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OPTIMUM DESIGN OF REINFORCED EARTH WALLS WITH METAL STRIPS; SIMULATION-OPTIMIZATION APPROACH

H. Ghiassian* and K. Aladini
Department of Civil Engineering, Iran University of Science and Technology, Tehran, Iran

ABSTRACT

Mechanically Stabilized Earth Walls (MSEW) and Reinforced Soil Slopes (RSS) are usually considered as cost-effective soil-retaining structures. By inclusion of tensile reinforcing elements 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. Based on limited data, reinforcement accounts for 45 to 65 percent of total cost. This paper couples a complete MSE wall design routine with a highly efficient optimization model for optimum design of mechanically stabilized earth walls. The design algorithm benefits from limit equilibrium technique to calculate the external and internal stability of the wall considering common safety factors. The proposed safety factors are treated as constraint to the problem. The optimization model uses GA to search for optimum combination of the design variables to satisfy the required safety factors. Integration of simulation- optimization approaches for optimum design of MSE walls is the first in its kind which has been overlooked in the literature. Application of the model in few case examples shows that up to 15 percent may be saved in design specific cost in relatively high walls.

Keywords: Design; mechanically stabilized earth; simulation; optimization

1. INTRODUCTION

Mechanically Stabilized Earth Walls (MSEW) and Reinforced Soil Slopes (RSS) are usually considered as cost-effective soil-retaining structures. By inclusion of tensile reinforcing elements in the soil, the strength of the soil can be improved significantly. In this case the vertical face of the soil/reinforcement system is essentially self supporting a facing system is usually used to prevent soil raveling between the reinforcing elements, which allows very steep slopes and vertical walls to be constructed safely.

MSE walls offer significant technical advantages and cost saving over common reinforced concrete retaining structures at sites with poor foundation conditions. It is shown that in poor foundation conditions the elimination of costs for foundation improvements such as piles and pile caps have resulted in cost savings of greater than 50 percent on

* Email-address of the corresponding author: h_ghiassian@iust.ac.ir (H. Ghiassian)

completed projects [1]. The reinforcement in the soil structure is basically used to construct a reinforced soil slope embankment at an angle steeper than could otherwise be safely constructed with the same soil.

Site specific costs of a soil-reinforced structure are a function of many technical and physical factors. In-situ soil type, cut-fill requirements, wall/slope size and type, available backfill materials, facing finish, temporary or permanent applications are among the most important parameters.

Total cost of a specific MSE/RSS structure depends on the cost of each of its principal components. Based on limited data, reinforcement accounts for 45 to 65 percent of total cost. High MSE/RSS structures have relatively higher reinforcement and lower backfill costs. Recent cost estimation and analysis suggest costs ranging from $110/m^2 to $260/m^2 ($10/ft^2 to $24/ft^2) as a function of height [1]. MSE/RSS systems can be described by the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing and connections.

Since the development of soil reinforcement concepts and their application to MSEW structure design, many design methods have been developed, practiced, and refined. Current practice consists of the geometric determination and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis.

Internal stability is treated as a response of discrete elements in a soil mass. This suggests that deformations are controlled by the reinforcements rather than total mass, which appears inconsistent given the much greater volume of soil in such structures. Therefore, deformation analyses are generally not included in current methods [2].

External stability evaluations for MSEW structures treat the reinforced section as a composite homogeneous soil mass and evaluate the stability according to conventional failure modes for gravity type wall systems.

Given the availability of different methods and research in the last decade, general agreement has been reached that a complete design approach should consist of Working Stress analyses, Limit Equilibrium analyses, and Deformation Evaluations [1].

The Working Stress method relies upon restrictive assumptions with regard to the state of stress in the soil. The Limit Equilibrium method essentially uses conventional slope stability analysis, modified to account for the reinforcement effect, for the global stability of the reinforced soil mass.

For routine design of retaining walls, all methods are normally considered applicable. Most of the methods use limiting equilibrium analysis to determine factor of safety against failure; however, they differ in their assumptions regarding stress distribution, failure surfaces, safety factors and the inclination of the reinforcement at the failure surface. The different design methods have been developed, modified, applied, and/or criticized by various authors [2-8]. Ehrlich and Mitchell [7] developed and evaluated a method for the internal design of reinforced soil walls based on working stresses. They used measurements from five full scale reinforced soil structures with a wide range of reinforcement types. It was shown that, in general, the stiffer the reinforcement system and the higher the stresses induced during compaction, the higher are the tensile stresses that must be resisted by the reinforcements.

