Optimization of Concrete Block Quay Walls

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Optimization of Concrete Block Quay Walls

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ABSTRACT

Marine structures are defined as a human made structures which are constructed for the purpose of port facilities and/or protecting the coastline, This research focus on studying and analyzing the stability of an important type of marine structure which is: concrete block quay wall, optimum design and stability of pre-cast concrete blocks quay walls consists of 15 row of blocks numbered respectively from bottom to top using the hollow blocks instead of solid ones by obtaining the resulting benefits of this replacement is investigated. Therefore, four stages of optimization under stability considerations have been adequated as. GEO5 software had been used for the purpose of determining the factors of safety against overturning and sliding for all structure and at each block interface and also determining the bearing pressures exerted by the quay wall to the existing ground for structural elements under all load combinations for all stages of optimization and using these pressures in hansen's equations for studying the stability of the block quay wall against (bearing capacity). SLOPE/W software had been used also for studying the stability of block quay walls against slip failure. The results show that the critical stage of optimizations is opt.(2), reducing the backfill internal angle of friction (φ) from 40˚ to 30˚, reduces the factors of safety against Bearing capacity and slip failure and Increasing the subsoil cohesion parameter (c), improves the bearing capacity factor of safety.

Key words: Marine structure, Quay wall, Stability, factor of safety, GEO 5, Slope/w

1. INTRODUCTION

Marine structures are defined as human made structures which are constructed for the purpose of port facilities and/or protecting the coastline such as (Quay walls, ship repair structures, rubble mound breakwaters, open piled breakwaters, groins and jetties, etc….), Some of these structures have classical common design. Others had been developed for the purpose of cost saving and other stability requirements.

Gravity structures are usually an excellent choice for marine structures where the seabed soil condition is appropriate. They should be designed and constructed to resist safely the vertical loads, trucks, cranes etc., as well as the horizontal loads from ship impacts, wind, soil pressure, etc., The afore mentioned loads vary according to the type of the terminal. This makes the design and construction of a quay wall interesting and complicated day by day. Therefore, several design guide lines are available to give recommendations for the design and construction of quay walls.

Optimum design and stability of pre-cast concrete blocks quay walls is the aim of this paper, this may be performed by comparing the stability factors of the solid type concrete blocks quay wall with that obtained for the same quay wall when replacing the solid blocks with hollow precast ones.

The main modes of failure of this gravity structure are: sliding, overturning, deep slip and foundation failure as shown in figure 1. Therefore, in the stability calculations, circular slip, bearing capacity of the foundation, sliding and overturning at all horizontal surfaces between blocks had been examined. To study the behavior of a block quay wall and to check the stability against probable different failure modes, a computer program has been developed. This program can easily consider the effects of different parameters such as section geometry of quay wall, material property and loading conditions in design. After reviewing design and construction considerations for such quay walls, available methods for optimum design of such structures are discussed and objective function, constraints and design variables are considered. The main constraints of the optimization problem in the present study are safety factors in various modes of failures.

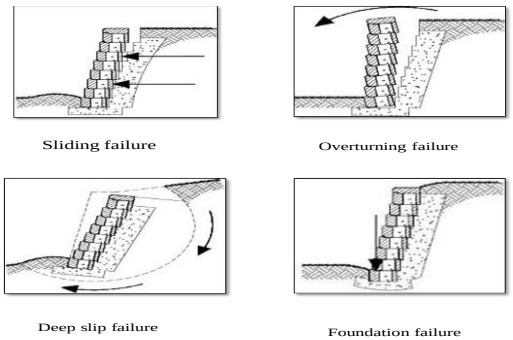

Figure 1: Main modes of failure

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The purpose of this paper is to verify the possibility of optimizing the dimensions of the quay wall using the hollow blocks instead of solid one by obtaining the resulting benefits of this replacement. GEO5 software had been used for the purpose of determining structures stabilities against overturning` and sliding for the studied quay walls to estimate the factor of safety for all structure and at each block interface and also determining the bearing pressures exerted by the quay wall to the existing ground for structural elements. SLOPE/W software had been used also for studying the stability of block quay walls to estimate the factor of safety against slip failure.

A number of case researches have been conducted related retaining wall Problems in the literature which defines the relationship between the block type quay walls and the earthquake. Among these *Chen and Huang* [2], *Ichii* [7], *Ichii* [8], *Karakus, et al.*[9], and *Gerolymos, et al*. [5].

Voortman, et al. [13] applied the concept of economic optimization to derive the appropriate safety level and at the same time the optimal geometry. Application to a design case shows that it can be economically optimal not to distribute the acceptable failure probability equally over all failure modes, but rather let one or two failure modes determine the total probability of failure.

El-Sharnouby, et al. [4] analyzed the stability of block wall against sliding, overturning and stresses under many factors such as depth of wall, pulling forces, soil characteristics and base stratum characteristics by using computer program (QWD).

Mirjalili [11] Introduced available methods of optimization on block quay wall cross section by using sequential quadratic programming (SQP) and found safety factor in various modes of failure and the results indicated that the cross section of the block quay wall has an important role on stability of the structure

Shafieefar and Mirjalili [12] presented an optimization for the cross section of block work quay walls using SQP method. The results of parametric studies carried out indicate that shear key, internal friction angle of back filling material and negative slope behind the blocks have considerable effect on cross section optimization.

Cihan, et al [3] studied the stability of block type quay wall which consists of two concrete blocks is investigated experimentally and numerically. During the experiments accelerations, pore pressures, soil pressures and displacements are measured for two blocks under different cycling loadings. *PLAXIS* V8.2 software program is used for numerical study to determine the material properties.

Madanipour, et al. [10] studied the parametric effect of cohesion in a silt layer of soil on the behavior of block quay walls under horizontal and vertical components of earthquake by using (ABAQUS) software. The results of the analysis show that under this study the effect of variations of cohesion of the silt layer, and the vertical component are negligible.

2. Methodology

2.1 Quay Wall Dimensions

2.1.1 Main solid quay wall

The block wall is consisting of solid concrete blocks from plain concrete. A precast concrete cope unit is then typically fixed at the top of the wall. we can also note that this solid quay wall consists of 15 row of blocks numbered respectively from bottom to top, every block is constructed from plain concrete has thickness 1 m unless block number 13 with thickness 0.3 m, while block number 14 has a thickness 0.7 m and block number 15 has a thickness 2 m.

2.1.2 Main Hollow Quay Wall

For this case, the dimensions of the hollow block quay wall are kept typically as same as the solid block quay wall. But, the solid blocks are replaced with hollow precast ones by making three holes through the quay wall. Every hole is filled *with* clayey gravel unless holes in blocks number 1, 14 and 15 which filled with reinforced concrete. While, block number #6 two holes are filled with clayey gravel and the third hole with reinforced concrete.

2.1.3 Optimization (1)

For this case, every hole of clayey gravel was optimized by decreasing its width by 0.5m width for each block. Therefore, the width of blocks number from 1 to 6 are minimized by 1.5 m, while blocks numbers from 7 to 15 are minimized by 1 m only, So we can note that the width of blocks from 1 to 6 are changed to 9.5m and the width of blocks from 7 to 15 are changed to 8.5m, while the height of the hollow block quay wall in optimization (1) is kept typically as same as the solid block quay wall after the minimizing process.

2.1.4 Optimization (2)

For this case, every hole of clayey gravel was optimized by decreasing it's width by 0.5m extra for each block. Therefore, the width of blocks numbers from 1 to 6 were minimized by 1.5 m, while blocks numbers from 7 to 15 were minimized by 1 m only.

Finally, we can note that the width of blocks numbers from 1 to 6 were minimized by 3 m, while blocks numbers from 7 to 15 were minimized by 2m from the beginning of optimization, To get the optimum cross section achieving stability requirements, So we can note that the width of blocks from 1 to 6 are changed to 8m and the width of blocks from 7 to 15 are changed to 7.5m, while the height of the hollow block quay wall in optimization (2) is kept typically as same as the solid block quay wall after the minimizing process.

The four cross sections adopted in all cases of optimization are shown in figure 2

2.2 Factors of Safety

The main constraints of the optimization problem in the present study are safety factors in various modes of failures, *British Standard* [1].

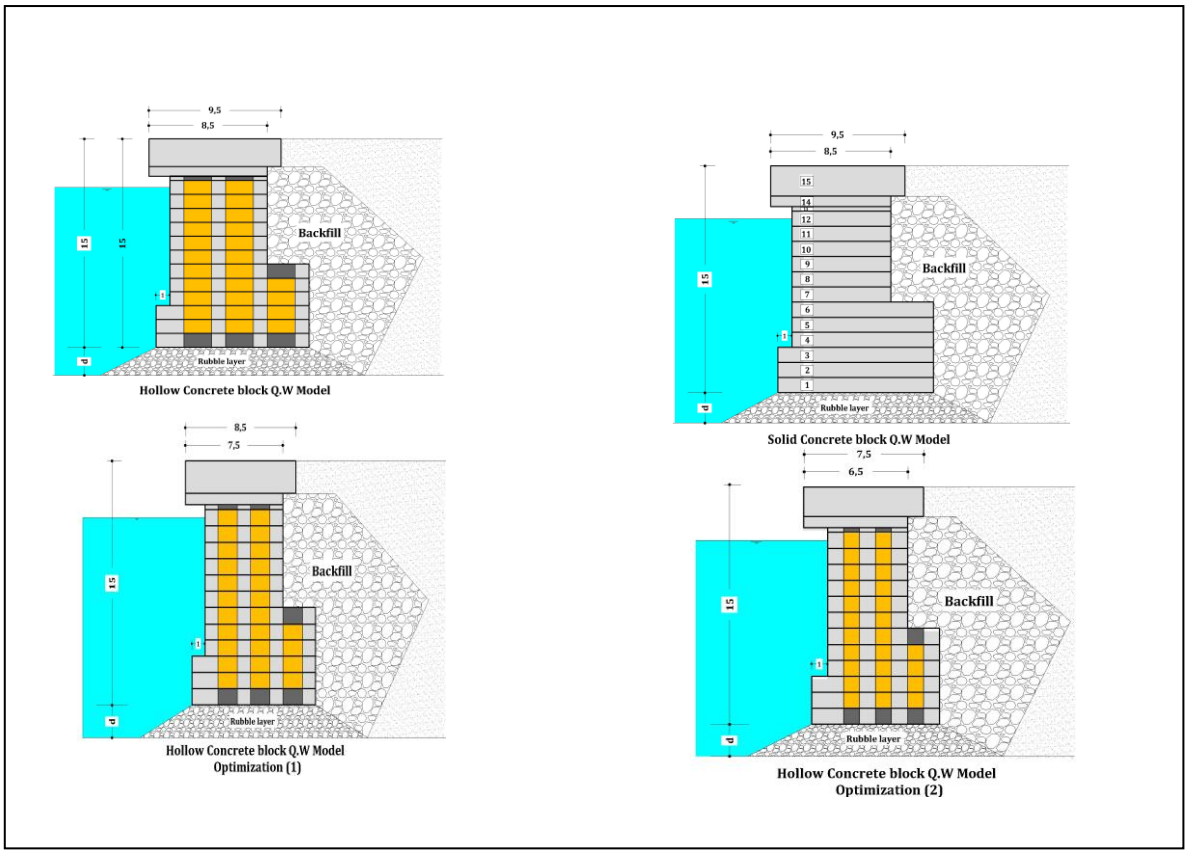


Figure 2: The four cross sections adopted in all cases of optimization

2.3 Geotechnical Data

The design soil profile and parameters adopted in the verification of the quay wall are provided in Table 1.

