Settlement Analysis of Back-to-Back Reinforced Retaining Walls

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Approval Sheet

This thesis entitled 'Settlement Analysis of Back-to-Back Reinforced Retaining Walls' by Akhila Palat is approved for the degree of Master of Technology from IIT Hyderabad.

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Dedicated to

My EP and Amma

Abstract

Mechanically Stabilized Earth (MSE) walls are one of the most common soil-retaining structures in the world. They are cost-effective soil-retaining structures that can tolerate large settlements compared to conventional gravity retaining walls. MSE walls are also an economical way to meet everyday earth retention needs for highway and bridge grade separations, railroads, commercial and residential developments. However, available literature and design guidelines for back-to-back MSE walls are limited. This study was conducted to investigate the effect of reinforced back-to-back MSE walls on wall settlements and facing deformations. Models of unreinforced and reinforced retaining walls were modeled and compared using commercially available finite element package PLAXIS 2D and a finite difference package FLAC 2D. Parametric studies were performed to determine the effect of types of facing, spacing between the reinforcements, axial stiffness of reinforcement, friction angles of backfill and foundation soil on foundation settlement, surface settlement, and horizontal displacement of the facing panel. The effects of varying spacing between reinforcements, and friction angle of backfill soil on the critical failure surface and the tension developed in the reinforcements were also studied.

Nomenclature

A	=	maximum ground acceleration coefficient
AASHTO	=	American Association of State Highway and Transportation Officials

Am	=	maximum wall acceleration coefficient at the centroid of the wall mass
B	=	width of the reinforcement
C	=	reinforcement effective unit perimeter
c	=	soil cohesion
c’	=	cohesion of the soil under effective stress conditions
d_{eq}	=	equivalent plate thickness
EA	=	axial stiffness
EI	=	flexural rigidity
e	=	eccentricity
F*	=	the pullout resistance (or friction-bearing-interaction) factor
FT	=	total earth pressure force
FS	=	overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads
FS_{MIN}	=	minimum factor of safety
FS_{PO}	=	factor of safety against pullout
f_{ms}	=	partial materials factor
f_s	=	partial factor against base sliding
f_p	=	partial factor for reinforcement pullout resistance
f_n	=	partial factor applied to economic ramifications of failure
H	=	vertical wall
I_{st}	=	influence factors (Poulos, 1967)
K_a	=	active lateral earth pressure coefficient
K_{af}	=	active lateral earth pressure coefficient of retained fill soil
L	=	total length of reinforcement
L_a	=	length of reinforcement in the active zone
L_e	=	embedment or adherence length in the resisting zone behind the failure surface
L_{aj}	=	length of the reinforcement beyond the line of maximum tension
M	=	mass of the active portion of the reinforced wall section assumed at a base width 0.5H
MSE	=	mechanically stabilized earth
MSEW	=	mechanically stabilized earth wall
n_c	=	global coarseness setting factor
P_{AE}	=	seismic thrust

P_{IR}	=	horizontal seismic inertia force
P_r	=	pullout resistance of the reinforcement per unit width
q_a	=	allowable bearing capacity
q_{ult}	=	ultimate bearing capacity
R_c	=	reinforcement coverage ratio b/S_h
R_h	=	horizontal factored disturbing force
R_v	=	vertical factored resultant force
SSR	=	Steady State Ratio
T_{max}	=	maximum reinforcement tension
T_{pj}	=	tensile force due to self weight and surcharge
T_{sj}	=	tensile force due to vertical strip loading
T_{fj}	=	tensile force due to horizontal shear
T_{cj}	=	tensile force due to cohesion
w_s	=	surcharge due to dead loads only
z	=	vertical depth
α	=	a scale effect correction factor to account for a non linear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data
α'_{bc}	=	adhesion coefficient between the soil and reinforcement
β	=	surcharge slope angle (MSEW)
δ	=	wall friction angle
γ_b	=	unit weight of the retained backfill
γ_f	=	unit weight of soil
φ	=	the peak friction angle of the soil
φ_p	=	peak angle of shearing resistance under effective stress conditions
θ	=	the face inclination from a horizontal
σ'_v	=	the effective vertical stress at the soil-reinforcement interfaces
σ_H	=	horizontal stress along the potential failure line
ν	=	poison's ratio

Contents

Chapter 1

1. Introduction

1.1 Overview

Mechanically Stabilized Earth Walls (MSEW) are cost-effective soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements (inclusions) in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. MSE Walls can be used to solve problems in locations of restricted Right-of-Way (ROW) and at marginal sites with difficult subsurface conditions and other environmental constraints. A typical cross section of MSE Wall is shown in Figure 1.1.

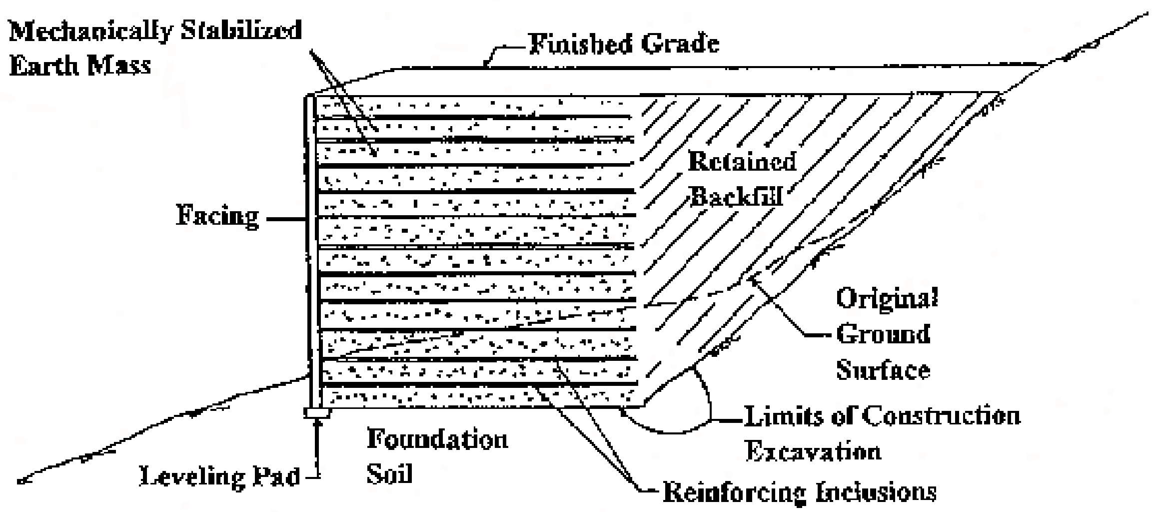


Fig 1.1: Principal Elements of a Mechanically Stabilized Earth mass [13]

AASHTO and BS codal guidelines are commonly used to perform external and internal stability checks and to determine the preliminary sizing of the reinforcement. Han and Leshchinsky (2008) conducted studies on the analysis of back-to-back Mechanically Stabilized Earth walls. Finite difference method based software- Fast Lagrangian Analysis of Continua (FLAC) - were used in their study. Parametric studies were carried out by varying the spacing between reinforcements, stiffness of the reinforcement, etc.

As per the traditional methods, the design of retaining wall structures is mainly based on bearing capacity criterion. But recent studies in the literature show that inclusion of reinforcements leads to greater performance and design based on bearing capacity may lead to a highly over conservative design. In this study, design of Mechanically Stabilized Earth (MSE) walls based on settlement criterion rather than depending on bearing capacity criterion is proposed. The effects of various stiffness values of reinforcement and soil properties on the settlement behavior of MSE structures were also evaluated. The numerical modeling was attempted in both finite element and finite difference based commercial softwares.

1.2 Objectives of the study

- Modeling Unreinforced and Reinforced Back-to-Back Retaining Walls using PLAXIS 2D- A Finite Element Software
- Modeling Unconnected and Connected Back-to-Back Reinforced Retaining Walls using FLAC 2D- A Finite Difference Software

1.3 Organisation of the study

Chapter 2 deals with the literature review. It mainly includes a review on the existing codal recommendations to model Reinforced Retaining Walls. Two codes that are popularly adopted - Federal Highway Administration (FHWA) Code and British Standard (BS) Code- are reviewed. External Stability Analysis and Internal Stability checks for reinforced retaining walls are also discussed. A brief study of the research studies conducted to model and analyze Reinforced Retaining Wall is included in this chapter.

Chapter 3 provides details on modeling Unreinforced and Reinforced Back-to-Back Retaining wall using a Finite Element Package PLAXIS 2D. Details on results validated using Davis and Poulos Elastic solution for unreinforced retaining wall are also included. The effects of axial stiffness of reinforcement, vertical spacing between reinforcement, deformation modulus of foundation soil, and backfill friction angle on the foundation settlement, surface settlement and facing panel displacement are quantified and discussed.

Chapter 4 deals with modeling connected and unconnected Back-to-Back Reinforced Retaining Wall using a finite difference package FLAC-2D. Prior to the analysis, a MSE wall was designed that meets all the stability check criteria. For the designed MSE wall, parametric Studies were then performed for different properties like type of facing, spacing between reinforcements, stiffness of reinforcement, internal friction angle of foundation and backfill.

Chapter 5 comprises of the results and discussions based on the above studies.

Chapter 2

2. Literature review

2.1 Introduction

Retaining structures are essential elements of every highway design. Retaining structures are used not only for bridge abutments and wing walls, but also for slope stabilization and to minimize the right-of-way for embankments. Reinforcements are inclusions provided within the engineering fill to absorb tensile loads or shear stresses, thereby reducing the loads which might otherwise cause the soil to fail in shear or by excessive deformation. Reinforcement may also be used to improve the performance of weak soils supporting embankments or other resilient structures.

2.2 Applications of MSE walls

Mechanically stabilized earth (MSE) walls have replaced traditional concrete retaining walls over the last 20 years. MSE walls have many advantages when compared to conventional reinforced concrete walls. Ease of installation along with relatively rapid construction of MSE walls are among the most important factors that favor their choice instead of traditional concrete walls. MSE Walls offer sufficient technical and cost advantages over conventional reinforced concrete retaining structures at sites with poor foundation soils. In such cases, the cost of improving or modifying the foundation soil by various methods, like piles or pile caps can be eliminated, which in turn results in cost savings of greater than 50% on completed projects. Reinforced MSE walls also find applications in many other areas:

- Temporary structures, which have been especially cost-effective for temporary detours necessary for highway reconstruction projects.
- Soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks.
- Construction sites with poor subsoil conditions (cost of ground modifications can be eliminated)

- Dams and seawalls, including increasing the height of existing dams.
- Sites prone to seismic activity

2.3 Advantages of MSE walls

The following advantages of MSE walls necessitates

a) Simple and rapid construction procedures that do not require large construction equipment.
b) Availability of more land for construction as very steep slopes can be used.
c) Elimination of the need for experienced craftsmen with special skills for construction.
d) Elimination of elaborate site preparation compared to other alternatives.
e) Elimination of the need for wall finishing.
f) Elimination of need for large space in front of the structure for construction operations. Hence it can be built in confined areas (where a concrete wall is almost impossible to be constructed) and thereby reducing right-of-way acquisition.
g) Elimination of the need for rigid, unyielding foundation support because MSE structures are tolerant to deformations.
h) Cost effective and have high seismic load resistance.
i) Construction of wall heights in excess of 25 m (80 ft).
j) Feasibility of various shapes and forms for facing wall.

2.4 Codal recommendations

The following two codes are mainly considered in the design of Reinforced Soil Retaining Walls:

1. FHWA Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines, FHWA-NHI-0043(2001)
2. BS 8006 Strengthened/Reinforced Soils and Other Fills, British Code of Practice (1995)

2.5 Forces acting on the wall

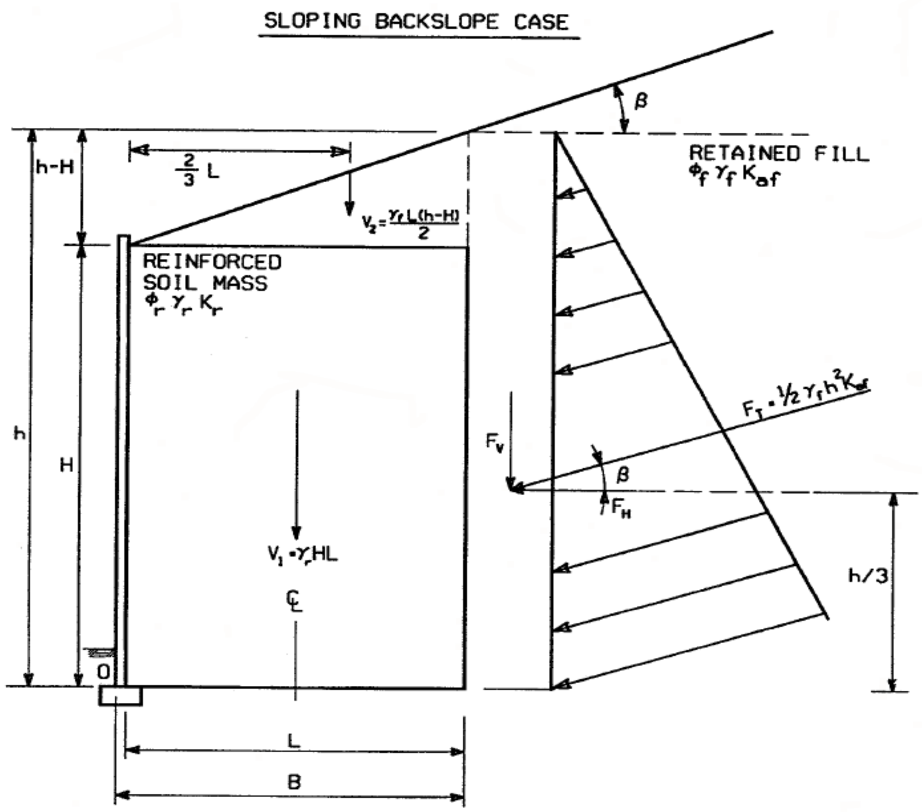


Fig 2.1: Representation of the forces acting on a retaining wall [13]

Major forces acting on the retaining wall includes (Fig 2.1):

a) Lateral Forces

   Earth pressure acting on the wall can be due to backfill and surcharge. The lateral earth pressure acting on the retaining wall is highly influenced by many factors like pore water pressure and shear strength of the soil

b) Live Loads due to point loads or uniform surcharge

c) Dead Load surcharges

d) Seismic Loads

2.6 Preliminary sizing of the reinforcement

As per the FHWA Code, the preliminary length of reinforcement is taken as equal to 0.7H or 2.5m, whichever is larger. In case of structures with sloping surcharge fills or other concentrated loads, the reinforcement length adopted is 0.8H or 1.1H, whichever is larger. H denotes the height of the wall.

As per the BS Codal provisions, the length of reinforcement to be adopted is highly dependent on the purpose for which the MSE wall is served for. Table 2.1 gives the length of reinforcements for various applications.

Table 2.1: Length of Reinforcements to be provided as per BS Code

Function	**Length of reinforcement**
Walls with normal retaining function	0.7H with minimum length of 3m
Bridge Abutments	Greater of (0.6H+2) or 7m
Trapezoidal Walls and Abutments	0.7 H in top half 0.4H in bottom half
Stepped Walls and Abutments	0.7 H
Walls subject to low thrust	0.6H with minimum length of 3m
Low Height Walls (< 1.5m)	Subject to particular considerations

2.7 External Stability Check

External Stability Analysis of MSE Walls includes global stability of the structure, bearing capacity of foundation soils, and settlement analysis of the proposed structure. In the external stability calculations, the reinforced section is treated as a composite homogeneous soil mass and the stability is evaluated according to the conventional failure modes for gravity type wall systems. The external stability calculations as per the FHWA and BS Codes are illustrated below:

2.7.1 FHWA Code

FHWA Code offers mainly five checks for external stability which includes:

a.) Sliding

b.) Overturning

c.) Bearing Capacity

d.) Overall Stability

e.) Settlement Estimate

2.7.1.1 Check for Sliding

A minimum Factor of Safety of 1.5 is required to resist sliding. Factor of Safety against sliding is calculated as (Eq. 2.1):

$$FS_{Sliding} = \frac{\sum Horizontal\ Resisting\ Forces}{\sum Horizontal\ Driving\ Forces} \tag{2.1}$$

2.7.1.2 Check for eccentricity (Overturning)

Eccentricity in a retaining wall is calculated using the equation below (Eq. 2.2):

$$e = \frac{F_T cos\beta\ \frac{h}{3} - F_T sin\beta\ \frac{L}{2} - V_2 \frac{L}{6}}{V_1 + V_2 + F_T cos\beta} \tag{2.2}$$

where, F_T denotes the total earth pressure force, V_1 denotes the weight of reinforced soil mass, V_2 denotes the weight of the sloping fill, and β denotes the surcharge slope angle (Fig 2.1).

As per the codal criteria, it must be ensured that eccentricity is less than L/6 in soils, and is less than L/4 in rocks. If eccentricity is greater than the above mentioned values, then a reinforcement of greater length is to be provided.

2.7.1.3 Check for Bearing Capacity

Generally two checks are made for the bearing capacity- Check for General Shear and Check for Local Shear.

Check for general shear:

The vertical stress, σ_v, at the base of the foundation is calculated using Eq.2.3:

$$\sigma_V = \frac{V_1 + V_2 + F_T\ sin\beta}{L - 2e} \tag{2.3}$$

The ultimate bearing capacity, q_{ult}, is calculated using classical bearing capacity equation (Eq. 2.4):

$$q_{ult} = c_f N_c + 0.5 \Upsilon_f L N_\Upsilon \tag{2.4}$$

It must be ensured that

$$\sigma_v \leq q_a = q_{ult}/FOS \tag{2.5}$$

where, q_a is the allowable load carrying capacity of the foundation soil and FOS is the factor of safety. FOS equal to 2.5 is commonly adopted in Equation 2.5. The vertical stress due to reinforced soil is decreased or the load carrying capacity of the foundation soil is increased by increasing the length of the reinforced zone.

Check for local shear

Local shear failure is characterized by "squeezing" of the foundation soil when soft/loose soils exist below the wall. To prevent large horizontal movements of the structure on weak cohesive soils, the following criterion is to be satisfied (Eq. 2.6).

$$\Upsilon H \leq 3c \tag{2.6}$$

where, ϒ is the unit weight of soil, H represents the wall height and c is cohesion of the soil.

2.7.1.4 Check for overall stability

A minimum Factor of Safety equal to 1.3 is to be provided to ensure the overall stability of the structure. It is mainly determined using the rotational or wedge analysis. Here the reinforced soil mass is considered as a rigid body, and failure surfaces that are completely outside a reinforced mass are only considered. But in the case of complex conditions such as changes in reinforced soil types or reinforcement lengths, high surcharge loads, sloping faced structures, significant slopes at the toe or above the wall, or stacked structures, compound failures must be considered. If the minimum Factor of Safety is not attained, it is essential to increase the reinforcement length or to improve the foundation soil.

2.7.1.5 Settlement estimate

Conventional settlement analysis must be carried out to ensure that the settlements are less than the performance requirements of the project (FHWA, Soils and Foundations Reference Manual, 2006). The differential settlement in a MSE wall structure must always be limited to 1/100. If this value is exceeded, either slip joints are to be provided which allow for independent vertical movement of precise concrete panels or consideration must be given to suitable ground improvement techniques.

2.7.2 BS Code

2.7.2.1 Check for bearing capacity

The imposed bearing pressure must satisfy the criterion that (Eq.2.7):

$$q_r \leq \frac{q_{ult}}{f_{ms}} + \Upsilon D_m \tag{2.7}$$

where, q_r is the factored bearing pressure acting on the base of the wall, q_{ult} is the ultimate bearing capacity of foundation soil, ϒ is the foundation soil density, D_m is the wall embedment depth, and f_{ms} is the partial material factor of safety.

2.7.2.2 Check for sliding

It must be ensured that the forward sliding of the structure between reinforced fill and subsoil should be restricted. Four cases are mainly taken into consideration (Table 2.2):

Table 2.2: Criteria for stability against sliding

For long-term stability	For short-term stability
With soil to soil contact at the base : $f_s R_h \leq R_v \frac{\tan \varphi'_p}{f_{ms}} + \frac{c'}{f_{ms}} L$	With soil to soil contact at the base : $f_s R_h \leq \frac{c_u}{f_{ms}} L$
With reinforcement to soil contact at the base : $f_s R_h \leq R_v \frac{\alpha' \tan \varphi'_p}{f_{ms}} + \frac{\alpha_{bc}' c'}{f_{ms}} L$	With reinforcement to soil contact at the base: $f_s R_h \leq \alpha'_{bc} \frac{c_u}{f_{ms}} L$

where, R_h is the horizontal factored disturbing force, R_v is the vertical factored resultant force, φ_p is the peak angle of shearing resistance under effective stress conditions, c' is the cohesion of the soil under effective stress conditions; c_u is the undrained shear strength of the soil; L is the effective base width for sliding, f_{ms} is the partial materials factor, f_s is the partial factor against base sliding, α' is the interaction coefficient, and α_{bc}' is the adhesion coefficient.

2.7.2.3 Settlement estimate

The total settlement of a reinforced soil structure comprises of the settlement of the foundation soil due to the overburden pressure imposed by reinforced soil structure, internal settlement of the reinforced soil fill which depends on the nature of the fill, and compaction and vertical pressure within the fill. Special care must also be taken to ensure that the differential settlement is within the tolerable limits.

2.8. Internal Stability Check

Internal Stability of a reinforced soil mass is mainly governed by the interaction between soil and reinforcement which occurs by friction or adhesion. Internal stability evaluations basically determines the length and spacing of reinforcement required in the soil mass as it is assumed that the deformations are mainly controlled by the reinforcement rather than the total mass. In addition, it covers all areas relating to the internal behavioral mechanisms, consideration of stress within the structure, arrangement and behavior of reinforcements and backfill properties.

2.8.1 FHWA Code

As per the FHWA Code, internal failure can occur in two ways:

a.) Failure by elongation or breakage of reinforcement

When the tensile forces in the inclusions become higher than the tensile strength of the reinforcement, the inclusions elongate excessively or break leading to large movements and collapse of the structure.

b.) Failure by pullout

When the tensile forces becomes larger than the pullout resistance (force required to pull the reinforcement out of the soil mass), it increases the shear stress in the surrounding soil leading to large movements and possible collapse of the structure.

2.8.1.1 Internal stability with respect to breakage

The maximum tension in each reinforcement layer is calculated as (Eq. 2.8):

$$T_{max} = \sigma_H S_V \tag{2.8}$$

where, σ_H is the horizontal stress along the potential failure line, and S_v is the vertical spacing.

In order to ensure safety against elongation or breakage of reinforcement, the following criterion must be satisfied (Eq. 2.9).

$$T_a \geq \frac{T_{max}}{R_c} \tag{2.9}$$

where, T_a is the allowable tension per unit width of the reinforcement, and R_c is the coverage ratio defined as $R_c = b/S_h$, where b is the gross width of the reinforcement, and S_h is the centre-to-centre horizontal spacing of reinforcement.

2.8.1.2 Internal stability with respect to pullout

The following criterion must be satisfied in order to ensure stability with respect to pullout of reinforcement (Eq. 2.10).

$$T_{max} \leq \frac{1}{FS_{PO}} F^* \Upsilon Z_p L_e C R_C \alpha \tag{2.10}$$

where, C is the reinforcement effective unit perimeter and is equal to 2 (for strip, grid and sheet- type reinforcement), α is the scale correction factor which depends on the compacted granular backfill, length, and extensibility of reinforcement, F* is the pullout resistance factor, Υz_p is the overburden pressure, and L_e is the length of reinforcement embedded in the resistance zone.

The above equation is also useful in determining the length of reinforcement in the embedded zone, i.e.

$$L_e \geq \frac{1.5\, T_{max}}{C F^* \Upsilon Z_p R_c \alpha} \geq 1m \tag{2.11}$$

If the above criteria are not satisfied:

i.) Reinforcement length has to be increased

ii.) Reinforcement with a greater pullout resistance per unit width must be used

iii.) Vertical Spacing between the reinforcements may be reduced which would reduce $T_{max.}$

2.8.2 BS Code

As per BS Code, internal stability of reinforcements can be calculated by using mainly the following three methods:

i.) Anchored Earth method

ii.) Tie Back Wedge method

iii.) Coherent Gravity method

The steps involved to check the internal stability of reinforcement are schematically given in Figure 2.2.

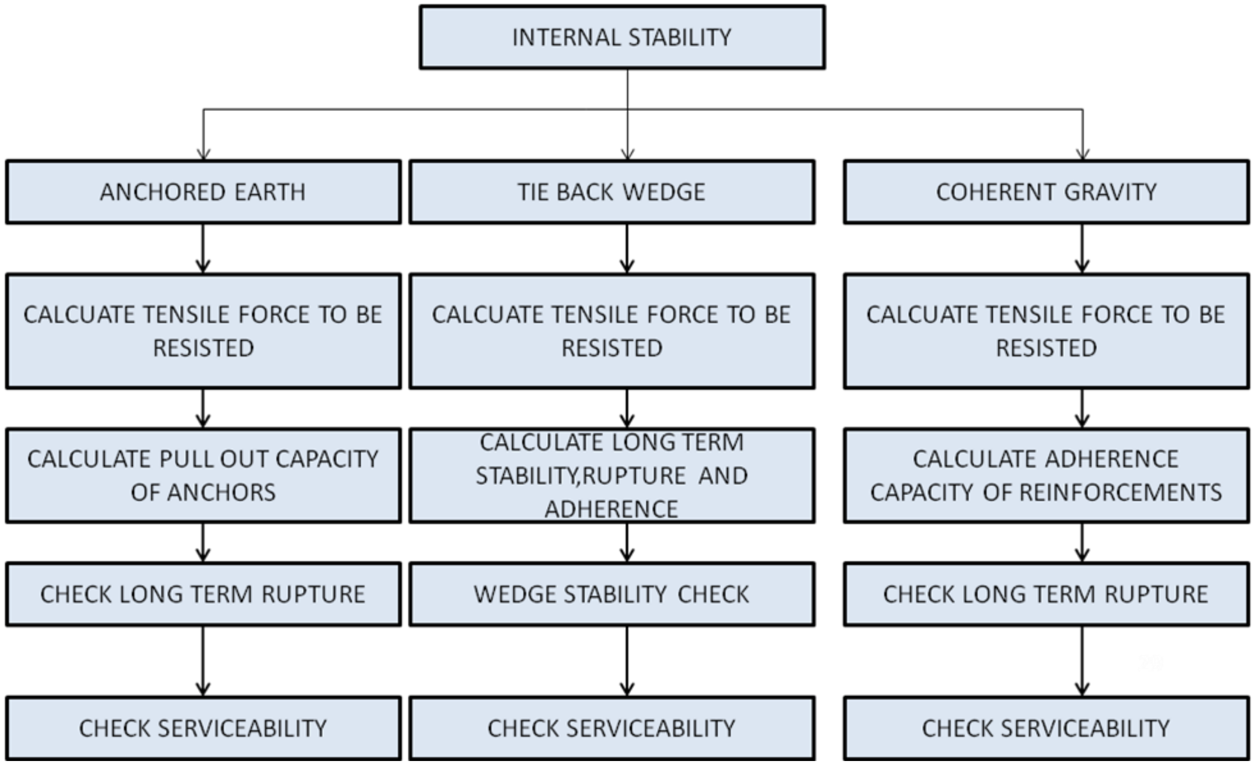


Fig 2.2: Schematic of Internal Stability Analysis Procedure followed in BS Code

2.8.2.1 Anchored earth method

Tensile force to be resisted by each layer is calculated as (Eq. 2.12):

$$T_j = T_{pj} + T_{sj} + T_{fj} - T_{cj} \tag{2.12}$$

where, T_{pj} denotes the tensile force due to self weight and surcharge, T_{sj} due to vertical strip loading, T_{fj} due to horizontal shear, and T_{cj} due to cohesion.

It must be ensured that the pullout capacity of the anchor reinforcing elements (after applying a partial factor for reinforcement pullout resistance and economical ramifications of failure) is greater than the tensile force of the reinforcement at that particular level (Eq.2.13):

$$\frac{P_{uj}}{f_p f_n} \geq T_j \tag{2.13}$$

where, P_{uj} is the ultimate pullout resistance of the anchor, f_p is partial factor for reinforcement pullout resistance, and f_n is partial factor applied to economic ramifications of failure.

The resistance of the reinforcing element should also be checked against rupture and adherence failures. Finally, it should be ensured that the post construction internal creep strain of polymeric reinforcement is well within the limits.

2.8.2.2 Tie back wedge method

The tensile force to be resisted by each layer of reinforcement is calculated using the above equation (Eq.2.12). The resistance of the reinforcing element should also be checked against rupture and adherence failures. In addition, the stability of the wedge is also checked to ensure that resistance acting on the potential failure plane in conjunction with the tensile resistance/bond of the group of reinforcements or anchors embedded in the fill beyond the plane are able to resist the applied loads tending to cause movement. For that, the following condition should be checked (Eq. 2.14):

$$\sum_{j=1}^{m}\left[\frac{P_j L_{ej}}{f_p f_n}\left\{\mu f_{fs}\Upsilon h_j + \mu f_f w_s + \frac{\alpha'_{bc} c}{f_{ms}}\right\}\right] \geq T \tag{2.14}$$

where, P_j is total horizontal width of top and bottom faces of the reinforcing element per meter run, T_{dj} design strength of reinforcement at the j^{th} level, L_{ej} is the length of the reinforcement in the resistant zone outside the failure edge, w_s surcharge due to dead loads only, f_p partial factor for reinforcement pull out resistance, f_n partial factor applied to economic ramifications of failure, α'_{bc} adhesion coefficient between the soil and reinforcement, c' cohesion of the soil measured under effective stress conditions, and f_{ms} is the partial material factor applied to c'. In addition, serviceability condition should also be checked for.

2.8.2.3 Coherent gravity method

The tensile force to be resisted by each layer is calculated using the above formula. The adherence capacity of each layer of reinforcement must satisfy the criterion (Eq. 2.14):

$$T_j \leq \frac{2B\mu}{f_p f_n}\int_{L-L_{aj}}^{L} f_{fs}\sigma_v(x)dx \tag{2.15}$$

where, f_p is the partial factor for reinforcement pull out resistance, B is width of the reinforcement, L total length of the reinforcement, L_{aj} length of the reinforcement beyond the line of maximum tension, μ coefficient of friction, $\sigma_v(x)$ is vertical stress along length x of the reinforcement, f_n is the partial factor for economic ramifications of failure, and f_{fs} partial load factor. Long-term rupture and serviceability conditions should also be checked for.

2.9 Effect of seismic loading

FHWA Code provides provisions to account for the seismic loads acting on the structure. Two cases are generally taken into consideration.

2.9.1 Case 1: Level backfill case

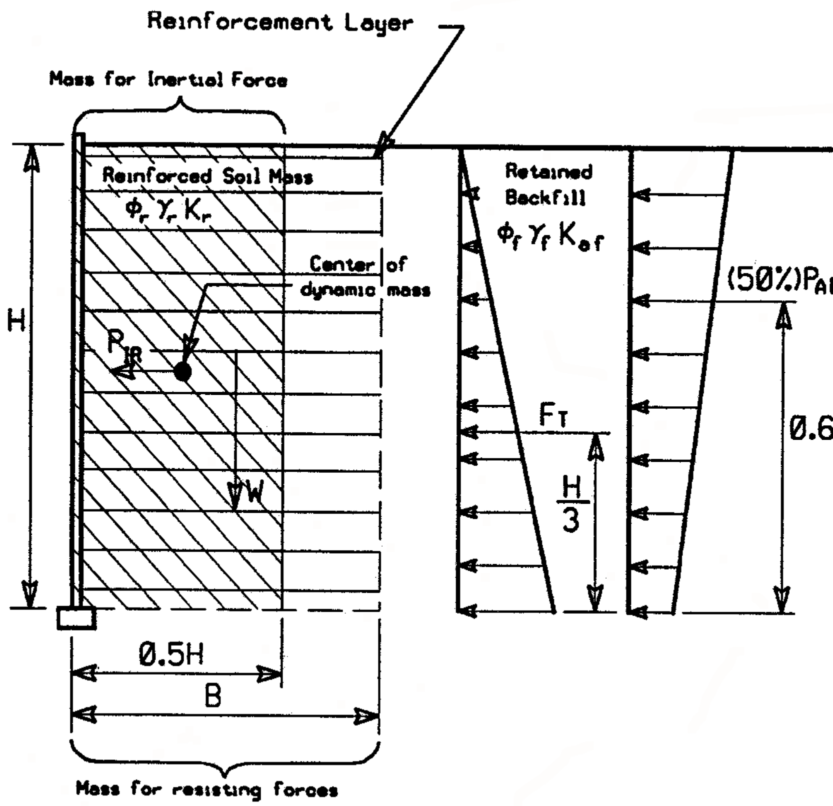


Fig 2.3: Effect of seismic loading on a level backfill case

In addition to the static thrust, a dynamic thrust (P_{AE}) is exerted by the retained backfill on MSE wall (Fig. 2.3) which is calculated using the equation (Eq. 2.15)

$$P_{AE} = 0.375 A_m \Upsilon_f H^2 \tag{2.16}$$

where, A_m is the maximum wall acceleration coefficient, and Υ_f is the unit weight of reinforced soil mass. Reinforced soil mass is also subjected to a horizontal inertia force which is given by (Eq. 2.16)

$$P_{IR} = M A_m \tag{2.17}$$

where, M is the mass of the active portion of the reinforced wall section assumed at a base width 0.5H.