Several studies have been undertaken to enhance the available methodologies for the analysis of reinforced earth walls. However, limited works have been reported in developing methods for their optimum cost design. Jie Han and Leshchinsky [9] presented a general unified scheme for limit equilibrium analysis to facilitate the design of flexible reinforced earth slopes and walls. The scheme yields a rational methodology to find the distribution of required tensile resistance along each reinforcement layer for a given reinforced slope/wall problem so that the factor of safety on the strength of soil is more or less uniform everywhere within the reinforced soil zone. Xue and Gavin [10] proposed a method for simultaneous determination of critical slip surface and reliability index for slopes.

Basudhar et al [11] employed the Sequential Unconstrained Minimization Technique (SUMT) in conjunction with conjugate direction and quadratic fit methods for multidimensional and unidirectional minimization to arrive at the optimal (minimum) cost of the reinforced earth wall. Choice of the initial designed length and strength of the reinforcement, which are the elements of the design vectors, were made in a way that it formed an initial feasible design vector. Chalermyanont and Benson [12] proposed a two-phase approach to develop a reliability-based design method for external stability of mechanically stabilized earth (MSE) walls. In the first phase, a parametric study was conducted using Monte Carlo simulation to identify parameters that affect the probability of external failure of MSE walls. In the second phase, a series of additional simulations were conducted where the significant parameters identified in the parametric study were varied over a predetermined range. Chalermyanont and Benson [13] conducted a parametric study using Monte Carlo simulation to assess how uncertainty in design parameters affects the probability of internal failure of mechanically stabilized earth (MSE) walls. Bishop's simplified method was used to conduct the internal stability analyses through a series of Monte Carlo simulations.

This paper intends to couple a complete MSE wall design routine with a highly efficient optimization model for optimum design of mechanically stabilized earth walls. The design algorithm benefits from limit equilibrium technique to calculate the external and internal stability of the wall considering 5 common safety factors. The proposed safety factors are treated as constraint to the problem. The optimization model uses GA to search for optimum combination of the design variables to satisfy the required safety factors. Integration of simulation-optimization approaches for optimum design of MSE walls is the first in its kind which has been overlooked in the literature. A simulation-optimization interaction loop (SOIL) is defined that cycles between the safety factor evaluation module of a MSE wall (i.e., the simulator) to couple the system stability and the GA optimization algorithm (the optimizer). The exchange of information between the simulator and the optimizer in the interaction loop facilitates convergence to an optimal solution. The coupled MSE wall simulation module and the GA optimization algorithm locate the reinforcements at appropriate locations with optimum spacing in vertical and horizontal directions. The model may equally be used to optimally design the MSE walls with other reinforcement means.

2. GENETIC ALGORITHM

In a GA, members of a population of abstract representations of candidate solutions to an optimization problem are stochastically selected, recombined, mutated, and then either eliminated or retained, based on their relative fitness. The GA technique has been notably developed by Goldberg (1989), who also gives an excellent introduction to the subject. The first step in GA formulation is to code the components of possible solutions in a chromosome. Each chromosome represents a potential solution consisting of the components of the decision variables (also known as genes) that either form or can be used to evaluate the objective function. The entire population of such chromosomes represents a generation. For a typical design problem, the genes consist of the design variables to be selected (decision variables) that are concatenated to form chromosomes.

For the purpose of exposition, suppose there is a MSE wall for which the best combination of 5 design parameters must be selected. The chromosome representing a solution to such a problem consists of 5 genes representing the decision variables of the problem. The fitness of a chromosome as a candidate design is a function of these genes and is obtained by evaluating the objective function of the problem. The objective function for a MSE wall may be defined as minimum total cost of the wall construction. In application of any search algorithm (i.e., GA) to design problems, infeasible chromosomes may be generated that fail to satisfy the system constraints, such as different safety factor and/or reliability requirements. The fitness of a chromosome is also a function of problem constraints and may be modified through the introduction of penalties when constraints are violated. The reproduction mechanism in a GA is composed of selection, crossover, and mutation. A number of representation, crossover, and mutation schemes have been proposed and successfully practiced [14].

3. STABILITY ANALYSIS

Since the development of soil reinforcement concepts and their application to MSEW structure design, a number of design methods have been proposed, used, and modified.

Considering the availability of different methods and research in the last decade, it is generally agreed that a complete design approach should consist of the following:

- Working stress analyses.
- Limit equilibrium analyses.
- Deformation evaluations.

Analysis of working stresses for MSEW Structures consists of

- Selection of reinforcement location and a control that stresses in the stabilized soil mass to be compatible with the properties of the soil and inclusions.
- Evaluation of local stability at the level of all reinforcements and prediction of progressive failure.

