Chapter 14

Seismic Behavior of Slopes and Embankments

Pyramid in Mexico was a place to worship God.

14.1 Classification of Seismic Failure of Artificial Embankment

Embankments such as river dikes as well as road and railway fills have been affected to different extents by earthquakes in the past. Since an earth embankment can be easily repaired, when compared with steel and concrete structures, it is important that the induced deformation is less than the allowable limit so that the damage is quickly restored.

Figure 14.1 illustrates a variety of residual deformation of earth fills which were experienced during past earthquakes. They are classified as 1) shallow surface sliding of slope, 2) development of slip surface within the body of embankment, 2') development of slip surface reaching the soft foundation soil, 3) slumping, and 4) densification. Another type of damage of fill is the one caused by fault action. A river-dike fill resting directly upon a fault was deformed as shown in Fig. 14.2 (Wufeng 霧峰 in Taiwan). This dike was quickly repaired as shown in Fig. 14.3.

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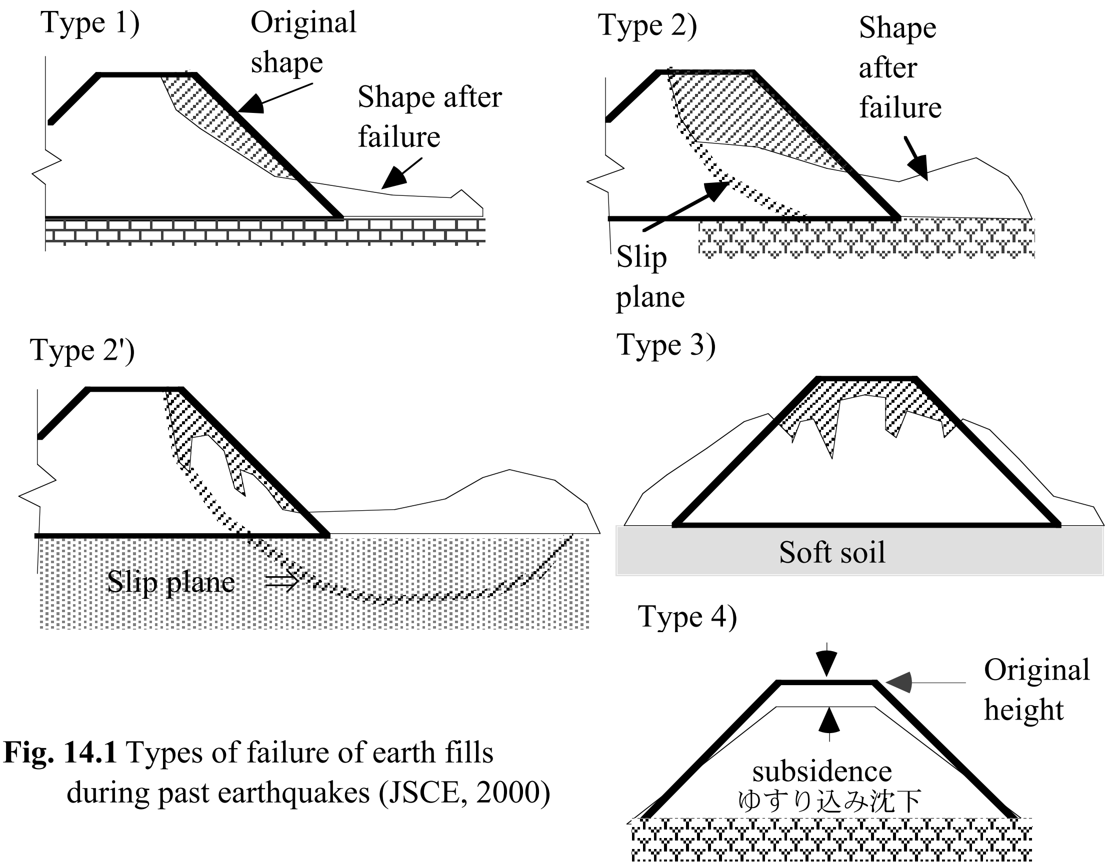


Fig. 14.1 Types of failure of earth fills during past earthquakes (JSCE, 2000)

Fig. 14.2 River dike distorted by fault action at Wufeng in Taiwan

Fig. 14.3 Repaired shape of dike in Wufeng

The quick restoration in Fig. 14.3 makes an important point that the restoration of earth structure is much easier and quick than that of steel or concrete structures. This issue will play an important role in the discussion on allowable seismic displacement in the performance-based design principle (Sect. 14.4 and Sect. 14.5).

Failure of an embankment may trigger additional problems. Fig. 14.4 shows a case of road embankment during the 2004 Niigata-Chuetsu earthquake (see Fig. 14.24 as well). The large displacement of a road fill destroyed many embedded pipelines. It is important that this site was situated upon a small valley, suggesting the potential instability of embankment upon such a small geology; see also Fig. 14.43.

Fig. 14.4 Effects of collapse of road embankment on embedded lifelines

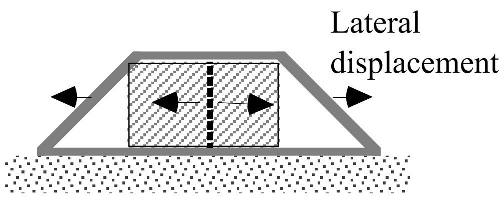


Fig. 14.5 Distortion of embedded structure due to deformation of highway embankment

Another example is demonstrated in Fig. 14.5. A highway embankment deformed in the horizontal direction and an embedded underpass concrete structure (culvert) was separated into two pieces. The separation was restored by placing steel plates. Thus, the stability of an embankment has to be discussed from the viewpoint of interaction with embedded facilities.

14.2 Example of Sliding Failure of Embankment Due to Earthquakes

Figure 14.6 manifests a development of a shallow slip plane (Type-1 failure in Sect. 14.1). Placed upon a small stream channel, this fill failed during the 1994 Hokkaido-Toho-Oki earthquake.

Fig. 14.6 Surface sliding of road embankment at Nakachambetsu site in 1994

A deeper slip plane inside a fill (Type-2 and 2') can generate a more significant damage. Fig. 14.7 shows a damaged shape of a fill at Kayanuma site (茅沼) which was constructed upon a small stream facing a peaty marsh deposit; 1993 Kushiro-Oki earthquake. It is possible that ground water flowed into the fill in place of flowing into the stream, raising the ground water table in the fill and increasing the weight. The increased seismic inertia force was not resisted by the peaty soil at the bottom (Fig. 14.8). Thus, the fill collapsed. This fill was reconstructed after the quake to the exactly same shape and failed once more during the 1994 Hokkaido-Toho-Oki earthquake.

Fig. 14.7 Failure of residential development site in Kayanuma in 1993

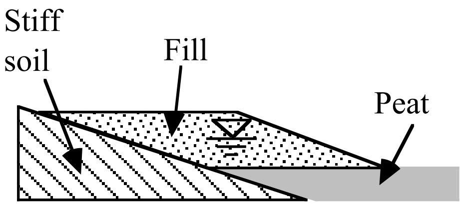


Fig. 14.8 Mechanism of failure of Kayanuma fill

Figure 14.9 illustrates a failure of a road embankment during the 2004 Niigata-Chuetsu earthquake. Situated upon a small valley, the failed earth is overtopped by stream water. This failure was probably caused by a combination of two reasons, which are elevated water table in the fill as well as the weak stream deposit that was not fully removed during construction. Thus, an embankment constructed upon a stream (集水地形) needs special care such as drainage and reinforcement at the toe of a slope.

Fig. 14.9 Failure of road embankment during the 2004 Niigata-Chuetsu earthquake

Fig. 14.10 Slip failure of fill part in cut-and-fill area (Midoriga-oka in Kushiro)

Many land development projects in hilly areas employ cut-and-fill construction to achieve a level ground surface. The interface between cut and fill can form a slip plane. Figure 14.10 indicates a sliding failure of this kind. During the 1993 Kushiro-Oki earthquake, only the bathroom of this house, which was situated upon a fill fell down, while the remaining part of the same house was intact.

