculated as follows:

$$\sigma_H = K_r [\gamma_r (Z + S_{eq}) \gamma_{EV-MAX}] \quad (5.29)$$

The value of K_r is obtained by assuming that:

(I) The value of ratio Kr/Ka is obtained by considering Figure 5.13.

(II) K_a is computed by using the Rankine formula.

The third case is the MSE wall with both backfill and live load surcharges. These cases are usually confronted in grade-separated roadways. Distribution of stresses from concentrated horizontal loads is illustrated at Figure 5.17. We accept the live load and soil surcharge as a uniform soil surcharge of height, h_{eq} ; the horizontal stress at any depth Z below the top of the MSE wall could be expressed as follows:

$$\sigma_H = K_r [\gamma_r (Z + h_{eq}) \gamma_{EV-MAX}] \quad (5.30)$$

To obtain the value of Kr some assumptions are made:

(I) The variation of Kr/Ka ratio illustrated in Figure 5.13 depends on depth and reinforcement type.

(II) K_a is obtained by using the Rankine formula.

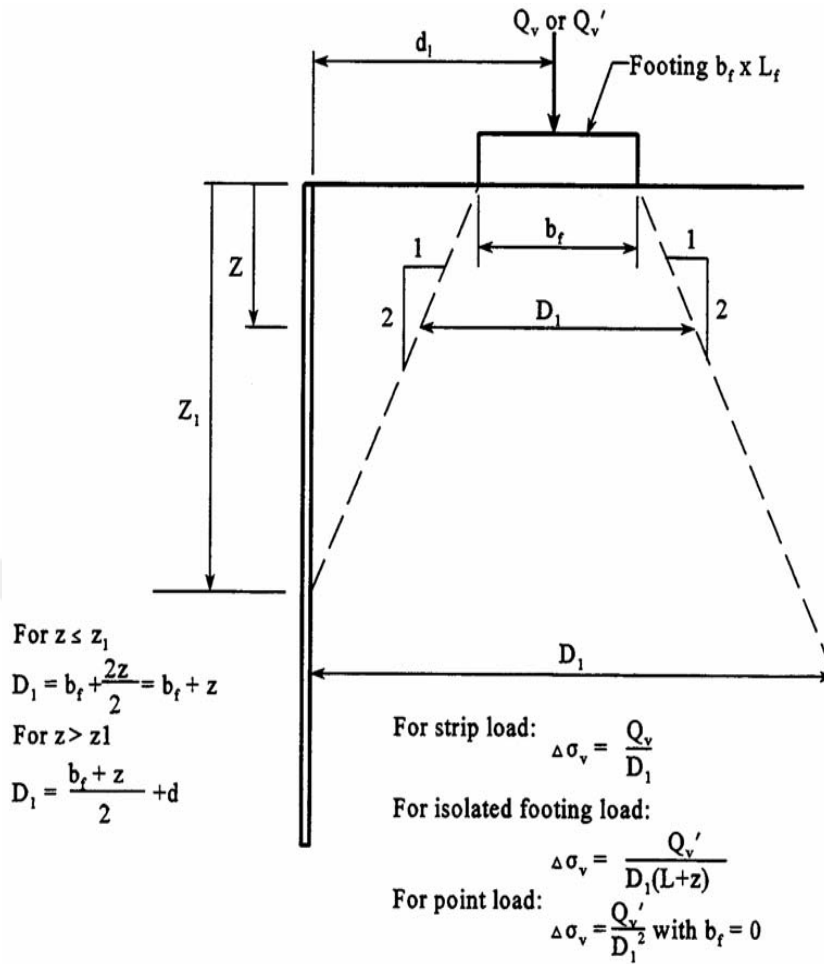


Figure 5.16 : Distribution of stress from concentrated vertical load for internal and external stability calculations (FHWA, 2009).

Where:

D_1 : Effective width applied load at any depth, calculated as shown above

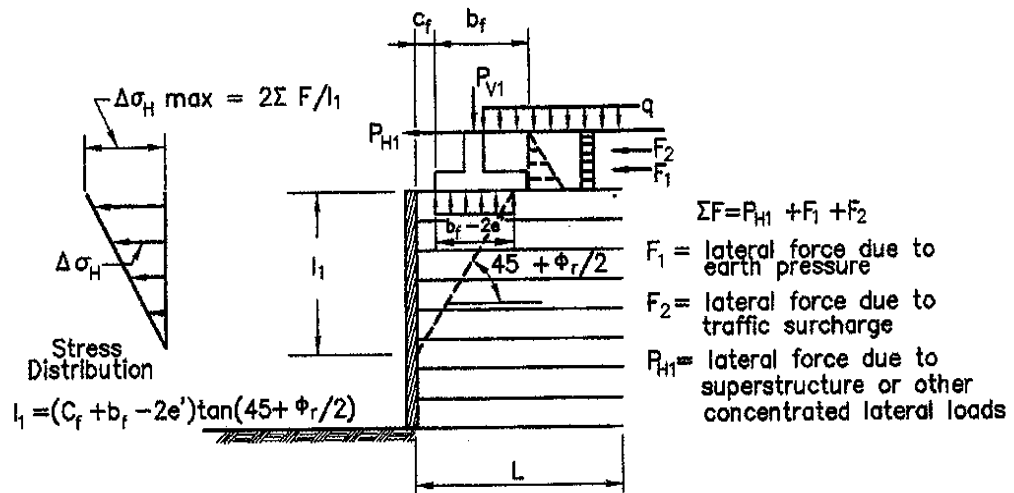
b_f : Width of applied load. For Footings which are eccentrically loaded (bridge abutment footings), set b_f equal to the equivalent footing width B' by reducing it by $2e'$, where e' is the eccentricity of the footing load (i.e., $b_f - 2e'$).

L_f : Length of footing Load per linear meter of strips footing

Q_v : Load per linear meter of strips footing

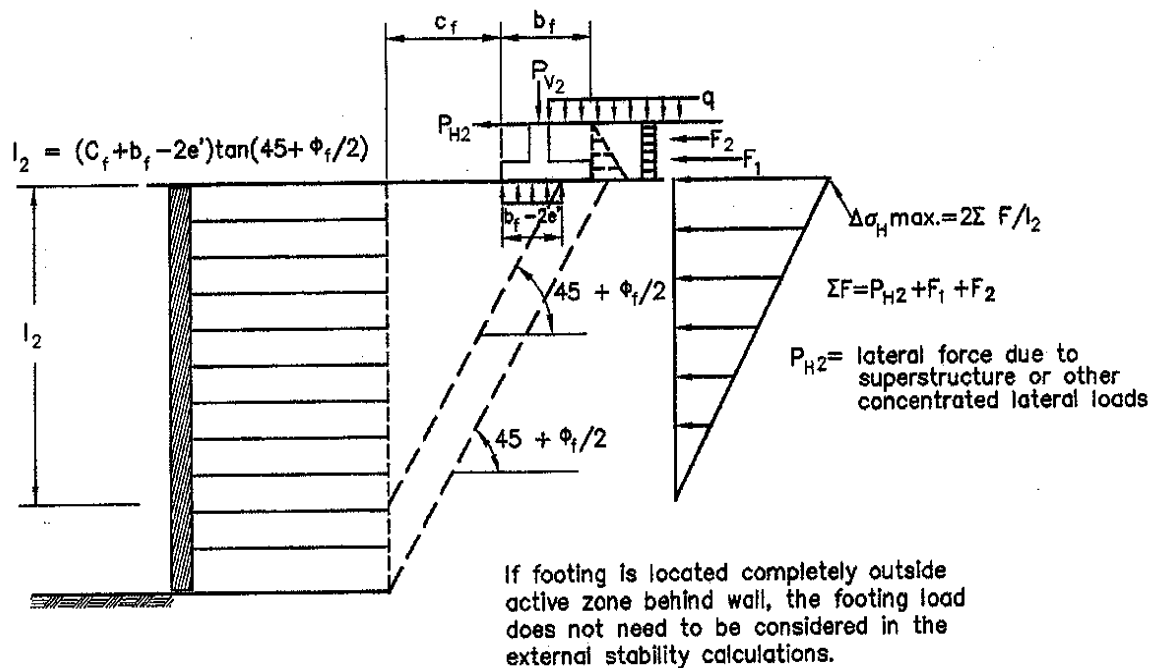
Q'_v : Load on isolated rectangular footing or point load

Z_1 : Depth where effective width intersects back of wall face $= 2d_1 - b_f$



e' = eccentricity of load on footing (see Figure 11.10.10.1-1 for example of how to calculate this)

a. Distribution of Stress for Internal Stability Calculations.



b. Distribution of Stress for External Stability Calculations.

Figure 5.17 : Distribution of stresses from concentrated horizontal loads (FHWA, 2009).

Assume the increased vertical stress due to the surcharge load has no influence on stress used to evaluate internal stability if the surcharge load is located behind the reinforced soil mass. For external stability, assume the surcharge has no influence if it is located outside the active zone behind the wall.

5.6.4 Maximum Tension Calculation

Maximum factored tension, T_{MAX} is obtained for each layer of reinforcement per unit width of wall based on the vertical spacing S_v from:

$T_{MAX} = \sigma_H S_v$ The term S_v is equal to the vertical reinforcement spacing for each layer where vertically adjacent reinforcements are equally spaced from the layer under consideration (Figure 5.18). The term, σ_H , is obtained from the level of the reinforcement, exactly at the midpoint between vertically adjacent reinforcement elevations. The important point, which is different from middle of the wall, and should be taken into consideration, is S_v at top and bottom of the wall. For these two parts (top and bottom layers of reinforcement) S_v is the total space between mentioned layer to top and bottom of the wall, which is the midpoint between the first and second layer. For better perception, S_v distances are shown in Figure 5.18.

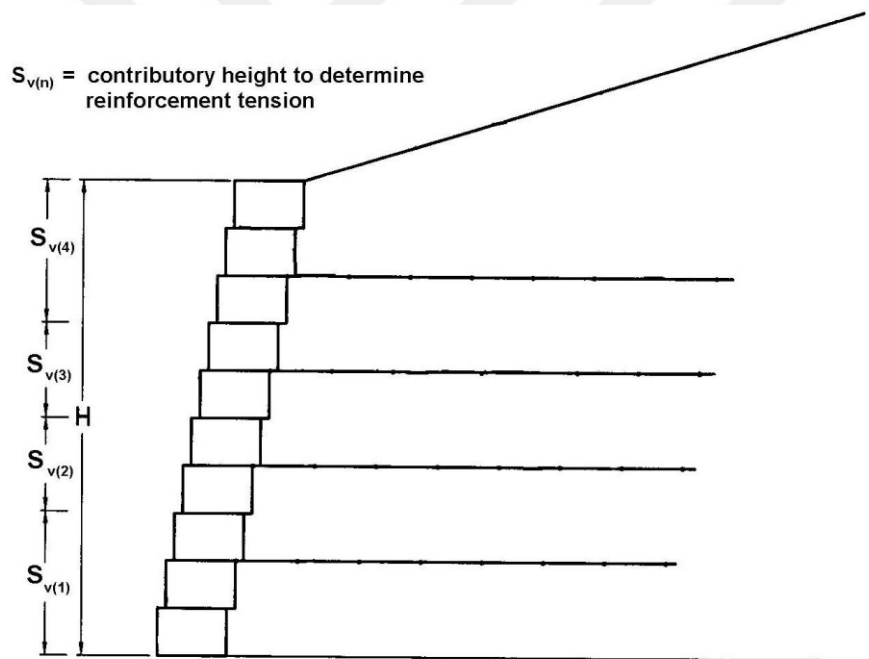


Figure 5.18 : Reinforcement load contributory height (FHWA, 2009).

At the top, middle, and the bottom layer of the reinforcement, where the spacing is not equal to adjacent layers, the maximum reinforcement tension is obtained from the product of the contributory height and the average factored horizontal stress acting upon that contributory height.

The maximum reinforcement tension, T_{MAX} , for the top and bottom layers of reinforcement and for intermediate layers that do not have equally spaced to adjacent

layers, is calculated as the product of the contributory height and the average factored horizontal stress acting upon that contributory height.

The average stress can be computed with regard to the average of the stress at the top and the bottom of the contributory height or at the midpoint of the contributory height.

The alternative way to calculate the maximum reinforcement tension for distinct reinforcements like metal strips, bar mats and geogrids could be calculated from equation $P_{TMAX-UWR}$ in terms of force per unit width of reinforcement as below:

$$P_{TMAX-UWR} = \frac{\sigma_H S_V}{R_C} \quad (5.31)$$

Where:

R_C : Is the ratio of the width of strip, sheet, or grid to the distance of center of the horizontal reinforcement to the adjacent reinforced level.

R_C : For full coverage reinforcement is equal to 1.

In precast concrete panels for the known reinforcement by known panel dimensions, T_{MAX} (force per unit width) can alternatively be calculated per reinforcement, $P_{TMAX-UWR}$, per panel width, defined as:

$$P_{TMAX-D} = \frac{\sigma_H S_V W_P}{N_P} \quad (5.32)$$

Where:

P_{TMAX-D} : Maximum factored load in discrete reinforcement element

W_P : Width of concrete panels

N_P : Number of discrete reinforcements per panel width; it actually depends on the type of panel

5.6.5 Calculation of soil reinforcement's resistance

The factored soil resistance is gained by product of three parameters, the nominal long-term strength, coverage ratio, and applicable resistance factor (ϕ).

The resistance factors for tensile rupture of MSE wall soil reinforcements are categorized by type of reinforcement in Table 5.2.

The factored tensile resistance T_r is equal to:

$$T_r = \phi T_{al} \quad (5.33)$$

T_{al} and T_r may be explained in terms of strength per unit width of wall, per reinforcement element, or per unit reinforcement width.

Table 5.2 : Resistance factors, for tensile and pullout Resistance for MSE walls (AASHTO, 2007).

Reinforcement Type and Loading Condition			Resistance Factor
Metallic reinforcement and connectors	Strip reinforcements ^(A)		
		Static loading	0.75
		Combined static/earthquake loading	1.00
		Combined static/traffic barrier impact ^(B)	1.00
	Grid reinforcements ^(A, C)		
		Static loading	0.65
		Combined static/earthquake loading	0.85
		Combined static/traffic barrier impact ^(B)	0.85
Geosynthetic reinforcement and connectors		Static loading	0.90
		Combined static/earthquake loading	1.20
		Combined static/traffic barrier impact ^(B)	1.20
Pullout resistance of tensile reinforcement (metallic and geosynthetic)		Static loading	0.90
		Combined static/earthquake loading	1.20
		Combined static/traffic barrier impact ^(B)	1.00

Notes:

- A. Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with AASHTO (2007) Article 6.8.3 and apply to net section less sacrificial area.
- B. Combined static/traffic barrier impact resistance factors are not presented in AASHTO.
- C. Applies to grid reinforcements connected to rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

5.6.6 Select grade of soil reinforcement elements at each level

After these three listed steps were determined:

- Soil reinforcement vertical layout.
- Factored tensile force at each reinforcement level.
- Factored soil reinforcement resistance.

The next step is selecting proper grades by considering the reinforcements strength and number of reinforcements (strips) for the accepted vertical reinforcement layout. Then with this layout, the pullout with extreme event loadings should be checked.

Stability with respect to breakage of the reinforcements requires that:

$$T_{MAX} \leq T_r \quad (5.34)$$

Where:

T_{MAX} : Maximum factored load in a reinforcement

T_r : Factored reinforcement tensile

5.6.7 Internal stability with respect to pullout failure

In order to check stability regarding the pullout of the reinforcements, the division of the factored effective pullout length to factored tensile load in the reinforcement should be greater than 1. Therefore, each layer of reinforcement should be checked, as the pullout resistance. The tensile loads may be different from reinforcement layer. Thus, the equation below should be satisfied:

$$\phi L_e \geq \frac{T_{MAX}}{F^* \alpha \sigma_v C R_C} \quad (5.35)$$

Where:

L_e : Length of embedment in resisting zone (Figure 5.19)

T_{MAX} : Maximum tension of reinforcements

ϕ : Resistance factor for soil reinforcement pullout

F^* : Pullout resistance factor which depends on depth

α : Scale correction factor

σ_v : Unfactored vertical stress at the reinforcement level in the resistant zone, by adding distributed dead load surcharges. For better perception refer to Figure 5.18.

C : Is equal to 2 for strip, grid, and sheet type reinforcement

R_C : Coverage ratio

Therefore, the required embedment length in the resistance zone beyond the potential failure surface can be computed as below:

$$L_o = \frac{T_{MAX}}{\phi F^* \alpha \sigma_V C R_c} \geq 1m \quad (5.36)$$

The length of the strips in active zone and Nominal vertical stress at the reinforcement level in the resistant zone is demonstrated at Figure 5.19.

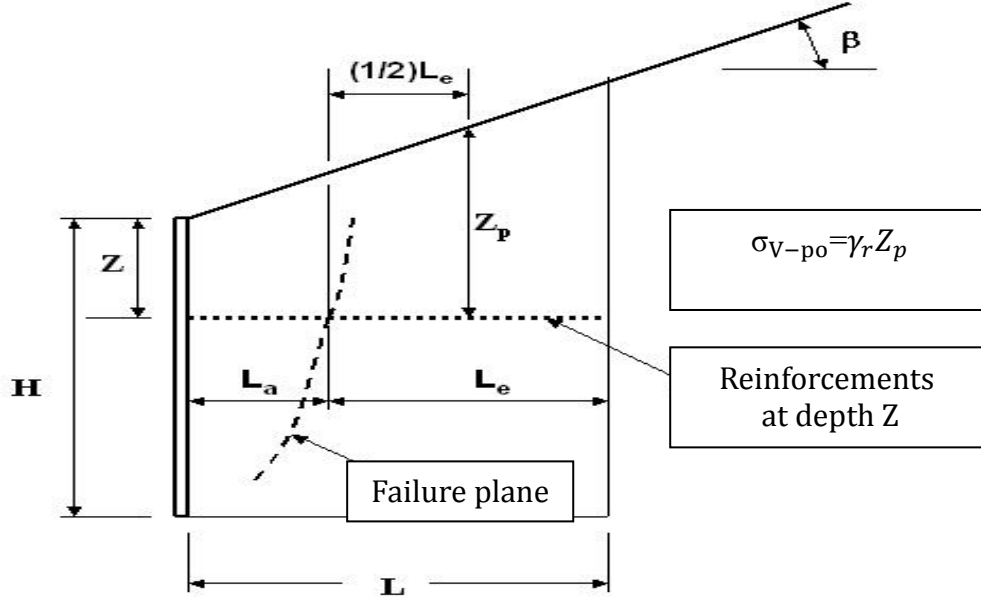


Figure 5.19 : Nominal vertical stress at the reinforcement level in the resistant zone, Beneath a sloping backfill (FHWA, 2009).

Therefore, the required embedment length in the resistance zone beyond the potential failure surface can be computed as below:

$$L_o = \frac{T_{MAX}}{\phi F^* \alpha \sigma_V C R_c} \geq 1m \quad (5.36)$$

When the traffic or other live loads exist above the wall, it is recommended to consider these points:

Maximum reinforcement tension must be calculated with the live loads, Pullout resistance could be obtained without considering the live loads.

The other possibility is the live loads being present near the front of the wall but not above the reinforcement embedment length. If the live load is located on the active and resistant zones at the same time or on the resistant zone alone, the pullout resistance and the calculation of T_{MAX} could be done without the live load (AASHTO, 2007).

If the criterion is not satisfied for all reinforcement layers, the reinforcement length must be increased, but if it still does not reach the criterion points, the reinforcement with a greater pullout resistance per unit width must be used, or the reinforcement vertical spacing may be reduced which would reduce T_{MAX} .

$$L = L_a + L_e \quad (5.37)$$

Where:

L_a : Is obtained from Figure 5.19 for simple structures not supporting concentrated external loads such as bridge abutments. Based on this figure, the following relationships can be obtained for L_a :

For MSE walls with extensible reinforcement, vertical face and horizontal backfill:

$$L_a = (H - Z) \tan(45 - \frac{\phi'}{2}) \quad (5.38)$$

Where:

Z : Depth to the reinforcement level

For walls with inextensible reinforcement, vertical face and horizontal backfill from the Base up to $H/2$:

$$L_a = 0.6(H - Z) \quad (5.39)$$

For the upper half of a wall with inextensible reinforcements, vertical face, and horizontal backfill is calculated from equation below:

$$L_a = 0.3H \quad (5.40)$$

For construction ease, a final uniform length is commonly chosen based on the maximum length required for each panel. However, if internal stability controls the length, it could be varied from the base, increasing with the height of the wall to the maximum length requirement based on a combination of internal and maximum external stability requirements.

5.6.8 Checking connection strength

The connection of the reinforcements with the facing should be designed for T_{MAX} for all limit states. The resistance factors for the connectors are the same as for the reinforcement strength and are listed in Table 5.2 (Article 11.10.6.2.2 (AASHTO, 2007)).

5.6.9 Connections to concrete panels

The metallic reinforcements for MSE systems are fabricated with segmental precast panels. These joint parts are structurally connected to the facing, and the reinforcements are connected to joints by bolting to a tie strip cast in the panel.

The capacity of the embedded connector as an anchorage must be checked by tests as required by Article 5.11.3 AASHTO (2007) for each geometry used.

5.6.10 Lateral movements

The evaluation of lateral wall movements in LRFD is the same as in ASD as the deformations are evaluated at the Service limit state.

In general, most internal lateral deformations of an MSE wall face usually occur during construction. Post construction movements, may take place due to post construction like surcharge loads, selected backfill settlements, or long-term settlement of the soil that is MSE wall is constructed.

The magnitude of lateral displacement depends upon fill placement and compaction technique, vibration of compaction, reinforcement extensibility, reinforcement length, reinforcement-to-facing connection details, and details of the wall facing. The rough estimate of probable lateral displacements of simple MSE walls that may occur during construction can be estimated based on empirical correlations (Figure 5.20). In general, increasing the length-to-height ratio of reinforcement, from its theoretical lower limit of $0.5H$ to the AASHTO specified $0.7H$, decreases the deformation by about 50 percent. For critical structures, such as bridge abutments, a more precise design analysis is required because more accurate calculations using numerical modeling may be warranted by the estimation of the deformation.

A deformation response analysis allows the design engineer to anticipate the performance of the structure with respect to horizontal (vertical) displacement. Horizontal deformation analyses are the most difficult and least certain of the performed analyses. In many cases, they are done only approximately. The results may impact the choice of facing, facing connections, or backfilling sequences.

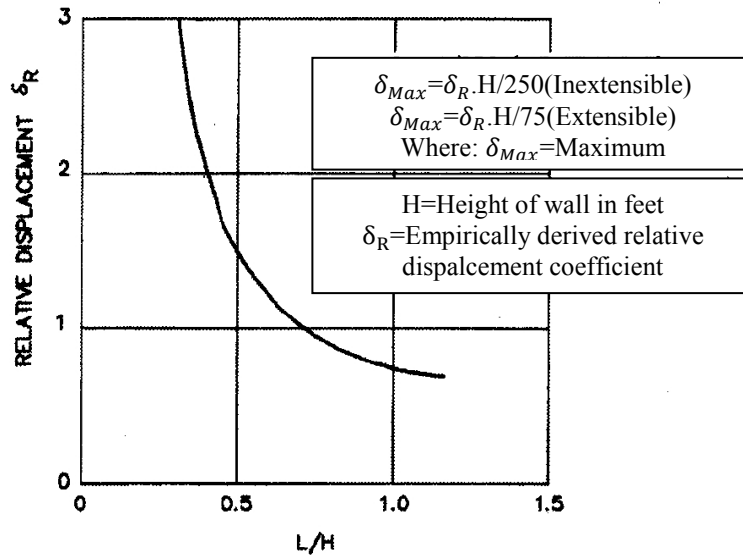


Figure 5.20 : Empirical curve for estimating lateral displacement during construction for MSE walls (Christopher et al, 1990).

This figure is only a guidance to estimation of displacements, Actual displacement will depend, in addition to the parameters addressed in the figure, on soil characteristics, compaction effort, and contractor (For $L = 0.7 H$).



6. INTRODUCTION TO THE SOFTWARE

6.1 Limit State Design

Limit state design (LSD) is also known as load and resistance factor design (LRFD), relates to a design method used in structural engineering. A limit state is actually is a design criteria condition which the structure beyond that no longer accomplishes the applicable design criteria. The mentioned terms may refer to a degree of loading or other forced and factors that have affected the structure. While the conditions refer to structural entirety, thickness of using material, durability or other design requirements is included. A structure designed by LSD is adjusted to resist against all probable actions may have to occur during its design life, and to remain safe for use, with an adequate level of reliability for each limit state.

The method of limit state design, acquired in the, It was introduced for a first time in USSR building regulations in 1955.

Limit state design consist of two criteria which must be satisfy:

- The ultimate limit state (ULS).
- The serviceability limit state (SLS).

In limit state design procedure is a numbers of assumption was made and the loads which the structure is exposure to them estimated sizes of members to check must be chosen and design criteria must be selected. The design criteria have a main aim that of ensuring a safe structure and ensuring the functionality of the structure in their expected life time.

6.1.1 Ultimate limit state

The ultimate limit state is a concurred and accepted computational condition that must be accomplish, by considering other subsidiary criteria, in order to fulfill the engineering requirements for strength and stability under design loads.

6.1.2 Serviceability limit state

After the first step is completed (ultimate limit state), the next step is the Service Limit State (SLS).

Service Limit State is the same ultimate limit state, it is not a physical situation and it is a computational control. The intention of this check is to prove that, the structure under the applying design loads, while applying specified magnitudes of imposed deformations, settlements, vibrations, or temperature gradients is safe and reliable.

6.1.3 Assumptions in limit state calculations

In practice the design of Reinforced Earth walls are done with limit state calculations, which are proved to be safe over the experiments of many years, and the quickest way to calculate the MSE walls. One of the basic assumption is made in the soil reinforcement is the interaction by a certain friction (NFP 94270).

Coefficient is exerted to the overburden pressure for computation of pull out resistance of soil. The section assumption is made in defining the active zone of the soil, as previously discussed, depending on the experimental studies, the researchers provided valuable documentation on the locus of the active zone and resisting zone's boundaries for both inextensible and extensible reinforcements (Figure 6.1).

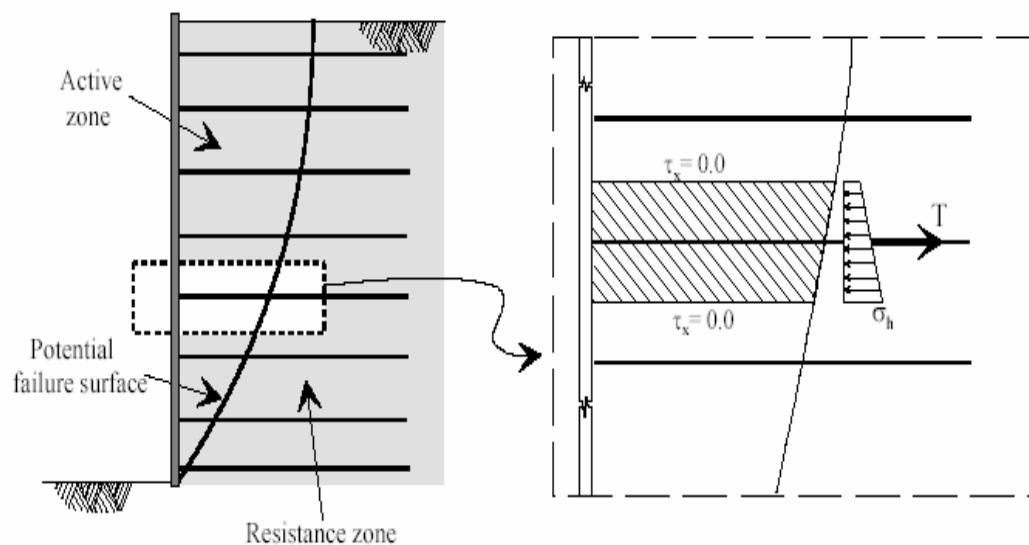


Figure 6.1 : Active and Resisting Zones in Reinforced Earth Structures (Liang, 2004).

