New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-45775-0

Seismic design of geosynthetic reinforced soils for railway structures in Japan

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ABSTRACT: Focusing on seismic design of geosynthetic reinforced soils, several features of a newly-revised design standard for railway earth structures in Japan are reported. Three ranks of seismic performance against level 1 and 2 earthquakes are assigned. Prescriptive measures are admitted, where use of primary and/or secondary geosynthetic reinforcements is mandated for embankments. Recommendation of verification procedures is made, where Newmark sliding block analysis is adopted against the level 2 earthquake load. No creep reduction is considered in setting the tensile strength of geosynthetic reinforcements against earthquake loads.

1 INTRODUCTION

1.1 *Background*

Recently, Japan suffered from several large earthquakes which caused extensive damage to earth structures. In some of them, reinforced soils performed very well (e.g., Tatsuoka et al., 1997 among others), and thus they were considered more frequently for the replacement of conventional earth structures in reconstruction works (Koseki et al., 2006 and Shinoda et al., 2007 among others).

Since the 1995 Hyogoken-Nanbu (Kobe) earthquake, in particular, the level of the design seismic load has been raised significantly, while introducing the concept of so-called level 2 earthquake load, which is defined as the maximum possible level over the design life of civil engineering structures (JSCE 2006). Meanwhile, the principle of performance-based design has been introduced as well. Under such circumstances, several design guidelines for new construction works of civil engineering structures in Japan have been revised or are under revision process, and reinforced soils are not the exceptions.

1.2 *Scope of the report*

In view of the above, it is attempted in this report to describe briefly the features of the design standard for railway earth structures (RTRI 2007) that has just been revised, focusing on the seismic design of geosynthetic reinforced soils (hereafter called as GRS).

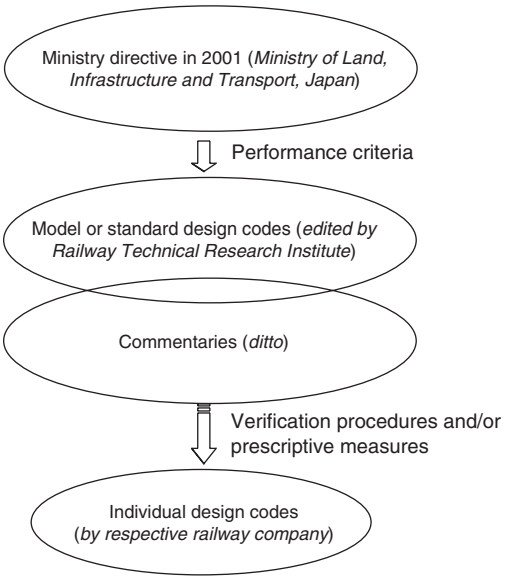

Figure 1. Evolution of design standards of railway structures in Japan (modified from Koseki et al., 2006).

As shown schematically in Figure 1 (Koseki et al., 2006), the revision was made following a directive by the Ministry of Land, Infrastructure and Transport in Japan issued in 2001. This directive mandated that structures should be designed based on performance criteria. It should be noted that the design standards for

Table 1. Composition of the design standard for railway earth structures and its commentaries (RTRI 2007).

Chapters	Pages
1. General	30
2. Design principles	32
3. Embankment	90
4. Cut and natural slopes	36
5. Base course	40
6. Subgrade	21
7. Reinforced soil (general)	28
8. Reinforced soil wall	17
9. Reinforced soil abutment	17
10. Reinforced cut slope	24
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foundations and retaining walls and for seismic design principles are currently under revision. Refer to Koseki et al. (2006) for the details of the relevant revision history of the whole design standards of railway structures in Japan.

As also shown in Figure 1, the design standard is accompanied by commentaries. Based on these, individual design codes will be implemented by railway companies in Japan.

2 DESIGN STANDARD OF RAILWAY EARTH STRUCTURES IN JAPAN

2.1 *Composition of the standard*

The table of contents of the newly revised design standard for railway earth structures and its commentaries (RTRI 2007) is listed in Table 1.