🕮 14.3 Example of Slumping of Embankment Due to Earthquakes

Slumping (Type 3 in Sect. 14.1) is another important kind of failure of an embankment. The 1983 Nihonkai-Chubu earthquake caused liquefaction in subsoil under Gomyoko Bridge (五明光橋) site on the western side of Hachiro-gata (八郎潟) Lake. Figure 14.11 shows that slumping of the fill resting upon soft lake deposit produced many cracks at the surface pavement. These longitudinal cracks suggest that the fill spread laterally. Figure 14.12 demonstrates an example failure of a road embankment at Chiebunnai Bridge (智恵文内橋) during the 1994 Hokkaido-Toho-Oki earthquake. This fill was constructed on a deposit of a small stream. Thus, an embankment resting upon soft soil deposit is vulnerable to earthquake-induced failure.

Fig. 14.11 Slump failure of road embankment due to subsoil liquefaction (Akita Sakigake Newspaper)

Fig. 14.12 Failure of road embankment at Chiebunnai Bridge in 1994

Fig. 14.13 Damaged shape of Tokachi riverdike in 1993 (Ministry of Construction)

Fig. 14.14 Cracks at the top of Nagara River dike caused by 1891 Nobi earthquake (Photo supplied by Ministry of Construciton)

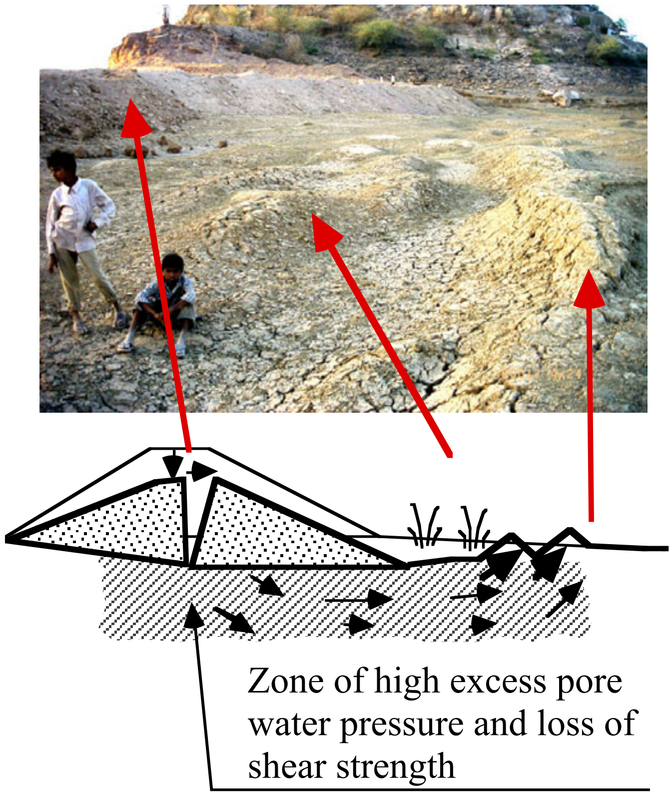


Fig. 14.15 Ground distortion in soft foundation of Chan Dam (2001 Gujarat earthquake in India; Towhata et al., 2002)

Fig. 14.16 Bridge out of service due to subsidence of approach embankment (Minami Yamabe Bridge of Ojiya City, 2004 Niigata-Chuetsu earthquake)

Figure 14.13 illustrates a damaged shape of the Tokachi river dike (十勝川堤防) during the 1993 Kushiro-Oki earthquake. Note that the center of the dike subsided more than the lateral slopes. Since an evidence of liquefaction (sand boil) was found here although the foundation soil was unliquefiable peat, the source of liquefaction was discussed. Sasaki (1998) stated that the original sandy body of the dike subsided into peat, expanded in its volume (loosening) and became liquefiable. For more details, refer to Sect. 17.4. Since this site took five months to repair, the conventional philosophy of quick repair before flooding comes did not function. Hence, a need of seismic resistance of important river dikes was recognized for the first time. Figure 14.14 shows a classical record of Nagara River dike which was destroyed by the 1891 Nobi earthquake. The longitudinal cracks suggest the tensile deformation of the dike in the lateral direction (Type 3 in Fig. 14.1). See in this figure the falling of railway bridge (Kansai Line) as well.

It is very important that the stability of foundation soil significantly affects the stability of an overlying embankment. Figure 14.15 illustrates a case of Chan Dam in Gujarat Province of India which was destroyed by an earthquake in 2001. Although the dam body was compacted in accordance with the regulation, not much attention was paid to the natural deposit under the dam body. The subsoil liquefied when earthquake occurred, and its large distortion destroyed the dam body. It is interesting that the heaving at the end of the distortion zone was visible at the time of the author´s site visit.

Densification of road embankment due to strong shaking results in subsidence. If this occurs near an abutment of a bridge, the road transportation is stopped (Fig. 14.16) and even an emergency traffic is prevented. Thus, although the main body of a bridge is of good seismic design, its function is stopped by the soil part.

14.4 Statistics on Types of Subsidence of Embankment Due to Earthquakes

The JSCE committee on earthquake engineering studied 47 cases of earthquake-induced distortion of fills. The number of cases belonging to each of four mechanisms as stated in Sect 14.1 is shown below:

5 cases of type 1, 23 cases of type 2, 12 cases of type 3, and 7 cases of type 4.

Although the cases were chosen in an as-random-as-possible way, the failure type 2 (development of a slip plane inside a fill) is the majority. It was reported further that all the failures of type 3 (slumping) were associated with development of high excess pore water pressure either in the fill body or in the foundation.

Figure 14.17 illustrates the ratio of subsidence/height in each case. It is shown therein that Type 2 (slip plane) and Type 3 (slumping) can generate larger subsidence/height ratio than other types. It is interesting that the maximum subsidence / height ratio is less than 0.7 which is consistent with the finding from river dike damage (Fig. 17.36).

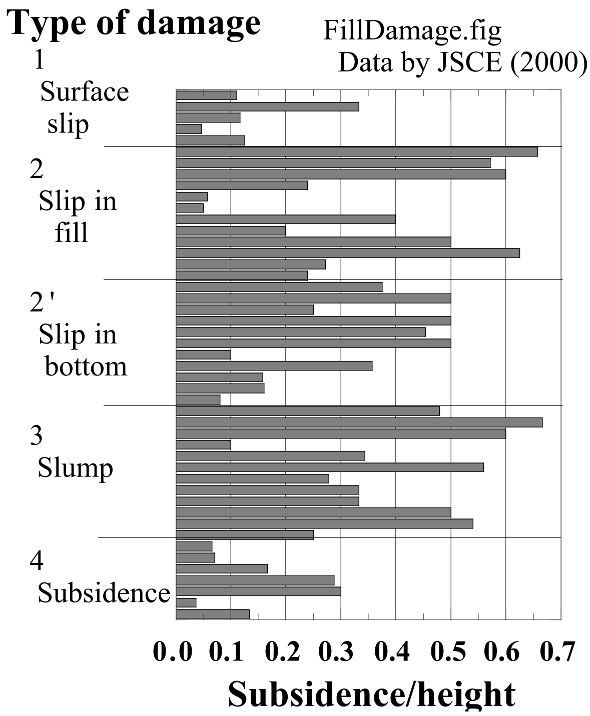


Fig. 14.17 Subsidence/height ratio of damaged fills (JSCE, 2000)