6.2 Introduction to Valdez Software

The Valdez program specifically prepared to analysis the MSE wall for the design of Reinforced Earth walls. This program is user friendly and provides two type of design cross section rectangular and trapezoidal. Further more you can design tired wall with various setback for walls. The method of design of Valdez program is based on the limit state design procedures. As I previously mentioned, the limit state design method is prepared in order to the limits of the materials used and the loads under the most critical conditions .

The essential control for external stability is made by Valdez program by considering the wall as a total (Reinforced zone and strips).

In the internal stability check is obtained by calculating the lateral earth pressure on panel segment (Facing panel based on program output is calculated as reinforced and unreinforced) of the wall system by considering the appropriate amount of strips which are working as friction elements.

Valdez program is widely used in MSE wall design for its capability of design in various geometries, and design calculation procedure in short time after the creation of the strip library on the different strips. However the limit state method is the quickest way to compute the exact and reliable quantity and length of reinforcing strips required for the retaining wall by considering the design codes of any countries (Durgunoğlu et al, 2007).

6.3 Finite Element Method (FEM)

Courant, who employ the Ritz method of numerical analysis and minimization of variational calculus to gain solutions to vibration systems This method (FEM) is a numerical technique for finding approximate solutions to boundary value problems for partial differential equations. It is also referred to as finite element analysis (FEA). FEM subdivides a large problem into smaller, simpler, parts, called finite elements. The simple equations that model these finite elements are then assembled into a larger system of equations that models the entire problem. After that it uses variational methods from the calculus of variations to approximate a solution by minimizing an associated error function.

6.4 Introduction to Plaxis Software

PLAXIS is a finite element package which is prepared for the two and three dimensional analysis of deformation and stability in geotechnical engineering. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time dependent and anisotropic behavior of soils and rock. Furthermore, since soil is multi-face material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. Although the modeling of the soil is an important issue, many projects require modeling structures and interaction of the structures with soil (Plaxis Manual, 2016).

6.5 Introduction to Slide Software

Slide is software which is developed by Rocscience for analyzing the two-dimensional slope stability. Slide program output can be used in both design and analyze of natural slopes or any kind of disturbed slopes like cuts, embankments and fill and MSE walls. Slide software has able to analyze a single user-defined or non-circular failure surface. In the cases with complex geometry this software has able to research and analyzed for composite surfaces comprising both a circular and non-circular component.

Slide has a user friendly graphical interface, and provides a wide range of modeling and data interpretation functionalities, and also the geometry could be imported from Autocad as dxf file.

It calculate safety factors for static case and by considering horizontal acceleration of region for seismic cases for circular and noncircular slope failure surfaces by using a number of widely used limit equilibrium analysis approaches such as the Bishop, Janbu, Mohr coulomb methods.

The graphical data interpreter gives users a rich set of tools for viewing and analyzing model results. As a result of the improvements to Slide, engineers now have a software package created specifically for performing the tasks involved in analyzing and designing slopes. By the finding slope failure surface, it is prepared wide range information for designing nailing and find out the active and resistance zone in slopes and cuts (Slide 5.0 User Manual, 1997).

6.6 Introduction to RocLab Software

RocLab is software for evaluating the rock mass strength parameters, according to the generalized Hoek-Brown failure criterion.

One of the problems in numerical modeling for rock mechanics is the inaccurate data for input for rock mass properties.

The numerical analysis programs, is not reliable, if the analyst does not have reliable input data for rock mass properties. The latest version of the Hoek-Brown failure criterion (Hoek and Brown, 1980), in conjunction with its performance in the software program RocLab, has shown a useful performance to estimate the rock properties.

6.7 Case Study Ankara – Sivas High Speed Railway Project-Reinforced Earth Walls

In the content of the Ankara-Sivas speed railway project – The high speed train will pass above the wall, the maximum height of wall which is supported the railway line is around 14 meters. All the design and application procedure were bid by Reinforced Earth Company (Figure 6.2).

This is the first time the Reinforced earth wall has been used for High-speed railway support in Turkey. All the design technology and data which has been used and expressed here exclusively belongs to Reinforced earth company and copying without permission is prohibited.

The preliminary designs prepared by the Reinforced earth company using the Valdez program, which has just given us the length of strips by applying defined safety factors. The program is not able to estimate the displacements in the walls because the designs were based on “Limit State” methods. Which is using the limits of the materials in design but does not take into account the strains formed by stresses. Therefore a new method to calculate the displacements is needed.

In this project because of the sharp cuts which is formed by Basalt, the preliminary designs were done in three alternative and three individual method, Because of the sensitivity of the project and heavy loads, author decided to do this project by three alternative geometry and a numerical analysis of the reinforced walls system.

Numerical analysis methods, as explained in section 6.1 to 6.6, are capable of calculating the displacements in any part of the defined system. This is done by the finite element program Plaxis 2016. This program is chosen because of its wide usage in geotechnical areas. As will be explained, the reinforced earth walls design, will do by limit state method, and then will check by the numerical analysis. In the evaluating of the design parameters the Rock lab program is used. In addition the slide 5.0 is used to calculate the stability and safety of the project.

This wall section is chosen from the most critical point, which has the maximum height. and the plan view is shown in Figure 6.3, Therefore the design is done for the mentioned height.

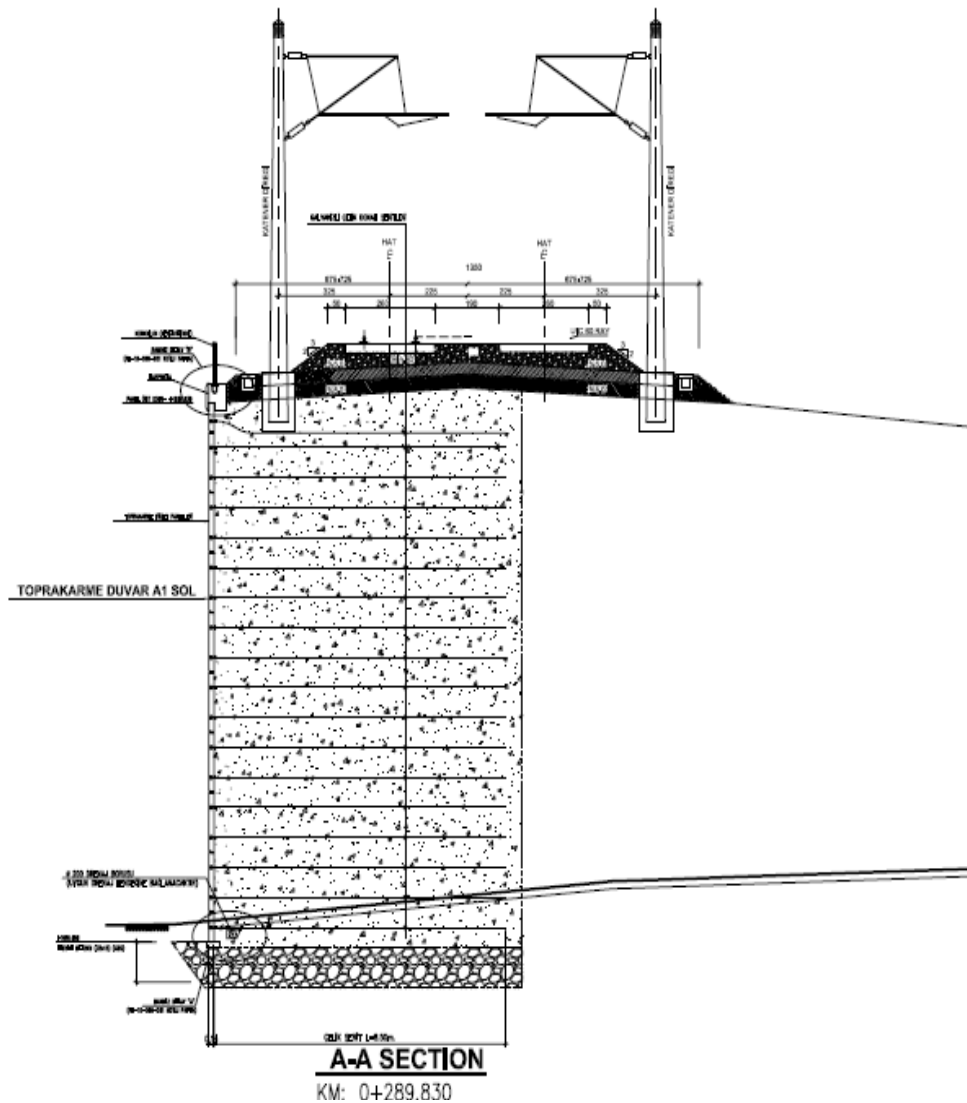


Figure 6.2 : Typical-Cross section of MSE wall for high-speed railways done by Reinforced earth company.

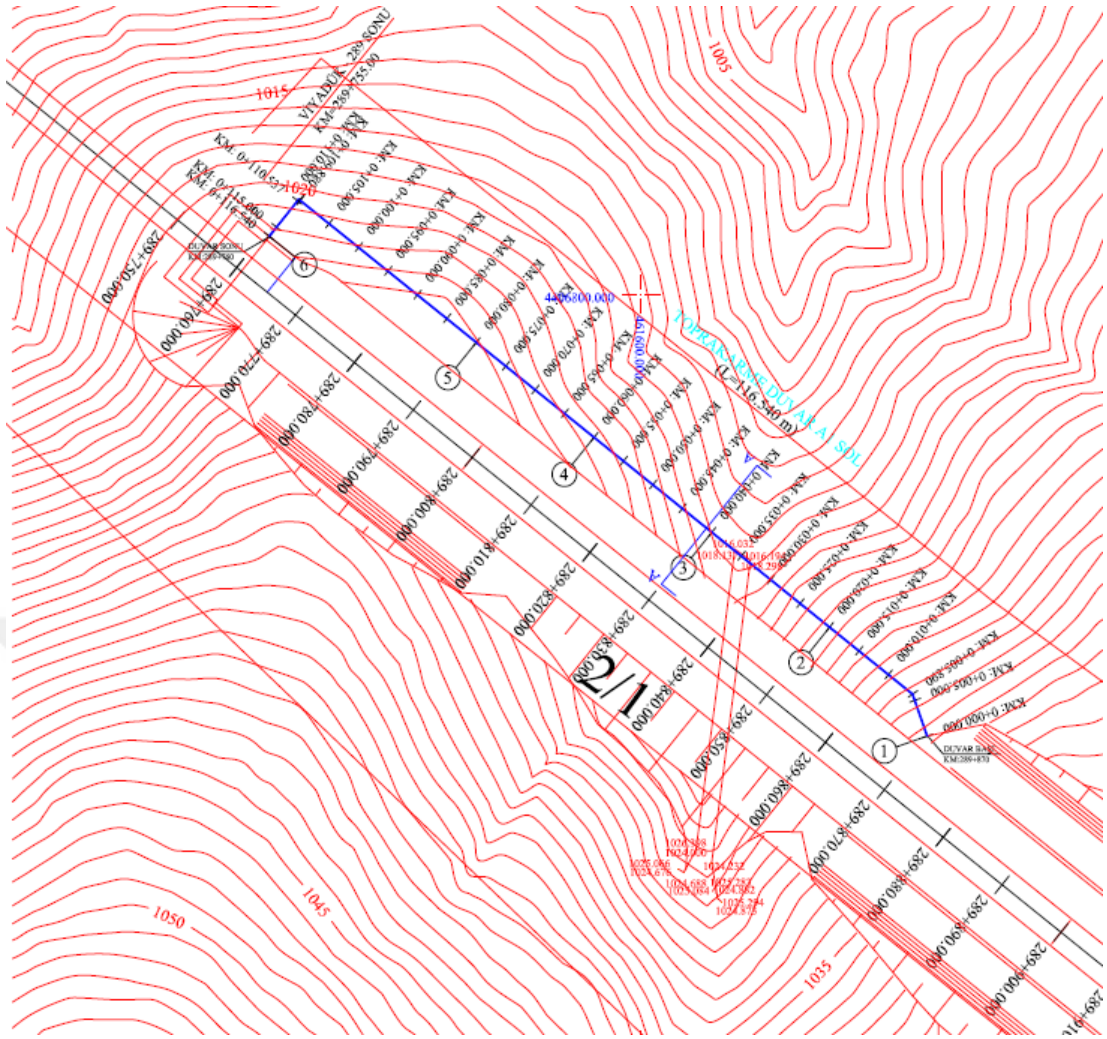


Figure 6.3 : Plan view of the project part 6- KM: 289 (Reinforced earth company, Turkey project 2016).

6.8 Design Criteria and Requirements for Valdez Anlaysia

6.8.1 Subsoil investigations

In the project zone, many regularly samples and boreholes were taken to investigate the subsoil.

The considered three boreholes are actually not located at the exact locus of the wall but they get the perspective about the geology of the region.

In this project because of the sensitivity of the design and in order to minimize the settlement the walls foundation are built on rock, the excavation were continued until to reach the bed rock or incompressible soil.

SPT values found were low in the borehole KM: 289+429 ($N < 20$) in the first 12 m the soil is Silty-Clay, after the depth 12 m to 25 m, the average SPT values are about ($N = 20$), and the soil is Sandy-Gravel.

In the second borehole KM: 289+549, the first 8m of the soil is Silty clay, but after the depth 8m, it turns to the stiff Basalt layer, and the borehole is continued until the depth 12.5m.

The third borehole is taken after the wall location, at the KM: 292+ 200. The soil layers is Stiff Basalt which is started after 30 cm at the surface and continued to the depth 140 m.

By taking all of the data from bore holes, We can elicit that by increasing the kilometer from 289+429 to 292+200, the Silty-Clay layer is diminish and the uniform stiff Basalt is taking place in the Upper layers. Geological profile of wall at KM: 0+289 are illustrated in Figure 6.4.

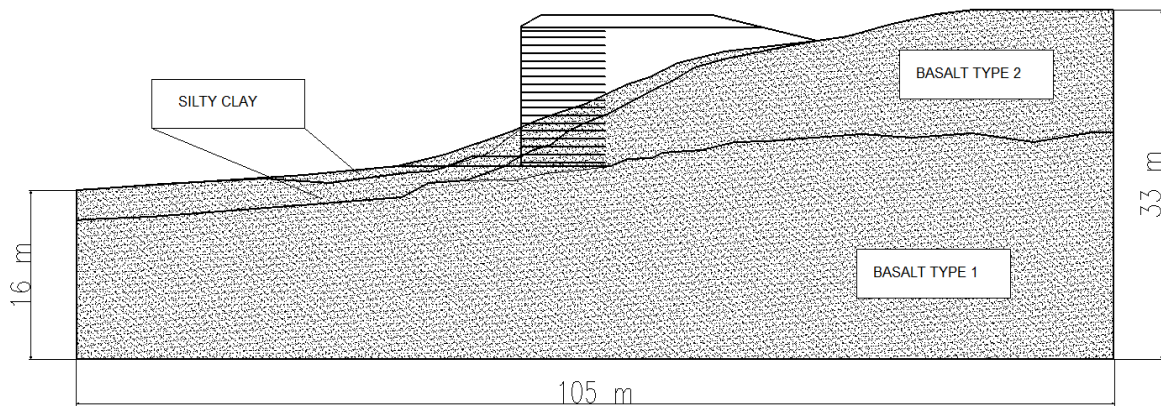


Figure 6.4 : Geological profile of cross section wall KM: 0+289.

For a closer and more precise look to soil layers and because of the existing of variable soil layers according to the boreholes, during the application of the wall several holes were excavated to determine the exact soil layer. In the situations which the soil in foundation of the wall is not suitable, the excavation continued to reach the stiffer layer.

According to geotechnical report and borehole's geology report, the foundation of MSE wall is based on Basalt rock, and the properties of the two type of stiff and decomposed Basalt Pazaecik Volkanitleri is obtain from SK-289+285 ,SK-289+549 and SK-292+200 boreholes.

The brief properties of Basalt type 1 and 2 are listed in Table 6.1 and 6.2, they taken from geotechnical report of the project.

Table 6.1 : Rock properties of Basalt type 1 (Pazarcik Volkaniteleri).

Strength	Strength degree(II)
Decomposition	(I_II)
stiffness	Very Stiff degree (I)
Uniaxial compressive strength	59 Mpa
Layer thickness	Massive
RQD%	Between 20
GSI	54

Table 6.2 : Rock properties of Basalt type 2 (Pazarcik Volkaniteleri).

Strength	Strength degree(II)
Decomposition	(II)
Stiffness	Very Stiff degree (II)
Uniaxial compressive strength	29 Mpa
Layer thickness	Massive
RQD%	Between 15
GSI	50

For numerical model and limit equilibrium analysis it is difficult to find the acceptable equivalent for friction angle and cohesive strength. Therefore, according the proposal of Hoek-Brown failure criterion (Figure 6.5 and 6.6) and in aftermaths at 2002 the “RocLab” software has been developed to allow for an acceptable means of solving and plotting the equations to getting realistic parameters in friction angle and cohesion. (Hoek et al, 2002 edition). In this case by considering Table 6.1 and 6.2 as input data for Roclab1.0 software we elicit the required parameters for Valdez and Plaxis input (Table 6.3 and 6.4).

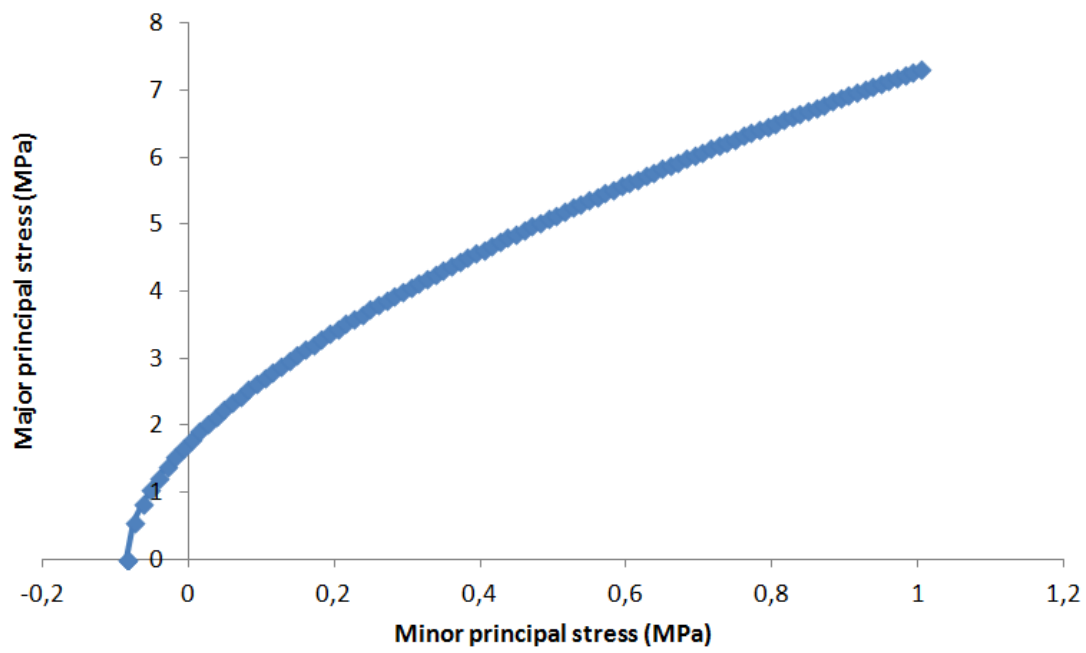


Figure 6.5 : Major principle stress plot minor principle stress for rock lab for Basalt type1.

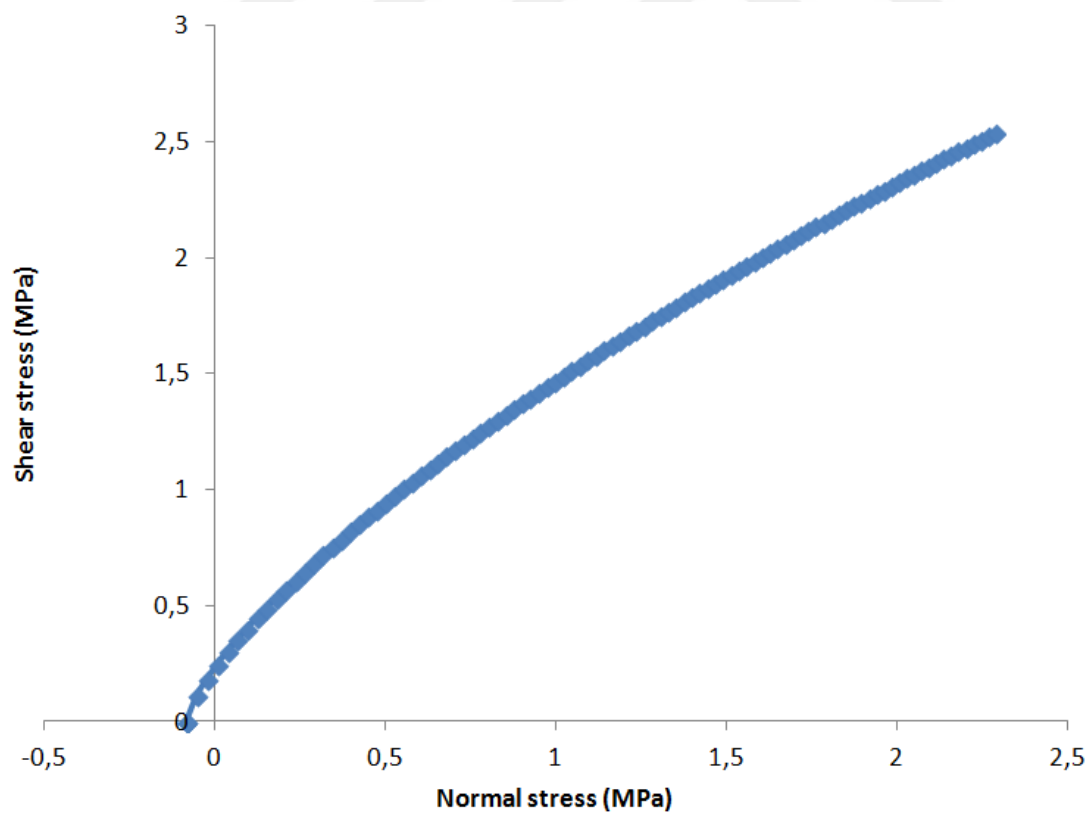


Figure 6.6 : Normal stress plot shear stress from rock lab for Basalt type 1.

Table 6.3 : Soil properties of foundation output from Rock lab for Basalt type 1.

For Basalt Type 1	E Modulus of elasticity (MPa)	γ Unsaturated (KN/m ²)	ϕ' Internal friction angle (degree)	c' Cohesion (KN)
Rock lab	5801	24	42	449
Plaxis (applying safety)	2417	24	42	10
Valdez(applying safety)		24	40	1

Table 6.4 : Soil properties of foundation output from Rock lab for Basalt type 2.

For Basalt Type 1	E Modulus of elasticity (MPa)	γ Unsaturated (KN/m ²)	ϕ' Internal friction angle (degree)	c' Cohesion (Mpa)
Rock lab	3231	21	46	140
Plaxis (applying safety)	1333	21	46	10
Valdez (applying safety)		21	33	10

6.8.2 Selected backfill material

MSE walls require high quality fill mass for better durability, beside the good drainage to avoid excess pore water pressure. In addition to obtain maximum interaction and friction between soil and reinforcement which leads to controlling the deformations, the selected and well graded, granular materials are necessary. The MSE systems depend on friction between the strips that act as reinforcing elements and the soil. In such cases, a material with high friction characteristics is specified and required.

These high performance requirements always limit the soils with high clay contents. In the point of reinforcement capacity, lower quality (limited to the specifications) wall fills could be used for MSE structures but in this point, the strip number in each panel and strips length, may not be economical. A high quality granular wall fill has the advantages of providing better durability for metallic reinforcement, and requiring less reinforcement by the better drainage capability. There are also

important advantages in using granular soils in the case of placement and compaction. Appropriate quality and specification for reinforced fill and design considerations for its use is prepared by Highways Technical Specifications in Turkey (Table 6.5). Suitable fill material shall be granular material which is classified between fine grained uniformly graded silty sand and coarse grained fragmented rock.

Table 6.5 : Selected backfill's grain size distribution according to percent of passing from sieve (KGM, 2013).

SIEVE SIZE (mm)	%PASSING
125 (5'')	100
75 (3'')	85-100
12.5 (1/2)	25-100
2 (No.10)	15-100
0.60 (No.30)	10-65
0.075 (No.200)	< 15

The other limitations that shall be considered when the galvanized steel strips will use as reinforcement are chemical-geotechnical conditions which are listed below.

- PH of the selected backfill must be between 5 and 10.
- CL (Chloride mg/Kg) content of the backfill must be less than 100.
- Total organic material should be less than two percent of total mass.
- Total Sulfate (SO₄ mg/kg) amount of the backfill must be less than 200.
- Minimum Resistivity should be equal to 3000 ohm.cm.

By considering the (KGM-2013) specification and chemical-geotechnical properties of selected back fill the mixed of crashed basalt and fine grain are used in order to reach proper gradation. The vital issue is that, whole system is working together, thus, backfill supplier should regularly check the backfill gradation and compaction.

The properties of selected backfill which is used in design and application of this project is listed below.

In order to gets better understanding of MSE walls working system under the loads and effect of the selected backfill material in deformation of the wall, by considering the constant strip amount, two type of selected backfill with maximum and minimum

friction angle are consider as selected backfill 1 and selected backfill 2. (Table 6.6 and 6.7). In additional three type of backfill material with different plastic and elastic properties are considered as selected backfill according to Hou (2009) and Hossain et al. (2012) and Damians et al. (2013). The properties which is considered in Plaxis analysis for these three article are illustrated in Table 6.8, 6.9 and Table 6.10.

Table 6.6 : Maximum accepted soil parameters (Selected backfill 1) used in modeling the MSE wall in this project.

Parameter	Selected Backfill
γ saturated(KN/m ²)	20
γ Unsaturated(KN/m ²)	20
ϕ' Internal friction angle	40
c' Cohesion	10
E Modulus of elasticity	40000
Ψ Dilatancy angle	10
ν Poissons's ratio	0.2
R_{inter} Interfaces strength reduction factor	0.8

Table 6.7 : Minimum accepted soil parameters (Selected backfill 2) used in modeling the MSE wall in this project.

