Introduction to Japanese codes for reinforced soil design

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ABSTRACT: Based on vast and very serious damage to railway structures during the 1995 Kobe Earthquake, the seismic design codes for railway structures were substantially revised. The revised codes for soil structures have following several new concepts and procedures: 1. introduction of very high design seismic loads (i.e., level 2) and three ranks of required seismic performance; 2. a recommendation for the use of geosynthetic-reinforced soil structures; 3. an evaluation of seismic performance based on residual displacement; 4. use of peak and residual shear strengths with well compacted backfill while ignoring apparent cohesion; 5. designs based on the limit equilibrium stability analysis; 6. an emphasis of good backfill compaction and good drainage; and 7. no creep reduction factor applied to design rupture strength of geosynthetic reinforcement.

1 SEISMIC DESIGN CODES OF RETAINING WALLS IN JAPANESE RAILWAY DESIGN CODES

1.1 *Introduction*

The design codes related to geosynthetic-reinforced soil structures that are most established and most referred to today in engineering practice in Japan are those described in the following design codes for railway structures:

- Railway Technical Research Institute (1997). Railway structure design standard – foundations/soil retaining structures, Maruzen (in Japanese).
- Railway Technical Research Institute (1999) - Railway structure design standard - seismic design, Maruzen (in Japanese).
- Railway Technical Research Institute (2007). Design standard for railway earth structures, Maruzen (in Japanese).

The most characteristic feature of these codes is that seismic design against very high seismic, which is likely to be most severe among those adopted in the world, is introduced. To make the above practical, several new design concepts and procedures were inevitably introduced.

The seismic design codes particularly for railway soil structures were revised based on vast and very serious damage to a great number of conventional type embankments, soil retaining walls and bridge abutments with unreinforced backfill for railways during the 1995 Hyogoken-Nambu Earthquake (i.e., so called the 1995 Kobe Earthquake) (Tatsuoka et al., 1997, 1998). The Japanese National Railway was privatized in 1987 and divided into six regional railway companies and a railway freight company. After the 1995 Kobe Earthquake, in collaboration with the Railway Technical Research Institute (RTRI) and directed by the Ministry of Transport, these companies decided to change the seismic design policy so that important railways are not closed for a long period because of failure of soil structures by earthquakes. In consultation with specialists of this topic (including the top and second authors of this article), the RTRI worked to substantially revise the seismic design codes for railway soil structures, as well as other structures. To illustrate the above, in the following, only the main characteristic features of the seismic design of soil retaining wall structures are described. Essentially the same concepts and procedures are relevant to embankments and bridge abutments with backfill.

It should be emphasized that today nearly all of the soil retaining walls designed and constructed following the new design codes are geosynthetic-reinforced soil retaining walls having staged-constructed full-height rigid facing (GRS RWs having FHR facing; Fig. 1.1). Nearly no conventional type soil retaining walls (e.g., gravity type, cantilever RC type ….) and no Terre Armee retaining walls are not constructed for railways. The total wall length of the GRS RW having FHR facing is now more than 100 km. It has also been the standard practice to reconstruct conventional type retaining walls and embankments that collapsed by earthquakes to this type of GRS RWs (Tatsuoka et al., 2006, 2007a, b).

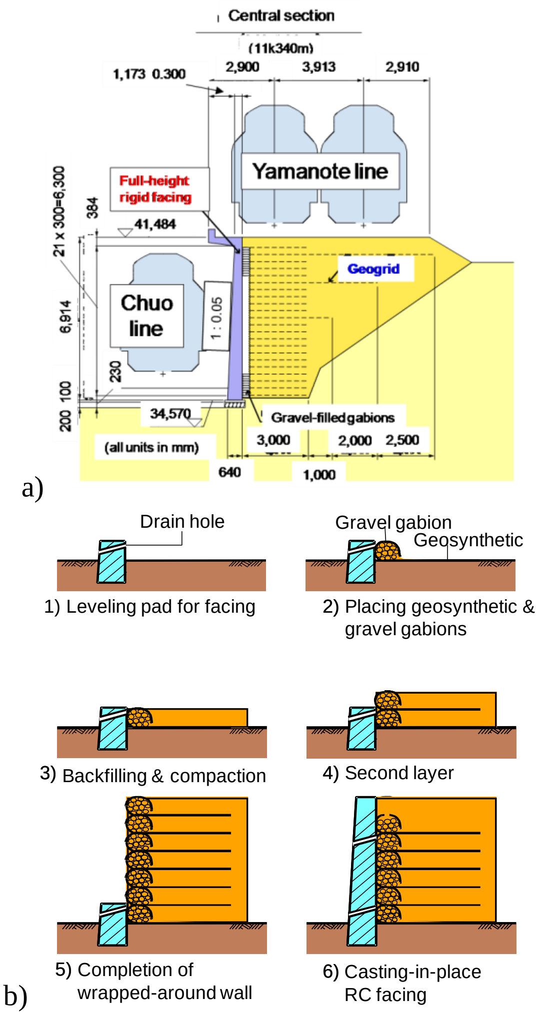


Fig. 1.1 GRS RW having FHR facing: a) a typical wall constructed for a period of 1995–2000 at Shinjuku, Tokyo; and b) construction procedure.