As one of the standard construction methods, soil reinforcing techniques including GRS are adopted in chapters 7 through 10, covering a subtotal volume of 86 pages that is about a quarter of the total volume excluding the appendix. The reinforced soil wall (chapter 8) and the reinforced soil abutment (chapter 9) deal with GRS retaining walls with a full-height rigid facing. The reinforced cut slope (chapter 10) deals with retaining walls to support cut slopes that are reinforced with nailing, micropiling or doweling.

In addition, in chapter 3 on embankment, use of secondary geosynthetic reinforcements is standardized. In this report, therefore, design of embankments will be also described in sections 2.2 and 2.4.

2.2 *Design flow of embankments*

Figure 2 illustrates the flow chart of the performance-based design of embankments that is specified in the design standard (RTRI 2007). After setting up the required performance in terms of safety, serviceability and repairability, decision is made whether prescriptive measures are adopted or not.

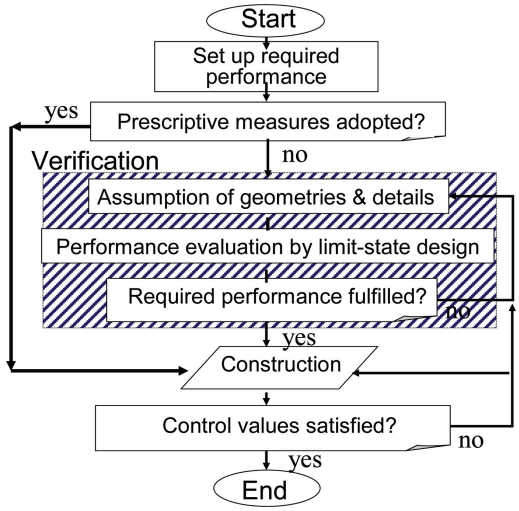

Figure 2. Flow of performance-based design of embankments.

The prescriptive measures have three different levels. Each of the prescriptive measures has been verified in advance to fulfill the corresponding performance rank, and thus no additional verification is required at the design stage.

On the other hand, if the prescriptive measures are not adopted, one needs to proceed to the verification process, including assumption of the geometries and structural details, performance evaluation based on limit-state design and confirmation of the required performance. In case the required performance is not fulfilled through the verification, one should modify the assumption and repeat the same procedures.

It should be noted that, in designing structures other than embankments, prescriptive measures are not available, and thus the verification process shall be implemented.

2.3 *Required seismic performance*

Table 2 summarizes the required performances of railway earth structures against two levels of design earthquake loads. In this table T_{des} is the design life of the structure. In general, the design life of a railway earth structure is assumed as 100 years.

For a level 1 earthquake load that is highly expected over the design life, it is required that all the earth structures will maintain their design functions without requiring repair work, i.e. will not exhibit excessive displacements (Performance rank III).

Against a level 2 earthquake load, which is defined previously in section 1.2, it is required that important earth structures can be restored to design function conditions with minimal repair (Performance rank II),

Table 2. Performance requirements for railway earth structures in Japan (modified from Koseki et al., 2006).

Action (design earthquake loads)	Level 1 (highly expected for T_{des})	Level 2 (maximum possible for T_{des})
Important structures	Performance I: Will maintain their expected functions without repair works (no excessive displacements)	Performance II: Can restore their functions with quick repair works
Others		Performance III: Will not undergo overall instability

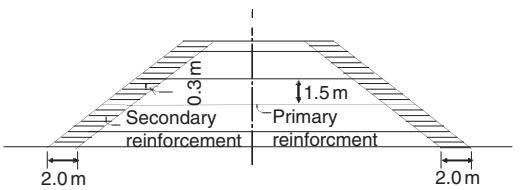

Figure 3. Cross-section of 6m-high embankment with performance rank I specified as prescriptive measure.

Table 3. Configurations of 6m-high embankment with different performance ranks specified as prescriptive measures.