Parameter	Selected Backfill
γ saturated(KN/m ²)	20
γ Unsaturated(KN/m ²)	19
ϕ' Internal friction angle	38
c' Cohesion()	1
E Modulus of elasticity()	40000
Ψ Dilatancy angle	8
ν Poissons's ratio	0.2
R_{inter} Interfaces strength reduction factor	0.8

Table 6.8 : Soil Parameters used in the model (Hossain et al, 2012)

Parameter	Selected Backfill
γ Saturated(KN/m ²)	22
γ Unsaturated(KN/m ²)	18.8
ϕ' Internal friction angle	34
c' Cohesion	1
E Modulus of elasticity	12500
Ψ Dilatancy angle	4
ν Poissons's ratio	0.32

Table 6.9 : Soil Parameters Used in the Model (Damians et al, 2013)

Parameter	Selected Backfill
γ Saturated(KN/m ²)	20
γ Unsaturated(KN/m ²)	19
ϕ' Internal friction angle	36
c' Cohesion	5
E Modulus of elasticity	30000
Ψ Dilatancy angle	6
ν Poissons's ratio	0.2

6.8.3 Loads and vibrations

When the railway structures are subjected to cyclic loads, it generates vibrations that are transmitted into the supporting ballast and any underlying structures like MSE wall that is supporting the train and rail line.

Table 6.10 : Soil parameters used in the model (Hou, 2009)

Parameter	Selected Backfill
γ Saturated(KN/m ²)	20
γ Unsaturated(KN/m ²)	18
ϕ' Internal friction angle	36
c' Cohesion	1
E Modulus of elasticity(KN/m ²)	16000
Ψ Dilatancy angle	6
ν Poissons's ratio	0.2
R_{inter} Interfaces strength reduction factor	0.67
K_x Permeability in horizontal direction(m/day)	1
K_y Permeability in vertical direction(m/day)	1

Initial investigations in measurement of vibration carried out by SNCF in France , they showed that, the train vibrations diminish rapidly with distance from the rail due to damping of energy by the ballast and earth under and adjacent to the ballast. By considering the MSE structures that are composed of select granular material, there is significant damping inherent in their design. Therefore, MSE walls are appropriate structures for use as retaining walls adjacent to supporting railways (Sankey and Hutchinson, 2011).

The next research is done in order to measure the effect of vibration in accelerations, earth pressures, deformations and tensions in the steel-reinforced earth mass. This full scale MSE wall structure test is done by Terre Armée Internationale, in 1987, vibrating rollers moving over the top of the full-size MSE walls to simulate heavily loaded rail trains, the walls were instrumented and the result of the measurement of changes shows that by the applying vertical vibrations, the coefficient friction stays unaltered Therefore, design of steel-reinforced MSE walls for static loads considered

due to weight of select backfill and rail loading is adequate to also accommodate the effects of rail vibrations (Sankey and Hutchinson, 2011).

The sensitivity of speeds of railway structures lines is much higher than those of normally roads and railways. The risk of any change in structure that might be caused by the dynamic effects of vibrations must also be guaranteed. Particularly, for structures under high-speed lines, the vibration frequencies can get or exceed the estimated resonance frequencies of the structures.

The full-scale vibration tests were carried out in 1975 on the experimental structure at Triel-sur-Seine (CEBTP, 1975), and Four years after this experiment the next test is done at a structures in Fremersdorf (Floss and Thamm, 1979).

The cyclic load of these tests are applied by vibration of compacting rollers, the frequencies applied were in the end comparatively high, 41 Hz at Triel-sur-Seine and 24.5 Hz at Fremersdorf, compared with a frequency of 32.4 Hz for a High Speed Train (HST). The results show that very little facing deformation and tensile increments in line with the static load is applied.

For various train and variable axel span, approximately 40 KPa measured under ballast for trains (Soyez, 2009). The displacements and accelerations measured in the two cases, at the facing deformations are very low (elastic displacements of less than 0.2 mm) and the over tensions in the reinforcing steel strips are in line with the effect of the amplitude of the load applied statically. The effects of vibration diffuse and dissipate quickly with depth in fill from the point of applying load from top of the wall. Thus, in a reinforced soil structure, the upper reinforcing strips are most likely to be disturbed by vibration (Soyez, 2009).

Mr Laurent Soyez in 2009 started the full scale test, at the Centred'Expérimentation Routière. Full instrumentation was installed on the 3.74 m high, 8 m wide structure (Figure 6.7), including deformation gauges , vertical and horizontal stress transducers and accelerometers in the selected back fill and slope fill is installed further ,the displacement transducers on the facing are measured.

Mr Soyez applied large number of cycles at the various frequencies to simulate the speed of train, he applied 2 million cycles at 16 Hz to simulate 170 Km/h train and also 5 million cycles at 28 Hz which is equal to 300 km/h trains, acting like 5 years of traffic at that one frequency alone. The outcome of this experiment is very

desirable, very little displacement in the facing panels approximately less than 0.3 mm was obtained.

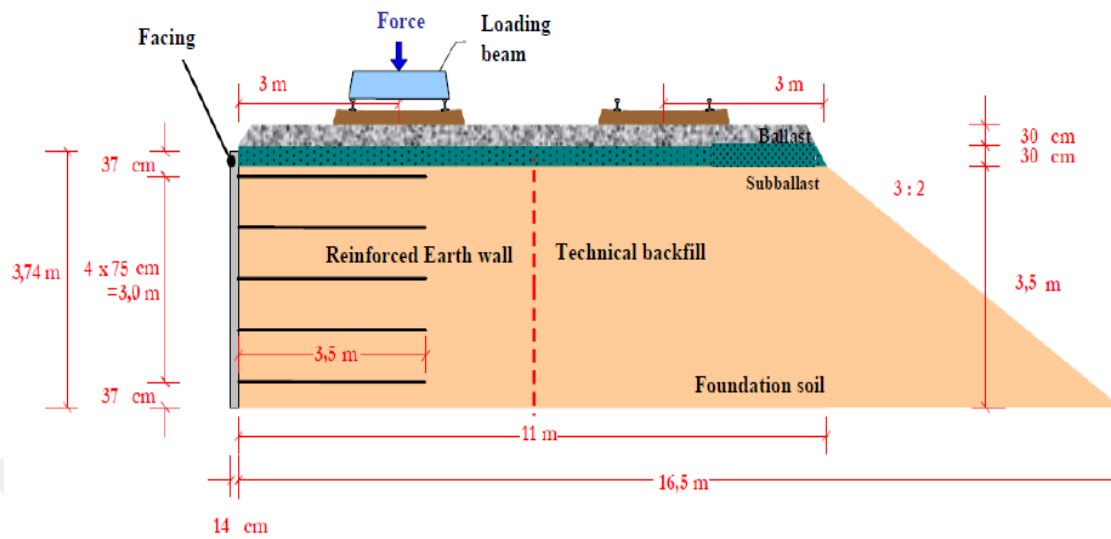


Figure 6.7 : Cross-section of the instrumented structure at CER in Rouen (Soyez, 2009).

The tension in the reinforcing strips always decreases slightly over the phase of the cycles, which could cause a small relaxation of the internal stresses, which is generated during the construction of the structure. It has been observed that there is no continuous failure mechanism, which has caused to disturb the function of the wall components and no sign that such a mechanism could commence in five years' time span (Soyez, 2009).

Wu and Brockbank, determined three considerations for the adjust design of heavily loaded structures (Wu and Brockbank, 2008):

- Facing compatible with the deformations of the fill – compressibility, and ability to comply with differential settlements.
- Low extensibility reinforcing strips to limit horizontal deformations. The reinforcement are high adherence galvanized steel strips are used in this project. Sufficient anchoring of the reinforcing strips in the fill, to prevent slippage of the reinforcing strips relative to the fill material.

In this project all of the Wu and Brockbank conditions are take into deliberation. The appropriate EPDM (Figure 6.8) pads and Special panels (dilatation panels) are used along the wall to ability to comply with differential settlements.

The high adherence galvanized steel strips (ribbed strips) reinforcement are used are in order to increases the friction between soil and strips and avoid slippage.

6.8.4 Appurtenant materials of construction (EPDM pads)

The MSE Walls system with concrete precast facing panels require bearing pads in their horizontal joints ,in order to provide some compressibility and movement between panels during elastic compression and settlement of the reinforced fill. In different settlement and movements it is preclude the concrete-to-concrete contact and it prevents the cracking between them. These materials are generally EPDM rubber or HDPE.

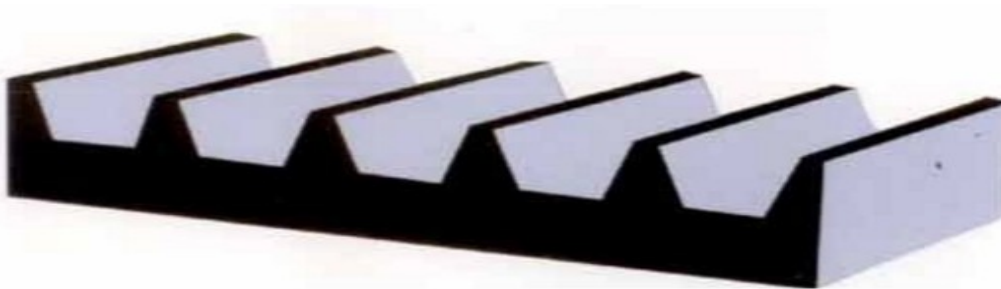


Figure 6.8 : Typical EPDM pad is used in MSE walls.

The type and number of EPDM joints are depends on the height of the wall .The compressibility and thickness of the horizontal joint material is a function of the wall height. Walls with heights of 15 m are requiring thicker or more compressible joints between them. Actually by increment of the height of the wall lateral pressure is increases, thus the thickness of the panels are changes from 14 cm to 18 cm. These joints must be accommodating larger vertical loads due to rise in weight of panels, especially in the lower third of the structure (FHWA, 2009).

6.8.5 Adhesion failure

In 1983 ,at Milville a test has been done in order to evaluate the friction coefficient by acting the reinforcement pull-outs from an instrumented structure subjected to vibrations plate which apply the 92 KN, by 25 to 30 Hz with vibrating plate (TAI 1987) .This experiment indicated that the reduction in pull-out strength takes place simultaneously with the reduction in immediate vertical stress; in conclusion that there was no loss of adhesion that is leads to failure is observed (Sankey and Hutchinson, 2011).

6.8.6 Governing load

In this project the exact train type is not clear but the supporting structure should be designed by considering high-speed train, technical consulting of the project, send the uniform distributed load for 22.5 kN/m^2 for each line (Figure 6.9), by considering the possibility of the two engine in one wagon the designer decided to multiply the uniform loads by two. The purpose of this thesis is to study the deformation of the structure in the highest section of the project's wall by considering the heaviest train. To approach this aim, The Cooper E90 diesel is chosen, this train receives its name from the maximum axle load that applies to the rail of 40 Ton. The axle loads for a Cooper E80 are illustrated in the Figure 6.10 (AREMA Manual, 2007).

In the area that is coverage four axles that is applying this load to the rail, the resulting surcharge at the level of under of rail ties is approximately 100 Kpa for Cooper E90. This is the uniform surcharge load that is specified by the rail companies for the design of a Reinforced Earth wall under heavy rail lines.(Brockbank et al, 2011).

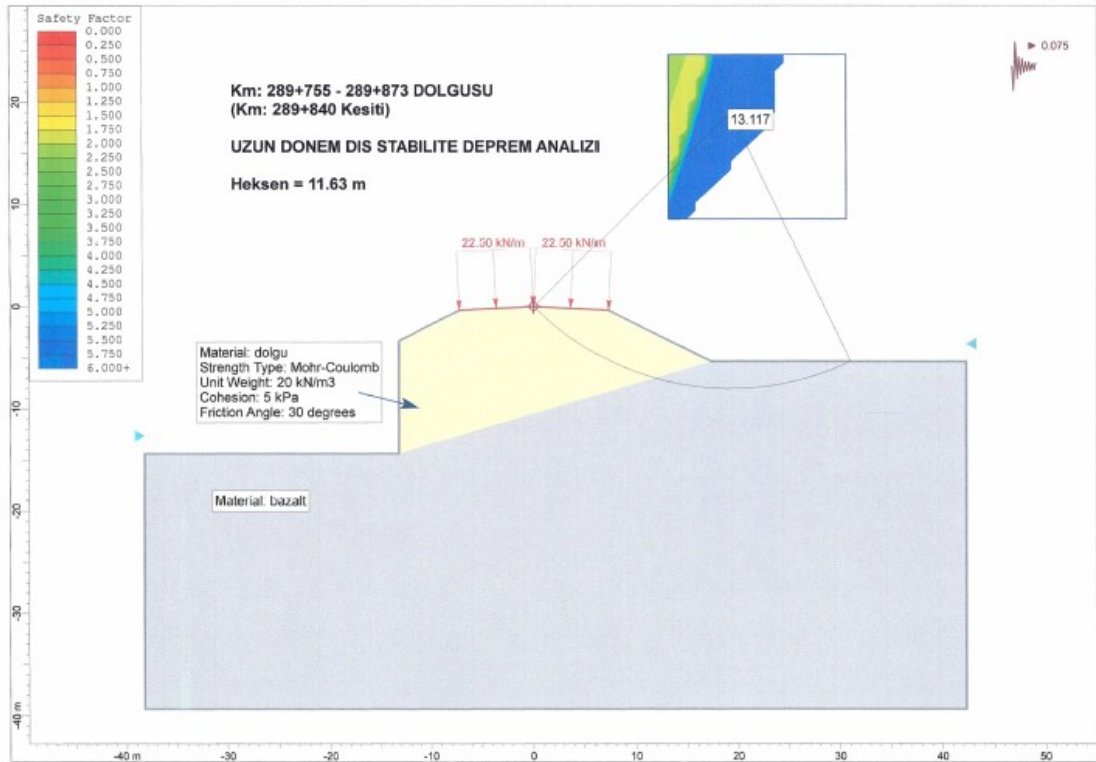


Figure 6.9 : Global stability of wall by considering 22.5 kN for each line.

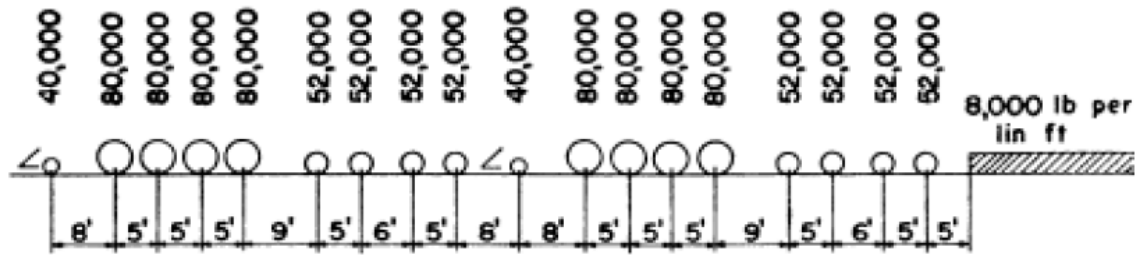


Figure 6.10 : Cooper E 80 axle load diagram (AREMA Manual, 2007)

6.9 Valdez Calculation of MSE Walls

6.9.1 Case One: Design of the wall with rectangular cross section

In design of the MSE wall, the Valdez program will be used, as mentioned in section 8, the Valdez software is based on limit stated method. The Valdez program checks the rupture of strips and adherence to prevent from pullout and checks the sliding at the base of the structure for static and seismic loads. In seismic areas, the check procedure is done by considering maximum horizontal acceleration of the region.

The parameters that the Valdez program requirements are generally based on cohesion and friction angle, both in selected fill material and foundation. During the soil investigation no water tables were reported. In other side the fill material in MSE wall is suitable for drainage, so in design procedure the water pressure is not taken into account.

The minimum strip length is considered between 0.5-0.7 of total height of wall. By increasing the pressure in bottom of the wall for the economic reasons two types of strips, in top levels 40 x 4 mm high adherence strips and in bottom 50 x 4 mm high adherence strips will be used.

In the first case the strips length is equal to 0.55 of the total height of the wall, that is approximately equal to 8,5 m. In this part the load type is rail way load and without considering extra safety factors, which previously mentioned in load governing section is accepted. Under the rail way railway's sleeper there will be 1.5 m Basalt fill with the slope angle of 33 degree is taken into account. The maximum horizontal acceleration is 0.2 g that is mentioned in geotechnical report. The screen shot of the wall geometry that is define to Valdez program is illustrated in Figure 6.11.

Safety factor for internal and external stability of the rectangular cross section MSE wall is illustrated at Table 6.11.

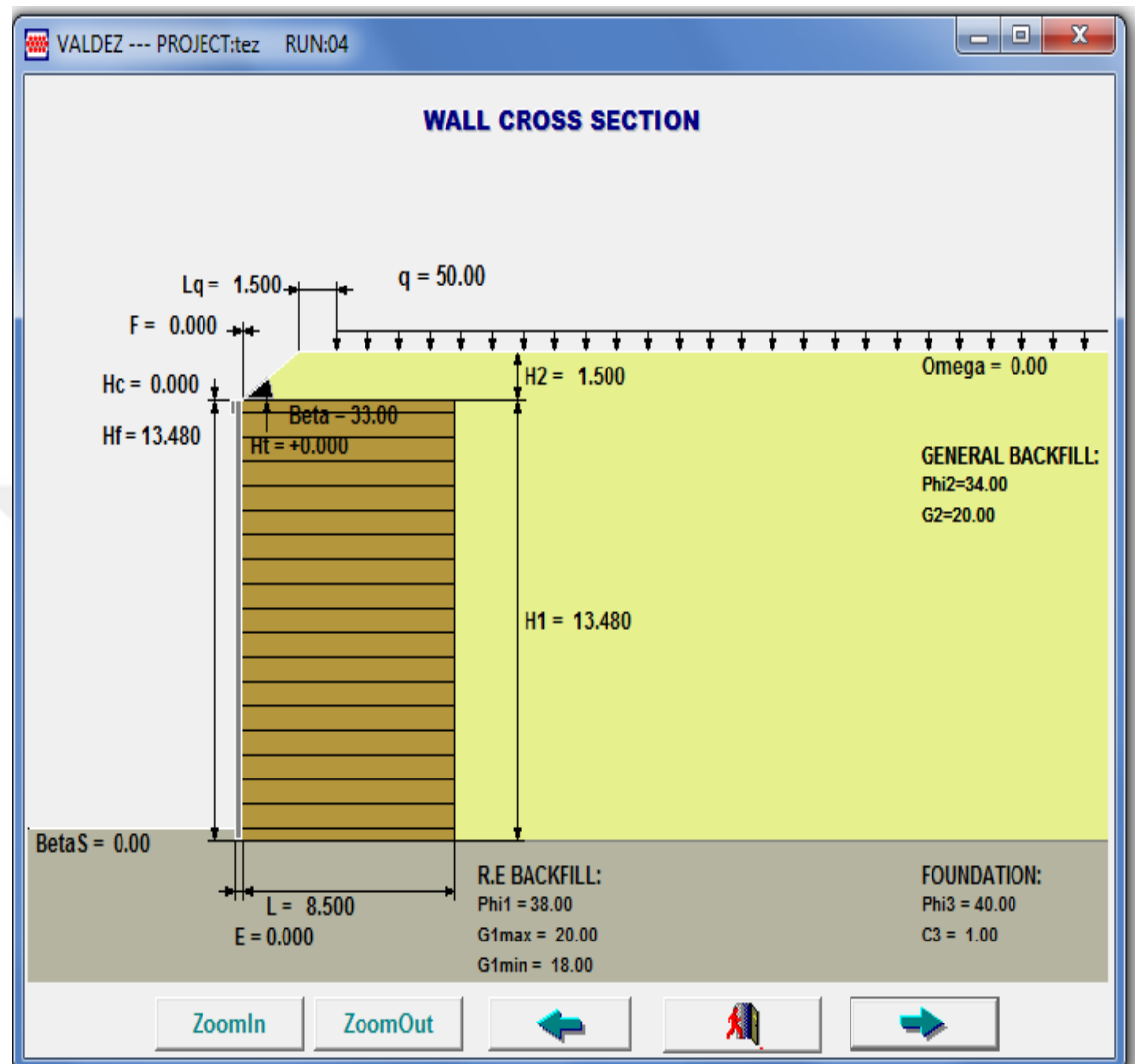


Figure 6.11 : Screen shot of the wall with rectangular cross section from Valdez software

6.9.2 Case Two : Design of the wall with trapezoidal cross section

Trapezoidal cross section type of reinforcement geometry is used because, the increment is happen in pull out capacity of the reinforcement from top to bottom of the wall by increasing the height of the soil mass from the top of the wall. The length of strips which is needed at the bottom is lower than from the top of the wall. The point should be considered, this type of geometry is recommended, when the foundation of the MSE structure is based on rock or competent foundation material, otherwise when the foundation or the soil under the strips is in cuts, it is exhibit minimal post construction settlements the whole function of strips is disturbed.

Table 6.11 : safety factor for internal and external stability of the rectangular MSE wall

Depth of Reinforcement from top of the wall	Number of joints (strips)	Length of strips	$\frac{Tr}{Tm}$ (50KPa)	$\frac{Tf}{Tm}$ (50KPa)	$\frac{Tr}{Tm}$ (100KPa)	$\frac{Tf}{Tm}$ (100KPa)
0.366	4	8.5 m-type (I)	3,61	2,37	2,59	2,37
1.115	4	8.5 m-type (I)	2,87	1,89	2,20	1,89
1.865	4	8.5 m-type (I)	2,45	1,86	1,96	1,86
2.615	4	8.5 m-type (I)	2,17	1,79	1,79	1,79
3.365	4	8.5 m-type (I)	1,98	1,67	1,67	1,67
4.115	4	8.5 m-type (I)	1,84	1,52	1,58	1,52
4.865	4	8.5 m-type (I)	1,68	1,45	1,46	1,45
5.615	4	8.5 m-type (I)	1,49	1,48	1,30	1,47
6.365	5	8.5 m-type (I)	1,67	1,82	1,47	1,80
7.115	5	8.5 m-type (I)	1,51	1,86	1,33	1,76
7.865	5	8.5 m-type (II)	1,71	2,31	1,51	2,20
8.615	5	8.5 m-type (II)	1,55	2,42	1,38	2,23
9.365	5	8.5 m-type (II)	1,42	2,53	1,26	2,24
10.116	6	8.5 m-type (II)	1,56	3,05	1,38	2,69
10.866	7	8.5 m-type (II)	1,67	3,56	1,48	3,11
11.616	7	8.5 m-type (II)	1,53	3,66	1,36	3,05
12.366	8	8.5 m-type (II)	1,61	4,17	1,42	3,40
13.116	8	8.5 m-type (II)	1,51	4,18	1,32	3,33

Where:

Tr : Allowable tensile loads of strips

Tm : Maximum tensile force

Tf : Strips pullout capacity

In the Valdez out put all of the safety factored is greater than 1 and the limit state conditions are reached.

The FHWA recommended using of Trapezoidal cross section type of reinforcement geometry when the competent foundation materials include materials with SPT N60 value greater than 50 and sound rock (FHWA, 2009)

In design of this type of wall with Trapezoidal cross section (Figure 6.12) requires special analysis after using simplified design rules for determining internal and external stability analyses.

After checking the internal and external stability the next control will be checking the safety for slope stability analysis. The most common programs that use reinforced soil stability and checking for both global and compound failure planes are Talren and slide software.

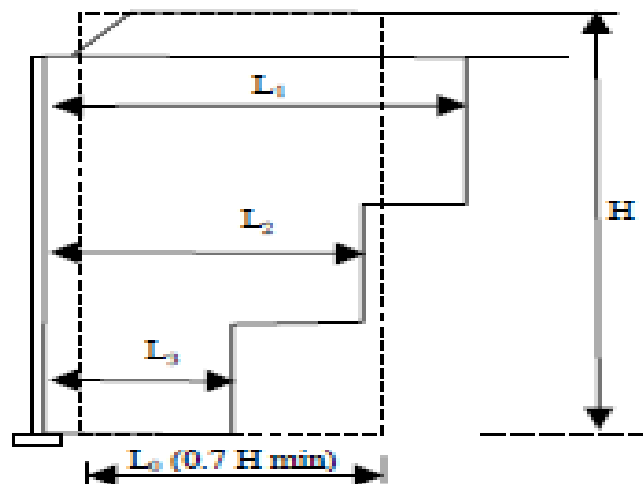


Figure 6.12 : Trapezoidal cross section type of reinforcement geometry (FHWA, 2009).

There are not any special criteria and limitation after controlling the mentioned stability analysis, but the FHWA has recommendation in length of the strips that is mentioned below:

Simplified design rules for these structures are as follows:

In Figure 6.12 the whole soil mass is illustrated by a rectangular block (L_0, H) having the same total height and the same cross-sectional area as the trapezoidal section for external Stability calculations. The maximum tensile force line is the

WALL CROSS SECTION

$L_q = 1.500$
 $q = 50.00$
 $F = 0.000$
 $H_c = 0.000$
 $H_f = 13.480$

$\beta = 90.00$
 $H_t = +0.000$
 $L = 9.00$
 $L = 8.00$
 $L = 7.50$
 $L = 7.00$

$H_1 = 13.480$
 $H_2 = 0.100$

$\Omega = 0.00$
GENERAL BACKFILL:
 $\Phi_2 = 34.00$
 $G_2 = 20.00$

$\beta_s = 0.00$
 $L = 9.000$
 $E = 0.000$

R.E BACKFILL:
 $\Phi_1 = 38.00$
 $G_{1max} = 20.00$
 $G_{1min} = 18.00$

FOUNDATION:
 $\Phi_3 = 40.00$
 $C_3 = 1.00$

ZoomIn ZoomOut

The safety factor for internal and external stability of the wall is demonstrated in Table 6.12.

In design of this type of MSE wall ,without symmetric geometry, in lower part of the wall the distance beyond the wall is not enough to extend the strips. Therefore, in the parts where the ground elevation codes are close to facing part, the a huge amount of excavations will be needed to provide adequate distance to extend the strips,Therefore an other system is required to avoid excessive costs.

Table 6.12 : Safety factor for internal and external stability of the Trapezoidal MSE wall.

Depth of Reinforcement from top of the wall	Number of joints (strips)	Length of strips	$\frac{Tr}{Tm}$ (50KPa)	$\frac{Tf}{Tm}$ (50KPa)	$\frac{Tr}{Tm}$ (100KPa)	$\frac{Tf}{Tm}$ (100KPa)
0.366	4	9 m-type (I)	3,59	2,21	2,49	2,25
1.115	4	9 m-type (I)	2,79	1,69	2,14	1,84
1.865	4	9 m-type (I)	2,34	1,92	1,91	1,83
2.615	4	9 m-type (I)	2,05	2,00	1,75	1,79
3.365	4	9 m-type (I)	1,85	2,00	1,64	1,71
4.115	4	9 m-type (I)	1,71	1,94	1,56	1,59
4.865	4	9 m-type (I)	1,61	1,83	1,45	1,54
5.615	4	9 m-type (I)	1,54	1,66	1,30	1,58
6.365	5	8 m-type (I)	1,77	1,67	1,46	1,66
7.115	5	8 m-type (I)	1,59	1,72	1,31	1,64
7.865	5	8 m-type (II)	1,80	2,11	1,48	2,05
8.615	5	8 m-type (II)	1,63	2,22	1,34	2,09
9.365	6	7,5 m-type (II)	1,79	2,52	1,46	2,32
10.116	6	7,5 m-type (II)	1,63	2,65	1,33	2,32
10.866	7	7,5 m-type (II)	1,74	3,14	1,40	2,68
11.616	7	7 m-type (II)	1,59	3,03	1,27	2,41
12.366	8	7 m-type (II)	1,66	3,45	1,31	2,67
13.116	8	7 m-type (II)	1,54	3,46	1,20	2,57

Where:

Tr : Allowable tensile loads of strips

Tm : Maximum tensile force

Tf : Strips pullout capacity

In this project two types of soil are exists, the lower part of wall, which is coinciding with three panels from bottom of the wall, is stiff Basalt type1, the excavation in this

part of the site is done by explosion. The second part is that coincide with other strips is Basalt type two ,by considering the danger and economical consideration of excavation ,the shored Reinforced Earth[®] wall system which is called TerraLink[™] is recommended. TerraLink[™] is patented product of Reinforced Earth Company.