1.2 *Characteristic features of the seismic design codes for railway soil structures*

1.2.1 *Scope*

The new seismic design codes have several characteristic and unique features including the followings (Koseki et al., 2006, 2007a&b, 2009). It is believed that these are also very useful to develop seismic design codes of soil structures relevant to other seismic zones in the world.

1. Very high design seismic loads (i.e., level 2) and three ranks of required seismic performance are introduced in the same way as other structures.

2. It is highly recommended to employ GRS structures as highly earthquake-resistant soil structures in place of conventional type embankments, soil retaining walls and bridge abutments with unreinforced backfill.

3. Performance against level 2 design seismic load is evaluated based on residual displacement.

4. When good compaction of the backfill is confirmed, the peak shear strength of the backfill can be used, in addition to the residual shear strength, which is equivalent to the conventional standard design value Apparent cohesion is ignored in the stability analysis of soil structures, including soil retaining walls with the backfill either unreinforced or reinforced.

5. The limit equilibrium stability analysis (i.e., statics and pseudo-statics as the first approximation of rigorous dynamics) is the basis for the design.

6. The backfill compaction is controlled to be of high level and good drainage arrangement should be ensured.

7. No creep reduction is applied to the design tensile strength of geosynthetic reinforcement in the seismic design, based on the fact that creep is not a mechanical degrading phenomenon.

1.2.2 *Two design seismic load levels and three required performance ranks*

Three ranks of seismic performance are assigned against “level 1 design seismic load” (equivalent to the conventional one) and “level 2 design seismic load“, which is newly introduced (equivalent to severe seismic loads experienced during the 1995 Kobe Earthquake) (Table 1.1). The required performance ranks are determined based on the importance of concerned structures; for example, soil structures supporting steel-reinforced concrete slabs for ballast-less tracks of high speed railways is required rank I, those supporting ballasted tracks for important railways is required rank II, and other non-critical soil structures are required rank III.

Table 1.1 Two design seismic load levels and three performance ranks

Design seismic loads / Structural type (required performance rank)	Level 1: highly expected for design life T_{des} (i.e., 100 years), equivalent to the conventional design EQ load	Level 2: maximum possible for T_{des}; newly introduced, equivalent to severe seismic loads experienced during the 1995 Kobe EQ
Very important soil structures (**rank I**)	will maintain their expected functions without repair works	will not exhibit excessive deformation (can restore their functions with quick repair works)
Important soil structures (**rank II**)		will not exhibit devastating deformation
Other non-critical soil structures (**rank III**)	will not collapse	Not specified

Level 1 seismic load used in the pseudo-static seismic stability analysis is assigned to be a horizontal seismic coefficient at the ground surface k_h equal to 0.2. It is assumed that the acceleration is not amplified inside soil structures. This seismic design procedure is equivalent to the conventional one used before the 1995 Kobe Earthquake.

On the other hand, level 2 design seismic load is assigned in terms of standard time histories of horizontal acceleration at the ground surface, which are used to evaluate the residual deformation of concerned soil structure by the *Newmark* sliding block analysis (explained later). They were obtained by applying a band-pass filter (0.3 - 4.0 Hz) to the design earthquake motions specified at the ground surface. Depending on the natural period T_g of the ground estimated at a given site, different wave forms and amplitudes are assigned (Table 1.2). It may be seen that the peak accelerations a_{max} are very high, in a range from 500 to 920 gals (cm/sec^2), and the largest value is assigned for the G2 ground consisting mainly of Pleistocene deposits.

Table 1.2. Maximum acceleration of level 2 design earthquake motions: the unit is gals (cm/sec^2).

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

G0 – G7: ground classifications listed below, determined based on the natural period T_g (the unit is 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 & G6: soft; G7: very soft.

1.2.3 *Recommendation of the use of geosynthetic-reinforced soil structures*

As level 2 seismic load described above has been introduced, it is now extremely difficult to design and construct cost-effective conventional type soil structures (i.e., unreinforced embankments and retaining walls and bridge abutments with unreinforced backfill) for railways in Japan. On the other hand, when the backfill is well-compacted and its effect on the design shear strength of backfill is taken into account, it becomes quite feasible to design and construct cost-effective GRS structures, such as the one illustrated in Fig. 1.1, that can perform satisfactorily against level 2 design seismic load. In this way, it is highly recommended to employ geosynthetic-reinforced soil structures, such as the GRS RWs (Fig. 1.1), in the new design codes.