Table 1: Geotechnical parameters.

NO	Soil type	Top elevation	γ_s KN/m³	Φ' Deg	C' Kpa
1	Rock fill	+1.00	20	30 and 40	0
2	Substructure rubble layer	-12.00	20	40	0
3	Sand fill	+3.00	19	36	0
4	subsoil	+3.00	18	25,28, 30,32 and35	0,20
5	Lower sand	-20.00	18	38	0

2.4 Loads

It is assumed that the block quay wall is designed to withstand the following loads as listed in Table 2.

Table 2: Loads applied on the block quay wall.

Type of load	Value
Dead load (A)	Weight of the quay wall
Minor Equipment on The Quay (VE)	900 KN
Storage Loads (UDL)	A uniform surcharge of (50 kN/m²)
Tidal Lag (TL)	0.5 m
Ship Mooring (M)	38 tonnes with a spacing of 15m
Seismic Loading (Q)	K_H= 0.0615 g K_V= 0.03075 g

2.5 Load Combinations

There are 13 load combinations are adopted in the verification of the block quay wall stability as shown in figure 3

Load Combinations												
1. Normal Operational Conditions					2. Extreme Conditions					3. Seismic Conditions		
Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10	Case 11	Case 12	Case 13
A + 50%M + UDL	A + 50%M + VE	A + UDL + 50%TL	A + VE + 50%TL	A + VE + 50%UDL	A + M + UDL	A + M + VE	A + M + TL	A + UDL + TL	A + VE + TL	A + 50% M + 50% VE + Q	A + 50% M +50% UDL + Q	A + 50% UDL + 50% VE + Q

Figure3: Load combinations adopted in the verification of the block quay wall stability

2.6 Plan of Work

In figure 4, all steps and variables used for reaching the optimum cross section under stability considerations are shown in details.

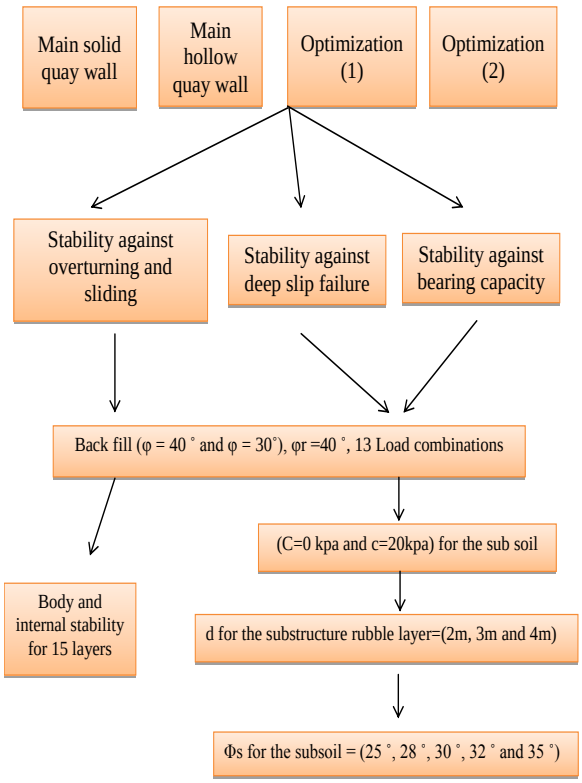


Figure 4: All steps and variables used for reaching the optimum cross section under stability considerations**.**

2.7 Used Soft wares for Optimization

GEO5 software had been used for the purpose of determining structures stabilities against overturning and sliding for the studied quay walls to estimate the factor of safety for all structure and at each block interface and also determining the bearing pressures exerted by the quay wall to the exiting ground for structural elements.

SLOPE/W software had been used also for studying the stability of block quay walls to estimate the factor of safety against slip failure.

Finally hansen's equations had been used for studying the stability of block quay walls to estimate the factor of safety against local foundation bearing capacity failure.

3. NUMERICAL MODELING

Numerical models involving FEM can offer several approximations to predict true solutions. The accuracy of these approximations depends on the modeler's ability to portray what is happening in the field. Often the problem being modeled is complex and has to be simplified to obtain a solution. Finite element method has become more popular as a soil response prediction tool. This has led to increased pressure on researchers to develop more comprehensive descriptions for soil behavior, which in turn leads to more complex constitutive relationship.

In this research, GEO5-software had been used for the purpose of determining structures stabilities against overturning, and sliding. While, SLOPE/W-software had been used also for studying the stability of block quay wall to estimate the factor of safety against slip failure.

3.1 Geo 5 (Prefab Wall)

The output of the program contains all facilities to verification overall stability and internal stability against overturning and sliding for the structure by calculating factors of safety against overturning and sliding. Also, the program can calculate the forces on the ground footing exerted from the analyzing.

The design is based upon a total factor of safety concept, in which factors The design of the block quay wall has to be considered in terms of both:

- Overall stability (overall stability of wall against sliding and overturning)
- Internal stability (over turning and sliding at each interface).

In this research, GEO 5 (prefab wall) was used for analyzing the stability of the block wall against overturning and sliding. The block quay wall is modeled in which the active earth pressure calculated by using coulomb theory, the passive earth pressure calculated by copout- kerisel, and finally the earth quake analyzed by Monobe-okabe theory. The structure properties are defined by geometry. The soil properties are defined by profiles, soil and assign icons. The forces due to surcharge load, applied forces and earth quake are defined. The chosen quay wall will be modeled using the finite element program GEO5 (prefab wall) as shown in figure 5.

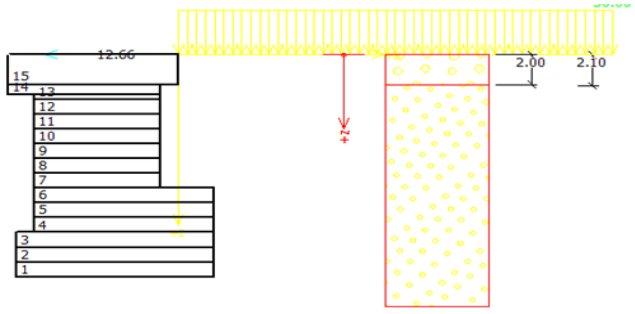


Figure 5: A screen shot of the input numerical model of the block wall

3.2 Slope/ W

The output program contains all facilities to examine the slope stability and view the lowest factor of safety and critical slip surface for all contained methods.

The stability against deep slip failure. (Failure by rotation of the soil mass) is checked by limit equilibrium approach using slope/w software.

In the analyses, the "half –sine function "is used to relate the normal forces to the shear forces between slices.

It's assumed entry and exit method for checking the stability of the block quay wall to obtain the minimum factor of safety, as shown in figure 6.

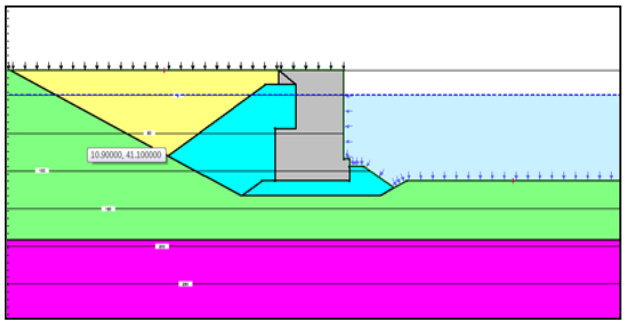


Figure 6: Quay wall numerical model by slope/w.

3.3 Bearing Capacity of Foundations

3.3.1 Hansen's bearing-capacity method

Hansen [6] proposed the general bearing-capacity case and N factor equations. This equation is readily seen to be a further extension of the earlier Meyerhof work. The extensions include base factors for situations in which the footing is tilted from the horizontal ***bi*** and for the possibility of a slope of the ground supporting the footing to give ground factors. The Hansen equation implicitly allows any D/B and thus can be used for both shallow and deep foundation.

4. RESULTS AND DISCUSSION

Results for all parts of the quay wall structure are summarized in this research.

4.1 Structural design consideration

The stability of the block quay wall against overturning and sliding for the four stages of optimization has to be considered in two terms under 13 load combinations by using GEO5 Prefab Wall software:

- Over all stability (over all stability of wall against sliding and overturning)
- Internal stability (overturning and sliding at each interface)

4.1.1 Body over all stability analysis

Figure 7 shows the overall stability of the block quay wall against overturning and sliding for the four stages of optimization under 13 load combination has been checked when changing the back fill characteristics from φ = 40˚ to φ = 30˚ and finally the minimum factor of safety for every load combination has been calculated.

The results show that the factors of safety against failure for both overturning and sliding showed variations according to the considered load combination case. The lowest factors of safety observed were obtained from load combination 12 (A+ 50%M + 50%UDL+ Q) for seismic conditions with average values for the four studied quay walls geometries as shown in Table 3.

Table 3: Variation between the factors of safety values for both overturning and sliding according to load combination 12#.

Backfill internal angle of friction, (φ)	Ave. F.O.S $_{OV.T}$	Ave. F.O.S $_{Sliding}$	Load combination 12 (Lowest factor of safety)
φ = 40˚	2.345	2.375	seismic conditions
φ = 30˚	1.985	1.323	

The results also show that reducing the backfill internal angle of friction (φ) from 40˚ to 30˚, reduces the factors of safety against overturning and sliding as shown in Table 4.

Table 4: Reduction in factor of safety values against overturning and sliding according to reducing the backfill internal angle of friction (φ) from 40˚ to 30˚.