TerraLink[™] elements are similar to those used for Reinforced Earth[®] solution. These elements are placed between well compacted backfill layers. The main feature is that reinforcements are linked to the back face nailing part, the existing structure or stiff blocks which enables combination or continuity of reinforcement between the narrow Reinforced Earth[®] wall and existing ground or structure.

Choice of appropriate link system which mainly depends on geometrical site configuration and the height of the wall is TerraLink[™]. For this purpose two types of linked system in TerraLink[™] are possible:

- The first choice is friction link system, according to this system the, the Ladder which is used between the reinforcements is ensures thst the system get its frictional connection, which is not enough space to extend the strips, the other important advantage of this system is the flexibility of system is kept .the frictional link system is feasible, when there is sufficient space to overlap the reinforcement.
- The second choice is direct connection, which is not usually preferred in this type of structure, to keep the flexibility of system.

In this part two type of connection is considered in order to comparison the deformations of two system.

The Valdez program does not help with the nail calculation; therefore, by considering the Valdez results we can find the instable joints and calculate the nail and friction joints.

6.10.1 Step one: MSE wall design

The design procedure for MSE wall is the same with rectangular cross section type.The length of strips in each level is given in Figure 6.14.

The safety factor for internal and external stability of the wall is listed at Table 6.13.

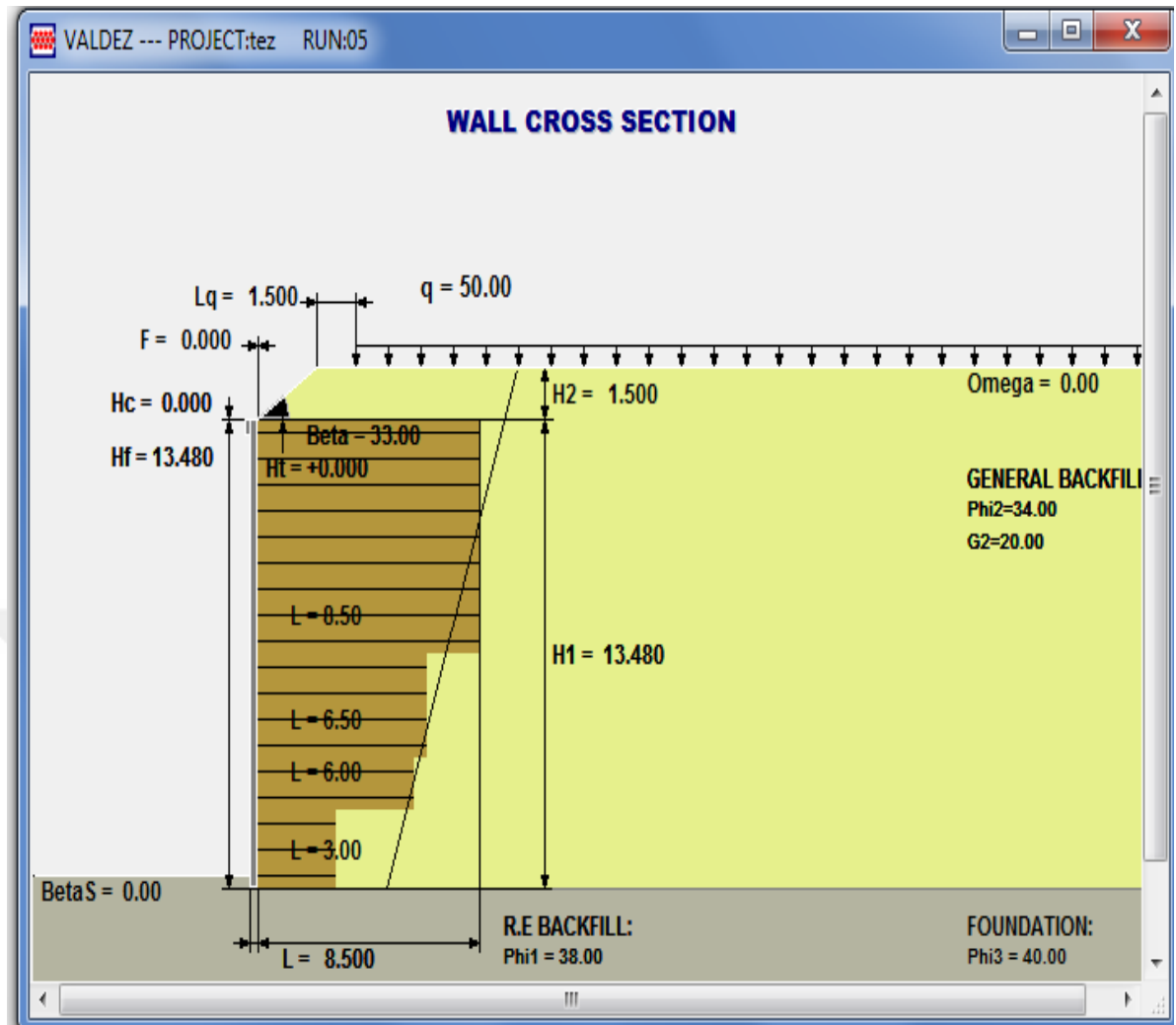


Figure 6.14 : Screen shot of the wall with Terralink TM cross section from Valdez software.

The first problem which leads to collapse of the wall, because, the maximum tensile forces are more than allowable tensile loads of strips in the last three layers, therefore the strips will rupture.

As mentioned in Table 6.13, in the last three layers the safety factors are less than 1, in this situation two possibilities are faced;

The second issue which the wall tackles with it is the length of the strips which is used in the lower layers of the wall is about 0.22 of the total height of wall and they are too short to provide adequate friction between soil and strips, thus the panels come out and strips do not hook them to soil. In this situation the special frictional ladders are used and connected to nails behind the wall.

Table 6.13 : safety factor for internal and external stability of TerraLink™ MSE wall.

Depth of Reinforcement form top of the wall	Number of joints (strips)	Length of strips	$\frac{Tr}{Tm}$ (50KPa	$\frac{Tf}{Tm}$ (50KPa)	$\frac{Tr}{Tm}$ (100KPa	$\frac{Tf}{Tm}$ (100KPa)
0.366	4	8,5 m-type (I)	3,61	2,30	2,58	2,30
1.115	4	8,5 m-type (I)	2,87	1,87	2,25	1,87
1.865	4	8,5 m-type (I)	2,45	1,84	2,00	1,84
2.615	4	8,5 m-type (I)	2,17	1,76	1,83	1,76
3.365	4	8,5 m-type (I)	1,98	1,62	1,70	1,62
4.115	4	8,5 m-type (I)	1,84	1,44	1,61	1,44
4.865	4	8,5 m-type (I)	1,68	1,34	1,49	1,34
5.615	4	8,5 m-type (I)	1,50	1,38	1,33	1,38
6.365	5	8,5 m-type (I)	1,68	1,69	1,50	1,69
7.115	6	6,5 m-type (II)	2,24	1,65	2,21	1,65
7.865	7	6,5 m-type (II)	2,32	1,95	2,26	1,94
8.615	7	6,5 m-type (II)	2,07	2,08	1,96	1,97
9.365	7	6,5 m-type (II)	1,85	2,19	1,71	1,97
10.116	7	6 m-type (II)	1,65	2,05	1,48	1,73
10.866	7	6 m-type (II)	1,48	2,03	1,29	1,67
11.616	8	3 m-type (II)	1,38	0,83	1,17	0,67
12.366	8	3 m-type (II)	1,00	0,68	0,80	0,54
13.116	8	3 m-type (II)	0,59	0,44	0,42	0,32

Tr: Allowable tensile loads of strips

Tm: Maximum tensile force

Tf: Strips pullout capacity

6.10.2 Step Two: MSE nail design

The soil nail is modeled as geogrid in Plaxis, geogrid properties for Plaxis input is illustrated in Table 6.14.

By modeling the soil nail in Plaxis we can get the maximum axial forces of nails furthermore the deformation of shotcrete and nail after the frictional joints are installed and therefore the maximum loads that are applied should be considered. Total displacement of soil nail wall without connection to wall is too small as shown in Figure 6.15.

Table 6.14 : Calculation of soil-nail properties as geogrid.

Soil Nail		
Active zone		
Properties	For 1 nail	φ 32
A_{nail}	0,000804248	m ²
E_{nail}	200.000.000	kN/m ²
EA_{nail}	160.849,54	kN
Açimento	0,00869907	m ²
$E_{\text{çimento}}$	10.000.000	kN/m ²
$EA_{\text{çimento}}$	86.990,70	kN
EA_{total}	247.840,24	kN
Vertical Nail Spacing	1,50	m
$EA_{\text{input value}}$	165.226,83	kN/m

6.10.3 Soil nail spacing

Soil nails are installed and designed in a grid form. In design procedure the frictional strips (Ladders) which is connected to nail is located equally in spacing, SH, that is often the same as the vertical nail spacing, SV by 1.5 m.

6.10.4 Soil nail inclination

Soil nails are generally installed at 10 to 20 degrees from the horizontal. Nail inclination angles less than 10 degrees should be avoided to prevent making voids, Voids can reduce the pullout resistance and decrease corrosion protection.

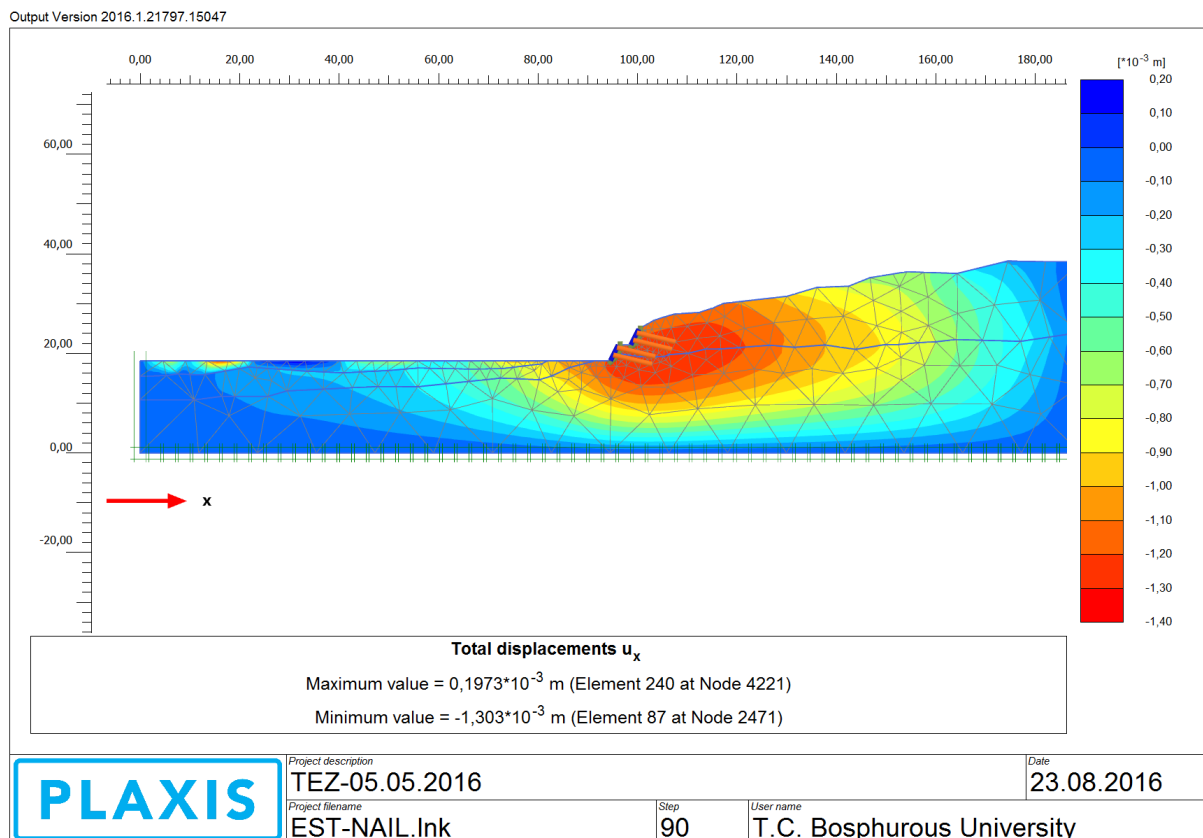


Figure 6.15 : Total displacement of Soil nail wall by Plaxis.

The most commonly inclination is 15 degrees, thus in this research , 15 degrees inclination is considered.

6.10.5 Soil nail length

To develop a preliminary cross section for analysis, the soil nail length can be estimated to be approximately $0.7 H$, where H is the wall height. In this case the height is variable from 14m to 12 m and $0.7 H$ is varies from 8.5 to 7 meters, by attending the rock properties of nail location 6 m length for nails is taking into account.

6.10.6 Bond strength

The bond strength (qu) is the (ultimate) shear strength of the soil-to-grout interface. This parameter is equivalent to a nominal bond resistance, as used in LRFD practice. In this manual, the term bond strength will be used for simplicity, regardless of the design platform being used (Table 6.15).

Table 6.15 : Estimated Bond strength for Nails in rock-Drilling method, Rotary Drilled (Elias and Juran, 1991).

Rock Type	Bond Strength, q_u (psi)
Marl/limestone	44 - 58
Phyllite	15 - 44
Chalk	73 - 87
Soft dolomite	58 - 87
Fissured dolomite	87 - 145
Weathered sandstone	29 - 44
Weathered shale	15 - 22
Weathered schist	15 - 25
Basalt	73 - 87
Slate/Hard shale	44 - 58

Bond strength for Basalt is between range 73-87 (Psi). The 73 (Psi) is equal to 503-600 KPa. The minimum value by applying 3 safety factor is accepted equal to 166 KN.

6.10.7 Pullout resistance per unit length

The ultimate pullout resistance per unit length of soil nail (r_{PO}) is equal to the bond strength multiplied by the perimeter of the bond zone.

$$r_{PO} = \pi q_u D_{DH} \quad (6.1)$$

Where:

q_u : Bond strength of the nail-grout-soil interface (force/unit area)

D_{DH} : Diameter of the drill hole

$$r_{PO} = \pi * 166 * 0.125 * 6 = 391.12 \text{ KN} = 39 \text{ Ton}$$

$$F_s = \frac{39}{10} = 3.9$$

6.10.8 Tensile resistance of the tendon

The nominal tensile resistance of the tendon is:

$$R_T = A_t * f_Y \quad (6.2)$$

Where:

A_t : Cross-sectional area of tendon

f_Y : Nominal yield resistance of tendon (force per area)

$$R_T = \left(\frac{\pi * R^2}{4}\right) * \left(\frac{f_Y}{1.15}\right) \quad (6.3)$$

$$R_T = \left(\frac{\pi * 0.032^2}{4}\right) * (42000/(1.15)) = 293.7 \text{ KN} = 29.37 \text{ Ton}$$

Maximum value of axial forces in nail part of TerraLink™ with 100 KN uniform load is obtained from Plaxis with connection is equal to 100 KN/m, therefore;

$$F_S = 1.15 * (29.37/10) = 3.37$$

6.11 Finite Element Analysis of MSE/ Hybrid Wall

The finite element mesh and displacement boundary conditions are used in the analysis for approximately 14m high wall with three type of geometry which is depend on strips lengths, are demonstrated in Figures 6.16, 6.17 and 6.18. The Reinforced earth wall analysis satisfies the 2-D plain strain condition, as the wall is extending along the elevation. The maximum wall height is taking into account for consider the maximum deformation in facing elements.

The type of strips and the length are considered by the results of Valdez software, Furthermore number of joints in each layer and the distances between joints are obtained by mentioned program.

6.11.1 Facing panel and shotcrete elements

The facing panels of Reinforced wall are two types, reinforced and unreinforced panels, for the thickness of 14 cm and 18 cm, which both of them are grade 30 precast concrete. Usually by considering lateral pressure on the facing panels, two or three panels from top elevation of wall are unreinforced .In bottom of the wall after 10 joints in facing panels according to recommendation of Reinforced Earth Company the 18cm panels are recommended. In all these analyses, the panels are simulated as beam elements. The beam elements are modeled as linear elastic material.

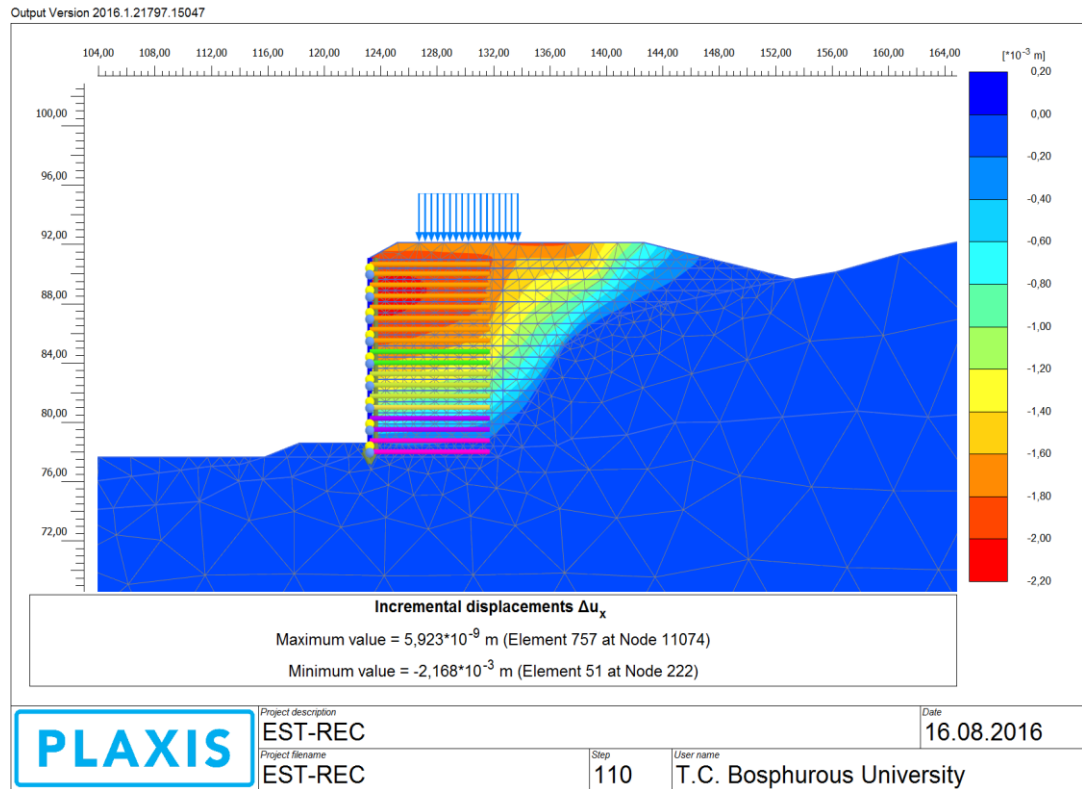


Figure 6.16 : Incremental displacement of wall with rectangular cross section.

The actual thickness of the wall is 14 cm for all panels, the elastic properties of facing elements are shown in Table 6.16, each panel is connected to another panel by EPDM pads. Therefore, the joints between the panels were considered as connection and the connections are set as free.

The other element which is used in the third geometry is shotcrete. The thickness is considered as 20 cm and the elastic properties of shotcrete elements are shown in Table 6.17.

6.11.2 Steel strips

Two type of High adherence galvanized steel reinforcing strip have been used in this case study, this technology has been used over thirty years without change. This gives us reliability and enables us to confirm, the allowance for corrosion of the steel

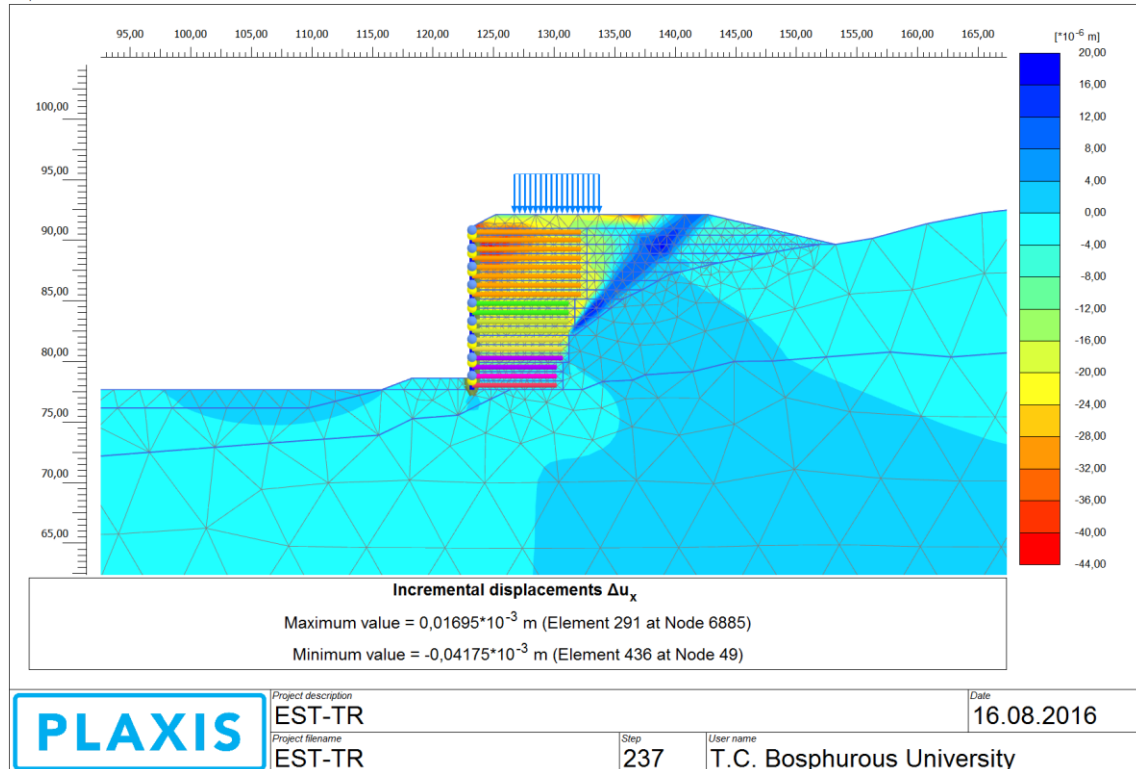


Figure 6.17 : Incremental displacement of wall with trapezoidal cross section.

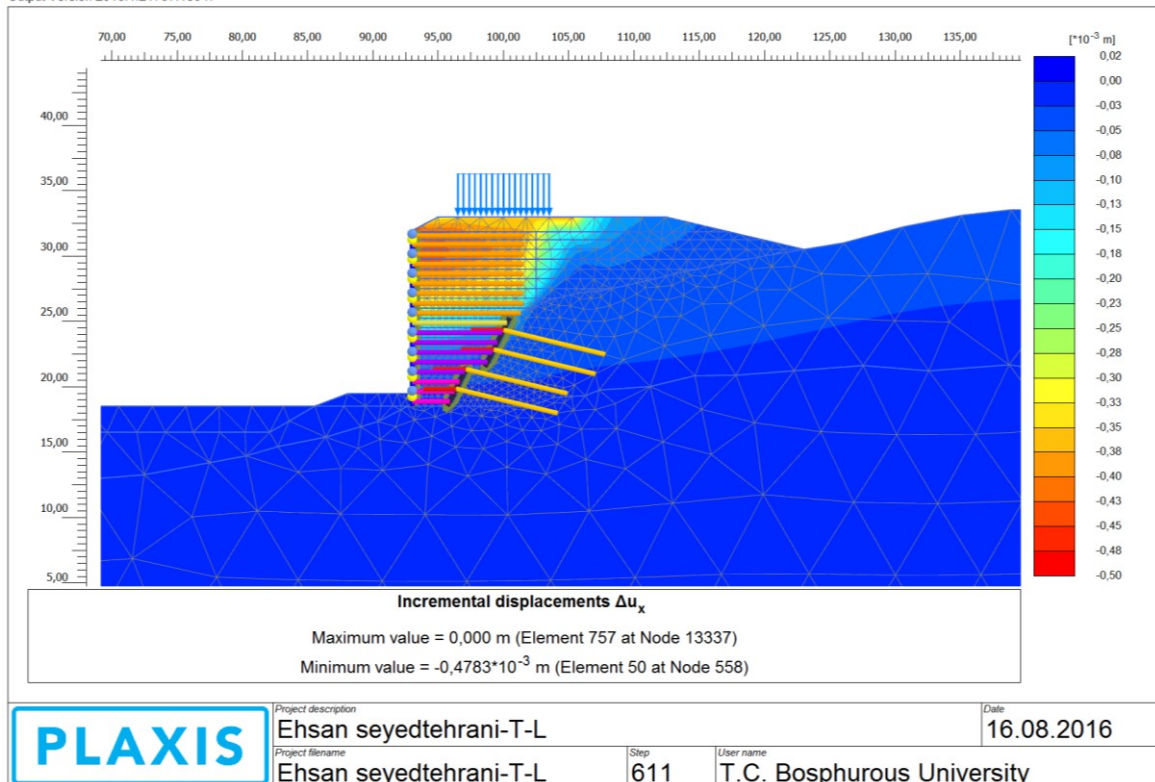


Figure 6.18 : Incremental displacement of wall with Terralink TM cross section.

Table 6.16 : Elastic properties of reinforced concrete facing panels.

Wall Thickness(M)	Material Type	EA (KN/m)	EI (KNm ² /m)	Linear Weight(KN/m/m)	Poisson's Ratio(ν)
0.14	Linear-Elastic	3.0E7	5.0E4	3.5	0.15

Table 6.17 : Elastic properties of shotcrete materials.

Wall Thickness(M)	Material Type	EA (KN/m)	EI (KNm ² /m)	Linear Weight (KN/m/m)	Poisson's Ratio (ν)
0.20	Linear-Elastic	5.600E6	18.70E3	5	0.2

strips in the soil. High adherence strips are tensile elements, which are able to carry tensile forces but not compressive forces.

For modeling the steel strips in MSE wall the geogrid element are chosen, because the geogrid material exhibits normal stiffness but not bending stiffness. Hence the galvanized steel strips in MSE wall are confronted by neglectable bending stiffness; this element is perfect element to model the reinforcing strips. Equivalent layer rigidity (KN/ml) for two type of strips by considering number of joints in each panel are listed in Table 6.17.

The Young's modulus for steel strips is 2.1E+08 MPa. The two type of adherence steel strips 40 x4 and 50 x4 and the special product of Reinforced earth company which is called Ladder type 2 is used in this case study calculation.