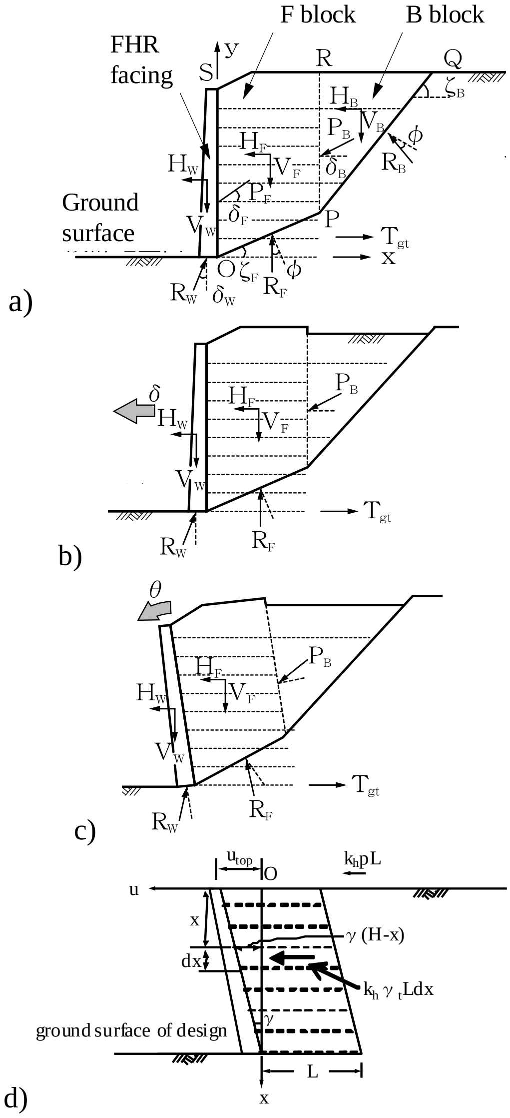


Fig. 1.2 Evaluation of residual deformation of typical GRS RW having staged constructed FHR facing: a) components considered in the two-wedge method; b) horizontal sliding along failure planes; c) overturning about the center of the bottom of FHR facing; and d) shear deformation of reinforced backfill zone.

1.2.4 *Evaluation of seismic performance of soil structures based on residual deformation*

The seismic performance of a given soil structure against level 1 seismic load is evaluated based on the factor of safety obtained by pseudo-static limit equilibrium stability analysis. This analysis method is also the basis for the evaluation of performance against level 2 seismic load based on residual displacements obtained by the *Newmark* sliding block theory (explained below). In the case of GRS RWs having FHR facing (Fig. 1.1), 1) horizontal sliding displacement (Fig. 1.2b); 2) overturning displacement (Fig. 1.2c); and 3) shear deformation of the reinforced backfill (Fig. 1.2d) are evaluated. In these analyses, the response amplification inside respective soil structures is ignored. Instead, the residual shear deformation of reinforced backfill zone, which is usually ignored in the seismic design of GRS RWs, is evaluated. The allowable residual deformations of a given soil structure is determined by the owner of that soil structure based on the criteria shown in Table 1.1. For example, for performance rank III, the ballasted track may allow a maximum residual settlement of 50 cm.

The time history of residual horizontal sliding displacement δ is obtained by integrating the equation of motion (Eq. 1.1) only when the safety factor F_R/F_D becomes lower than unity:

$$M\ddot{\delta} = F_D - F_R \qquad (1.1)$$

where, referring to Fig. 1.2a, F_D and F_R are the sliding force and resistance obtained by:

$$\begin{aligned} F_R &= R_{Wx} + R_F \cdot \sin(\phi - \varsigma_F) + \sum T_{gt} \cdot \cos\beta \\ F_D &= H_W + H_F + P_B \cdot \cos(\delta_B - \alpha) \end{aligned} \qquad (1.2)$$

After having reached the peak value φ_{peak}, the angle of internal friction φ starts dropping toward the residual value $\varphi_{residual}$ as long as yielding continues. A conservative assumption that φ suddenly drops from φ_{peak} to $\varphi_{residual}$ is employed in the modified *Newmark* method described in the Japanese railway design codes. With actual backfill, the stress fully drops only after a shear deformation increment that is essentially proportional to the particle size takes place (Tatsuoka, 2001). A possible increase in the tensile resistance of reinforcement associated with residual deformation of the wall is also ignored as a conservative simplification. The residual rotational angular displacement of the wall θ is obtained in the similar way as above by integrating Eq. 1.3 only when the safety factor M_R/M_D becomes lower than unity:

$$J\ddot{\theta} = M_D - M_R \qquad (1.3)$$

where M_D and M_R are the overturning moment and the resisting moment, both defined about the center of the bottom of the FHR facing. The residual horizontal displacement at the crest of wall due to residual overturning angular displacement becomes much smaller than the residual horizontal sliding displacements when several layers of geosynthetic reinforcement at higher levels of the wall are made longer, as in the case shown in Fig. 1.1a.

Referring to Fig. 1.2d, the residual shear displacement at the wall crest u_{top} is obtained as γH, where γ is the residual shear strain of the reinforced backfill zone, which develops only when the horizontal seismic coefficient k_h exceeds a given specified yield value k_y. The equation to evaluate γ is obtained by assuming that the external work done by seismic load is equal to the internal work done by the shear deformation of the reinforced backfill zone having a length equal to L. The shear modulus of the backfill and the value of k_y were obtained based on the model shaking table tests and calibrated by analysis of the performance of a GRS RW at Tanata that survived the 1995 Kobe Earthquake but exhibited noticeable residual deformation.