Actual overall Factor of safety	Operating Conditions		
	Normal Conditions	Extreme Conditions	Seismic Conditions
Reduction in F.O.S $_{OV.T}$	25.3 %	22.6 %	15.3 %
Reduction in F.O.S $_{Sliding}$	51.3 %	49.9 %	44.3 %

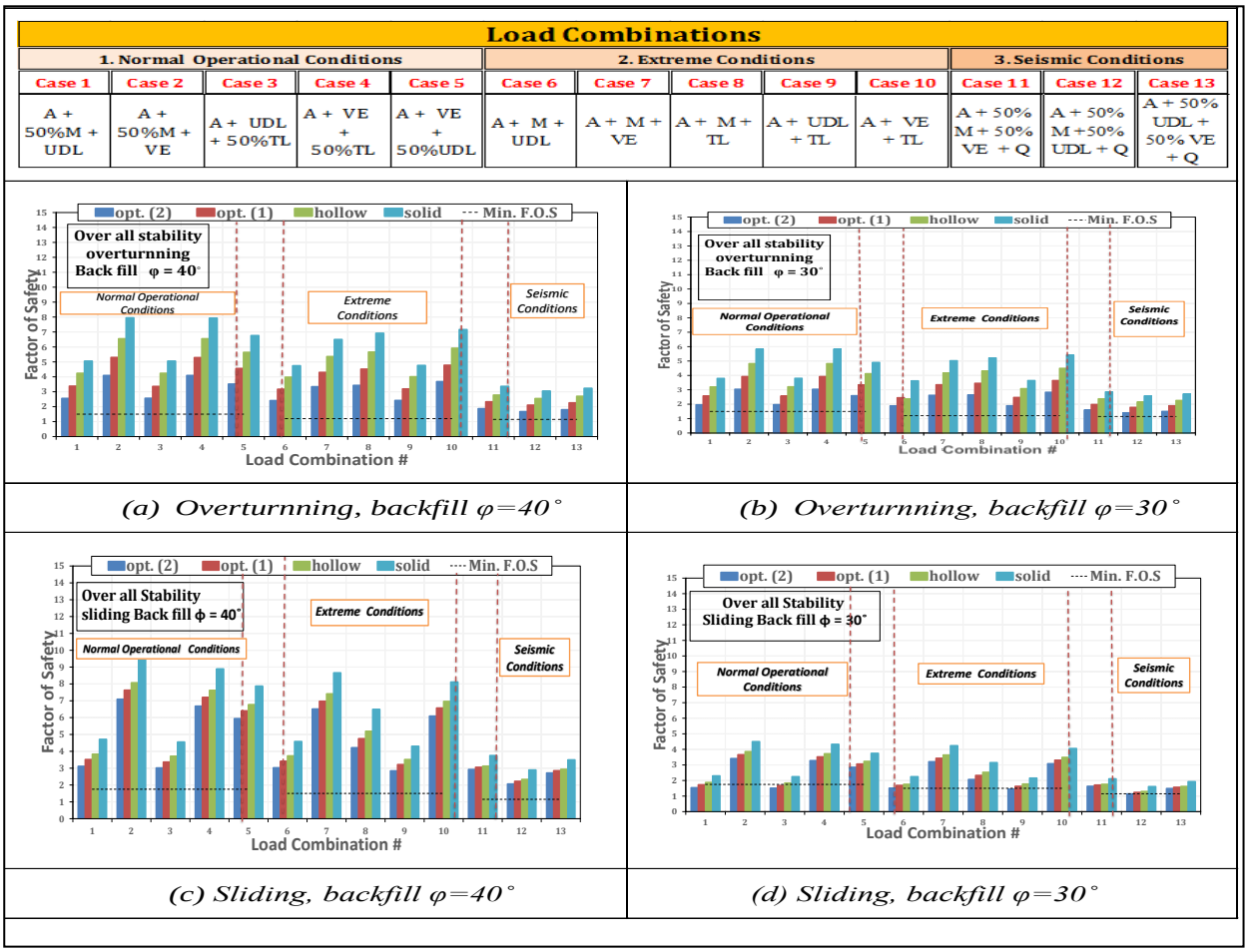


Figure 7: Relationship between all stages of optimization under 13 load combinations when changing the backfill characteristics from (φ = 40˚) to (φ = 30˚) for the overall stability against overturning and sliding.

4.1.2 Block internal stability analysis

The internal stability of layer 2 will be presented in this research which located on the bottom of the quay wall and considered the most important layer in analyzing.

1- Stability against overturning

From figures 8 (a and b), the results show that the layer 2 is safe against overturning for both back fill with φ = 30˚and φ = 40˚, but in general the factor of safety increased in case of φ = 40˚ in all stages of optimizations. Also, it is clear that the factor of safety under seismic conditions (load combination 11, 12 and 13) is lower than the other conditions and approaching to the value of the minimum factor of safety of seismic conditions especially in case of back fill of φ = 30˚.

Moreover, it is noticed that the load combinations including the vertical load (VE) increased the factor of safety such as (load combination 2, 4 and 10) while the load combinations including surcharge load (UDL) decreased the factor of safety such as (load combination 1, 3, 6 and 9) for all stages of optimization.

2- Stability against sliding

As shown in figure 8.c, it is concluded that the stability of the block quay wall for all stages of optimization under all load combinations against sliding is safe for back fill with φ = 40˚. While, as shown in figure 8. d, the factors of safety in case of back fill with φ = 30˚ are lower than their values in case of back fill φ = 40˚ in all stages of optimizations under all load combinations.

Also, as mentioned before that the critical load combinations (load combination 1, 3, 6 and 9) are unsafe in case of opt. (2), which means that these load combinations play an important role in the stability of the structure. Finally, the results of studying the stability against sliding for layer 2 showed that the structure is safe against sliding for all stages of optimizations under all load combinations in case of back fill with φ = 40˚, while the structure is unsafe in stage of opt.(2) with the case of back fill with φ = 30˚. Therefore, in order to solve this problem, it could be suggested to increase the shear resistance of the structure opt. (2) within the use of back fill with φ = 30˚ in order to make the structure safe.

Similar to the results obtained for the overall stability of the quay walls, the reduction of the backfill internal angle of friction (φ) from 40˚ to 30˚, reduce also the factors of safety against overturning and sliding.

The calculated factors of safety represented versus load combinations at every interface (beneath layers), provide a clean picture for the possibility of optimizing the studied quay walls and specify the load combination case

at which the factor of safety may exceed the permitted values belong to the operation conditions.

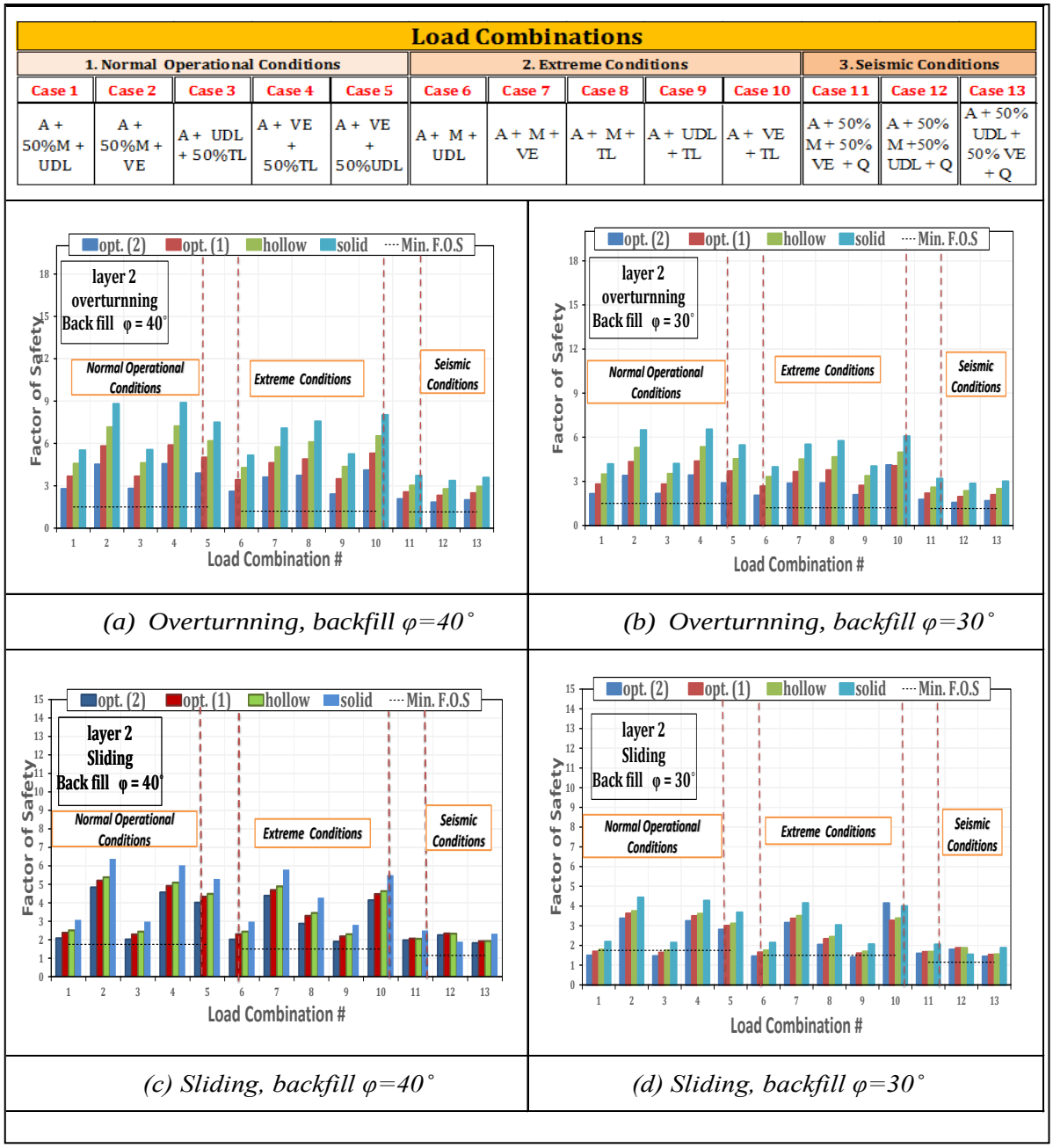


Figure 8: Relationship between all stages of optimization under 13 load combinations with back fill characteristics of (φ = 40˚) and (φ = 30˚) for the internal stability of layer 2 against overturning and sliding.

4.2 Geotechnical design consideration

In this section, the stability of the bock quay wall in all stages of optmizations considering the following geotechinal failure modes has been studied. The failure modes have been evaluated for the quay wall as follows:

- Foundation failure (bearing capacity).
- Deep slip failure.

4.3 Results of stability against foundation failure

4.3.1 Case of Back fill with φ = 40˚ , (φr = 40˚ and c=0 kpa)

4.3.1.1 Effect of Gravel bed depth, (d).

- **Substructure rubble layer, (d) = 2 m:**

In this section, the soil assumed to be silt with high plasticity with φ_s = 25˚ as in figure 9. a, while the soil assumed to be silt with low plasticity with φ_s = 28˚ as in figure 9. b. The results of these figures show that the

block quay wall is unsafe under all load combinations for all stages of optimization.

The soil assumed to be uniform fine sand with $\varphi_s = 30^\circ$ as shown in figure 9. c, while the soil assumed to be uniform fine sand with $\varphi_s = 32^\circ$ as in figure 9. d. Finally, the soil assumed to be well graded sand with $\varphi_s = 35^\circ$ as in figure 9.e. Finally, the results showed that the case of optimization (2) is considered the critical case of optimization which indicates that the constant substructure rubble layer depth must be increased in order to overcome this failure problem.