In other side geogrid element is a continuous sheet in each defined layer in the plain strain direction, and the forces in the strips are exactly converted into equivalent layer rigidity in a meter. Therefore by considering the number of strips in three meter and length of strips in each layer we can obtain the equivalent layer rigidity for one meter. Thickness tolerance is and sacrificial thickness at long term is taking into account in the equivalent layer rigidity. Equivalent layer rigidity (KN/ml) for various type and number of strips are demonstrated in Table 6.18.

Table 6.18 : Equivalent layer rigidity (KN/ml) for various type and number of strips

Number of strips for three meters	Strip Type	Name in Plaxis	Equivalent Layer Rigidity (KN/ml)
4	40x4mm	40x4mm-4 joints	44800
5	40x4mm	40x4mm-5 joints	56000
5	40x5mm	40x5mm-5 joints	70000
6	40x5mm	40x5mm-6 joints	84000
7	40x5mm	40x5mm-7 joints	98000
8	40x5mm	40x5mm-8 joints	112000
6	Friction elements	Ladder	140000

6.11.3 Interface element

The interaction between facing panel and selected backfill is modeled by using interface elements. In order to reduce the friction between the structural element and the soil we can use interface element. In the MSE wall the interface is define between facing panel and selected back fill. The interface value, are defined as termed as R_{inter} , which has a value between 0.01 and 1.0, The 0.01 value means there is practically no friction between the structural element and the soil. The upper bound value of 1.0 means the structural element and the soil is completely rigid and cannot slip one another, therefore the default value 1 is set for the interaction between panel and soil.

In modeling steel strips by using geogrid (tensile) element in plaxis. In practice the when we use granular material with the minimum friction angle 36 degree, pressure of soil around the strips is completely in contact with the surface body. Therefore, failure surface take place not between the steel strips and the soil, but within the soil seams that stick to the ribbed steel strips body and the soil around it. It means the full frictional force of the soil can be developed; therefore, no interface element should be introduced less than 1.

In this model the interaction between soil and steel strips is defined as,
 $R_{inter} = \tan \phi = 0.8$.

6.12 Pseudostatic Slope Analysis

Limit equilibrium analysis has generally been used to compute the stability of slopes under both static and seismic loads. The limit equilibrium calculation is based on analyzing the cross section to obtain possible potential slide mass which is called the failure surface. It is feasible to find a circular or non-circular surface. The confined area between the slope face (outer layer) and the failure surface, is cognized as the failure mass, these are subdivided into a series of slices which are then analyzed for equilibrium by several computational methods.

In the cases which is located in earthquake prone zones, horizontal and vertical pseudo static (seismic) coefficients, k_h and k_v , are used to calculate the horizontal and vertical forces caused by considering potential earthquake, as shown in Figure 6.19. By considering the horizontal and vertical seismic coefficients and by adding the F_v and F_h forces to the overall equilibrium calculation for the individual slices creating the failure surface (Figure 6.20).

According to the Newmark's studies in displacement-type analysis (Newmark, 1965) and field observations illustrate that the Pseudo static method can be useful in evaluating the performance of embankments constructed of soils that do not lose significant strength during earthquakes. Such soils include clays, clayey soils, dry or moist cohesion less soils, and dense cohesion less soils (Seed, 1979).

In pseudo static stability analysis, selection of an accurate seismic coefficient is the most crucial and difficult part of this analysis. In theory, the seismic coefficient values should depend upon the measurement of the amplitude of the inertial force induced in the slope by the dynamic forces generated during an earthquake.

As shown in Table 6.18, there are no specific rules for selection of an appropriate seismic coefficient for design. However, the different selection criteria suggest that the seismic coefficient should be based on the anticipated level of acceleration within the failure mass and should correspond to some fraction of the anticipated peak acceleration (Kramer, 1996).

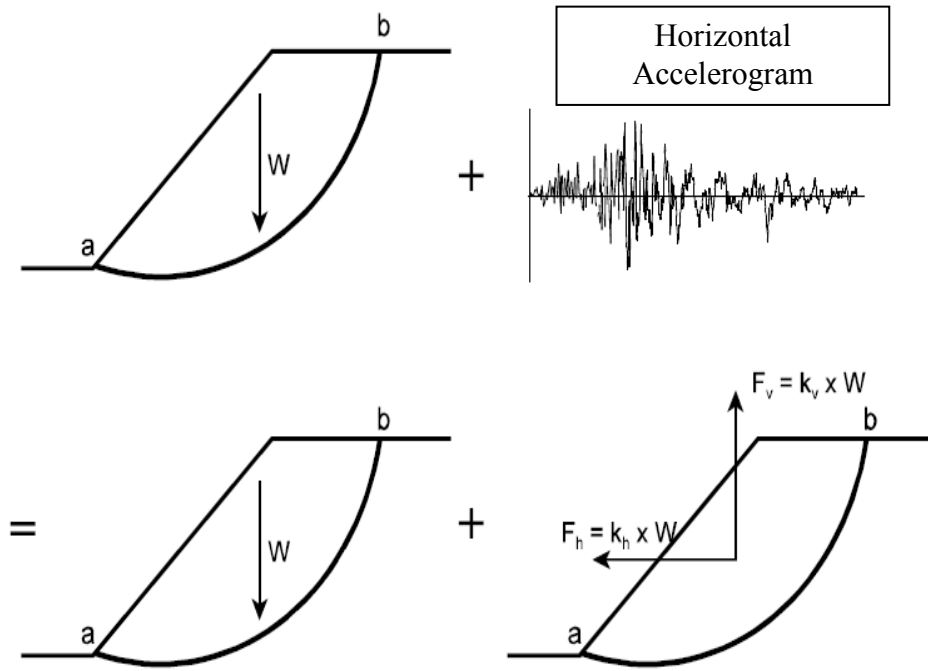


Figure 6.19 : Pseudostatic analysis approaches (Melo and Sharma, 2004).

Table 6.19 : Recommended horizontal seismic coefficients (Melo and Sharma, 2004).

Horizontal Seismic Coefficient, k_h	Description	
0.05 - 0.15	In the United States	
0.12 - 0.25	In Japan	
0.1	“severe” earthquakes	Terzaghi [4]
0.2	“violent, destructive” earthquakes	
0.5	“catastrophic” earthquakes	
0.1 - 0.2	Seed [2], FOS ≥ 1.15	
0.10	Major Earthquake, FOS > 1.0	Corps of Engineers [5]
0.15	Great Earthquake, FOS > 1.0	
$\frac{1}{2}$ to $\frac{1}{3}$ of PHA	Marcuson [6], FOS > 1.0	
$\frac{1}{2}$ of PHA	Hynes-Griffin [7], FOS > 1.0	

FOS = Factor of Safety. PHA = Peak Horizontal Acceleration, in g's.

6.13 Earthquake Potential Evaluation of the Region

The seismic risk map of Sivas -Yozgat is illustrated at Figure 6.20, The region that the wall will be built is located in second third zone, which we expected horizontal acceleration is $a_0/g=0.2$.

This map is prepared by Turkey Seismic Risk Map, General Directorate of Disaster Affairs Earthquake Research Department.

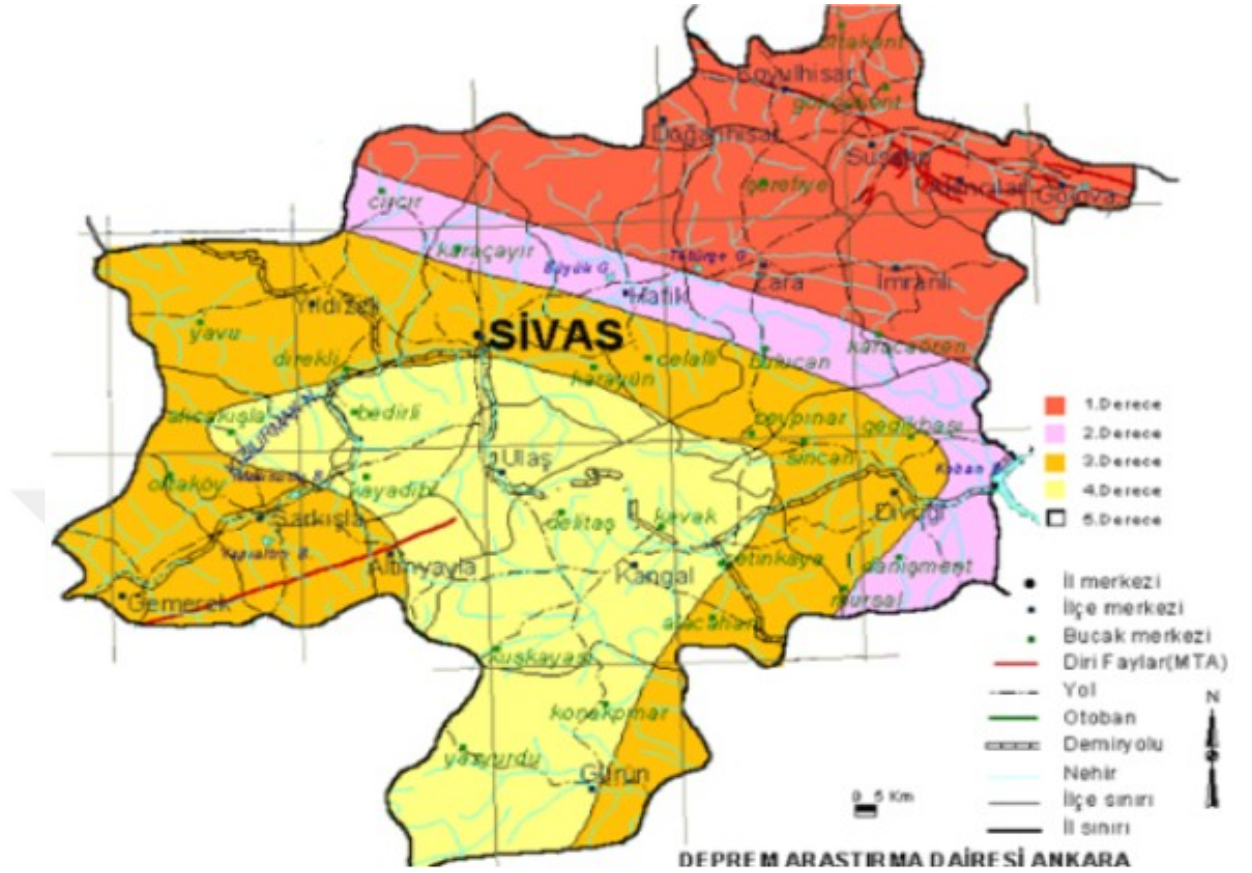


Figure 6.20 : Turkey Seismic Risk Map (2006).

6.14 Soil Properties and Soil Models Method

6.14.1 Linear elastic model (LE)

The Linear Elastic (LE) model is based on the Hooke's law of isotropic elasticity. It is the most uncomplicated model available which only involves two basic parameters, Young's modulus (E) and Poisson ratio (ν). The Linear Elastic model is not appropriate for modeling soil behavior because soil behavior is highly nonlinear and irreversible, but it is appropriate to model stiff volumes in soil like concrete walls and also intact rock (Plaxis Material Models Manual, 2016).

6.14.2 The Hardening Soil Model (HS)

The Hardening Soil model is an advanced model for the simulation of soil behavior. In the Mohr-Coulomb model, the states of stress are limiting by means of the friction angle, cohesion c and the dilatancy angle, But in Harding soil model (Table 6.20),

the soil stiffness is described much more accurately by using three different input stiffnesses.

Table 6.20 : Input parameters of HS model (Plaxis Material Models Manual, 2012)

Symbol	Parameter
ϕ'	Internal friction angle
c'	Cohesion
ψ	Dilatancy angle
E_{50}^{ref}	Reference secant stiffness from drained triaxial test
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading
E_{ur}^{ref}	Reference unloading/reloading stiffness
m	Power for stress-level dependency of stiffness
ν_{ur}	Unloading/reloading Poisson's ratio
K_0	Coefficient of earth pressure at rest = $1 - \sin \phi$
R_f	Failure ratio

In way of comparison the Mohr-Coulomb model and the Hardening Soil is also an account for stress-dependency of stiffness moduli. This means that all stiffnesses increase with pressure (Figure 6.21). Hence, all three input stiffnesses relate to a reference stress, usually taken as 100 KPa (Plaxis Manual, 2016).

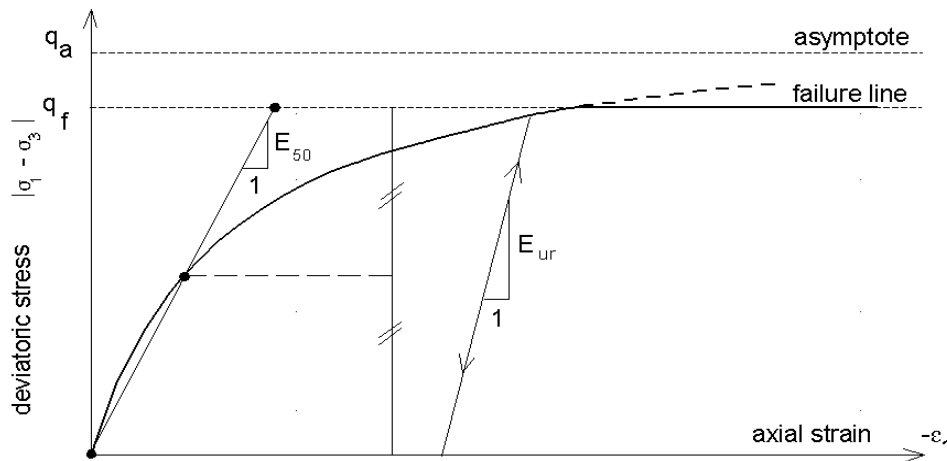


Figure 6.21 : Hyperbolic stress strain response curve of hardening soil model.

6.14.3 Hook Brown method for rock

The Hoek-Brown failure criterion for rock masses is generally admitted and has been used in a large number of projects all over the world. Hoek-Brown failure criterion for rock masses has been found to be acceptable, while there are still exists some

uncertainties and inaccuracies, that have made this criterion convenient to apply into numerical models and limit equilibrium programs. Particularly, there was a problem in finding an acceptable equivalent friction angle and cohesive strength for a given rock mass since the publication of the criterion in 1980.

The material behavior of rock differs from the behavior of soil in the sense it is generally stiffer and stronger. The dependency of the stiffness on the stress level is almost negligible, so stiffness of rock can be considered constant. On the other hand the dependency of the shear strength on the stress level is significant. In this respect, heavily jointed or weathers rock can be regarded a frictional material. A first approach is to model the shear strength of rock by means of the Mohr-Coulomb failure criterion.

However, considering the large range of stress levels where rock may be subjected to, a linear stress-dependency, as obtained from Mohr-Coulomb, is generally not a sufficient. Furthermore, rock may also show a significant tensile strength. The Hoek-Brown's low failure criterion is a better non-linear approximation of the strength of rocks. It involves shear strength as well as tensile strength in a continuous formulation. Together with Hook's law of isotropic linear elastic behavior it forms the hook-Brown model for rock materials.

Original Hoek-Brown criterion, defined by the equation:

$$\sigma'_1 = \sigma'_3 - \sigma_{ci} \left(m_b \frac{-\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (6.4)$$

Where σ'_1 and σ'_3 are the major and minor effective principal stresses at failure σ'_{ci} is the uniaxial compressive strength of the intact rock material and m and s are material constants. Where m_b is a reduced value of the material constant m_i and is given by:

$$m_b = m_i \exp \left(\frac{GSI-100}{28-14D} \right) \quad (6.5)$$

S and A are constants for the rock mass given by the following relationships:

$$S = \exp \left(\frac{GSI-100}{9-3D} \right) \quad (6.6)$$

$$a = \frac{1}{2} + \frac{1}{6} \left[\exp \left(\frac{-GSI}{15} \right) - \exp \left(\frac{-20}{30} \right) \right] \quad (6.7)$$

6.14.4 Modulus of deformation

The rock mass modulus of deformation is given by:

$$E_m(GPA) = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_{ci}}{100}} * 10^{((GSI-10)/40)} \quad (6.8)$$

Equation 6.8 applies for $\sigma_{ci} \leq 100$ MPa. For $\sigma_{ci} > 100$ MPa, use equation 11b.

$$E_m(GPA) = \left(1 - \frac{D}{2}\right) * 10^{((GSI-10)/40)} \quad (6.9)$$

by the inclusion of the factor D, to allow for the effects of blast damage and stress relaxation.

6.14.5 Mohr-Coulomb criterion

Since most geotechnical software is still written in terms of the Mohr-Coulomb failure criterion, it is necessary to determine equivalent angles of friction and cohesive strengths for each rock mass and stress range. This is done by fitting an average linear relationship to the curve generated by solving equation 2 for a range of minor principal stress values defined by $\sigma_t < \sigma_3 < \sigma'_{3max}$, as illustrated in Figure . The fitting process involves balancing the areas above and below the Mohr-Coulomb plot. This results in the following equations for the angle of friction and cohesive strength:

$$\varphi' = \sin^{-1} \left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}} \right] \quad (6.10)$$

$$C' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3n})^{a-1}/((1+a)(2+a)))}} \quad (6.11)$$

The Mohr-Coulomb shear strength τ , for a given normal stress σ , is found by substitution of these values of φ' and C' in to the equation: The equivalent plot, in terms of the major and minor principal stresses (Figure 6.22), is defined by:

$$\sigma'_1 = \frac{2C' \cos \varphi'}{1 - \sin \varphi'} + \frac{1 + \sin \varphi'}{1 - \sin \varphi'} \sigma'_3 \quad (6.12)$$

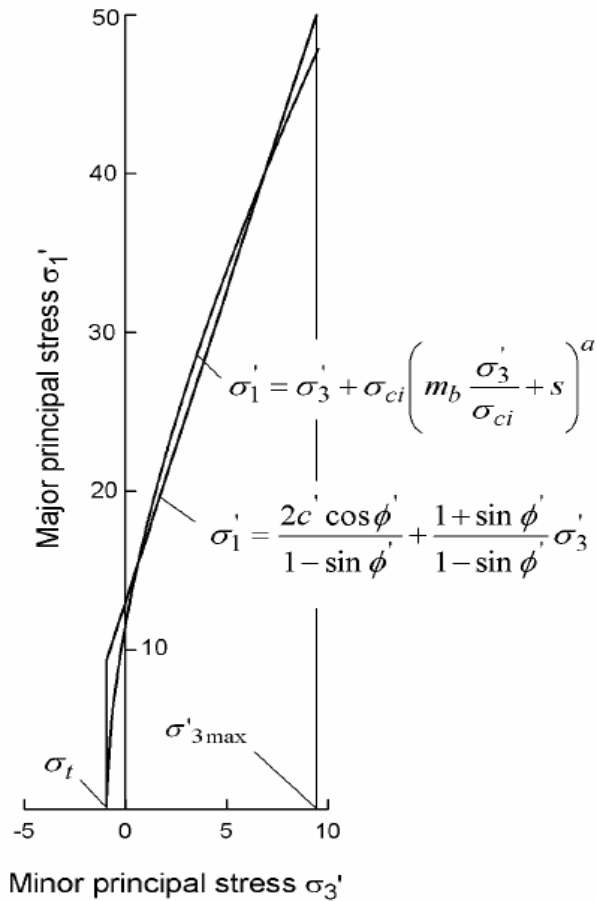


Figure 6.22 : Minor principal stress plot major principal stress (Hoek et al, 2002).

6.15 Parametric Study

The parametric study is carried out by repeating the finite element analysis with the previously mentioned soil model for three cases by different geometry. For the same height, while keeping other parameters remain unchanged. Variable parameters are changed for four alternatives, to see the effect of difference parameters on deformation of wall for two different loads.

The parametric study is performed using confirmed soil model, total of 32 analyses are done to find out the effect of parameters in deformation. In some cases the source of supplying backfill material is changed during the application, therefore the lack of a reference is seen, to anticipate the deformations. Parameters that is used as selected backfill materia's in parametric study are illustrated in Table 6.21 and 6.22. The Valdez program which is acceptable widely used in design of MSE walls project, it does not consider the cohesion and young modulus and Poisson's ratio in calculation of internal stability of wall, it just considers friction angle and unit weight in the

calculation. Therefore it is useful to determine the effect of these parameters in high importance cases.

Table 6.21 : Variable properties of soil in parametric study.

For Rectangular Geometry Wall				
Selected Backfill		Variable Parameter		Value
Selected Backfill-1	Cohesion (c')		10 KN	
	Cohesion (c')		20 KN	
	Cohesion (c')		30 KN	
	Cohesion (c')		40 KN	
selected Backfill-2	Modulus of elasticity (E)		10000	
	Modulus of elasticity (E)		20000	
	Modulus of elasticity (E)		30000	
	Modulus of elasticity (E)		50000	
Selected Backfill-3	Poisson's ratio (v)		0.1	
	Poisson's ratio (v)		0.2	
	Poisson's ratio (v)		0.3	
	Poisson's ratio (v)		0.4	
Selected Backfill-4	Internal friction angle (ϕ')	Dilatancy angle (Ψ)	32	2
			34	4
			36	6
			38	8

Table 6.22 : Minimum selected backfill properties of selected backfill in this project.

Parameter	Selected Backfill
γ Saturated(KN/m ²)	20
γ Unsaturated(KN/m ²)	20
ϕ' Internal friction angle	38
c' Cohesion	1
E Modulus of elasticity	40000
Ψ Dilatancy angle(degree)	8
v Poisson's ratio	0.2

6.15.1 Mobile loads

In the MSE wall by using Steel strips, the Inextensibility of the reinforcement acts as a hooping of the reinforced fill. Generally the strips and granular reinforced strips are stuck together and the lateral deformation is very limited and as the fill is granular and compacted therefore, no plasticity mechanism happens. This is very obviously obtained by using numerical model that is produced by Flac 6.0 software, in this analysis the construction phases and loading are applied corresponding to railway loading on two tracks.

In this model the plasticization zones of soil that is in shear and it appears fundamentally along a vertical plane behind the reinforced mass soil, therefore, there is no plasticization observed (Figure 6.23) in the reinforced fill zone (Freitag et al, 2011).

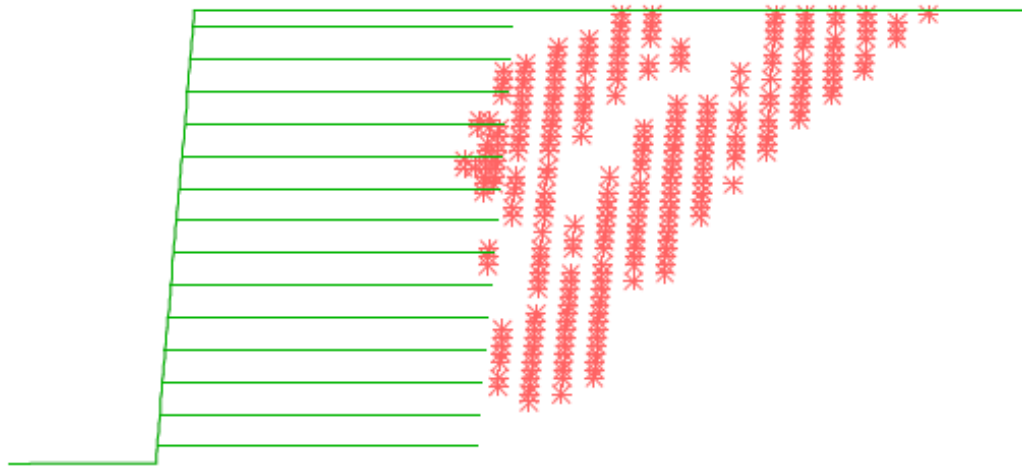


Figure 6.23 : Plasticisation (crosses) in the earth retained by the reinforced soil structure with inextensible reinforcing strips (Freitag et al, 2011).

6.16 Safety Factors for MSE Wall Structure

6.16.1 Plaxis safety factor software

In plaxis (2016) the safety analysis is done by using the ϕ/c reduction method, $\tan(\phi)$ and c is reduced according to the rule:

$$\Sigma Msf = \tan(\phi'_{input}) / \tan(\phi'_{reduced}) = c'_{input} / c'_{reduced} \quad (6.13)$$

Because of this (artificial) strength reduction, we may confront out-of-balance forces in the model. This out-of-balance will be solved by the calculation kernel, which will effect in deformations. The additional displacements that are showed in safety steps do not have the physical; actually the incremental displacements and incremental shear strains are calculated in one step before safety and indicate the likely failure mechanism in mentioned step.

When the failure is happens the soil strength has inchmeal reduction corresponding to strength reduction factor can be accepted as a factor of safety on soil strength, this is the idea

of ϕ/c reduction. We can elicit that whether failure occurs by considering the ϕ/c reduction mechanism the small reduction of strength is applied this leads to a large change of strains and displacements. By using more additional steps in calculation of safety factor by a ϕ/c reduction, the constant value of safety is reached, because of generate more additional displacements (Plaxis Manual 2016).

6.17 Analysis Methods of Slide Software

The limit equilibrium analysis methods to be used for the Slide analysis in this project are :

- Bishop Simplified (Bishop, 1955).
- Janbu Simplified (Janbu, 1973).

Modified Bishop's Method of Analysis

The Modified Bishop's method is a bit different from the other method of slices in limit equilibrium. In this method the normal interaction forces between the two adjacent slices are accepted to be collinear, Therefore, the resultant inter slice shear force is equal to zero.

Janbu's simplified Method of Analysis

In the method of slices in limit equilibrium the resultant of inter-slice forces are horizontal. An empirical correction factor is used to account for inter-slice shear forces. Compound of two methods is chosen to calculate the safety factor in slide. The soil parameters are chosen according Tables 6.6 to 6.10. The selected back filled

is modeled as Mohr-Coulomb and the rocks behind and under wall are modeled by using Hook -Brown method.

6.18 Analysis and Evaluations

6.18.1 Deformation analysis for MSE wall with rectangular cross section

The first case is the MSE wall with rectangular cross section geometry (Figure 2.24). For the better understanding of the working system, the mentioned geometry is analyzed according to maximum and minimum material properties. which is defined from company as a selected backfill for this case, in addition for the effect of other backfill properties from other cases and parametric comparison with them, three selected backfill are taken from articles as illustrated in Tables 6.8 ,6.9 and 6.10.

The important factor in design of MSE wall is the deformation and factor safety for the case. The deformation should not exceed the safe range according to each counter's criteria as well as the safety factor for expected life time, therefore it is important to take into account the erosion dependent in service life.

The maximum value of total displacements in -X direction is for both 50 KN and 100 KN (Figure 6.24) uniform distributed loads is for Hossain et al. (2012) by 19 cm and 21 cm. the minimum deformation is observed for selected backfill 1 that has the maximum friction angel between the selected backfill materials.

The maximum value of deformation in -X direction for facing panels by considering the two type of the train loads for 50 KN and 100 KN (Figure 6.25) is belong to Hossain et al. (2012) by 11cm and 13 cm.

The total displacements in soil mass and the total displacement of facing panels in -X direction for both 50 KN and 100 KN are listed in Table 6.23.

The total settlement of a MSE walls can damage the function of the wall in some aspect, however, these kind of structures is able to tolerant the limited differential settlements and usually the facing panels which determines the limits to settlement (Table 6.24).

In this case, the sensitivity of the project and by considering the comfort level of train, design should be done in order to minimize the deformation, therefore the EPDM pads quality has the important role.