	Rank I	Rank II	Rank III
Slope	1:1.8	1:1.5 (upper) 1:1.8 (lower)	1:1.5
Primary reinforcement	Yes	Basically no	No
Secondary reinforcement	Yes	Yes	Yes

while the other earth structures will not undergo overall instability (Performance rank I).

2.4 *Prescriptive measures for embankments*

When the prescriptive measures are adopted for embankments, different configurations of reinforcement arrangement as well as the slope geometry are employed depending on the performance rank.

For example, for a 6 m-high embankment, the primary reinforcements are used with rank I, as shown in Figure 3 and Table 3. They are placed for the full width of the embankment at a vertical spacing of 1.5 m. With ranks II and III, on the other hand, only the secondary reinforcements are used to protect the slope for the width of 2.0 m and to enhance the specified height (=0.3 m) of fill lift during construction. The design tensile strength of the primary and secondary reinforcements shall be 30 and 2 kN/m, respectively.

For a 6 m-high embankment, the slope shall have no berm, while its angle is varied depending on the performance rank as listed in Table 3. It should be noted that the type of the fill material and the required degree of compaction to be secured during construction are also varied depending on the performance rank.

2.5 *Design tensile strength of geosynthetics reinforcement*

The design strength of geosynthetics reinforcements are determined based on the following procedures.

The design strength T_d is assigned by applying a material factor γ_g to the characteristic strength T_a (Eq. 1), which is obtained as a product of the derived strength T_k and the modification factor ρ_m (Eq. 2).

$$T_d = \gamma_g \times T_a \tag{1}$$

$$T_a = \rho_m \times T_k \tag{2}$$

The derived strength T_k is evaluated by Eq.3 using the average value T_{AVE} and the standard deviation σ_x of the measured strengths, where the factor a is set equal to 0.67.

$$T_k = T_{AVE} - a \times \sigma_x \tag{3}$$

The material factor γ_g is given as a product of several reduction factors α_i (Eq. 4), while the modification factor ρ_m is set equal to unity against accidental actions such as the level 2 earthquake load and to 0.8 otherwise.

$$\gamma_g = \prod \alpha_i \tag{4}$$

Table 4 summarizes the combination of reduction factors α_i in evaluating the design tensile strength of reinforcements under five different situations. Reductions are made considering the combined effects of alkalis (α_1), damage during construction (α_2), creep load (α_3), impact load (α_4) and cyclic load (α_5). It should be noted that the design strengths against earthquake loads as well as train load can be assigned without considering the possible effects of creep reduction. This is consistent with recent findings on the load-strain-time behavior of geosynthetic reinforcement products as reported by Greenwood et al. (2001) and Tatsuoka (2003) amongst others.

2.6 *Verification against level 1 earthquake load*

As explained in section 2.2, if the prescriptive measures are not adopted, one needs to proceed to the verification process. In case of GRS retaining walls,

Table 4. Combination of reduction factors.

Design situations (action combinations)	Reduction factors				
	α_1	α_2	α_3	α_4	α_5
Permanent	Yes	Yes	Yes	No	No
Permanent + variable (train load)	Yes	Yes	No	No	Yes
Permanent + major variable (level 1 earthquake) + minor variable	Yes	Yes	No	Yes	No
Permanent + accidental (e.g., level 2 earthquake) + minor variable	Yes	Yes	No	Yes	No
Temporary state during construction	No	Yes	No	No	No

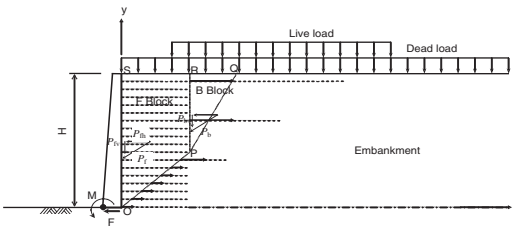

Figure 4. Modeling of GRS retaining wall for internal stability analysis.