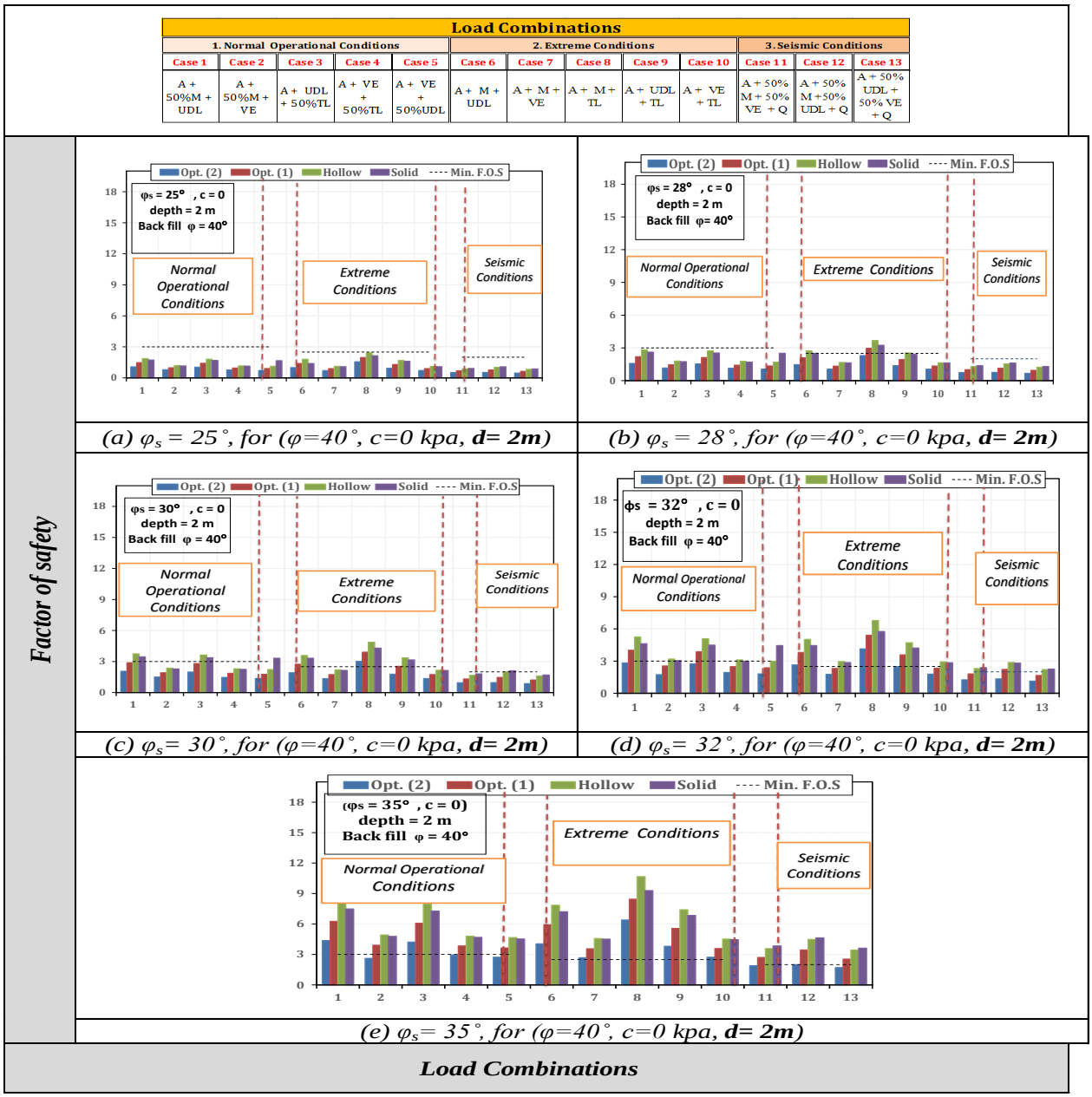


Figure 9: Effect of changing the sub-soil characteristics, (d) = 2m, $\varphi = 40^\circ$, ($\varphi_r = 40^\circ$ *and* $c = 0$ *kpa).*

- **Substructure rubble layer, (d) = 3m:**

Figures 10.a to 10.d show that the block quay wall is unsafe under all load combinations for all stages of optimization and the values of factor of safety are less than the minimum factor of safety even after changing the substructure rubble layer depth to 3m. Within the case of increasing φ_s to 35°, the block quay wall became full safe against foundation failure under all load combinations for all stages of optimization as shown in figure 10.e.

- **Substructure rubble layer, (d) = 4m:**

Figures 11.a to 11.c show that the block quay wall is unsafe under all load combinations for all stages of optimization, but figures 11.d and 11.e show that the block quay wall became more safe against foundation failure under all load combinations for all stages of optimization. By the end case of changing the angle of

friction (φ_s) for the foundation soil with a constant substructure rubble layer depth = 4m, it is concluded that as the value of angle of friction (φ_s) increases subsequently, the factor of safety of bearing capacity is increased and the better cases are occurred when the angle of friction (φ_s) = 32˚ and 35˚. So, increasing the properties of the foundation soil (φ_s) by increasing the substructure rubble layer depth is playing an important role for the stability against foundation failure (bearing capacity).

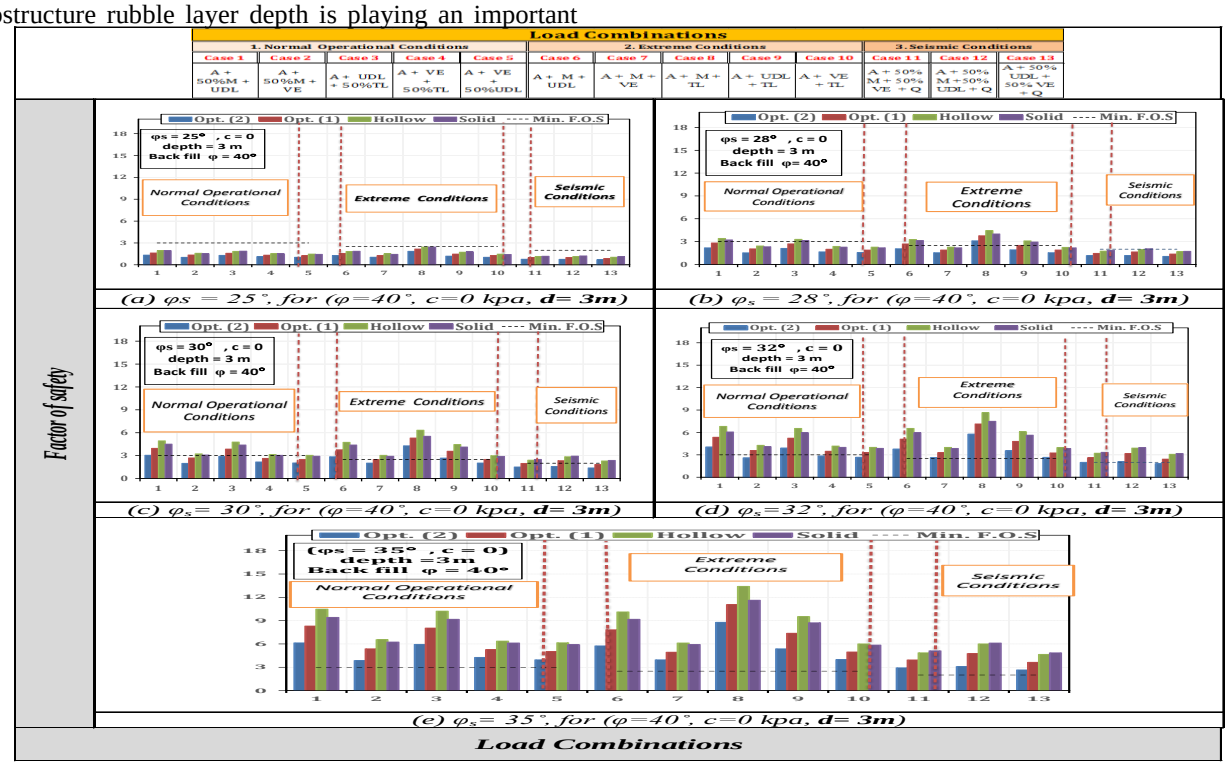


Figure 10: Effect of changing the sub-soil characteristics, (d) = 3 m, φ = 40˚ , (φr = 40˚ and c = 0 kpa).

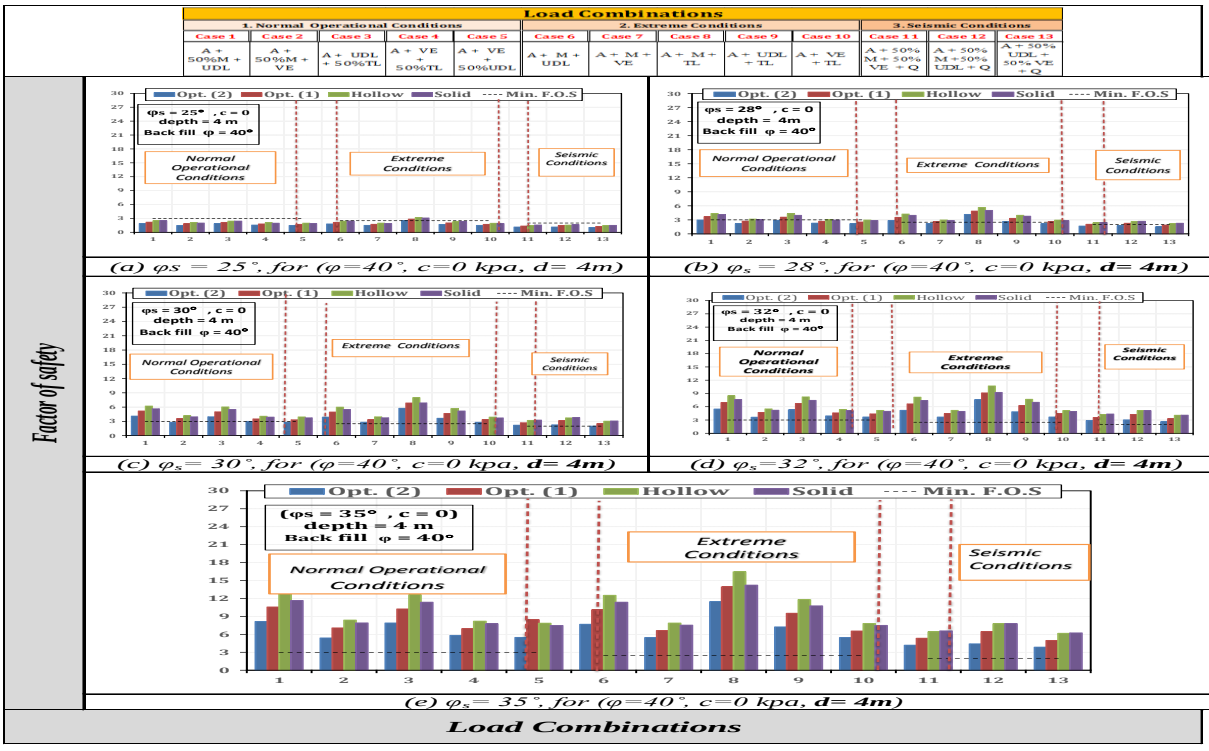


Figure 11: Effect of changing the sub-soil characteristics, (d) = 4 m,*φ = 40˚ , (*φ_r = 40˚ *and c = 0 kpa).*

4.3.2 Case of Back fill with φ = 40˚ , (φ_r = 40˚ and c=0 kpa)

4.3.2.1 Effect of Gravel bed depth, (d).

- **Substructure rubble layer, (d) = 2 m:**

In this section, the cohesion of the foundation soil has been increase from *c* = 0 kpa to *c* = 20 kpa for all cases of changing the sub soil characteristics (φ_s) from φ = 25˚ to φ = 35˚ when the substructure rubble layer depth is preserved constant at d = 2m for all stages of optimization under all load combinations as shown in figure 12.