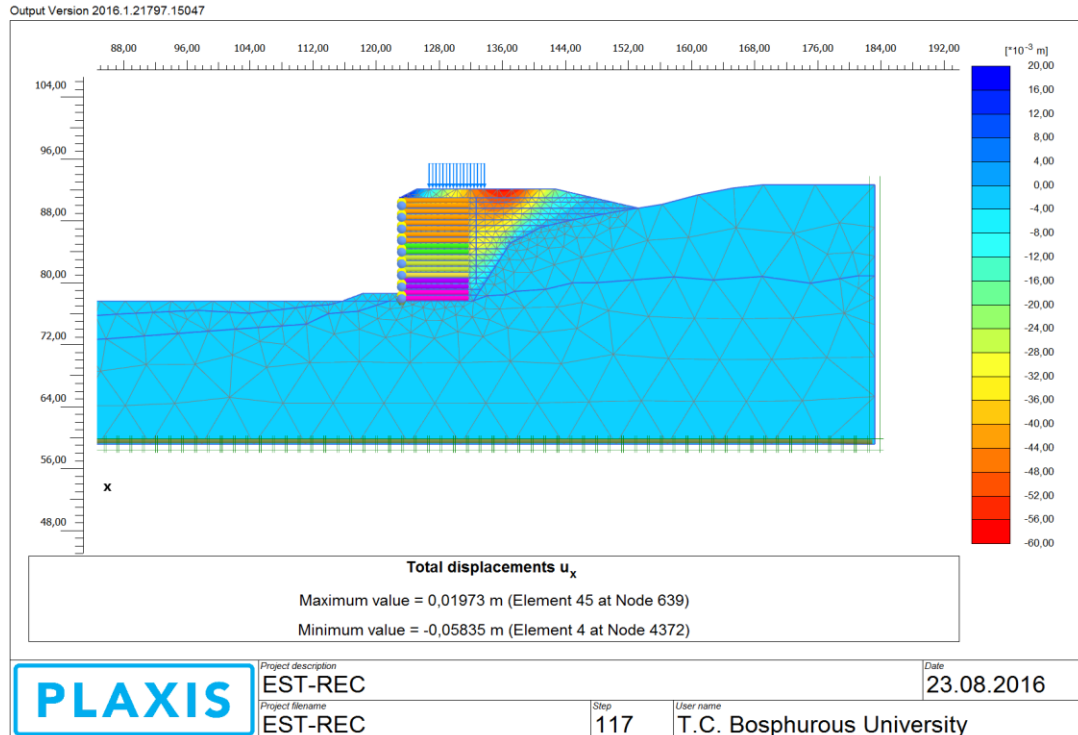


Figure 6.24 : Total displacements in -X direction for 100KN (Plaxis 2016).

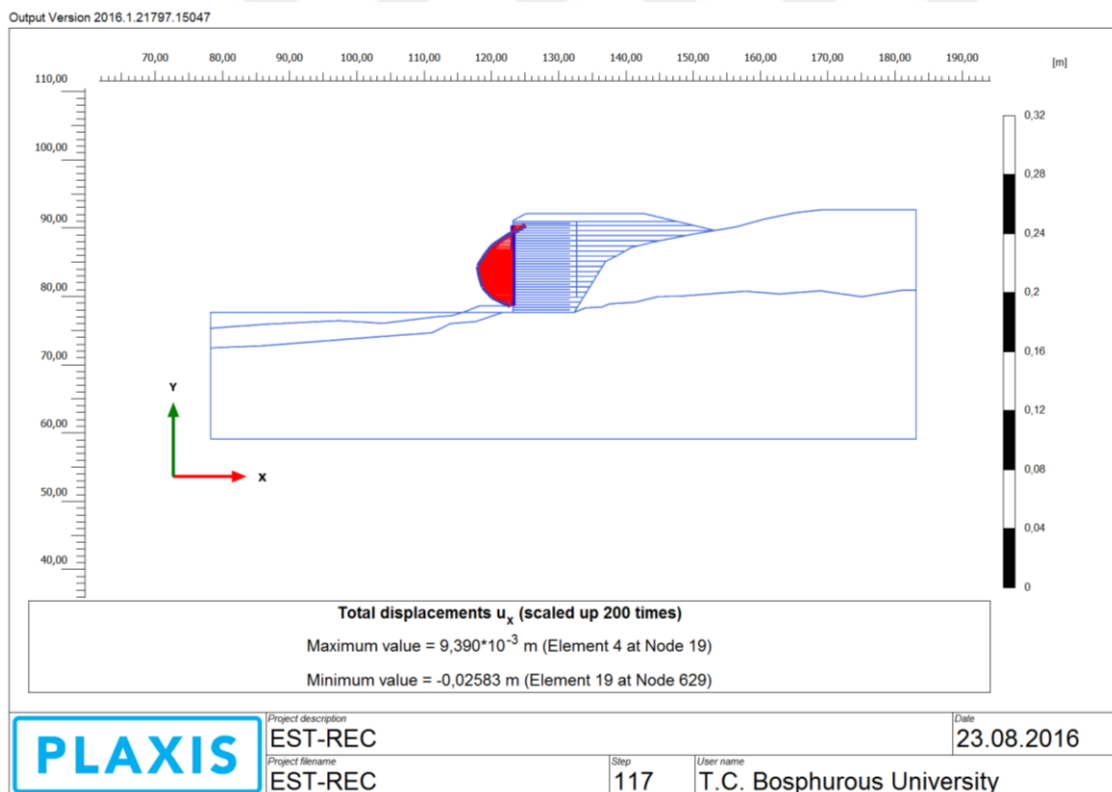


Figure 6.25 : Total displacements of facing panels in -X direction for 100 KN, (Plaxis 2016).

Table 6.23 : Total displacements in soil mass and the total displacement of facing panels in -X direction.

Selected Backfill	Uniform Load	Total Displacement(U_x) in meter	Total facing panel Displacements(U_x) in meter
(Hou, 2009)	50 KN	0,1523	0,06839
	100 KN	0,169	0,0812
(Hossain et al, 2012)	50 KN	0,1937	0,1191
	100 KN	0,2156	0,1385
(Damians et al, 2013)	50 KN	0,07719	0,03293
	100 KN	0,08572	0,03974
(Selected backfill 1)	50 KN	0,05268	0,02099
	100 KN	0,05835	0,02583
(Selected backfill 2)	50 KN	0,0617	0,02443
	100 KN	0,06865	0,03024

Table 6.24 : Guide to the effects of settlements (BS 8006-1, 2010).

Maximum differential settlement	Comment
1 in 1000	Not normally significant
1 in 200	Full height panels may be affected by joints closing or opening. Normal safe limit for segmental blockwalls.
1 in 100	Normal safe limit, without special measures, for discrete concrete panel facings
1 in 50	Normal safe limit for semi-elliptical steel face elements. Discrete concrete panels may suffer closed joints if special measures not included
1 in < 50	Soft facings might suffer distortion affecting their retaining ability

NOTE There is no intended firm limit between categories. This is a preliminary guide only.

By considering the total height of the wall, which is equal to 14 m and the maximum vertical deformations of facing panel which is equal to 5,8 mm (Figure 6.26), the differential settlements that will take place in first category, are not normally significant. The other selected back fill materials also are safe and it could be considered in second category.

6.18.2 Deformation analysis for MSE wall with trapezoidal cross section

The second case is the MSE wall with trapezoidal cross section, which means that the length of strips is shortened step by step from top to bottom of the wall. For the better understanding of the trapezoidal MSE wall system, the mentioned geometry is analyzed in maximum and minimum selected backfill properties, which are defined from company for selected back fill for this case, in addition for better apperceiving and parametric comparison with other soil properties for other cases, three soil parameters are taken from Table 6.8 to 6.10. For the total settlement of a MSE walls the all deformation in -Y direction (Figure 6.26) according to BS 8006-1 (2010), are in first or second category. Therefore, there is not significant problem in vertical displacements. The maximum value of total displacements in -X direction (Figure 6.27) for both 50 KN and 100 KN uniform distributed loads is gained for selected backfill properties by Hou (2009), with 27 cm and 25 cm. The minimum deformation is observed for Selected backfill 2. It has the maximum friction angel between the studied selected backfill materials. The maximum value for total deformation in -X direction (Figure 6.28) for facing panels by considering 50 KN and 100 KN loads is obtained for Hossain et al. (2012) by 7,2 cm and 8,9 cm, as the results shows, by increasing the load from 50 KN to 100 KN, approximately the deformation is increase by the rate of 10% .

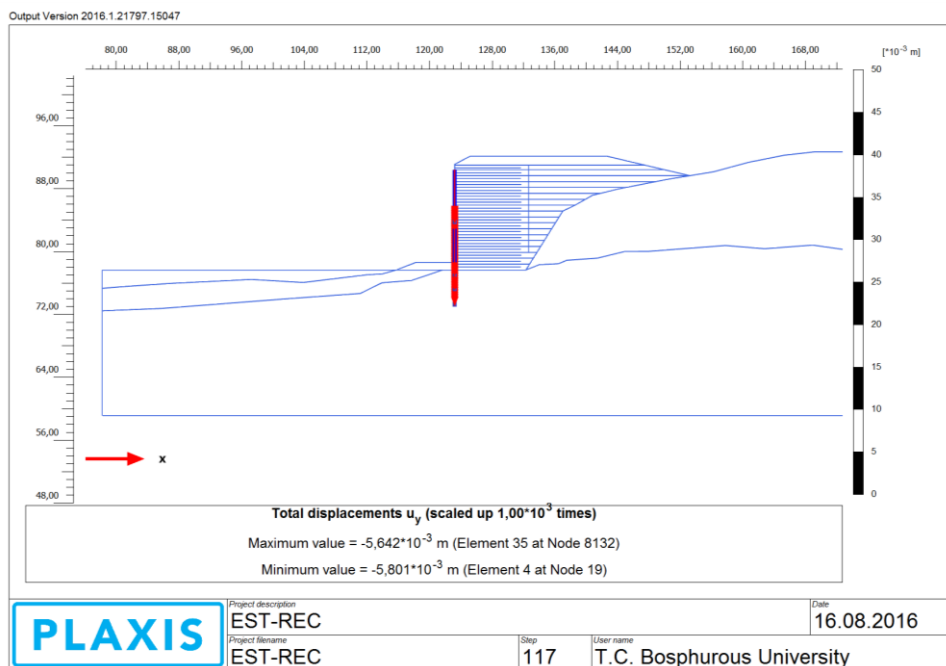


Figure 6.26 : Total facing panel displacements in -Y direction for 100 KN (Plaxis , 2016).

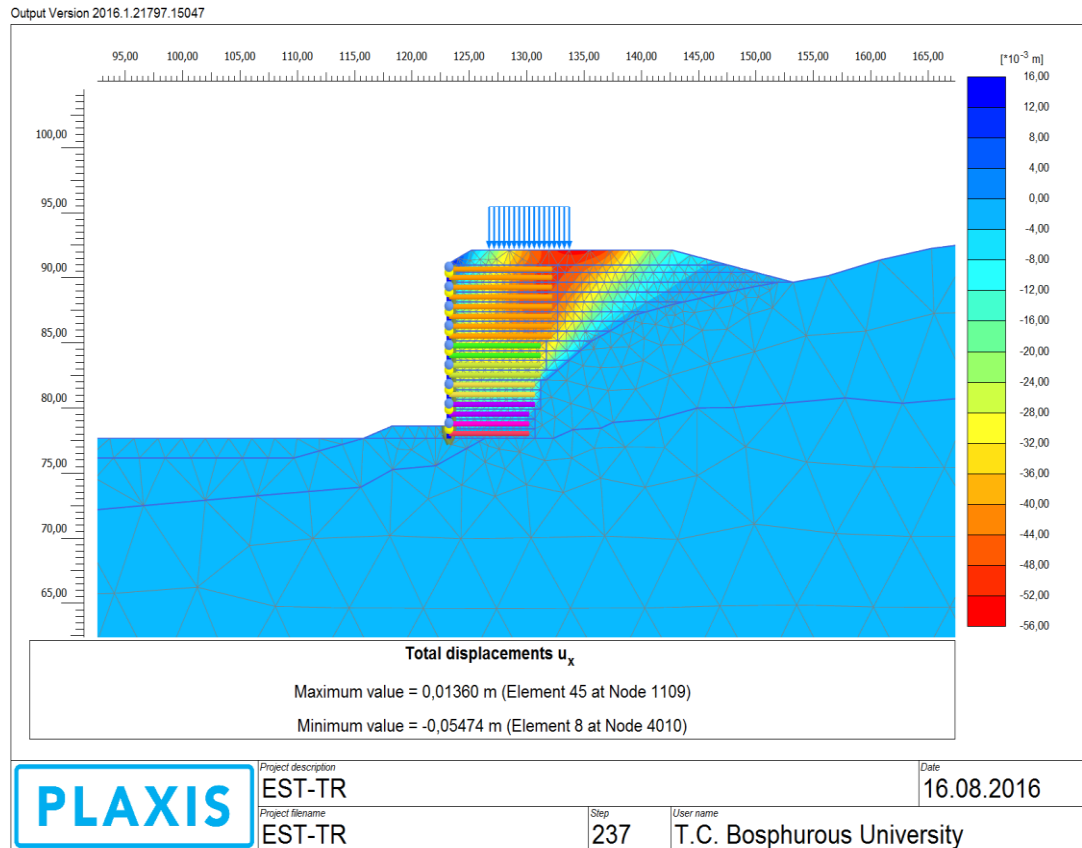


Figure 6.27 : Total displacements in -X direction for 100 KN, (plaxis 2016).

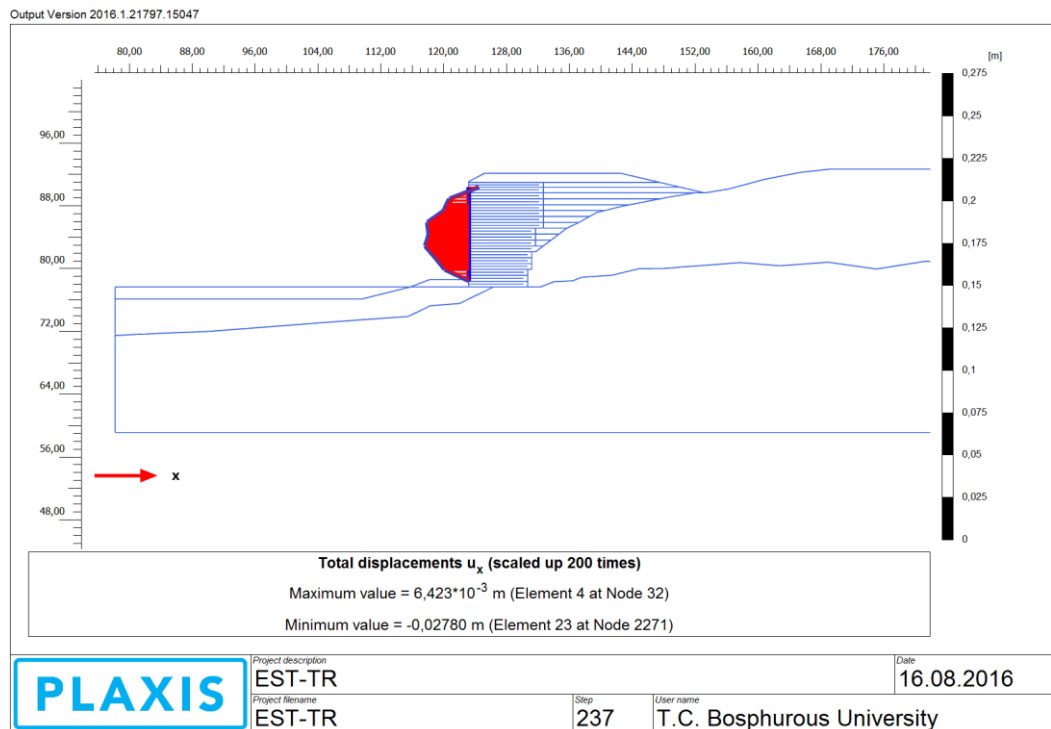


Figure 6.28 : Total displacements of facing panels in -X direction for 100 KN, (plaxis 2016).

The total displacements in soil mass and the total displacement of facing panels in -X direction for both 50 KN and 100 KN are listed in Table 6.25.

The differential settlements that is illustrated in Figure 6.29, will take place in first category, are not normally significant. The other selected back fill materials also are safe and it could be considered in second category.

Table 6.25 : Total displacements in soil mass and the total displacement of facing panels in -X direction.

Selected Backfill	Uniform Load	Total Displacement (U _x) in meter	Total facing panel Displacements (U _x) in meter
(Hou, 2009)	50 KN	0,2572	0,06162
	100 KN	0,2697	0,06925
(Hossain et al, 2012)	50 KN	0,1821	0,07221
	100 KN	0,204	0,08978
(Damians et al, 2013)	50 KN	0,07228	0,03211
	100 KN	0,08135	0,03879
(Selected backfill 1)	50 KN	0,04843	0,02318
	100 KN	0,05474	0,0278
(Selected backfill 2)	50 KN	0,06243	0,02738
	100 KN	0,0688	0,03226

6.18.3 Deformation analysis for hybrid MSE-soil nail wall

TerraLink™ technique and elements are similar to those used for Reinforced Earth® solution. These elements are placed between well compacted backfill layers (selected backfill). The primary characteristic is that reinforcements are linked to the back face nailing part, the existing structure or stiff blocks which enables combination or continuity of reinforcement between the narrow Reinforced Earth® wall and existing ground or structure.

The research is carried out in order to evaluate the comparison of two systems with previously mentioned parameters. The first case is the Terralink™ with direct connections and the second case is Terralink™ without direct connections.

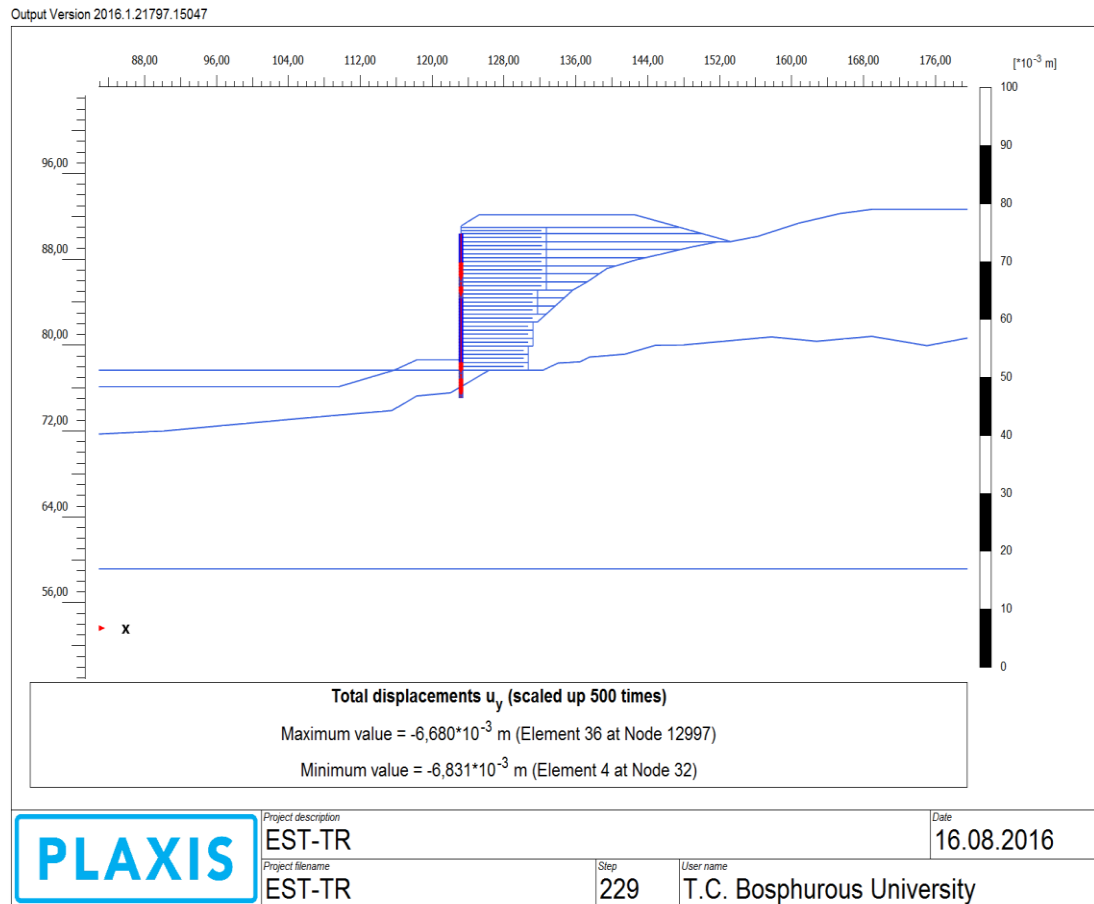


Figure 6.29 : Total facing panel displacements in -Y direction for 50 KN (Plaxis , 2016).

The maximum total displacement in -X direction (Figure 6.30) for both 50 KN and 100 KN loads is obtained 10,6 cm and 11,9 cm ,that is belong to Hossain et al. (2012). Beside that the minimum total displacement in -X direction for the heaviest train load was gained by selected backfill 1 by 3,2 cm.

Then maximum deformation in facing panels (Figure 6.31) are respectively reached by Hossain et al. (2012) and Hou (2009), with the lowest young's modulus and friction angel.the maximum facing panel deformation occurred in the haviest train.

The total displacements in soil mass and the total displacement of facing panels in -X direction for both 50 KN and 100 KN are listed in Table 6.26.The total settlement of a MSE walls the all deformation according to BS 8006-1 (2010), are in first and second category .therefore, there is not significant problem in vertical displacement. The first case is The Terralink™ without direct connections.

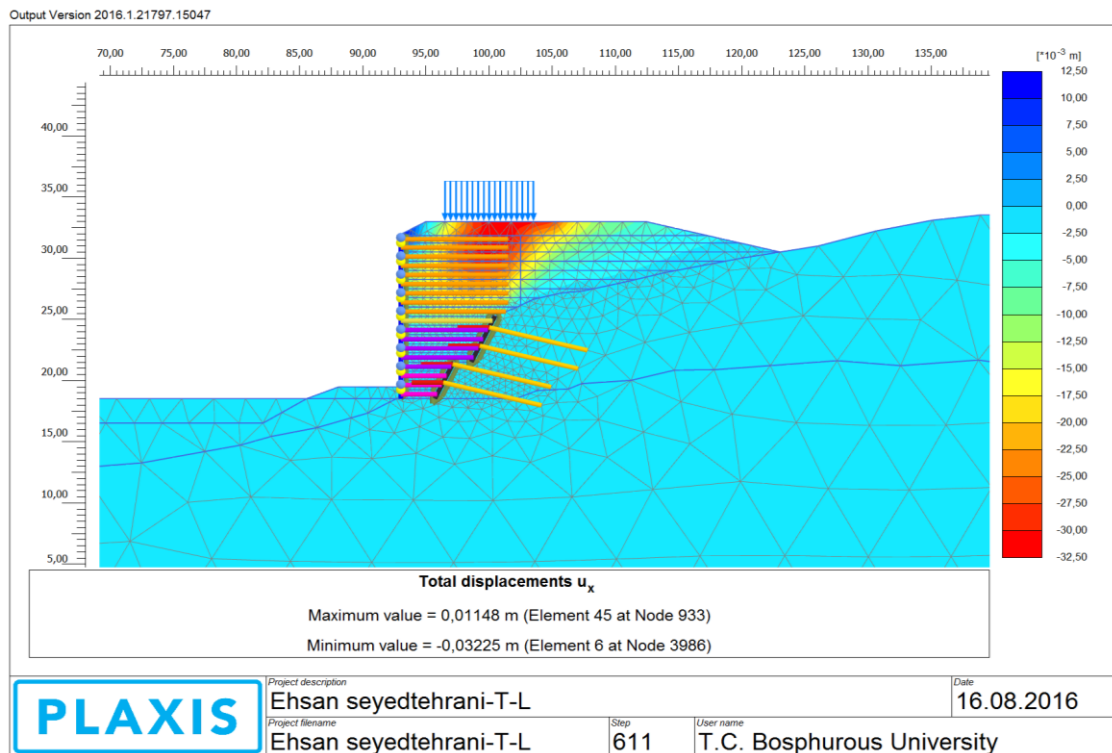


Figure 6.30 : Total displacements of facing panels in -X direction load, Hybrid MSE wall with direct connection (Plaxis, 2016).

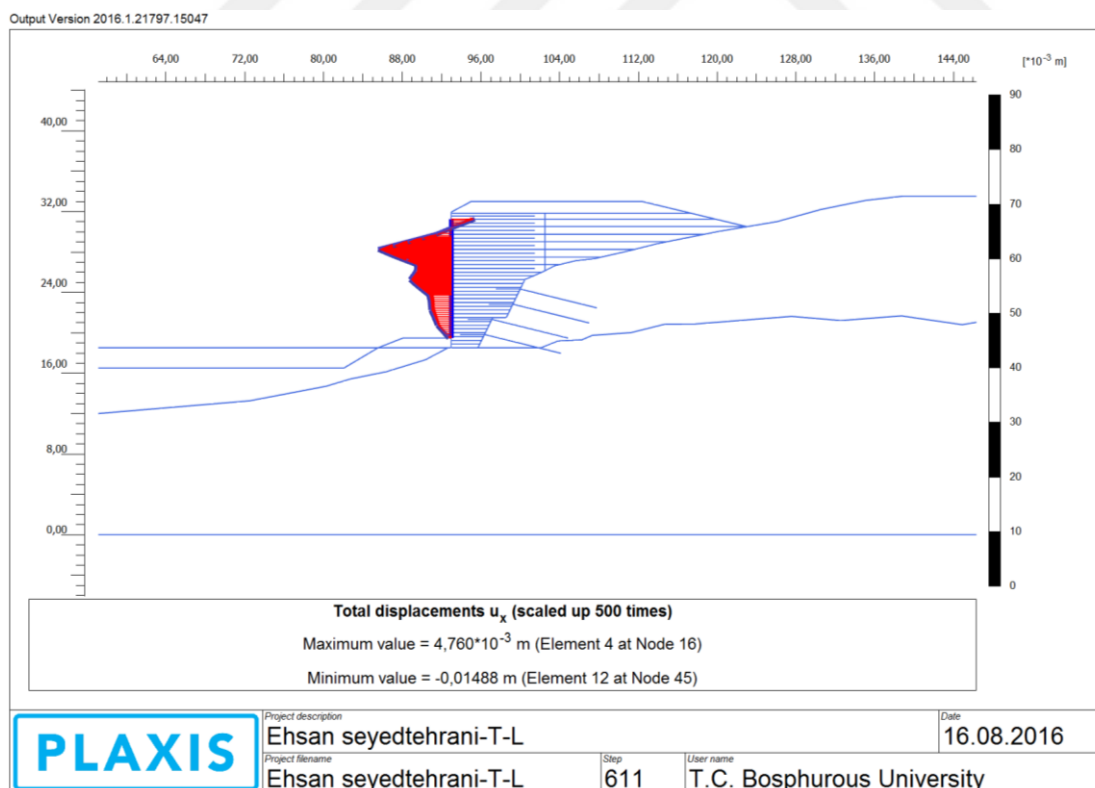


Figure 6.31 : Total displacements in -X direction for 100 kN load, Hybrid MSE wall with direct connection (Plaxis, 2016).

Table 6.26 : Total displacements in soil mass and the total displacement of facing panels in -X direction with direct connection.