2.9.2 Case 2: Sloping backfill case

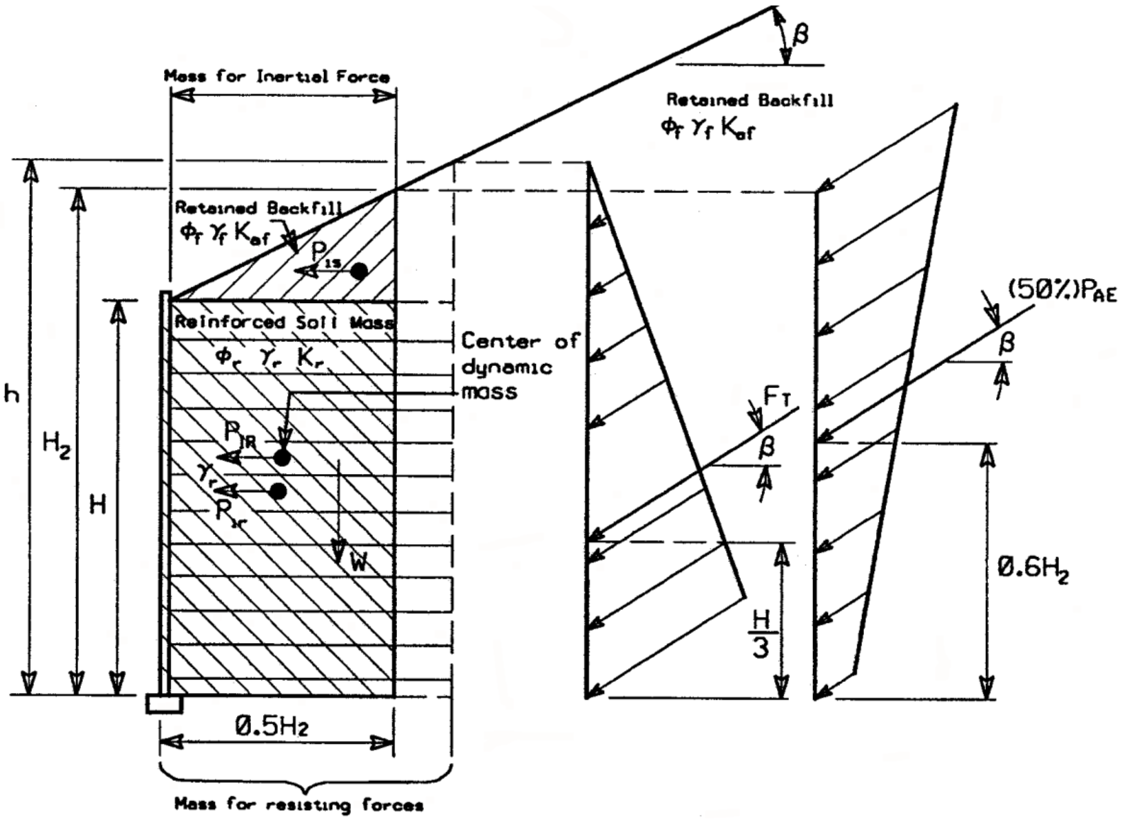


Fig 2.4: Effect of seismic loading on a sloping backfill case

The total inertia force acting on the reinforced soil mass (Fig 2.4) is calculated as the sum of the inertial force caused by the acceleration of the reinforced backfill and by the acceleration of sloping surcharge (Eqs. 2.17, 2.18, 2.19).

$$P_{IR} = P_{ir} + P_{is} \tag{2.18}$$

$$P_{ir} = 0.5 A_m \Upsilon_f H_2 H \tag{2.19}$$

$$P_{is} = 0.125 A_m \Upsilon_f H_2^2 \tan\beta \tag{2.20}$$

The dynamic horizontal thrust (P_{AE}) exerted by retained fill on MSE wall is calculated as

$$P_{AE} = 0.5 \Upsilon_f H_2^2 \Delta K_{AE} \tag{2.21}$$

where, k_{AE} is the total seismic earth pressure coefficient calculated based on Mononobe-Okabe general expression.

In this procedure, 50% of the seismic thrust (P_{AE}) and full inertial force (P_{IE}) are added to the static forces to get the total force acting on the structure. Checks are then made for sliding stability, eccentricity and bearing capacity in the same manner as for non-seismic loading condition. It should be ensured that that the computed factors are greater than or equal to 75% of minimum static safety factors and that the eccentricity falls within L/3 for both soil and rock.

2.10 Studies on the behavior of Back-to-Back Mechanically Stabilized Walls

Han and Leshchinsky (2008) conducted studies on the behavior of back-to-back MSE Walls. In their study, numerical methods based on limit equilibrium method (Simplified Bishop's Method) were adopted to investigate the effect of width to height ratio and backfill friction angle on settlements, critical slip surface, and tensile forces mobilized in the reinforcement. In addition to this, the effect of connecting reinforcements in the middle was also investigated. FLAC 2D software was used to develop the model. The geometric and material properties of the model are given in Fig 2.5.

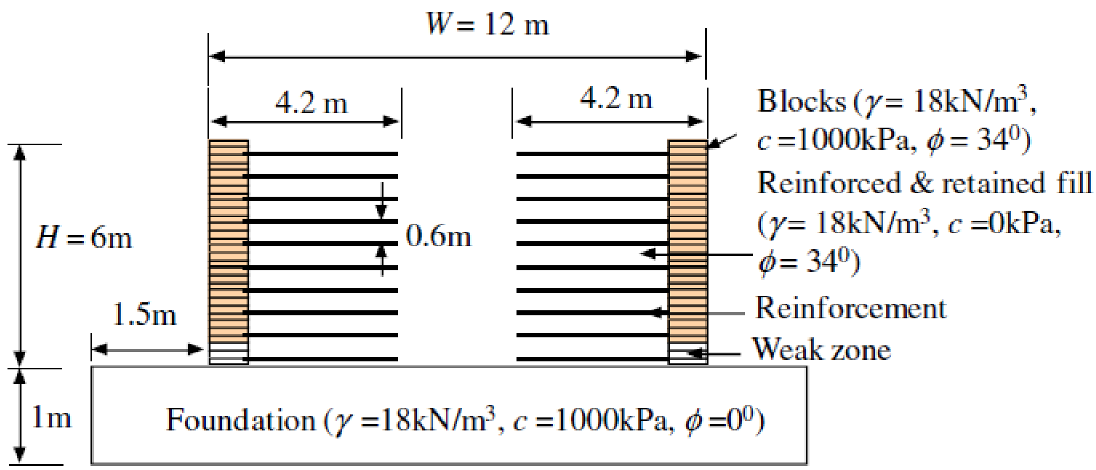


Fig 2.5: Model developed in FLAC 2D (Han and Leshchinsky, 2008)

The height of MSE wall was fixed as 6m. Mohr-Coulomb failure criterion was used to model the interface between facing blocks, the reinforced soil, retained soil, and the foundation soil. Reinforcements were modeled as cable elements. Since FHWA guidelines specify the minimum length of reinforcements to be provided in MSE walls as 0.7 times the height of retaining structure, the length of reinforcement equal to 4.2m was adopted. A weak zone of dimensions equal to 0.3m wide and 0.4m high was also provided at the base of the facing to ensure that the critical surface passes through the toe. A parametric study was carried out by varying the W/H ratio (equal to 2.0, 1.4 and 3.0), and angle of shearing resistance of backfill friction angle (equal to 25° and 34°). The effects of these parameters on the shape and location of critical failure surfaces, distribution of maximum tension with height, and lateral earth pressure behind the reinforced zone were studied.

Viswanadham and Katkar (2011) conducted similar studies on the back-to-back MSE walls using a finite element package - PLAXIS 2D. Geometric and material properties were adopted as in the previous study (Fig 2.6). A 15-noded triangular element was used to model the soil elements, and 6-noded triangular element was used to model the reinforcements. The average size of the mesh was equal to 0.153m. The axial stiffness of reinforcement was taken as 60 kN/m. Vertical fixity was provided to the horizontal surface of the foundation, and horizontal fixity to the vertical face of foundation. An interaction coefficient value, R_{inter}, equal to 0.97 was adopted for the backfill and geosynthetic interface, and foundation soil and geosynthetic interface.

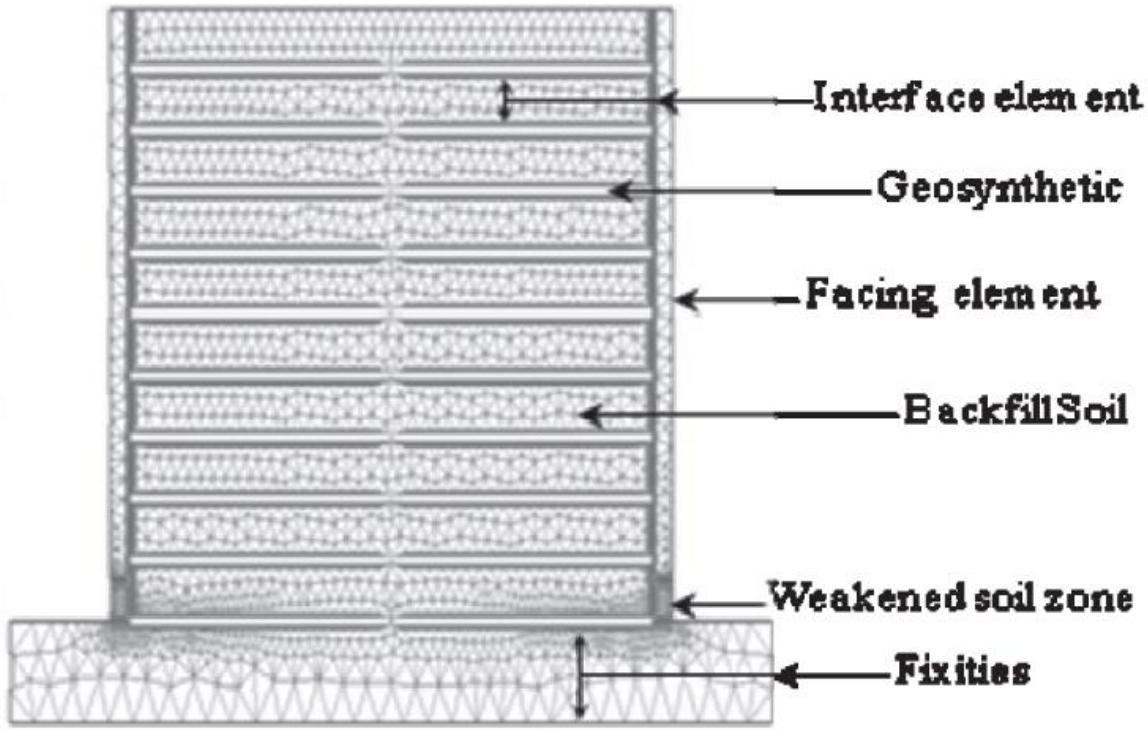


Fig 2.6: Model developed in PLAXIS 2D (Viswanadham and Katkar, 2011)

Checks were performed to ensure that no plastic stress points were generated at the end of initial stress generation stage. Eight cases were mainly considered with various W/H ratios and two different friction angles equal to 25^o and 34^o. The effect of these parameters on displacement profile, the maximum tension variation in geosynthetics, and displacement of the facing element were studied.

2.10.1 Conclusions based on the above study

The shape and location of the critical slip surfaces of back-to-back walls were determined based on the contours of shear strain rate in FLAC 2D and based on displacement contours in PLAXIS 2D. When the W/H ratio of the wall is increased, the critical surfaces from one wall do not enter into the other, and the two walls behave independently. But when the W/H ratio is reduced to 1.4, the reinforcements are almost connected and the failure surface from one reinforced zone will interact with that of the other reinforced zone. In addition, the interaction of the critical failure surface is found to be significant for a low friction angle of the backfill (φ equal to 25^o). Similar trend is also observed in case of settlements of MSE walls, i.e., when the distance between walls reduces, the resulting settlements decrease. The displacements are found to be minimum for the connected case.

By analyzing the variation of maximum tension in the reinforcement along the height of the wall, it is indicated that the maximum tension is observed for a backfill with low friction angle compared to that of a backfill with high friction angle. But, the mobilized geosynthetic forces do not vary much by changing the distance between the walls. However, tensile forces are less in the connected case than the unconnected case because pullout from the middle of the model is impossible.

As per the FHWA guidelines, the lateral earth pressure for external analysis can be ignored when D = 0 where D is the distance between the back-to-back walls. Hence, no active thrust is expected to be developed in the case where W/H = 1.4. But as per the above numerical study, it is indicated that the lateral earth pressure exists behind the reinforced zone even for W/H = 1.4 (i.e. no retained fill). Hence, designing the MSE Walls based on FHWA guidelines for active earth thrust would lead to an unsafe design. Lateral earth pressure increases with an increase in W/H ratio, and decreases with an increase in the soil backfill friction angle.

When the displacement of the facing element was taken into consideration for backfill soil with $\varphi = 25^{o}$, bulging of the wall occurred at the mid height of the wall. While for $\varphi = 34^{o}$, the displacement was uniform. Hence it emphasizes the importance of using backfills with higher friction angle. Displacement was also observed to be less for connected case, and it increases with an increase in the distance between walls.

2.11 Studies on the effect of foundation soil properties on reinforced soil retaining wall

Damians and Bathurst (2013) conducted studies to determine the effect of varying the foundation soil stiffness on settlement in steel reinforced soil wall systems constructed with precast concrete panel. Analysis was performed using Finite element package- PLAXIS 2D. Beam elements were used to model concrete panels and bearing pad joints. Geogrid element type available as structural element in PLAXIS was used to model reinforcement elements. They possess only axial stiffness and can transmit load to the surrounding soil through interface shear. Interface elements were used to model strength and stiffness between soil and reinforcement elements and between the soil and concrete facing panels. Studies indicated that the settlement at the base of the wall is largely influenced by the stiffness of the foundation soil. When the deformation modulus of the foundation soil was 10MPa, the settlement at the toe was found to be approximately 300mm and for an increase in the deformation modulus of foundation soil to 1000MPa, the settlement got reduced to approximately 3mm.

Bilgin and Mansour (2014) performed parametric study to investigate the effect of unit weight and friction angle of foundation soil on the minimum length of reinforcements required in the design. It was determined that an increase in the unit weight of foundation soil affects the bearing capacity mode and has no effect on the other failure modes. But a change in the friction angle of foundation soil affects sliding as well as the bearing capacity mode, i.e., as the friction angle of the foundation soil increases, the minimum reinforcement length required decreases. For the case when foundation soil is weak, longer reinforcements are required to distribute wall pressures to a larger area. It was also determined that the pullout, eccentricity and overturning failure modes are not affected by varying the friction angle of the foundation soil.

2.12 Studies on the effect of backfill material properties on reinforced soil walls

Hatami and Bathurst (2005) investigated the influence of backfill type and material properties on the performance of reinforced soil segmental retaining walls under working stress conditions. A numerical model was used and 6m high segmental wall was considered in the study with the facing column modeled as 40 rows of solid masonry concrete blocks. The properties of sand backfill were selected on the basis of triaxial tests conducted for the same, and reinforcements were modeled as two-noded, elasto-plastic cable elements. Results indicated that the facing deflections diminish in magnitude as the soil strength increases due to an increase in friction angle or increase in soil cohesion or both. It was also noted that the reinforcement loads are greater for the walls with weaker backfills.

Rowe and Ho (1997) conducted studies to evaluate the effects of material properties on the behavior of a reinforced retaining wall with full length of facing panel, and a hinged toe. They found that an increase in backfill soil friction angle φ (i.e., an increase in the soil shear strength), led to a decrease in the forces required to cause internal equilibrium.

Bilgin and Mansour(2014) conducted studies to evaluate the effects of friction angle of backfill/retained soil on the minimum length of reinforcement required to satisfy the design criteria for MSE walls under various conditions. The friction angle range considered was between 25^{o} and 45^{o}. The results indicated that as the friction angle of the backfill/retained soil increases, the required reinforcement length decreases for all external failure modes. This is due to a reduced lateral earth pressure coefficient for increased friction angles.

2.13 Effect of facing panel rigidity on the performance of reinforced soil retaining walls

Viera et al. (2013) performed a numerical study on continuous facing panel wall to study the influence of rigidity of facing panel on horizontal displacements of the wall, and the reinforcement tensile loads within geosynthetic reinforcement. Two dimensional finite difference software, FLAC 2D, was used to model a wall of height equal to 6m with ten horizontal reinforcement layers of uniformly spaced, and attached to a continuous facing panel. The length of the reinforcement was equal to 4.2m. Studies were carried out by varying the bending stiffness of facing panel, and it was determined that wall bending stiffness has a great influence on the pattern of lateral displacements. It was also found that, if the facing panel bending stiffness was increased from $11kN/m^2$ to $421.9kN/m^2$, the maximum displacement decreased from 0.56% of height to 0.48% of height, i.e., for an increase of rigidity by 38 times, a decrease in 14% was observed for the maximum value of displacement.

Rowe and Ho (1998) performed a numerical study to investigate the horizontal deformations of a reinforced soil retaining wall with a continuous facing panel. It was concluded that the maximum horizontal displacement of the facing wall decreases by 15% when the wall bending stiffness increases by 100 times.

Chapter 3

Modeling in PLAXIS 2D

3.1 PLAXIS 2D – an overview

PLAXIS 2D is a commercial finite element software intended to perform 2-D deformation and stability analysis in geotechnical engineering. The geometry of the model can be easily defined in soil and structures modes using PLAXIS 2D. The staged construction mode allows for simulation of construction and excavation process by activating and deactivating soil clusters and structural objects. The calculation kernel enables a realistic simulation of the non linear, time dependent and anisotropic behavior of soils or rock. The output consists of a full suite of visualization tools to check the details of the 2D soil-structure model.

3.2 Finite element method

The Finite Element Method (FEM) is a popular discretization technique in structural mechanics. The basic concept in physical interpretation of FEM is the subdivision of the mathematical model into disjoint (non-overlapping) components of simple geometry called 'finite elements'. The response of each element is expressed in terms of finite number of degrees of freedom characterized as the value of an unknown function or functions at a set of nodal points. Unlike finite difference models, finite elements do not overlap in space.

3.3 Finite element model

In the finite element method, a continuum is divided into a number of elements. Each element consists of a number of nodes. Each node has a degree of freedom that corresponds to discrete values of unknowns in the boundary value problem to be solved. Finite element calculations are becoming an increasingly important tool to predict soil behavior in and around construction sites.

In the present analysis, 15-noded triangular elements are used for discretization (Fig 3.1). This provides a fourth order interpolation for displacements and the numerical integration involves 12 Gauss points (stress Points). The 15-node triangle is a very accurate element that has produced high quality stress results for difficult problems. But the main disadvantage associated with a 15- noded triangular element is that it leads to more memory consumption, and hence slower calculations and operation performance.

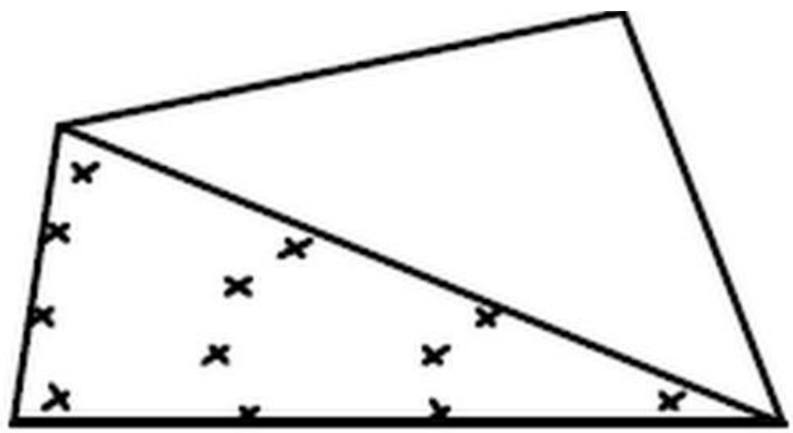


Fig 3.1: Nodes in a 15-noded triangular element

3.4 Validation of the model

Model of an unreinforced retaining wall developed in PLAXIS 2D was validated with Poulos and Davis (1967) elastic Solution.

A miniature model of unreinforced retaining wall of height 3m and width 6m was modeled in PLAXIS 2D (Fig 3.2) with the foundation soil modeled as linear elastic. The depth and width of foundation soil was taken as 12m and 50m, respectively. The wall is constructed in ten phases with each phase of height equal to 0.3m. The vertical settlement of the foundation, vertical and shear stresses at a certain depth obtained from PLAXIS model were compared with the elastic solution given by Poulos and Davis to ensure that the geometry and meshing used in the model are appropriate.

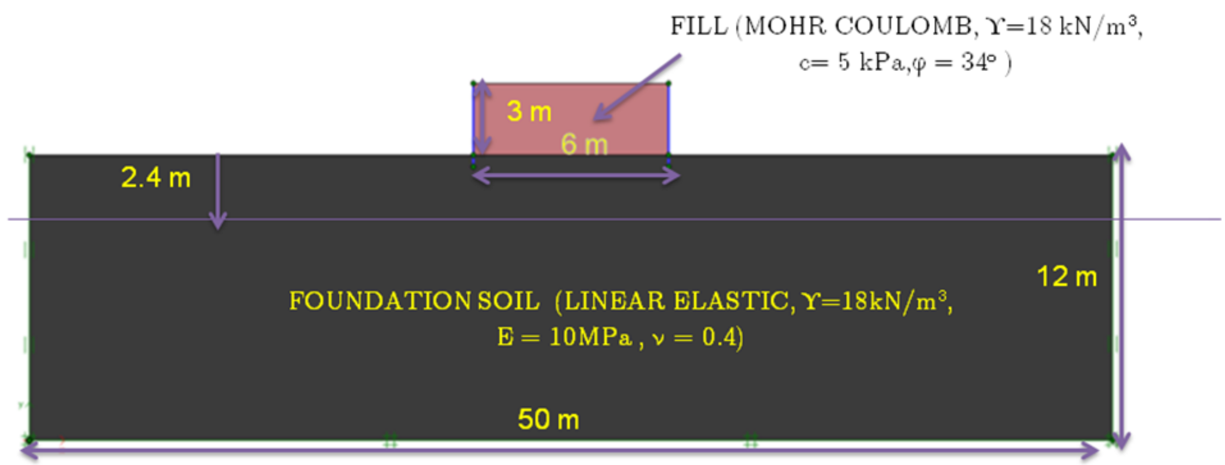


Fig 3.2: Plaxis model of Unreinforced Retaining Wall

3.4.1 Comparison of vertical settlement

Poulos and Davis (1967) have provided elastic solutions to most of the loading relevant to soil and rock mechanics in the form of charts, tables and explicit solutions.

Fig 3.3 shows the schematic of strip loading on a finite soil layer. A uniform vertical load of intensity p/unit area and width ‘B’ is applied over a rigid base of depth ‘h’.

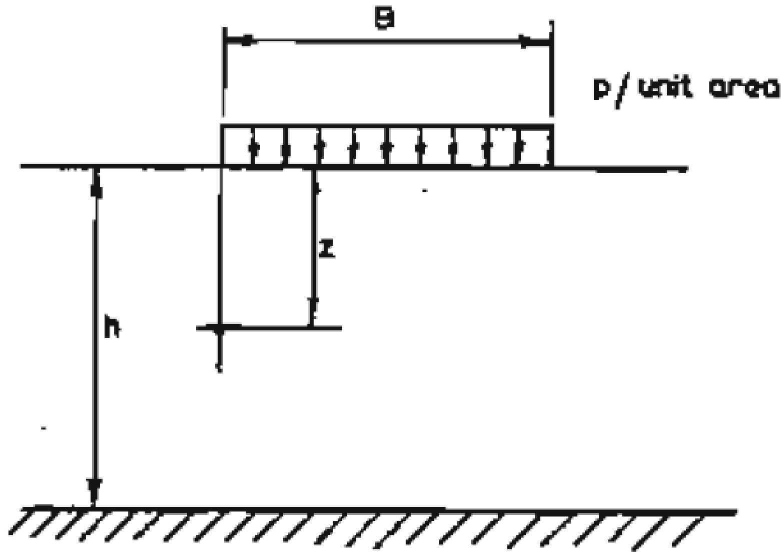


Fig 3.3: Schematic of Loading on a Finite Soil Layer (Poulos 1967)

Fig. 3.4 shows the Influence Factors (I_{st}) for the vertical displacement (ρ_z) beneath the edge of the strip proposed by Poulos (1967).

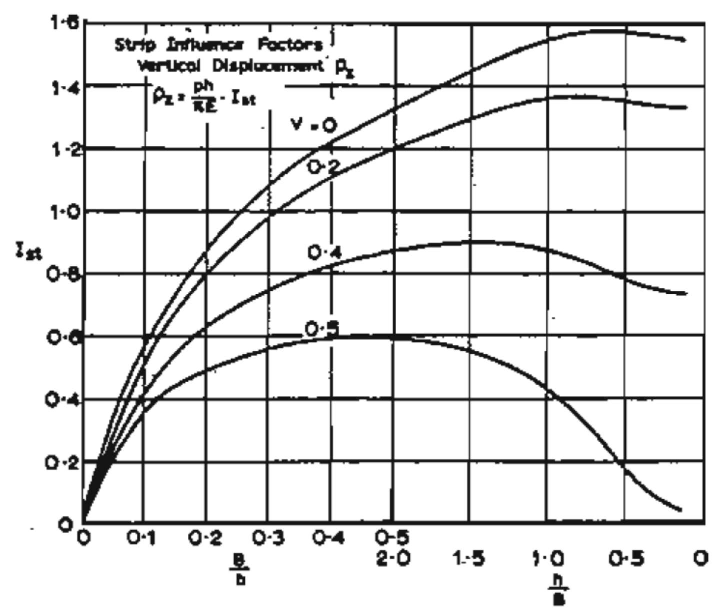


Fig 3.4:Influence Factors for Vertical Displacement(Poulos 1967)

For a B/h value equal to 0.5 and Poisson's ratio equal to 0.4, the value of I_{st} can be obtained from the chart as 0.87. A similar case with h= 12m, B = 6m and Poison's ratio (ν) = 0.4 was modeled in PLAXIS 2D. The vertical displacements obtained from PLAXIS model were then compared with the elastic solutions

Table 3.1 gives the summary of comparison of results obtained from elastic solution and PLAXIS. The comparison is also represented in Fig.3.5.

Table 3.1: Vertical Settlements from Elastic Solution and PLAXIS

PHASE	FROM ELASTIC SOLUTION(in mm)	FROM PLAXIS ANALYSIS(in mm)	PERCENT DIFFERENCE
1	2.1	2.3	8.69
2	4.1	4.8	14.58
3	6.2	7.2	13.89
4	8.0	9.5	15.79
5	10.3	12.0	14.17
6	12.3	14.0	12.14
7	14.4	16.0	10
8	16.5	18.0	8.33
9	18.5	19.5	5.13
10	20.6	20.7	0.48

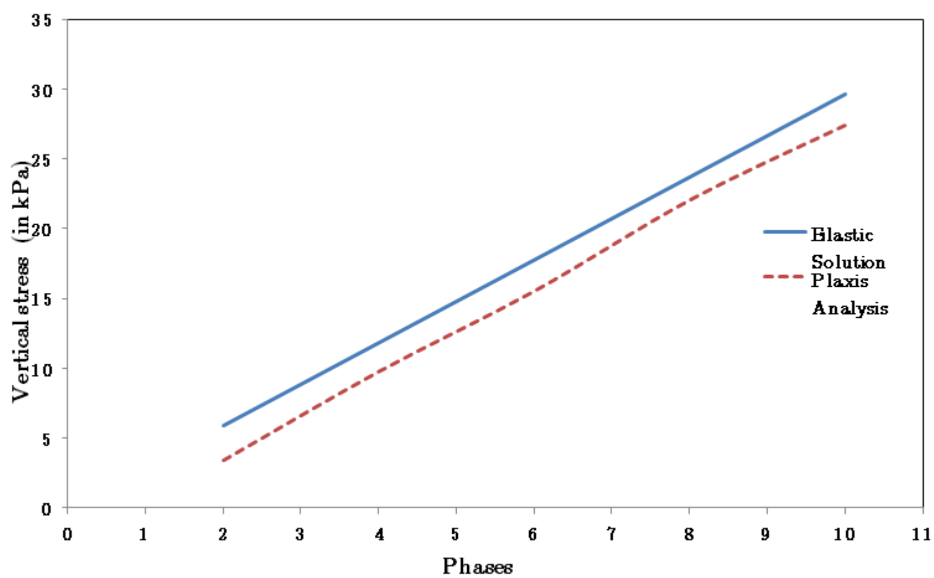


Fig 3.5: Comparison of Vertical Settlement from Elastic Solution and PLAXIS

3.4.2 Comparison of Vertical Stress

The vertical stress Influence Factor for a Poisson's ratio ν equal to 0.4 is shown in Fig 3.6. An arbitrary value of 2.4m was taken for 'z' where z represents the distance from the top of the rigid base. Hence for a B/h = 0.5 and z/h = 0.2, the value of I_{st} = 1.55. Vertical stress can thus be calculated for all the phases.

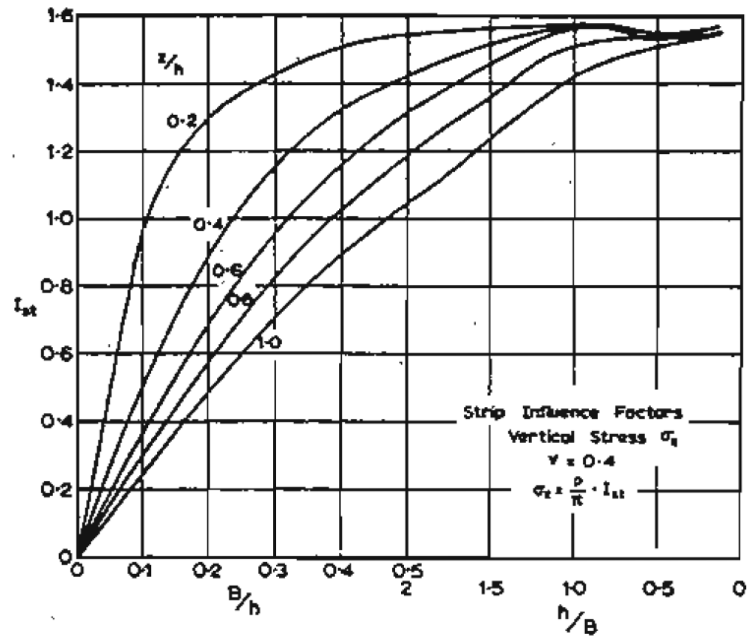


Fig 3.6:Influence Factors for Vertical Stress (Poulos 1967)

Vertical Stress at a depth of 2.4m below the top layer of foundation soil was calculated from PLAXIS, and was then compared with elastic solutions. The comparison results are tabulated in Table 3.2 and represented in a graphical form in Fig 3.7.

Table 3.2: Vertical Stress from Elastic Solution and PLAXIS

PHASE	FROM ELASTIC SOLUTION (in kPa)	FROM PLAXIS ANALYSIS (in kPa)	PERCENT DIFFERENCE
2	5.9	3.4	42.37
4	11.8	9.8	16.95
6	17.8	15.6	12.36
8	23.7	22.1	6.75
10	29.6	27.5	7.09

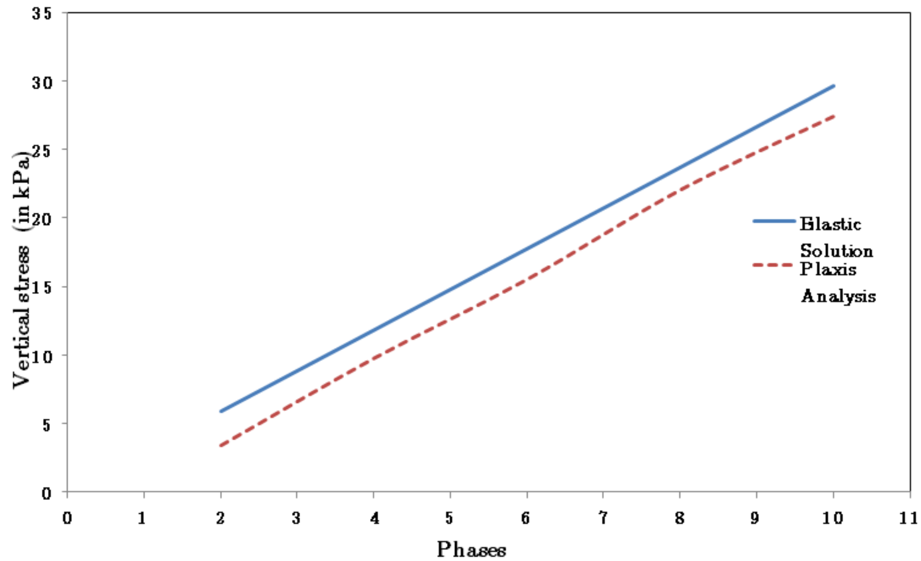


Fig 3.7: Comparison of Vertical Stress from Elastic Solution and PLAXIS

3.4.3 Comparison of Shear Stress

A comparison was also made for the shear stress in order to ensure that the results obtained from PLAXIS are in accordance with the elastic solutions.

The shear stress Influence Factor for a Poisson's ratio ν equal to 0.4 is shown in Fig 3.8. Corresponding to an arbitrary depth of z = 2.4, the value of I_{st} can be obtained as 0.75. Shear Stress can thus be calculated for all the phases.