Limit equilibrium analysis consists of a check of the overall stability of the structure. The types of stability that must be considered are external, internal, and combined.

External stability involves the overall stability of the stabilized soil mass considered as a whole and is evaluated using slip surfaces outside the stabilized soil mass. Internal stability analysis consists of evaluating potential slip surfaces within the reinforced soil mass. In some cases, the critical slip surface is partially outside and partially inside the stabilized soil mass and a combined external/internal stability analysis may be required.

Deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to horizontal and vertical displacement. In addition, the influence and variations in the type of reinforcement on the performance of the structure can be evaluated.

Current practice consists of determining the geometric and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis.

Analysis Method, Inextensible Reinforcements (e.g. metal strips)

The current method of limit equilibrium analysis uses a coherent gravity structure approach to determine external stability of the whole reinforced mass. This approach is basically similar to the analysis for any conventional or traditional gravity structure. For internal stability evaluations, it considers a bi-linear critical slip surface. The assumed critical slip surface divides the reinforced mass in active and resistant zones and requires that an equilibrium state be achieved for successful design.

The state of stress for external stability, is assumed to be equivalent to a Coulomb state of stress with a wall friction angle δ equal to zero. For internal stability a variable state of stress varying from a multiple of Ka to an active earth pressure state, K_a, is used for design.

3.1 External Stability Analysis

External stability evaluations for MSEW structures treat the reinforced section as a composite homogeneous soil mass and evaluate the stability according to conventional failure modes for gravity type wall systems.

As with classical gravity and semi-gravity retaining structures, three potential external failure mechanisms are usually considered in analysis of MSE walls. They include [2]:

a. Sliding on the base.
b. Overturning which limits the location of the resultant of all forces
c. Bearing capacity.

The Factor of Safety for sliding is usually calculated as the ratio of the shear resistance along the base of the MSE wall to the active thrust due to lateral earth pressure. The Factor of Safety for overturning is calculated as the ratio of the resisting moment to driving moment. The resistance moment is caused by the weight of the wall, whereas the driving moment is caused by the active thrust about the toe of the wall. The Factor of Safety for bearing capacity is calculated as the ratio of the ultimate bearing capacity of the foundation soil to the stress imposed by the weight of backfill soil.

The coefficient of active earth pressure, K_a, is computed using the Rankine theory with friction angles corresponding to the cells next to the backfill soil mass [1,2]

$$K_a = tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right) \tag{1}$$

The total active thrust is computed as:

$$F_T = \frac{1}{2} K_a \gamma H^2 \tag{2}$$

Meyerhof's bearing capacity equation is often used to calculate the bearing capacity of the foundation soil under an MSE wall [15]. Bearing capacity theory assumes that the soil is homogeneous. Thus, when computing the bearing capacity, the friction angle and unit weight of the foundation soil are assigned as the arithmetic mean of the friction angle and unit weight of foundation soil in an "effective area" of the foundation [12]. Bearing capacity (q_{ult}) of the cohesionless foundation may be computed as [12]

$$q_{ult} = \frac{1}{2}(L - 2ec)\gamma_F N_\gamma \tag{3}$$

Where,

$$N_\gamma = (N_q - 1)\tan(1.4\varphi_F) \tag{4}$$

$$N_q = exp(\pi \tan\varphi_F)\tan^2(\frac{\pi}{4} + \frac{\varphi_F}{2}) \tag{5}$$

eccentricity due to the active thrust

$$ec = \frac{F_T H}{3W} \tag{6}$$

The weight of reinforced soil mass is easily determined as

$$W = \gamma HL \tag{7}$$

Now, the equivalent uniform vertical stress on the base, σ_V, may be calculated as

$$\sigma_V = \frac{W}{L - 2ec} \tag{8}$$

The friction ratio of the reinforced soil-foundation interface is computed as

$$\mu = Min\,(\tan\varphi_F, \tan\varphi_R, \tan\varphi_B) \tag{9}$$

Having determined all parameters, the sliding, overturning, and bearing capacity safety factors can be calculated as

$$FS_{SL} = \frac{W\mu}{F_T} \tag{10}$$

$$FS_{BC} = \frac{q_{ult}}{\sigma_V} \tag{11}$$

$$FS_{OT} = \frac{3WL}{2F_T H} \tag{12}$$

These safety factors are to exceed the predefined values for structures external stability.