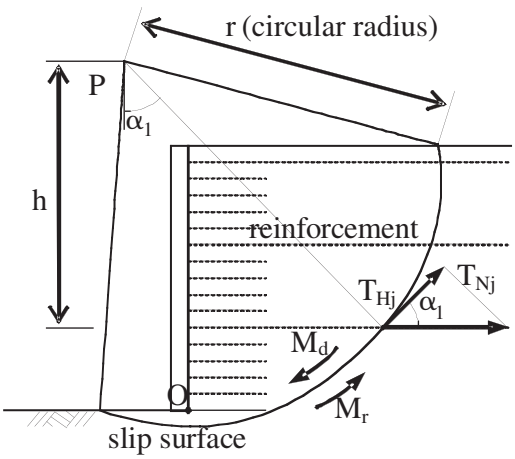

Figure 5. Modeling of GRS retaining wall for external stability analysis.

the performance requirement for level 1 earthquake load as explained in section 2.3 is verified through stability analyses using load and resistance factors against internal instability (Figure 4), external instability (Figure 5), and facing failure (Figure 6).

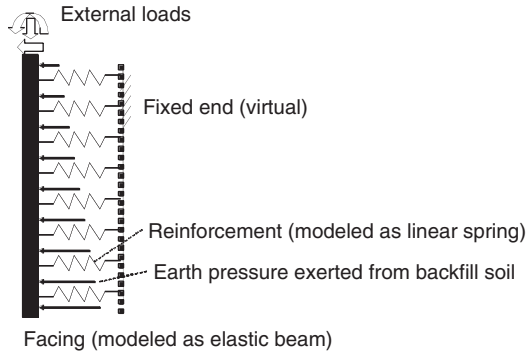

Figure 6. Modeling of facing and reinforcements.

Internal stability analysis is conducted with respect to base sliding and overturning. For example, the stability against base sliding is verified using the following equation:

$$\gamma_i \times \frac{H_{Rd}}{H_{Ld}} \leq 1.0 \tag{5}$$

where γ_i is the structure factor (set equal to unity in general); H_{Rd} is the response value of base sliding force; H_{Ld} is the limiting value of base sliding force.

The response value of base sliding force is evaluated as:

$$H_{Rd} = \gamma_H \times (P_{fh} + W_{EQ} + F_H) \tag{6}$$

where γ_H is the load factor (set equal to unity in general); P_{fh} is the horizontal component of the resultant force of earth pressure exerted from the backfill; W_{EQ} is the horizontal inertia force of facing; F_H is the external load applied to the top of the facing (e.g., due to the existence of noise barrier). As schematically shown in Figure 4, the resultant force of earth pressure is evaluated based on the two-wedge method.

The limiting value of base sliding force is evaluated as:

$$H_{Ld} = f_{ri} \times (\Sigma T_i + W_{BS} + W_{hp}) \tag{7}$$

where f_{ri} is the resistance factor (set equal to 0.80 against level 1 earthquake load); T_i is the design tensile resistance of reinforcement; W_{BS} is the design shear resistance mobilized at the bottom of facing; W_{hp} is the design horizontal resistance mobilized at the embedded part of the facing. The design tensile resistance of reinforcement is evaluated as the smaller value between the design tensile strength of reinforcement T_d that has been explained in section 2.5 and

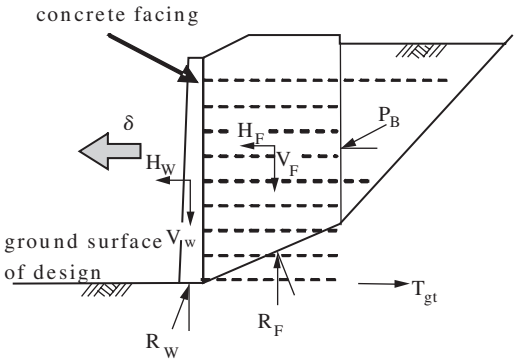

Figure 7. Modeling of GRS retaining wall for evaluation of base sliding.