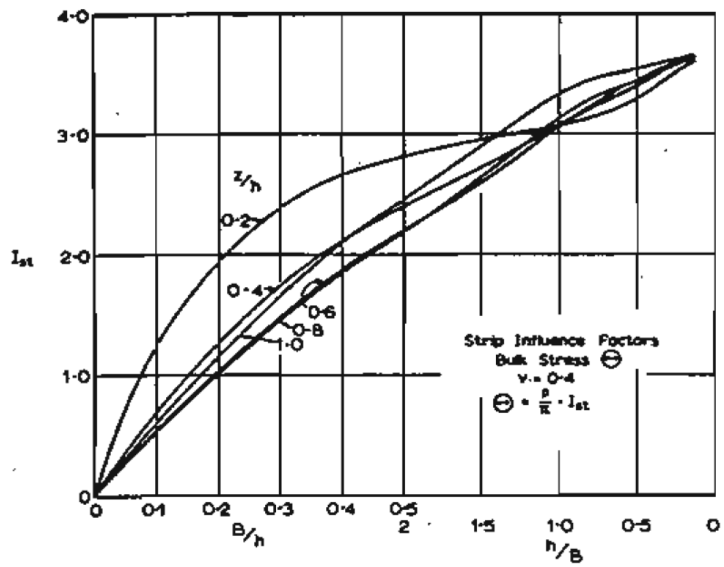


Fig 3.8:Influence Factors for Shear Stress (Poulos 1967)

Shear stress at a depth of 2.4m below the top layer foundation soil was also calculated using PLAXIS and then compared with the values obtained from elastic solutions. The comparison results are tabulated in Table 3.3 and represented graphically in Fig 3.9.

Table 3.3: Shear Stress from Elastic Solution and PLAXIS

PHASE	FROM ELASTIC SOLUTION (in kPa)	FROM PLAXIS ANALYSIS (in kPa)	PERCENT DIFFERENCE
2	2.54	2.89	12.11
4	5.08	5.76	11.8
6	7.62	8.35	8.74
8	10.16	10.89	6.7
10	12.7	12.8	0.78

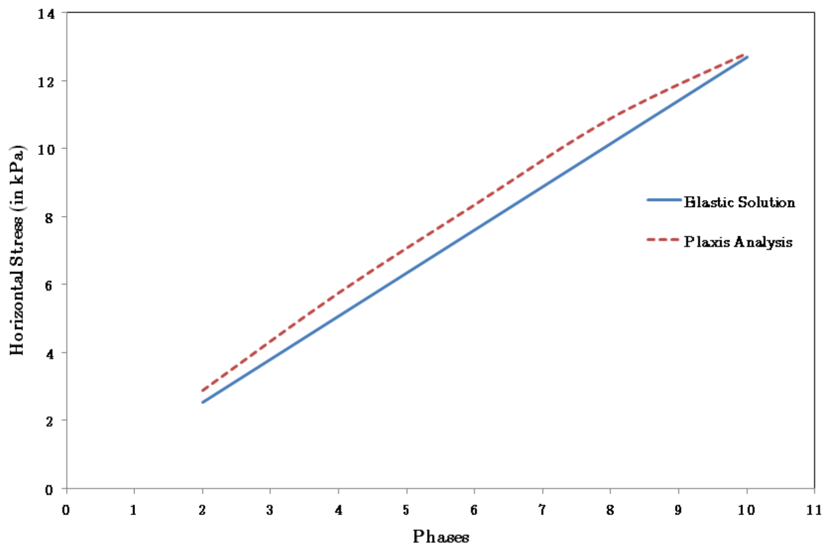


Fig 3.9: Comparison of Shear Stress from Elstic Solutions and PLAXIS

3.5 Fixation of bottom boundary of reinforced retaining wall

As per Schmertmann's method (Schmertmann 1978), the depth of influence is dependent on the type and shape of loading. For an axi symmetric loading, strain influence factor, I_z, varies linearly from 0.1 at the bottom of the footing to a peak value, I_{zp}, at a depth of B/2. The strain influence factor then decreases to zero at a depth of 2B. While for a plane strain case, I_z varies linearly from 0.2 at the bottom of the footing to I_{zp} at a depth of B. The strain influence factor then decreases to zero at a depth of 4B. In the present model, plane strain condition exists and hence any loading on the top of soil will have an influence up to 4B; i.e., 4 times the width of footing. Since the width of the footing used for the analysis is equal to 12m, the influence depth becomes equal to 48m.

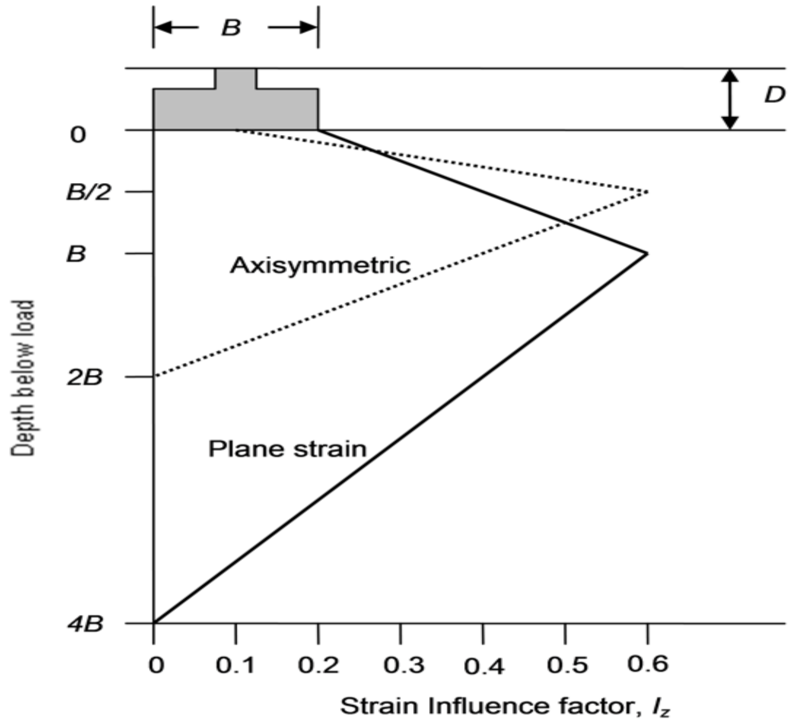


Fig 3.10 : Strain Influence Factors vs Depth below the Load (Schmertmann 1978)

Trials were done using PLAXIS 2D software for several depths of bottom boundary (equal to 10m, 36m, 48m, 60m and 80m). Fig 3.11 (a) through Fig 3.11 (e) shows the models for various depths of bottom boundary.

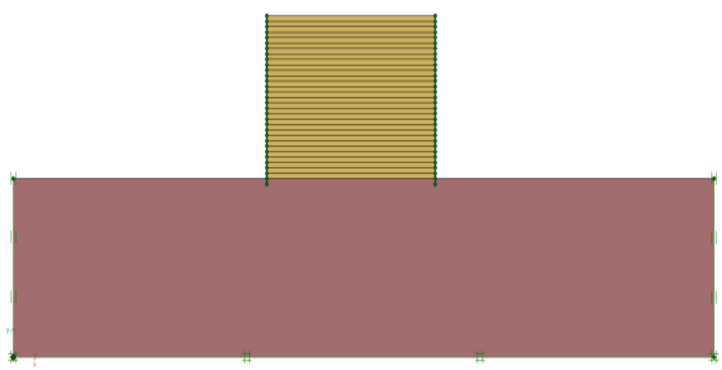


Fig 3.11(a): PLAXIS Model for 10m deep foundation soil

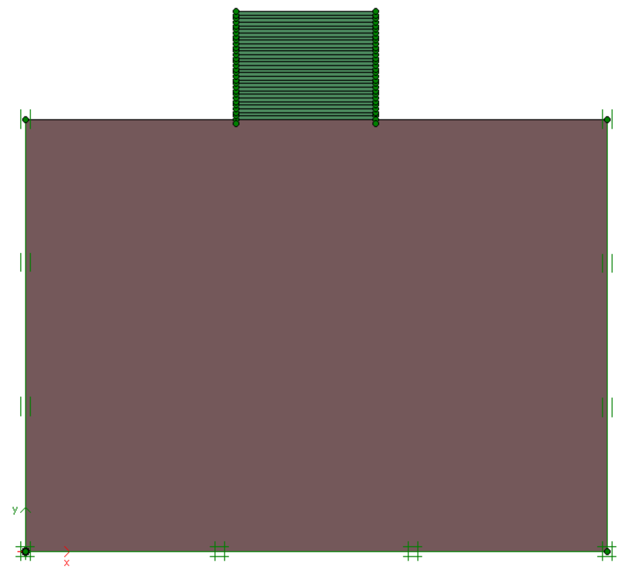
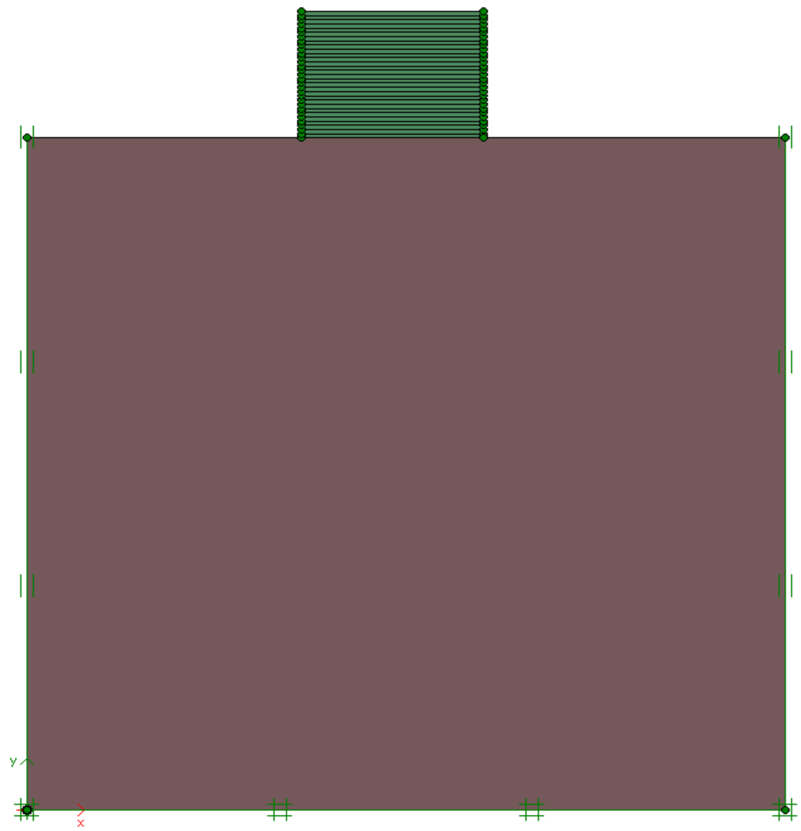


Fig 3.11(b): PLAXIS Model for 36m deep foundation soil



3.11(c): PLAXIS Model for 48m deep foundation soil

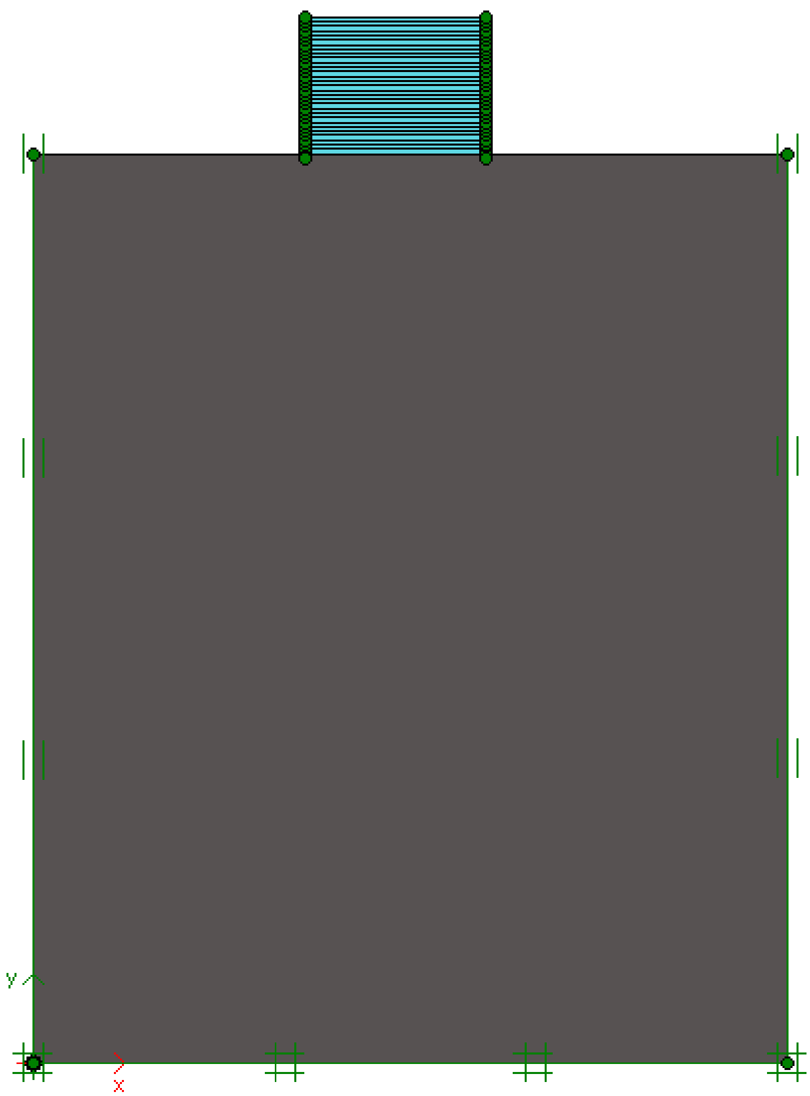


Fig 3.11(d): PLAXIS Model for 60m deep foundation soil

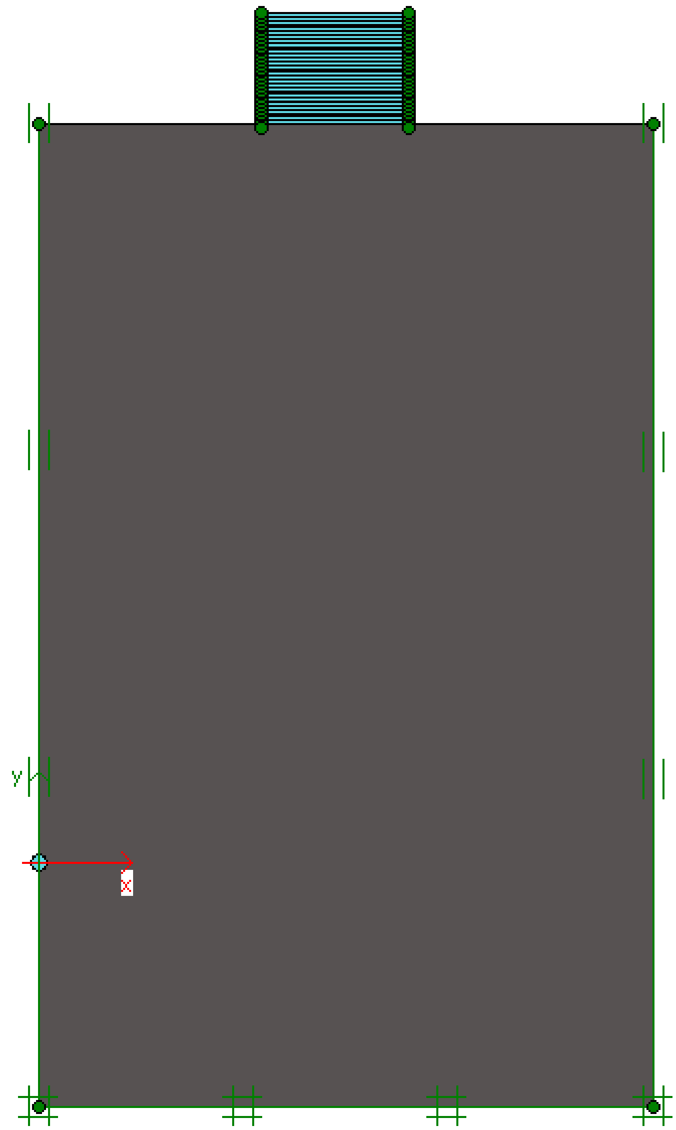


Fig 3.11(e): PLAXIS Model for 80m deep foundation soil

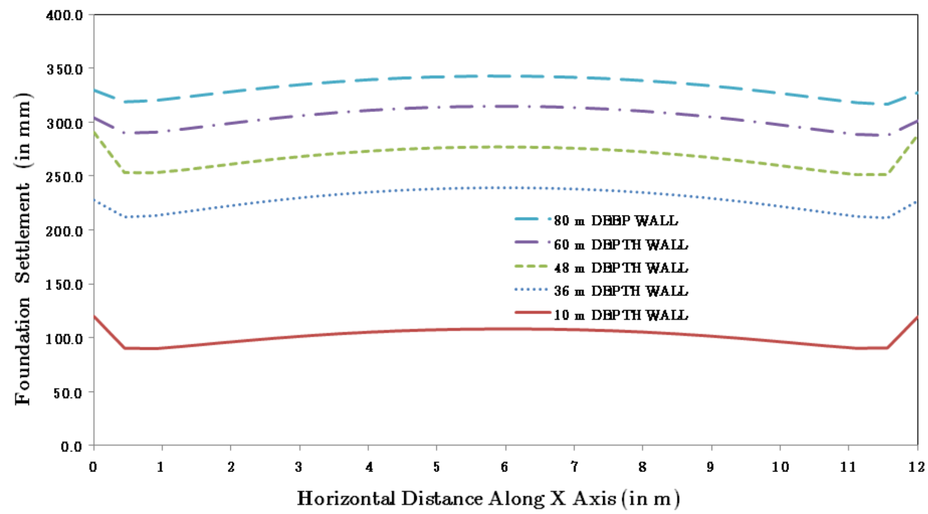


Fig 3.12: Settlement of Foundation Soil vs the Distance from Left Facing for Different Depths of Foundation

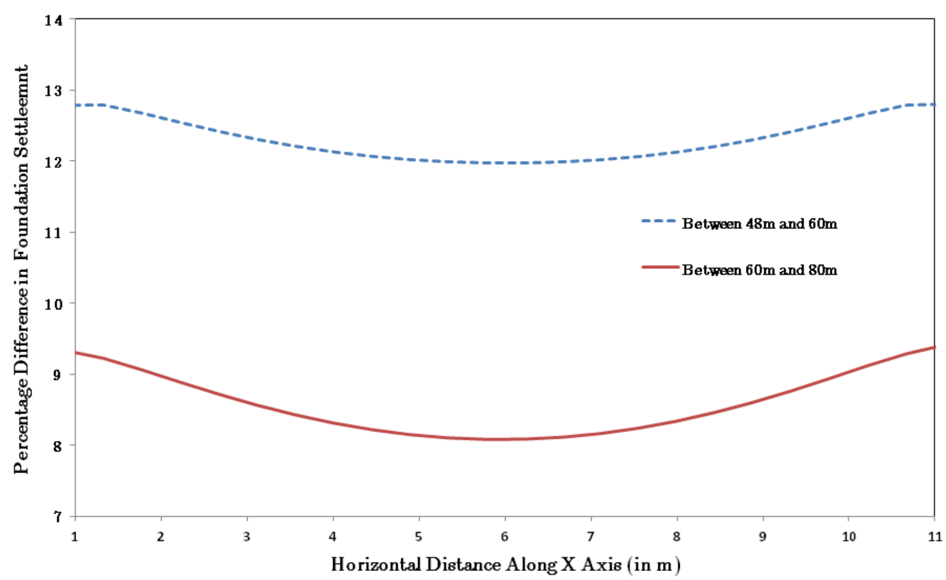


Fig 3.13: Comparison of Percentage Difference in Foundation Settlement between 48m and 60m deep, and 60m and 80m deep foundation

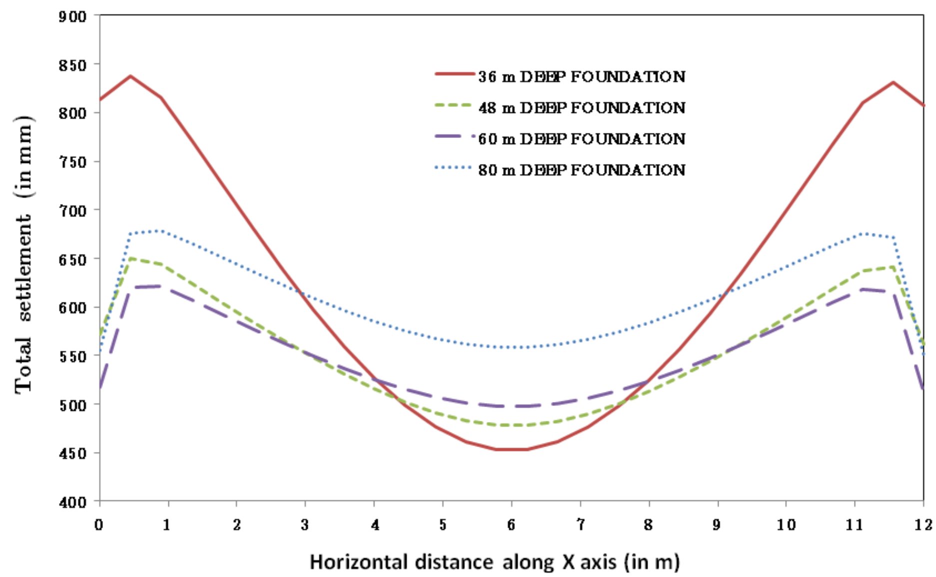


Fig 3.14: Surface Settlement Analysis for different depths of Foundation

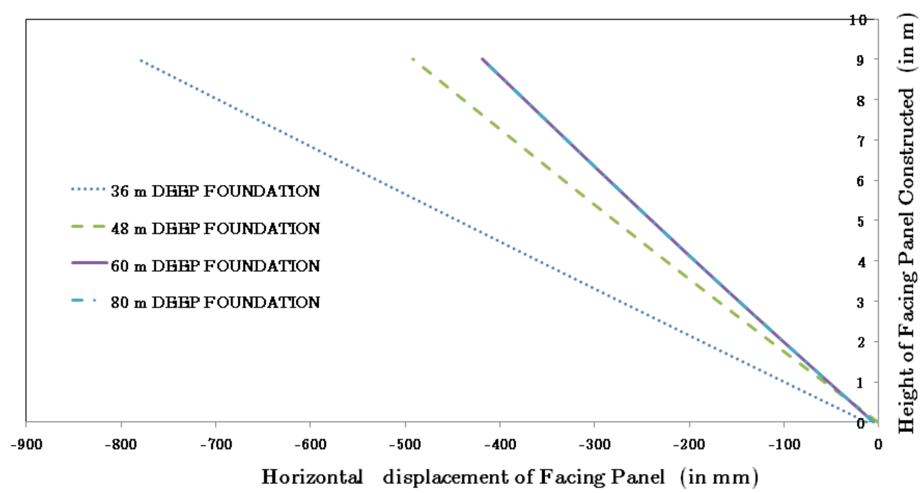


Fig 3.15: Horizontal Displacement of Facing Wall vs Height of Panel Wall for different depths of Foundation

It is observed from the above graphs that (Fig 3.12 to Fig 3.15), when the depth of foundation was increased from 48m to 60m, percentage difference in settlement ranges from 12% to 13%. But, when the foundation depth was increased from 60m to 80m, the percentage difference in foundation settlement was found to be less than 10%. Similarly, facing panel displacement was observed to give the same values for 60m deep and 80m deep foundations. Hence, the bottom depth of foundation soil was fixed as 60m in order to cover all the compressible soil layers that might contribute to the total settlement of the structure.

3.6 Modeling reinforced retaining wall

3.6.1 Model geometry

Back-to-Back retaining walls of different heights 3m, 6m, 9m, 12m and 15m were modeled in PLAXIS 2D with full length facing panel on both sides. The wall was constructed in layers with each layer taken as 0.3m thickness. The width of reinforced soil zone is assumed to be 12m and is founded on a 60m deep and 50m wide foundation soil. Plate elements are used to model facing panel, and are embedded 0.3m into the foundation soil in order to ensure that the movement is restricted. The length of reinforcements is assumed to be 4.5m on both sides in accordance with the codal recommendations. Fig. 3.16 shows the model developed for a reinforced retaining wall of height equal to 9m.

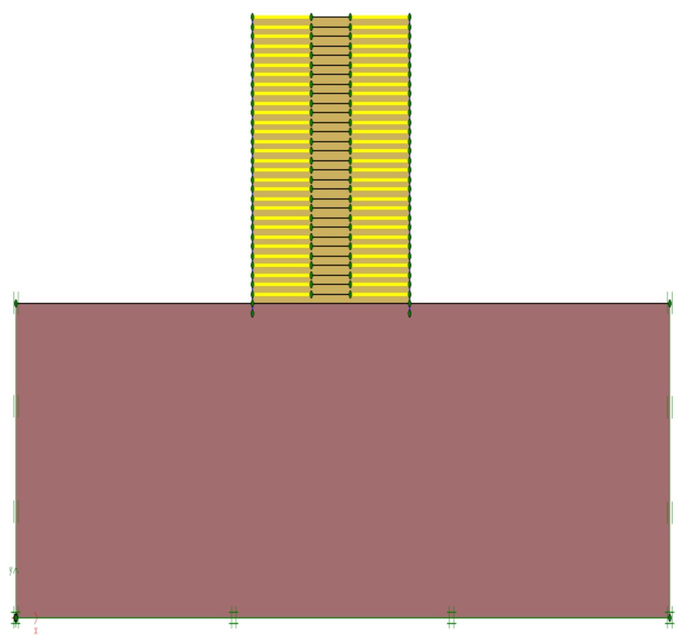


Fig 3.16: PLAXIS Model of Reinforced Retaining Wall (9m high)

3.6.2 Material models

3.6.2.1 Soil

Mainly two soil models are used for modeling reinforced and foundation soils: Mohr-Coulomb and Linear Elastic models.

MOHR-COULOMB MODEL

The linear-elastic, perfectly plastic (Mohr-Coulomb model) is used widely to model the soil behavior. In this case, a constant average stiffness is assumed for the soil layer. Due to this constant stiffness, computations tend to be relatively fast and first estimate of deformations can be obtained. It involves mainly five parameters, i.e., E and ν for soil elasticity, φ and c for soil plasticity, and ψ as an angle of dilatancy.

LINEAR-ELASTIC MODEL

The linear-elastic model is based on Hooke's law of isotropic elasticity. It involves two basic parameters, i.e., Young's Modulus (E) and Poisson's Ratio (ν). Linear Elastic model is not so appropriate to model soil behavior because it is insufficient to capture the essential features of soil. However, it can be used to model stiff volumes in soil like concrete walls or intact rock formations.

Table 3.4 gives the materials properties used in this model.

Table 3.4: Soil Properties

TYPE	MATERIAL	UNIT WEIGHT (kN/m^3)	MODULUS OF ELASTICITY E(kPa)	POISSON'S RATIO	COHESSION (kPa)	INTERNAL FRICTION ANGLE
FOUNDATION SOIL	LINEAR ELASTIC	18	100,000	0.3	-	-
BACKFILL	MOHR COULOMB	18	96,000	0.3	5	35

3.6.2.2 Interfaces

Interfaces are mainly used in PLAXIS 2D to enable soil-structure interaction without which, the soil and structure are tied together and no relative displacement (slipping/gapping) is possible between structure and soil. A 'virtual thickness', an imaginary dimension used to define material properties of interface, is assigned to interface. The higher the virtual thickness, the higher are the elastic deformations. The default value of virtual thickness is 0.1. A typical application of interfaces would be in a region which is intermediate between fully smooth and rough. The roughness of the interaction is modeled by choosing a suitable value for the strength reduction factor for the interface (R_{inter}). This factor relates the interface strength (wall friction and adhesion) to the soil strength (friction angle and cohesion). In general, for real soil-structure interaction, the interface is weaker and more flexible than surrounding soil which means that the value of R_{inter} should be less than 1. A reduced value of R_{inter} not only reduces the interface strength but also the interface stiffness. In this case, a R_{inter} value equal to 0.97 was adopted throughout the study.

3.6.2.3 Reinforcements

Geogrids are flexible elastic elements that represent a grid or sheet of fabric, and are mainly used to model soil reinforcements in PLAXIS 2D.They are slender structures with an axial stiffness but with no bending stiffness. It can sustain only tensile forces and no compression. In PLAXIS, a geogrid is created by selecting a geogrid material dataset in the material database, and assigning this material to one or more surfaces. The only material property of a geogrid is an elastic normal (axial) stiffness 'EA' which can be selected in the material database. Each geogrid element is defined by five nodes for the case of a 15-

noded soil elements employed in the model. Geogrids are placed in horizontal layers to unify the mass of the composite MSE wall structure to increase the resistance of the wall to the destabilizing forces generated by the retained soils and surcharge loads. To achieve a composite MSE wall structure, geogrids must possess adequate tensile strength to be placed in sufficient layers and must develop sufficient connection and anchorage capacity to hold the composite structure together.

3.6.2.4 Plate elements

Plate elements are used to create structural objects (or slender structures in the ground) with a significant flexural rigidity (or bending stiffness) and a normal stiffness. Plates can be used to simulate the influence of walls, shells or linings extending in z direction. The most important properties of plates, that can be assigned in PLAXIS includes its Flexural Rigidity and Axial Stiffness. From these two parameters, an equivalent plate thickness (d_{eq}) can be calculated from the equation (Eq.3.1):

$$\sqrt{\frac{12\,EI}{EA}} \tag{3.1}$$

Five-node plate elements are used along with 15-noded soil elements. Plates can be either elastic or elasto-plastic. Elasto-plastic plate elements are used in this model study to model concrete facing. The properties used for the plate elements are as follows:

EA = 12000000kN/m

EI = 120000kNm^2/m

w = 8.3kN/m/m

M_p = 1e12kNm/m

N_p = 1e12kN/m

Here, EA (axial stiffness), EI (flexural rigidity) and w (specific weight of the plate which is entered as a force per unit length per unit width in the out of plane direction) denotes the stiffness properties of the plate, and Mp (maximum bending moment) and Np (maximum axial force) denotes the strength properties of the plate.

3.6.3 Boundary conditions

Standard Fixities boundary condition is adopted throughout the model. This is done to impose a set of general boundary conditions to the geometry model. These Boundary conditions are generated according to the following rules:

a) Vertical geometry lines for which the *x*- coordinate is equal to the lowest or highest *x*-coordinate (right and left boundaries of the model) in the model obtain a horizontal fixity (u_x=0).

b) Horizontal geometry lines for which the *y*-coordinate is equal to the lowest *y*-coordinate (bottom boundary) in the model obtain a full fixity (u_x= u_y=0).

Standard Fixities can be used as a convenient and fast input option for many practical applications.

3.6.4 Meshing

When the geometry is fully defined and material properties have been assigned to all clusters and structural objects, the geometry has to be divided into finite elements in order to perform finite element calculations. A composition of finite elements is called a mesh. The basic type of element in a mesh is a six-noded triangular element or 15-noded triangular elements. 15-noded triangular elements are used throughout the model. PLAXIS 2D allows for a fully automatic mesh generation of finite element meshes. This generation of mesh is based on robust triangulation procedure. Although PLAXIS 2D automatically applies local mesh refinements, meshes that are automatically generated may not be accurate enough to produce acceptable numerical results. The mesh generator requires a general meshing parameter which represents the average element size L_e (Eq. 3.2). In PLAXIS, this parameter is calculated from the outer geometry dimensions (x_{min}, x_{max}, y_{min}, y_{max}) and a global coarseness setting defined in the mesh menu.

$$L_e = \frac{n_c}{12}\sqrt{(x_{max} - x_{min})(y_{max} - y_{min})} \tag{3.2}$$

Distinction is made between five levels of global coarseness as Very Coarse, Coarse, Medium, Fine and Very Fine. For this particular model, very fine mesh is adopted where the value of global coarseness setting factor (n_c) equal to 0.5, i.e., around 1000 elements are created. The average element size and number of generated triangular elements depends on this global coarseness setting factor. A mesh composed of 15-noded elements gives a much finer distribution of nodes, and thus gives much more accurate results than a similar mesh composed of six-node elements. On the other hand, the use of 15-noded elements is more time consuming than using six-noded elements.

3.7 Results and discussions

Figures 3.17 to 3.25 shows a comparison study of foundation settlement, surface settlement and horizontal displacement of facing panel between unreinforced and reinforced retaining walls of heights equal to 3m, 6m, 9m, 12m and 15m. Parametric Study was also performed by varying the axial stiffness of reinforcement from 500kN/m to 5000kN/m.

