Geotechnical Engineering for Disaster Mitigation and Rehabilitation

Proceedings of the 2nd International Conference GEDMAR08, Nanjing, China
30 May – 2 June, 2008
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Editors:

Hanlong Liu
An Deng
Jian Chu

Sponsored by National Natural Science Foundation of China
Books Series of National Key Subject in Geotechnical Engineering
PREFACE

This 2nd International Conference on Geotechnical Engineering for Disaster Mitigation and Rehabilitation (GEDMAR08), held at the Hohai University, Nanjing, China from 30 May to 2 June, 2008, is one of the activities of International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee TC39: Geotechnical Engineering for Coastal Disaster Mitigation and Rehabilitation. It is organized jointly by Hohai University, Chinese Institution of Soil Mechanics and Geotechnical Engineering (CCES), and Chinese Society of Environmental Geotechnics (CSRME) under the support of TC39, TC4 on Earthquake Geotechnical Engineering and Associated Problems, and the Joint Working Group on Geotechnical Engineering for Disaster Mitigation and Rehabilitation (JWG-DMR). This conference is the second in the series. The first conference was held at the Furama Riverfront Hotel in Singapore from 12-13 December 2005.

One hundred and forty-four papers from 20 countries and regions are included in this Proceedings. The papers were selected from more than 200 abstract submissions after a rigorous review process. This Proceedings contains 7 keynote and special invited plenary lectures written by international renowned experts and 18 special session and invited papers that reflect the special topics discussed in this conference. Not all the keynote or invited papers are included in this Proceedings due to various constraints. The other 119 papers cover a range of topics including disasters related to earthquake, landslide, soil dynamics, risk assessment and management, slopes, disaster mitigation and rehabilitation and others.

It is hoped that this Proceedings will be a useful source of reference to geotechnical engineers and professionals in other disaster related fields.

Editors

H.L. Liu, A. Deng and J. Chu
ACKNOWLEDGEMENTS

The Editors gratefully acknowledge the significant contributions from the following people and organizations:

- Staff of GeoHohai and Hohai University for their support in organizing this conference;
- Chinese Institution of Soil Mechanics and Geotechnical Engineering (CCES) and Chinese Society of Environmental Geotechnics (CSRME) for co-hosting the conference;
- Members of Conference Steering Committee, members of ISSMGE TC39: Geotechnical Engineering for Coastal Disaster Mitigation and Rehabilitation, members of ISSMGE TC4: Earthquake Geotechnical Engineering and Associated Problems, in particular, Professor Takaji Kokusho, Chair of TC4, and members of the Joint Working Group on Geotechnical Engineering for Disaster Mitigation and Rehabilitation (JWG-DMR) for their support to the organization of this conference;
- Members of the International Advisory Committee, in particular, Professor P. S. Séco e Pinto, President of ISSMGE, Professor M. R. Madhav, Vice-President for Asia, ISSMGE for their support and advice, and Prof. Jie Han, The University of Kansas, USA, for his advice and help on conference organization;
- Keynote lecturers and invited special plenary lecturers;
- Special session organizers, special session speakers, invited speakers and panelists;
- Sponsors, National Natural Science Foundation of China and Hohai University, for their generous sponsorship;
- Supporting staff and students to the conference, in particular, Mr Liang Chen.
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FLOW SLIDES OF UNDERWATER SAND DEPOSITS
IN JAMUNA RIVER BED

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When excavation was under progress by dredging through the sandbar deposit in Jamuna River in Bangladesh, a number of slips occurred underwater. Features of the slips are first described herein together with the results of in-situ investigations on the ground conditions. In the fluvial deposit in the Jamuna riverbed the sand is known to contain several percent of mica mineral composed of plate-shaped grains. The inclusion of mica has been known to make the sand behave more strain-softening leading to increased vulnerability to flow type deformation. This was conceived to have been the seminal cause of the underwater slides. To confirm this aspect, the sand was recovered from the river site and triaxial tests were performed in the laboratory extensively. The outcome of the tests was compiled and arranged in a manner where the residual strength could be evaluated in a general framework of interpretation on sand behaviour. The results of the tests showed that the mica-containing sand from Jamuna River site exhibited contractive or strain-softening behaviour over a wide range of void ratio. The residual strength at the steady-state deformation obtained in the present test scheme was used to provide an explanation for the flow-type instability of the slopes in the light of what actually happened during the underwater excavation in Jamuna River.