3.2 Internal Stability Analysis

The response of discrete elements in a soil mass is controlled by internal stability. This suggests that deformations are controlled by the reinforcements rather than total mass, which appears inconsistent given the much greater volume of soil in such structures.

Internal failure of a MSE wall can occur in two different ways [1].

1. The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the reinforcements become so large that the reinforcements elongate excessively or break, leading to large movements and possible collapse of the structure. This mode of failure is called failure by elongation or breakage of the reinforcements.
2. The tensile forces in the reinforcements become larger than the pullout resistance, i.e., the force required to pull the reinforcement out of the soil mass. This, in turn, increases the shear stresses in the surrounding soil, leading to large movements and possible collapse of the structure. This mode of failure is called failure by pullout [1].

In a simple reinforced soil wall one may assume that the most critical slip surface can approximately be bilinear in the case of inextensible reinforcements and passes through the toe of the wall, Figure 1.

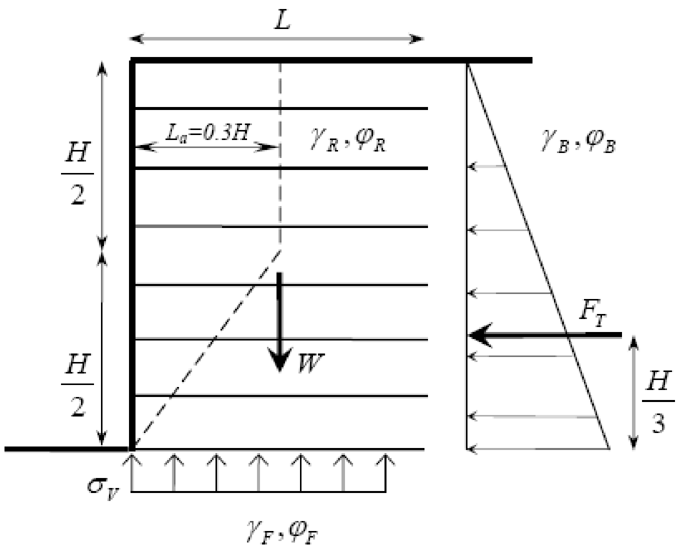

Figure 1. Mechanically stabilized earth wall model

Assuming no wall friction, the active earth pressure coefficient is determined using a Coulomb earth pressure relationship, Therefore, for r a vertical wall the earth pressure reduces to the Rankin equation [1, 2]

$$K_a = tan^2(\frac{\pi}{4} - \frac{\varphi}{2}) \tag{13}$$

The horizontal stresses σ_H along the potential failure line at each reinforcement level may be calculated from the weight of the retained fill $\gamma_r z$:

$$\sigma_V = z\,\gamma_R \tag{14}$$

$$\sigma_H = \sigma_V\,K_r \tag{15}$$

The resulting K/K_a for inextensible reinforcements ratio decreases from the top of wall to a constant value below 6 m [1].

Maximum tension T_{max} in each reinforcement layer is now determined based on the vertical spacing S_V, and horizontal spacing S_H as

$$T_{Max} = \sigma_H \, S_H \, S_V \tag{16}$$

Which leads to the maximum stress, P_{max} , in each reinforcement layer

$$P_{Max} = \frac{T_{Max}}{b\,t} \tag{17}$$

Where b and t are width and height of elements. Yielding factor of safety may now be determined as

$$FS_Y = \frac{F_Y}{P_{Max}} \tag{18}$$

The total length of reinforcement, L, required for internal stability is then determined from

$$L = L_e + L_a \tag{19}$$

Where L_e is the embedded length of the element, and L_a is obtained from Eqs. 20 and 21 for simple structures.

For walls with inextensible reinforcement from the base up to $H/2$

$$L_a = 0.6(H - z) \tag{20}$$

For the upper half of a wall with inextensible reinforcements

$$L_a = 0.3H \tag{21}$$

And then

$$FS_{PO} = \frac{2\,z\,\gamma_R\,L_e\,F^*\,b}{T_{Max}} \tag{22}$$

F^* obtain by interpolation from 2.0 at the top of the wall to $tan\ \varphi$ at depth=6 m [1].

4. SIMULATION-OPTIMIZATION MODEL

Optimum design of Mechanically Stabilized Earth walls reinforced with metal strips is carried out. Standard design procedures developed by Federal Highway Administration (FHWA) have been adopted.

A MSEW simulator and an efficient optimization algorithm are needed to develop a simulation- optimization model for optimum design of MSE walls. This paper develops a MSE walls simulator module based on the well-known method developed by US Department of Transportation, Federal High Way Administration, (FHWA, 2001). It calculates the factors of safety corresponding to sliding, overturning and bearing capacity for external stability and two factors of safety against reinforcement strength and pullout for internal stability analysis. The simulator module is linked with a GA algorithm especially developed for this purpose, forming an optimal MSE wall design model (GA-MSEW).