3.7.1 3m, 6m and 9m high wall

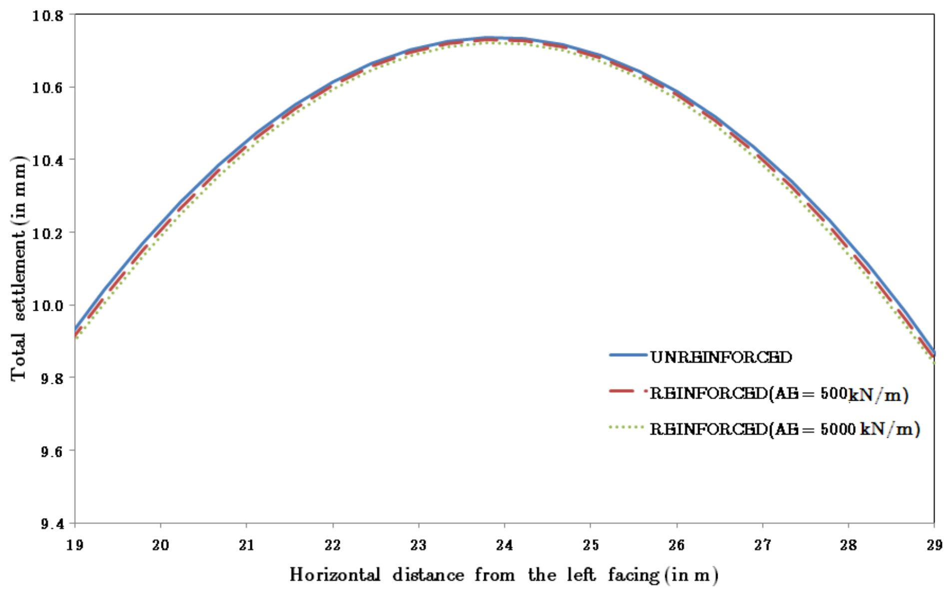


Fig 3.17: Foundation Settlement Analysis for 3m high wall

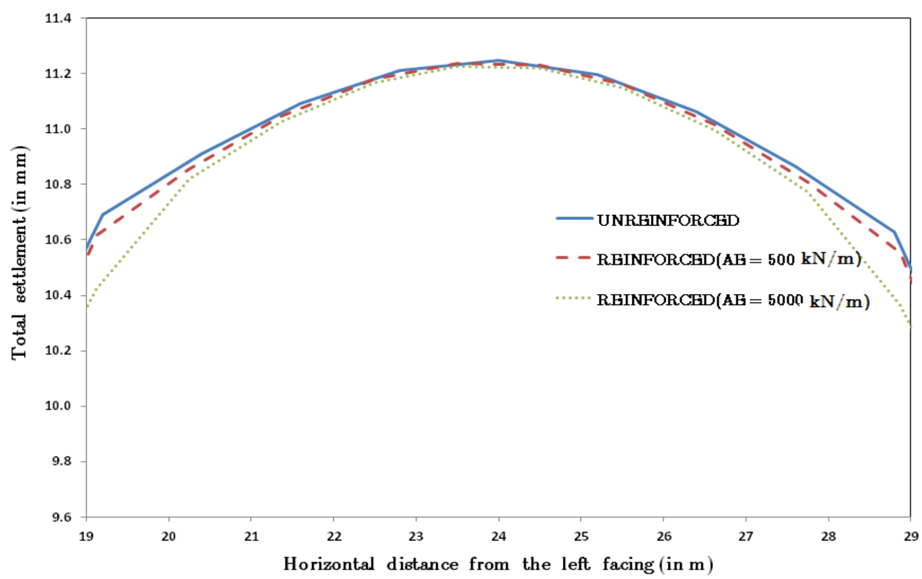


Fig 3.18: Surface Settlement Analysis for 3m high wall

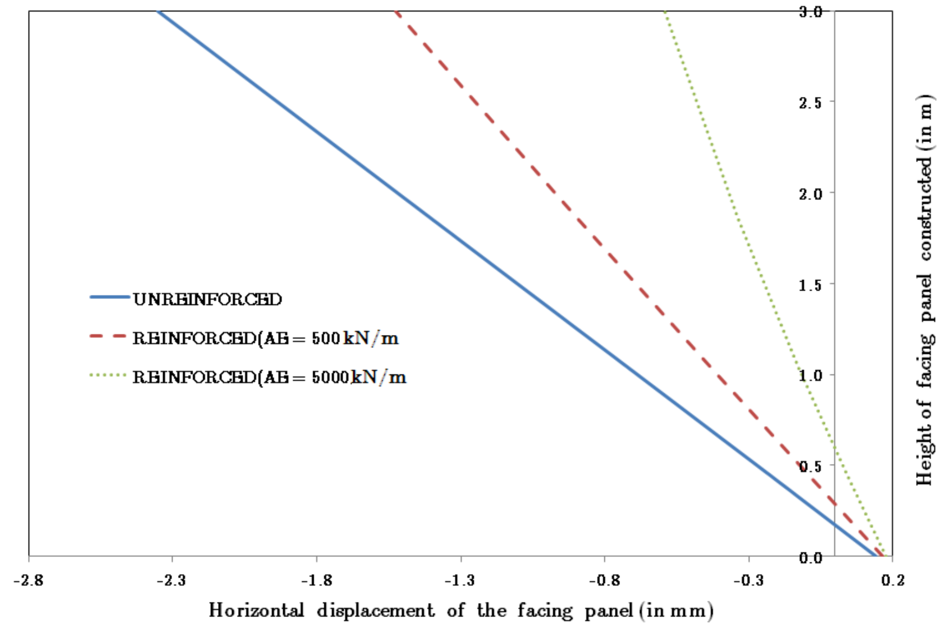


Fig 3.19: Facing Displacement Analysis for 3m high wall (a negative value of displacement indicates movement away from the wall)

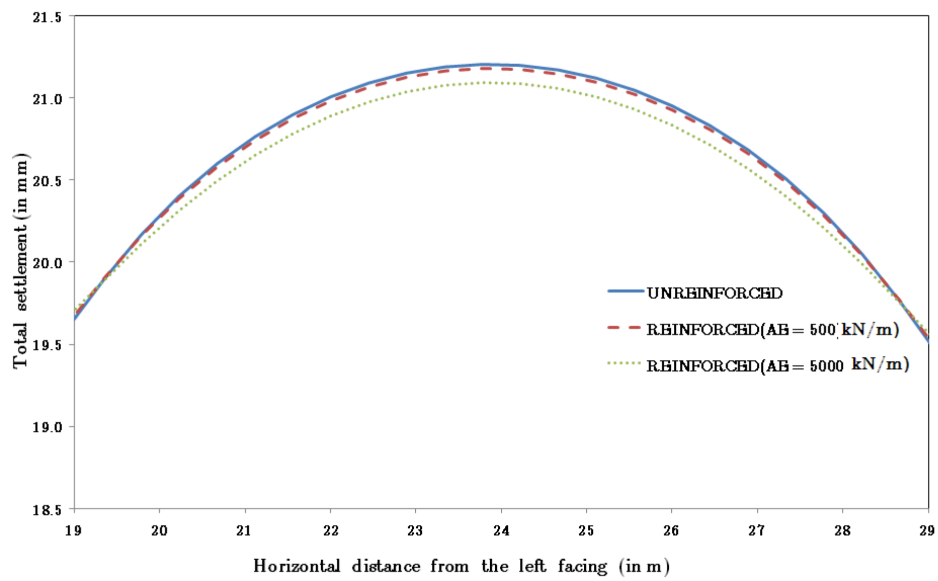


Fig 3.20: Foundation Settlement Analysis for 6m high wall

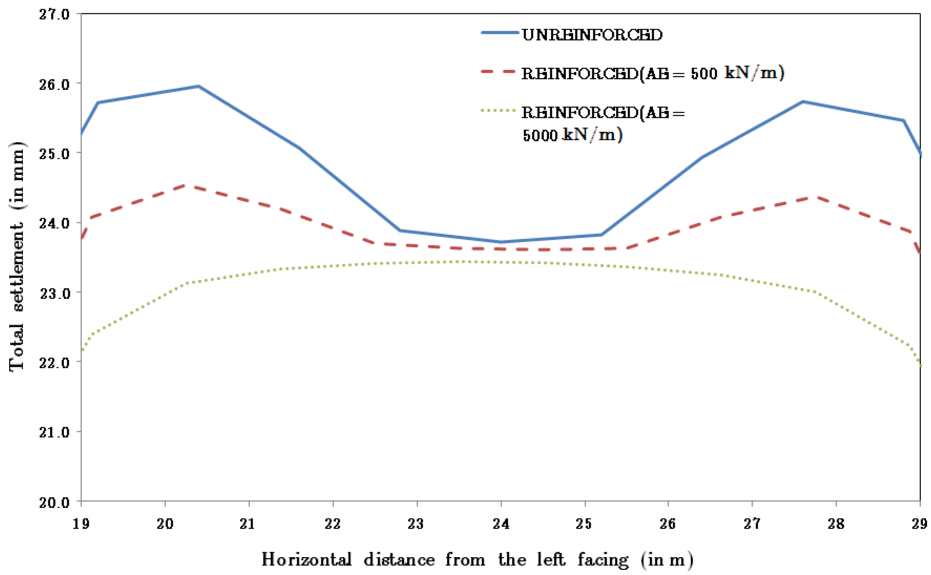


Fig 3.21: Surface Settlement Analysis for 6m high wall

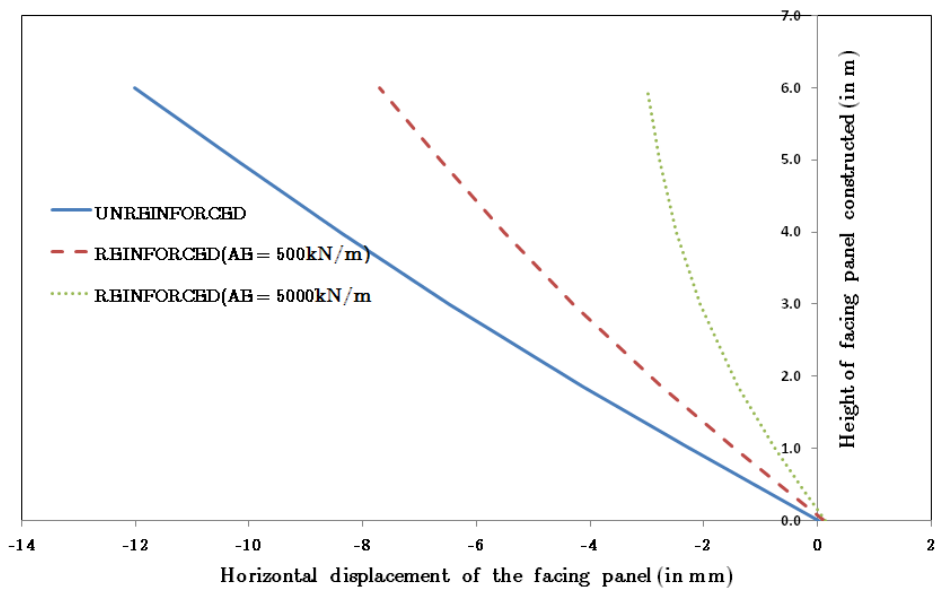


Fig 3.22: Facing Displacement Analysis for 6m high wall

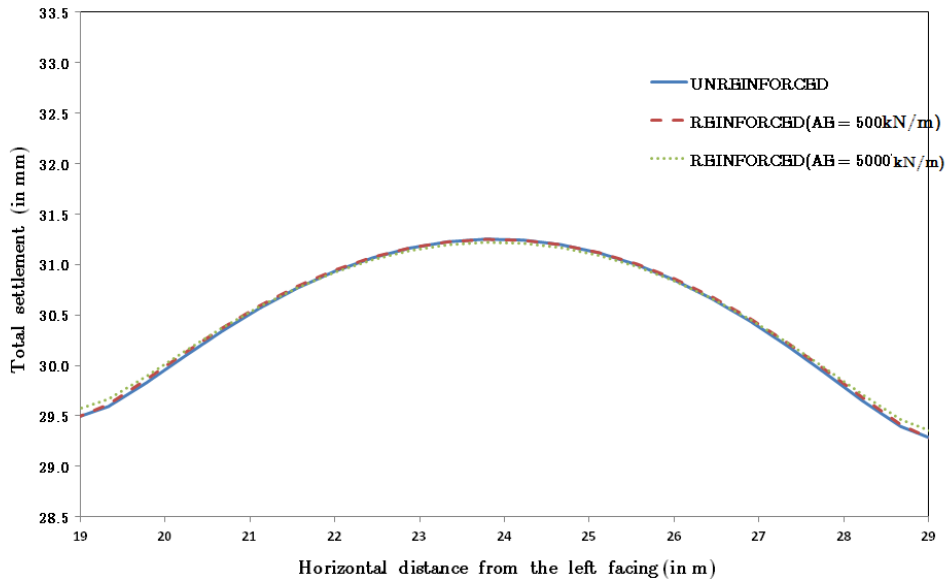


Fig 3.23: Foundation Settlement Analysis for 9m high wall

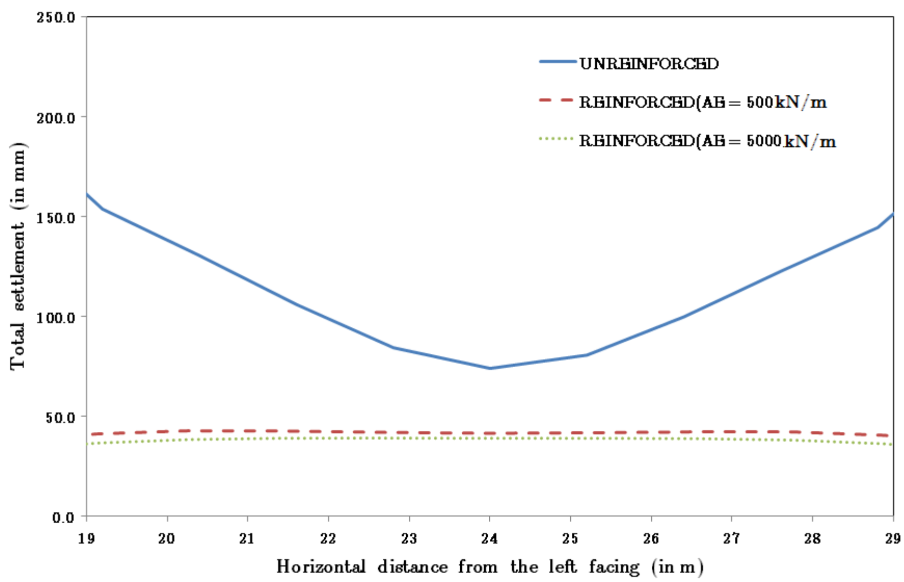


Fig 3.24: Surface Settlement Analysis for 9m high wall

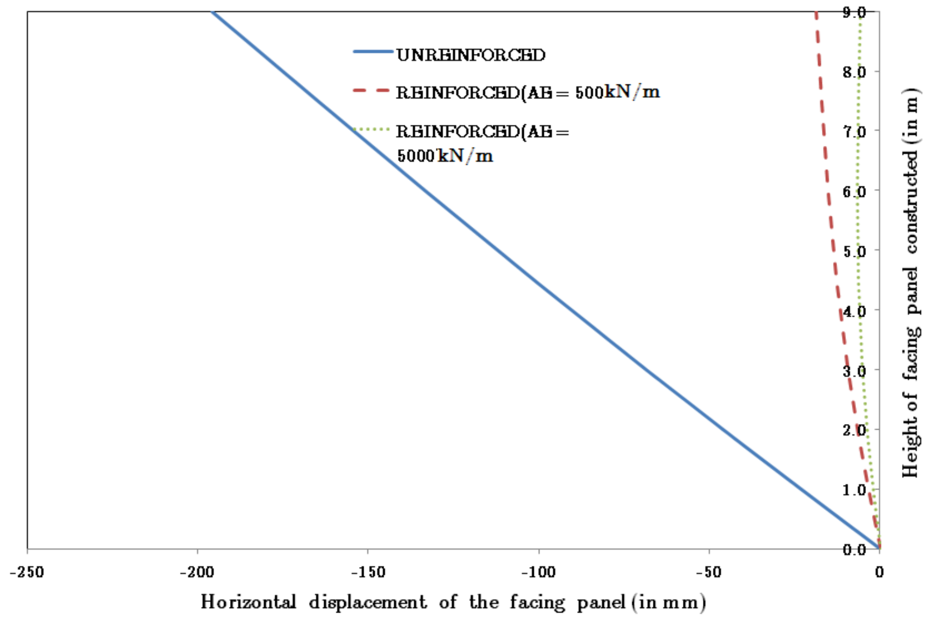


Fig 3.25: Facing Displacement Analysis for 9m high wall

It was observed that the foundation settlement, surface settlement and the facing panel displacement increases with an increase in the height of the wall. Reinforcing the retaining wall leads to a reduction in the foundation settlement, surface settlement and horizontal facing panel displacement. It was noted that the reduction caused by reinforcing the retaining wall is more evident for retaining walls of greater heights. That is facing panel displacement gets reduced by 38% for a 3m high wall (Fig 3.19) while for a 9m high wall, a reduction of 88% can be observed (Fig 3.25) Increasing the axial stiffness of reinforcement also leads to a reduction in the settlements and deformations of MSE Walls. Considerable reduction can be observed in the displacement of facing panel by increasing the stiffness of reinforcement from 500kN/m to 5000kN/m. That is a decrease of 67%, 75% and 89% was observed in 3m high wall (Fig 3.19), 6m high wall (Fig 3.22) and 9m high wall (Fig 3.25), respectively.

3.7.2 Comparison study

In the case of Back-to-Back wall desigh, the distance between two opposite walls is a key parameter to determine the analysis method to be adopted (FHWA 2001). However existing design methodologies do not provide a clear design methodology for the analysis of Back-to-Back MSE walls.

Han and Leshchinsky (2007) performed several studies to determine the effect of varying the spacing between reinforcements and backfill friction angle on the critical failure surface and tension developed in the reinforcement. The model developed by Han and Leshchinsky (2007) is shown in Fig 3.26.

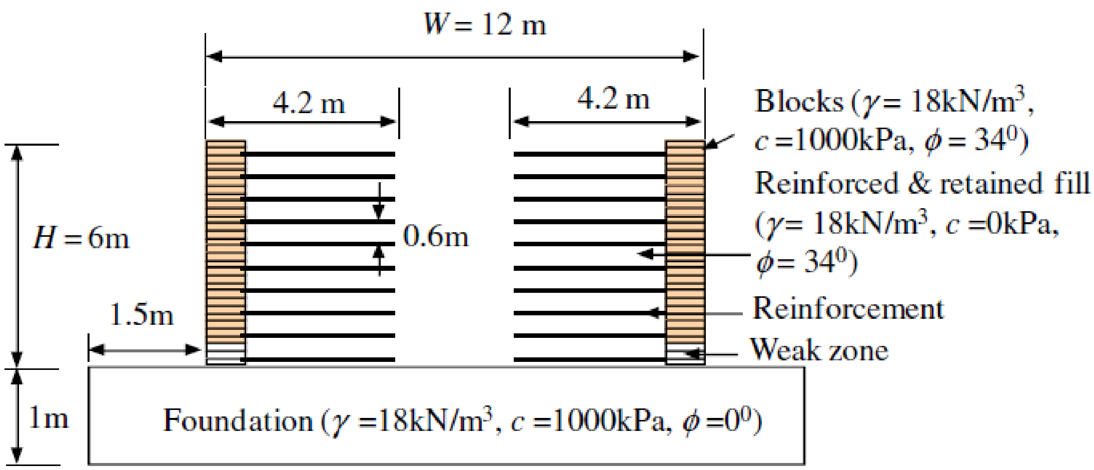


Fig 3.26: Model of Back-to-Back Reinforced Retaining wall (Han and Leshchinsky, 2007)

A similar model with the same properties and parameters was developed in PLAXIS 2D software (Fig 3.27). The model was developed in PLAXIS to perform an extensive parameteric study.

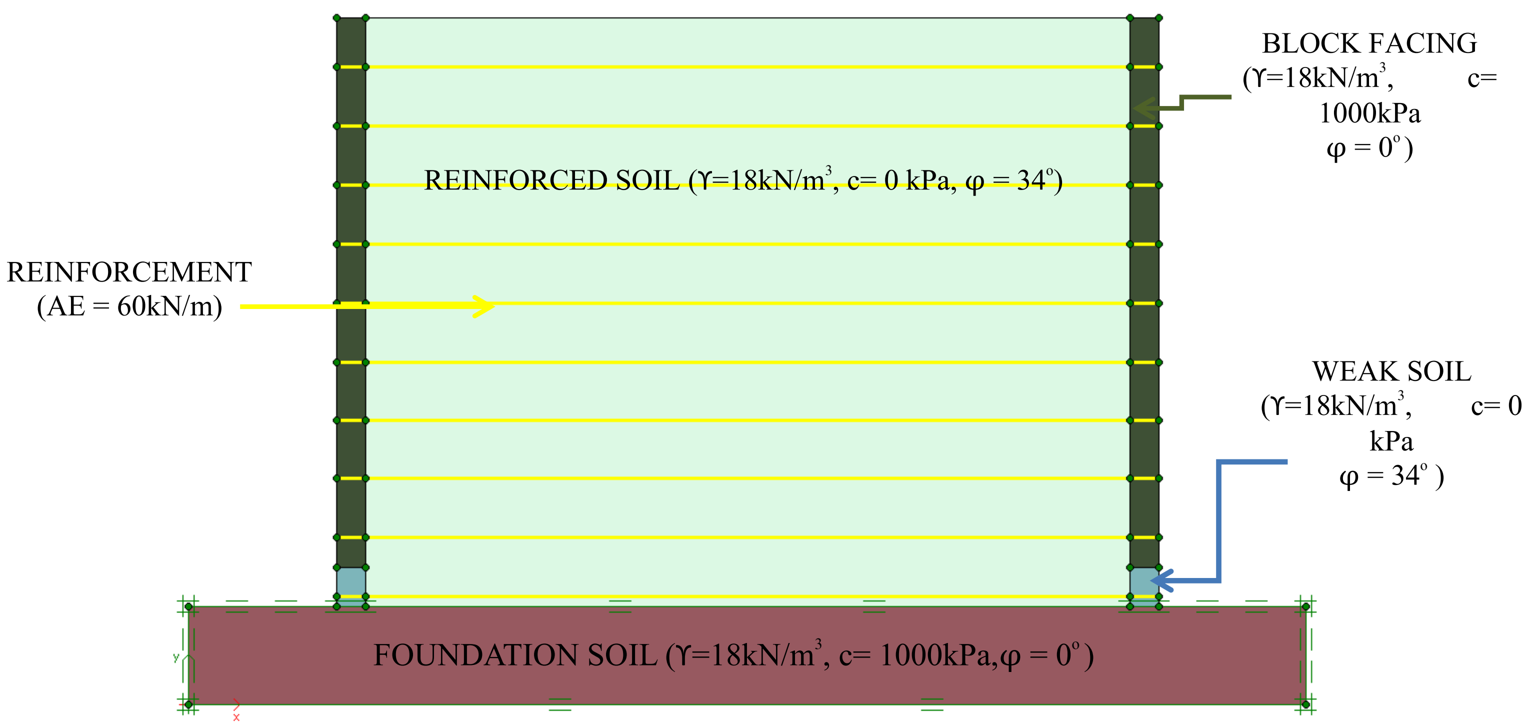


Fig 3.27: PLAXIS Model of Back-to-Back Reinforced Retaining wall

Comparison studies were performed by varying the W/H ratios and backfill Friction angle. Three trials were done by varying the W/H ratios; i.e, W/H = 1.4, W/H = 2, W/H = 3. Trials were also done for two values of backfill friction angle; i.e φ = 25 and φ = 34. The models developed in PLAXIS is as follows (Fig 3.28 to Fig 3.30) :

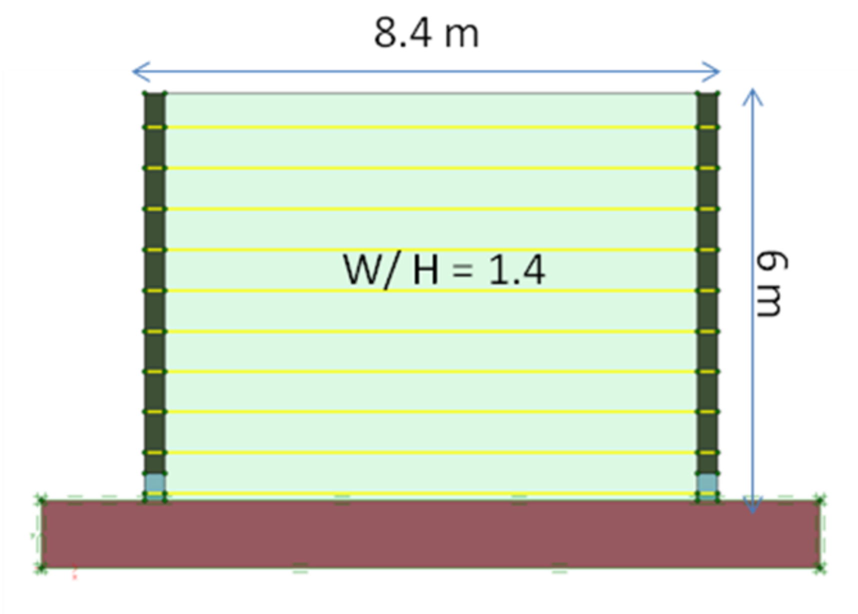


Fig 3.28: Case 1:- W/H = 1.4

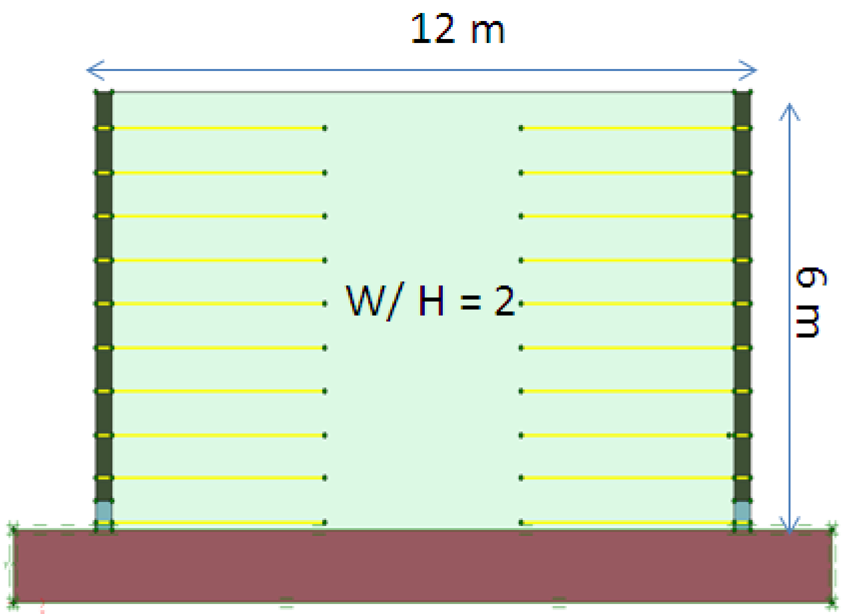


Fig 3.29: Case 2:- W/H = 2

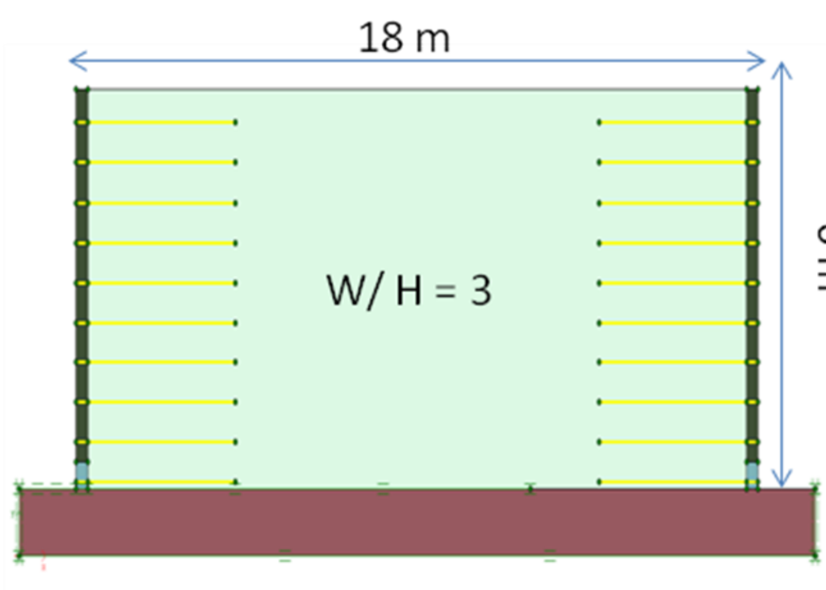


Fig 3.30: Case 3:- W/H = 3

3.7.2.1 For the case when $\varphi = 25^{\circ}$

Fig. 3.31 through 3.33 show the deformation profile, contour of displacements, and the horizontal displacement of facing panel with backfill friction angle equal to 25°. Fig. 3.34 shows the maximum tension mobilized in the reinforcement.

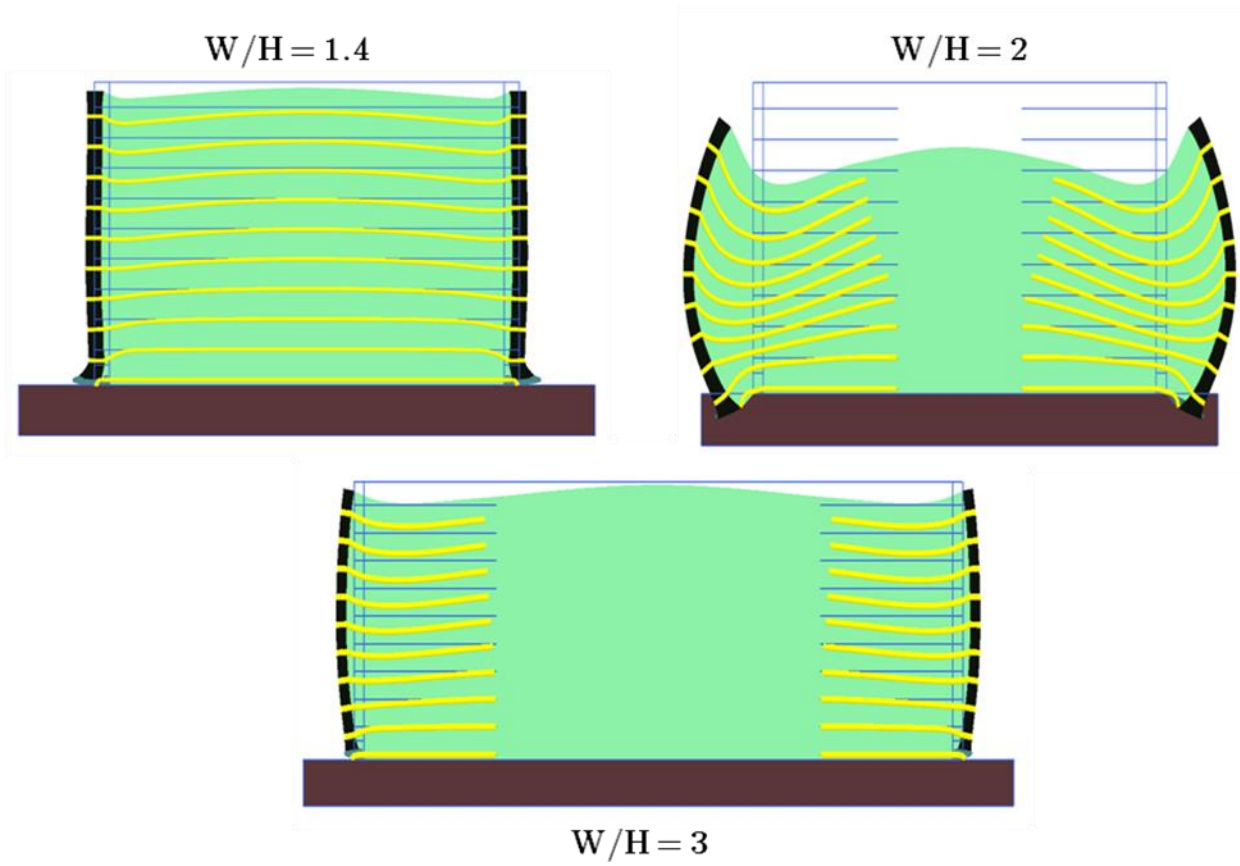


Fig 3.31: Deformation plots with backfill friction angle equal to 25°

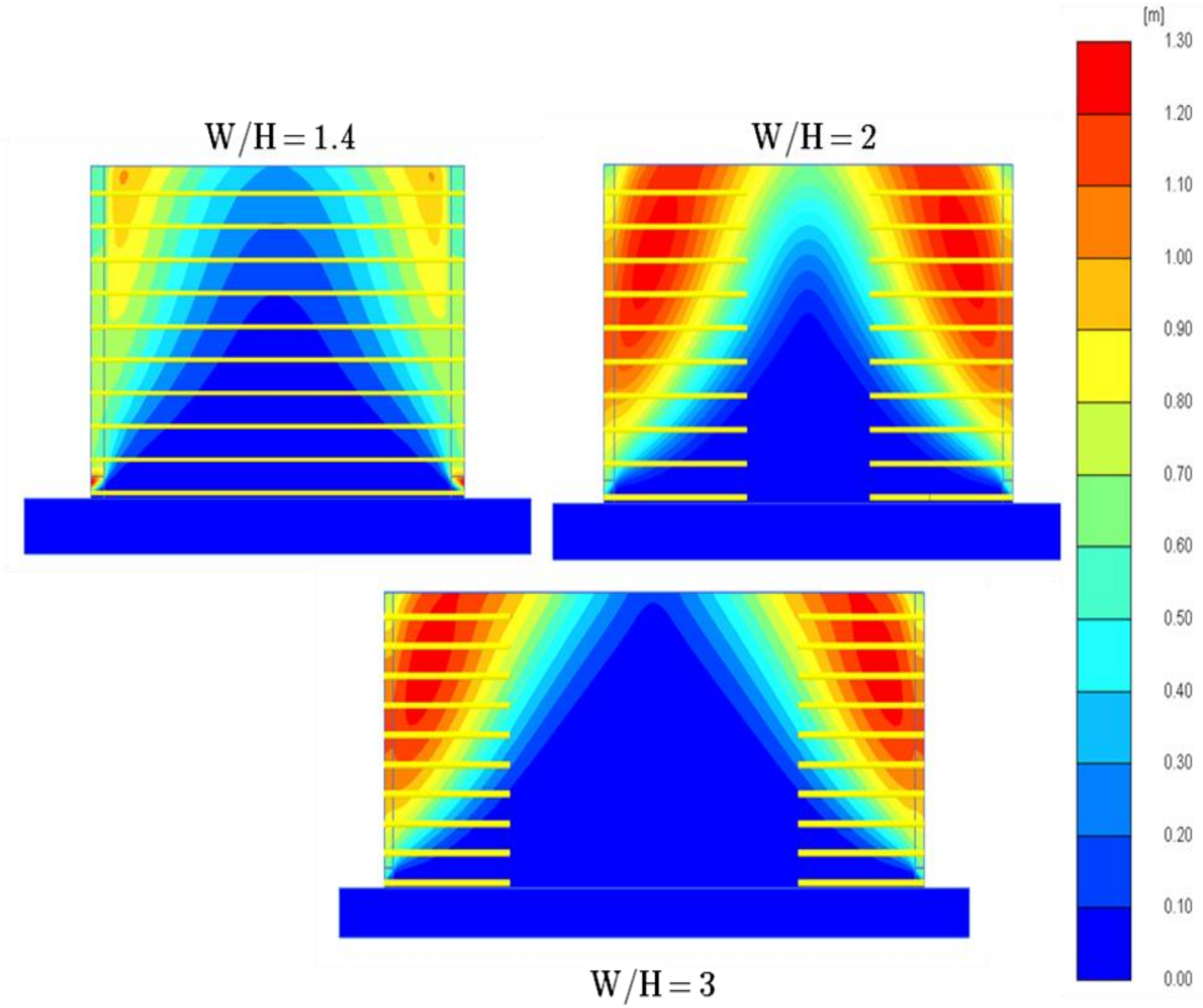


Fig 3.32: Contour plots of total displacements with backfill friction angle equal to 25°

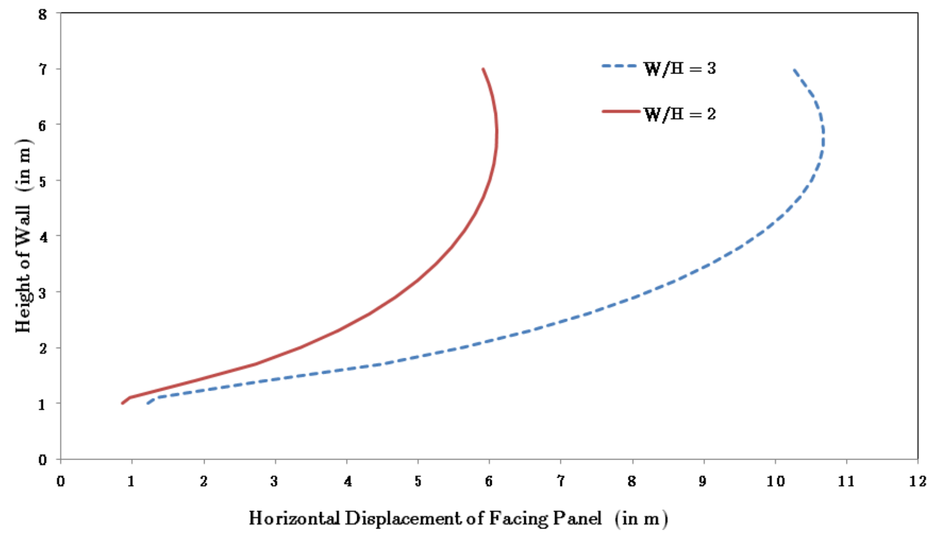


Fig 3.33: Facing Displacement vs height of wall with backfill friction angle equal to 25°

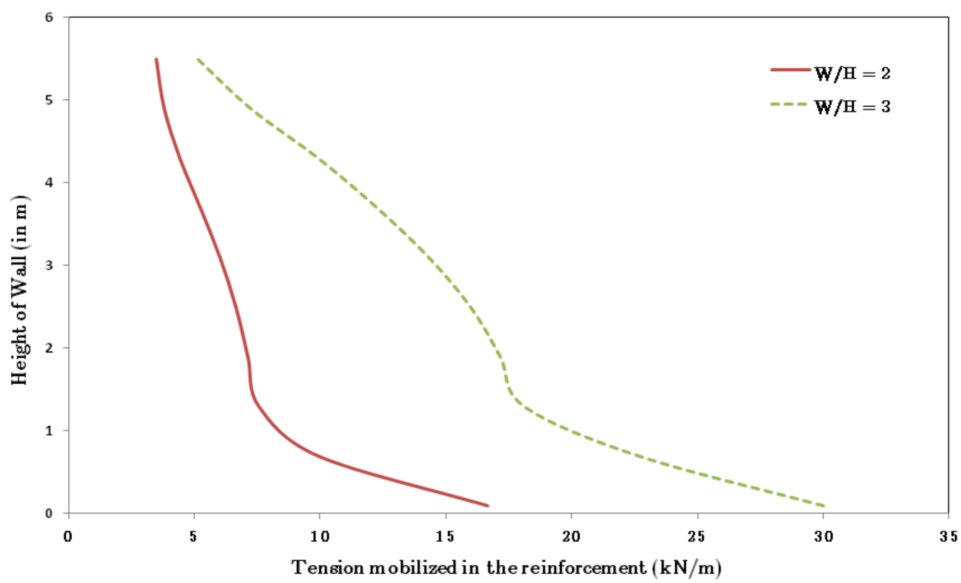


Fig 3.34: Maximum tensile force developed in the reinforcement or an internal friction angle 25°

3.7.2.2 For the case when $\varphi = 34^{\circ}$

Fig. 3.35 through 3.37 show the deformation profile, contour of displacements, and the horizontal displacement of facing panel with backfill friction angle equal to 34°. Fig. 3.38 shows the maximum tension mobilized in the reinforcement.