1.2.5 *Use of peak and residual strengths of backfill*

In the seismic design against level 1 seismic load, the standard design shear strengths of backfill are used in the similar way as the previous codes. These values are very similar to the residual shear strengths of backfill. The values used in the wall design are denoted as φ in Table 1.3.

Table 1.3. Standard design values of density and shear strength for wall design

Standard design values / Soil type	Soil unit weight (kN/m^3)	φ for seismic design against level 1 load	Seismic design against level 2 load	
			φ_{peak}*)	$\varphi_{residual}$
Type 1: well-graded sand & gravel	20	40°	55°	40°
Type 2: other ordinary types of sand & gravelly sand	20	35°	50°	35°
Type 3: poorly-graded sand	18	30°	45°	30°
Type 4: cohesive soil	18	30°	40°	30°

Type 1: SW & GW; Type 2: GP, G-M, G-C, G-V, S-M and GM & GC with fines content less than 30 %; and Type 3: other soil types with fines content less than 30 %; and Type 4: fines content more than 30 %.

*) These values can be used only when the compacted degree of density satisfies the specification (explained in Section 1.2.7). Otherwise, $\varphi_{residual}$ should be used.

The standard design shear strength values listed in Table 1.3 were determined by conservative judgments of the results from a comprehensive series of drained triaxial compression tests on many different backfill types representative of the railway soil structures in Japan. It is to be noted that, with gravelly and sandy soils, the apparent cohesion, which is basically due to suction in unsaturated backfill, is ignored (i.e., c= 0) in the wall design under not only static but also seismic loading conditions. This is based on conservative considerations that the apparent cohesion due to suction may decrease or even may disappear by heavy rainfall and therefore is not reliable. By following the same concept, the saturated unit weight of soil is used. Despite that it may be too conservative, in particular under seismic loading conditions, the c= 0 concept is also applied to clayey soil in these codes.

It is known that, even with GRS RWs having FHR facing, residual deformation when subjected to level 2 seismic load may become too large if calculated using these standard φ values. In the stability analysis of RWs (with the backfill unreinforced or reinforced) against level 2 seismic load, when good compaction of the backfill can be confirmed, the use of the peak friction angle φ_{peak} in addition to the residual friction angle $\varphi_{residual}$ (equivalent to the conventional standard design value) is allowed. Furthermore, even higher shear strength values of the backfill can be used if they are confirmed by relevant laboratory stress-strain tests.

1.2.6 *Limit equilibrium stability analysis as the basis of the design.*

The limit equilibrium stability analysis under static and dynamic loading conditions is the basis of these design codes. Earth pressure in full-scale retaining walls (RWs) with unreinforced backfill and tensile geosynthetic loads in full-scale GRS RWs measured under ordinary non-critical conditions are usually much smaller than respective design values. These values are not referred to in these Japanese railway design codes, because RWs should be designed for critical conditions, typically when subjected to heavy rainfalls or severe earthquakes in Japan, while the measured values usually do not include effects of heavy rainfall (i.e., loss of suction and others) and those of severe seismic loads. Moreover, the actually operated shear strength of well compacted backfill may be larger than conservative design values, which makes the measured values smaller than the design values.

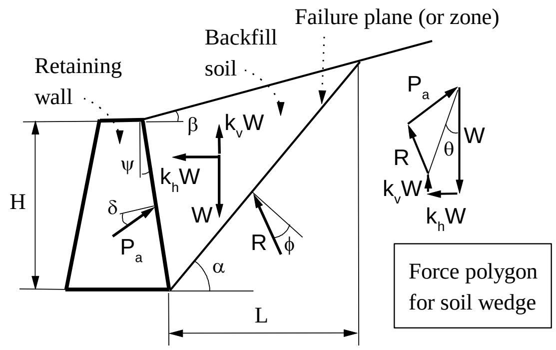


Fig. 1.3 Retaining wall with unreinforced backfill having a single linear failure plane under general seismic loading conditions (Koseki et al., 1997).

The following two specific methods based on pseudo-static limit equilibrium stability analysis using both peak and residual shear strengths of backfill are introduced:

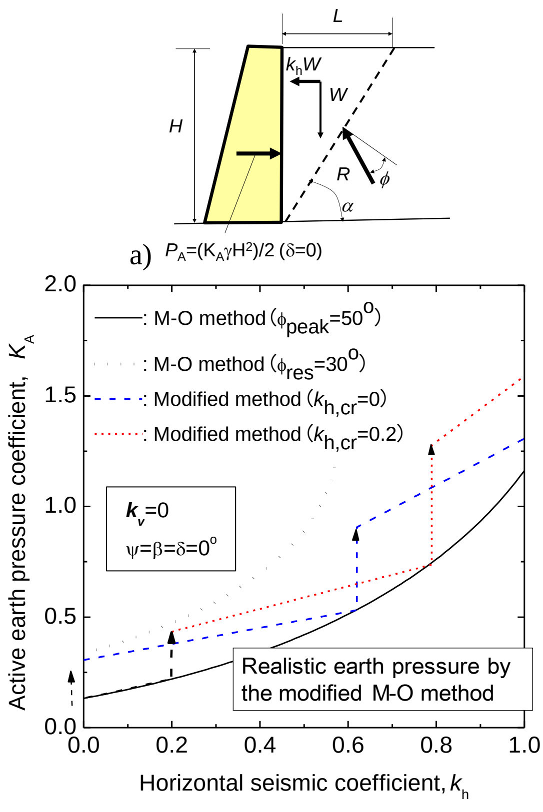
<u>Modified *Mononobe-Okabe* seismic earth pressure theory:</u> The original *Mononobe-Okabe* theory evaluates the effects of seismic inertia forces on the earth pressure in the framework of Coulomb's theory assuming that the stress-strain behaviour of soil is isotropic and perfectly plastic while using such a single linear failure plane in the unreinforced backfill as shown in Fig. 1.3. As the friction angle φ is kept constant everywhere and every time, the failure plane moves for every change in the input seismic load. When the input motion is continuously increasing, the failure plane continuously becomes deeper (i.e., the angle α continuously decreases).

In actuality, however, the compacted backfill exhibits significant strain-softening in the post-peak regime. That is, the φ value drops from the peak value (φ_{peak}) toward the residual value ($\varphi_{residual}$) only inside a shear band (i.e., a failure plane), while the peak value of φ is kept equal to φ_{peak} in the other unfailing zones (Tatsuoka et al., 1998, 2001). Therefore, when the input acceleration level becomes higher after the first failure plane has been formed at a certain input acceleration level, this failure plane develops further, without forming another deeper failure plane, until the input acceleration level becomes large enough to form a new one. Therefore, multiple failure planes are formed stepwise in the backfill after the maximum acceleration level increases exceeding the critical value at which the first failure plane is formed during a given earthquake. Based on this consideration, Koseki et al. (1997) proposed to modify the original *Mononobe-Okabe* theory taking into account this effect of strain-softening associated with shear banding.

Figs. 1.4b and c compare the horizontal earth pressure coefficient K_A and the size of the failure zone when only the horizontal seismic load (k_h) is applied, obtained by the original and modified *Mononobe-Okabe* theories for the simple RW configuration (Fig. 1.4a). In this analysis, it is conservatively assumed that φ suddenly drops from φ_{peak} to $\varphi_{residual}$, like the modified *Newmark* method explained above. The following trends may be seen from Figs. 1.4b & c:

1) The K_A value by the modified theory increases with jumps at several values of horizontal seismic coefficient k_h with a continuous increase in k_h.
2) The K_A value by the modified theory is always smaller than the value by the original theory using $\varphi_{residual}$ (i.e., the value by the conventional seismic design), while it is always larger than the K_A value by the original theory using φ_{peak}. These results indicate that the K_A values by the original theory using $\varphi_{residual}$ and φ_{peak} are, respectively, conservative and on the unsafe side.
3) The failure zone by the modified theory becomes larger stepwise with a continuous increase in k_h.
4) The failure zone by the modified theory is consistently smaller than both of those by the original theory using φ_{peak} and $\varphi_{residual}$. This trend is consistent with the model shaking table tests (Koseki et al., 2007a & b) and field observations (Tatsuoka et al., 1997, 1998).

It is to be noted that good compaction of the backfill can be rewarded and encouraged by the use of high φ_{peak} values in the modified theory in the seismic design of RWs with the backfill unreinforced or reinforced.



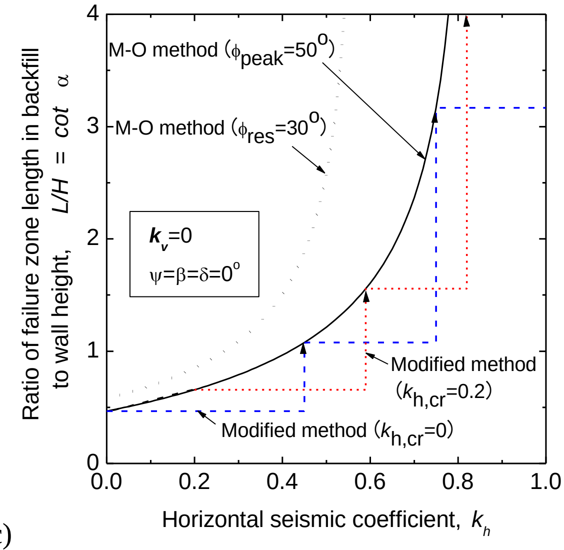
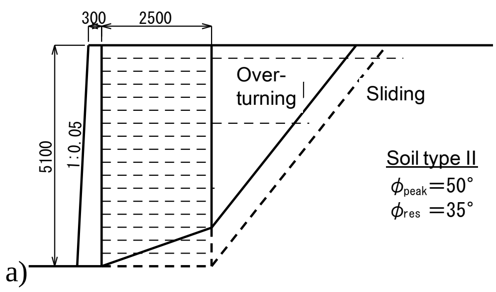
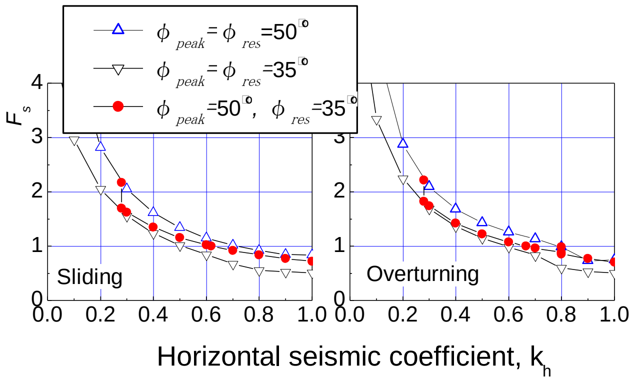


