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Computerized design method for geosynthetic-reinforced soil retaining walls for railway embankments

K. Horii & H. Kishida
Chuo Kaihatsu Corporation, Tokyo, Japan

M. Tateyama
Railway Technical Research Institute, Tokyo, Japan

F. Tatsuoka
Institute of Industrial Science, University of Tokyo, Japan

Abstract : This paper describes the design method and computer program for the geosynthetic-reinforced soil retaining wall system (GRS-RW system) having a cast-in-place rigid facing. This system has been used extensively to reconstruct railway embankments in Japan. The design method is limit-equilibrium-based and encompasses internal stability, external stability and local failure of facing. The analysis of external stability against circular slip failure uses the simplified slice method, while the analysis of internal stability against the sliding and overturning of wall uses the two-wedge method, taking into account the effect of facing rigidity. The structural analysis of facing assumes the facing as an elastic beam supported by the geosynthetic reinforcement layers working as elastic springs. These analyses procedures have been coded in a newly developed program. Given the data for configuration of embankment, ground condition, soil properties, the design parameters of facing, the spacing, length and mechanical properties of reinforcement, and load conditions, the factor of safety can be computed on a personal computer. Some working examples of this design method are given.

1. INTRODUCTION

In large cities of Japan, the demand for good transportation access between downtown and suburban areas is increasing rapidly. Accordingly, the extension of many existing railway systems is being planned to relieve traffic congestion due to limited transportation capacity. However, the high price of land, especially near the centers of the big cities, is a serious constraint to land acquisition for such railway expansion. As a result, closer attention is being given to the potential use of the slope portion of existing embankments. Most existing embankments possess a gentle slope, on average 1 : 2 in vertical : horizontal, thereby providing potential new area above the slope for development, if this can be accomplished in a cost-effective manner. Therefore, application of the steepest possible embankment method, reconstructing a gentle slope to a steep or near-vertical wall, is required in order to utilize the unused space available above the slope of existing embankment.

The geosynthetic-reinforced soil retaining wall (GRS-RW) system, which is a sort of hybrid wall system of mechanically reinforced earth wall with a cast-in-place full-height rigid facing, has been developed by the Railway Technical Research Institute (RTRI) in cooperation with Institute of Industrial Science, University of Tokyo (Tatsuoka et al., 1989, Murata et al., 1990, Tatsuoka, 1993). The advantages of this method include:

1) Wall deformation is very small due to the use of full-height continuous rigid facing.

2) An existing gentle slope of embankment can be reconstructed with no or minimal slope excavation for placing reinforcements, since the GRS-RW system utilizes a relatively short-length geosynthetic reinforcement. This is achieved because the shape of reinforcement is planar and the vertical spacing between reinforcement layers is a relatively small (i.e., 30 cm).

3) Neither heavy construction machinery nor propping of the full-height facing during wall construction are necessary, thereby reducing the required space during construction as well as limiting the adverse effect of construction activities on street traffic.

4) A cast-in-place concrete facing serves to protect the geosynthetics from the effect of ultraviolet rays, mechanical damage actions and fire hazard while

serving also for aesthetic appeal.

Since its development, the use of GRS retaining walls has increased rapidly for railway embankments. Recently, use for highway embankment is being planned. Demand has therefore increased for GRS retaining walls for varying types of site conditions, including a wide variety of ground conditions and embankment types.

A computer program has been developed to respond to this demand as the GRS retaining wall cannot be designed in a timely manner by manual computations or, effectively, by design chart under such varying conditions (Tateyama et al., 1991; CKC,1992). This paper describes the design method for GRS retaining walls and the computer program. Further, the application of this design method is illustrated through a design example computed by the program.

2. DESIGN METHOD

2.1 Design approach

Fig. 1 shows a schematic flow chart for the design procedure of the GRS-RW system. Given data for the configuration of embankment, ground conditions, backfill soil properties and external load conditions, assuming the spacing, length and mechanical properties of geosynthetic reinforcement, the external stability and internal stability of a given GRS retaining wall, and local stability of facing can be assessed (Table 1). The external stability is evaluated by the conventional simplified slice method or so called Fellenius method. The internal stability is evaluated by the two-wedge method taking into account the effect of facing rigidity. Local stability of facing is analyzed by the elastic beam structural analysis method. Using these methods, safety factors against circular slip failure, overturning, sliding and bending collapse of facing, respectively, are computed. Further, when needed, the stability of the supporting ground, and the settlement of ground and its effect, including the liquefaction potential of ground due to earthquake, are evaluated. This design method, as coded, is based on the most recently revised version of the Japan Railway Structure Standard (RTRI, 1992; Ministry of Transport, 1992).

The design computations are performed using a personal computer. The program yields the factor of safety against several types of failure mode. The factors of safety for external and internal stability as computed are then compared with the required

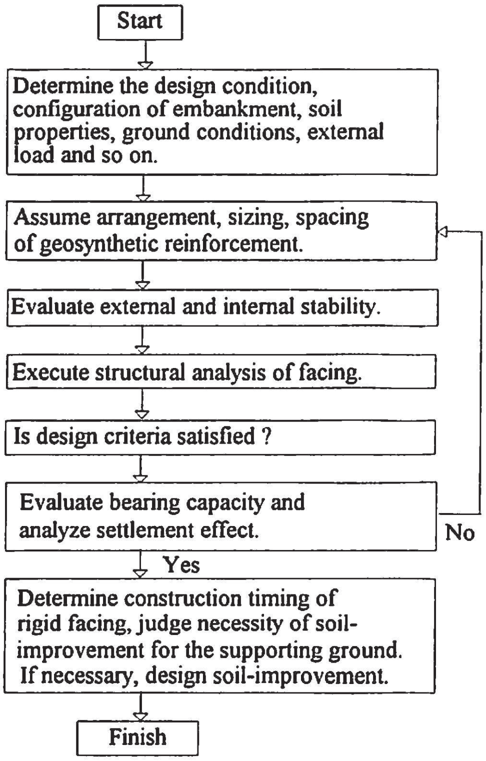


Fig. 1 Design procedure of the GRS retaining wall system.

Table 1 Stability analysis for the GRS Retaining Wall system.

Stability	Failure Mode	Method of Analysis
External Stability	Circular Slip Failure	Simplified Slice Method
Internal Stability	Overturning and Sliding Failure	Two-Wedge Method
Local Failure	Bending Collapse of Facing	Elastic Beam Structural Analysis

Table 2 Required factor of safety corresponding to the different loading conditions.