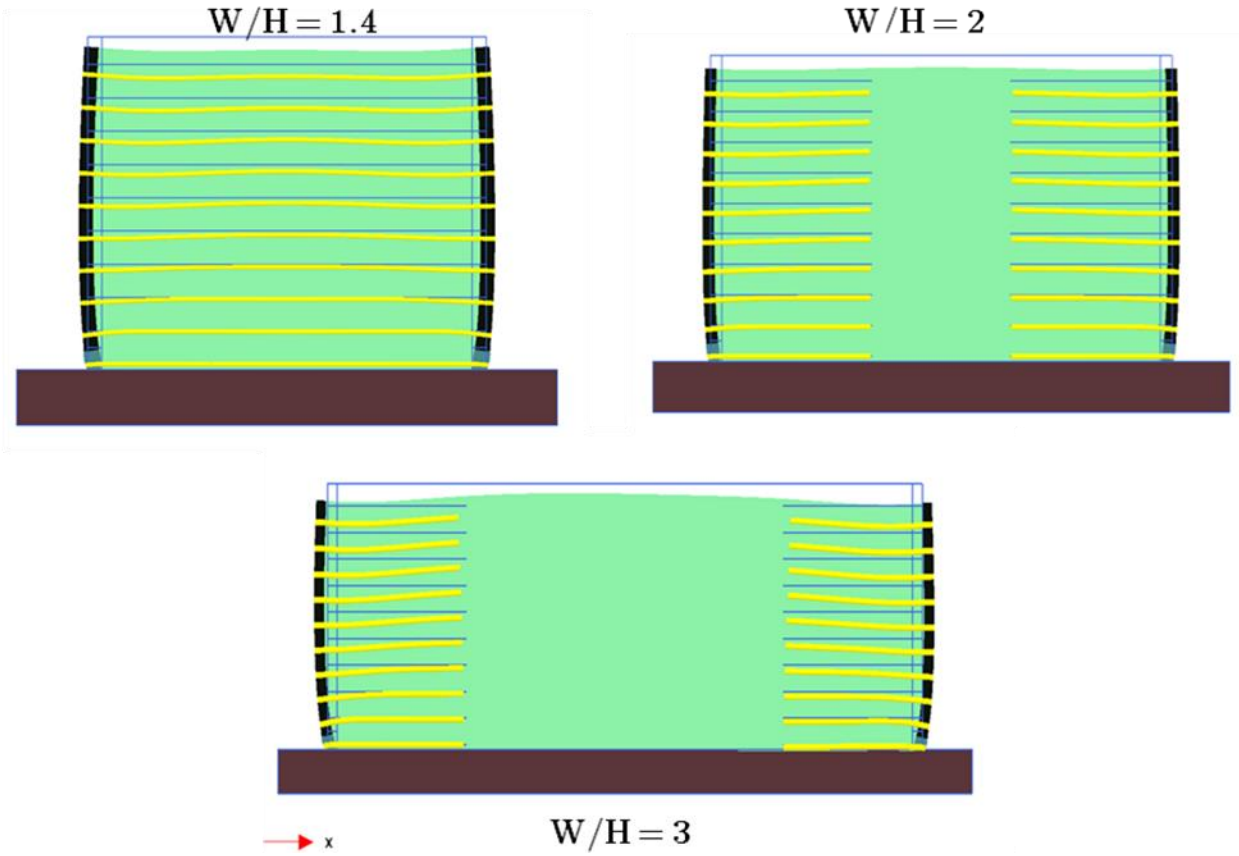


Fig 3.35: Deformation plots with backfill friction angle equal to 34°

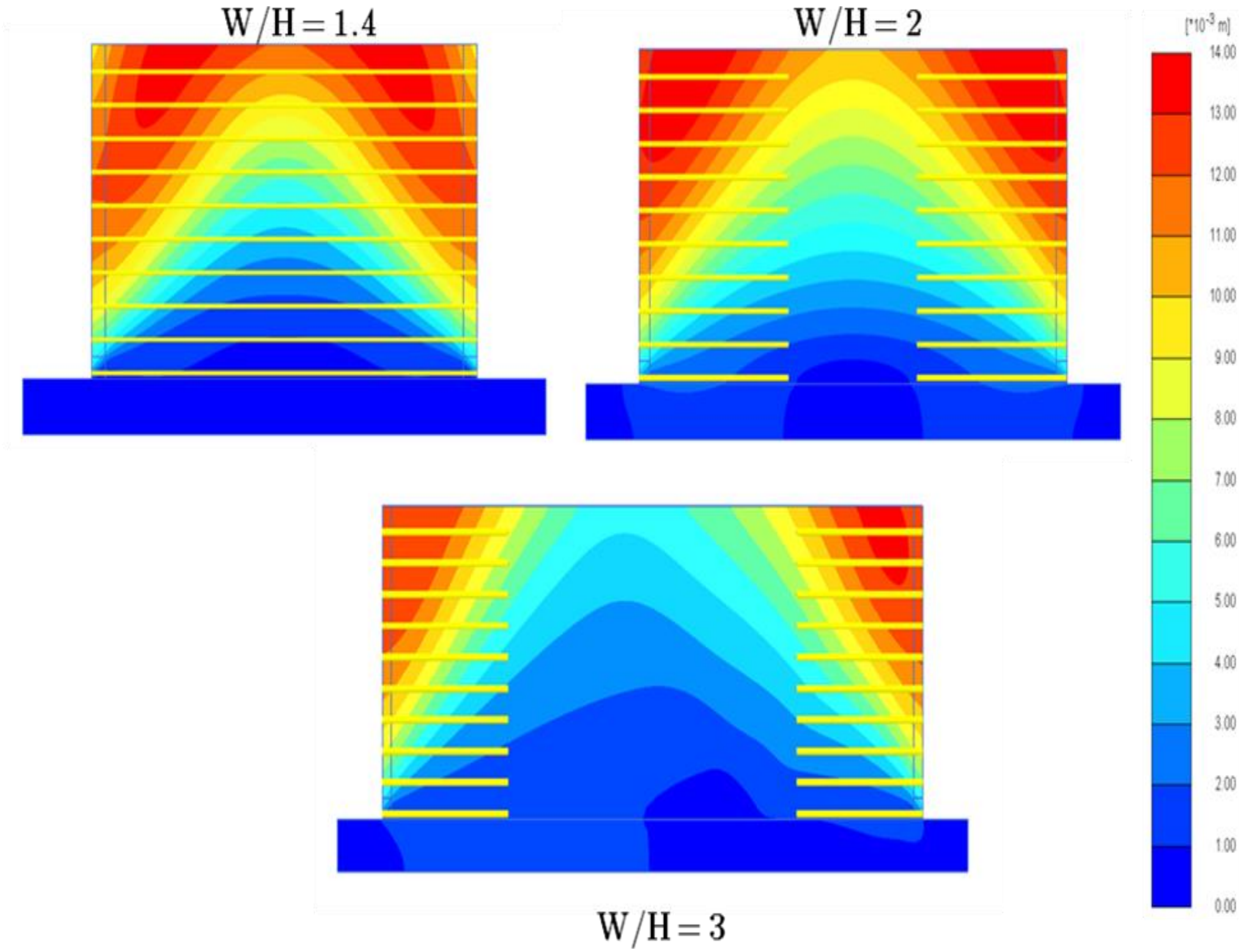


Fig 3.36: Contour plots of total displacements with backfill friction angle equal to 34°

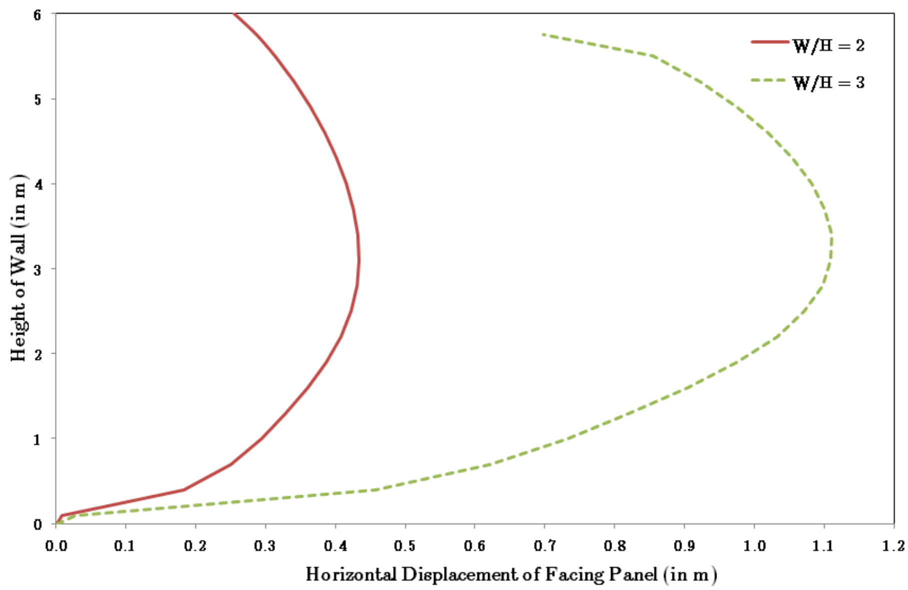


Fig 3.37: Facing Displacement vs height of wall with backfill friction angle equal to 34°

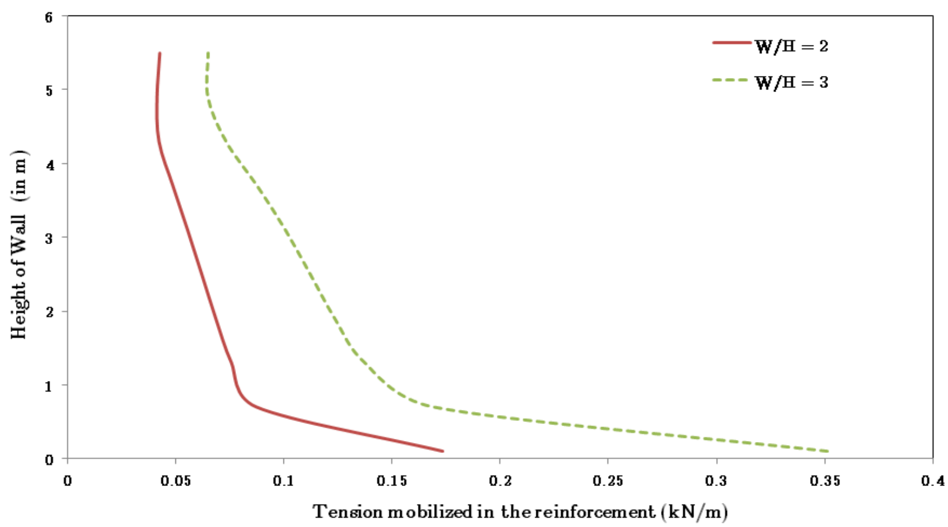


Fig 3.38: Maximum tensile force developed in the reinforcement or an internal friction angle 34°

3.7.2.3 Discussion of Results

It was observed that as the distance between the walls decreases, the resulting displacement also decreases, and a drastic reduction was observed displacement when the reinforcements are connected. It was evident from the contour plots that the critical surface interacts in case of connected reinforcements and the critical surface from one wall do not enter into the other when the W/H ratios are increased (Fig 3.32 and Fig 3.36). Horizontal displacements of wall facing increase as the distance of separation between the walls increases. It is evident from Fig 3.33 and Fig 3.37 that the displacement is more for a wall with low quality backfill ($\varphi=25^{o}$) than a wall with high quality backfill ($\varphi=34^{o}$). From Fig 3.34 and Fig 3.38, it is indicated that similar trend is observed in the case of forces mobilized in the reinforcement. Mobilized geosynthetic forces increases with an increase in the distance between the walls. Maximum tension was found to be high for backfill with low friction angle compared to a backfill with high friction angle.

Chapter 4

Modeling in FLAC 2D

4.1 Problem definition

Back-to-Back Mechanically Stabilized Walls are encountered in highway applications such as narrow ramps and turning lanes. However, available literature on design guidelines for Back-to-Back MSE walls is limited. This study was conducted to investigate the effect of reinforcing Back-to-Back MSE walls on foundation settlement, surface settlement, and horizontal displacement of facing panel. A finite difference method based computer program, FLAC (Fast Lagrangian Analysis of Continua), was used to perform the analysis of retaining wall. FLAC was selected for this research because of its excellent capability to model geotechnical engineering related stability problems and its extended programming ability. Although numerical analyses using FDM usually has longer iteration times than FEM, with the development of high-speed computers, this is not a major shortcoming. A wall of height equal to 6m was modeled with full length facing panels of nominal thickness. Backfill material properties were kept constant. Reinforcement was modeled as cable elements (two-dimensional structural elements). Interfaces were considered between soil and facing wall, and also between soil and reinforcement.

4.2 FLAC – an overview

FLAC is a two-dimensional, explicit finite difference program used for computations in the field of engineering mechanics. This program simulates the behavior of structures built of soil, rock, or other materials that may undergo plastic flow when yield limits are reached. Materials are represented by elements or zones which form a grid that is adjusted by the user to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or non-linear stress/strain law in response to the applied forces or boundary restraints. The material can yield and flow and the grid can deform (large-strain mode) and move with the material that is represented. The explicit Lagrangian scheme and the mixed discretization zoning ensure that the flow is modeled accurately.

4.3 Finite difference program

Finite Difference Method is perhaps the oldest numerical technique used for the solution of sets of differential equations (given initial values and/or boundary values). In the Finite Difference method, every derivative represented by a set of governing equation is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space. Figure 4.1 illustrates the general calculation procedure in FLAC.

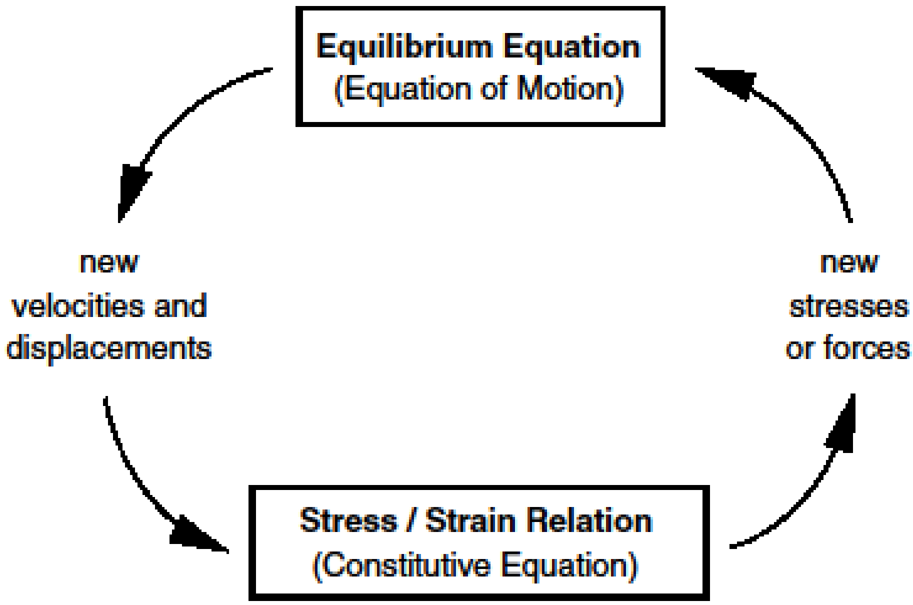


Figure 4.1: General calculation procedure in FLAC

The method first invokes the equations of motion to derive new velocities and displacements from stresses and forces. The strain rates are then derived from velocities, and new stresses are obtained from strain rates. It takes one time step for every cycle around the loop.

4.4 Lagrangian analysis

The incremental displacements are added to the co-ordinates so that the grid moves and deforms with the material it represents. This is termed as Lagrangian formulation. The constitutive formulation at each step is a small-strain formulation; but is equivalent to large-strain formulation over many steps.

4.5 Modeling retaining wall

4.5.1 Model Geometry

A back-to-back wall of height equal to 6m was modeled with full length facing panels on both sides. The width of the reinforced zone was taken as 12m.The reinforced soil was assumed to rest on a foundation soil of width and depth equal to 80m and 60m, respectively (Fig 4.2). The solid body is divided into a grid points composed of quadrilateral elements. Internally, FLAC subdivides each element into two overlaid sets of constant-strain, triangular elements. Reinforcements were attached to the facing panels. Standard Fixities were provided along the boundary. Interfaces are provided between the soil and facing panel, and also between reinforcement and soil. The facing panel is embedded to a depth equal to 0.5m into the foundation soil to ensure that there is no movement. Fig 4.3 shows an enlarged view of the model.

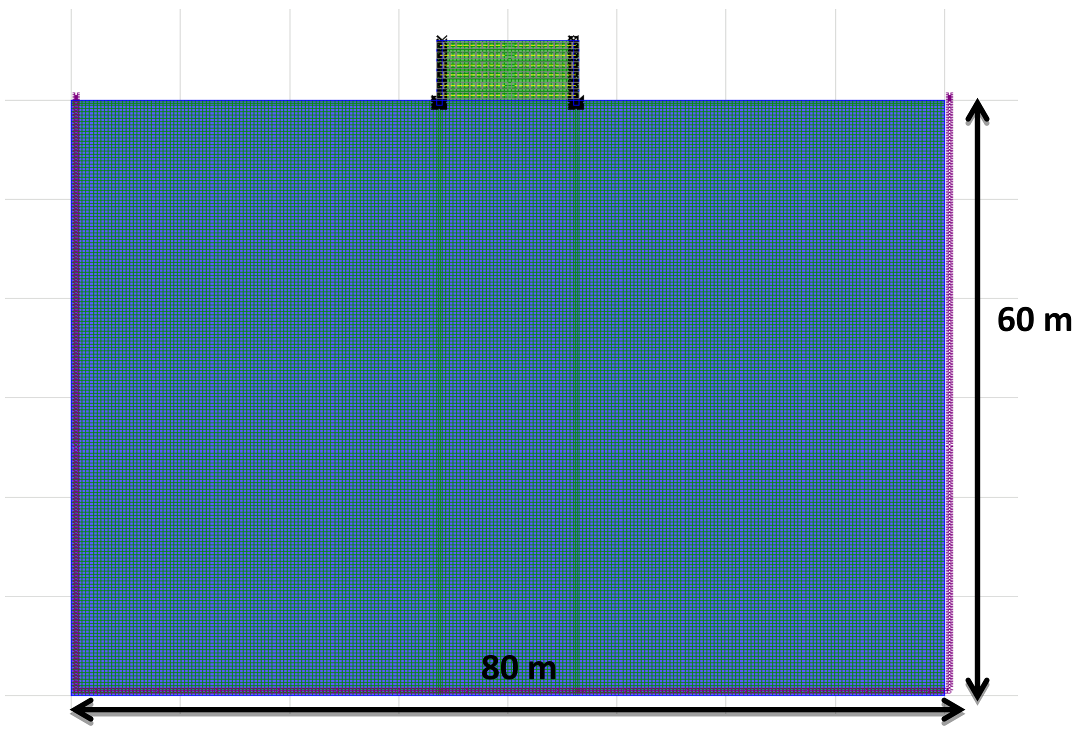


Fig 4.2: Reinforced Retaining Wall Model developed in FLAC

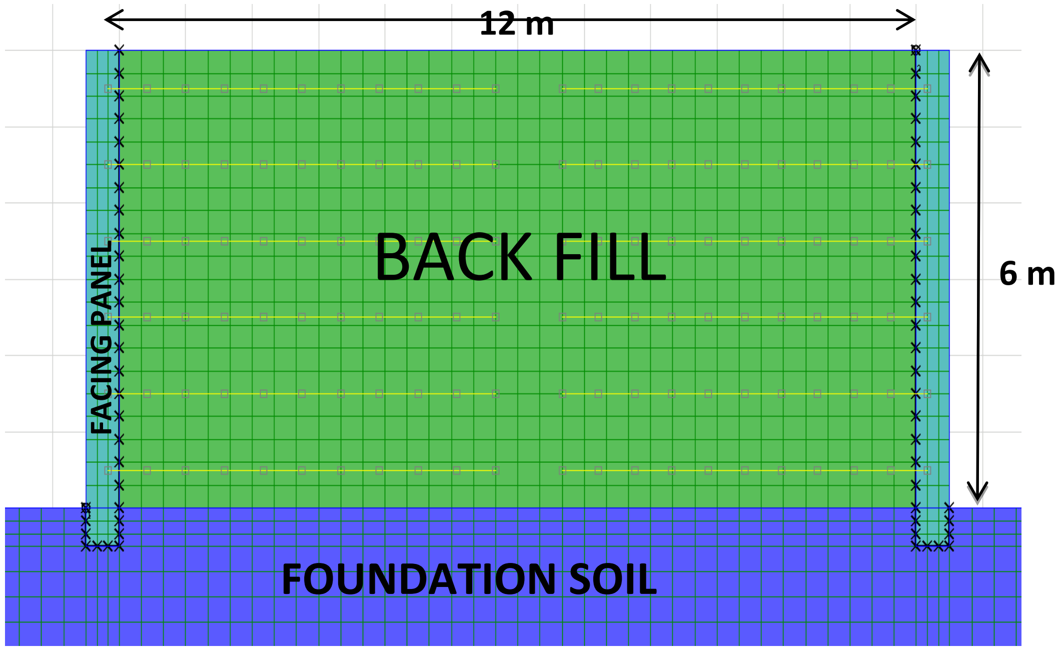


Fig 4.3: Enlarged View of the Backfill

4.5.2 Material Models

4.5.2.1 Soil

Both the foundation soil and back fill are modeled using Mohr-Coulomb criterion. The failure envelope for this model corresponds to a Mohr-Coulomb criterion (shear yield function) with tension cutoff (tensile yield function). The Mohr-Coulomb material model is an elasto-plastic material model. Facing panel was assigned properties corresponding to that of concrete.

Advantages of using the Mohr-Coulomb material model for the backfill include:

1. The model provides a yielding criteria (failure envelope) similar to the backfill material (soil), and

2. The input properties of Mohr-Coulomb material model can be obtained from routine soil strength tests such as direct shear test and triaxial test.

Table 4.1: Properties of Soil and Facing Wall

TYPE	MATERIAL	UNIT WEIGHT (kN/m^3)	MODULUS OF ELASTICITY E (Pa)	POISSON'S RATIO	COHESSION (kPa)	BACKFILL FRICTION ANGLE(°)
FOUNDATION SOIL	MOHR COULOMB	2400	1,00,000	0.3	5	40
BACKFILL	MOHR	1800	1,00,000	0.3	5	34

	COULOMB					
FACING	MOHR COULOMB	1800	285e8	0.3	1000	34

Table 4.1 gives the material properties of foundation soil, reinforced soil, and facing wall.

4.5.2.2 Reinforcements

Both beam and cable elements can be used to model reinforcements in FLAC. In this particular model, cable elements are used for this purpose. This is because of their ability to provide sufficient information on the reinforcement stress-strain distribution, as well as simpler interface input properties and better computational time efficiency compared to the beam elements. A cable element is made of an axial elastic material element (cable) with interface elements (grout) around it. When cable elements are used to model reinforcements, the properties assigned to the cable element automatically takes into account of the effect of interface between the reinforcement and soil. The advantages of using cable elements include simple geometry, direct axial stress-strain information, and time savings in computation.

The properties assigned to cable elements are as follows:

Modulus of Elasticity (E) = 125e6 Pa

Bond Stiffness = 3e7 N/m/m

Bond Strength = 0 kPa

Bond Friction angle = 25^{o}

Perimeter = 2m

Area = $0.004m^2$

Tensile Strength = 2e6N

4.5.2.3 Interfaces

Interfaces are planes upon which slip/separation are allowed, that simulates the presence of faults, joints or frictional boundaries. FLAC provides interfaces that are characterized by Coulomb sliding and/or tensile separation to enable the simulation of the contacts between different materials. Interfaces have the properties of friction, cohesion, dilation, and normal and shear stiffness, and tensile strength. Interface elements can be modeled to enable full interaction between the structural objects and the surrounding soil.

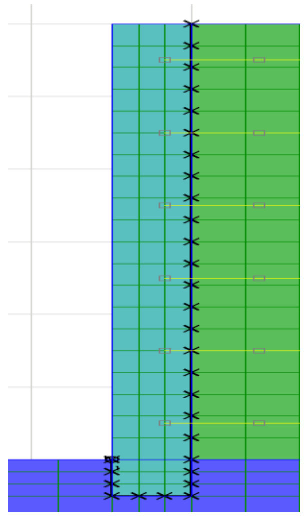


Fig 4.4: Enlarged view of interfaces used in the model.

In the current model, interfaces were provided between reinforced soil and the wall facing element, and also between foundation soil and wall facing element (Fig 4.4). The following properties of interface are given:

Normal Stiffness: 1e7 Pa/m

Shear Stiffness: 1e5 Pa/m

Cohesion = 10 Pa

Dilation Angle = 11°

Friction Angle = 25°

In this study, the normal stiffness of all interfaces was assigned with a very high value, i.e., at least ten times the stiffness of the materials that were connected by the interfaces. This modeling technique was used to avoid material elements penetrating the interfaces and causing numerical instability.

4.5.3 Boundary Conditions

Fixities are defined as prescribed displacements at geometry line with displacement equal to zero. Fixity can be provided by selecting the geometry line and applying either horizontal (u_x=0), vertical (u_y=0), and total fixity (u_x=u_y=0), or by selecting the geometry line and applying a standard boundary condition available. By selecting standard boundary condition, FLAC automatically implies a set of the following boundary conditions:

a) Vertical geometry lines for which the *x*- coordinate is equal to the lowest or highest *x*-coordinate (right and left boundaries of the model) in the model obtain a horizontal fixity (u_x=0).

b) Horizontal geometry lines for which the *y*-coordinate is equal to the lowest *y*-coordinate (bottom boundary) in the model obtain a full fixity ($u_x = u_y = 0$).

Standard Boundary Conditions were adopted in the model.

Special care was also taken while setting boundary conditions to avoid any artificial reaction forces, or deformations to occur. Since settlement of the retaining structure is a major issue, the foundation soil was extended to a depth that would cover all the compressible soil layers that might contribute to the total settlement of the structure.

Hence studies were done to fix the boundaries (left, right, and bottom) of foundation soil by varying the depth from 30m to 60m, and width from 50m to 80m.

From the results obtained, the width of the foundation soil was fixed as equal to 60m and depth as 80m.

4.5.4 Meshing

High-quality meshes are crucial for the stability, accuracy, and fast convergence of numerical simulations. Element shape is an important parameter in obtaining accurate results. Since mesh generation can be a very time consuming process, it is also necessary to be able to judge if a given mesh will perform well enough for a given model, or if more effort needs to be made to improve its quality. FLAC also provides provision to increase the accuracy of meshing the model by adding multipliers to the virtual zone density. In this model, mesh convergence studies were done for the model, and fine mesh with a mesh density ratio of 4*4 was adopted for the analysis. That is the mesh refinement was done four times finer in both x and y direction.

4.5.5 Equilibrium criteria

To ensure that the results of the numerical analysis have converged and that the unbalanced forces within individual material elements have been minimized, it is necessary to set up equilibrium criterion. Each grid point in the FLAC model is surrounded by up to four material elements. The algebraic sum of forces contributed by these surrounding elements at any specific grid point should converge to zero when the model reaches equilibrium. This algebraic sum of forces acting on the grid point is defined as the unbalanced force. The Steady State Ratio is defined as the ratio of maximum unbalanced force to the representative internal force. Limits were set on the unbalanced force and/or stress ratio of the material elements as the equilibrium criteria. That is a system is considered to be in equilibrium only when the maximum value of Steady State Ratio is 10e-3. In the current model, the value of Steady State Ratio (SSR) was approximated to be 0.0010.

4.6 MSEW Software

MSEW Software was used as a design aid for the design and analysis of Mechanically Stabilized Earth Walls.

A reinforced retaining wall of height equal to 6m was modeled in MSEW software with an embedment depth of 0.5m. Tables 4.2 through 4.4 give the properties of soil and geogrid. Full height precast concrete panel was adopted for the facing. An uniformly distributed load was applied over the reinforced soil zone.