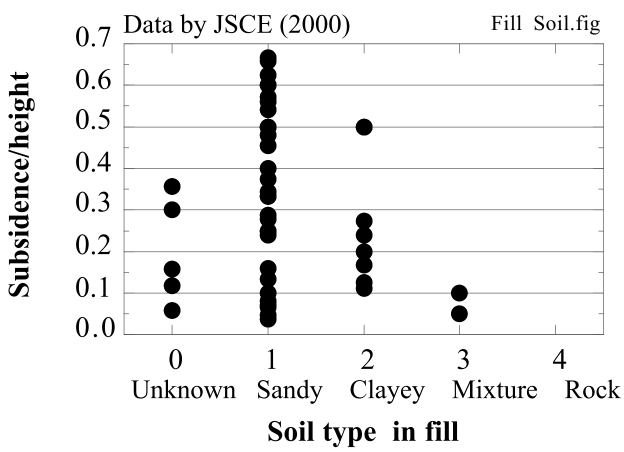


Fig. 14.18 Effects of soil type in embankment on extent of subsidence (JSCE, 2000)

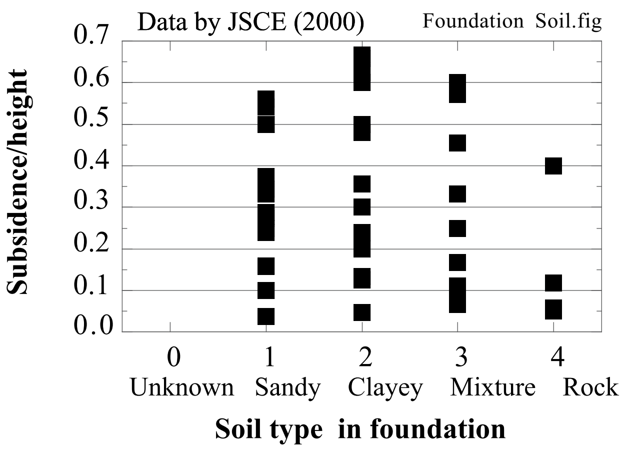


Fig. 14.19 Effects of soil type in foundation on extent of subsidence (JSCE, 2000)

Figure 14.18 illustrates the magnitude of subsidence (subsidence/height ratio) changing with the type of soil in the embankment. It is found that sandy fill material can induce significant subsidence. Furthermore, Fig. 14.19 examines the effects of soil type in foundation on the extent of subsidence. Again, sandy foundation can cause large subsidence. These findings are probably related to excess pore water pressure development and consequent liquefaction either in fill or in foundation. It is important in Fig. 14.19 that clayey (or peaty) foundation soil might generate significant subsidence as well.

14.5 Performance-Based Seismic Design

The conventional principle in seismic design of geotechnical structures has been based on the allowable stress concept. Therein the factor of safety, F_s, is calculated by

$$F_s = \frac{\text{Resistance}}{\text{Static Load} + \text{Seismic Load}} \quad (14.1)$$

and should be greater than unity. The seismic load in (14.1) is related to the expected intensity of earthquake. Figure 14.20 illustrates the variation of maximum acceleration which has been recorded during earthquakes. It is noteworthy that the intensity of acceleration increased suddenly after 1990. For examples, see the 1993 Kushiro record in Fig. 6.23 and the 1994 Tarzana record in Fig. 6.25. This situation led to the increased intensity of design earthquakes and the magnitude of seismic load in (14.1).

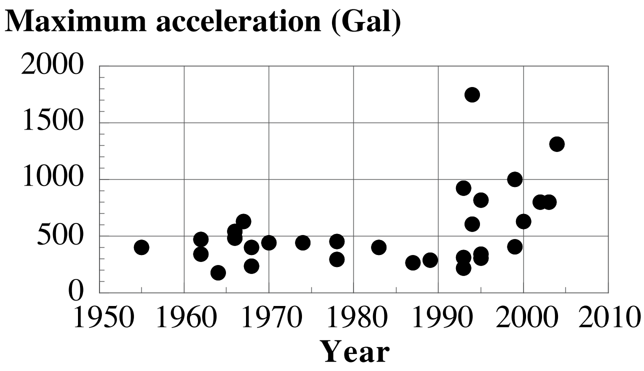


Fig. 14.20 Variation of maximum acceleration in recent earthquakes

Figure 14.20 does not mean that the seismic activity of the earth planet changed after 1990. Actually, since earthquake observation network was installed in many countries, the probability of obtaining very strong motions increased.

One of the problems in geotechnical engineering caused by the increased seismic load is that it is hence difficult to maintain the factor of safety in (14.1) greater than unity. This is certainly because the shear resistance of soil is limited. Thus, an alternative idea is desired. Evidently, it is not good to replace all the earth structures by reinforced concrete.

Fig. 14.21 Total collapse of road bridge in Taiwan, 1999 (after 1999 ChiChi earthquake in Taiwan)

Fig. 14.22 Quick construction of temporary road embankment after collapse of bridge (1999, Taiwan)

It should be stressed that performance of geotechnical structures is different from those of steel and concrete structures. Even if the factor of safety is far less than unity, the situation may still be different from what is called total collapse. The most important feature is the quick restoration. A typical example was presented in the restoration of damaged dike in Taiwan (Fig. 14.3). Figure 14.21 illustrates a total collapse of an important bridge in Taiwan at the time of the 1999 ChiChi earthquake. When the author visited the site two weeks after the quake, a temporary road had already been constructed by earth fill (Fig. 14.22). Figure 14.23 manifests a similar example of river dike restoration after the 2003 Tokachi-oki earthquake.

Fig. 14.23 Quick restoration in dike of Ushishubetsu River in Hokkaido

Fig. 14.24 Significant subsidence of road embankment resting on small valley topography caused by 2004 Niigata-Chuetsu earthquake

Fig. 14.25 Quick construction of temporary road next to the site of Fig.14.24

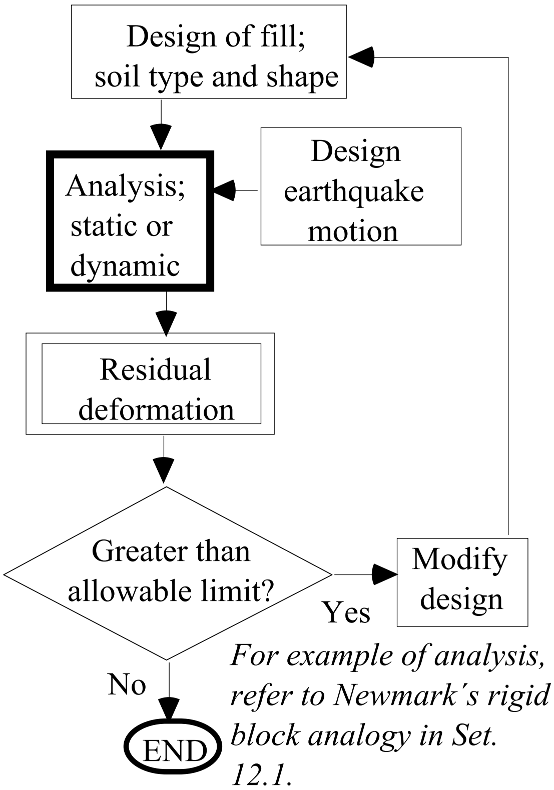


Fig. 14.26 Flow of seismic design based on allowable deformation

Quick restoration and construction are important and unique feature of geotechnical structures. The total collapse of a road embankment in Fig. 14.24 appears to be too significant to be allowed. The seismic stability of this part of road embankment was bad because it was placed many decades ago on soft valley deposits. From the viewpoint of oil pipeline and communication lifelines which were embedded in this embankment, this collapse was certainly not allowable. From the viewpoint of road, in contrast, this case may still be allowable. Fig. 14.25 shows the reason; a detour road was constructed next to this collapsed fill within a few days after the quake. Hence, the road traffic was not stopped for many days.

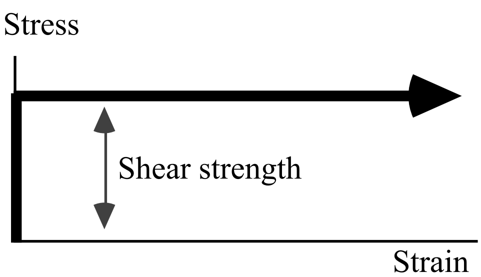


Fig. 14.27 Rigid perfectly plastic model of soil

The future principle in seismic design will allow the factor of safety to be less than unity and maintain the residual deformation within an allowable extent (Fig. 14.26). Since the seismic load is not of a static nature as (14.1) hypothesizes, its effect lasts for a short time (Sect. 5.12), and does not cause an infinite magnitude of deformation even if the factor of safety is less than unity.