The simulator module receives trial solutions with reinforcements lengths, width, thicknesses, and spacings as decision variables to find the internal and external safety factors. The trial solutions with insufficient safety factor (nonfeasible solutions) are penaized to reduce their fitness values in the selection process. Penalized and nonpenalized solutions are transferred to the GA optimization module for generation of new trial solutions. The new solutions are checked against the required safety factors in the simulator. This procedure continues till the minimum cost feasible solution is identified.

In its general form, searching for an optimal solution, GA search for an optimal design that will be composed of (1) encoding the design variables; (2) generation of an initial population of trial solutions; (3) computation of trial solutions cost; (4) MSEW analysis simulation of the trial solutions; (5) determination of total penalized cost; (6) computation of fitness index; and (7) generation of new population via selection, crossover, and mutation operation. Steps 3–7 are then repeated until convergence is achieved.

The total cost of a reinforced earth structure may consist of; (1) reinforcement, (2) reinforced wall fill, (3) installation, (4) facing units, (5) leveling pad, and (6) engineering and testing costs. In most of design cases the last four items remain almost unchanged and may be treated as constant parameters. Therefore, without loss of generality, they may be removed from the objective function which is to be minimized. The first two items in total cost vary from one design to another as decision variables change. In this model, the part of the total cost which accounts for the first two items (i.e., the reinforcement and fill costs) is referred to as Design Specific Cost (DSC). Therefore the model attempts to minimize the DSC subject to physical, structural, and other specified constraints. All constraints are handled by penalizing the infeasible solutions.

The mathematical presentation of the optimization model may be summarized as follows

$$Min\ Cost = \left(c_1\ n\ L\ b\ t\ \frac{\gamma_{steel}}{S_H} \right) + \left(c_2\ \gamma_R\ H\ L\ \frac{1000}{g} \right) \tag{23}$$

Where, c_1 and c_2 are cost factors for reinforcements and wall fill and n is number of reinforcement layers.

Subject to

$$FS_{SL,min} \leq FS_{SL} \tag{24}$$

$$FS_{BC,min} \leq FS_{BC} \tag{25}$$

$$FS_{OT,min} \leq FS_{OT} \tag{26}$$

$$FS_{Y,min} \leq FS_Y \tag{27}$$

$$FS_{PO,min} \leq FS_{PO} \tag{28}$$

$$L_{min} \leq L \leq L_{max} \tag{29}$$

$$S_{H,min} \leq S_H \leq S_{H,max} \tag{30}$$

$$S_{V,min} \leq S_V \leq S_{V,max} \tag{31}$$

$$b_{min} \leq b \leq b_{max} \tag{32}$$

$$t_{min} \leq t \leq t_{max} \tag{33}$$

4.1 Input Parameters

The input parameters for the optional design procedures are presented in Table 1.

Table 1. Input parameters and notations for optimum design model

Parameter	**Symbol**	**Value**
Wall height	H	6.3, 7.8, 9.3, 10.8m
Angle of internal friction of the reinforced soil	φ_R	34 degree
Unit weight of the reinforced soil	γ_R	18.8 kN/m^3
Angle of internal friction of the backfill	φ_B	30 degree
Unit weight of the backfill	γ_B	18.8 kN/m^3
Angle of internal friction of the foundation	φ_F	30 degree
Allowable bearing capacity of the foundation soil	γ_F	300 kPa
Ultimate strength of metal strips	F_Y	413700 kN/m^2
Minimum factor of safety against sliding	FS_{SL}	1.5
Minimum factor of safety against bearing capacity	FS_{BC}	2.0
Minimum factor of safety against overturning	FS_{OT}	2.0
Minimum factor of safety against reinforcement strength	FS_Y	1.8
Minimum factor of safety against reinforcement pullout	FS_{PO}	1.5
Reinforcement strips cost factor	c_1	\$1/kg
Wall fill cost factor	c_2	\$3/1000kg

4.2 Model Application

Based on the proposed mathematical presentation a computer program is developed in FORTRAN and solutions are obtained using an iterative technique and optimal search method.