By the end case of increasing the angle of friction (φ_s) in addition to increasing the cohesion *c* to 20kpa for the foundation soil with a constant substructure rubble layer depth d = 2m, it is concluded that the critical angle of friction occurred at φ_s = 25˚ while the best angle of friction occurred at φ_s =35 ˚. So, increasing the properties of the foundation soil (φ_s) and (c) playing an important role in stability against foundation failure (bearing capacity) for the same depth. Also, the results showed that the case of optimization (2) is considered the critical case of optimization which indicates that the constant gravel bed depth must be increased in order to overcome this failure problem. Following the same procedures, the effect of substructure rubble layer depths of d= 3 and 4 m are introduced in figures 13 and 14, respectively.

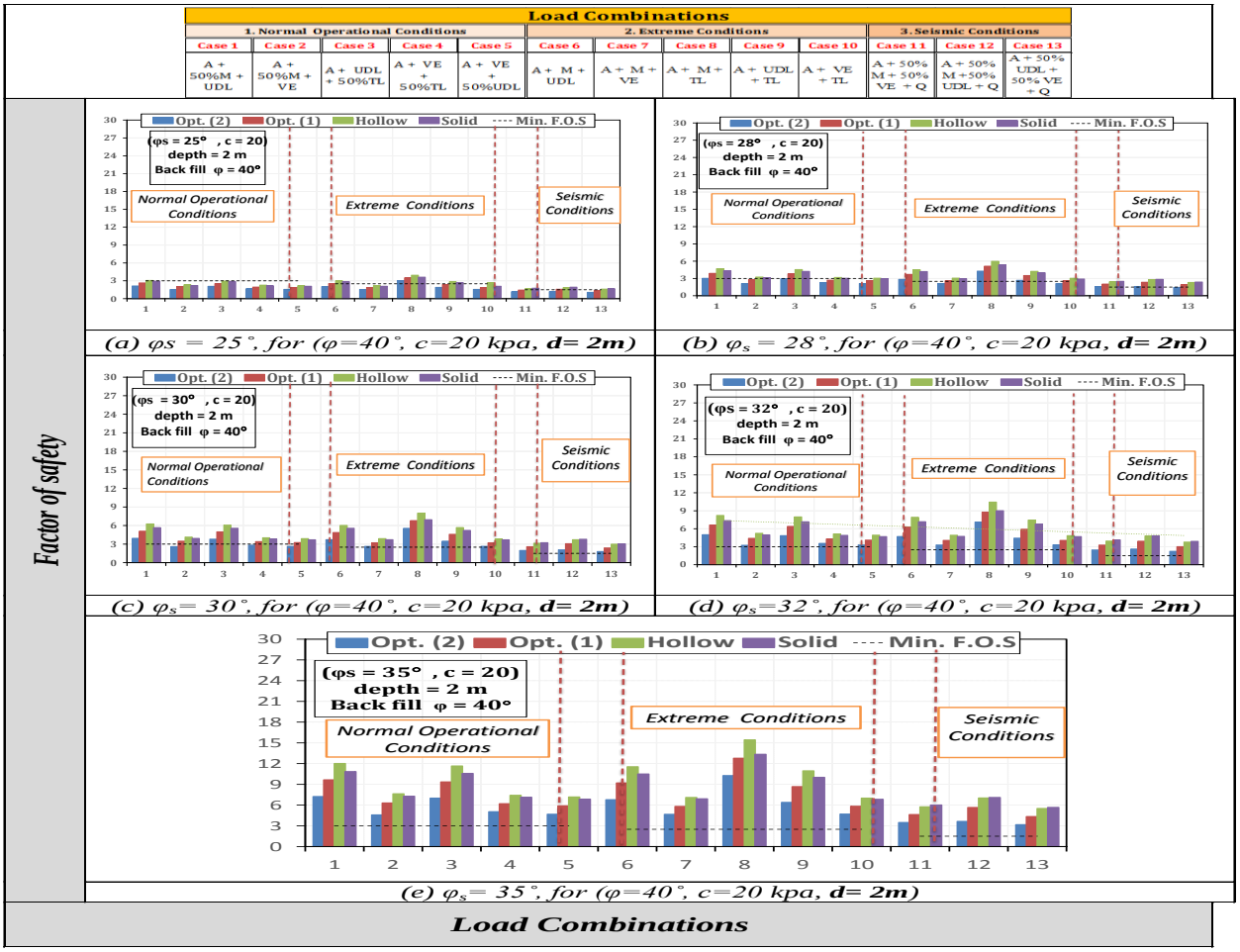


Figure 12: Effect of changing the sub-soil characteristics, (d) = 2 m,*φ = 40˚ , (φ_r = 40˚ and c = 20 kpa).*

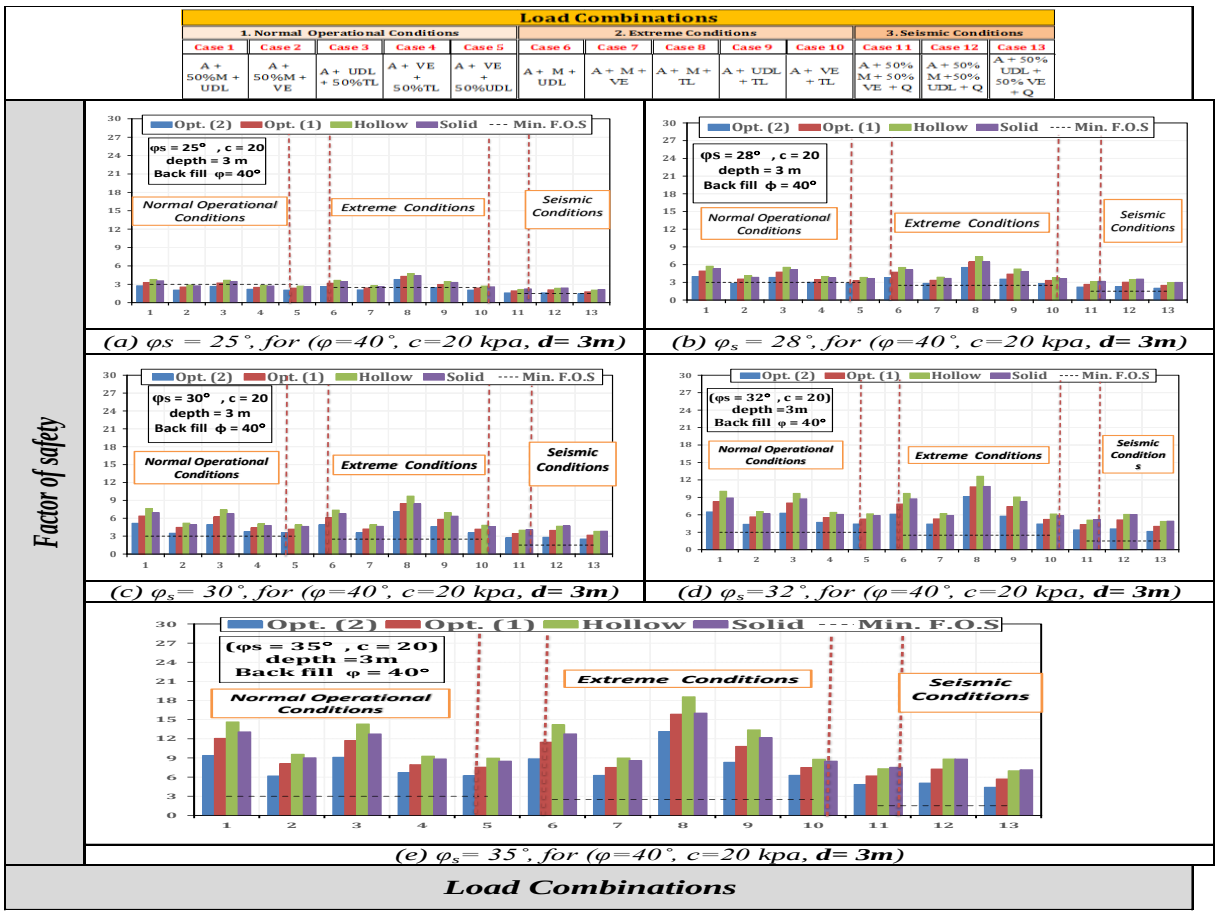


Figure 13: Effect of changing the sub-soil characteristics, (d) = 3 m $\varphi = 40^{\circ}$, ($\varphi_r = 40^{\circ}$ *and c = 20 kpa).*