Stability	Loading State	Required Factor of Safety
External Stability	Dead Load	1.4
	Live Load	1.4
	Earthquake Load	1.1
Internal Stability	Dead Load	2.0
	Live Load	1.5
	Earthquake Load	1.25

factors of safety, which correspond to the different loading states listed in Table 2. If the computed factor of safety is insufficient, the design factors for geosynthetics are modified, and the above-mentioned computation procedure is repeated in order to achieve the appropriate design. Several trial computations are generally necessary in order to identify the optimum design.

2.2 General Description of GRS Retaining Wall system

Fig. 2 shows the cross-section of a typical GRS retaining wall with a rigid facing. Geosynthetic reinforcement is embedded in each soil layer commonly 30 cm thick. The length of reinforcement is stipulated at more than 35 % of wall height or a minimum length of 1.5 m. This specification assures the safe stability of the fill embankment under construction before casting in place a concrete rigid facing. The longer geosynthetic reinforcement sheet is laid at a vertical interval of 1.5 m, in order to facilitate quality control of soil compaction. The range of length of reinforcement is usually up to the line corresponding to the angle of soil repose. This construction procedure consists of the following:

1) The embedded portion of the small footing (or leveling pad) for the facing is excavated, and reinforced concrete footing (or leveling pad) is cast-in-place.
2) The first geosynthetic reinforcement is laid on the ground.
3) Sandbags (or sand gabions) are placed at the shoulder of each soil layer to confine the soil adjacent to the wall face.
4) The geosynthetic sheet is wrapped around the sandbag, and overlapped.
5) The earth fill materials are placed on the geosynthetic sheet and compacted.
6) The above construction procedure is repeated until the required height of wall is achieved.
7) Steel reinforcement is constructed in front of the wrapped-around wall.
8) Concrete forms are constructed and concrete is cast in place. The GRS retaining wall is thus completed.

There are many kinds of geosynthetics suitable for reinforced soil. However, the geosynthetic reinforcement used in a GRS retaining wall must have sufficient tensile strength. The tensile strength is determined by laboratory testing under specified loading condition. The design tensile strength of geosynthetic reinforcement is determined

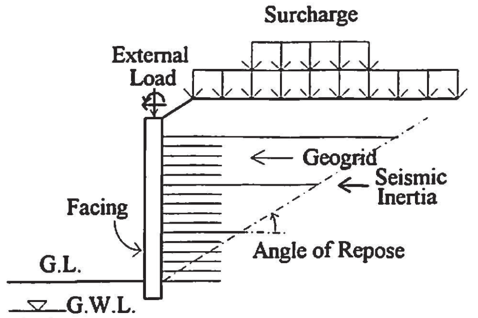


Fig. 2 Typical section of GRS retaining wall.

considering several factors of reduction due to alkaline deterioration (if needed), mechanical damage during construction, creep effect, impact or instantaneous loading, and cyclic loading.

Alkaline deterioration is considered due to contact with concrete materials. This deterioration is negligible for Vinylon, which is widely used for actual GRS retaining walls. The reduction factor α_1 is defined by the ratio of tensile strength after submersion in pH12 alkaline solution for a period of 700 days to the standard tensile strength. The standard tensile strength is determined by uniaxial test with a strain rate of 5 % per minute. If the maximum strain tested is more than 15 %, then the standard tensile strength is taken as the value of tensile force at a 15% strain.

Mechanical damage is considered as breakage or tearing of geosynthetic reinforcement due to compaction in gravely soil. The reduction factor α_2 in the case of geogrid is defined by the ratio of the number of intact threads of geogrid after the specified condition of spreading and compaction in gravely soil to the total number of threads tested.

Creep reduction is considered as the strength reduction of reinforcement subjected to long term loading. The reduction factor α_3 is defined by the ratio of the maximum tensile force (at which creep rate does not exceed 3.5×10^{-5} per hour in a period of 500 hours, beginning 24 hours after the start loading) to the standard tensile strength.

Reduction due to impact loading is considered as caused by earthquake or instantaneous loading. The reduction factor α_4 is defined by the ratio of the impact loading strength to the standard tensile strength. The impact loading strength is determined by the maximum instantaneous loading force which does not produce breakage or strain in excess of 15 % under loading conditions of 3 repetitions with the frequency less than 1 Hz, beginning 24 hours after loading at magnitude of 30 % of the standard tensile strength.

The reduction due to cyclic loading is considered due to repeated train loads above the wall. The cyclic reduction factor α_5 is defined by the ratio of the cyclic loading strength to the standard tensile strength. The cyclic loading strength is determined by the tensile strength after cyclic loading of 1.5 million times with a frequency 20 Hz at an amplitude of 1.0 kgf/cm² (98 kPa) above a sustained stress 1.0 kgf/cm².

The material commonly used for geosynthetic reinforcement is geogrid, made of Vinylon coated with poly-vinyl carbonate. The configuration for geosynthetics (geogrid) is netting with an aperture of 2 cm. Under laboratory testing of geosynthetics conducted by RTRI, the above reduction factors are given as follows: α_1= 0.98, α_2= 0.93, α_3 =0.7, α_4= 0.9, α_5= 0.8.

There are several types of loads; i.e. the tramway and train load on the wall, the external load of the noise barrier fence installed on the top of the rigid facing, and seismic load. Design is checked for the loading conditions according to the Railway Structures Standard approved by Ministry of Transport (1992). The loading conditions consist of dead load state, live load state, and earthquake load state. Dead load state is considered self weight. Live load state is considered train load and wind load superimposed on self weight. Earthquake load state is considered inertia force due to earthquake superimposed on self weight.

In design computations, the reduction factors described previously are combined in accordance with the loading state. In other words, for dead load state $\alpha_j = \alpha_1 \bullet \alpha_2 \bullet \alpha_3 \approx 0.6$; for live load state $\alpha_i = \alpha_1 \bullet \alpha_2 \bullet \alpha_5 \approx 0.7$; and for earthquake load state $\alpha_e = \alpha_1 \bullet \alpha_2 \bullet \alpha_4 \approx 0.8$, respectively. Thus, the design tensile strength is given as follows:

$$T_a = \alpha T_k \tag{1}$$

where, T_a = design tensile strength, T_k = standard tensile strength of geosynthetic reinforcement measured at $\varepsilon \leq 15$ %, and α = reduction factor corresponding to the loading state.

Furthermore, the pullout resistance force of the geosynthetic reinforcement embedded in soil is defined as follows;

$$T_p = 2\,(c + \sigma \bullet \tan\phi)\,L/F_f \tag{2}$$

where T_p = design pullout resistance force, c = cohesion of soil, ϕ = internal friction angle of soil, σ = over-burden pressure, L = length of reinforcement for resisting pullout (anchorage length), and F_f = safety factor against pullout (for dead load state F_f = 2.0; for live load state F_f = 1.5; for earthquake load state F_f = 1.25).