INTRODUCTION

The vast expanse of the flood plain in Jamuna River in Bangladesh has experienced volatile shifts of river course during the flood period. In a large project to construct a long bridge, it was considered necessary to protect the abutment of the bridge from scouring and erosion in which the level of the riverbed is purported to change by more than 10m overnight. In order to provide a countermeasure, excavation of underwater channels was executed by dredging the sand deposit by ships through the sandbar area. The aim was to reinforce the underwater slope with stones and geotextiles on the side of abutment. During the excavation, a number of slides took place underwater thereby inhibiting the operation of construction. In-situ investigations were carried out extensively by Dutch company and consultants and original design modified.

To clarify the cause of the slides, various kinds of investigations were carried out both in the field and in the laboratory. Although the causes were variously speculated, one of the
points unanimously recognized was the fact that the fluvial sand deposit in Jamuna River contains several percents of mica mineral exhibiting peculiar behaviour of deformation. In recognition of this, the extensive studies were performed in the laboratory by Hight et al. (1999) and various factors such as anisotropic mode of deposition were addressed as possible seminal cause leading to highly collapsible nature of the mica-containing sand. At the Tokyo University of Science, the scheme of laboratory studies had been underway to clarify the steady-state deformation characteristics of sandy soils. Some of the results of tests were reported by Ishihara et al. (2003).

With an aim to examine behaviour of Jamuna River sand in terms of the framework established as above, a large amount of Jamuna River sand was shipped to Japan and multiple series of triaxial tests were performed to clarify the deformation characteristics of the sand. The results of the undrained triaxial compression and extension tests will be introduced in this paper within the framework of data arrangements and interpretation established thus far.

As a result of the study, it was pointed out that the use of the major principal stress would be most appropriate to take into account effects of confinement on the residual strength of sand, and if based on this, there is no need to consider the effect of various $K_c$-conditions at the initial stage of anisotropic consolidation. The outcome of these laboratory tests was incorporated into a simple analysis to examine the instability of the slopes observed in Jamuna River. The analysis targeted for the post-failure conditions was performed based on the residual strength. The consequence of these studies will be described in the following pages. This paper is a modified version of the paper with the same content which was published previously by Ishihara and Tsukamoto in 2007.

**PROJECT**

In the middle reaches of the Jamuna River in Bangladesh about 110km northwest of Dhaka, a 4.8km-long bridge called Bangabandhu Bridge connecting the towns of Sirajganj and Bhuapur was planned and constructed in 1995-1999. Its location is shown in Figure 1. The Jamuna is a shifting braided river, consisting of numerous channels which change their width and course significantly with reasons. Thus, training the river to ensure that it would continue to flow under the bridge corridor was the most difficult technical challenge of the project.

As shown in the more detailed map in Figure 2, the width of the river channel was about 11km. This area is the vast expanse of flood plain and had suffered severe destruction over the years due to intense flooding over the river channel and its surroundings. Devastation was particularly conspicuous at the time of the flooding in 1987 and 1988. In some areas, river channels are purported to have shifted their courses overnight through several hundred meters. The tendency of the drift is reported to have been westwards whereby involving a huge amount of sandy soils removed by scouring in the riverbed in the west side of the Jamuna River.
Figure 1. General location map

Figure 2. Locations of the Guide Bunds in Jamuna bridge site
In the design of the abutment of the Bangabandhu Bridge, it was considered mandatory to implement some countermeasures against the deleterious effects due to such scouring and to duly control the river channel. With this aim, construction of guide bunds was planned on both sides of the river as shown in Figure 2. Of particular importance was the construction of the West Guide Band, as it was intended to protect the bridge abutment behind the river from scouring or erosion of the riverbed. The construction consisted of excavating the riverbed by dredging the sand by ships and placing erosion-protecting armors such as geotextiles and stones over the underwater slopes on the west side. A typical cross section with an armored slope is shown in Figure 3. The location and horseshoe-shaped plan view of the Guide Bunds are displayed in Figure 4. The trench varying from 22 to 30m in depth was dug below water by means of cutter-suction dredgers which were operated from ships at the site of each guide bund.

![Cross section for the dredging of West Guide Band for the bridge at Jamuna River site](image)

**Figure 3. Cross section for the dredging of West Guide Band for the bridge at Jamuna River site**

**UNDERWATER SLIPS ON EXCAVATED SLOPES**

The West Guide Bund was constructed at the site of a recently formed sand island as seen in Figure 2. The materials forming the dredged slopes were composed of young, rapidly deposited sediments. The detailed plan view of the excavation is shown in Figure 4 and a typical section (E-W section) across the dredged channel is displayed in Figure 5.