To examine the performance of the proposed model, few case examples are used. The first example is directly taken from FHWA (2001) which is solved to illustrate the standard method proposed by FHWA and widely practiced. Other test examples are specially designed to verify the performance of the proposed approach. All other case examples are solved using the proposed standard method of FHWA. The results are then compared with those of the proposed simulation-optimization procedure. The specification of the first test example is presented as follows:

4.3 First Example Problems (adapted from FHWA, [1])

Assumption:

The total design height of the wall = 7.8 meter with respect to gutter grade.
Traffic surcharge = 9.4kN/m^2.
Seismic coefficient = 0.05g, therefore no seismic design required.
Effective angle of internal friction of the reinforced soil = 34 degree.
Unit weight of the reinforced soil = 18.8 kN/m^3.
Effective angle of internal friction of the backfill = 30 degree.
Unit weight of the backfill = 18.8 kN/m^3.
Effective angle of internal friction of the foundation soil = 30 degree.
Allowable bearing capacity of foundation soil = 300 kPa
Cost of the wall fill = $3/1000 kg.
Cost of the steel = $1/kg.
Design life of structure = 75 year.

The assumptions equally hold for the other test examples. The solution to the first problem (H=7.8m) is identified using the proposed model. Total population of 40, maximum generation of 200, cross over and mutation probabilities of 0.8 and 0.1 are used to address a good near optimal solution. The rate of convergence of the solutions toward the final solution for 10 test runs is presented in Figure 2. The specifications of the best solution are presented in Table 2. The same table compares the results of the standard method of FHWA and those of the proposed model. The results show that the model may save up to 9 percent for 7.8 meter high wall on design specific cost as defined earlier.

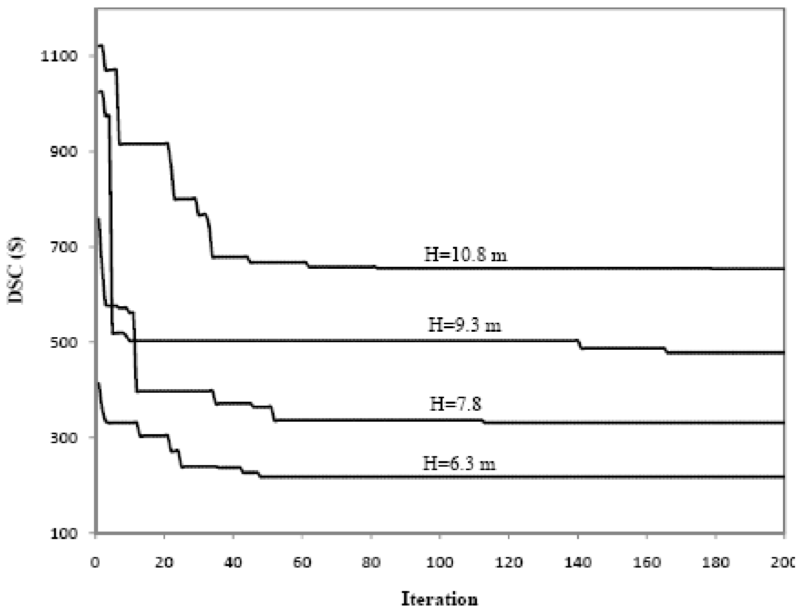

Figure 2. Rate of convergence of the solutions

To further verify the performance of the proposed method, 3 more cases are considered. For all cases the assumptions of the first case hold. Results for the same assumptions and loading and wall heights ranging from 6.3 meter to 10.8 meter are presented in Tables 3-5. For all cases the design parameters are also derived from the standard method of design proposed at FHWA (2001). Cost saving increase for higher walls and decreases for shorter walls. For the cases under consideration the model saves up to 15 and 10 percent in DSC for 10.8 and 6.3 meter high walls, respectively. With the stated population and generation numbers, all tests runs ended up with solutions better than those resulted of the FHWA standard design method [1].

It is interesting to note that the design parameters for the standard method of FHWA remain unchanged regardless of cost variations on reinforcement strips and/or wall fill cost factors. In another words, the results of FHWA design procedure is insensible to the cost factors. In real world problems, however, the designer accounts for the cost variations of the design elements.