Finally, the design resistance force is defined by the following equation;

$$T = \min\{T_a, T_p\} \tag{3}$$

where T = design resisting force, T_a = design tensile strength, and T_p = pullout resistance force. This design resisting force is used as the available resistance for the stability analysis described later.

In this method, basically the entire geosynthetic strength is used for the analysis of the ultimate failure condition regards of elevation. This is an approximation method on the unsafe side. This factor is, however, covered by a degree of conservatism in several steps of analysis (the use of reduced value of ϕ and the nature of Eqs. (1), (2), (4), (7) and (8)).

2.3 External Stability Analysis

A GRS retaining wall should be stable against any slip failure which may or may not pass through the reinforcement. The typical failure mode is assumed to be a circular slip failure. The computation method for slip circle failure utilizes the so called modified Fellenius method. The Fellenius method is known to be more conservative as the slope becomes steeper in slope stability analysis. Its main advantage is the simplicity of its application (i.e., simple formulation, no trial and error, etc.). In this method, the Fellenius method has been employed as a safe side one, with while most engineers are familiar.

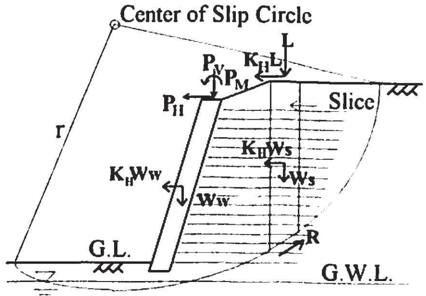


Fig. 3 Schematic diagram of the simplified slice method.

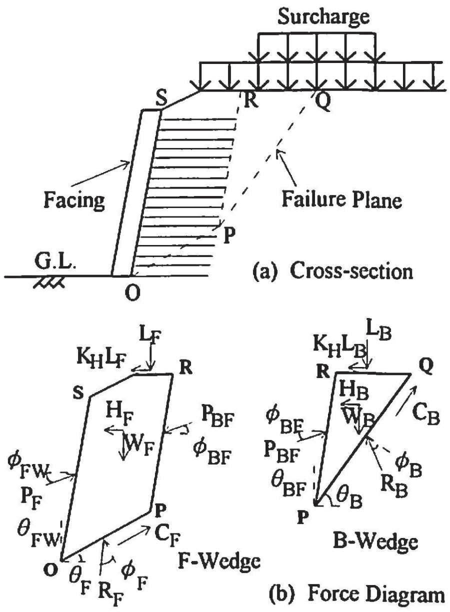


Fig. 4 Two-wedge analysis for GRS-RW system

The effect of earthquake force is represented by the static horizontal force equal to the weight of the wedge of soil multiplied by a seismic coefficient. A schematic diagram is shown in **Fig. 3**.

The factor of safety against the circular slip failure is defined by the following equation:

$$F_s = \left\{ \frac{\Sigma M_{rg} + \Sigma M_{rs}}{\Sigma M_{ds} + \Sigma M_{dw} + \Sigma M_{df}} \right\}_{min} \qquad (4)$$

where M_{rs} = resisting moment of soil at the slice base determined by the Coulomb's failure criterion, M_{rg} = resisting moment of geosynthetics pullout resistance force, M_{ds} = driving moment of soil slice weight and the horizontal force due to earthquake, M_{dw} = driving moment of weight of facing, and M_{df} = driving moment of the external load on the top of facing. Σ signifies the summation for all slices.

The reinforcement effect of geosynthetics (M_{rg}) is comprised of two factors. One is the resistance against sliding which is the parallel component to the slip surface produced by the tensile force of geosynthetics. The other factor is the incremental frictional resistance of soil due to the increase in the normal stress caused by the component perpendicular to the slip surface of tensile force of geosynthetics.

The procedure for stability computation against base failure including toe failure is as follows:

1. The center of the slip circle is set.
2. The radius of slip circle is set, extending beyond the embedded depth of facing so as not to pass through the rigid facing (i.e., part of the effect of rigid facing).
3. The soil mass is divided into several vertical slices.
4. The driving and resisting moments for each slice are computed.
5. The sum of all the driving and resisting moments for all slices is obtained.
6. The factor of safety is computed by Eq. 4.
7. The above procedure is repeated varying the radius and location of slip circles until the lowest factor of safety for critical circular slip failure is identified.

2.4 Internal Stability Analysis

The stability of the reinforced zone can be evaluated by the following two methods:

a) The conventional method based on the two-wedge method (Tatsuoka, 1993) to evaluate the stability of the reinforced soil mass (together with a facing for a completed wall), against direct sliding failure and overturning.

b) The simplified method also based on the two-wedge method to evaluate the stability of the rigid facing against sliding and overturning.

In both methods, the location of the resultant force R_F along the base of the active zone (**Fig. 4**) should be assumed since it is statically indeterministic. The practical design method described in this paper follows the second method. In this method, the location of P_F is made deterministic by assuming the distribution pattern of earth pressure on the back face of facing (**Fig. 5**). The results of the two methods are compared in the later section; they shows comparable results for ordinary configurations of retaining walls constructed by the GRS-RW system.

For the second method, it is assumed that the facing and reinforcement region in backfill behaves as a unified body. The resultant force against the facing is computed by means of force equilibrium equation for the backfill, which is also necessary for both the stability computation against overturning and sliding of facing, and the structural analysis of facing.

The two-wedge failure mode is shown in **Fig. 4**.
The front wedge includes the region of reinforcement domain "OPRS". The back wedge is the triangle region of backfill indicated as "PQR"

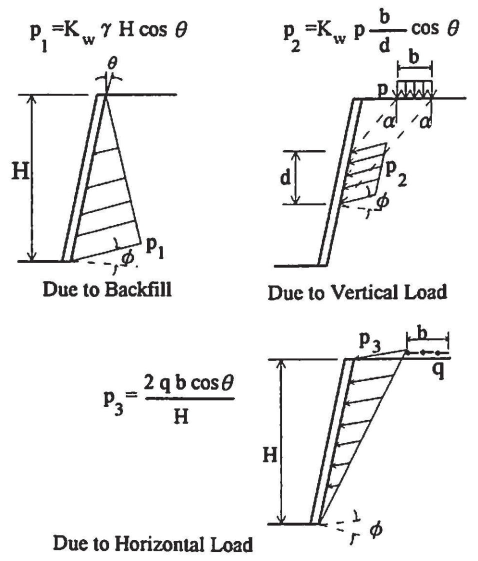


Fig. 5 Earth pressure distribution due to backfill and surcharge on wall.