Table 4.2: Soil Properties used for modeling

SOIL PROPERTIES			
Soil	**Unit Weight (in kN/m3)**	**Friction Angle**	**Cohesion (kPa)**
Reinforced Soil	18	34^{o}	0
Foundation Soil	24	40^{o}	0
Facing	18	34^{o}	1000
WATER TABLE DOES NOT AFFECT THE BEARING CAPACITY			

Table 4.3: Geogrid Properties

GEOGRID PROPERTIES	T_{ult}	500kN/m
	Coverage Ratio	1
	Friction Angle Between Geogrid-Soil Interface(ρ)	25^{o}
	Pullout Resistance Factor	0.7tan(φ)
	Scale Effect Correction Factor(α)	0.8

Table 4.4: Wall Geometric Properties

WALL PROPERTIES	Design Height(including embedment depth)	6.5m
	Embedment Depth	0.5m
	Batter	0^{o}
	Back Slope (β)	0^{o}
	Back Slope Rise	0m

Table 4.5: Loading on the Wall

UNIFORM SURCHARGE	
Uniformly Distributed Dead Load	22 kPa
Uniformly Distributed Live Load	12 kPa

Table 4.6: Reinforcement Data

TRIALS WERE CARRIED OUT BY VARYING THE NUMBER OF REINFORCEMENTS		
REINFORCEMENT	**NUMBERS**	**LENGTH**
TYPE 1	20	4.5
TYPE 2	10	4.5
TYPE 3	6	4.5

Table 4.7 *Results* from Analysis

ANALYSIS OF RESULTS	TYPE	FACTORS OF SAFETY		
		BEARING CAPACITY	**SLIDING**	**PULLOUT** (Minimum Pullout resistance for topmost reinforcement)
	TYPE 1	41.52	4.12	7.58
	TYPE 2	22.58	2.53	3.53
	TYPE 3	***14.58***	***2.105***	***2.362***

Table 4.5 and Table 4.6 indicate the data input for loading and reinforcement data used in the MSEW software. Table 4.7 represents a summary of the values of Factors of Safety obtained for three different numbers of reinforcement. After analyzing the results, it was concluded that six layers of reinforcement of length equal to 4.5m with an ultimate tensile capacity of 500kN/m can be used for the analysis, and the same wall was considered for further modeling and analysis of back-to-back walls in FLAC 2D software. The vertical spacing between reinforcements was taken as 1.0m.

4.7 Load Application

The same model was used throughout the analysis. A uniform load of 34kPa was then applied along the width of reinforced soil zone. This was done to simulate the practical condition above the soil layer. The pavement system was modeled using a surcharge of 21.5kPa. The traffic surcharge is taken as 11.5kPa based on AASHTO guidelines that recommend surcharge of height equal to 0.67m and unit weight equal to 18.9kN/m^3 to represent the traffic loading at the top of embankment. The total surcharge used to represent the traffic and pavement was obtained by adding together the pavement surcharge of 21.5kPa and traffic surcharge of 11.5kPa (which equals 33kPa).

The model was also subjected to gravity loading. Large-strain movements were also allowed in the model before solving.

4.8 Results and discussions

4.8.1 Wall height – 6m

4.8.1.1) Comparison 1 – Single facing, Unconnected Back-to-Back and Connected Back-to-Back walls

A comparison study was done for the above three cases; i.e.

a.) Reinforced Retaining Walls with Single Facing (Fig 4.5)
b.) Back-to-Back Unconnected Reinforced Retaining Walls(Fig 4.6)
c.) Connected Back-to-Back Reinforced Retaining Walls(Fig 4.7)

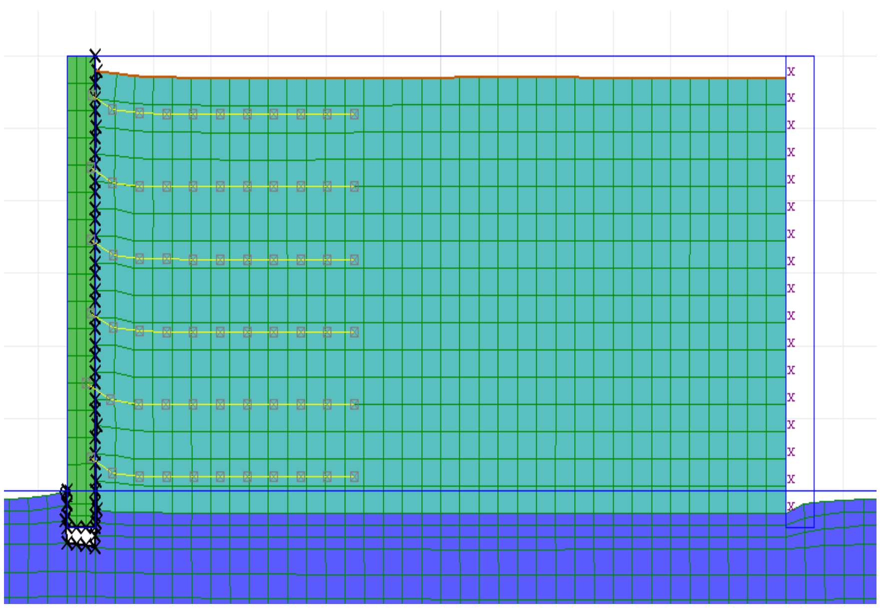


Fig 4.5: MSE Walls with single facing

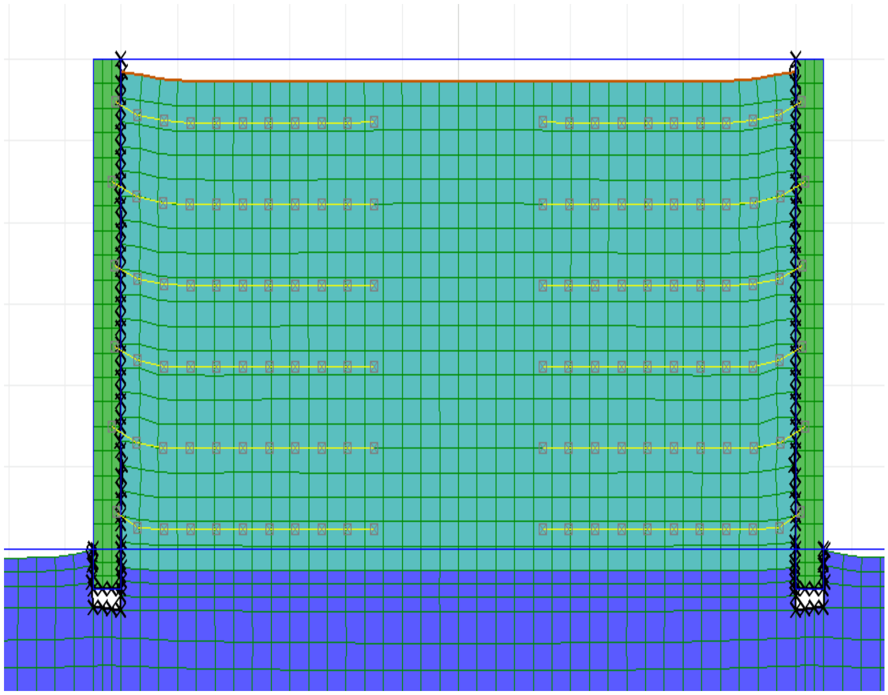


Fig 4.6: Back-to-Bach Unconnected MSE Walls

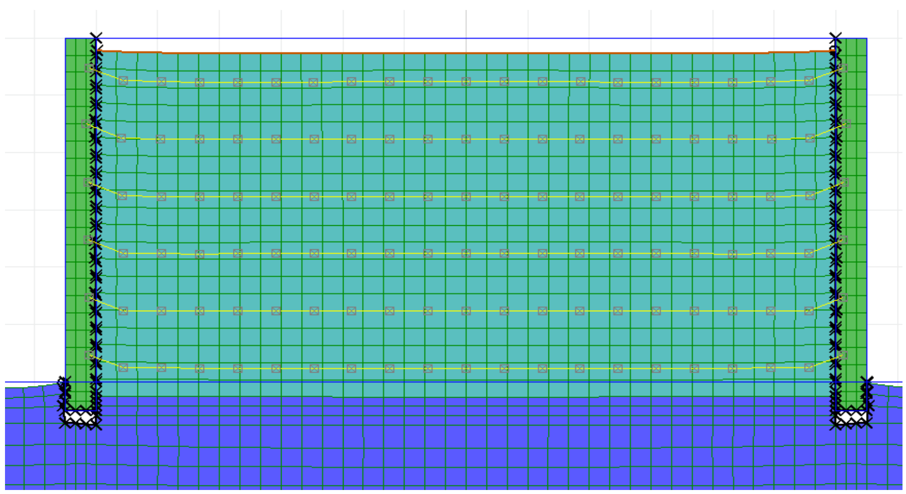


Fig 4.7: Connected Back-to-Back MSE Walls

After analyzing the above cases, a comparison was done for the Foundation settlement (Fig 4.8), Surface settlement (Fig 4.9) and Horizontal Displacement of Facing Panel (Fig 4.10). The results obtained are shown in the form of plots.

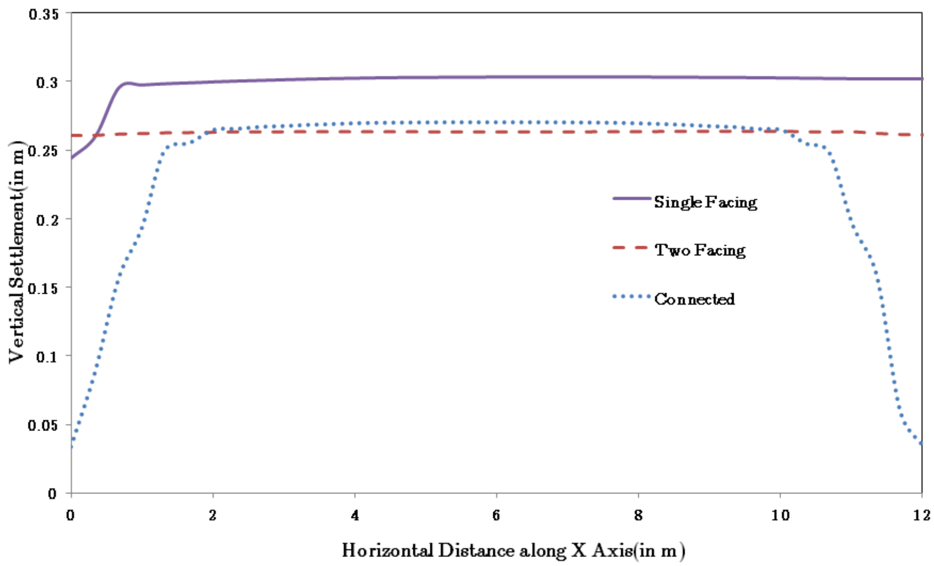


Fig 4.8: Foundation Settlement Analysis for different types of MSE Walls

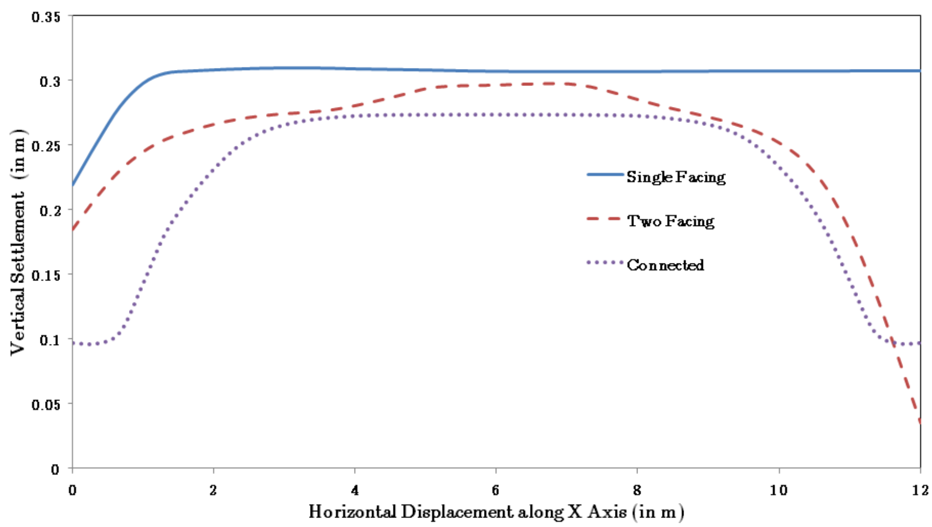


Fig 4.9: Surface Settlement Analysis for different types of MSE Walls

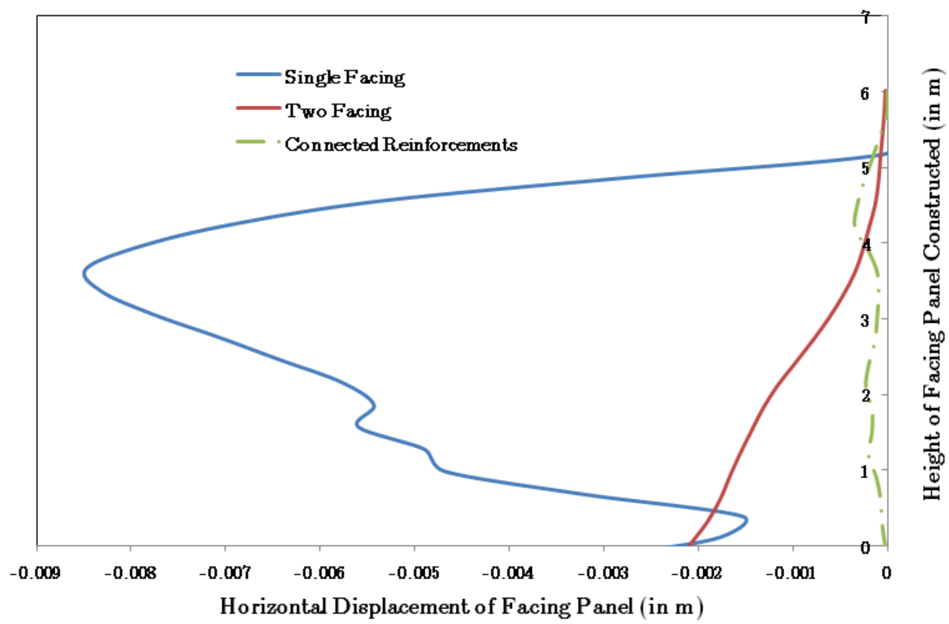


Fig 4.10: Facing Panel Displacement Analysis for different types of MSE Walls

4.8.1.2 Comparison 2: Varying the spacing between Reinforcements

Comparison studies were done by varying the spacing between the reinforcements, i.e., by increasing the lengths of reinforcements. Trials were done for mainly four cases.

a.) Case 1: Length of reinforcement = 4.5m (spacing = 3m). The model is represented in Fig. 4.11
b.) Case 2: Length of reinforcement = 5.5m (spacing = 1m). The model is represented in Fig. 4.12
c.) Case 3: Length of reinforcement = 5.9m (spacing = 0.2m). The model is represented in Fig. 4.13
d.) Case 4: Connected Reinforcements. Fig 4.14 represents the model for this case

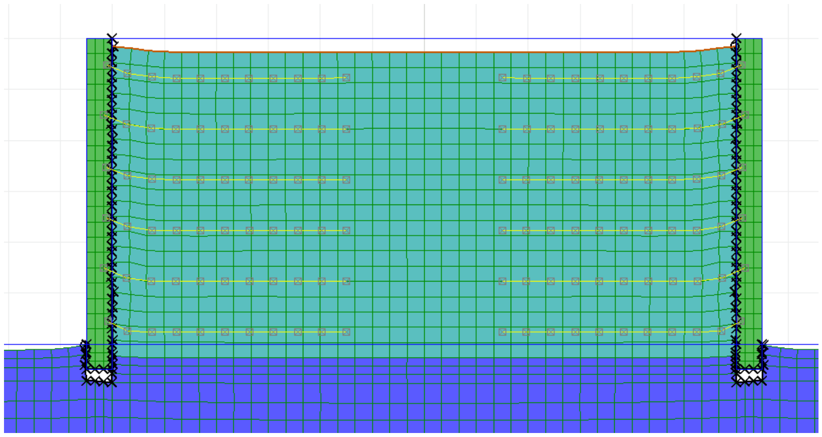


Fig 4.11: Spacing = 3m

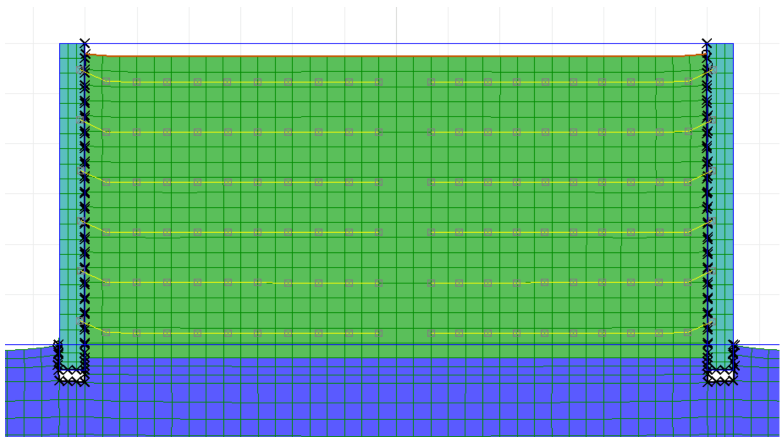


Fig 4.12: Spacing = 1m

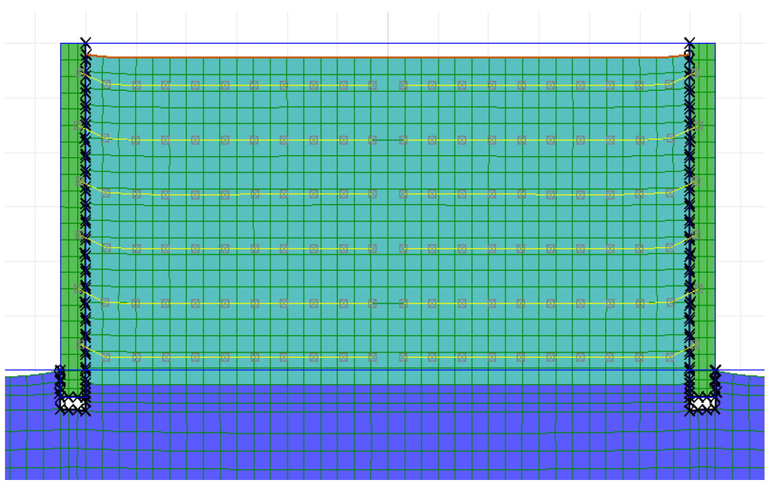


Fig 4.13: Spacing = 0.2m

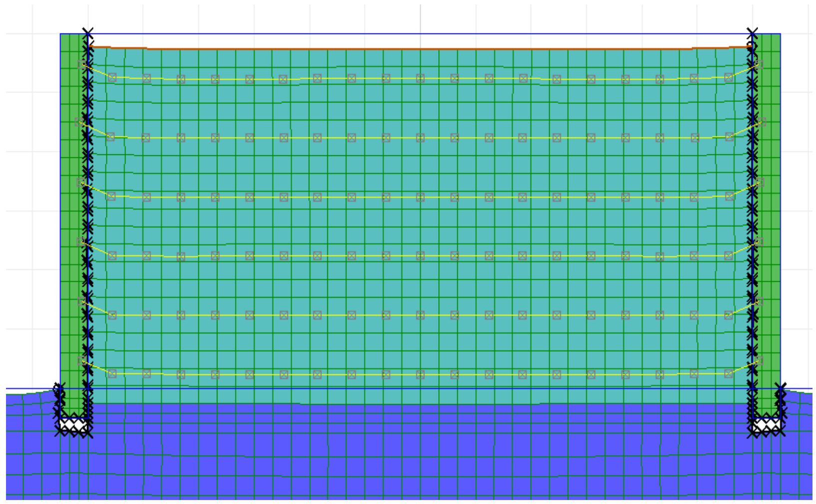


Fig 4.14: Connected Reinforcements

As in the previous cases, Foundation Settlement, Surface Settlement and Facing Panel Displacements were compared for all the four cases. Fig 4.15, 4.16, and 4.17 illustrates the comparisons.

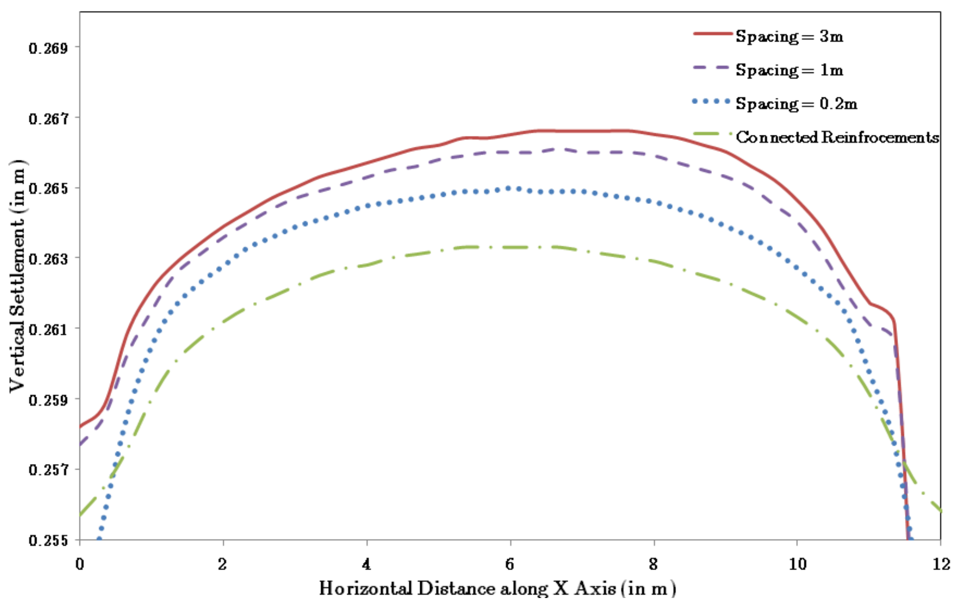


Fig 4.15: Foundation Settlement Analysis for different spacing of Reinforcements

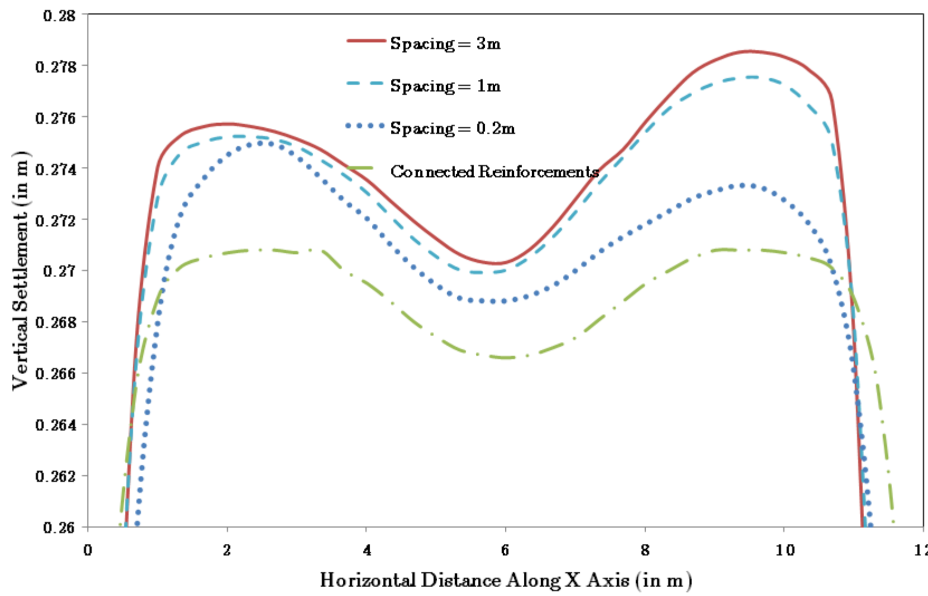


Fig 4.16: Surface Settlement Analysis for different spacing of Reinforcements

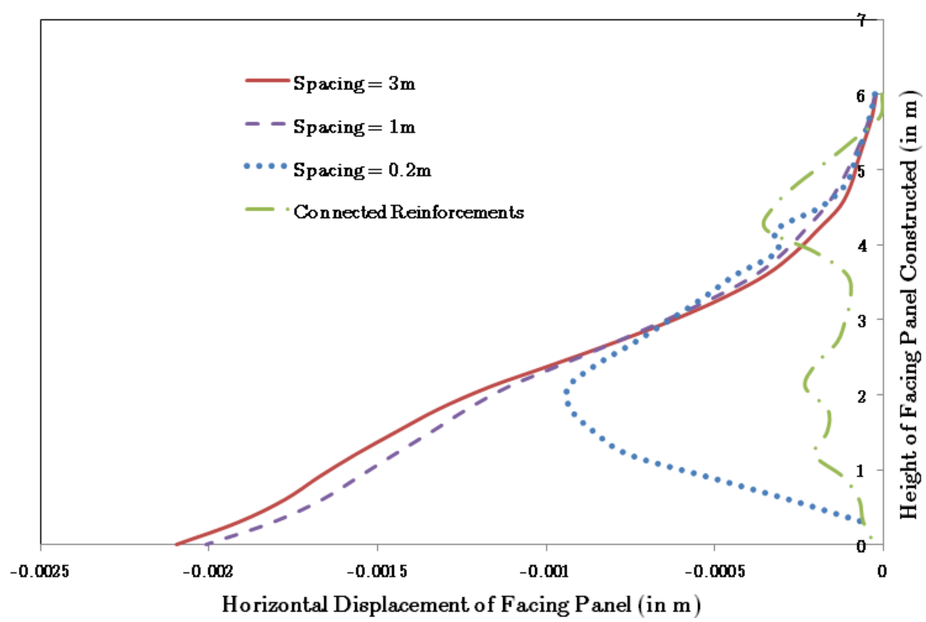


Fig 4.17: Facing Displacement Analysis for different spacing of Reinforcements

4.8.1.3 Comparison 3: Varying the Stiffness of Reinforcements

A parametric study was done for different stiffness of reinforcement, i.e., by changing the reinforcement stiffness from 500kN/m to 50,000kN/m. A comparison was done for the same properties. Fig 4.18 illustrates a comparison made for the foundation settlement. Fig 4.19 and Fig. 4.20 illustrate the comparison for surface settlement and horizontal displacement of facing panel.

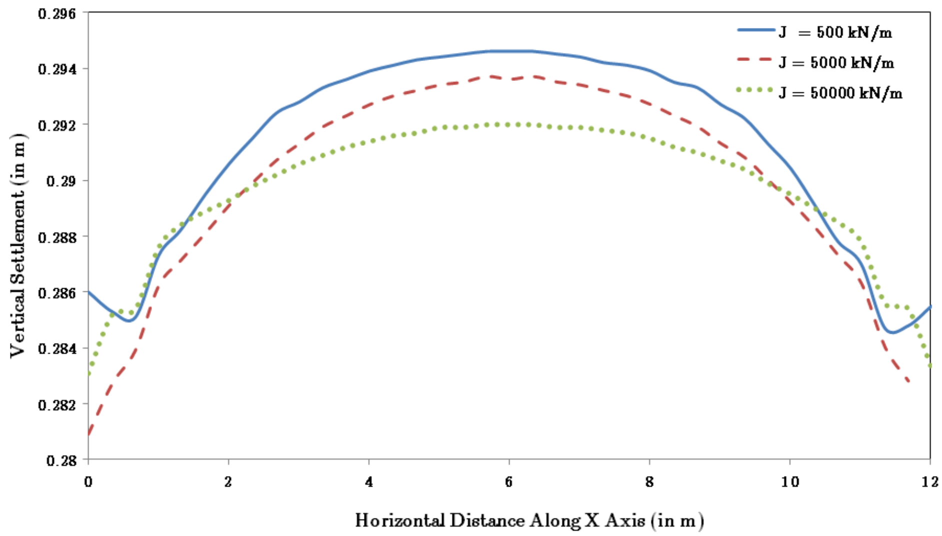


Fig 4.18: Foundation Settlement Analysis for different stiffness of Reinforcements

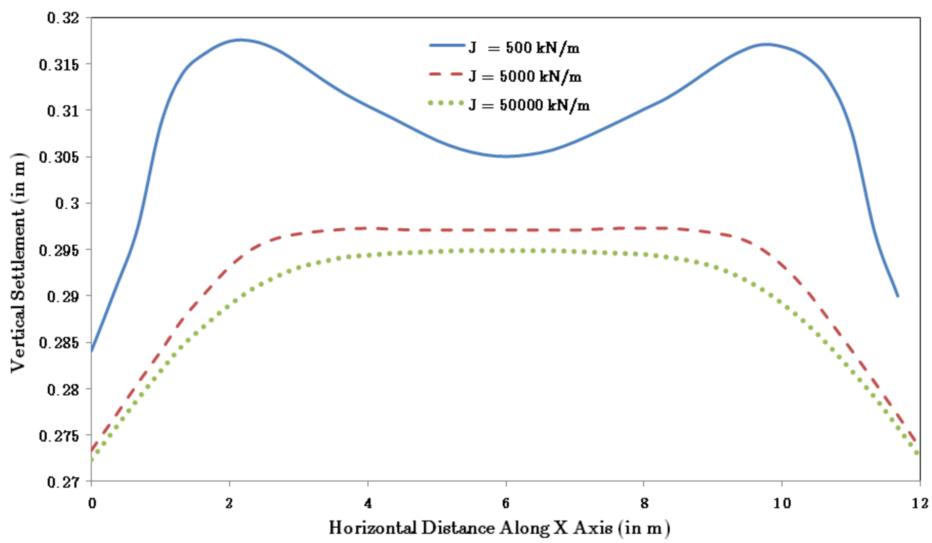


Fig 4.19: Surface Settlement Analysis for different stiffness of Reinforcements

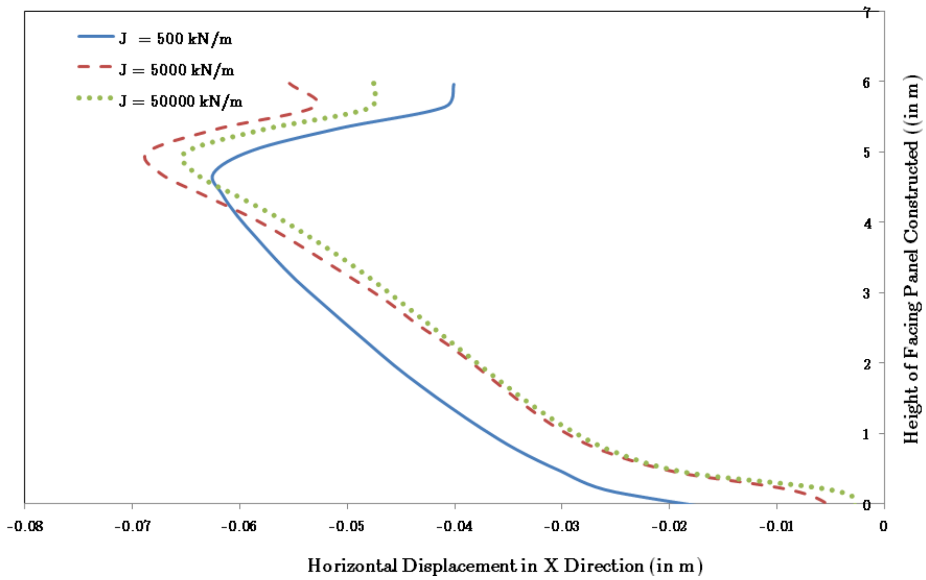


Fig 4.20: Facing Displacement Analysis for different stiffness of Reinforcements

4.8.1.4 Comparison 4: Varying the friction angle of Backfill

Comparison for foundation settlement, surface settlement and facing displacement by varying the friction angle of reinforced soil with all the properties kept the same. Three values were chosen for the analysis, i.e., $\varphi=28^{\circ}$, $\varphi=34^{\circ}$ and $\varphi = 38^{\circ}$. Results obtained are given in Figure 4.21 to 4.23.

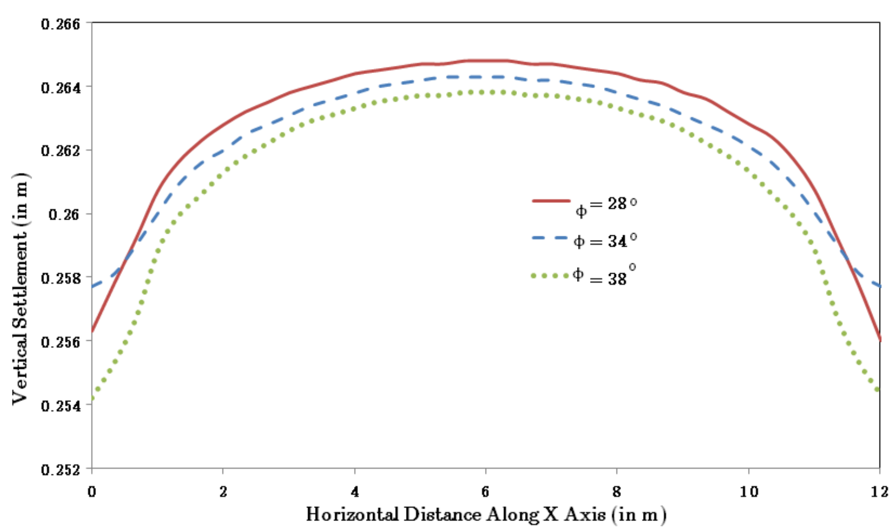


Fig 4.21: Foundation Settlement Analysis for different values of friction angle of backfill

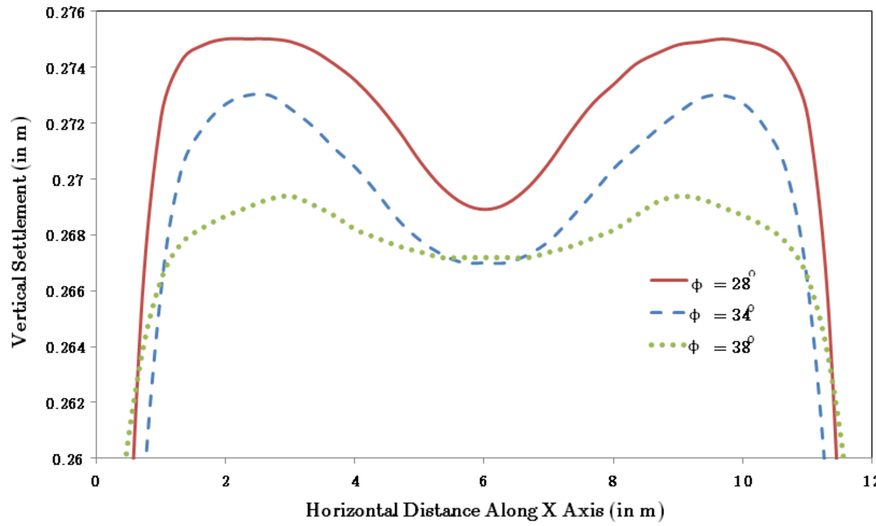


Fig 4.22: Surface Settlement Analysis for different values of friction angle of backfill

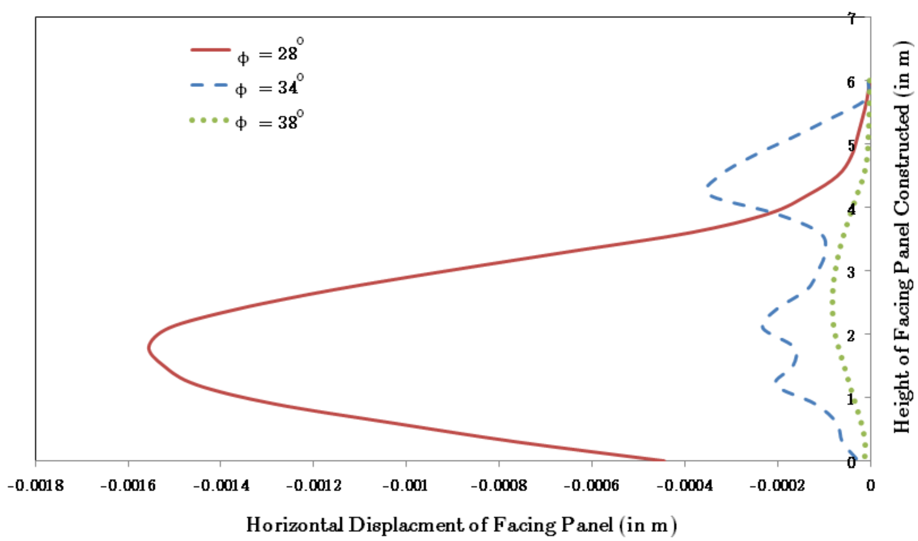


Fig 4.23: Facing Displacement Analysis for different values of friction angle of backfill

4.8.1.5 Comparison 5: Varying the Friction Angle of Foundation Soil

Trials were conducted for two different values for the friction angle of foundation soil, viz. 30° and 40°. The results of the analysis are as follows (Fig 4.24, 4.25 and 4.26).