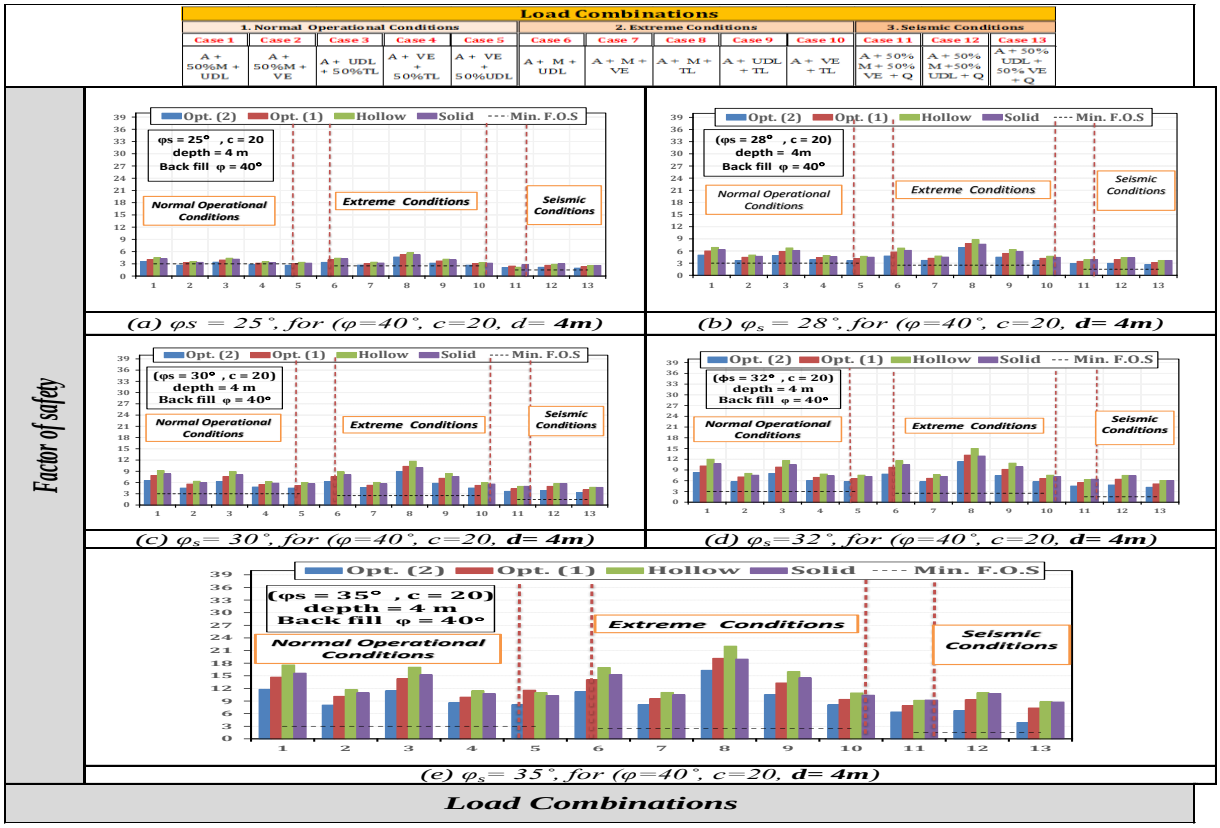


Figure 14: Effect of changing the sub-soil characteristics, (d) = 4 m,$\varphi = 40^{\circ}$, ($\varphi_r = 40^{\circ}$ *and c = 20 kpa).*

Figure 15 shows the effect of increasing the sub-soil angle of friction, φ_s on foundation bearing capacity, for different substructure rubble layer depths d = 2, 3, and 4 m, constant substructure rubble layer internal angle of friction φ_r = 40˚, constant backfill with φ = 40˚, and constant subsoil cohesion c = 20 kpa for the case of optimization (2) which considered the critical case of optimization. The results show that the factor of safety increased by the incresing the values of the angle of friction (φ_s) and substructure rubble layer depths, (d) subsequently.

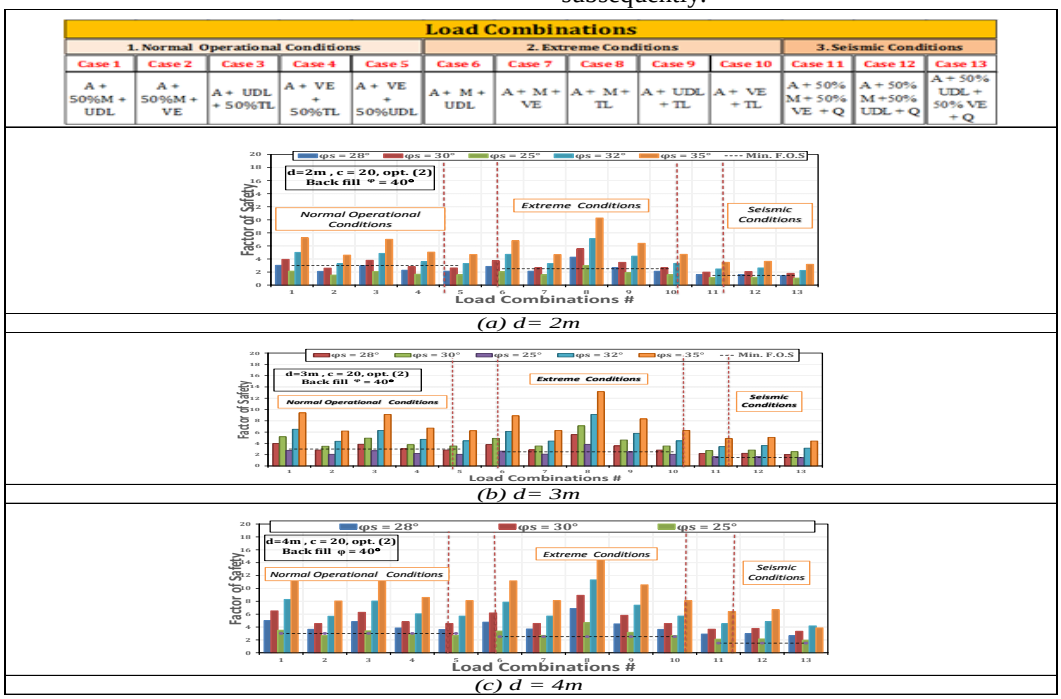


Figure 15: Effect of increasing the sub-soil angle of friction, φ_s for different values of substructure rubble layer depths, (d) on foundation bearing capacity, Case Opt.(2), *(φ = 40˚, φ_r = 40˚ and c = 20 kpa).*

In order to declare the pervious results, figure 16 shows the effect of changing the substructure rubble layer depths with d = 2m, 3m, and 4m at critical and best angle of friction φ_s = 25˚ and 35˚, respectively. This figure is done for the critical stage of optimization in which opt. (2) the critical stage of optimization is considered here.

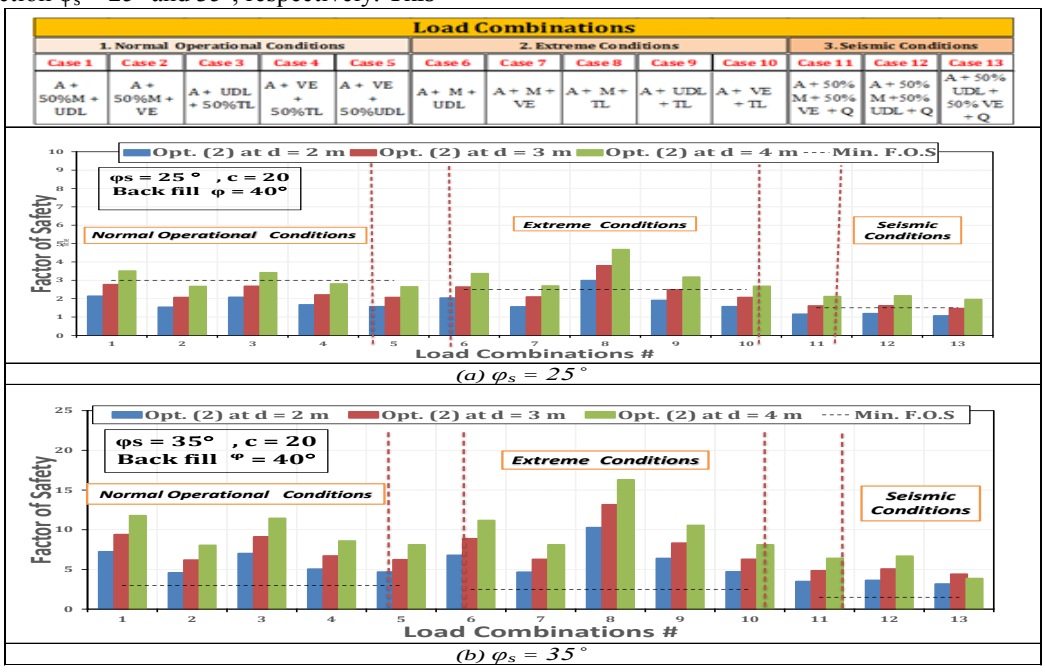


Figure 16: Effect of increasing substructure rubble layer depths, (d) for different values of sub-soil angle of friction, φ_s on foundation bearing capacity, Case Opt.(2), *(φ = 40˚, φ_r = 40˚ and c = 20 kpa).*

4.3.3 Case of Back fill with φ = 30˚ , (φ_r = 40˚ and c=0 kpa and 20 kpa)

Following the same procedures presented in section 4.3.1, the research investigations had been carried out for the block quay walls with internal angle of friction of backfill, φ = 30˚. Considering the backfill properties is constant (φ = 30˚), the investigations were performed for substructure rubble layer depths, d = 2, 3, and 4 m; subsoil internal angle of friction, φs = 25˚ , 28˚ , 30˚ , 32˚ and 35˚ with subsoil cohesion parameter c = 0 kpa and 20 kpa. All investigations were performed with constant internal angle of friction of substructure rubble layer, φr = 40˚. Part of these results is given in figures 17 to 19 in the form of comparison to the previous similar studied cases presented in section having backfill angle of friction φ = 40˚.

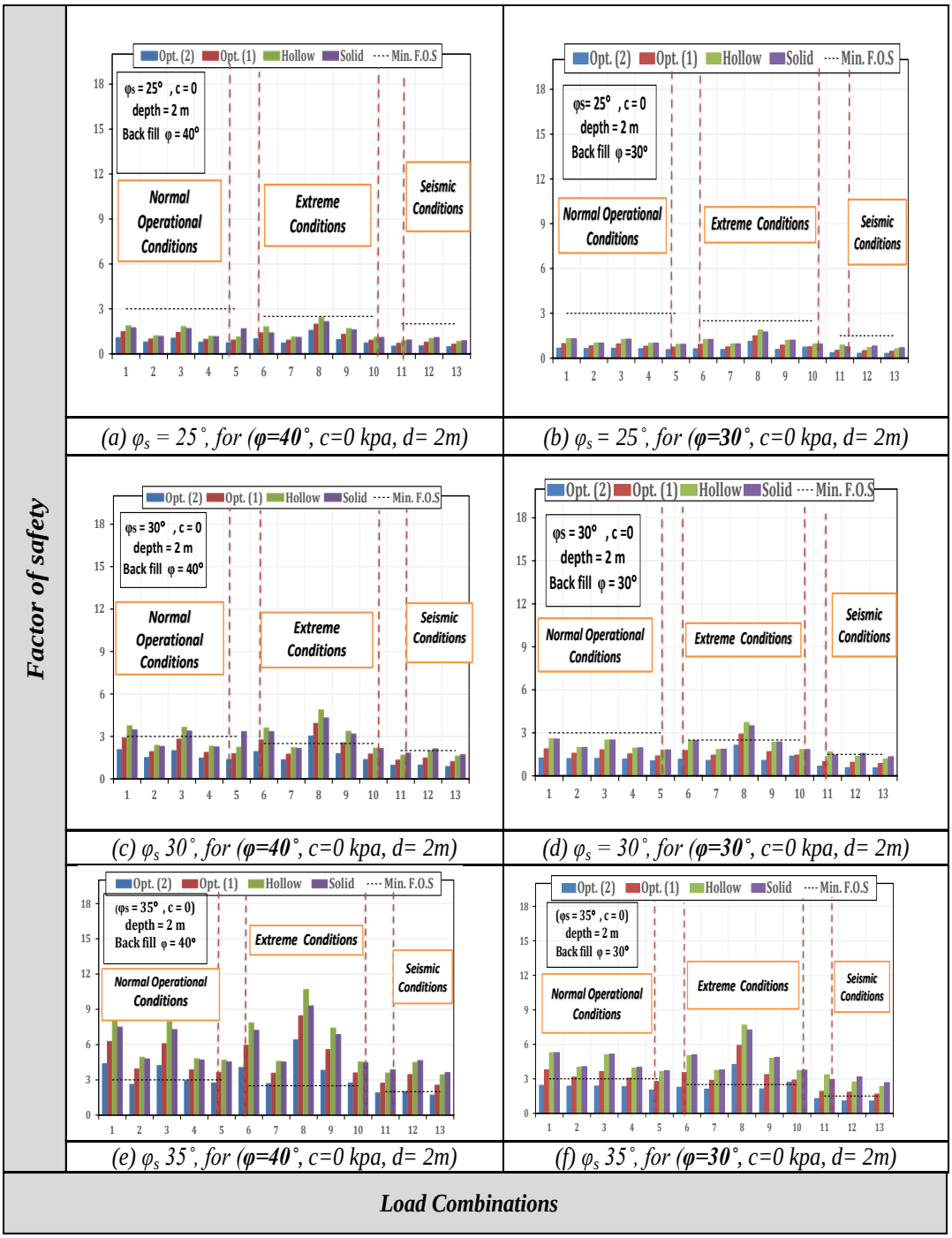


Figure 17: Effect of increasing backfill angle of friction, φ on foundation bearing capacity, (d = 2m, φ_r = 40˚ and c = 0 *kpa*).