The performance-based design evaluates the residual deformation/displacement of an earth structure undergoing a design earthquake motion. Thereinafter, the calculated residual deformation/displacement at the end of the earthquake is compared with a prescribed allowable value. If the calculated value is greater than the allowable limit, the original design has to be modified. Figure 14.24 illustrates a conceptual flow of this principle. This new principle and the conventional one based on the static seismic coefficient and the factor of safety are compared in Table 14.1.

In spite of the simple idea as above, the new design principle is not yet fully in practice because of the following reasons:

1. Prediction of residual deformation which remains after earthquake is not easy. It is difficult even with an advanced nonlinear finite element analysis. For most of earth structures, an advanced elastoplastic analysis is not feasible because ordinary earth structures do not afford expensive insitu investigations and laboratory tests. Soil parameters have to be determined only by SPT-*N*, cone penetration, and likes.
2. In this respect, the Newmark´s rigid block analogy (Sect. 12.1; Newmark, 1965) is one of the good choices because it assumes a very simple, rigid, perfectly plastic constitutive relationship (Fig. 14.27). It should be borne in mind, however, that the determination of appropriate strength parameters (drained, undrained, static, or dynamic) is still disputable. Furthermore, the Newmark method is not appropriate when deformation of soil is less than failure strain because the method assumes the development of failure mechanism (rigid and perfectly plastic behavior). Liquefaction-induced large deformation is out of scope as well, because liquefaction-induced displacement is a consequence of large strain in place of shear failure mechanism (slip plane) as Newmark assumed.
3. There is not a clear idea on how to determine the allowable deformation. From the viewpoint of limit state design, it is not evident what kind of limit state is appropriate for geotechnical earthquake design; serviceability, restoration, or ultimate failure. If somebody says that 50 cm is the allowable limit, what about 55cm? What is the reason to insist on 50 cm? Most probably, the allowable limit is related to risk to human life, damage cost, pause period of expected service, influence to region/nation, time for restoration, and others. It is important to take into account the situations in Figs. 14.22, 14.23, and 14.25.

Table 14.1 Features of seismic design principle based on allowable deformation

	New design principle based on allowable deformation	Conventional principle based on factor of safety
Input earthquake effects	Time history of acceleration	Static inertia force
Nature of soil	Rigid perfectly plastic (Fig. 14.24) or nonlinear stress–strain model	Rigid perfectly plastic
Method of calculation	Dynamic analysis to solve equation of motion	Static calculation on limit equilibrium
Criteria of design	Residual deformation < Allowable limit	Seismic factor of safety >1.0 (maybe >1.05?)

14.6 Inquiry on Allowable Seismic Displacement

Since there are many uncertainties in determination of allowable limit of seismic residual deformation and/or displacement, inquiries were made by JSCE (2000) to engineers and officers who were involved in restoration of damaged geotechnical structures after major earthquakes in 1990s (earthquakes in Hokkaido and Kobe). The author was a chairman of a committee in charge of this study (Towhata, 2005).

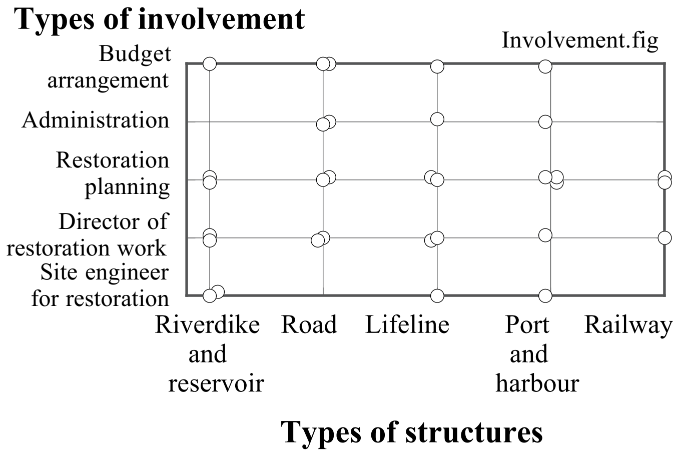


Fig. 14.28 Types of involvement of inquired people

The inquired people were involved in restoration in such manners as taking the initiative of reconstruction at sites, design of reconstructed structures, administration, and others; see Fig. 14.28. It was expected that the difficult experiences gave people reasonable ideas about the extent of allowable displacement. Note that their answers are personal and do not represent any official view of their institutes and/or companies. On the contrary, those who simply use the facilities (e.g., passengers of trains) were not included because they often demand too much safety. Moreover, the concerned structures include harbor quay walls, river dikes, irrigation dams, roads, and embedded lifelines.

Table 14.2 Factors that affect the allowable displacement

Importance	Factors			
	Human life	Negative effects to public	Difficulty in restoration	Cost of restoration
1	14	3	0	1
2	1	**12**	0	1
3	1	0	4	7
4	0	0	7	5

The first question addressed the key issue in determination of allowable displacement. As shown in Table 14.2, human life was chosen as the absolutely most important issue. This issue, however, was eliminated from further discussion because seismic failure of geotechnical structures does not affect human life significantly; most victims are killed by collapse of houses or failure of natural slopes. Among the remaining choices of negative effects to the public (社会的迷惑), difficulty in restoration, and cost of restoration, 12 out of 14 answers chose the negative effects as the second important issue (Table 14.2). Thus, the following discussion will focus on measures to reduce the negative effects. Being contrary to the initial expectation, the restoration cost was not chosen probably because the inquired people belonged to public sectors.

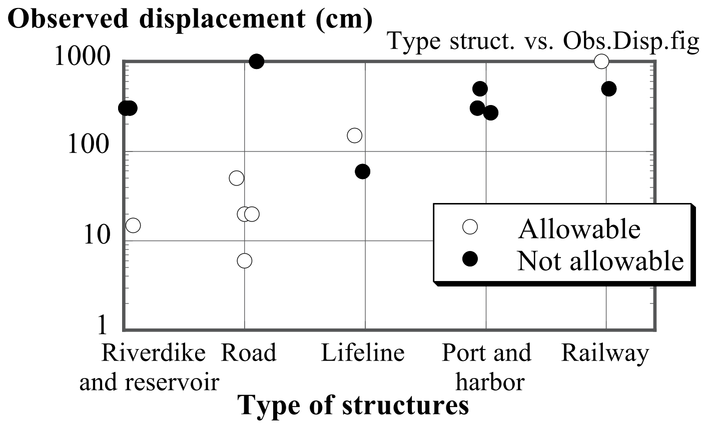


Fig. 14.29 Relationship between magnitude of displacement and people´s attitude

The next question was asked to whether or not the people allow the geotechnical damage that they experienced. Figure 14.29 plots the observed displacement values by using two different symbols depending on whether the displacement is allowed or not. Here, “to allow” means to consider the results of the quake as what should be restored without need for seismic reinforcement. Conversely, “not to allow” means that measures should have been taken in order to mitigate the damage prior to the quake.

Moreover, the displacement in this section is the biggest one in each structure whether it is vertical or horizontal. Figure 14.29 gives a simple idea that displacement greater than 200 cm is not allowed, although the 1000-cm displacement of a railway embankment was considered allowable.

Costs for restoration and reconstruction do not affect the idea on allowability as shown in Fig. 14.30. This is consistent with the opinions shown in Table 14.2.