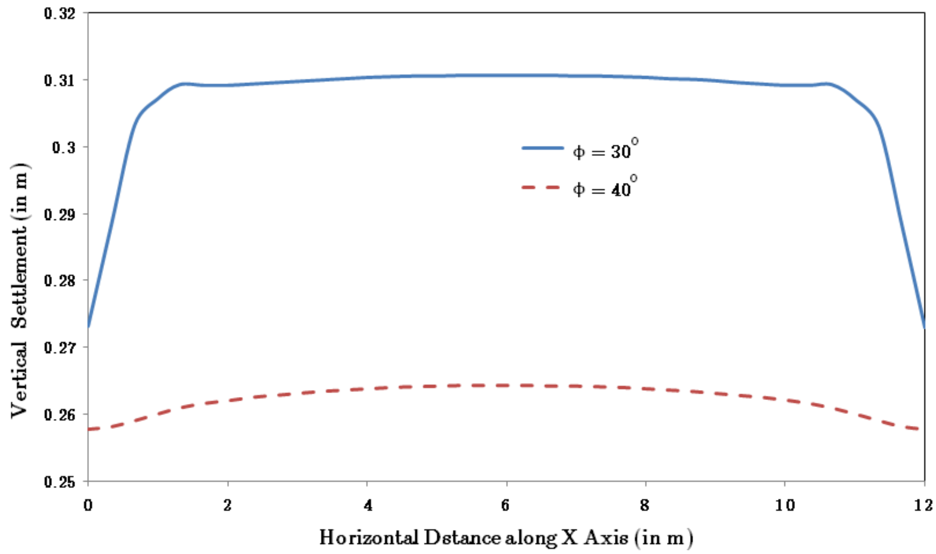


Fig 4.24: Foundation Settlement Analysis for different values of friction angle of foundation soil

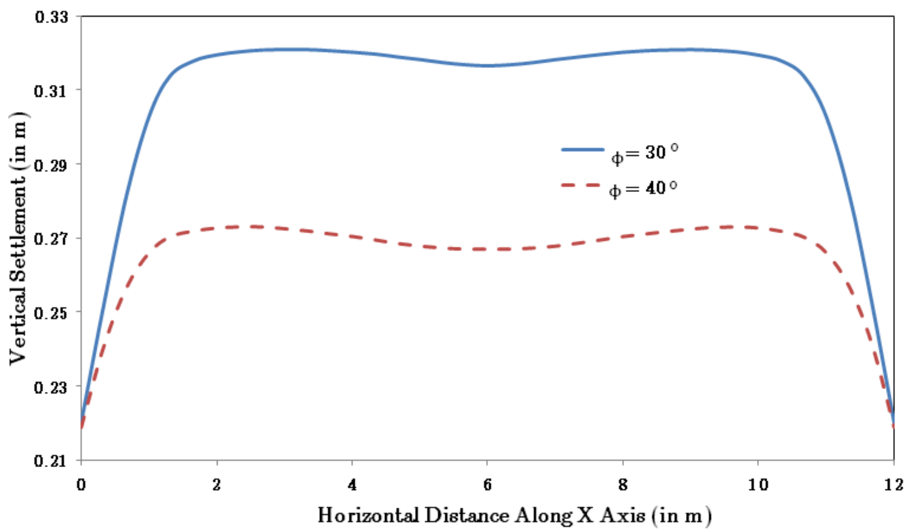


Fig 4.25: Surface Settlement Analysis for different values of friction angle of foundation soil

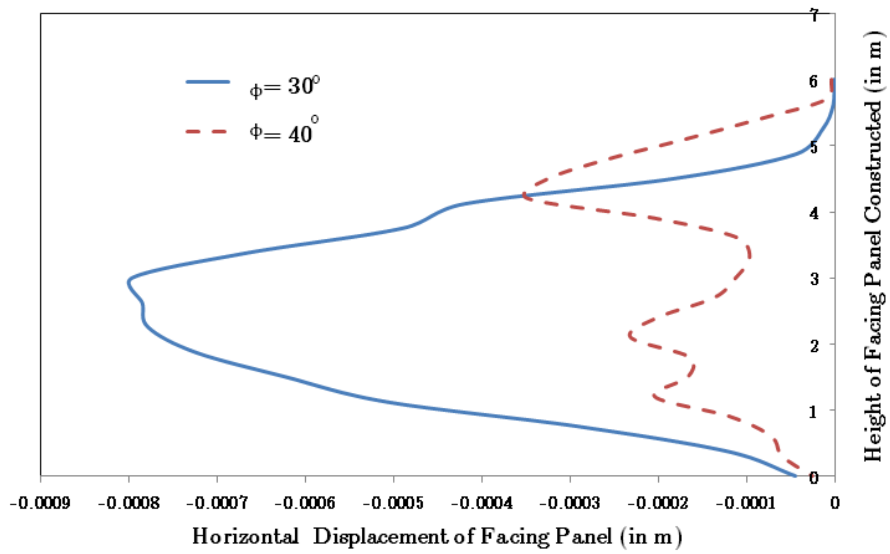


Fig 4.26: Facing Displacement Analysis for different values of friction angle of foundation soil

Similar comparisons were done for wall heights of 10m and 15m.

Results obtained are given below (Fig. 4.27 through 4.41).

4.8.2 Wall height – 10m

4.8.2.1 Comparison 1 – Single facing, Unconnected Back-to-Back and Connected Back-to-Back walls

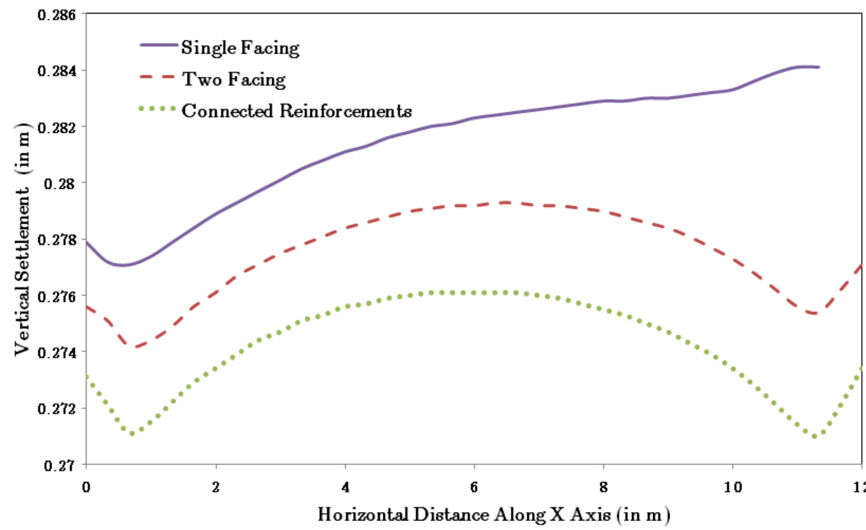


Fig 4.27: Foundation Settlement Analysis for different types of facing

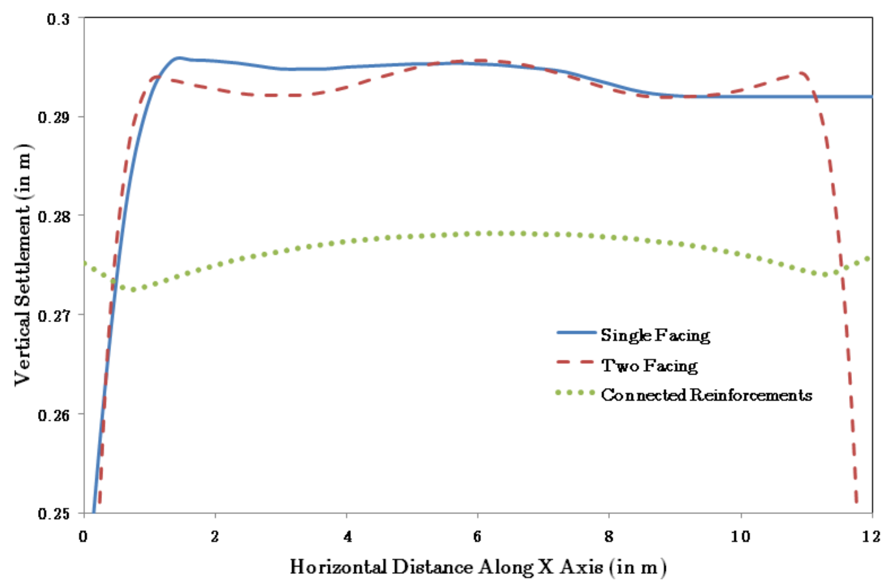


Fig 4.28: Surface Settlement Analysis for different types of facing

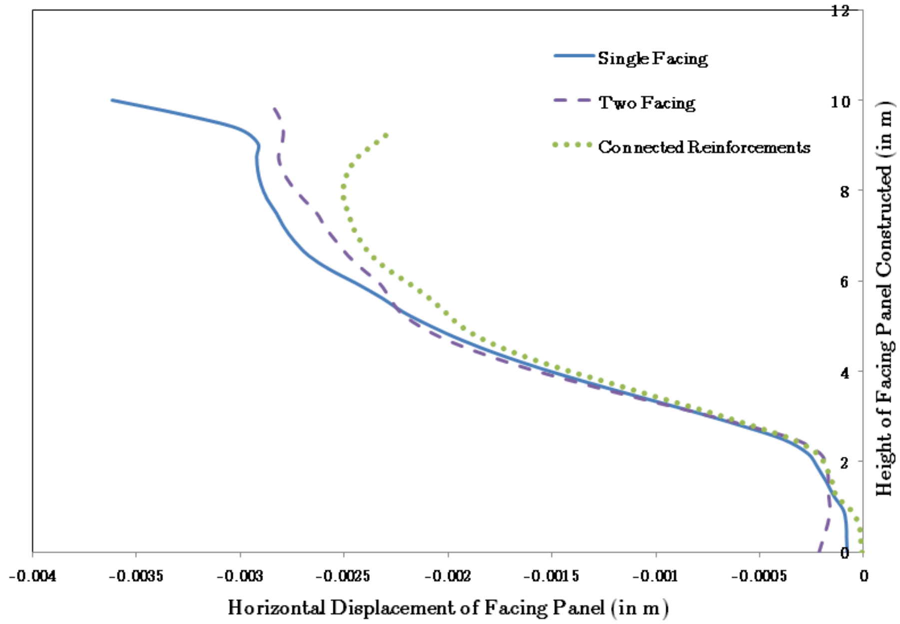


Fig 4.29: Facing Displacement Analysis for different types of facing

4.8.2.2 Comparison 2: Varying the spacing between Reinforcements

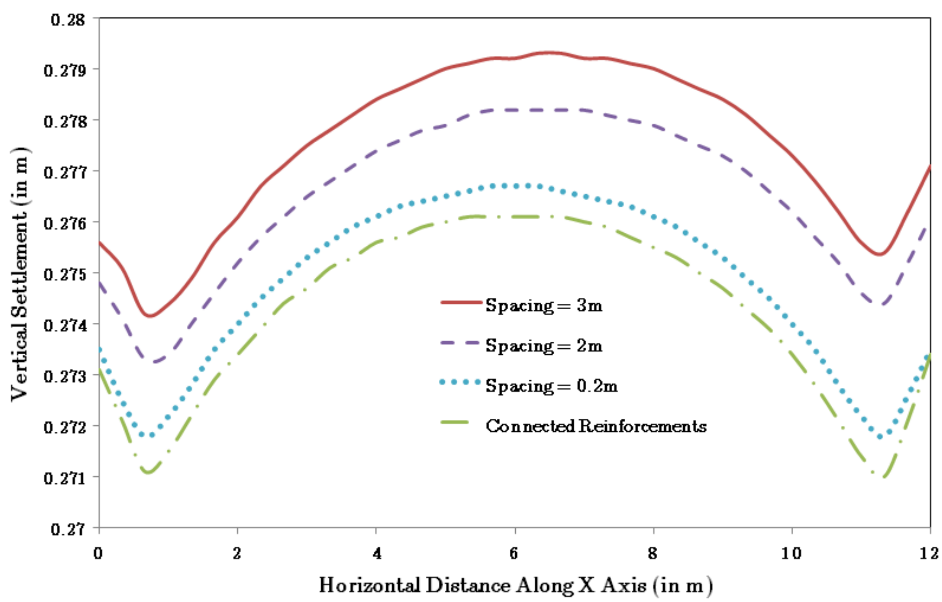


Fig 4.30: Foundation Settlement Analysis for different spacing of reinforcement

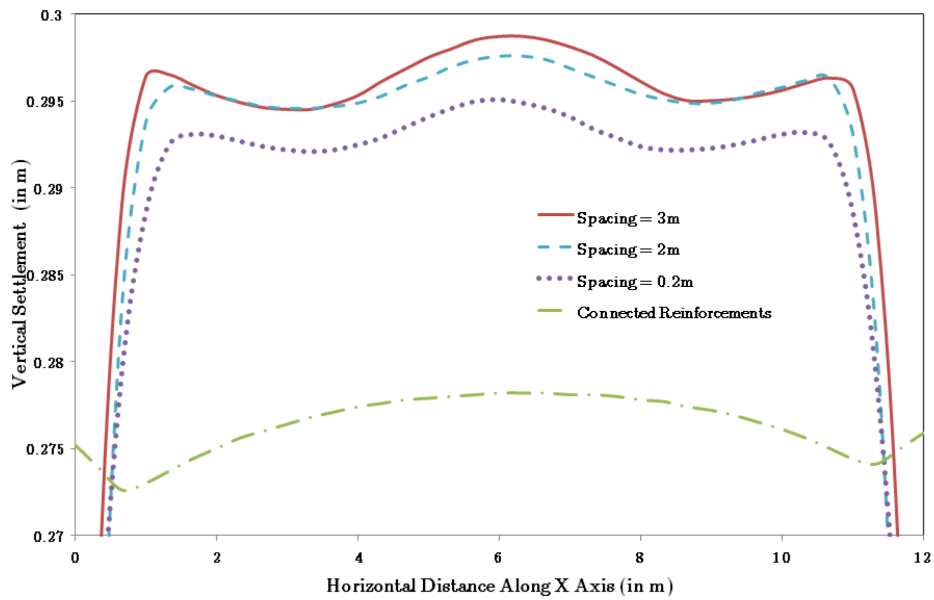


Fig 4.31: Surface Settlement Analysis for different spacing of reinforcement

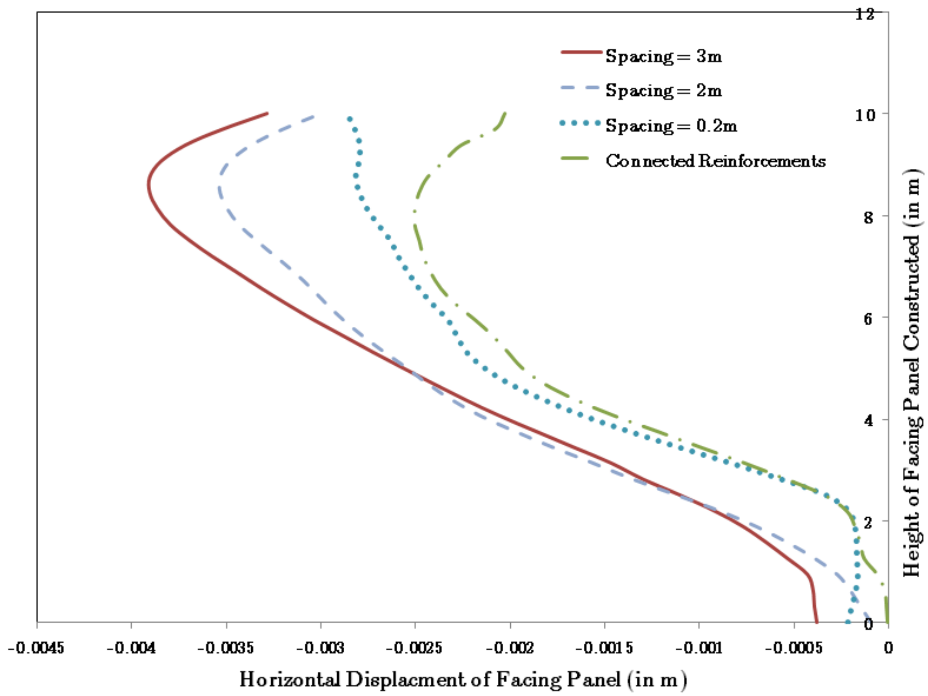


Fig 4.32: Facing Displacement Analysis for different spacing of reinforcement

4.8.2.3 Comparison 3: Varying the stiffness of Reinforcements

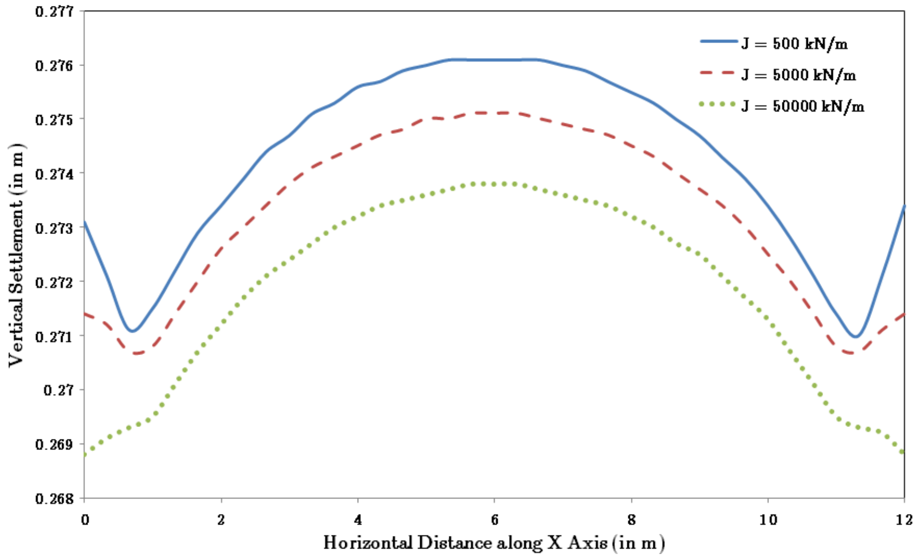


Fig 4.33: Foundation Settlement Analysis for different stiffness of reinforcements

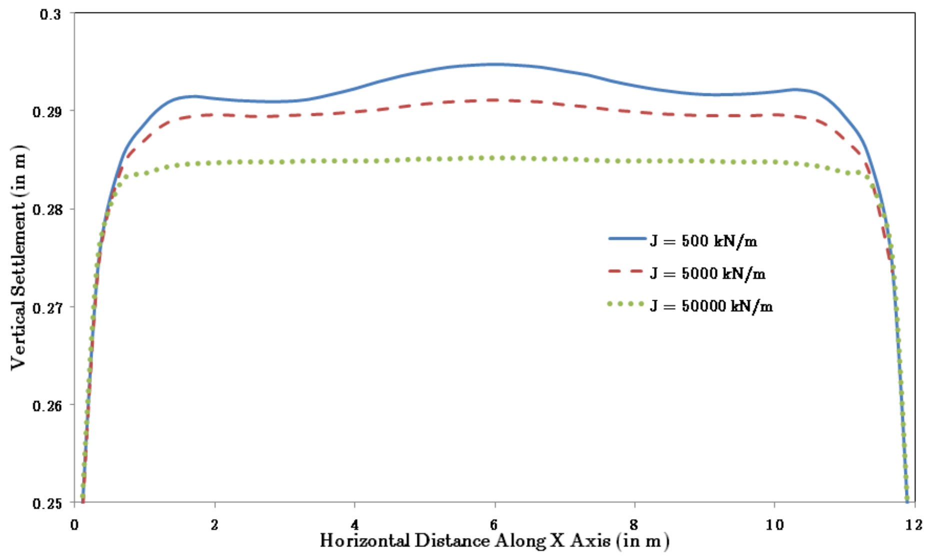


Fig 4.34: Surface Settlement Analysis for different stiffness of reinforcements

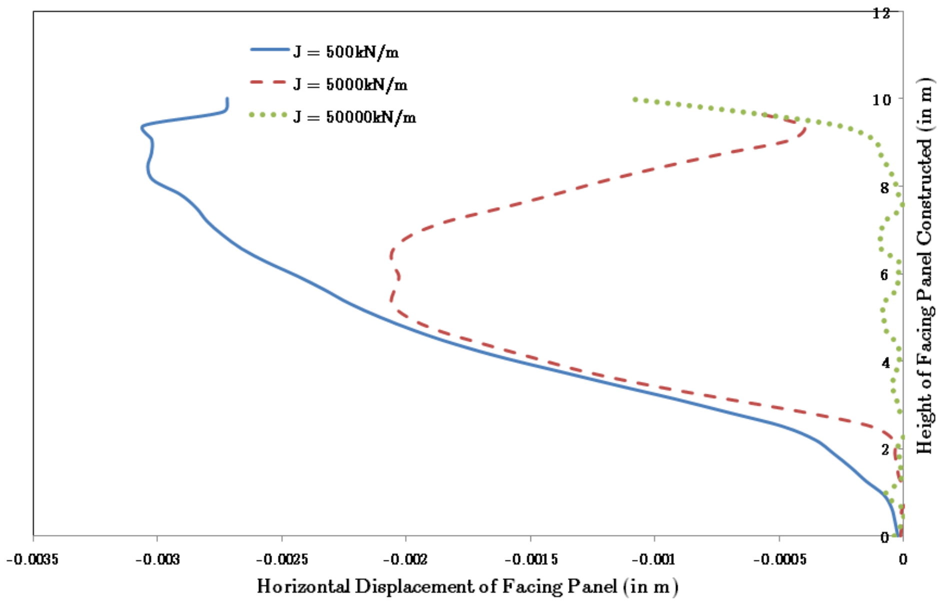


Fig 4.35: Facing Displacement Analysis for different stiffness of reinforcements

4.8.2.4 Comparison 4: Varying the friction angle of Foundation soil

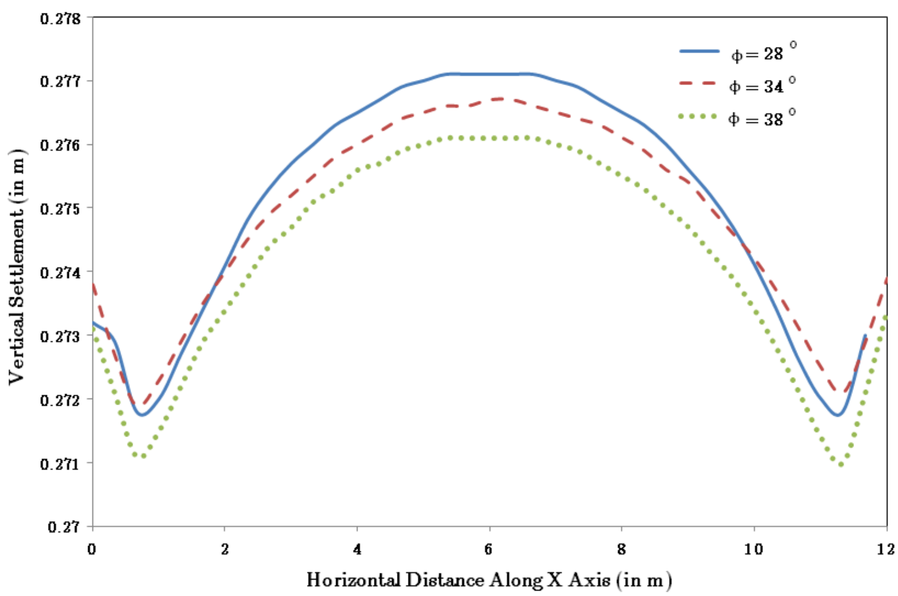


Fig 4.36: Foundation Settlement Analysis for different friction angle of foundation soil

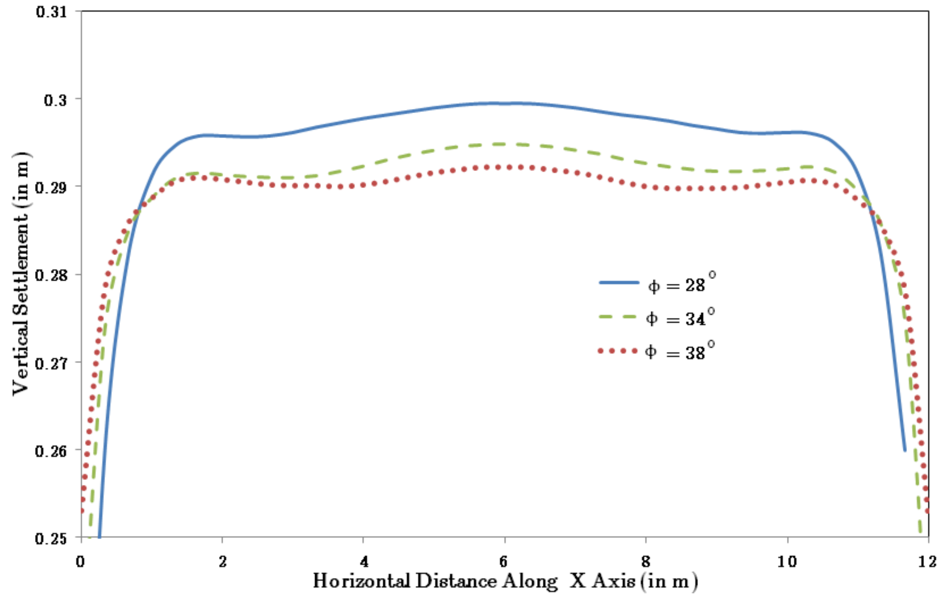


Fig 4.37: Surface Settlement Analysis for different friction angle of foundation soil

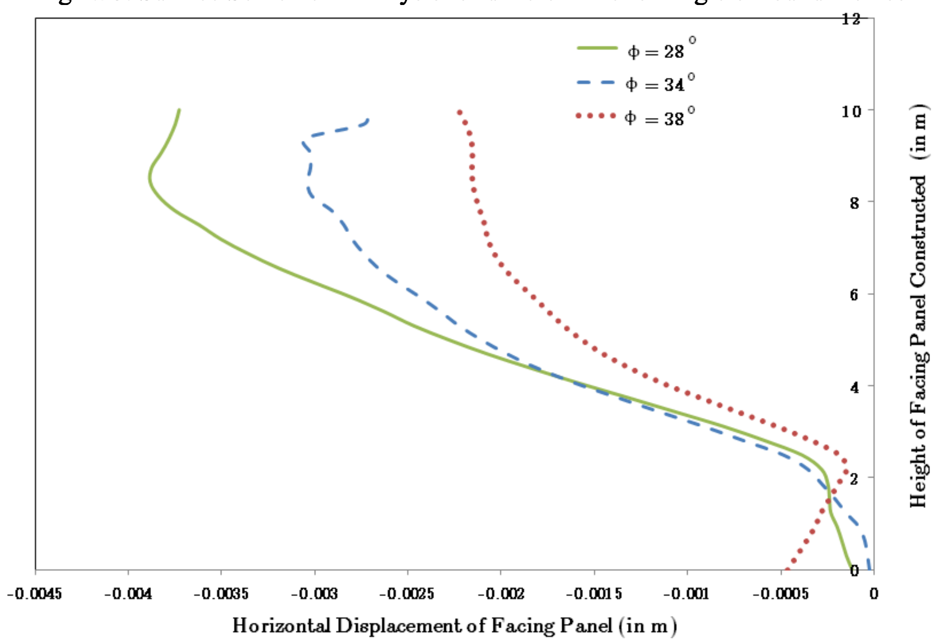


Fig 4.38: Facing Displacement Analysis for different friction angle of foundation soil

4.8.2.5 Comparison 5: Varying the friction angle of Backfill

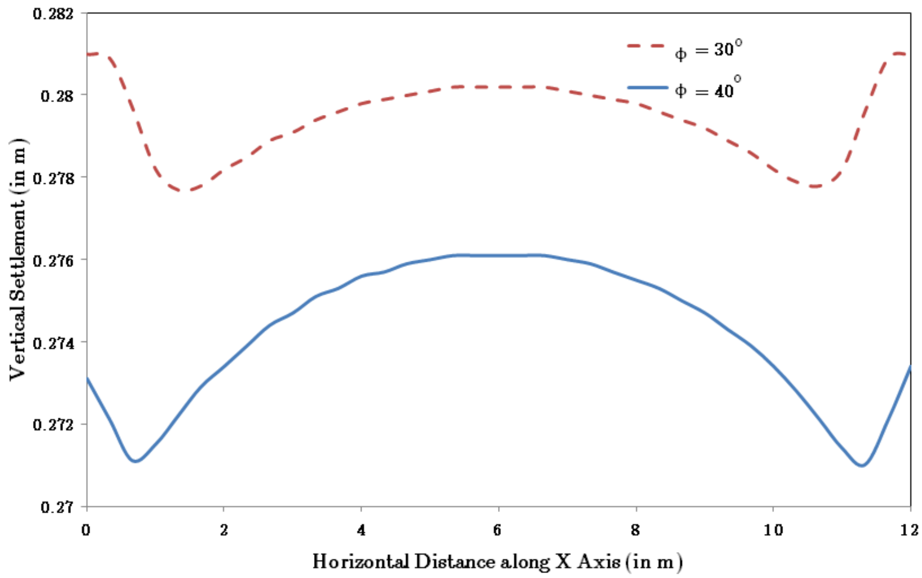


Fig 4.39: Foundation Settlement Analysis for different friction angle of backfill

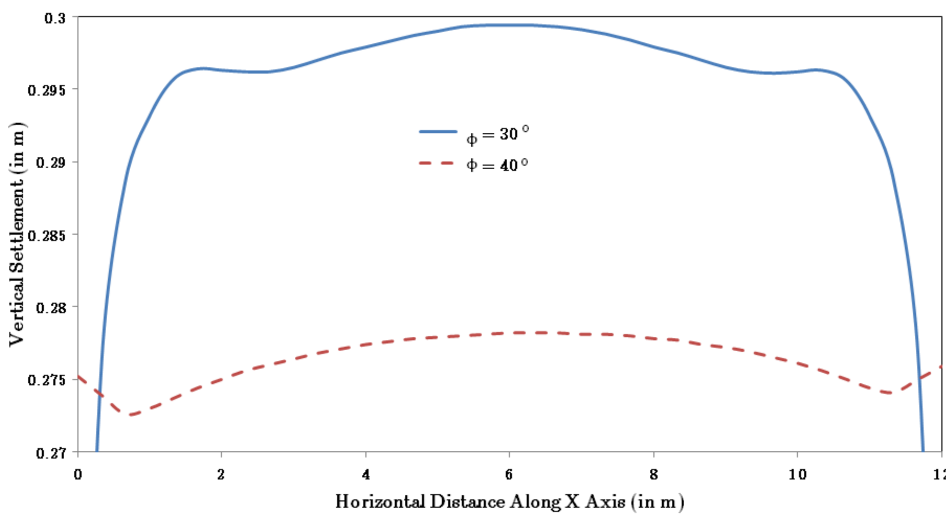


Fig 4.40: Surface Settlement Analysis for different friction angle of backfill

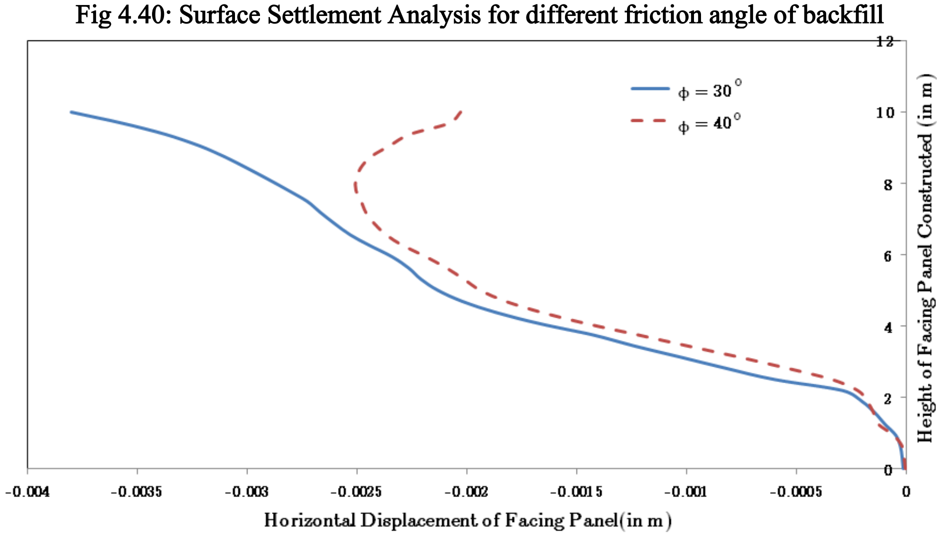


Fig 4.41: Facing Displacement Analysis for different friction angle of backfill

4.8.3 Wall height – 15m

The results of the analysis carried out for a wall height of 15m are illustrated from Fig 4.42 to Fig 4.56.