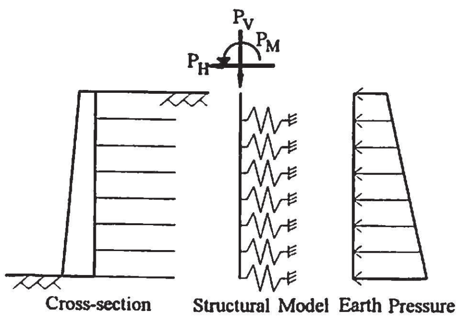


Fig. 6 Structural model of rigid facing with earth pressure.

region. A limit equilibrium state is assumed for these wedges. The unknown forces to be evaluated are given on the basis of this limit equilibrium state by the following equations:

$$P_{BF}=\frac{\{(H_B + K_H L_B)\cos(\theta_B - \phi_B) + (W_B + L_B)\sin(\theta_B - \phi_B) - c_B\cos(\phi_B)\}}{\cos(\phi_B + \phi_{BF} - \theta_B - \theta_{BF})} \tag{5a}$$

$$R_B = \frac{- (H_B + K_H L_B) + P_{BF}\cos(\phi_{BF} - \theta_{BF}) + c_B\cos(\theta_B)\}}{\sin(\theta_B - \phi_B)} \tag{5b}$$

$$P_F=\frac{\{(H_F + K_H L_F)\cos(\phi_F - \theta_F) - (W_F + L_F)\sin(\phi_F - \theta_F) + P_{BF}\cos(\phi_F + \phi_{BF} - \theta_F - \theta_{BF}) - c_F\cos(\phi_F)\}}{\cos(\phi_{FW} + \phi_F - \theta_{FW} - \theta_F)} \tag{5c}$$

$$R_F=\frac{\{(H_F + K_H L_F) + P_{BF}\cos(\phi_{BF} - \theta_{BF}) - P_F\cos(\phi_{FW} - \theta_{FW}) - c_F\cos(\theta_F)\}}{\sin(\phi_F - \theta_F)} \tag{5d}$$

where W = weight of the wedge, H = horizontal force due to earthquake ($H = K_H \bullet W$), K_H = seismic coefficient, L = surcharge load, R = reacting force acting on the failure surface of wedge, P = resultant force between two wedges and acting force on facing, c = cohesion of soil, ϕ = internal angle of friction of soil, and θ = inclination angle of failure surface. The subscript symbols B and F signify the back wedge and front wedge, respectively. The subscript symbol W signifies the rigid facing.

2.5 Distribution of Earth Pressure on the Back Face of Facing

The distribution of earth pressure acting on the facing must be known in order to compute the overturning moment of the wall and structural analysis discussed later. Therefore, it is assumed that the earth pressure produced by the backfill and surcharge on the crest is obtained as shown in **Fig.** 5; namely,

1) The earth pressure due to the weight of backfill acts on the facing as a triangularly distributed load.

2) The earth pressure due to vertical surcharge load **p** acts as a load which is distributed from assumed planes with an inclination angle α of 40 degrees relative to the vertical.

3) The earth pressure due to horizontal surcharge load **q** on the crest of wall acts as an inverted triangular distribution.

The resultant force acting on the facing computed by the two-wedge method must be equal to the integral of the above distributed earth pressure plus the horizontal outward external load on the crest; namely,

$$P_F = K_w(\gamma H^2 / 2 + pb) + qb \tag{6a}$$

A coefficient of active reinforced earth pressure K_w is determined so as to satisfy the above condition, which is given as:

$$K_w = \frac{P_F - qb}{\gamma H^2/2 + pb} \quad (6b)$$

where P_F = resultant force due to earth pressure, γ = unit weight of soil, p = vertical component of surcharge, q = horizontal component of surcharge (= $K_H p$), K_H = seismic coefficient for p, H = height of wall, and b = surcharge load width.

The factor of safety against overturning of the rigid facing is defined by the following equation:

$$F_s = \left\{ \frac{M_{rw} + M_{rf} + M_{rs} + M_{rb} + M_{rg}}{M_{ow} + M_{of} + M_{os}} \right\}_{min} \quad (7)$$

where M_{rw} = resisting moment of self weight of facing, M_{rf} = resisting moment of external load on the top of facing, M_{rb} = resisting moment due to the earth pressure acting on the boundary between front and back wedges parallel to the axis of facing, M_{rs} = resisting moment of the component parallel to the facing of the difference between the earth pressure acting on the back face of facing and that acting on the boundary between front and back wedges (this component is assumed to act on the back face of facing), M_{rg} = resisting moment of geosynthetic tensile forces, M_{ow} = overturning moment due to the inertia of self weight of facing, M_{of} = overturning moment due to external load acting on the top of facing, and M_{os} = overturning moment of the normal component of earth pressure acting on the back face of facing.

The reaction on the facing base is not considered, which is equivalent to the assumption that the reaction is located at the facing toe as the moment is calculated about the toe of facing. In this computation, the effect of the embedding of facing is not taken into account. It is reasonable to consider that the facing and backfill behave more-or-less as a monolith. Accordingly, the overturning failure of facing as predicted based on Eq. (7) without the term M_{rb} does not mean the failure of the entire wall. Therefore, so that Eq. (7) can analyze the stability of the entire wall, which increases as the reinforcement length increases, the term M_{rb} has been included in Eq. (7). As shown later, without the term M_{rb}, the stability of wall becomes smaller.

The factor of safety against sliding of the facing is defined as follows:

$$F_s = \left\{ \frac{F_{rg} + F_{rb}}{F_{ds} + F_{dw} + F_{df}} \right\}_{min} \quad (8)$$

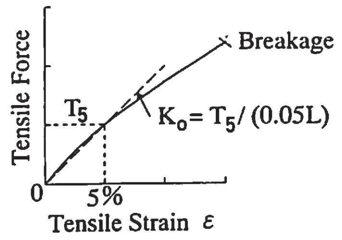


Fig. 7 Method used in the current design method to obtain the "spring constant" of reinforcement.

where F_{rg} = horizontal resistance of geosynthetic reinforcement, F_{rb} = shearing resistance at the bottom of facing. F_{ds} = horizontal component of the earth pressure acting on the back face of facing, F_{dw} = equivalent horizontal force of facing weight multiplied by the seismic coefficient, and F_{df} = horizontal component of external load on the top of facing.

The procedure of computing internal stability is as follows (see **Fig. 4**) :

1. Location "P" is set along an arbitrary chosen line parallel to the back face of facing (n.b., usually the back face of the main reinforced zone becomes to the critical failure plane).
2. The failure surface "PQ" of the back wedge is assumed.
3. The internal force P_{BF} is computed by Eq. (5a).
4. The maximum value of $P_{BF,max}$ is determined varying the inclination of failure surface "PQ" (the angle θ_B), through steps 1~3.
5. The resultant force P_F of earth pressure acting on the back face of facing is computed using $P_{BF,max}$ by Eq. (5c).
6. The factors of safety against overturning and sliding are computed by Eqs. (7) and (8), respectively.
7. By varying the location of point P, the above computational procedure is repeated to obtain the minimum factors of safety against overturning and sliding, respectively, and to identify the critical two-wedge failure surfaces. The smaller value of the safety factors against overturning and sliding controls the actual failure of wall.