The slope on the west side was to be protected by the geotextiles-stone armor against the scouring, because the bridge abutment was to be installed due west of the West Guide Bund. Thus the underwater slope on the west side designated as “permanent slope” was designed so as to have a gentle slope of 1:5.0 in the middle portion. On the contrary, the slope on the east side of the dredged channel was to be left unprotected. Even though slides occur and the sand bar disappears in future due to scouring or erosion, it was considered it did not matter. Thus, the eastern slope was designed to form a steeper slope with an angle of 1:3.0 and designated as “temporary slope” in Figure 5.
The dredging work began northwards in October 1995 from the southern rim of the sand bar. As the dredging proceeded, slope failures occurred on the permanent slope on November 19th in 1995 in the cross section 1270 and another on November 22nd in the cross section.
They are respectively called W1 and W2 slide as shown in Figure 4. On December 3, 1995, the largest-in-scale slide denoted by W3 took place on the permanent slope at a location of Chainage 1550. This slide covered an area of about 150m wide and 150m long over the permanent slope. Afterwards many failures were found to have occurred on the temporary slope during the period of rising water level in 1996. Eleven of them were larger in scale than W3, and these are indicated by the symbol WT 6, 7, 9, 9E, 13, 15, 16, 17, 18, 22, 24 in Figure 4. Many of these slides delayed the progress of construction works, but of most serious concern were the failures on the permanent slope, because they had to be repaired to construct an erosion-free slope. In recognition of the instability with an angle of 1:3.5, the design cross section on the permanent slope was changed so as to have a slope of 1:6.0 near the bottom and for the temporary slope, the angle was changed from 1:3.0 to 1:5.0 as illustrated in Figure 5. Then, the dredging work to full depth was resumed to finish excavation of the trench. As the dredging went on, a number of slope failures began to occur again but only on the temporary slope. The failures occurred mostly during the period of March to June in 1996. The exact locations of all of these slope failures including those prior to and after the design change as well are indicated together in Figure 4.

CAUSES OF SLOPE FAILURES

There are two aspects to be distinguished in elucidating mechanisms of slope failures in sand deposits. These are the seminal cause and the consequence of failures.

Causative incidents

It was difficult to precisely identify generic causes of the slides which occurred apace in underwater environments. It was envisioned that the over cutting, overstepping or rapid cutting associated with the dredging operation had been responsible for triggering the slips. There were slow falls in the water level of the river after storms of the order of 0.1m per day over a period of five days. There might have been other seminal causes leading to the slips. The factors such as wave actions and thunderstorms were also suspected to have triggered the slips. No matter what causes might have been, it is certain that the sand deposits had been in a precarious state narrowly keeping the stability when the excavation was made.
Consequences of failure

After the initial failure is triggered involving a small or medium deformation, the soil may or may not develop large displacement later on. One kind of sand deposits might induce only limited deformation which is tolerable, but in another type a fairly large amount of displacement will continue further on. In the latter case, the level of devastation incurred will be intolerably large. Thus, identification of the damage level in terms of continuation or discontinuation of deformation after triggering of the failure will pose an important aspect in recognizing the feature of the problem particularly in saturated sand deposits under water.

The identification of the consequence of failure as above can be made generally by examining the state of an existing deposit as to whether it will exhibit contractive or dilative behaviour after it has undergone triggering. In the case of the sand deposit in the Jamuna riverbed, the deposit seems to have had characteristics showing the contractive or strain-softening type of behaviour in which the residual strength at a largely deformed state is reduced significantly leading to an intolerable level of deformation after the slips were triggered. In the above context, soil characteristics were investigated in details in the field as well as in the laboratory as described below.

INVESTIGATION OF SOIL CONDITIONS

Following the occurrence of the slides as well as at the time the design was made, multiple series of tests were conducted both in the field and in the laboratory to elucidate nature and properties of soils which are deemed to be a cause of the flow type slides.