Table 2. The design variables and design specific cost obtained from conventional FHWA design method and optimum design method (GA-MSEW) for H=6.3 m

Design Method	H (m)	Design Variables					DSC ($)
		L (m)	S_H (m)	S_V (m)	b (m)	t (m)	
FHWA	6.3	4.50	0.43	0.75	0.03	0.004	242
GA-MSEW	6.3	3.73	0.20	1.59	0.047	0.003	218

Table 3. The design variables and design specific cost obtained from conventional FHWA design method and optimum design method (GA-MSEW) for H=7.8 m

Design Method	H (m)	Design Variables					DSC ($)
		L (m)	S_H (m)	S_V (m)	b (m)	t (m)	
FHWA	7.8	5.50	0.75	0.75	0.05	0.004	362
GA-MSEW	7.8	4.60	0.45	1.12	0.074	0.003	331

Table 4. The design variables and design specific cost obtained from conventional FHWA design method and optimum design method (GA-MSEW) for H=9.3m

Design Method	H (m)	Design Variables					DSC ($)
		L (m)	S_H (m)	S_V (m)	b (m)	t (m)	
FHWA	9.3	6.50	0.75	0.75	0.06	0.004	544
GA-MSEW	9.3	5.47	0.37	1.34	0.057	0.004	478

Table 5. The design variables and design specific cost obtained from conventional FHWA design method and optimum design method (GA-MSEW) for H=10.8m

Design Method	**H (m)**	**Design Variables**					**DSC ($)**
		L (m)	**S_H (m)**	**S_V (m)**	**b (m)**	**t (m)**	
FHWA	10.8	7.60	1.50	0.75	0.09	0.006	773
GA-MSEW	10.8	6.33	1.02	0.54	0.067	0.004	654

A sensitivity analysis was conducted to determine how the effective angle of internal friction of the reinforced soil, φ, and unit weight of the reinforced soil, γ, affect the optimum design variables and design specific cost. All sensitivity tests are conducted for the 7.8 meter high case example with the same parameters. Effective angle of internal friction of the reinforced soil and unit weight of the reinforced soil were systematically varied through the ranges recommended by Chalermyanont and Benson (2005). For a fixed γ, Figure 3 shows how positive and negative changes in φ modify the final design and the associated DSC. As expected, for a fixed unit weight, an increase in the angle of the internal friction of the reinforced soil decreases the reinforcement and results in overall cost reduction. Conversely, for a fixed φ, in soils with larger unit weights, reinforcement cost is larger compared to the lighter soils. In another words, in soils with the same effective angle of internal friction, mechanical stabilization of the soils with larger unit weights are more expensive than those of smaller ones. Partial results of the conducted sensitivity tests are presented in Figure 4.

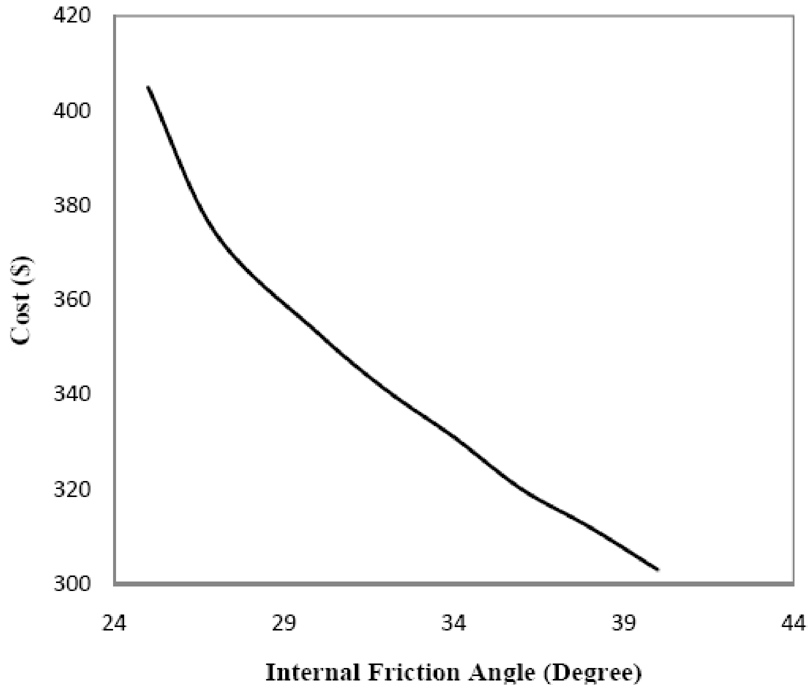

Figure 3. Sensitivity analysis of internal friction angle for given specific unit weight