2.6 Structural Analysis

The facing must be strong enough against earth pressure acting on the back face of facing and the external forces acting on the top of facing such as those from noise barrier structures, fencing or electric poles on the top of facing. The structural analysis of facing is done by assuming that the facing acts as an elastic beam supported by the geosynthetic reinforcement layers considered as a series of elastic springs as illustrated in **Fig. 6**.

Here, the tensile force in geosynthetic reinforcement occurs only in case the facing displaces outward. Conversely, in case the facing displaces inward, only compressive backfill reaction occurs because the geosynthetic reinforcement is not effective in compression. Thus, the equation which governs an elastic beam supported by a series of elastic springs is as follows:

$$EI \cdot y'''' + K_g \cdot y = p \tag{9}$$

where EI = flexibility of facing, y = out-of-plane deflection of beam, y'''' = the fourth derivative of deflection y, K_g = modulus of subgrade lateral reaction of backfill, and p = distributed earth pressure (either that derived from overturning computation or sliding computation, whichever is the larger value).

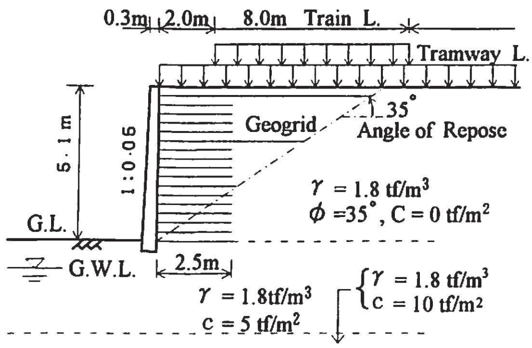


Fig. 8 Wall cross-section of design example.

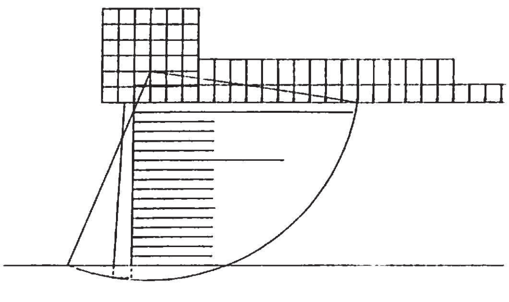


Fig. 9 Critical slip circle (Live load state).

The tensile force in geosynthetic reinforcement produced by the displacement of facing is defined by the following equation.

$$T = K_s \cdot y \tag{10}$$

where T = tensile force of geosynthetic reinforcement, K_s = spring constant of geosynthetic reinforcement, and y = deflection of beam (i.e., facing). The spring constant K_s for geosynthetic reinforcement can be estimated by the following equation;

$$K_s = \alpha_s \frac{T_5}{(0.05 \cdot L)} \tag{11}$$

where α_s = modified factor for soil confining effects, which is 1.0, T_5 = tensile force at a 5 % strain of geosynthetics (see **Fig. 7**), and L = length of geosynthetics. The value of α_s suggested above is a very conservative value. Therefore, the lateral displacement of facing computed by using this value will be largely over-estimated. This point will be discussed in detail later.

The external forces acting on the facing consist of self weight, inertia of earth pressure and loads on the top of facing. All these forces are considered in the structural analysis. Computation for structural analysis utilizes the finite element method; i.e., displacement method. The iterative method is used for this computation, because the stiffness matrix is changed in accordance with the direction of facing displacement, inward or outward. This computation procedure is as follows:

1. The facing is subdivided into finite elements.
2. The element stiffness matrix and load vector are generated, and assembled into total stiffness matrix and total load vector, respectively.
3. The spring stiffness of geosynthetic reinforcement is added to the total stiffness matrix in the case of facing displaced outward; otherwise, the spring stiffness matrix of backfill reaction is added to the total stiffness matrix (in case the facing displaces inward). The spring stiffness of the backfill reaction usually employed in the current design is 1.0 kgf/cm³.
4. The externally applied concentrated loads are added to the total load vector.
5. The algebraic equations generated in the above are solved for unknown displacements and rotation of facing.
6. The convergence of displacement is judged, and if the error of convergence is not less than the allowable value, steps 2 to 5 are repeated.

7. The section force and stresses are computed for each element.

3. FUNCTIONS OF COMPUTER PROGRAM

The design conditions for GRS retaining walls encountered in the field include different complicated configurations of embankment, multi-layered soil deposits, different loads with varied magnitude, etc. Accordingly, a computer program written in Quick-Basic (Microsoft), which can address designs for most commonly used types of railway embankments, was developed. More than 10 km of GRS retaining walls have been designed using this computer program since 1990.

In this paper, the type of geosynthetic reinforcement discussed is geosynthetic sheet reinforcement only. The program also deals with the design of GRS-RW bridge abutment which supports directly a bridge girder on the crest. Also, in the case of railway construction it is necessary to include various utility structures such as electrical pole, noise barrier structures, etc. In the case of a light structure with relatively small load, the structure may be constructed on the top of facing itself. However, in the case of relatively larger structural load, the structure is attached by means of rib to the facing to increase facing rigidity. In the case of very large loads, the foundation of the structure is embedded in the reinforced zone behind the face.

4. DESIGN EXAMPLE

Fig. 8 shows a cross-section of a railway retaining wall as a design example. The height of the wall is 5.1 m, the soil is a cohesionless soil (i.e., sand) with an angle of internal friction of 35 degrees and a unit weight of 1.8 tf/m^3. The supporting ground of the wall is comprised of two clayey layers. Soil properties of subsoil in the upper layer is: cohesion = 5 tf/m^2 and unit weight = 1.8 tf/m^3, and for lower subsoil: cohesion = 10 tf/m^2 and unit weight = 1.8 tf/m^3. The ground water table exists at 1.0 m deep below the ground surface. The facing is steel-reinforced concrete with a top width of facing of 30 cm, an inclination of the front face of 1 (V) : 0.05 (H), a vertical back face of facing, and an embedded depth of 40 cm.