Fig. 1.4 Comparison of the original and modified *Mononobe-Okabe* theories: a) considered wall configurations; b) horizontal seismic coefficient; and c) the size of failure zone (Koseki et al., 1997).

The pseudo-static limit equilibrium stability analysis by the two-wedge (TW) method of GRS RWs (Figs. 1.2a & 1.5a) that uses both φ_{peak} and $\varphi_{residual}$ is a direct extension of the modified *Mononobe-Okabe* theory. Fig. 1.5b compares the overall safety factors for failure by sliding and overturning obtained by the TW method using φ_{peak} and $\varphi_{residual}$ with those by the TW method using either φ_{peak} or $\varphi_{residual}$ for soil type II. Similar results are obtained for the other soil types. The wall configurations in this case (Fig. 1.5a) are as follows: the wall height H= 5.1 m; the facing is 0.3 m-thick at the top; the surcharge on the backfill crest= 1.0 tonf/m^2 (10 kPa); the basic length of reinforcement is 2.5 m with a vertical spacing of 0.3 m with several layers at higher levels extended to a line at an angle of $\varphi_{residual}$; the design rupture strength of reinforcement T_d= 30 kN/m; the friction angle at the interface between the reinforcement and the backfill φ_B = "$\varphi_{residual}$ of the backfill"; and the friction angle at the bottom of the facing δ_w= $\varphi_{residual}$. It is assumed that the first failure takes place in the backfill when k_h= 0.28 for soil type II, above which the residual shear deformation of the reinforced backfill zone takes place. It may be seen from Fig. 1.5b that the safety factor by the TW method using φ_{peak} and $\varphi_{residual}$ is in between the value by the TW method using $\varphi_{residual}$, which is equivalent to the value by the conventional design, and the one by the TW method using φ_{peak}.





b)

Fig. 1.5 a) Typical configurations of GRS RW with FHR facing and critical failure planes (unit in mm); and b) results of stability analysis for soil type II (Table 1.3) (Horii et al., 1998).

<u>Modified *Newmark* block sliding theory:</u> The critical k_h values (when the safety factor becomes unity) obtained from Fig. 1.5b are plotted in Fig. 1.6a. Fig. 1.6b shows the residual displacements obtained by the modified *Newmark* block sliding theory (explained in 1.2.4), based on the results presented in Fig. 1.6a and other similar ones. Note that these analyses assume good compaction of the backfill. In these analyses, time histories of horizontal acceleration on the ground surface obtained by using the parameters assigned for the four different soil types (Table 1.3) while based on the one recorded in Kobe during the 1995 Kobe Earthquake were used. It may be seen that the residual wall deformation decreases by using backfill of higher quality. This result also encourages the use of higher quality backfill, as well as better backfill compaction, to construct GRS RWs.

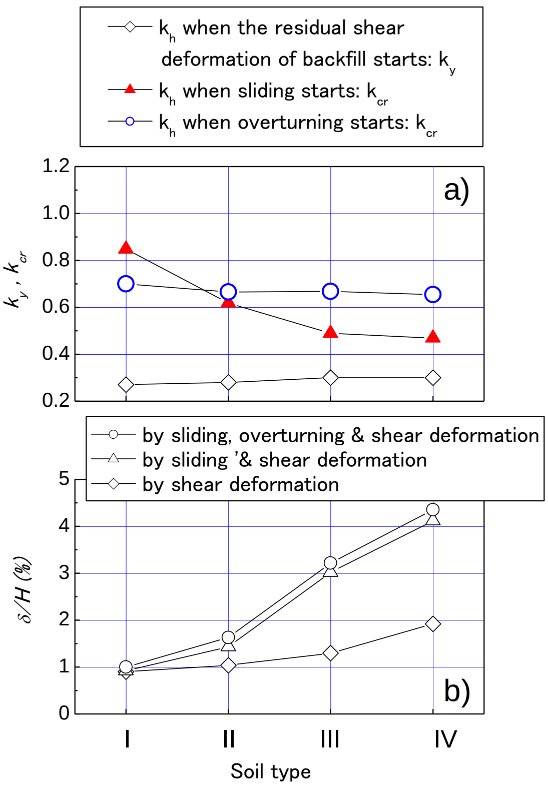


Fig. 1.6 a) Critical seismic coefficient when horizontal sliding or overturning failure starts (k_{cr}) and assumed yield k_h value above which residual deformation of reinforced backfill takes place (k_y); and b) calculated residual horizontal displacements at the crest of the wall for different soil types (Horii et al., 1998).