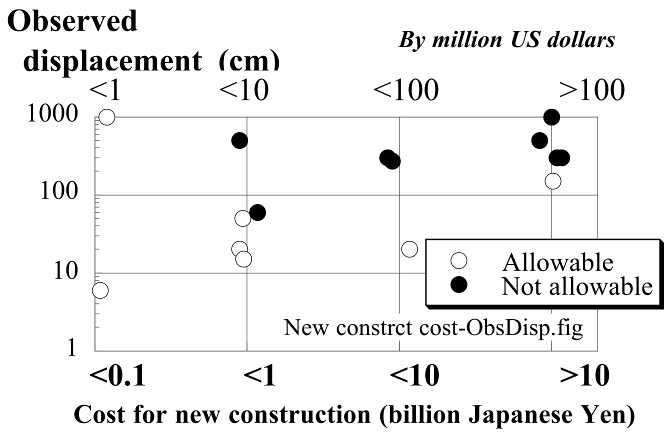


Fig. 14.30 Relationship between observed displacement and cost for restoration

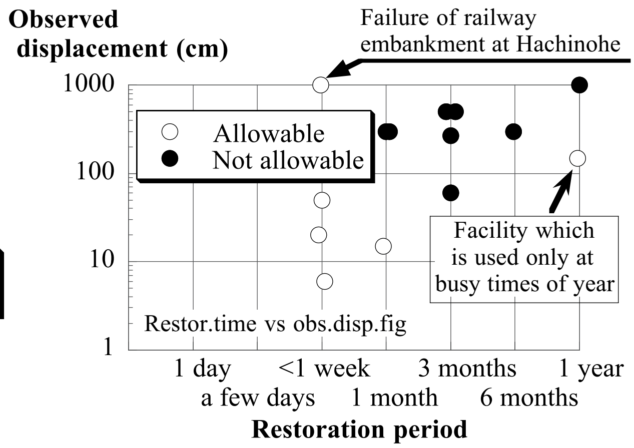


Fig. 14.31 Relationship between observed displacement and time needed for restoration

Since the negative effects to the public is a very important issue, efforts were made to understand the details. First, it is easy to understand that railway customers are in touble if train service stops for a long time. In this regard, the time needed for restoration was studied. Figure 14.31 illustrates that the restoration period longer than one month is taken seriously. People on the contrary allow period without service until one month after big earthquakes.

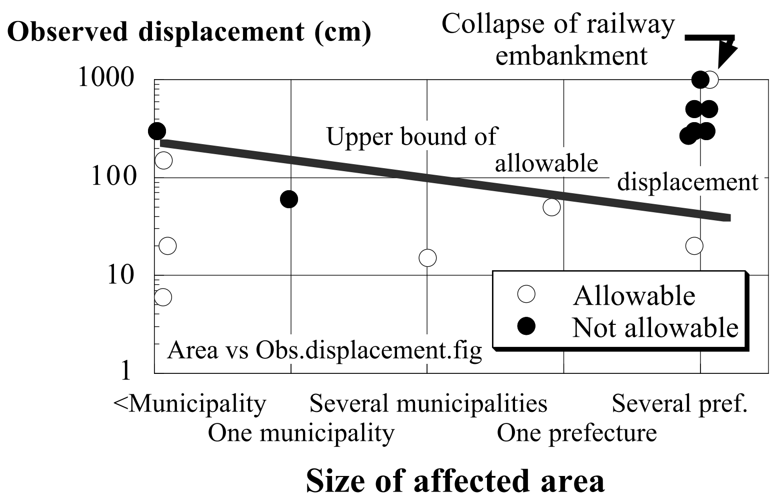


Fig. 14.32 Relationship between size of affected area and observed displacement

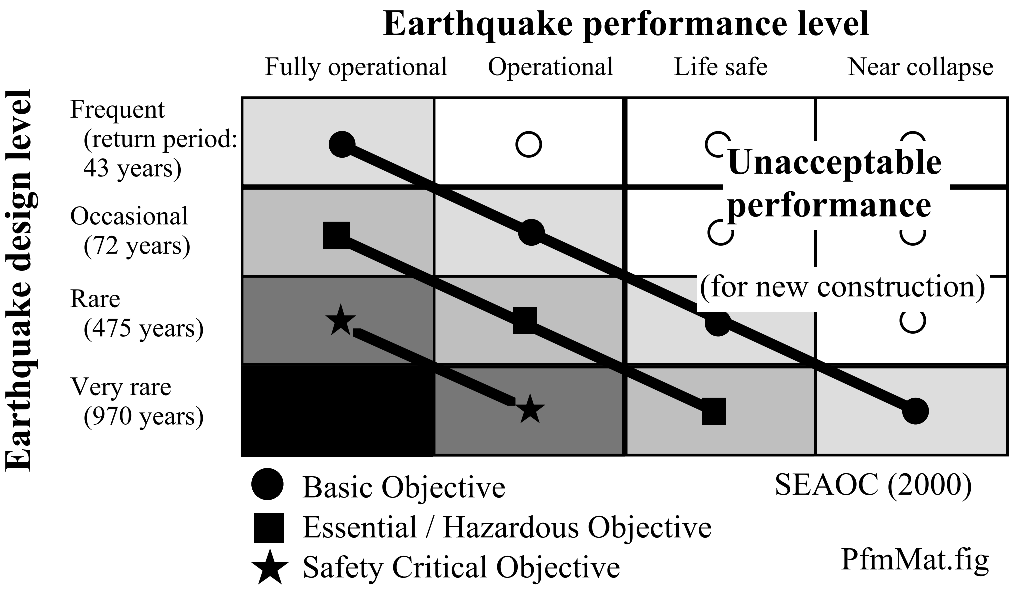
The second component of the negative effects to the public lies in the size of the affected public. The effects to the whole nation are certainly more significant than those to a small village. In this regard, the present study inquired people about "the size of the affected area" as the simplest parameter that was easy to answer, although the size of the affected population or the size of the affected economy are more suitable for further studies. To make the answering even easier, the inquired people were requested to answer in terms of the municipal units. In Fig. 14.32, one municipality stands for a city, for example, with a population of tens of thousand to a few million. As for the prefecture, it is a good instruction to state that Japan has 47 prefectures in total. It appears reasonable to state that less extent of displacement is allowed when the affected area is greater.

In summary, experienced engineers and officers wish to mitigate the negative effects to the public and the negative effects consist of two factors that are the time without service (restoration time) and the size of the affected public.

14.7 Principle of Performance-Based Seismic Design and Life Cycle Cost

Experiences of strong acceleration, typically greater than 700 Gal, during the earthquakes in 1990s urged the seismic coefficients in conventional seismic design principle to be raised substantially. Consequently, design procedure found that soil cannot resist the increased seismic inertia force as specified by the revised design requirement. This is the reason why a displacement-based design principle is desired, which allows a conventional factor of safety less than unity. It is therein desired that the consequence of seismic factor of safety which is less than unity still remains within an allowable extent. The allowable extent varies with different seismic performance requirements. Since the design relies on the earthquake-induced displacement/ deformation which is a seismic performance of a structure, the design procedure is called performance-based design.

Table 14.3 Example of performance matrix (redrawn after SEAOC, 2000)



A similar approach has been investigated widely in other fields of earthquake engineering. An example of *performance matrix* is shown in Table 14.3 where the consequence of earthquake response in a building is classified into four performance levels, and this level varies with the importance of structure (basic, essential/hazardous, and safety critical) and the intensity or rareness of design earthquake. Even in the worst case, it is required that a total collapse which would claim many casualties should be avoided. Each performance in the table may be related to different kinds of limit states (serviceability, restorability, and ultimate state) in design principles.

The recent discussion on performance-based seismic design takes into consideration the universal idea in Table 14.3. What is special in geotechnical engineering is (1) the extent of allowable damage is evaluated by the magnitude of *allowable displacement*, and (2) the importance of structure is represented by the size of affected area combined with time accepted for restoration (Sect. 14.6). These ideas account for the engineer´s opinion obtained by the aforementioned inquiry.

The results of inquiries in Sect. 14.6 are further introduced in what follows. Discussion here is focused on the allowable displacement in place of the observed (real) displacement in Sect. 14.6. Since the allowable displacement is the one in the mind of those who had difficult times in restoration after big earthquakes, there is a reasonable thinking behind.