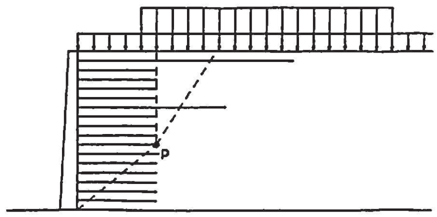
On the top of facing, a noise barrier fence is constructed, which applies to the facing horizontal load of 0.9 tf/m caused by wind loading and momentum load of 1.35 tf-m/m. This wind load is considered only for the live load state. The seismic coefficient adapted is 0.2, for the use in external and internal stability analyses. Tramway load = 1.0 tf/m^2 and train load = 2.5 tf/m^2 on the embankment are considered. The geosynthetic reinforcement used is a geogrid having a standard tensile strength equal to 3 tf/m and a length of 2.5 m. Some longer geogrid sheets are spread at a vertical spacing of 1.5 m. These contribute largely to increase the resistance of wall against overturning under earthquake loading condition.

Fig. 9 shows the critical slip circle in the case of live load state for an external stability analysis computed by this program. The critical slip circle found is located outside of the reinforced region. The values of the safety factor against this circular slip failure as computed are listed in Table 3, which shows that this design satisfies the required safety factors, and are therefore judged to be stable for circular slip failure.

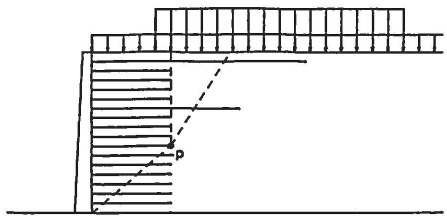
Fig. 10 shows the critical failure planes in the case

Table 3 Factor of safety for circular slip failure.

	Fs computed	Fs required
Live Load	1.523	1.4
Earthquake Load	1.329	1.1



(a) Overturning



(b) Sliding

Fig. 10 Critical failure surfaces by two-wedge method (Live load state).

of live load state with wind load acting outward. There is no difference in the location of failure planes for overturning and sliding. However, in the earthquake load state, the critical failure planes are located deeper than that for the live load state. The safety factor values as computed are listed in Table 4. The comparison with the required factors shows that this design assures internal stability.

Fig. 11 shows the distribution diagrams in the case of live load state for the deflection and rotation of the facing, the tensile force of reinforcement (geogrid) at the connection to the back face of facing, and the bending moment and shearing forces in the facing. The quantity in each diagram is normalized by dividing by its maximum value, which is shown above each diagram. It is seen from this figure that the deflection of facing varies with depth linearly. This means that the facing behaves as a rigid facing. The maximum deflection of facing shown is 4.2 cm, but this does not represent the actual deflection; i.e., in all the past cases, the observed deformation of the facing is substantially less than the computed deformation. This is because the spring constants at the connections between reinforcement and facing are grossly underestimated so that conservative (i.e., over-estimated) stress values in the facing are obtained. It is also because in the course of the deflection computation the external load which is applied to the facing includes all loads in estimating the stresses in the facing. In reality, as the facing is cast-in-place after the full wall height is constructed, the earth pressure from the backfill soil, acting on the gabions, is transferred to the back face of facing, however the deflection due to the same does not occur.

Table 5 shows the results of the structural analysis. It is seen that, despite that the earth pressure on the back face of facing is not largely reduced by soil reinforcement, the maximum values of tensile force, bending moment and shearing force are substantially lower than those for conventional RC cantilever retaining walls. It is to be noted that this is because the facing is supported at many levels by reinforcement layers.

5. DISCUSSION

5.1 Stability analysis for overturning

Since the GRS-RW system allows the use of

Table 4 Factor of safety for overturning & sliding.

	Overturning	Sliding	Required
Dead Load	4.000	3.815	2.0
Live Load	2.358	2.896	1.5
Earthquake Load	2.136	1.897	1.25

Table 5 Result of structural analysis.

Loading State	Max. Tensile Force (tf)	Max. Bending Moment (tf-m)	Max. shearing Force (tf)
Dead Load	0.475	0.219	0.458
Live Load	0.494 0.699	-1.836 1.957	-0.990 1.150
Earthquake Load	0.798	0.323	0.798

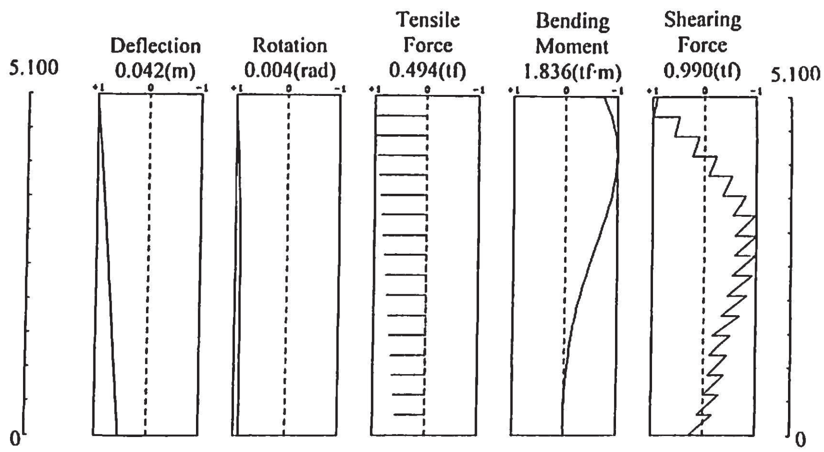
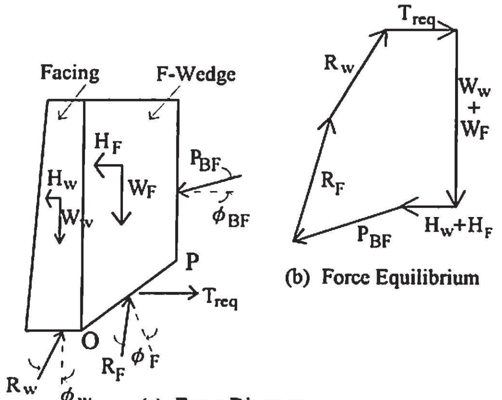
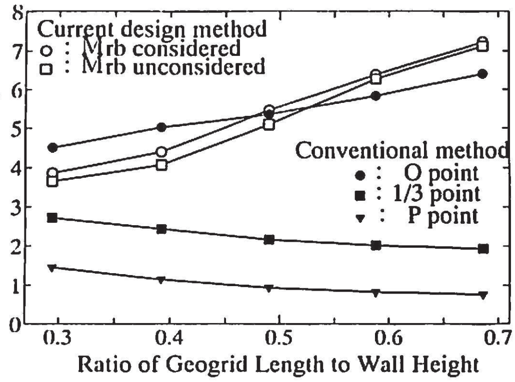


Fig. 11 Section force distribution diagram (Live load state).