Selected Backfill	Uniform Load	Total Displacement (U _x) in meter	Total facing panel Displacements (U _x) in meter
(Hou, 2009)	50 KN	0,08353	0,03197
	100 KN	0,09369	0,04149
(Hossain et al, 2012)	50 KN	0,1067	0,05036
	100 KN	0,1198	0,0622
(Damians et al, 2013)	50 KN	0,04302	0,01723
	100 KN	0,04802	0,0231
(Selected backfill 1)	50 KN	0,02899	0,01056
	100 KN	0,03225	0,01488
(Selected backfill 2)	50 KN	0,03788	0,01308
	100 KN	0,04158	0,01782

The maximum total displacement in -X direction (Figure 6.32) for the minimum and maximum train loads is belongs to soil parameters was taken by Hossain et al. (2012) with 19,9 cm and 20,9 , in addition the maximum deformation in facing panels (Figure 6.33) are respectively reached by Hossain et al. (2012), it has the lowest friction angle among other selected backfills properties.

In both two cases the deformation in total and facing panels are acceptable. By contrasting the directly connected and the TerraLink™ system without connection technique, the minimum deformation is gained in directly connected by reducing deformation approximately 2%.The deformation of shotcrete (Figure 6.34) is acceptable and for 100 KN is about 3,3mm is obtained.

The total settlement of a MSE walls the all deformation according to BS 8006-1 (2010), are in second category . Therefore, there isn't remarkable obstacle in vertical displacement. The total settlement of a MSE walls are according to BS 8006-1 (2010), in first and second category . Therefore, there is not considerable problem in vertical displacement. The total displacements in soil mass and the total displacement of facing panels in -X direction for both 50KN and 100 KN are listed in Table 6.27

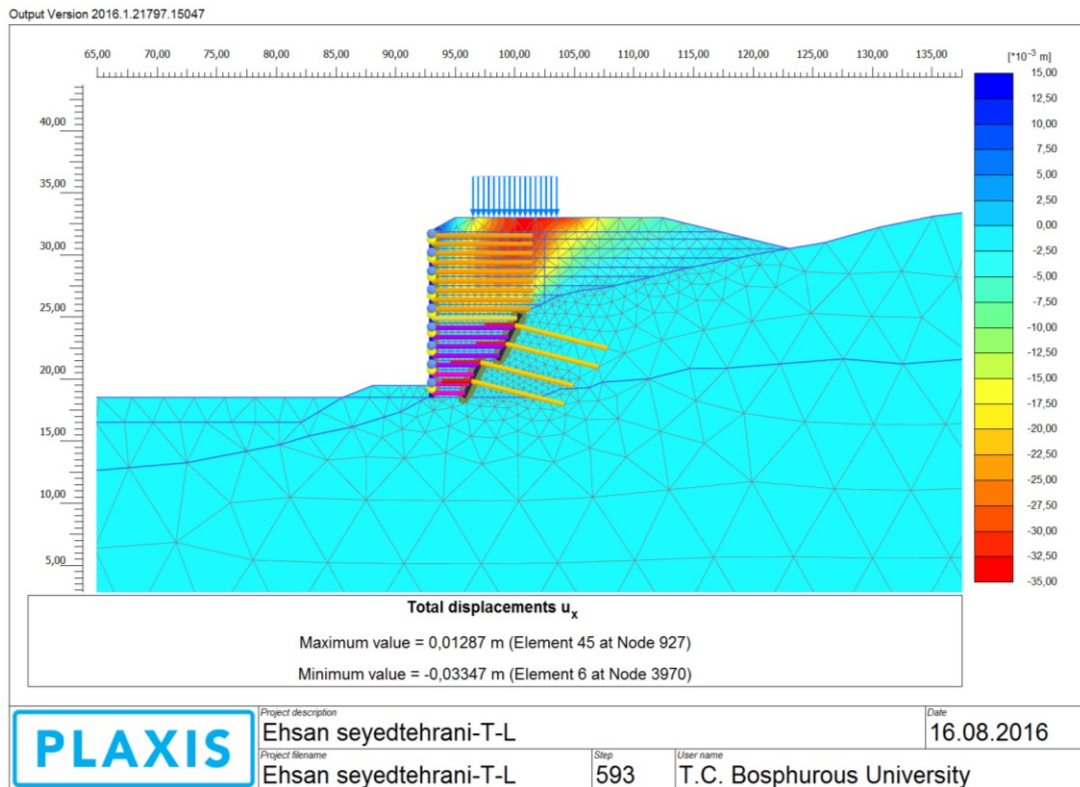


Figure 6.32 : Total displacements in -X direction for 100 kN load, Hybrid MSE wall without direct connection (Plaxis 2016).

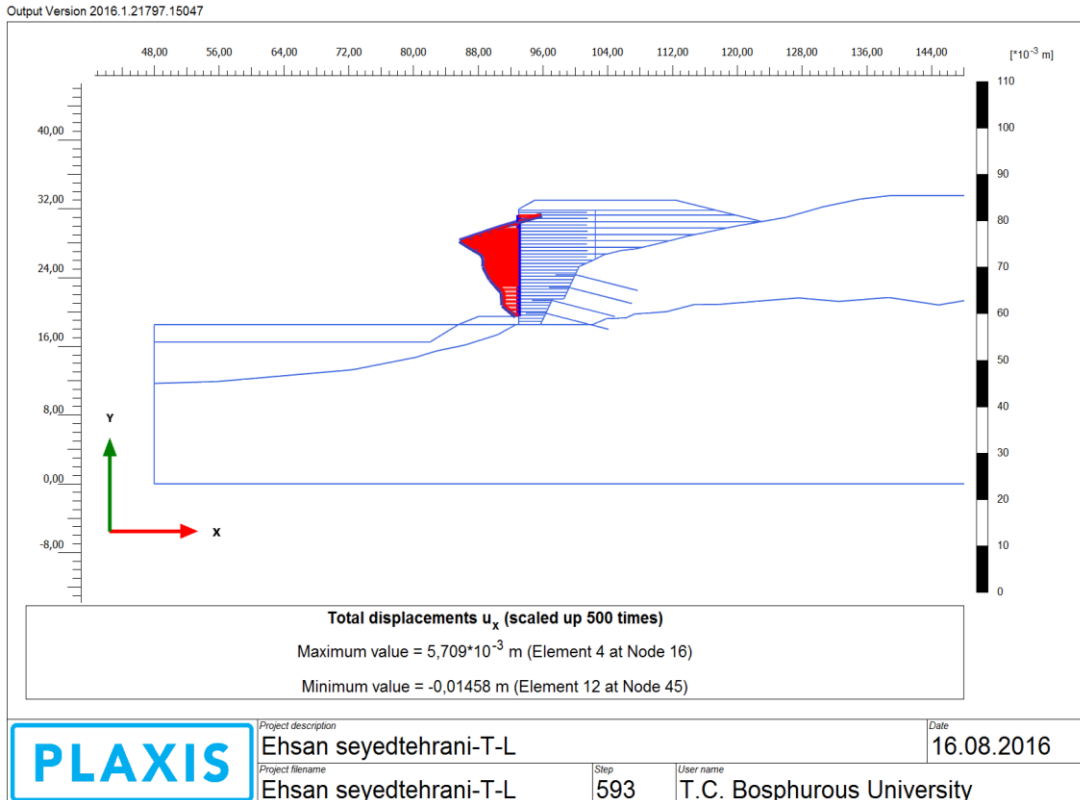


Figure 6.33 : Total displacements of facing panels in -X direction load, Hybrid MSE wall without direct connection, (Plaxis 2016).

The rupture of friction elements by considering the maximum axial force from Plaxis are checked and the range of safety of strips and friction element are between 1,1 and 1,4 is gained.

Table 6.27 : Total displacements in soil mass and the total displacement of facing panels in -X direction.

Selected Backfill	Uniform Load	Total Displacement(Ux) in meter	Total facing panel Displacements(Ux) in meter
(Hou, 2009)	50 KN	0,09036	0,03511
	100 KN	0,01013	0,04529
(Hossain et al, 2012)	50 KN	0,1991	0,07392
	100 KN	0,2099	0,0843
(Damians et al, 2013)	50 KN	0,04483	0,01579
	100 KN	0,05035	0,02217
(Selected backfill 1)	50 KN	0,03009	0,01005
	100 KN	0,03347	0,01458
(Selected backfill 2)	50 KN	0,03958	0,01387
	100 KN	0,04342	0,01801

For the nails the maximum pullout resistance per unit length and tensile resistance of the tendon are respectively, 391.12 KN and 293.7 KN are calculated. Axial forces in the soil nail before the friction element and strips are connected must not be considered in design, because it is very small (Figure 6.35). For the critical case the Terralink™ with connection and 100 KN load the maximum axial forces which is applied to nail is 117 KN/m (Figure 6.36) which satisfies the nail pullout resistance and tensile resistance.

6.18.4 Safety factors for MSE walls

The finite elements Plaxis 2016 and slide 5.0 (Figure 6.37) are used to calculate the safety factors. In the both soft ware were calculated by considering persuade static seismic method for the region. The safety factors for the previously mentioned properties are listed in Table 6.28.

Slide program uses limit equilibrium methods for both simplified Bishop Method and simplified Janbu method which gives low value in the factors for all geometries and

all selected backfill, otherwise finite element method gives high safety factors for rectangular and trapezoidal shape wall by average 8,3% .In complex geometry and system Terralink™, the limit equilibrium in compared to the finite element method seem to be more conservative by about 48%.

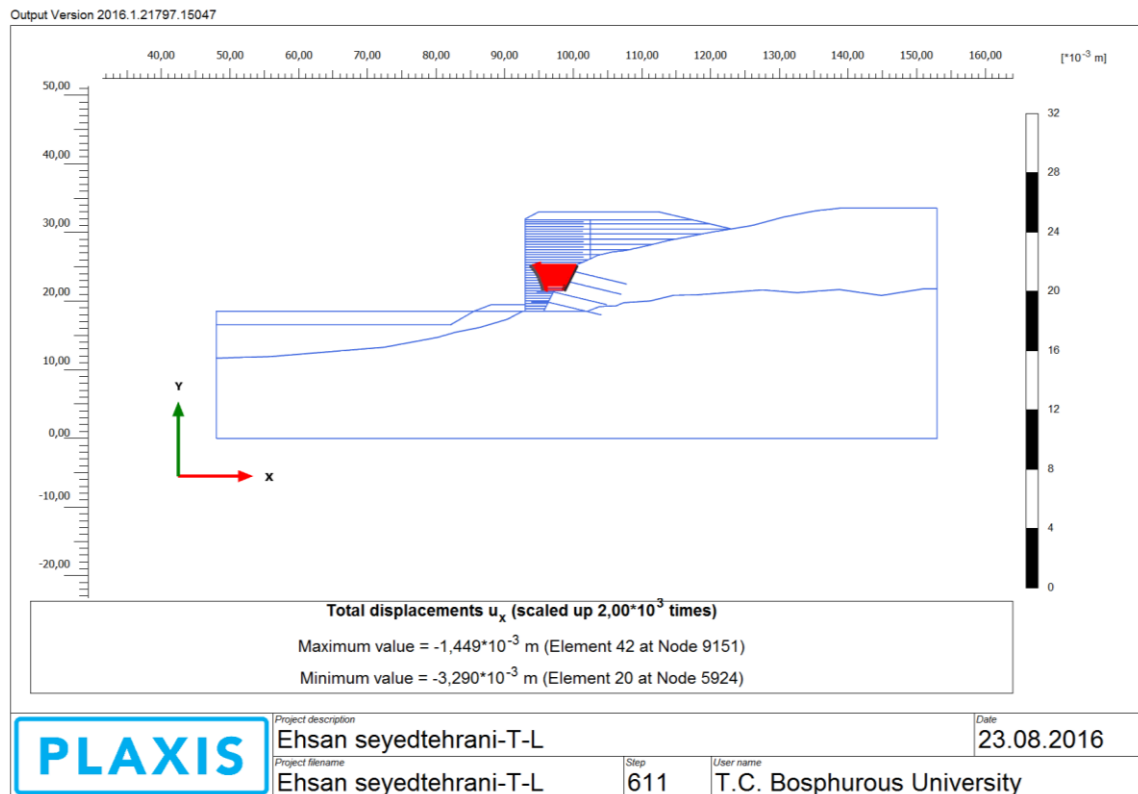


Figure 6.34 : Total shotcrete displacements in -X direction for Hybrid MSE wall with direct connection and 100 KN load.

6.19 Parametric Study for MSE Wall with Rectangular Cross Section

After the proposal of three type of MSE geometry as an alternative method for the project, the rectangular shape MSE wall which is more common in is taken to parametric study.

The selected backfill material 2 is accepted to use as a backfill material in this project. This parametric study is carried out in order to give an understanding of the effect of various parameters in deformation of wall for both total facing and soil mass in -X direction.

Deformation in soil mass is occurs, when a load is applied to it, if the material is elastic, the soil mass return to its original shape after the load is removed.

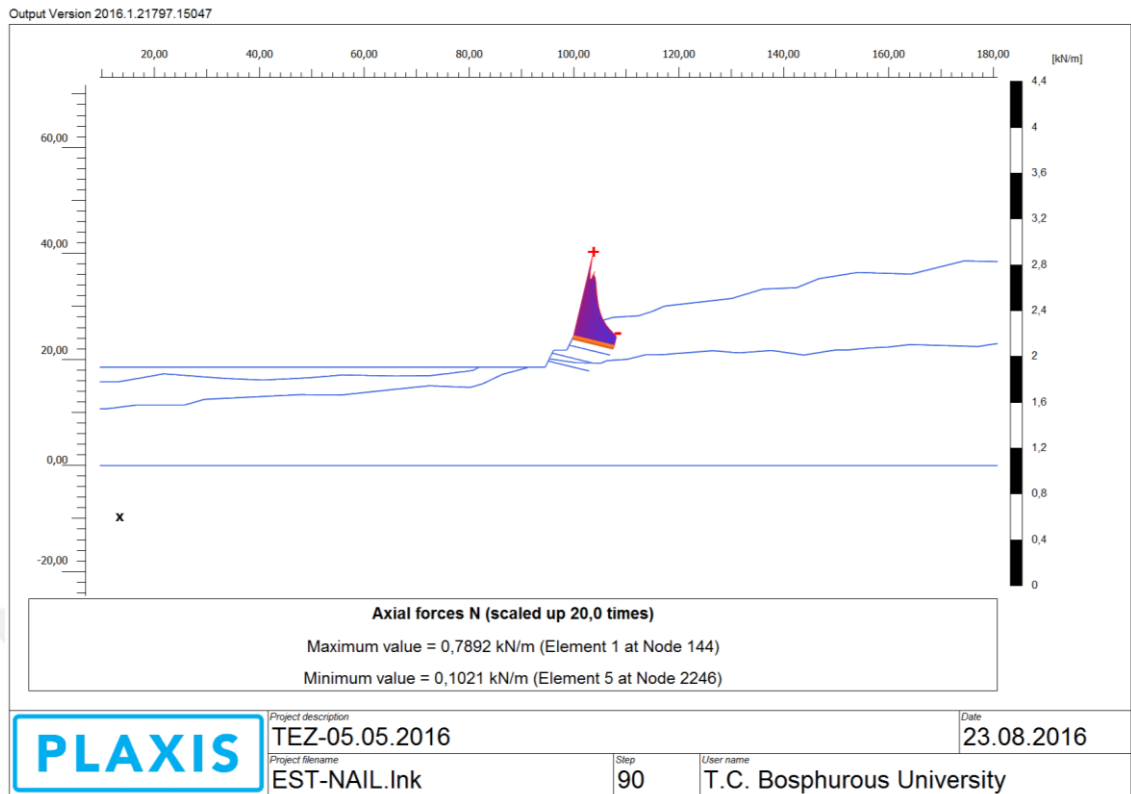


Figure 6.35 : Axial forces in the soil nail before the friction element and strips are connected.

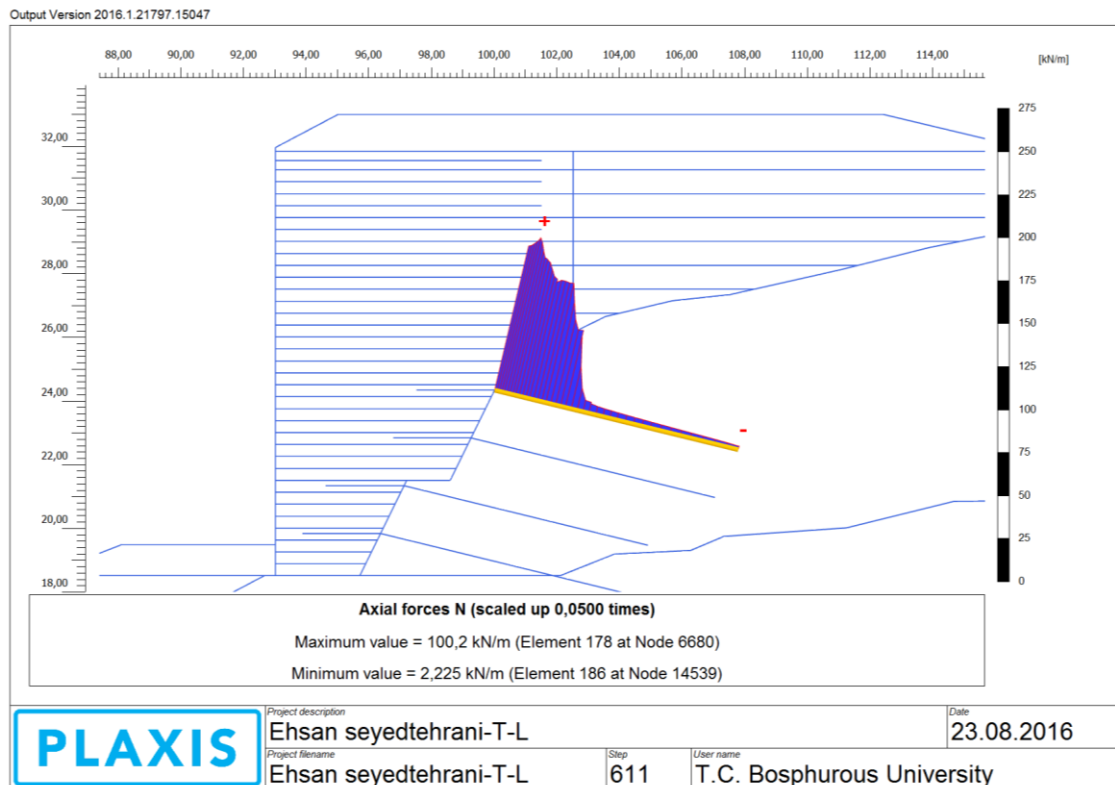


Figure 6.36 : Soil nail under 100 kN load and with connection to friction elements and strips.

By accepting the selected backfill as homogeneous and elastic material the elastic parameters are carried out. The safety factor that is achieved by Slide program which is uses Limit equilibrium analysis in comparisiom with Plaxis software wich is based in Finite element method in simple geometry is approximately the same but in complex geometry like TerraLink™, it is more than two times conservative.

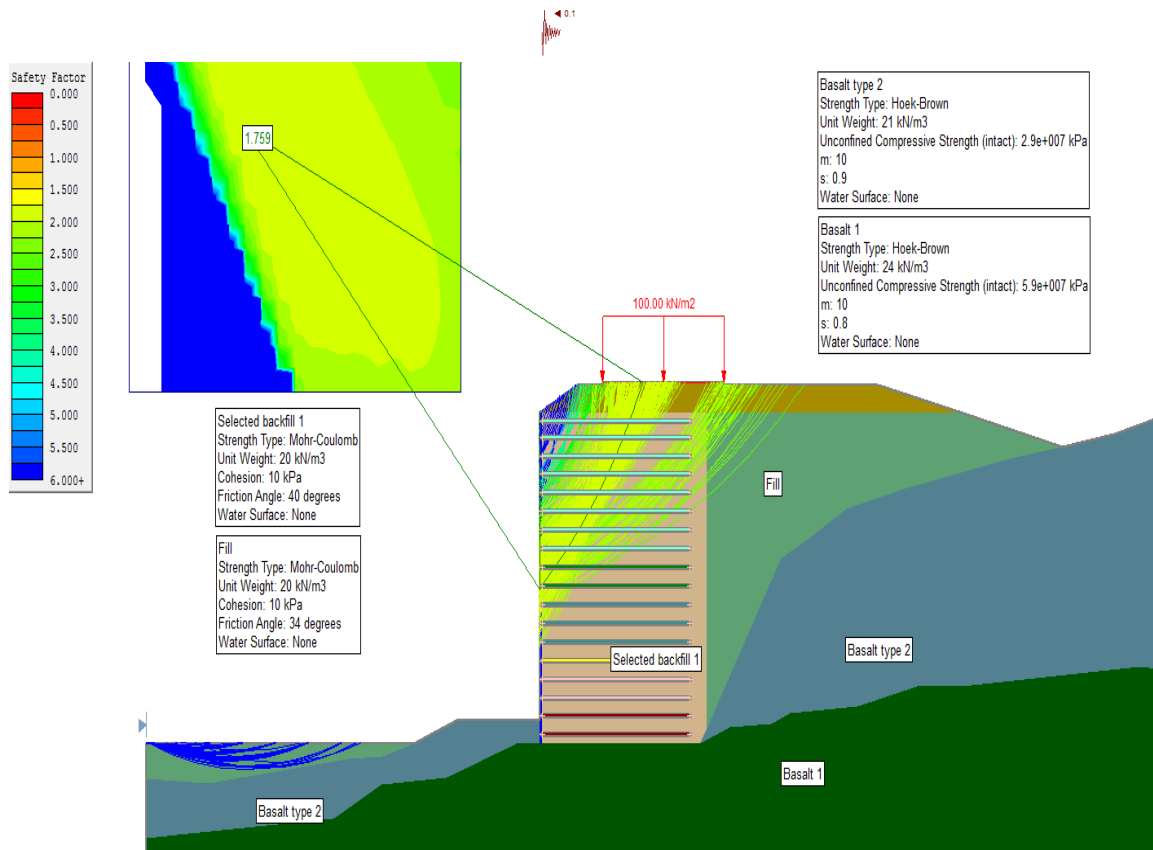


Figure 6.37 : Screen shot of safety factor calculation by slide 5.0 software

6.19.1 Young's modulus and poisson's ratio effects in deformation of wall with rectangular cross section

In analysis of an elastic isotropic material there are requires only two constants the first one is Young's modulus and the second one is Poisson's ratio, if we assume that soils are isotropic elastic materials, by considering that, the strips and granular reinforced strips are stick together and the lateral deformation is very limited and as the fill is granular and compacted material, therefore, no plasticity mechanism happens, so, we have a analytical tool to predict a soil's response under loading.

Table 6.28 : Safety factors for three geometry and two type of loads for mentioned selected backfills.

Selected Backfill	Sowftware	Loads	Rectangular	Trapezoidal	Terralink without Connection	Terralink without Connection
(Selected backfill 1)	Plaxis(Σ Msf)	50 KN	2,145	2,258	3,042	2,936
		100 KN	2,197	2,319	2,923	2,844
	Slide(FOS)	50 KN	1,907	1,743	1,718	1,465
		100 KN	1,844	1,734	1,672	1,416
(Selected backfill 2)	Plaxis(Σ Msf)	50 KN	1,871	1,954	2,623	2,529
		100 KN	1,938	2,048	2,575	2,494
	Slide(FOS)	50 KN	1,753	1,636	1,59	1,341
		100 KN	1,694	1,632	1,554	1,302
(Hou, 2009)	Plaxis(Σ Msf)	50 KN	1,665	1,745	2,418	2,342
		100 KN	1,746	1,855	2,396	2,322
	Slide(FOS)	50 KN	1,694	1,577	1,533	1,286
		100 KN	1,633	1,573	1,496	1,247
(Hossain et al, 2012)	Plaxis(Σ Msf)	50 KN	1,434	1,564	2,228	2,188
		100 KN	1,485	1,661	2,251	2,059
	Slide(FOS)	50 KN	1,617	1,523	1,465	1,222
		100 KN	1,562	1,52	1,432	1,188
(Damians et al, 2013)	Plaxis(Σ Msf)	50 KN	1,755	1,828	2,537	2,445
		100 KN	1,835	1,917	2,482	2,397
	Slide(FOS)	50 KN	1,741	1,599	1,563	1,315
		100 KN	1,672	1,593	1,522	1,272

By keeping the other soil properties constant and Young's modulus and Poisson's ratio variable, effect of Young's modulus in MSE wall deformation for both facing and whole soil mass in -X direction is illustrated in Figure 6.38.

During elastic deformation under drained condition, the soil volume decreases (contracts), and after initial yield the soil volume increases (expands) to the critical state and does not change volume thereafter.(Muni Budhu).

With increasing the load from 50 KN to 100 KN there is not significant change in deformations is observed. The most considerable change in deformation is assessed, by increasing the elastic modulus, the results shows that the deformation reduces by approximately 40%, for each 10000 KPa increments in selected backfill properties. With increasing the load and Poisson's ratio there is not significant change in deformation is determine (Figure 6.39).

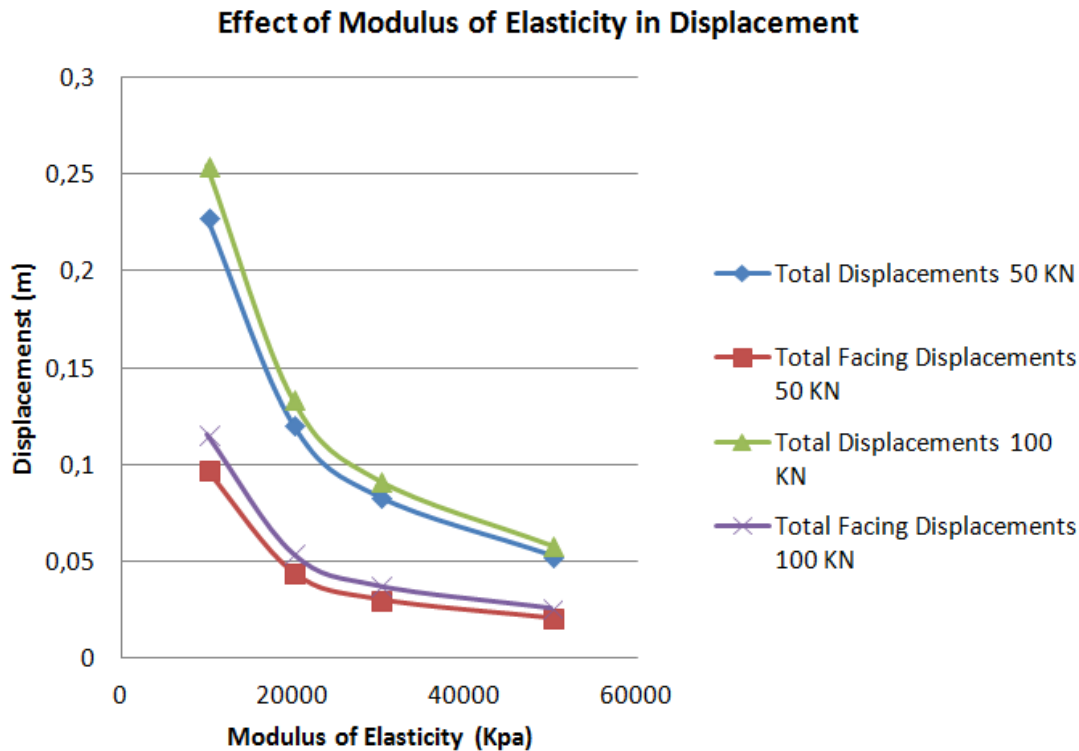


Figure 6.38 : Effect of modulus of elasticity in MSE wall displacements.