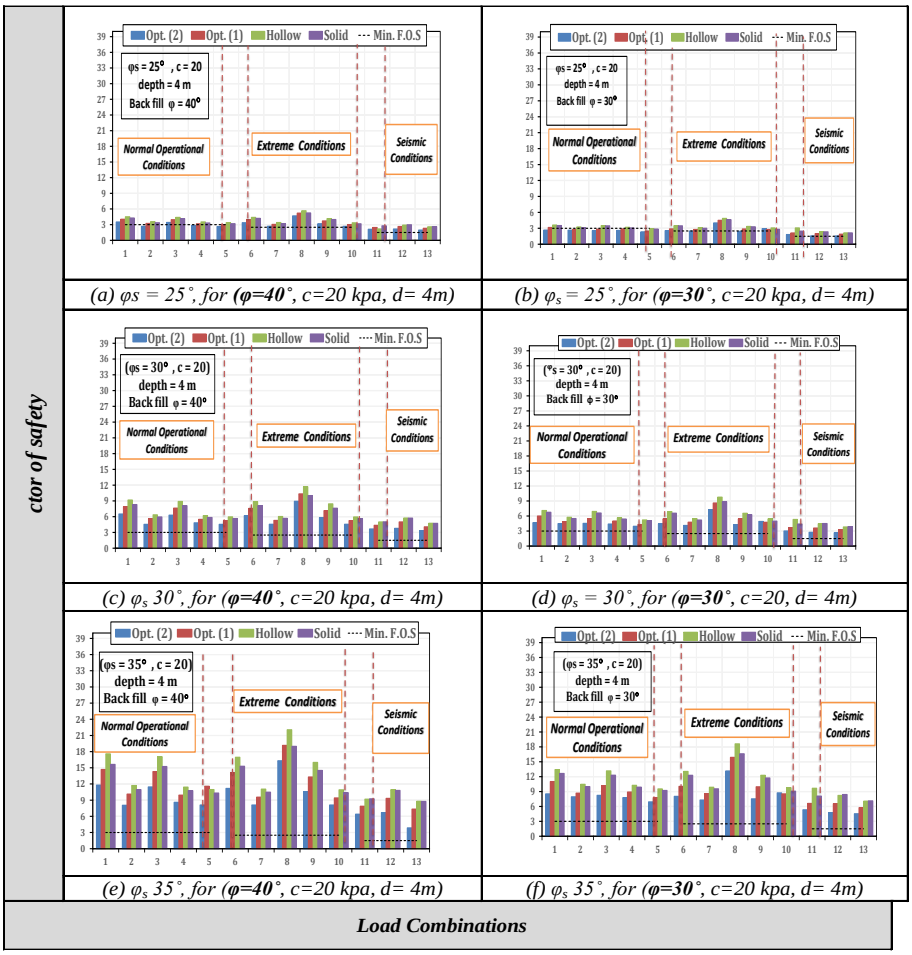


Figure 18: Effect of increasing backfill angle of friction, φ on foundation bearing capacity, (d = 4m, $\varphi_r = 40^\circ$ and c = 20 *kpa*).

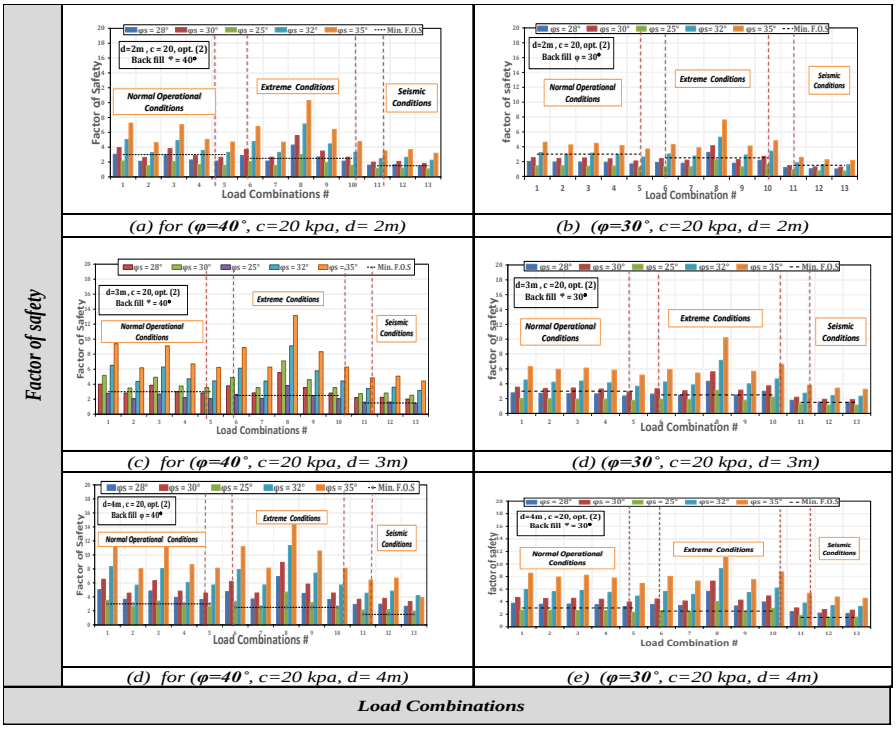


Figure 19: Effect of decreasing backfill angle of friction, φ on foundation bearing capacity for different values of substructure rubble layer depths, (d) Case Opt.(2), ($\varphi_r = 40^\circ$ *and c = 20 kpa)*.

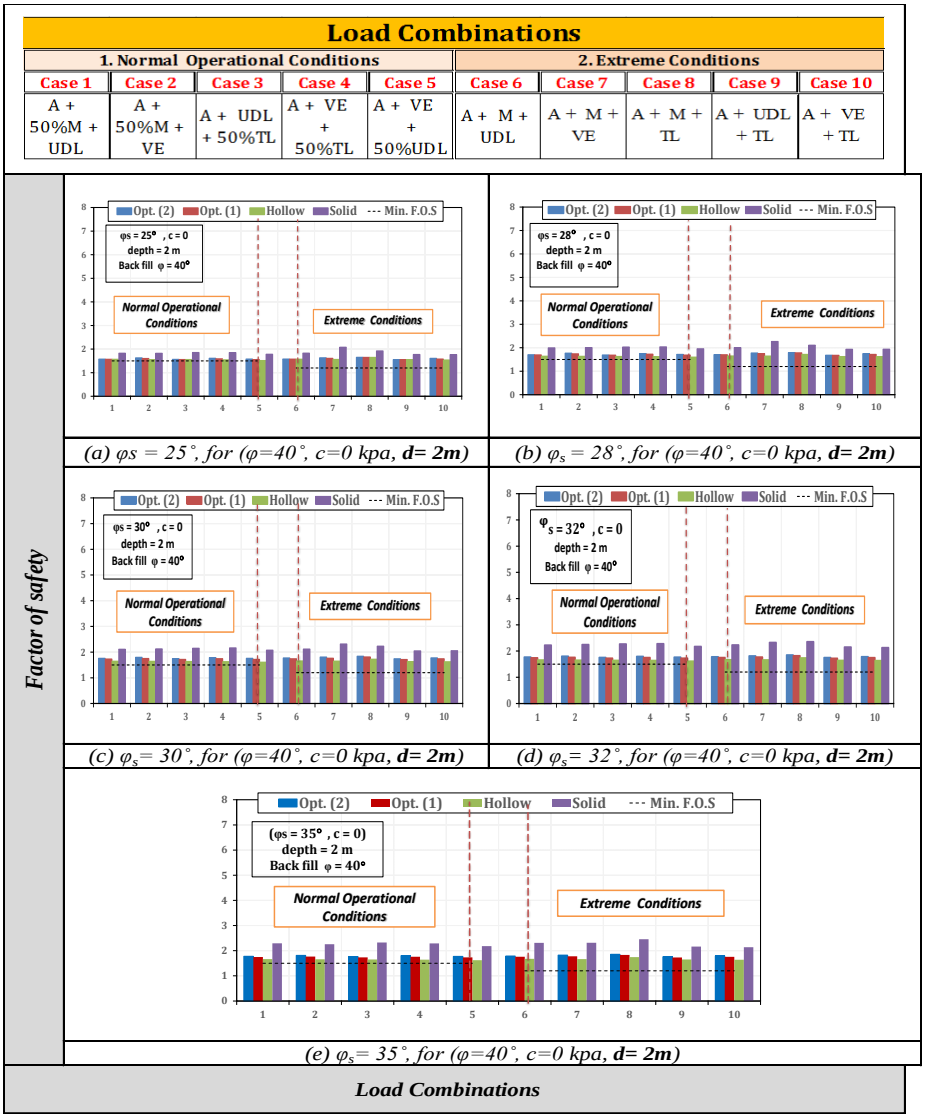
The calculated factors of safety against foundation failure represented versus load combinations provide a clean picture for the possibility of optimizing the studied quay walls and specify the load combination case at which the factor of safety may exceed the permitted values belong to the operation conditions.

4.4 Deep slip failure (global)

Figures 20.a to 20.e show factor of safety against deep slip failure versus load combinations for backfill of φ = 40˚, cohesion of the foundation soil from c = 0 kpa and substructure rubble layer depth, d = 2m with different values of internal angle of friction for the subsoil, φ_s = 25˚, 28˚, 30˚, 32˚ and 35˚, respectively. The figures show that the factor of safety against deep slip failure does not affect significantly with the change of φ_s for all studied load combinations.