4.8.3.1 Comparison 1 – Single facing, Unconnected Back-to-Back and Connected Back-to-Back walls

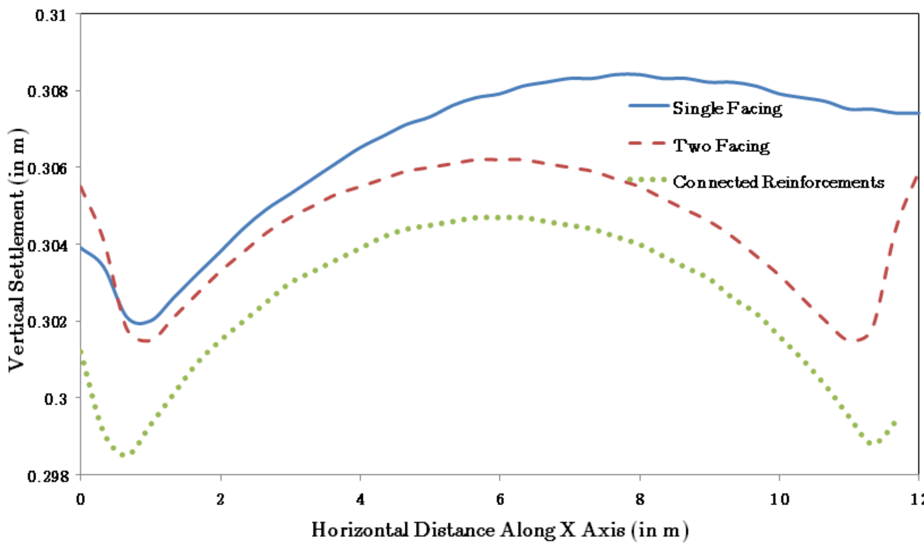


Fig 4.42: Foundation Settlement Analysis for different types of facing

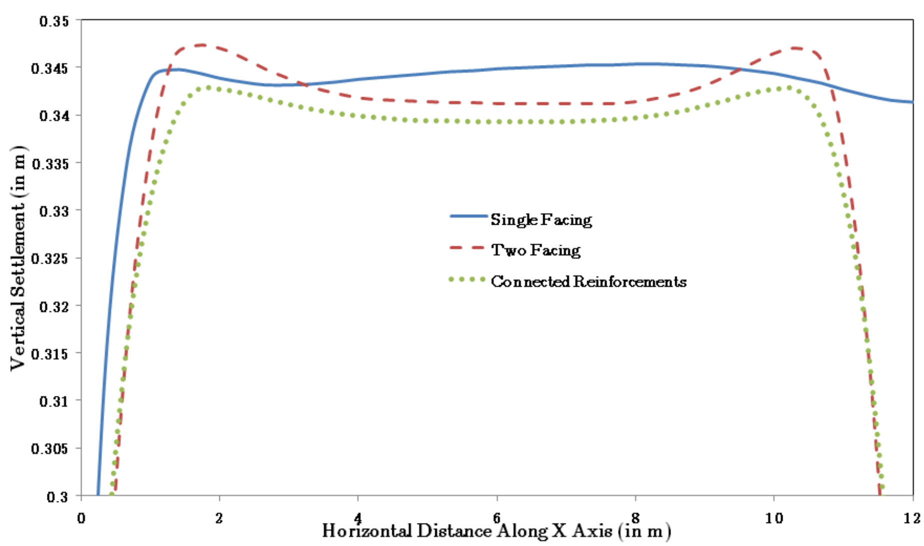


Fig 4.43: Surface Settlement Analysis for different types of facing

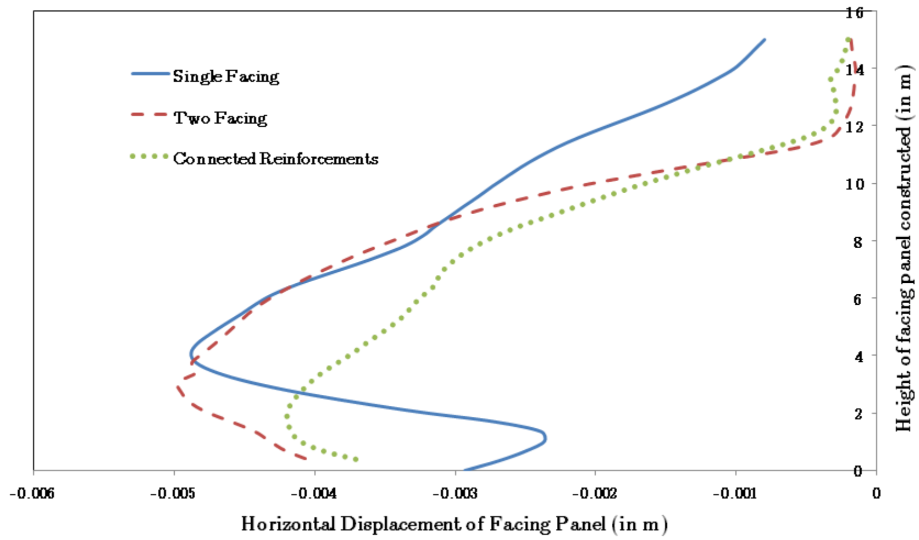


Fig 4.44: Facing Displacement Analysis for different types of facing

4.8.3.2 Comparison 2: Varying the spacing between Reinforcements

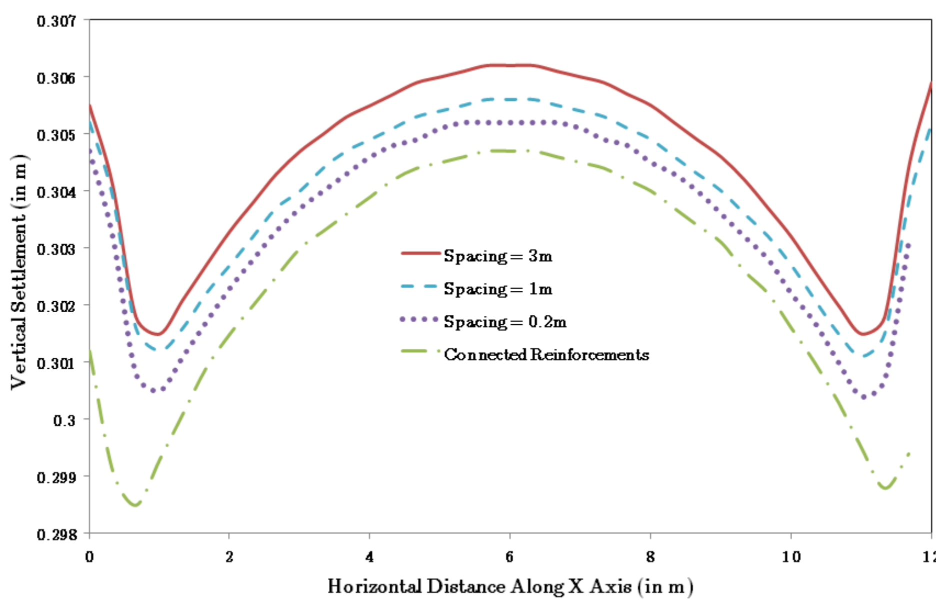


Fig 4.45: Foundation Settlement Analysis for different spacing between reinforcements

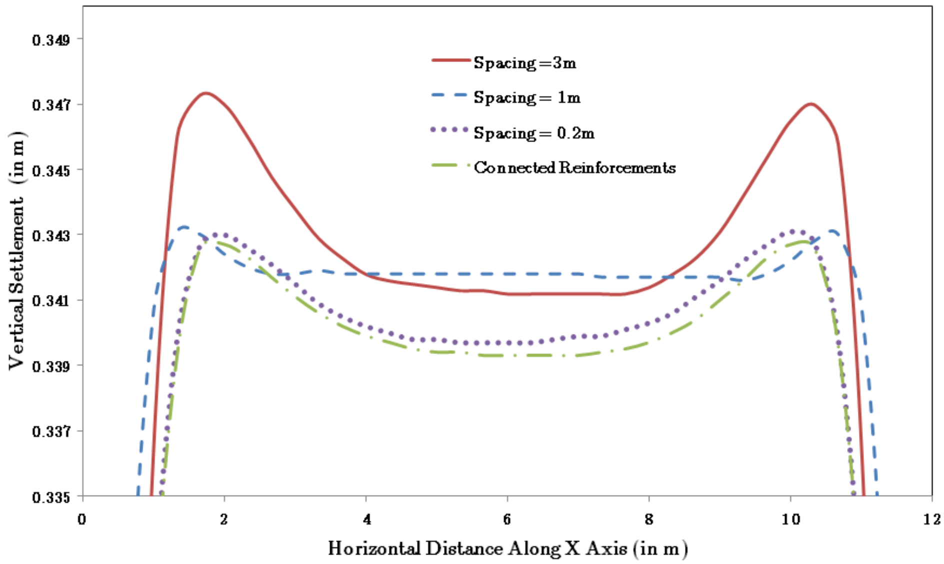


Fig 4.46: Surface Settlement Analysis for different spacing between reinforcements

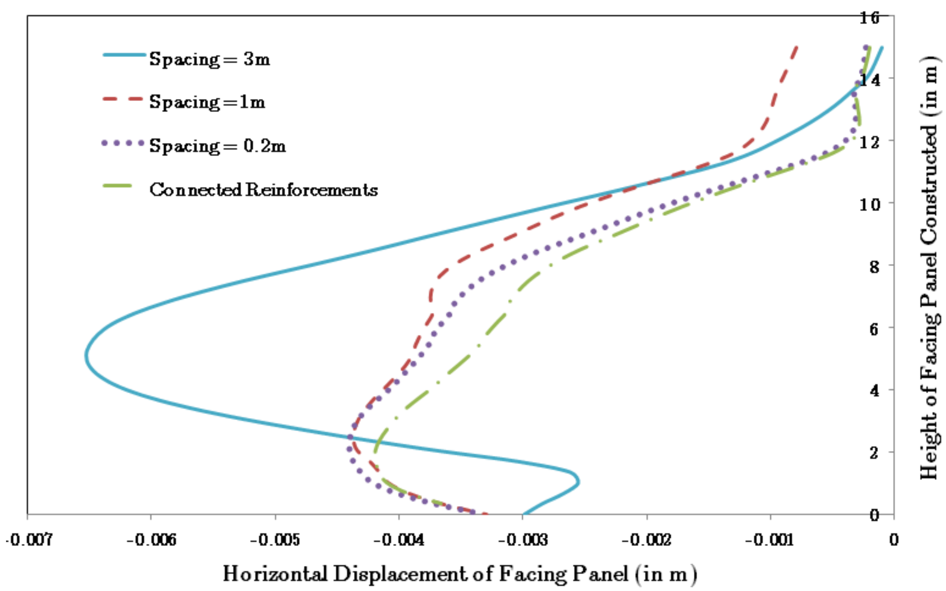


Fig 4.47: Facing Displacement Analysis for different spacing between reinforcements

4.8.3.3 Comparison 3: Varying the stiffness of Reinforcements

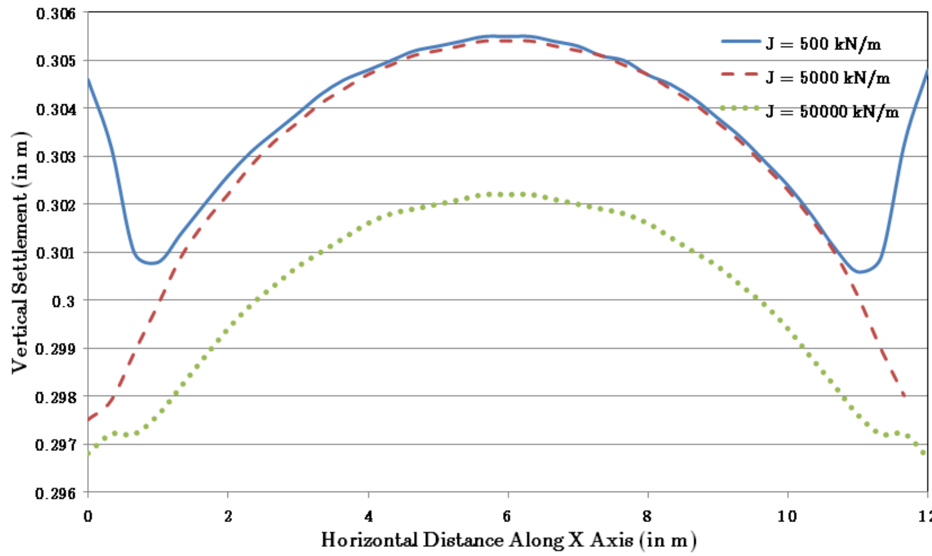


Fig 4.48: Foundation Settlement Analysis for different stiffness of reinforcements

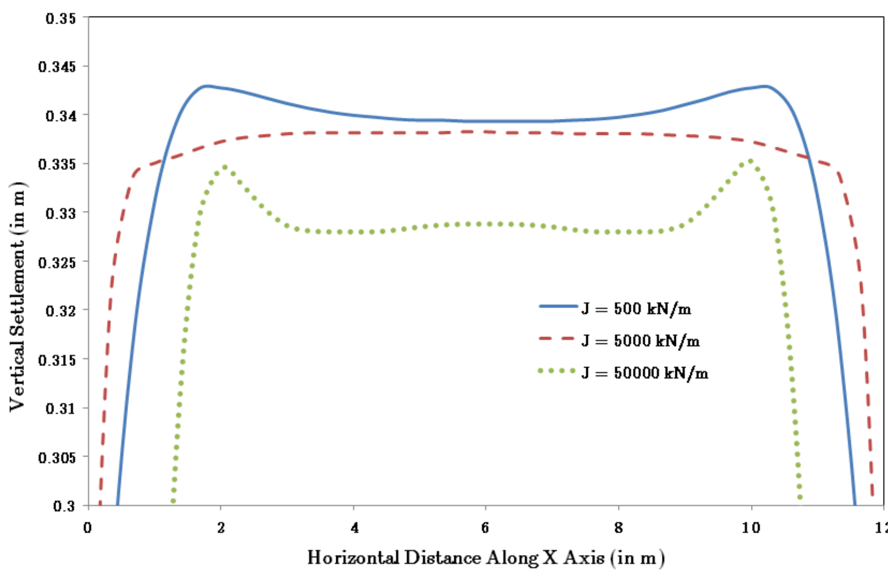


Fig 4.49: Surface Settlement Analysis for different stiffness of reinforcements

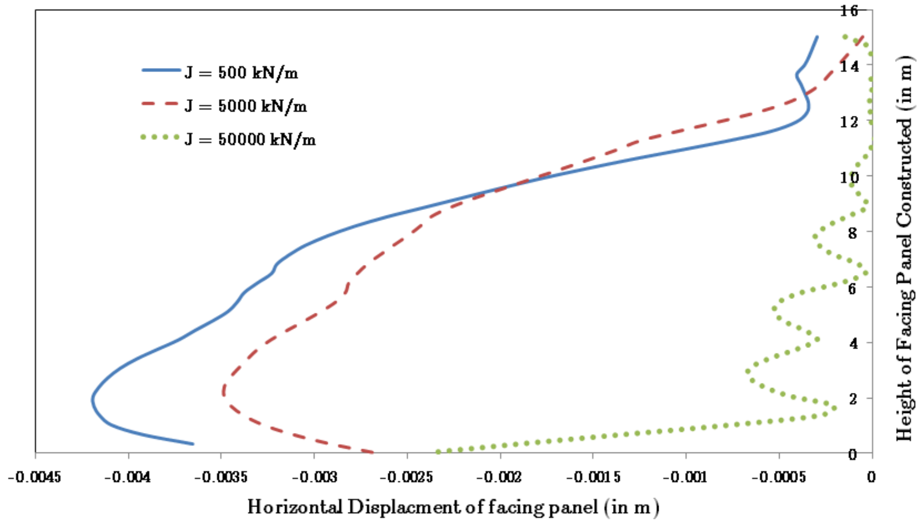


Fig 4.50: Facing Displacement Analysis for different stiffness of reinforcements

4.8.3.4 Comparison 4: Varying the friction angle of Foundation soil

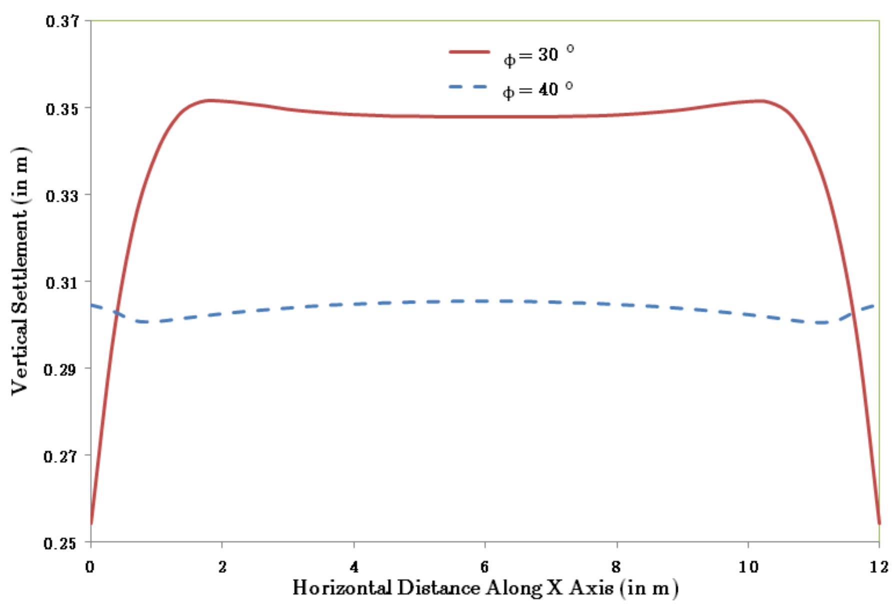


Fig 4.51: Foundation Settlement Analysis for different friction angle of foundation soil

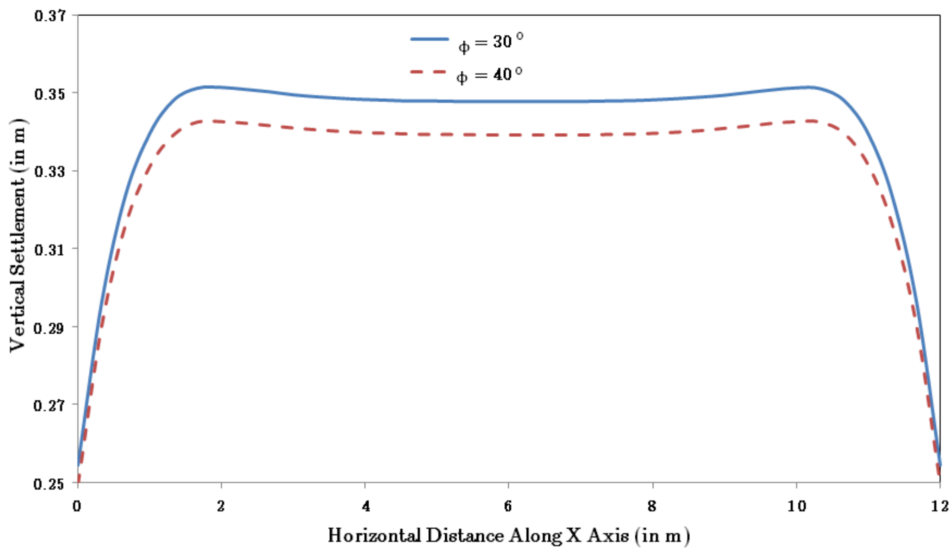


Fig 4.52: Surface Settlement Analysis for different friction angle of foundation soil

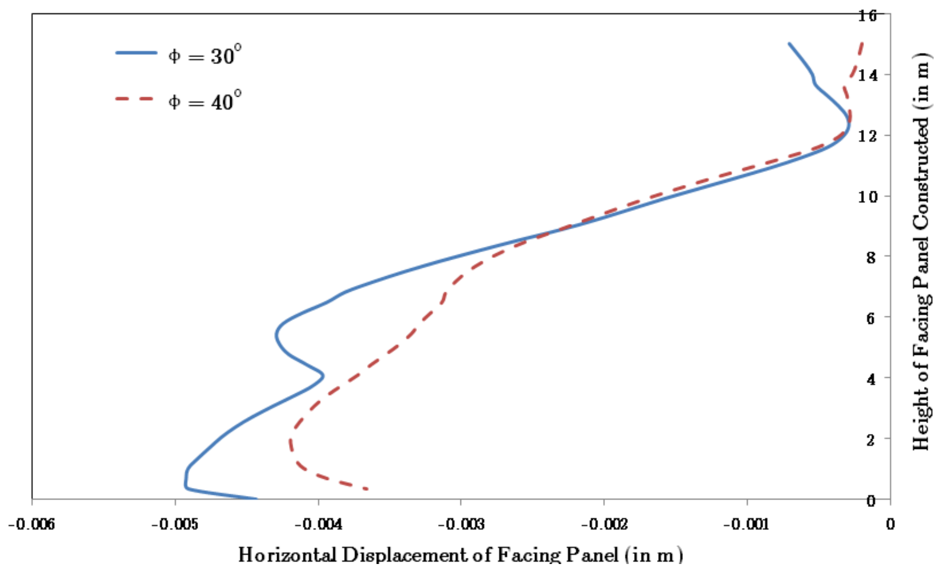


Fig 4.53: Facing Displacement Analysis for different friction angle of foundation soil

4.8.3.5Comparison 5: Varying the friction angle of Backfill

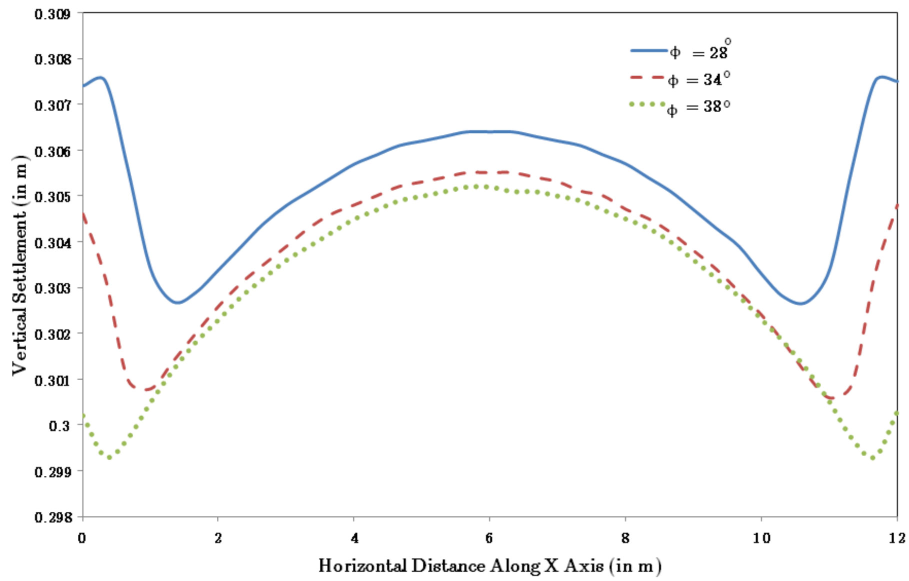


Fig 4.54: Foundation Settlement Analysis for different friction angle of backfill

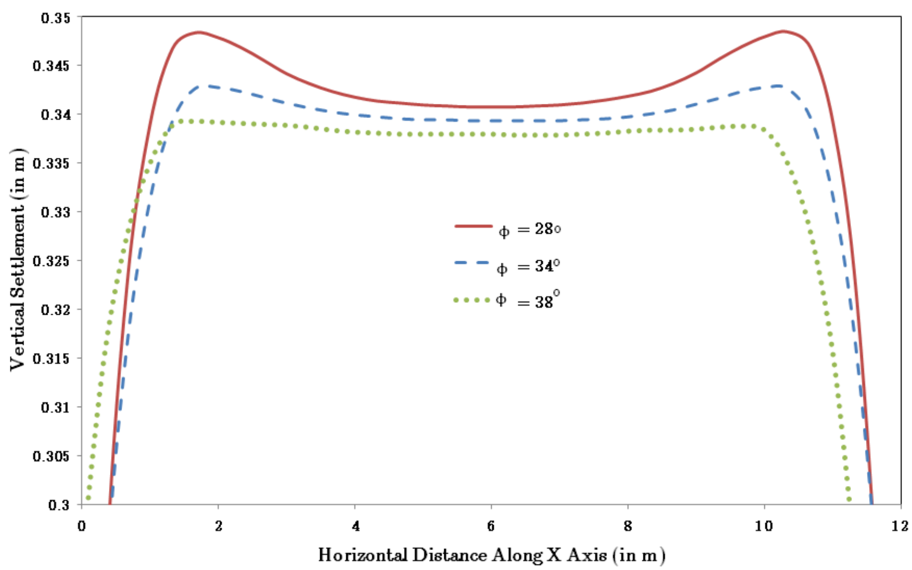


Fig 4.55: Surface Settlement Analysis for different friction angle of backfill

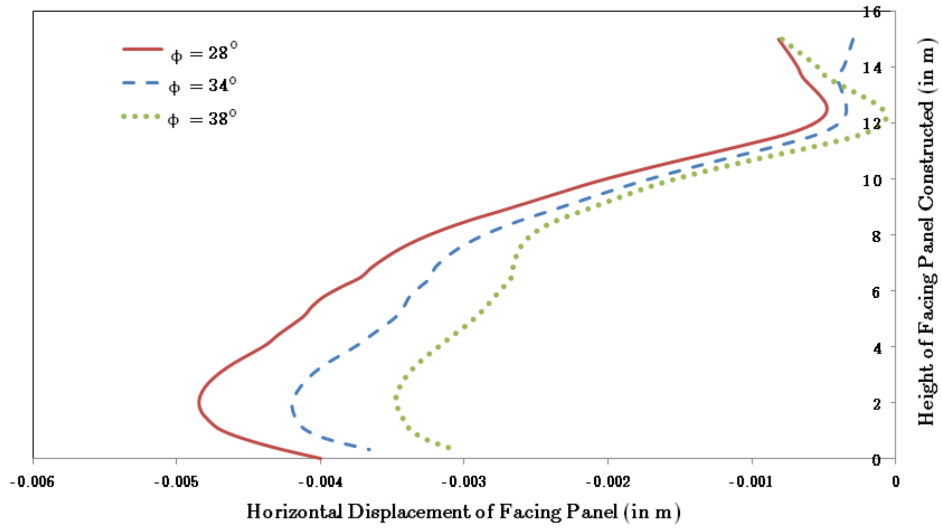


Fig 4.56: Facing Displacement Analysis for different friction angle of backfill

It was observed from the analysis that the stability of MSE wall decreases with an increase in wall height. Foundation settlement, surface settlement and horizontal displacement of the facing panel were found to be higher for single facing MSE walls. The value decreases for back-to-back walls. It was also noted that connecting the reinforcements in the middle increases the maximum tension mobilized in the same resulting in a decrease in the settlement.

Settlements and deformations in the MSE wall was found to be significantly affected by the spacing between the reinforcements. Fig 4.11 to 4.13, Fig 4.27 to 4.29, and Fig 4.42 to 4.44 clearly indicates that as the spacing between the reinforcements reduces, foundation settlement, surface settlement and horizontal facing panel displacement also decreases, i.e., the stability of wall increases. Axial stiffness of the reinforcement used for modeling the retaining wall also plays a very important role in determining the stability of MSE Walls. As the stiffness of reinforcement increases, the axial force mobilized in the reinforcement also increases correspondingly, (i.e., a stiffer reinforcement mobilizes more load) which leads to a decrease in foundation settlement, surface settlement and horizontal displacement of the facing panel. Wall settlements and facing deformations reduces with an increase in the backfill soil friction angle. Fig 4.24 to 4.26, Fig 4.39 to 4.41 and Fig 4.54 to 4.56 clearly indicate the effect of backfill friction angle of the stability of MSE Wall. As friction angle of foundation soil increases, the load that is transferred to the foundation soil increases proportionally, hence resulting in reduction of foundation settlement, surface settlement and facing panel deflection. MSE wall with a foundation soil friction angle of 40° gives lesser settlements and deflections compared to a wall with foundation soil friction angle 30° for all wall heights.

Chapter 5

Conclusions

In this study, model of reinforced and unreinforced back-to-back retaining walls were developed and parametric studies were performed for the same using the finite element package Plaxis 2D and finite difference package Flac 2D.

5.1 Case 1: Modeling in Plaxis 2D

- Models of Unreinforced and Reinforced Back-to-Back Retaining walls were developed.
- Back-to-Back unreinforced and reinforced retaining walls of height equal to 3m, 6m and 9m were modeled and compared.
- As the height of the wall increases, the foundation settlement, surface settlement and the facing panel displacement increases.
- Considerable reduction was observed in the foundation settlement, surface settlement, and horizontal displacement of the facing panel by reinforcing the retaining wall. This reduction was observed to be more for greater heights of retaining walls, i.e., for a retaining wall of height 3m, a

reduction of 38% was observed in the facing panel displacement after reinforcing, whereas when the wall height was increased to 9m, a reduction of 88% can be observed.

- When the stiffness of reinforcement was increased to 5000kN/m from 500kN/m, no significant change was observed in the foundation settlement and the surface settlement decreases by a small value.
- Significant reduction is observed in the displacement of facing panel by increasing the stiffness of reinforcement. That is a decrease of 67%, 75% and 89% was observed in 3m high wall, 6m high wall and 9m high wall, respectively, for tenfold increase in reinforcement stiffness.

A study was performed to determine the effect of varying the spacing between the reinforcements and backfill soil friction angle on the critical failure surface and the tension developed in the reinforcements. The deformation plots and facing panel displacements were also taken into consideration.

- As the distance between the walls increases, a larger deformation was observed for the wall and this deformation gets reduced considerably once the reinforcements are connected.
- For a soil backfill of friction angle 25°, a decrease in the spacing between reinforcements resulted in a reduction of 67% in the facing panel displacement, and 45% in the tensile forces mobilized in the reinforcements.
- Similarly, for a soil backfill of friction angle 34°, the facing panel displacement and mobilized geosynthetic forces were reduced by 50% and 51%, respectively, when the W/H ratio was decreased from 3 to 2.
- The critical slip surfaces behave independently as the distance between the walls increases, and it overlaps for the case when reinforcements for the two walls are connected.
- A reduction of about 95% was observed for the facing panel displacement and tension mobilized in the geosynthetic reinforcements by increasing the backfill soil friction angle from 25° to 34°. This indicates the importance of using higher friction angle for the soil backfill.

5.2 Case 2: Modeling in Flac 2D

- Models of Connected and Unconnected Back-to-Back Reinforced Retaining walls were developed.

- Comparison studies were conducted using the Finite Difference package Flac 2D in mainly five areas for walls of height 6m, 10m and 15m .The areas of comparison study includes:
    - Reinforced Retaining Wall with single facing, Back-to-Back Unconnected Reinforced Retaining wall, Back-to-Back Connected Reinforced Retaining Wall
    - Varying the spacing between the reinforcements as 3m, 1m, 0.2m and connected case.
    - Varying the stiffness of reinforcement from 500kN/m to 5000kN/m
    - Varying the friction angle of foundation soil, viz. 30^{o} and 40^{o}
    - Varying the friction angle of backfill as 28^{o}, 34^{o} and 38^{o}

Based on the comparison study, the following conclusions are made:

- Foundation Settlement, Surface Settlement and Horizontal Displacement of Facing Panel are high for MSE walls with single facing compared to Back-to-Back Reinforced Retaining walls. The value decreases further when the reinforcements are connected.
- The spacing between the reinforcements has significant effect on the deformation of reinforced retaining walls. As the spacing between the reinforcements reduces, wall deformation reduces (i.e. the stability of wall increases), which is indicated by the decrease in foundation settlement, surface settlement and horizontal facing panel displacement
- The stability of a Reinforced Retaining Wall is highly dependent on the stiffness of reinforcement. As the reinforcement stiffness increases, the axial force mobilized in the reinforcement also increases correspondingly. This will lead to a decrease in foundation settlement, surface settlement, and horizontal displacement of the facing panel.
- Backfill friction angle has considerable effect on the settlements and displacements of reinforced retaining wall. Both foundation settlement and surface settlement decreases with an increase in the soil backfill friction angle. It is also noted that the horizontal displacement of the facing panel diminishes in magnitude to a great extent as the soil strength increases due to an increase in backfill friction angle.
- Settlement at the surface and foundation as well as the displacement of facing panel is proportional to and is largely influenced by the deformation modulus of foundation soil, i.e., a reinforced retaining wall with a foundation soil stiffness of 40MPa gives lesser settlements and deflections compared to a wall with foundation soil stiffness 30MPa. That is foundation settlement decreases by 20%, 38% and 50% in 6m, 10m and 15m high walls respectively.

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