First, Fig. 14.33 shows the relationship between the allowable restoration time and the allowable displacement. When those people allowed longer restoration time probably due to very strong shaking or reduced importance, the greater residual displacement is allowed. Second, the upper bound in Fig. 14.34 illustrates that the allowable restoration time becomes shorter when the affected area (affected population and economy) becomes larger. This certainly implies that important structures have to be restored more quickly. Further note in these figures that the idea of allowable displacement is significantly variable, suggesting that direct decision on the magnitude of allowable displacement is not an easy task; if 30 cm is allowed, why 35 cm is not allowed?

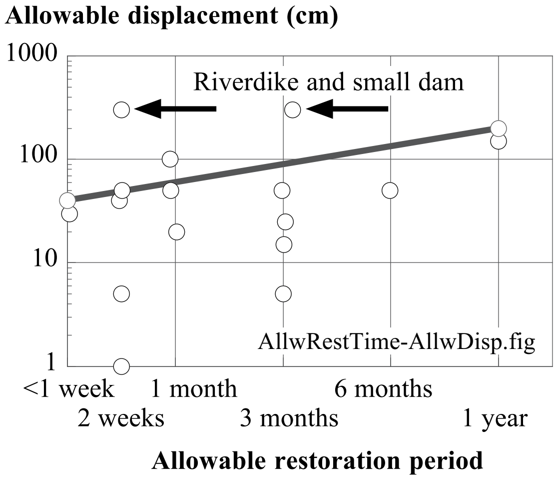


Fig. 14.33 Relationship between allowable restoration time and allowable displacement

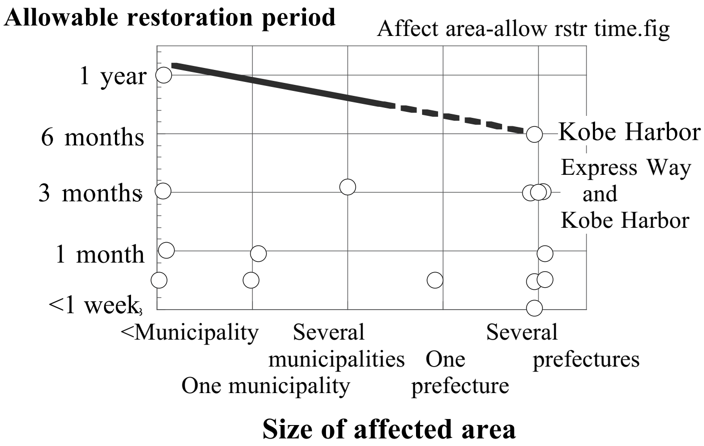


Fig. 14.34 Relationship between size of affected area and allowable restoration period

Table 14.4 Concept of matrix of allowable displacement

Importance of structures	Intensity of design earthquake	
	Strong	Rare and extremely strong
More	Not damaged	Restorable damage
Less	Restorable damage	Avoid collapse

It is aimed that the performance-based design principle for geotechnical structures under seismic effects follows the idea in Table 14.4 which is a revision of more universal Table 14.3. Emphasis is therein placed on importance of structure and time needed for restoration. Note that the negative effects to the public caused by seismic damage consist of the size of affected area and the duration of time without service; both are taken into account in Table 14.4 by “importance” and “restoration.”

To make the idea more realistic, Fig. 14.35 summarizes the opinions in Figs. 14.33 and 14.34. It is therein proposed to first decide the size of the affected area. It is a single village if a small bridge is the target structure. Conversely, the whole nation is the affected area if a nation´s No.1 highway is the target. This decision seems easy. The second decision is then made of the allowable restoration time: only a few days or several months, etc. This decision is easier than that on allowable displacement. Based on these two decisions, the magnitude of the allowable displacement is determined by this figure.

According to Fig. 14.35, a structure which affects several prefectures is an important structure. Hence, smaller displacement is allowed. However, when a design earthquake is a very rare one whose recurrence period is hundreds of years, a longer restoration period should be allowed. Then the allowable displacement should be increased to some extent. Once the allowable displacement is thus determined, design and prediction of displacement are made. Thus there are three important components in performance-based seismic design of geotechnical structures which are

1. Determination of allowable displacement
2. Design and construction
3. Practical but reliable prediction of earthquake-induced displacement

Note that prediction has to be reasonably cheap and easy. Advanced numerical analysis is not appropriate for ordinary bridge abutment etc. It is evident that the reliability of prediction relies on the quality of soil investigation in which accuracy as well as detection of spatial variation of soil properties is important.

There have been several attempts to determine the allowable displacement. Their consequences are tabulated in Table 14.5.

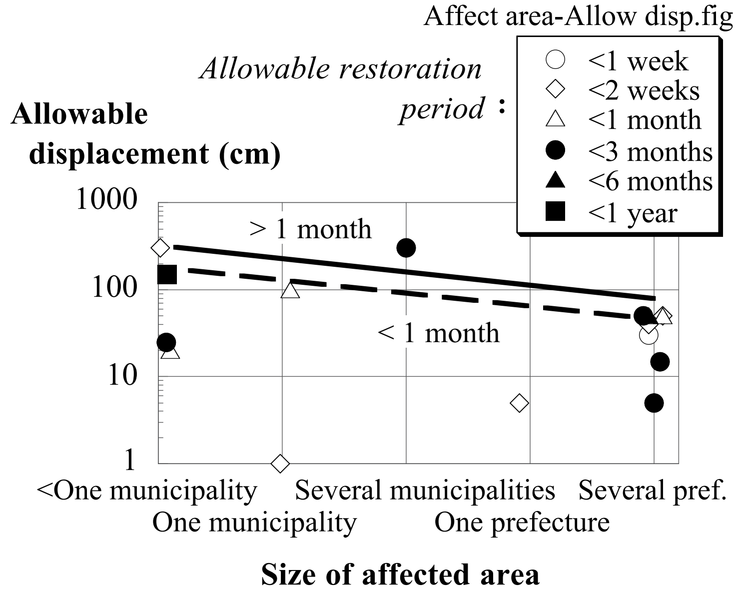


Fig. 14.35 Relationship between allowable displacement and size of affected area in terms of allowable restoration time

Table 14.5 Examples of allowable displacement of geotechnical structures in Japan

Railway	River dike
No damage : negligible displacement Minor damage : subsidence < 20 cm Quick restoration : 20–50 cm Longer restoration : >50 cm	Top and river side : displacement < 50 cm Super river dike (Fig. 7.13) with urban development: < 20 cm

The performance-based design principle will be extended in future to the idea of minimization of life cycle cost. The life cycle cost (*LCC*) of a geotechnical structure stands for the combination of the initial construction cost (C_i), the maintenance cost (C_m) during the service period (life cycle of *N* years) of a concerned structure, and the cost caused by a natural disaster (C_e) such as an earthquake:

$$LCC = C_i + C_m + \sum_{k=1}^{N} P_k \, C_{e,k}\,, \qquad (14.2)$$

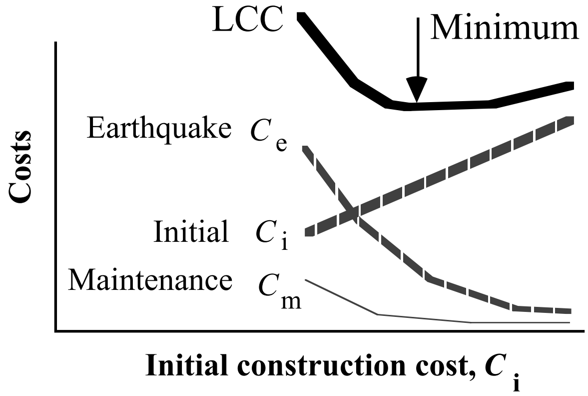


Fig. 14.36 Conceptual illustration of minimization of life cycle cost (LCC)

in which the third term on the right-hand side calculates the earthquake damage cost for the *k*th year from k = 1 to *N*. This cost is calculated by using the probability of damage, P_k, and the induced cost, $C_{e,k}$, in the respective year. It is expected that the initial construction at a reasonable cost (not too expensive but not too cheap) can keep the maintenance and disaster costs at reasonably low levels and the entire LCC would be minimized (optimization); see Fig. 14.36.