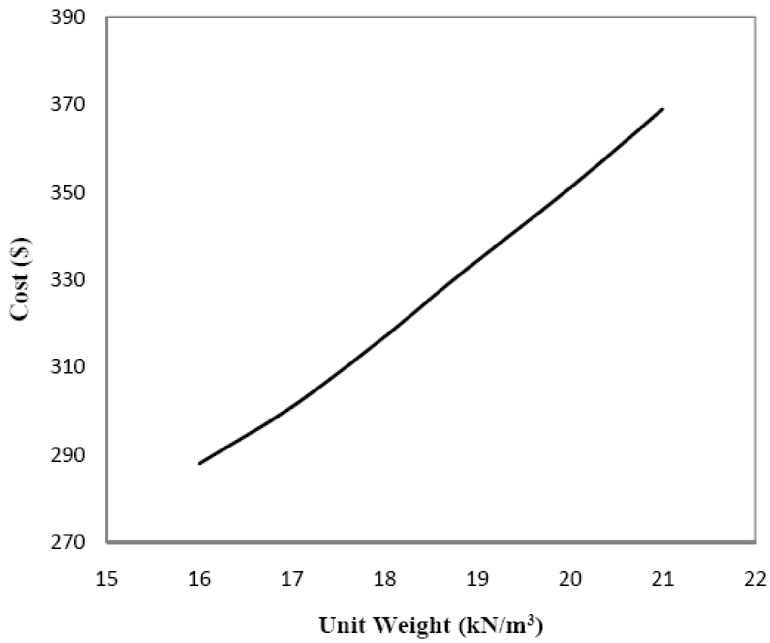

Figure 4. Sensitivity analysis of unit weight for given internal friction angle

5. CONCLUSIONS

MSE walls offer significant technical advantages and cost saving over common reinforced concrete retaining structures at sites with poor foundation conditions. A simulation-optimization model was presented for optimum design of MSE walls. Considering few case examples with different heights, it was observed that design specific cost may be reduced up to 15 percent, following the proposed optimum design procedure over conventional design approach. Cost saving is more pronounced for higher walls compared to shorter ones. In soils with the same effective angle of internal friction, mechanical stabilization of the soils with larger unit weights are more expensive than those of smaller ones. It was shown that the coupled simulation-optimization approach is quite effective in addressing the most desirable design considering with different safety factors as constraint to the model.

REFERENCES

1. Elias V, Christopher B. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, National Highway Institute, Department of Transportation FHWA, Washington, D.C., USA, 2001.
2. Collin JG. Earth wall design, PhD thesis, University of California, Berkeley, USA, 1986.
3. Bonaparte R, Holtz RD, Giroud JP. Soil reinforcement design using geotextiles and geogrids. In: Fluent JE Jr (ed) Geotextile testing and the design engineer, STP 952, ASTM, Philadelphia, 1987, pp. 69–116.
4. Schneider HR, Holtz RD. Design of slopes reinforced with geotextiles and geogrids, *Geotextiles and Geomembranes*, **3**(1986) 29–51.
5. Leshchinsky D, Perry EB. A design procedure for geotextile-reinforced walls, In: Geosynthetics’87, Industrial Fabrics Association International, St. Paul, Minn, (1987) 95–107.
6. Schmertmann GR, Chouery-Curtis VE, Johnson RD. Design charts for geogrid-reinforced soil slopes. In: Geosynthetics’87, Industrial Fabrics Association International, St. Paul, Minn, 1987, pp. 108–120.
7. Ehrlich M, Mitchell JK. Working stress design method for reinforced soil walls, *Geotechnical Engineering*, **120**(1994) 624–45.
8. Zornberg JG, Sitar N, Mitchell JK. Limit equilibrium as basis for design of geosynthetic reinforced slopes, *Geotechnical and Geoenvironmental Engineeirng*, **124**(1998) 684–98.
9. Jie Han, Leshchinsky D. General analytical framework for design of flexible reinforced earth structures, *Geotechnical and Geoenvironmental Engineering*, **130**(2006) 1427–35.
10. Xue JF, Gavin K. Simultaneous determination of critical slip surface and reliability index for slopes, *Geotechnical and Geoenvironmental Engineering*, **133**(2007) 878–86.
11. Basudhar PK, Vashistha A, Deb k, Dey A. Cost optimization of reinforced earth walls, *Geotechnical and Geological Engineering*, **26**(2007) 1–12.
12. Chalermyanont T, Benson CH. Reliability-based design for external stability of mechanically stabilized earth walls, *Geotechnical and Geoenvironmental Engineering*,

5(2005) 196–205.

13. Chalermyanont T, Benson CH. Reliability-based design for internal stability of mechanically stabilized earth walls, *Geotechnical and Geoenvironmental Engineering*, **130**(2004) 163–73.
14. Goldberg DE. *Genetic Algorithm in Search, Optimization and Machine Learning*, Addison-Wiley, New York, US, 1989.
15. Mayerhof G. Some recent research on the bearing capacity of foundations, *Canadian Geotechnical Journal*, **1**(1963) 16–26.