1.2.7 *Backfill compaction and drainage*

It is among the very important lessons learned from failure of a great number of embankments and conventional type retaining walls by recent heavy rainfalls and severe earthquakes that good backfill compaction and good drainage are among the essential keys to prevent such failures. To facilitate as good as possible compaction of the backfill for the GRS RW having FHR facing (Fig. 1.1), it is specified that the vertical spacing between vertically adjacent geosynthetic layers is 30 cm, while the standard compacted lift of soil layer is 15 cm. In the design codes, it is allowed to use the φ_{peak} values listed in Table 1.3 in the design of soil structures against level 2 seismic load only when the degrees of compaction D_c measured at a given site satisfy specified criteria: for example, for very important soil structures that are required to exhibit performance rank I against level 2 seismic load, both of the following criteria should be satisfied:

i) all measured values of D_c based on the Standard Proctor ≥ 92 %; and the average ≥ 95 %; and
ii) all measured values of the coefficient of vertical sub-grade reaction (K_{30}) obtained by plate loading tests using a 30 cm-diameter ≥ 70 MN/m^2; and the average ≥ 110 MN/m^2.

Even when the average value of D_c is around 90 %, the compaction is accepted if both of the following criteria are satisfied:

i) all measured values of D_c ≥ 87 %; and the average ≥ 90 %; and
ii) all measured values of K_{30} ≥ 110 MN/m^2; and the average ≥ 150 MN/m^2.

Good drainage is another key for high seismic performance of soil structures. With the GRS RW having FHR facing (Fig. 1.1), gravel bags placed at the shoulder of each soil layer to help better backfill compaction during the wall construction are expected to function also as a drainage layer after the wall completion. The water percolating from the inside of backfill into the gravel bags is drained to the outside of the wall through small pipes arranged for every 2 to 4 m^2 in the facing.

1.2.8 *Design tensile strength of geosynthetic reinforcement*

In most of the current design procedure, the design rupture strength (T_d) for long-term static loading conditions of a given geosynthetic reinforcement type is obtained by applying a set of reduction factors to "tensile rupture strength by fast loading test of new product (T_{ult})". As illustrated in Fig. 1.7, these reduction factors account for: 1) installation damage; 2) the possibility of creep rupture; 3) long-term degradation; and 4) overall safety factor. With respect to a reduction factor to avoid creep failure under long-term static loading conditions, it is specified in the Japanese railway design codes that the T_{ult} value is reduced to a value at which the creep failure does not take place at the end of 50 years. It is postulated that the above condition is satisfied if the strain rate after 500 hours is smaller than 3.5 x 10^{-5}/h in all three creep loading tests on a given type of geosynthetic reinforcement.

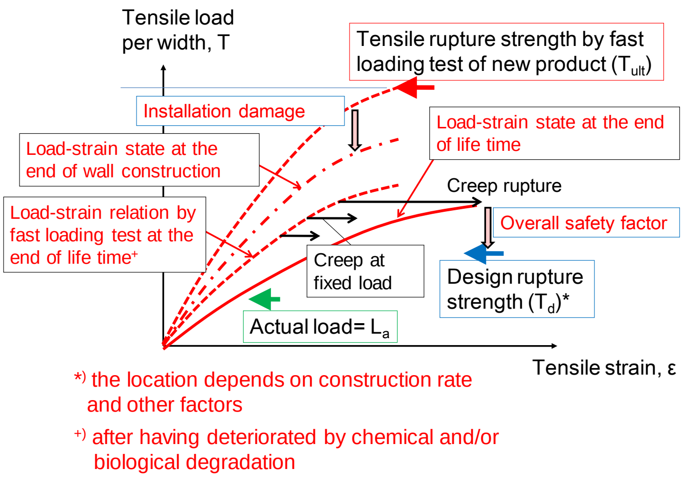


Fig. 1.7 Procedure to obtain the design rupture strength (T_d) of geosynthetic reinforcement under long-term static loading conditions compared with the value actually measured under ordinary non-critical condition (L_a).

On the other hand, in evaluating the design tensile strength of geosynthetic reinforcement against seismic loads, no creep reduction factor is taken into account in the Japanese railway design codes. This procedure is based on the fact that creep is not a mechanical degrading phenomenon (e.g., Greenwood et al. 2001; Tatsuoka et al., 2004, 2006; Tatsuoka, 2008; and Kongkitkul et al., 2007a, b), as illustrated in Fig. 1.8. Lines 1, 2 and 3 indicate three different residual strengths that are available when loaded at respective strain rates after having been subjected to constant load equal to the unfactored strength or any lower load. That is, unless the material property degrades with time by chemical and/or biological effects, the original strength of a given geosynthetic reinforcement for a given strain rate at rupture is maintained until late in its service life. When subjected to seismic loads after some long service period under constant load conditions, the original strength at a strain rate that is much higher than the value immediately before the start of this seismic event can be fully activated. According to this design concept, it is not necessary to reduce the original rupture strength by using a creep reduction factor that is determined to avoid creep rupture under static loading conditions in the seismic design of geosynthetic-reinforced soil structures.