6.19.2 Cohesion and friction angel effects in deformation of wall with rectangular cross section

Shear strength of soil is the magnitude of the shear stress that a soil can sustain. When a shear force is applied to a cube of muddy soil or rock at zero normal pressure, the outcome is shear deformation which is accompanied by measurable resistance. The resistance force per unit area is cohesion. In natural soils, cohesion results from electrostatic bonds between clay and silt particles the shear strength of a soil is thus the sum of the cohesive and frictional contributions. Usually in design of MSE wall the selected back fill's cohesion is neglected, actually clay content of soil is disturbed the wall function. With increasing the cohesion by four times the maximum displacement of soil mass and facing panels are reduced by 1,5 cm (Figure 6.40), therefore the high content of clay in soil mass is not recommended.

The effect of friction angel in total displacements in -X direction (Figure 6.41) is reduced with increment of friction angel, by increasing the friction angel from 32 degree to 38 degree the maximum displacement for 50 KN load and 100 KN is reduced about 14%.

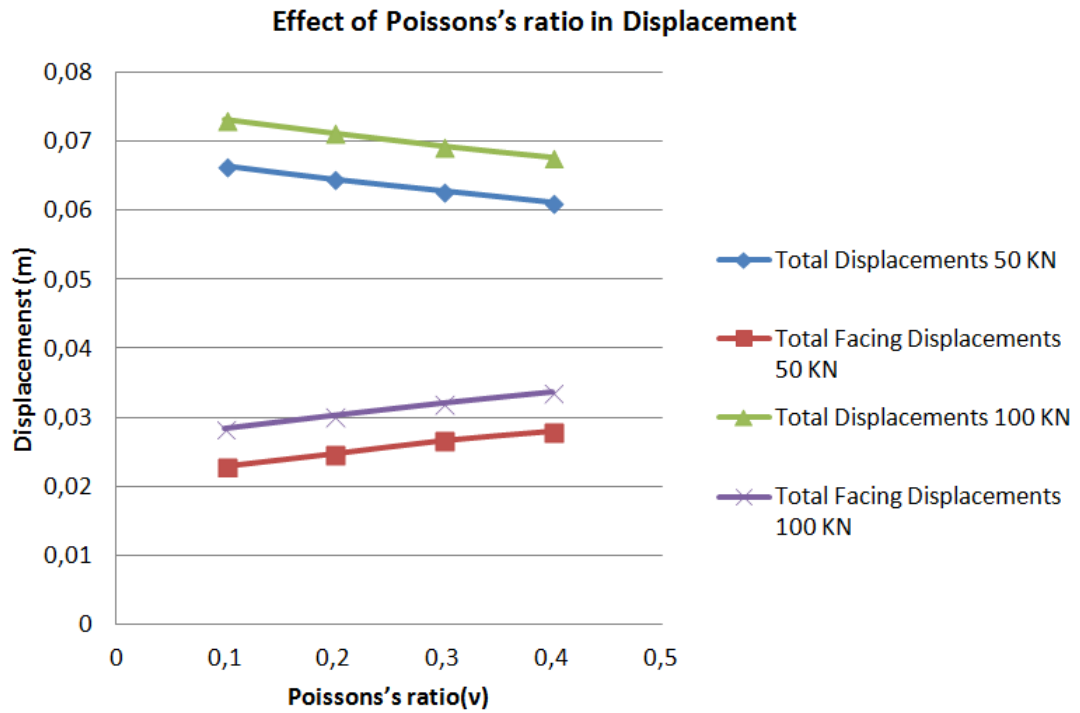


Figure 6.39 : Effect of Poisson's ratio in MSE wall displacements.

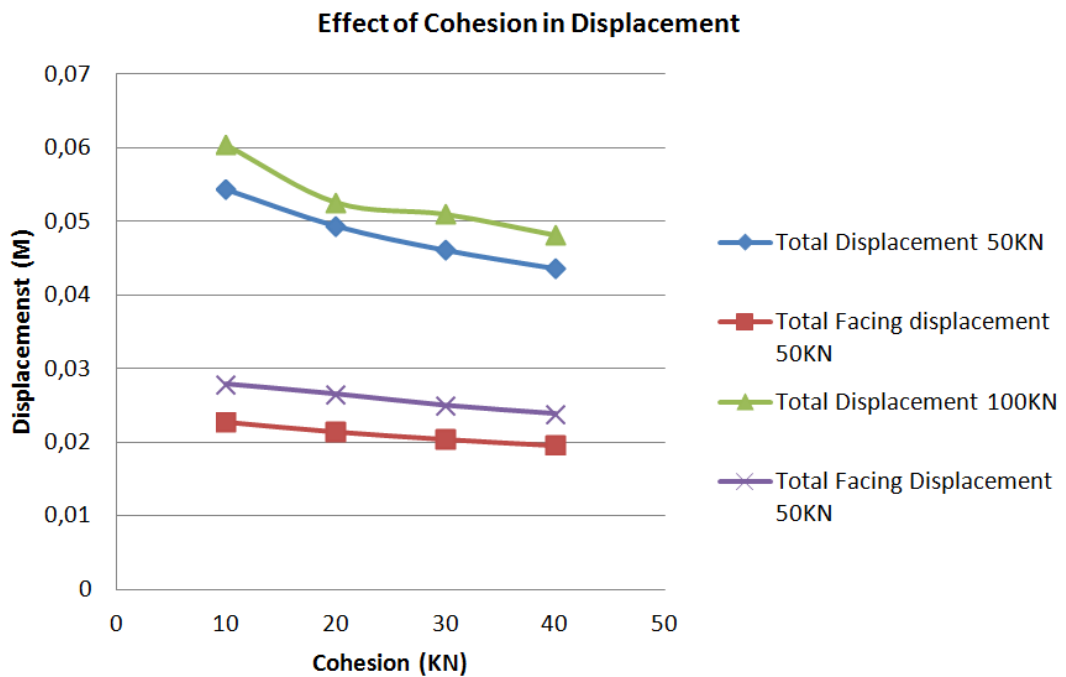


Figure 6.40 : Effect of cohesion in MSE wall displacements.

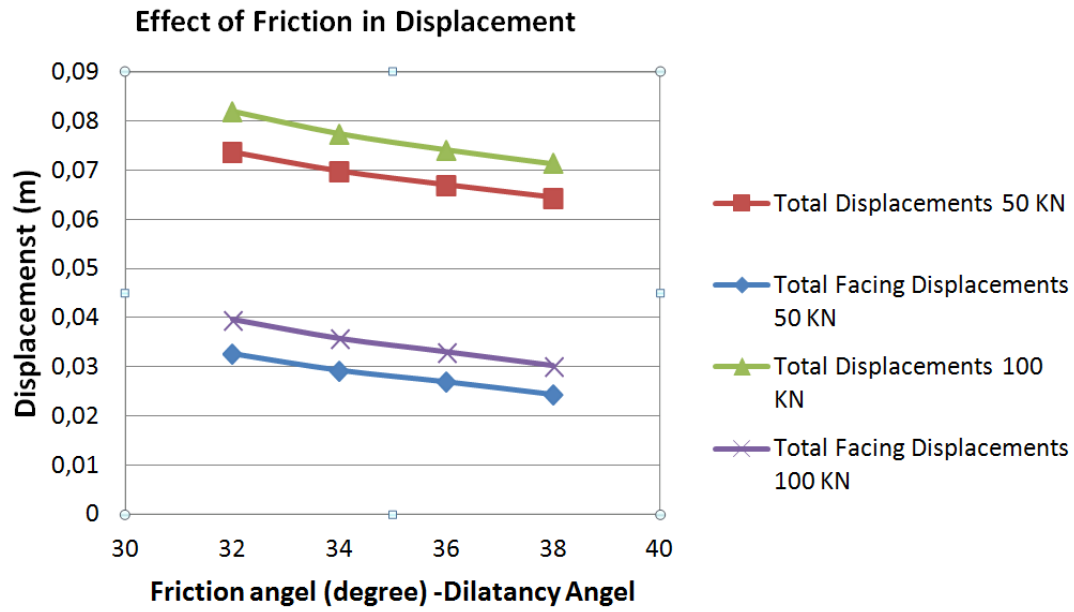


Figure 6.41 : Effect of friction angel (degree) -Dilatancy angel in MSE wall displacements.

7. RESULTS AND DISCUSSION

This research is focused on numerical modeling of Ankara-Sivas High speed railway project – The high speed train passes on a track above the wall with the maximum wall height of fourteen meters. This is the first time the Reinforced earth wall has been used for High-speed railway support in Turkey. All the design technology and data which has been used and expressed here exclusively belong to Reinforced Earth Company and copying any data without permission is strictly prohibited.

The sensitivity of the project leads to predict the wall deformations, by advance numerical modeling of reinforced earth wall for two different types of trains. In this project, the uniform distributed load is defined as 22.5 KN/m^2 for each line from the technical consulting of the project. Possibility of the two engines in one wagon is probable, therefore by doubling the uniform load, we can reach to load that contains engine load. The aim of this research is to study the deformation of the structure in the highest section of the project's wall by considering the heaviest train. To approach this purpose, the Cooper E90 diesel train is chosen, this train receives its name from the maximum axle load that applies to the rail of 40 tons. The axle loads for a Cooper E80 are illustrated in Figure 6.10 , the approximate resulted value of 100 Kpa surcharge under of rail ties for Cooper E90 is accepted.

The preliminary designs were prepared using the Valdez program, The Valdez program is specifically prepared to analysis the MSE wall for internal and external specified strip amount for cross section. This program is user friendly and provides two types of design cross-sections, rectangular and trapezoidal. Furthermore you can design Tired wall with various setbacks for wall steps. The design method of Valdez program is based on the “Limit State” design procedure which has just given the length of strips by applying defined safety factors. The program is not able to estimate the displacements in the walls because as previously mentioned, the designs were based on “Limit State” methods. “Limit State” is using the limits of the

materials in design but does not take into account the strains formed by stresses. Therefore a new method to calculate the displacements is needed.

Finite element method of analysis is capable of calculating the displacements in any part of the defined system. The finite element program Plaxis 2016 is chosen to calculate the deformation for facing panels and whole system safety.

The reinforced earth walls design was done according to “Limit State” method by Valdez program, and then the deformation in each part of wall was checked by Plaxis program.

In the evaluating of the design parameters the Rock lab program was used for evaluating the Basalt parameters for both Valdez and Plaxis programs. In addition the Slide 5.0 is used to calculate the stability and evaluate the safety factor of the project. Slide 5.0 uses the limit equilibrium analysis method, Simplified Bishop and simplified Janbu are considered in calculations.

By considering the geological geometry of the site, the sharp cuts which is formed by Basalt is seen. Therefore, it leads to prepare the preliminary designs in three individual alternatives.

The first case is the MSE wall with a rectangular cross-section. For the better understanding of the working system, for all cases the mentioned geometry is analyzed according to maximum and minimum material properties which are defined as selected backfill 1 and 2. In addition to effect of other backfill properties from other cases and parametric comparison with them, three selected backfills are taken from three articles which is the properties of selected backfill close to this case.

Under the railway's sleeper there will be 1.5 m Basalt fill with the slope angle of 33 degree is taken into account. Therefore by considering the slope height on top of the wall, it reaches to about 15.5 m.

The second case is the MSE wall with Trapezoidal cross section MSE wall. This type of reinforcement geometry is used because the increment happens in pull out capacity of the reinforcement from top to the bottom of the wall by increasing the height of the soil mass from the top of the wall. The length of strips which is needed at the bottom is lower than the top of the wall. It is crucial that this type of geometry should be recommended, when the foundation of the MSE structure is based on rock or competent foundation material.

The third system is known as TerraLink technology. This system is the combination of MSE wall with soil nail system. This type of MSE wall has no symmetric geometry. In lower part of the wall the distances beyond the wall and back of the wall is not enough to extend the strips. Therefore, in the parts where the ground elevation codes are close to facing panels, a huge amount of excavations will be needed to provide adequate distance to extend the strips. Therefore another system is required to avoid excessive costs.

In this project, two types of soil exists, the lower part of the wall which is coinciding with three panels from bottom of the wall, is stiff Basalt type1. The excavation in this part of the site is done by explosion. The second part that coincide with other strips is Basalt type two ,by considering the danger and economical consideration of excavation. For the Second part the shored Reinforced Earth[®] wall system which is called TerraLink[™] is recommended. TerraLink[™] is patented product of Reinforced earth company.

TerraLink[™] elements are similar to those used for Reinforced Earth[®] solution. These elements are placed between well compacted backfill layers. The main feature is that the reinforcements are linked to the back face nailing part of the existing structure or stiff blocks which enables combination or continuity of reinforcement between the narrow Reinforced Earth[®] wall and existing ground or structure.

Deformation analysis for MSE wall is done for three individual type geometry, the brief results of analysis for each geometries are listed below :

- 1) The first alternative is the MSE wall with a rectangular cross-section. The maximum value of total displacements in X direction for both 50 KN and 100 KN uniform distributed loads is selected as it is mentioned by Hossain et al. (2012) backfill material by 19cm and 21 cm. The maximum value of deformation in X direction for facing panels is evaluated as it is mentioned by Hossain et al. (2012) by 11cm and 13 cm. The minimum deformation is observed for selected backfill 1, which has the maximum friction angle between the selected backfill materials.
- 2) Another solution is the MSE wall with trapezoidal cross section. The maximum value of total displacements in X direction for both maximum and minimum uniform distributed loads is gained for selected backfill properties

by Hou (2009), with 27cm and 25 Cm. The minimum deformation is observed for selected backfill 1. The maximum value for total deformation in X direction for facing panels considering 50 KN and 100 KN loads is obtained by Hossain et al. (2012) material properties by 7,2 cm and 8,9 cm, as it can be seen, by increasing the load from 50 KN to 100 KN, the deformation increases approximately by the rate of 10% .The friction angle and elastic modulus have the main effect in minimizing the deformation in MSE walls.

- 3) The last alternative design is TerraLink™ technique. In this system, elements are similar to those used for Reinforced Earth® solution. This system is combination of MSE wall and soil-nail wall system. TerraLink™ solution for this case has two alternatives:

- i. TerraLink™ system with direct connection

In the alternative with direct connection, the facing elements are directly connected with joints to the shotcrete elements. Although in this system, the flexibility of system is limited more else not recommended from TAI, but the minimum deformation is gained in comparisom with the TerraLink™ technique without connection. The maximum total displacement in X-direction loads is obtained as 10.6 cm and 11.9 cm ,which is mentioned by Hossain et al. (2012). Besides the minimum total displacement in X direction for the heaviest train load was gained by selected backfill 1 by 3,2 cm. Then maximum deformation in facing panels is respectively reached by Hossain et al. (2012) and Hou (2009) in selected backfill properties, with the lowest Young's modulus and friction angle. The maximum facing panel deformation occurs in the heaviest train. The total settlement of a MSE walls according to BS 8006-1 (2010), are in the first and second categories. Therefore, there is not a significant problem in vertical displacement. The first case is The Terralink™ without direct connections.

- ii. TerraLink™ system without direct connection

The maximum total displacement in X direction for the minimum and maximum train loads belongs to soil parameters as it is taken by Hossain et al. (2012) with 19.9 cm and 20.9. In addition to the maximum deformation in facing panels are respectively reached by Hossain et al. (2012), it has the lowest friction angle among other selected backfills properties.

After proposing of three types of MSE geometry as an alternative method for the project, the rectangular shape MSE wall which is more preferable in design, is taken to parametric study.

In analysis of an elastic isotropic material, only two constants are required. The first one is Young's modulus and the second one is Poisson's ratio. If we assume that soils are isotropic elastic materials, by considering that, the strips and granular reinforced strips stick together and the lateral deformation is very limited and the fill is granular and a compacted material. Therefore, no plasticity mechanism happens, so we have an analytical tool to predict the soil's response under loading. By keeping the other soil properties constant and Young's modulus and Poisson's ratio variable, the results show that by increasing the load from 50 KN to 100 KN, no significant increment in deformations is observed. The most considerable change in deformation is assessed, by increasing the elastic modulus, the results show that the deformation is reduced by approximately 40%, for each 10000 KPa increments in selected backfill properties. In design of the MSE wall, selected backfill's cohesion is neglected, actually clay content of soil disturbs the wall function. By increasing the cohesion to four times, the maximum displacement of soil mass and facing panels are reduced by 1.5 cm, therefore the high content of clay in soil mass is not recommended. The effect of friction angle in total displacements in -X direction is reduced with increment of friction angle, by increasing the friction angle from 32 degree to 38 degrees, the maximum displacement for 50 KN load and 100 KN is reduced about 14%.

The safety factor achieved by Slide program which uses Limit equilibrium analysis in comparison with Plaxis software which is based on Finite element method, in simple geometry is approximately the same but in complex geometry like TerraLink™, it is more than two times conservative.



REFERENCES

- AASHTO** (2002). Standard Specifications for Highway Bridges, 17th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO** (2007). LRFD Bridge Design Specifications. 4th Edition, with 2008 and 2009, Interims, American Association of State Highway and Transportation Officials, Washington, D.C.
- Alhabshi, A.** (2006). Finite Element Based Design Procedures For Mse/Soil-Nail Hybrid Retaining Wall Systems, Doctor Of Philosophy, Graduate Faculty of Texas Tech University.
- Allen, T. M., Bathurst, R. J., Lee, W. F., Holtz, R. D. and Walters, D. L.** (2004) "A New Method for Prediction of Loads in Steel Reinforced Walls", ASCE Journal of Geotechnical and Geo-environmental Engineering, Vol. 130, No. 11, pp. 1109-1120.
- American Railway Engineering and Maintenance-of-Way Association.** (2007). Mechanically stabilized embankments. Section 7.
- Ankara-Sivas hızlı tren projesi.** (2015). Ozkar inşaat, plan view.
- Bishop, A. W.** (1955). "The use of the Slip Circle in the Stability Analysis of Slopes". Géotechnique. 5: 7. Doi:10.1680/geot.1955.5.1.7.
- British Standards Institution.** (2010). "Code of practice for strengthened/reinforced soil and other fills." BS 8006, Milton Keynes, U.K.
- Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G.** (1998). "Manual for Design and Construction Monitoring of Soil Nail Walls," Report FHWA-SA-96-69R, Federal Highway Administration, Washington, D.C. California, USA.
- CEBTP** (1975). Procès verbal d'essais – Mur expérimental de Triel sur Seine (Test report – Experimental wall at Triel sur Seine). Unpublished.
- Christopher, B. R., Gill, S. A., Juran, I., and Mitchell, J. K.** (1990). "Reinforced Soil Structures, Volume 1, Design and Construction Guidelines," Report No. FHWA-RD-89/043, Federal Highway Administration, Washington, DC.
- Clouterre Recommendations** (1991). Soil Nailing Recommendations 1991," English Translation, Presses de l'Ecole Nationale des Ponts et Chaussées, Paris, France.

- Collin, J. G.** (1986). Earth Wall Design, Doctor of Philosophy Dissertation, University of Berkeley, 419 p.
- Durgunoğlu H. T., Kaya T., Özbatur M.,** (2007)." Donatılı Zemin Duvar Yapılarının Depremde Davranışı" TMMOB Zemin Mekaniği ve Temel Mühendisliği 1. Özel Konulu Sempozyum, İstanbul.
- Ekli, O.** (2006). Donatılı Zemin Kenarayak Duvarlarının Nümerik Analizi, Master thesis bosphorus university civil engineering department.
- Elias, V. and Christopher, B. R.,** (1997). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines, Federal Highway Administration, FHWA SA-96-071, Washington, D.C., 371 p.
- Elias, V., and Juran, I.** (1991). "Soil Nailing for Stabilization of Highway Slopes and Excavations," Report No. FHWA-RD-89-198, Federal Highway Administration, Washington, DC.
- Feritag, N. & Brancaz, D. & Lucas, E.** (2011). Apromising Solution For The Development of Railways. GEORAIL 2011 International Symposium, 1-10.
- FHWA** (1996). U.S. Department of Transportation Federal Highway Administration. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. United State of America. National Highway Institute.
- FHWA** (2009). U.S. Department of Transportation Federal Highway Administration. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. United State of America. National Highway Institute.
- FHWA** (2015). U.S. Department of Transportation Federal Highway Administration. Chapter 3: Numerical Analysis. Effects of Geosynthetics Reinforcement Spacing on The Behavior of Mechanically Stabilized Earth Wall.. United States of America. National Highway Institute.
- Floss R. and Thamm B. R.** (1979). Field measurements of a Reinforced Earth retaining wall under static and dynamic loading. Colloque international sur le renforcement des sols : Terre Armée et autres techniques, Paris, 1979, Vol. III, pp. 183-188.
- Gassler, G.** (1988). Soil nailing-Theoretical Basis and Practical Design. PROCEEDINGS IN GEOTECHNICAL SYMPOSIUM THEORY AND PRACTICE OF EARTH REINFORCEMENT 1988, Volume - , Number -; Page(s) 283 To 288.
- Gässler, G., and Gudehus, G.** (1981). "Soil Nailing-Some Aspects of a New Technique," in Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3., Session 12, Stockholm, Sweden, 665-670.
- Hoek, E. & Torres, C.C. & Corkum, B.** (2002). Proc. NARMS-TAC Conference, Toronto, 2002, 1, 267-273.

- Hoek, E. and Brown, E. T.** (1980). *Underground Excavations in Rock*, London, Instn Min. Metall.
- Hossain, M. S., Kibria, G., Khan, M. S., Hossain, J., and Taufiq, T.** (2012). "Effects of backfill soil on excessivemovement ofMSE wall." *J. Perform.Constr. Facil.*, 10.1061/(ASCE)CF.1943-5509.0000281, 793–802.
- Janbu, Nilmar.** (1973). RC Hirschfeld and SJ Poulos, eds., "Slope stability computations", In *Embankment-dam Engineering*, Jon Wiley and Sons Inc., NY: 40P.
- Jewell, R. and Pedley, M.** (1992). "Analysis for Soil Reinforcement with Bending.
- Juran, I., Baudrand, G., Farrag, K., and Elias, V.** (1990). "Kinematical limit analysis for design of nailed structures," *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 116, No. 1, pp. 54-72.
- Karayolları Genel Müdürlüğü.** (2013). *Karayolları Genel Müdürlüğü Teknik Şartnamesi*.
- Kramer, S. L.** (1996). "Geotechnical Earthquake Engineering." Prentice-Hall, Inc., Upper Saddle River, New Jersey 07458, 1996, pp. 434-437.
- Kulhaway, F. H., and Mayne, F.** (1990). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.
- Lazarte, C. A., Elias, V., Espinoza, R. D., and Sabatini, P. J.** (2003). "Soil Nail Walls," *Geotechnical Engineering Circular No. 7*, Report No. FHWA-IF-03-017, Federal Highway Administration, Washington, DC.
- Liang R. Y.** (2004). *MSE Wall and Reinforcement Testing At Mus-16 Bridge Site*, September .
- Melo, C. & Sharma, S.** (2004). *Seismic Coefficients for Pseudostatic Slope Analysis*. 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 2004, Paper No. 369.
- Meyerhof, G. G.** (1956). "Penetration Tests and Bearing Capacity of Cohesionless Soils," *Journal of the Soil Mechanics and Foundations Division*, American Society of Civil Engineers, Vol. 82, No. SM1, Reston, VA, 1-19.
- Newmark.** (1965). "Effects of earthquakes on dams and embankments." *Géotechnique*, Vol. 15, No. 2, pp. 139-160.
- NFP-94270.** (2009). *Calcul Géotechnique – Ouvrages de Soutènement – Remblais Renforcés et Massifs en sol Cloué (Geotechnical Design - Retaining Structures – Reinforced fill and soil-nailed masses)*. AFNOR
- Damians., R. J. Bathurst., A. Josa., A. Lloret., and P. J. R. Albuquerque.** (2013). "Vertical-Facing Loads in Steel-Reinforced Soil Walls." *Journal of Geotechnical and Geoenvironmental Engineering* © ASCE/ september 2013.
- PLAXIS.** (2016). *Plaxis Material Models Manual*. Plaxis bv P.O. Box 572, 2600 AN DELFT, Netherlands.

- Poulos, H.G. & Davis, E.H.** (1980). Pile Foundation Analysis and Design. Rainbow Bridge Co, The University of Sydney.
- Reinforced Earth Company.** (1999). "Reinforced Earth Retaining Walls", TAI Group.
- Reinforced Earth Company.** (2009). "Reinforced Earth Retaining Walls", TAI Group.
- Reinforced Earth Company.** (2016). Ozkar inşaat railway report.
- Sankey, J. E. & Hutchinson, D.** (2011). Experience with Support of Railways Using Mechanically Stabilized Earth in the USA. GEORAIL 2011 International Symposium, 485-493.
- Schlosser, F.** (1983). "Analogies et différences dans le comportement et le calcul des ouvrages de soutènement en Terre Armée et par clouage des sols," (Trans: Similarities and differences in the behavior and design of retaining structures of reinforced earth and soil nailing), Annales de L'Institut Technique de Bâtiment et des Travaux Publics, No. 418, Series: Sols et Fondations, Paris, France.
- Schmertmann, J.H.** (1975). "Measurement of In-Situ Strength," in Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, Raleigh, NC, Vol. 2., 55-138.
- Seed HB.** (1979). "Considerations In the Earthquake-Resistant Design of Earth and Rockfill Dams." Géotechnique, 1979, Vol. 29, No. 3, pp. 215-263.
- HOU, S.E.** (2009). Deformation analysis of Reinforced Earth Wall Subjected to applied loads, Master thesis Universiti Teknologi Malaysia, civil engineering department.
- SLIDE 5.0.** (1997). Slide user Manual Book. 1991-1997 Silicon Graphics, Inc.
- Soyez L.** (2009). Contribution à l'étude du Comportement des Ouvrages de Soutènement en Sol Renforcé, Soumis à des Charges d'Exploitation Ferroviaire (Charges Dynamiques et Cycliques) (Contribution to the Study of the Behaviour of Reinforced Soil Retaining Structures, Subjected to Railway Service Loads (Dynamic and Cyclic Loads)). Doctoral Thesis for the Ecole Nationale des Ponts et Chaussées.
- Stocker, M. F., Korber, G. W., Gässler, G., and Gudehus, G.** (1979). "Soil Nailing," in International Conference on Soil Reinforcement I, Paris, France, Vol. 2, 469-474.
- Terre Armée Internationale.** (1987). The Design of Reinforced Earth Structures Supporting Railway Lines, Documentary File No. 4.
- Turkey Seismic Risk Map.** (2006). General Directorate of Disaster Affairs Earthquake Research Department.
- Wu P. and Brockbank W. J.** (2008). Design Considerations of Earth Reinforced Structures Using Inextensible Reinforcements in Heavy Load Surcharge Support Capacity. New Horizons in Earth Reinforcement.

CURRICULUM VITAE



Name Surname: Ehsan SEYEDTEHRANI

Place and Date of Birth: Iran-Orumiyeh – 30/08/1985

E-Mail: seyedtehrani@itu.edu.tr

EDUCATION:

B.Sc.: Civil Engineering, University of Mahabad, 2009.