In-situ tests

Deep borings were performed at three locations, viz., B1 on the west bank, and B2 and B3 on the east bank, as shown in Figure 6. The standard penetration test (SPT) was also conducted at these boring sites. At the site B3, the measured N-value was, for example, 15 at a depth of 9.25m. Considering the use of a free falling hammer in the SPT practice at the Jamuna River sites, the energy level is deemed as about 80% of the theoretical energy in hammer dropping. This energy level actually consumed for penetration is considered about the same as the level normally achieved in the Japanese practice. The SPT N-value at a shallow depth of 6.25m was found to be 7 at B3, 10 at B2 and 19 at the site B1. These values are relatively small indicating the presence of loose sand layer which might be responsible for triggering the slope failure.

Dutch one-penetration tests (CPT) were also performed at the stage of feasibility study and design at 15 locations as indicated by C1 to C13 shown in Figure 6. The results of CPT showed q_c-values of 4-5MPa at depths from 6-8m at the location CDI which is close to the site B3. This value indicated as well the presence of a loose sand layer at this depth. After the failures, an additional set of CPT was carried out at the shoulders of the excavation as shown in Figure 7. The results of the CPT are reported by Yoshimine et al. (2001) as displayed in Figure 8 in terms of the q_c-value and sleeve friction ratio F. It may be seen that the q_c-value at the depth of 10m takes values ranging between 5-12MPa, indicating that the sand is in loose states of deposition. By comparing the SPT N-value and the CPT q_c-value obtained each in their vicinity, an empirical correlation was established by Delft Geotechnics as follows,
Figure 6. Location of soil investigations at Jamuna multipurpose bridge construction

Figure 7. Location of the slips by arrows and cone penetration tests (CPT) (from Yoshimine et al., 2001)
$q_c = 0.31 \cdot N_{60}$ \hspace{1cm} (1)

where $N_{60}$ indicates the SPT N-value corresponding to 60% of the theoretical energy. In view of the 80% of the energy achieved in the Jamuna River investigation, the above relation would be rendered to,

$q_c = 0.37 \cdot N_{80}$ \hspace{1cm} (2)

where $N_{80}$ indicates SPT N-value obtained by the free fall hammer which is considered to exert 80% of the theoretical energy in the SPT operation. The relative density at the site was estimated based on the data of SPT and CPT. One of the typical data by Hight et al. (1999) is shown in Figure 9, where it may be seen that the relative density takes values around 50%, but the majority of data indicate values less than 65%.

Figure 8. Results of CPT along the shoulder of the dredging in the West Guide Band (After Yoshimine et al., 2001)

Figure 9. A typical profile of relative density at a site in Jamuna River
Laboratory tests

Sand samples were obtained in-situ by means of the tube sampling technique. However, the degree of disturbance appears to be somewhat high in loose deposits of sand and therefore the outcome of the laboratory triaxial tests on such samples is considered not truly representative of the conditions of sand deposits prevailing in the field.

Apart from the laboratory tests on intact sand samples, it was discovered that the sand in the Jamuna River contains several percent of mica and its presence was suspected to have created conditions of the deposits which were highly vulnerable to triggering failure and consequent flowage of the sand. The distribution of mica content versus depth is shown in Figure 10. Thus, attention has been drawn to the peculiar behaviour of mica-containing sand by several investigators. One of the results of simple shear tests reported by Hight et al. (1999) is reproduced in Figure 11, in which the behaviour of clean silica sand is compared with that of the same sand but containing 1% mica.

Figure 10. Distribution of mica content at sites of the Jamuna River
Figure 11. Effect of 1% mica on the undrained behaviour of sand in simple shear (Hight et al., 1999)

It can be seen that the loose sand with a void ratio of $e=0.70$ exhibits ductile behaviour with a tendency to dilate at largely strained states, bringing up the effective stress path along the failure line. In stark contrast, the sand with $e = 0.74$ containing 1% mica is shown to be brittle and have a potential to collapse at medium strains resulting in a small residual strength at a largely strained state. Hight et al. (1999) argued that, because of the aspect ratio of the mica plate as high as approximately 50:1 compared to the rotund sand particles, the presence of even 1% mica by weights is approximately equivalent to that of 25% of mica by number of grains.

In an effort to investigate more thoroughly the behaviour of mica-containing sand, multiple series of triaxial tests have been conducted at the Tokyo University of Science on reconstituted samples of the sand recovered from the Bangabandhu Bridge site in Bangladesh. The details of the tests procedures and the manner in which data are arranged are described in more detail in the paper by Tsukamoto et al. (2007). The conduct of the tests on the Jamuna River sand and its outcome will be described somewhat in details in the following pages.