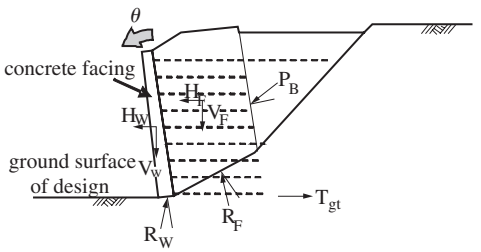

Figure 8. Modeling of GRS retaining wall for evaluation of overturning.

the pull-out resistance of the reinforcement T_p. that is evaluated as:

$$T_p = f_{rg} \times (\sigma_{vi} \times \tan\phi \times 2l_i + c \times 2l_i) \quad (8)$$

where f_{rg} is the resistance factor (set equal to 0.80 against level 1 earthquake load); σ_{vi} is the effective vertical stress acting on the ith reinforcement; l_i is the effective length of the ith reinforcement; ϕ and c are the internal friction angle and cohesion of the backfill soil, respectively.

2.7 *Verification against level 2 earthquake load*

For structures with performance ranks II and III, performance requirement for level 2 earthquake loads is verified in terms of their residual deformations using Newmark sliding block analyses and other numerical analyses. In case of GRS retaining walls, base sliding displacement of the retaining wall (Figure 7), overturning displacement (Figure 8), and shear deformation of the reinforced backfill (Figure 9) are evaluated.

It should be noted that, the residual shear deformation of the reinforced backfill has not been considered in many of the other existing design codes which adopt

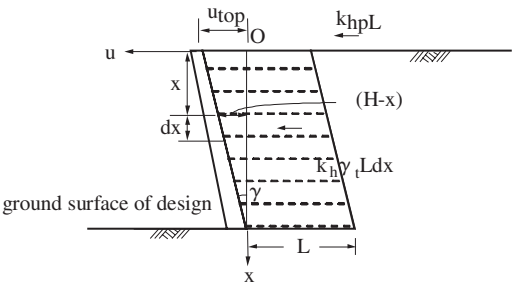

Figure 9. Modeling of GRS retaining wall for evaluation of shear deformation of reinforced backfill.

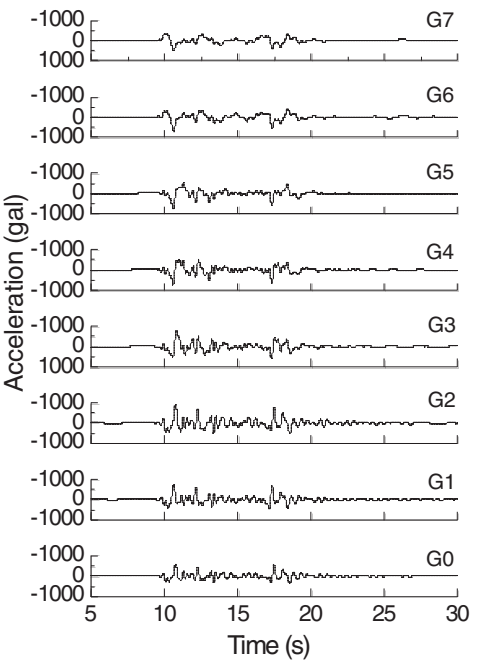

Figure 10. Time history of level 2 design earthquake motions (refer to Tables 5 and 6 for ground type classification and amplitude of maximum acceleration).

the assumption that the reinforced backfill behaves as a rigid body.

In conducting the Newmark sliding block analysis, one needs to specify the design earthquake motions. They are specified in the design standard as shown in Figure 10. They were obtained by applying a band-pass filter (0.3–4.0 Hz) to the design motions specified at the ground surface levels in the relevant design standard (RTRI, 1999). Depending on the natural period T_g of the ground, which is evaluated using Eq. 9 based on the profile of shear wave velocities, different wave forms and amplitudes are assigned as listed in Tables 5 and 6. The peak accelerations a_{max} are in the range

Table 5. Ground type classification based on natural period T_g (unit in seconds).