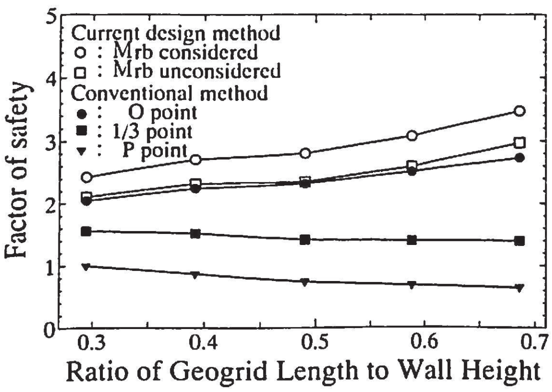


(a) Force Diagram (b) Force Equilibrium

Fig. 12 Two-wedge stability for overturning of the reinforced zone (conventional two-wedge method).



(a) Dead load state



(b) Earthquake load state

Fig. 13 Safety factor against overturning (considering the thickness and weight of facing with longer geosynthetic layers).

relatively short reinforcement, the stability analysis for overturning is important. In the following, the simplified method described above will be compared with the conventional two-wedge analysis (**Fig. 12**).

The factor of safety against the overturning of the front wedge F is defined by the following equation:

$$F_s = \left\{ \frac{M_{rw}+M_{rf}+M_{rg}}{M_{ow}+M_{of}+M_{or}+M_{os}} \right\}_{min} \tag{12}$$

where M_{rw} = resisting moment of the total self weight of facing and the wedge F, M_{rf} = resisting moment of external load on the top of facing, M_{rg} = resisting moment of geosynthetic tensile forces activated at the lower and back boundaries of the front wedge F, M_{ow} = overturning moment due to the inertia of the total self weight of facing and wedge F, M_{of} = overturning moment due to external load acting on the top of facing and the crest of the front wedge F, M_{or} = overturning moment due to the reaction R_F on the lower boundary of wedge F, and M_{os} = overturning moment of earth pressure P_{BF} (normal plus tangential components) acting on the boundary between the front and back wedges F and B. In this computation, the effect of the embedding of facing is not taken into account.

The safety factors were obtained for a completed wall with a rigid facing calculating the moment around the toe of facing. As to the location of the reaction R_F along the bottom boundary of the front wedge F, which controls the value of M_{or}, the following three assumptions were made (**Fig. 12**):

a) the toe of the reinforced zone of backfill (the point O),

b) at the one third point from the point O on the lower boundary of the front wedge F, and

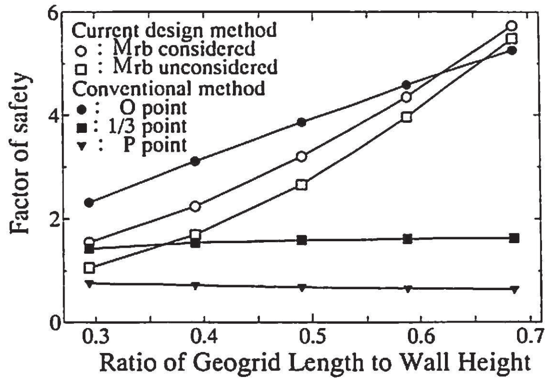
c) the back end of the lower boundary of the front wedge F (the point P) (very conservative assumption).

For the GRS-RW system with a rigid facing and a densely reinforced zone, probably the assumption a) may be reasonable. The location of the reaction R_w along the bottom of facing was fixed at the facing toe; this method is equivalent to that used in the current design method.

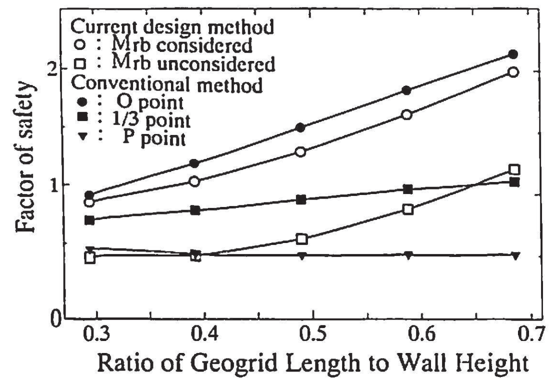
The computation based on Eq. (12) was made for the configurations shown in **Fig. 8**, except for removing the surcharge on the wall crest and external load on the top of facing, while varying the length of reinforcement using a safety factor against pull-out of reinforcement equal to 1.0.

The following two cases were analyzed:

(1) The longer geosynthetic layers exist as shown in **Fig. 8**. Results are shown in **Figs. 13 and 15**.



(a) Dead load state



(b) Earthquake load state

Fig. 14 Safety factor against overturning (considering the thickness and weight of facing without longer geosynthetic layers).

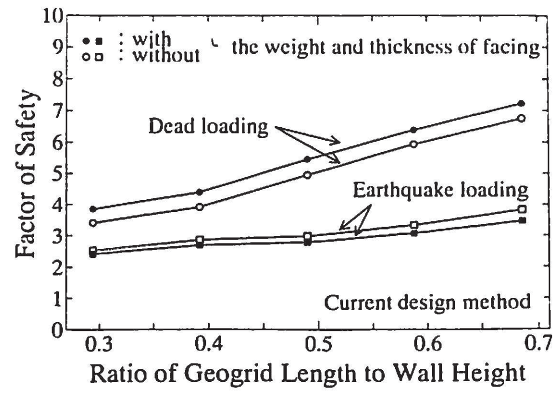


Fig. 15 Comparison of safety factor against overturning for the presence of facing (with longer geosynthetic layers).

(2) The longer geosynthetic layers have been removed. This case was to examine their effect on the stability of the wall. Results are shown in **Figs. 14** and **16**.

The safety factors against overturning were also obtained by the simplified method (Eq. 7). Similarly, the safety factors by the current design method were obtained without the term M_{rb} (= resisting moment of the component parallel to the axis of facing of the earth pressure acting on the boundary between the front and back wedges).

Results are shown in **Fig. 13**(a) and **Fig. 14**(a) for dead load state and in **Fig. 13**(b) and **Fig. 14**(b) for earthquake load state, respectively. The following may be inferred from the figures:

1) For the conventional two-wedge method, the effect of the location of reaction point on the lower boundary of the front wedge **F** is very large. The results obtained when the reaction is located at the 1/3 point from the point **O** on the lower boundary of the wedge **F** and the point **P** (the back end of the lower boundary of the front wedge **F**) decreases as the geogrid length increases. This runs counter to the experimental results, and therefore these assumptions are not acceptable. Furthermore, the result with the assumption of the reaction point (the point **P**) may be too conservative. On the other hand, the result when the reaction is located at the toe of wall (the point **O**) is more reasonable.

2) The safety factor by the simplified method without the term M_{rb} are consistently smaller than those with the term M_{rb}. The difference is particular large in Fig. 14(b) (the most critical condition). This result indicates that the design based on this method is too conservative (and actually is not used in the current practice).