CONDUCT OF TRIAXIAL TESTS ON BANGLADESH SAND

Material properties

The sand with 4.5% to 5% percents mica recovered from the bridge site has a typical grain size distribution curve shown in Figure 12. It is a sand with $D_{50} = 0.2$ mm and contains 10% non-plastic fines. Its specific gravity is $G_s = 2.745$ and the maximum and minimum void ratios as measured by the Standard of the Japanese Geotechnical Society were $e_{\text{max}} = 1.202$ and $e_{\text{min}} = 0.602$, respectively.
Test procedures

In the triaxial tests, the samples with 6cm in diameter and 12cm in length were prepared by the method of wet tamping, in which moist sand was placed in the mould with varying energy of compaction. By this method, it was possible to prepare reconstituted samples with a widely varying range in void ratio. After preparing the samples, de-aired water was circulated to achieve a state of full saturation with a B-value greater than 0.95. Then, the samples were consolidated anisotropically under different $K_c$-conditions. The axial load was then increased under undrained condition, when the mode of the test was triaxial compression. The triaxial extension tests were also performed by decreasing the cell pressure under undrained conditions.

Results of tests

The results of the undrained compression test on samples with void ratios ranging between 0.804 and 0.871 are displayed in Figure 13, where the deviator stress $q = \sigma_1 - \sigma_3$ is plotted versus the effective mean confining stress defined as $\sigma' = (\sigma_1 + 2\sigma_3)/3$. The saturated samples were consolidated with a vertical stress of $\sigma'_v = 98kPa$ and a lateral stress of $\sigma'_h = 49kPa$ producing an initial state of $K_c=0.5$. It may be seen in Figure 13 that the dilatancy behaviour is exhibited when the sample is prepared with a void ratio less than about 0.83, but otherwise the sample is contractive. It is to be noticed that the sample with $e=0.871$ has reached a steady-state with a deviator stress of $q = 15kPa$, which is smaller than the initially applied deviator stress of $q = 49kPa$. It is seen in Figure 13(b) that a large deformation began to occur at an early stage of load application and continues further on until an axial strain of 20% developed.

The smallness of the deviator stress at the steady state as compared to the deviator stress at the outset would be regarded as a criterion to indicate an unstable condition where flow-type deformation could be triggered if the peak shear stress is passed over by application of a slight agitation at the beginning. Another series of tests with the same initial lateral stress of
\[ \sigma'_{1c} = 49 \text{kPa} \] but with an increased \( K_c \)-value of 0.7 is demonstrated in Figure 14 for samples with various void ratios where the general tendency is seen to be the same as those shown in Figure 13. It may be seen in Figure 14 that the sample consolidated with a deviator stress \( q_c = \sigma'_{1c} - \sigma'_{3c} = 20 \text{kPa} \) with a void ratio of 0.827 has reached a steady-state with the deviator stress \( q = 95 \text{kPa} \) which is much greater than the initially applied deviator stress of 20kPa. In such a condition, the flow type deformation will not be induced because of the gain in shear strength as compared to the initially applied shear stress. The last series of the tests with \( K_c = 1.0 \) are demonstrated in Figure 15 where it is apparently noted that the specimen with \( e = 0.767 \) exhibited highly dilative behaviour. The characteristic feature of deformation as deduced from the above test results may be summed up as follows.

1. It is apparent that the deformation characteristics become more dilative with decreasing void ratio and vice versa. The void ratio for the threshold can be read off as approximately \( e = 0.83 \) from Figures 13 to 15.

2. Although not clearly displayed in the test results shown in Figures 13 and 14, a number of tests on Jamuna River sand with greater confining stresses or tests on other sands have
coherently shown that the deformation characteristics become more contractive with increased $K_c$-value at the stage of anisotropic consolidation. This is consistent with the main conclusion derived by Chern (1995), Vaid and Chern (1985), Kato et al. (1999) and Tsukamoto et al. (2007).

3. Similar undrained triaxial tests have been performed on several other sands in Japan which do not contain mica. The outcome of the tests on such non-mica sand showed that the sand behaviour becomes more starkly dilative or strain hardening with decrease in void ratio. In contrast, the mica-containing sand from Jamuna River tends to become contractive or to remain at an intermediate state between contractive and dilative whereby developing large deformation at constant shear stress $q_s$ and effective confining stress $p_\sigma'$. This implies that the mica-containing sand would easily be put into a steady-state over a wide range of void ratio. This is consistent with the results of the tests reported by Hight et al. (1999).