The problems to be overcome may be as what follows:

1. For geotechnical structures, the length of life (*N* years) is not clear; river dikes of more than 1,000 years old are still used today.
2. Maintenance becomes necessary not only because of the quality deterioration of a constructed earth structure but also because of the underlying natural soil condition (consolidation).
3. Seismically induced cost consists of the direct cost (restoration) and the indirect cost (economic loss). The latter is particularly difficult to evaluate. For example, the entire economic loss due to a big earthquake can be evaluated, but how much of that loss is caused by a single particular failure of a highway embankment?

An example calculation of LCC was conducted on an expressway embankment (Ishihara et al. 2007). Figure 14.37 shows a cross section of an embankment studied which is underlain by very soft clay. It is possible upon a strong earthquake, therefore, that a shear failure mechanism is activated through the embankment and the soft subsoil, resulting in subsidence of the road pavement and cars may crash into it. Consequently, passengers in those cars are killed in such an accident. Moreover, the restoration of the expressway takes time, and the function of the expressway stops for a long time, thus causing economic loss in regional and even national economy. During the period of restoration, many vehicles come into local small roads and the number of traffic accidents may increase.

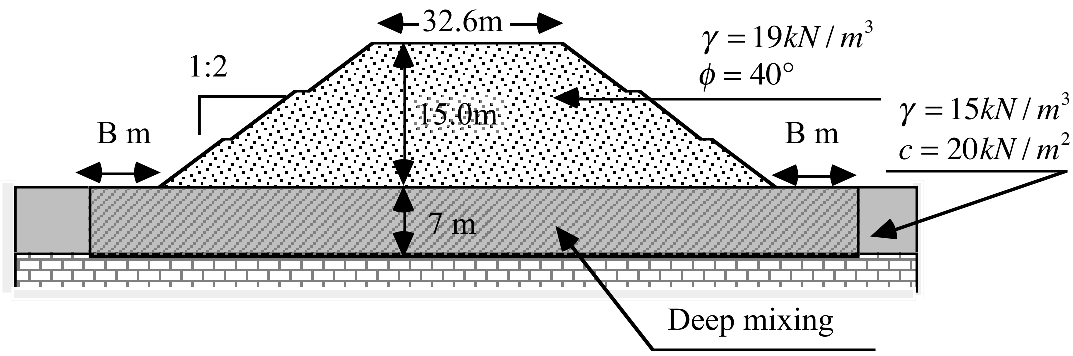


Fig. 14.37 Cross section of expressway embankment with soft subsoil (Ishihara et al. 2007)

The present study employed deep mixing (mixing clay with cement; Sect. 26.13) of clay as a mitigation of soft clay. The size of deep mixing and the ratio of improved soil mass were variables. For definition of the size of deep mixing (B), see Fig. 14.37 The types of cost which were included in the LCC calculation are as what follows:

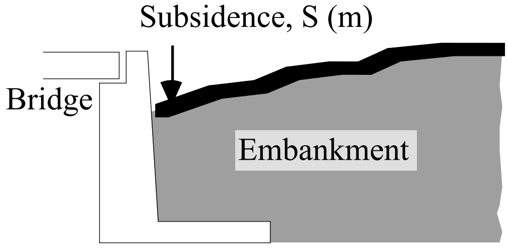


Fig. 14.38 Subsidence of road pavement as a cause of car crush

1. Initial construction cost in which the expense for deep mixing increases as the range and quality of soil improvement increase
2. Direct damage cost upon earthquake:
   – Restoration of embankment
   – Human life due to car crush into subsidence (S in Fig. 14.38)

3. Indirect damage cost after earthquake;
   - Elongated travel time (due to shifting from expressway to local roads)
   - Missing toll fee income
   - Increased traffic accidents in local roads
   - Reduced air pollution along expressway (environmental issue)

By referring to many practices in both public and private sectors, the number of days needed for restoration was determined (Fig. 14.39). The restoration time is totally different for S greater than or smaller than 15 cm because of different types of restoration. On the basis of this S, the costs were determined as second-order functions of S. See Fig. 14.40 for details.

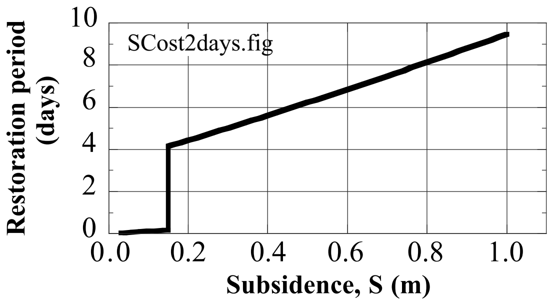


Fig. 14.39 Number of days for restoration

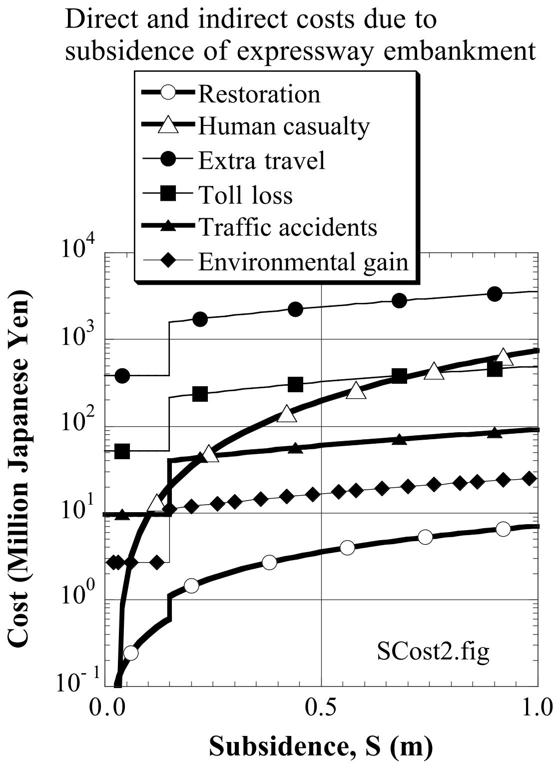


Fig. 14.40 Damage costs in express way as functions of subsidence

Fig. 14.41 Good condition of ancient irrigation dam in Sri Lanka

(a) Variation of LCC with extent of soil improvement (b) Change of LCC with initial construction cost

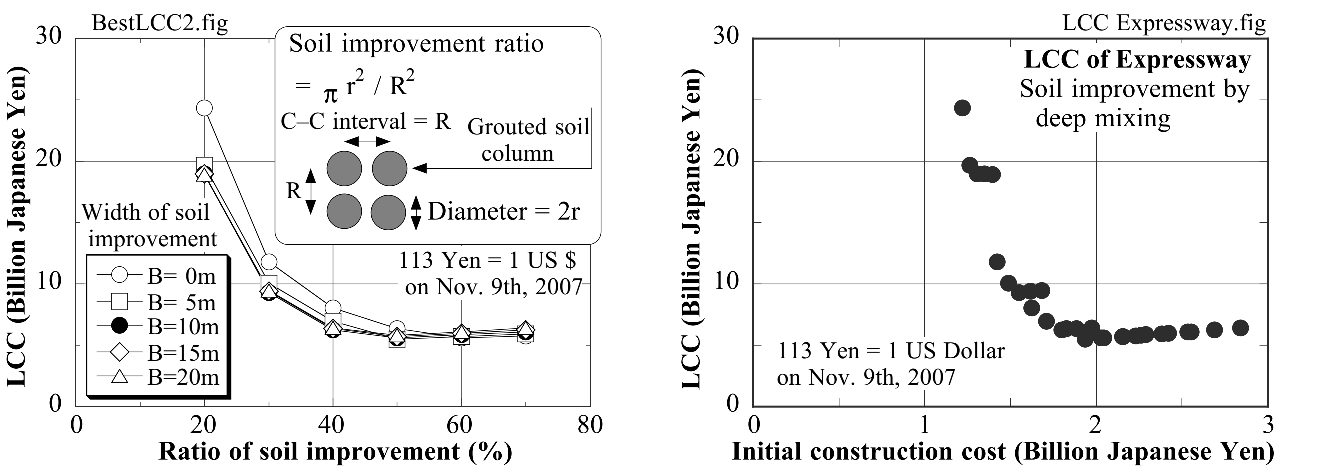


Fig. 14.42 Minimum LCC of expressway embankment

Note here that the aforementioned economic cost (negative effects to economic activity) is not considered explicitly here. This is because of an advice from a socioeconomic specialist that this cost is already and somehow taken into account by the missing toll fee cost. Although there may be different opinions on this issue, there is no unanimous idea at this moment.