3) The safety factor by the simplified method with the term M_{rb} is similar to that by the conventional two-wedge method assuming the reaction R_F at the toe of the front wedge **F** (point **O**). This result indicates that the currently used design method (i.e., the simplified method with the term M_{rb}) is reasonable.

In the following, the effects of the weight and thickness of rigid facing are studied by the current design method with the term of M_{rb}. Note that the facing without weight and thickness has the overall rigidity as that with weight and thickness, and the center for rotation is located at the toe of the reinforced zone (point **O**).

Fig. 14 shows the comparison of safety factor against overturning with and without the weight and thickness of facing for the case with the longer geogrid layers as shown in **Fig. 8**. **Fig. 16** shows the similar result for the case without the longer geogrid layers. It is seen that in both **Figs. 15** and **16**, the safety factor with the weight and thickness of facing is consistently larger than that without facing under dead load state. However. in case of earthquake load state, the differences becomes either small or even the reverse result is obtained (**Fig. 15**). The reason for this is because the increase in the resisting moment due to the self weight of facing and the increase in the rotation arm by the bottom width of facing is set off by the increase in the overturning moment due to the seismic inertia of facing.

5.2 Spring constants for structural analysis of facing

In the current design method, the tensile resistance of reinforcement at the connection to the back face of facing (i.e., the spring constants used in the structural analysis of facing) is defined as the lateral outward displacement of the facing multiplied by the tensile rigidity of reinforcement. In such case, the interaction between the adjacent soil and reinforcement is ignored; namely, the tensile rigidity used is the value K_0 obtained at 5 % elongation strain by the tension test while fixing the total length of reinforcement at both ends (**Fig. 7**). This method provides substantially conservative (underestimated) values as shown below.

As described in Tateyama et al. (1993, this volume), outward lateral loading tests were performed at the top of the facing of the full-scale GRS retaining wall (**Fig. 17**). The spring constants K_c were back-calculated based on the current design method described in section 2.6 using these results against the load factor (the ratio of the lateral load T to the ultimate value T_f). In this calculation, a non-linear spring which was constant with depth was assumed. The result is shown in **Fig. 18**. **Fig. 19** shows the ratio of the back-calculated spring constants K_c to the design value K_0 (= 120 kgf/cm).

It may be seen from **Figs. 18** and **19** that the current design method uses substantially under-estimated spring constants (which provide substantially large stresses in the facing). These results indicate that the facing can be made much simpler than the currently used one without loosing the stability and rigidity of GRS retaining wall.

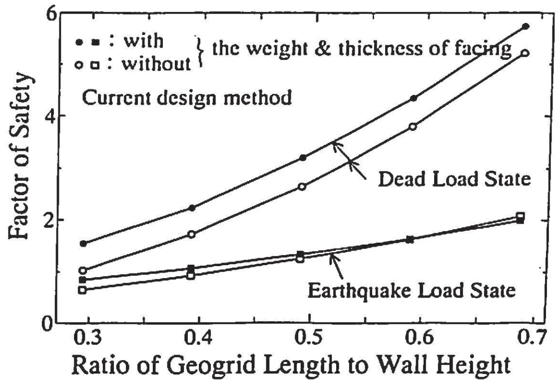


Fig. 16 Comparison of safety factor against overturning for the presence of facing (without longer geosynthetic layers).

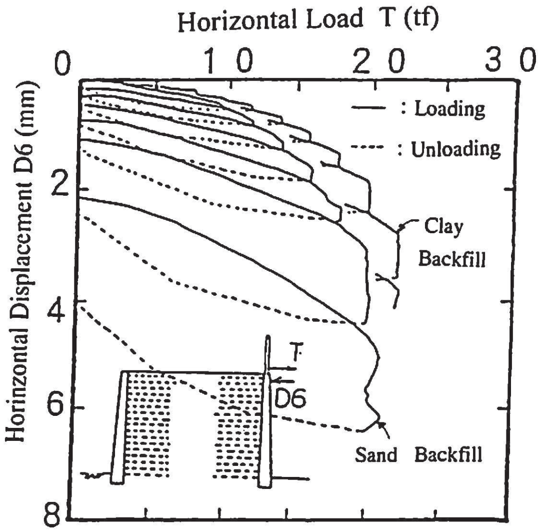


Fig. 17 Summary of the result of outward lateral loading at the top of the facing of full-scale GRS retaining wall (Tateyama et al., 1993, this volume).

6. CONCLUSIONS

This paper describes the design method of the GRS retaining wall with a rigid facing, the computer program based on this design method, the function of the computer program, a design example conducted by this program and some discussion on the design method. The computer program is applicable to complex ground formations and wall configurations, while the design by the program is both economical and practical.

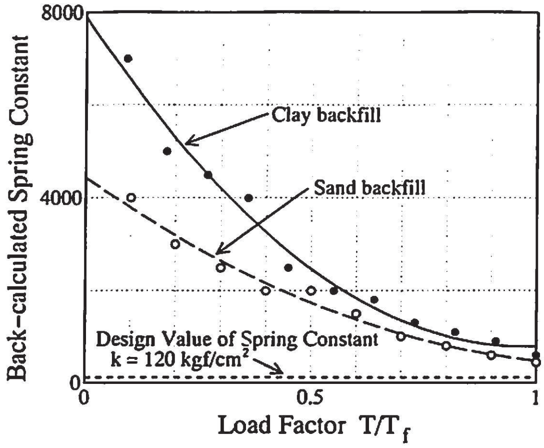


Fig. 18 Back-calculated spring constants from the result of outward lateral loading at the top of the facing of the full-scale GRS retaining wall.

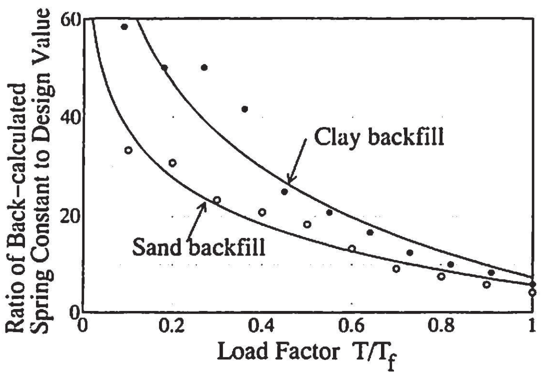


Fig. 19 Ratio of the back-calculated spring constants K_c to the design value K_0 (= 120 kgf/cm).

This computer program can be applied to retaining wall structures for highway as well as railway, which include those used as a bridge abutment and GRS retaining walls holding a pier foundation embedded in the backfill or at the top of or the outer face of facing.

However, an issue still to be addressed is the need to incorporate into the program a function for automatic search of critical failure surface and auto dimensioning and arrangement of the reinforcement.

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