FLOW CHARACTERISTICS OF JAMUNA RIVER SAND

From a number of other tests including those shown in Figure 13 through 15, the void ratio $e$, deviator stress $q_s$ and effective mean confining stress $p_\sigma'$ at the steady state were read off and plotted in Figure 16 in terms of $e$ versus $p_\sigma'$. In this plot, those data from the tests with $K_c=1.0$ are displayed with white circles and those from anisotropically consolidated samples with $K_c$-values less than 1.0 are shown with black circles. It can be seen in Figure 16 that the steady state line is established uniquely irrespective of the $K_c$-conditions.

In the whole program of the tests executed at the Tokyo University of Science, it was shown, however, that the most appropriate parameter indicative of the effects of confinement at the steady state is not necessarily the mean principal stress $p_\sigma'=(\sigma_1'+2\sigma_3')/3$, but instead the major principal stress $\sigma_1'$ alone. In accordance with this concept, the plot is made of the same data set as alternatively displayed in Figure 17 in terms of the void ratio.
versus the effective major principal stress $\sigma'_1$, at the steady state. It can be seen that the steady-state line is established equally well again irrespective of $K_c$-conditions.

![Figure 16. Steady-state line for Jamuna River sand as a plot of void ratio and effective mean principal stress](image)

![Figure 17. Steady-state line for Jamuna River sand as plots of void ratio and effective major principal stress](image)

It is to be recalled that the steady-state lines established in Figures 16 and 17 all do pertain, not to the initial un-deformed state of the sand, but to the state where the sample is largely deformed to a strain level greater than about 5%. One might argue that the specification of a state of soil at the time it is already deformed may not be appropriate. Thus, it is claimed to be a parameter at the initial state prior to load application that is more convenient and hence desirable in view of its practical application when soil behaviour is to be estimated for subsequent application of shear stress. Looking back over the development of soil mechanics, one can realize that, in the majority of problems, such as consolidation and undrained shear strength, it was the initial state of the confining stress that was taken up as a parameter to specify the soil behaviour in the subsequent loading. In the context as above, the initial state of void ratio and the major principal stress $\sigma'_1$ before the application of shear stress were
picked up from the whole file of test data on Jamuna River sand and they are plotted in the diagram of Figure 18. In this diagram, a line is drawn which differentiates the sample behaviour between contractive and dilative. Such a line was called the Initial Dividing Line (Ishihara, 1996). It may be seen in Figure 18 that the Initial Dividing Line (IDL) can be obtained consistently irrespective of the $K_v$-state of samples at the time of anisotropic consolidation. It is to be noticed here that the steady-state line, quasi-steady state line and initial dividing line are almost coincident particularly in the range of a small confining stresses less than about $\sigma_{1c}=100$kPa. The angle of internal friction at the steady state derived from the same data is shown in Figure 19. Although the values are slightly different between the triaxial compression test (TC-test) and triaxial extension test (TE-test), the average value would be taken as $\phi_s=30^\circ$.

![Figure 18. Initial dividing line for Jamuna River sand as plots of void ratio and initial major principal stress](image1.png)

![Figure 19. Angle of internal friction for the steady state](image2.png)

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RESIDUAL STRENGTH OF JAMUNA RIVER SAND

It has been customary to define the residual strength, \( S_u \), by referring to the minimum shear stress at the quasi-steady state (QSS) which is mobilized at the state of phase transformation for sands exhibiting contractive behaviour. In the mica-containing Jamuna River sand, the test data in Figures 13 to 15 show that the quasi-steady state (QSS) is almost coincident with the steady state (SS). Thus, these two states could be regarded practically identical for the mica-containing Jamuna River sand being considered. By denoting the deviator stress at this state as \( q_s = \sigma_{1s}' - \sigma_{3s}' \), the residual strength is expressed as (Ishihara, 1996).