Fig. 1.9 shows typical tensile loading tests on a PET geogrid that support the design concept described above. In one test, sustained loading (SL) was applied for 30 days during otherwise monotonic loading (ML) at a constant strain rate. The rupture strength from this test is essentially the same as those obtained by two continuous ML tests without an interruption of SL at an intermediate stage. This data set clearly indicates that, upon the restart of ML at a constant strain rate, the load-strain relation soon rejoins the one during continuous ML loading and the rupture strength does not decrease by SL at an intermediate stage, but it is rather unique function of the strain rate at rupture.

In the Japanese railway design codes, the design rupture strength (T_d) required for a given GRS RW is determined by limit equilibrium stability analysis under critical conditions, which are expected to be encountered only limited times during the life time of a given soil structure. In determining T_d, tensile loads measured in full-scale GRS RWs under ordinary non-critical condition (L_a in Fig. 1.7) are not referred to. The measured values could be considerably smaller than the design values (T_d) due firstly to apparent cohesion due to suction, which may decrease considerably or may disappear during heavy rainfall and therefore is ignored in the design. Another reason is that the shear strength that is operated in actual walls could be higher than the values used in the design, as the design values are usually specified to be conservative considering a possible variance in the quality and the degree of compaction in the backfill.

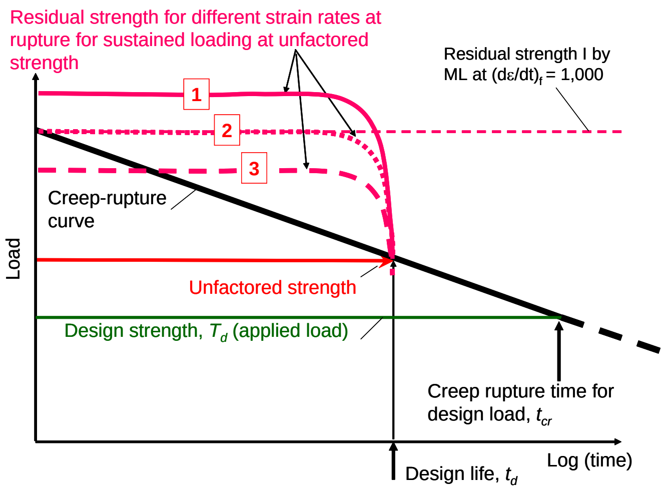


Fig.1.8 Effects of strain rate at rupture on residual strength (Tatsuoka et al., 2004)

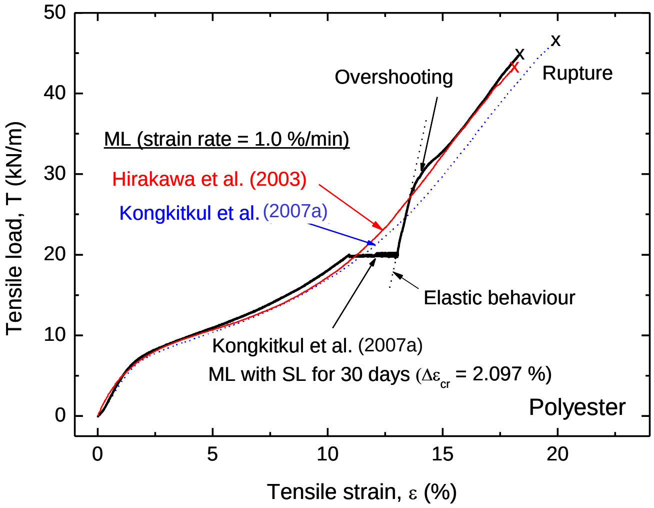


Fig. 1.9 Comparison of tensile load - strain relations from three ML tests with and without creep loading for 30 days at an intermediate load level, a PET geogrid (Kongkitkul et al., 2007a).

2 CONCLUSIONS

The seismic design codes for Japanese railway soil structures were revised substantially after the 1995 Kobe Earthquake so that soil structures can perform satisfactorily during very high seismic loads, equivalent to those experienced during that earthquake. This revision was possible only by introducing several new design concepts and procedures, which include: introduction of three required performance ranks based on the importance of a given soil structure; strong recommendation of the use of geosynthetic-reinforced soil structures; evaluation of seismic performance based on residual displacements; the use of peak and residual shear strengths with well compacted backfill while ignoring apparent cohesion; design based on the limit equilibrium stability analysis; emphasis of good backfill compaction and good drainage; and no creep reduction factor applied to obtain design rupture strength of geosynthetic reinforcement.

The use of the GRS technology to construct new type bridges that are highly earthquake-resistant and highly cost effective (Tatsuoka et al., 2009) is also highly recommended in the design codes.

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