G0-G2	G3	G4	G5	G6	G7
Less than 0.25	0.25–0.5	0.5–0.75	0.75–1.0	1.0–1.5	More than 1.5

G0: Rock deposit; G1: firm base deposit; G2: Pleistocene deposit; G3: moderate; G4: moderate to soft; G5 and G6: soft; G7: very soft.

Table 6. Maximum acceleration of level 2 design earthquake motions (unit in gals).

G0	G1	G2	G3	G4	G5	G6	G7
578	732	924	779	−718	−741	−694	−501

Table 7. Results from Newmark sliding block analyses on 6 m-high embankments with different performance ranks.

	Rank I	Rank II		Rank III
Slope	1:1.8	1:1.5	1:1.8	1:1.5
Primary reinforcement	Yes	No	No	No
Secondary reinforcement	Yes	Yes	Yes	Yes
Residual displacement*	10.6 cm	36.0 cm	61.8 cm	96.5 cm

* Against level 2 design earthquake motion for G2 ground.

between 500 and 920 gals, and the largest value of a_{max} is assigned for the G2 ground consisting mainly of Pleistocene deposits.

$$T_g = 4 \times \sum_{i=1}^{N} \left(\frac{h_i}{V_{s_i}} \right) \tag{9}$$

where N is the total number of soil layers; h_i and V_{Si} are the thickness and the shear wave velocity of the ith layer, respectively.

For example, results from the Newmark sliding block analyses on the 6m-high embankment with different performance ranks specified as prescriptive measures (see section 2.4) are shown in Table 7. In setting the level 2 design earthquake motion, the severest ground condition (i.e., the condition of G2 ground) was assumed. It should be noted that the embankment with performance rank II was simplified into two kinds of configurations having a uniform slope angle.

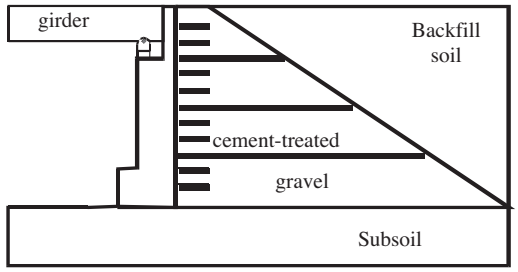

Figure 11. GRS abutment having cement-treated gravel for reinforced backfill (Aoki et al., 2005).

As a result, the embankment with performance rank I suffered from a residual displacement of about 10 cm, while those with performance rank II underwent residual displacements in the range of 40 to 60 cm. Further, the embankment with performance rank III suffered from a residual displacement of about one meter. Such different performances are expected when they are subjected to level 2 earthquake motion.

2.8 *Seismic design of reinforced soil abutment*

In chapter 9 of the design standard, design of GRS abutments having cement-treated gravel for reinforced backfill, as developed by Aoki et al. (2005) and schematically shown in Figure 11, is described.

In their seismic design, the abutment body and the reinforced backfill are verified with respect to safety against level 1 earthquake load and repairability against level 2 earthquake load.

For example, in verifying the repairability of the abutment body, a pseudo-static non-linear push-over analysis is conducted against the inertia force of the girder, while considering the tensile reaction of the reinforcements and assuming that the reinforced backfill does not exert any earth pressure to the body.

3 SUMMARY

The features of seismic design of GRS structures as specified in the newly-revised design standard for railway earth structures in Japan can be summarized as follows.

Three ranks of seismic performance against level 1 & 2 earthquakes are assigned considering the importance of the structure. The level 2 design earthquake motions have the maximum acceleration levels in the range of 500 to 920 gals.

Use of prescriptive measures is admitted in designing embankments. In setting their standard cross-section, use of primary and/or secondary geosynthetic reinforcements is mandated.

Recommendation of verification procedures is made. Against the level 2 earthquake, Newmark sliding block analysis is adopted as well as the evaluation of residual shear deformation of reinforced backfill in case of GRS retaining walls.

No creep reduction is considered in setting the tensile strength of geosynthetic reinforcements against earthquake loads.

Cement-treated gravel is used for reinforced backfill in case of GRS abutments to support bridge girders.

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