Another point in the list of considered cost is that maintenance cost is not included. Although the authors expected that maintenance cost is reduced by a better construction effort, the reality was found different. Interviews with officers of government and other authorities revealed that maintenance of embankment consists mainly of regular cleaning and cutting grasses, which are not affected by the quality of construction. This implies that the maintenance cost is constant, independent of the construction cost. Hence, maintenance was removed from further study.

There are many uncertainties in geotechnical evaluation of LCC. Firstly, geotechnical materials do not decay with time in such a manner as steel and concrete. Therefore, it is difficult to define explicitly the life of a geotechnical structure. For example, there are many ancient earth dams (irrigation reservoirs) which are still in use (Fig. 14.41). After discussion, the life of geotechnical structures was set equal to 80 years, similar to practice in other types to structures. The second question concerned the cost of human life, which was an extremely difficult and sensitive issue. Certainly there are a wide range of opinions and practices in evaluation of human life in monetary units, namely missing income or annual economic products. It may be advisable to refer to travel insurance in which the loss of life due to accidents is evaluated in terms of money. In case of the author, the maximum insurance money is 100 million Japanese Yen in 2006.

Table 14.6 Comparison of conventional seismic design and LCC-based design

	Conventional design	LCC design
Soil improvement	CDM	CDM
B	0 m	10 m
Ratio of improvement	30%	40%
Seismic safety factor	1.12	1.58
Probability of damage (subside. 0.15 m) / year	5.02×10^{-3}	2.30×10^{-3}
Initial construction cost	1.42	1.80
Seismic risk	10.36	4.47
LCC	11.78	6.27

Unit of cost : Billion Japanese Yen, 113 Yen = 1 US $ on Nov 9th, 2007

The intensity of future earthquakes were given in a probabilistic manner. Many earthquake records were put in an embankment model (Fig. 14.37) and the subsidence, S, was calculated by repeating the Newmark rigid block analogy (Sect. 12.1). Finally, the seismic costs were evaluated by substituting many S values in the empirical formulae (Fig. 14.40) in a probabilistic manner and added together. Thus, the conducted analysis was a Monte Carlo probabilistic simulation.

The variation of LCC with the quality of soil improvement is illustrated in Fig. 14.42. The better quality of deep mixing (grouting for solidification of soft soil, Sect. 26.13) increases the initial construction cost but reduces the subsidence (S) and the seismic cost. Consequently, the optimum (least) LCC was achieved by the B value of 10 m and the soil improvement ratio = 50%. Further efforts of soil improvement do not reduce LCC anymore.

Table 14.6 compares details of the road embankment designed either by a conventional approach (seismic factor of safety = 1.12) or the LCC approach (mean factor of safety = 1.58, but varying probabilistically). Although the latter requires more initial construction cost, the overall cost in its life cycle is smaller.

14.8 Restoration of Damaged Fill Resting on Soft Soil

Fig. 14.43 Failed shape of Itoizawa road embankment in 1993

Fig. 14.44 Small creek at the place of embankment failure (Photo taken by T. Honda in May, 2000)

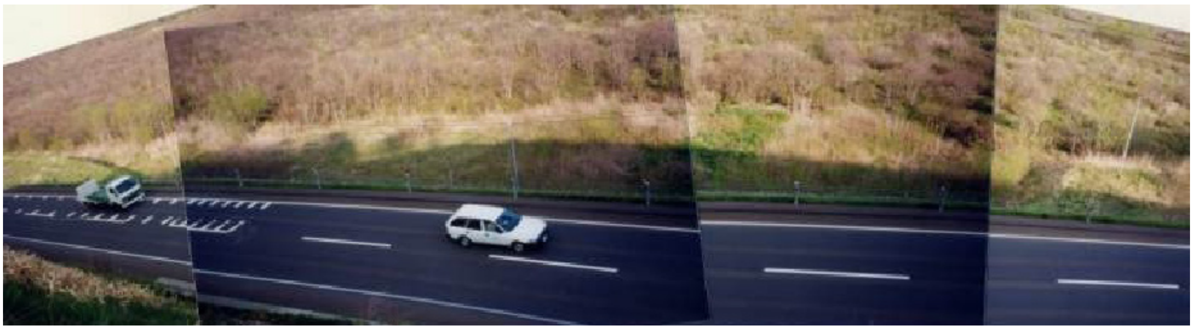


Fig. 14.45 Overall view of Itoizawa embankment in year 2000 (photo by T. Honda)

The 1993 Kushiro-Oki earthquake triggered a failure of an important road embankment at Itoizawa site (糸魚沢) between Akkeshi (厚岸) and Nemuro (根室) in Hokkaido. This road was situated along the foot of hills which faced a marsh (swamp) of soft stream deposit. Figure 14.43 indicates the failed shape of the road embankment. This failure occurred at a place where the road crossed the exit of a small stream into the marsh (Fig. 14.44). Thus, an embankment resting on soft soil collected much water from a valley behind and failed easily upon shaking. The slope stability of the embankment had been improved before the quake by placing berms (腹付け盛土 on left and right sides of Fig. 14.45) at the bottom of the fill. This was however insufficient on the embankment resting on softer subsoil.

Fig. 14.46 Repaired toe of road embankment in Itoizawa (photo by T. Honda)

The damaged fill was restored by constructing reinforced earth fill (soil with geogrids, plastic sheets, geotextiles, etc) while installing more water drainage pipes and replacing the base soft soil by gravels. Gravel gabions were placed at the toe of the slope for better shear strength and drainage (Fig. 14.46).

Note that the slope toe is the place where stress concentrates and shear strain becomes greater than in other part of the fill. Together with the effects of drainage from inside the new embankment, consequently, no damage occurred here upon the 1994 Hokkaido-Toho-Oki earthquake.

In addition to reinforcement, the type of fill materials has to be considered. Asada (2005) compared SPT-*N* values in residential fills which were affected during the 1978 Miyagiken-oki earthquake around Sendai City. While *N* values exhibit substantial variation within each site, the mean value in Fig. 14.47 clearly decreases with years after completion of filling. This infers that the filled geomaterials were affected by ground water and disintegrated with time (slaking of such materials as mudstone). This is in contrast with a common idea that soil increases its rigidity with time (ageing: Sect. 10.12).

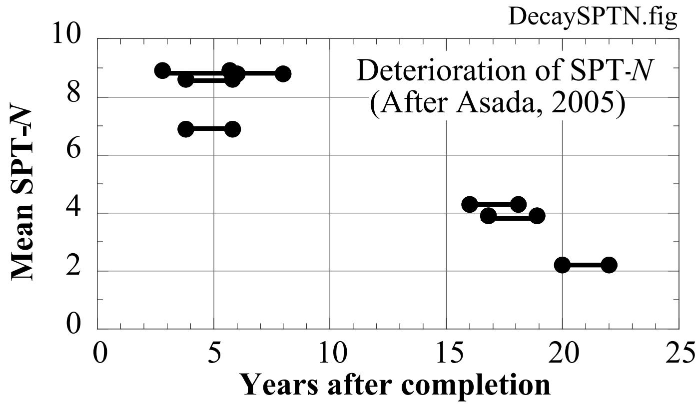


Fig. 14.47 Deterioration of fill material with time (after Asada, 2005)

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