While, Figures 21.a to 21.e show factor of safety against deep slip failure versus load combinations for backfill of φ = 40˚, cohesion of the foundation soil from c=0 and substructure rubble layer depth, d = 4 m with different values of internal angle of friction for the subsoil, φ_s = 25˚, 28˚, 30˚, 32˚, and 35˚, respectively. The figures show that the factor of safety against deep slip failure also does not affect significantly neither by the change of φ_s nor by the change of substructure rubble layer depth, d, for all studied load combinations.

Figures 22.a to 22.f show the effect of soil cohesion parameter beneath the foundation, c on the safety factor against slip failure at constant substructure rubble layer depth, d = 2m. This was carried out by comparing each studied case with c = 0 kpa to its similar case with c = 20 kpa. The figure shows also that the safety factors are affected slightly with the change of cohesion parameter.

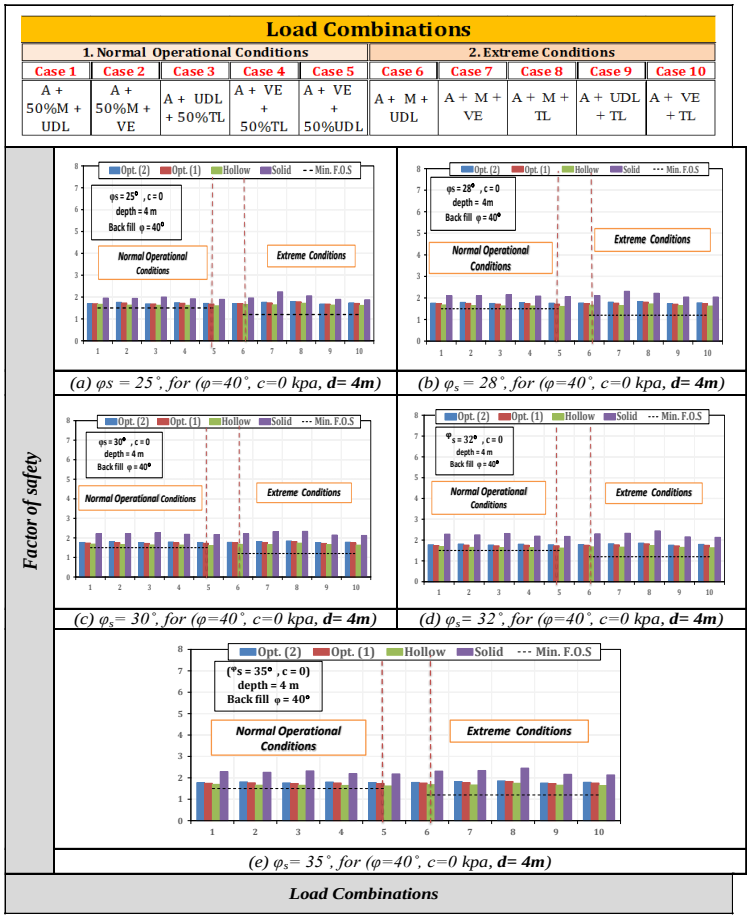


(a) φs = 25˚, for (φ=40˚, c=0 kpa, **d= 2m**) (b) φ_s = 28˚, for (φ=40˚, c=0 kpa, **d= 2m**)

(c) φ_s= 30˚, for (φ=40˚, c=0 kpa, **d= 2m**) (d) φ_s= 32˚, for (φ=40˚, c=0 kpa, **d= 2m**)

(e) φ_s= 35˚, for (φ=40˚, c=0 kpa, **d= 2m**)

Figure 20: Effect of changing the sub-soil characteristics on slip failure stability, (d) = 2m, φ = 40˚, (φ_r = 40˚ and c = 0 kpa).

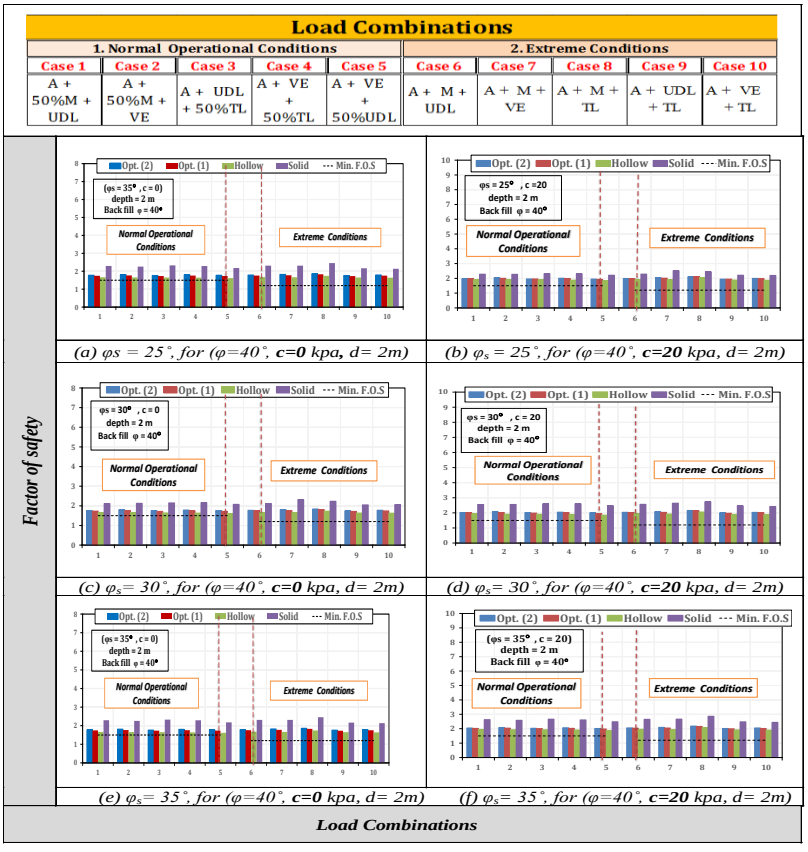


(a) φs = 25˚, for (φ=40˚, c=0 kpa, **d= 4m**) (b) φ_s = 28˚, for (φ=40˚, c=0 kpa, **d= 4m**)

(c) φ_s= 30˚, for (φ=40˚, c=0 kpa, **d= 4m**) (d) φ_s= 32˚, for (φ=40˚, c=0 kpa, **d= 4m**)

(e) φ_s= 35˚, for (φ=40˚, c=0 kpa, **d= 4m**)

Figure 21: Effect of changing the sub-soil characteristics on slip failure stability, (d) = 4m, φ = 40˚, (φ_r = 40˚ *and c = 0 kpa)*



(a) φs = 25˚, for (φ=40˚, **c=0** kpa, d= 2m) (b) φ_s = 25˚, for (φ=40˚, **c=20** kpa, d= 2m)

(c) φ_s= 30˚, for (φ=40˚, **c=0** kpa, d= 2m) (d) φ_s= 30˚, for (φ=40˚, **c=20** kpa, d= 2m)

(e) φ_s= 35˚, for (φ=40˚, **c=0** kpa, d= 2m) (f) φ_s= 35˚, for (φ=40˚, **c=20** kpa, d= 2m)

Figure 22: Effect of changing the sub-soil characteristics, (c) on slip failure stability, (d) = 2m, φ = 40˚, φ_r = 40˚

Through the calculations carried out in the current study using: backfill, subsoil and substructure rubble layer with different characteristics; several diagrams representing factors of safety for foundation failure (slip failure) versus load combinations had been introduced which concluded the followings:

- Reducing the backfill internal angle of friction (φ) from 40˚ to 30˚, reduces the factors of safety against foundation failure (slip failure) for all studied load combinations with little values,
- Increasing the substructure rubble layer depth, (d) doesn't effect on slip failure factor of safety,
- Increasing the subsoil internal angle of friction (φ_s), improves the slip failure factor of safety with little values,
- Increasing the subsoil cohesion parameter (c), improves the slip failure factor of safety with little values,
- The calculated factors of safety against deep slip failure represented versus load combinations provide a clean picture for the possibility of optimizing the studied quay walls and specify the load combination case at which the factor of safety may exceed the permitted values belong to the operation conditions.

5. CONCLUSIONS

The present work demonstrates a verification study for the ability of optimizing the block quay wall for four stages under 13 load combinations. The first stage is to study the stability of the solid block quay wall under all load combinations; the second stage is to study the stability of the quay wall when replacing the solid concrete blocks by hollow precast ones by making three holes through the quay wall. Every hole is filled with clayey gravel under all load combinations, in the third stage (opt. 1) every hole of clayey gravel are optimized by decreasing its width by 0.5m and studying the stability under all load combinations, finally in four stage (opt. 2) every hole of clayey gravel was optimized by decreasing it's width by 0.5m extra for each block while the height of the block wall is kept constant along the four stages of optimization. The analyzed results of the study including the factor of safety against sliding. Overturning, deep slip failure and bearing capacity obtained for the four stages of optimization under all load combinations had been presented when changing the back fill characteristics, the cohesion of the subsoil and the depth of the substructure rubble layer. It could be concluded that, reducing the backfill internal angle of friction (φ) from 40˚ to 30˚, reduces the factors of safety against overturning, sliding, bearing capacity and deep slip failure.

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"دراسه الوصول لقطاع أمثل لحائط رصيف من البلوكات الخرسانيه"

من المؤكد ان المنشات البحريه لها دور كبير وفعال في تسهيل اعمال الموانئ بالاضافه الي حمايه خط الشاطئ و لذلك فان هذا البحث سوف يلقي الضوء علي دراسه وتحليل الاتزان لنوع مهم من المنشات البحريه وهما حائط رصيف من البلوكات المفرغه بالنسبه للرصيف من البلوكات المفرغه فانه مكون من 15 صف من البلوكات المفرغه المرقمه تدريجيا من اسفل الي اعلي فان الهدف الرئيس من دراسته الوصول للقطاع الامثل عن طريق تقليل الابعاد له مع تحقيق الاتزان عن طريق استبدال البلوكات الصلبه باخري مفرغه مختلفه ل 4 مراحل من تقليل أبعاد القطاع تدريجيا تحت تاثير 13 حاله تحميل مختلفه واستنتاج معاملات الامان لكل مرحله من مراحل تقليل الابعاد تدريجيا وتحديد معاملات الامان لكل مرحله ضد الانزلاق والدوران عن طريق استخدام برنامج GEO 5 , قياس مدي تحمل التربه تحت تاثير كل حاله تحميل خلال جميع مراحل تقليل ابعاد القطاع عن طريق استخدام Hansen's Equations واخيرا دراسه الاتزان ضد دائره الانهيار عن طريق استخدام بزنامج Slope/W . وقد اوضحت النتائج ان المرحله الحرجه من تقليل الابعاد هيا المرحله الاخيره وهي opt 2 بالاضافه الي ذلك تقليل زاويه الاحتكاك الداخلي للكوم الخلفي من الردم يقلل من مدي تحمل التربه للانهيار كما ان زياده تماسك التربه c يحسن من مدي تحمل التربه للانهيار.