\[
S_u = \frac{q_s}{2} \cos \phi \cdot \frac{M}{2} \cos \phi \cdot p_s', \quad M = \frac{6 \sin \phi}{3 - \sin \phi} \quad (3)
\]

where \( p_s' \) is the mean effective confining stress at the QSS or SS as defined by \( p_s' = (\sigma_{1s}' + 2 \sigma_{3s}')/3 \), \( M \) is a parameter related to the angle of phase transformation in the \( p' - q \) plot, and \( \phi \) is the angle of internal friction at QSS or SS. When normalizing the residual strength \( S_u \) to the initial mean principal stress, it is written as

\[
\frac{S_u}{p_c'} = \frac{M}{2} \cos \phi \cdot \frac{1}{r_c}, \quad \text{where} \quad r_c = \frac{p_c'}{\sigma_{1c}'} = \frac{\sigma_{1c}'+2\sigma_{2c}'}{\sigma_{1c}'+2\sigma_{3c}'} \quad (4)
\]

If the residual strength is normalized to the initial major principal stress \( \sigma_{1c}' \), it is written as

\[
\frac{S_u}{\sigma_{1c}'} = \frac{M}{2} \cdot \frac{3 - \sin \phi}{3(1 + \sin \phi)} \cdot \cos \phi \cdot \frac{1}{r_c}, \quad r_c = \frac{\sigma_{1c}'}{\sigma_{1c}'} \quad (5)
\]

From the majority of data on undrained triaxial compression tests “TC-test”, the residual stress at largely deformed state of the Jamuna River sand was read off and normalized to \( p_c' \) and \( \sigma_{1c}' \). Similar undrained triaxial tests were also performed in the extension mode of deformation which is referred to as “TE-test”. The residual strength from the TE-test was also normalized to \( p_c' \) and \( \sigma_{1c}' \). The outcome of such a data compilation is demonstrated in Figure 20, in terms of the plot of the normalized residual strength versus the \( K_c \)-values employed in each of the tests. Among the cluster of data points, the initial states of the samples exhibiting the contractive behaviour in subsequent undrained loading are indicated by black symbols and those showing dilative response by white symbols.

It is to be noted that those data with smaller values of \( S_u/p_c' \) correspond to loosely prepared samples and with increasing values of \( S_u/p_c' \), the samples become denser. A line dividing the two types of behaviour is drawn for each of the TC-tests and TE-tests. The two lines thus determined each from TC and TE tests are indicated in Figure 20, together with the empirical equations suggested for each of these threshold conditions. The same data sets are plotted in Figure 21 now choosing the residual strength normalized to the initial major principal stress \( \sigma_{1c}' \). From the two kinds of plot shown in Figures 20 and 21, the following observation can be made.

1. In the case when the residual strength is normalized to the initial mean confining stress, the value of \( S_u/p_c' \) tends to increase with decreasing \( K_c \)-value, and also it depends upon the mode of deformation as to whether the loading is triaxial compression or triaxial extension.
2. If the residual strength is normalized to the initial major principal stress, the value of \( \frac{S_u}{\sigma'_{1c}} \) is determined uniquely irrespective of the \( K_c \)-value in the anisotropic consolidation, and also independently of whether the mode of deformation is triaxial compression or triaxial extension. For the mica-containing sand from the Jamuna River site, the threshold value of the normalized residual strength separating conditions of contractive and dilative behaviour is found to be \( \frac{S_u}{\sigma'_{1c}} = 0.26 \) as accordingly indicated in Figure 21.

3. Each data in Figure 21 belongs to the samples having different void ratios but a close examination of the data has shown, although not shown explicitly in the figure, that for the state with a given void ratio, the normalized residual strength \( \frac{S_u}{\sigma'_{1c}} \) takes a constant value irrespective of the \( K_c \)-condition at the time of consolidation. Thus, the limiting value of \( \frac{S_u}{\sigma'_{1c}} = 0.26 \) can be taken as the upper bound of \( \frac{S_u}{\sigma'_{1c}} \)-values within the range of the void ratio in which the sand exhibits contractive behaviour.
In the type of plots shown in Figures 20 and 21, the looseness or denseness of the samples is not indicated explicitly in the figures. In order to visualize this effect, the same test data are now shown in Figure 22 in terms of the \( S_{\sigma_0}/\sigma'_c \) plotted versus the void ratio, \( e \). It may be seen in the figure that effects of the minor principal stress, that is, \( \sigma'_c \)-value in the TC-test and \( \sigma'_c \)-value in the TE-test, are not so important but the influence of the mode of deformation as to whether it is TC-test or TE-test would be somewhat significant with the mode of the TE-test giving smaller normalized residual strength \( S_{\sigma_0}/\sigma'_c \) as compared to that from the TC-tests. It is to be noticed that no matter whether the mode is TC-test or TE-test, the upper limit of \( S_{\sigma_0}/\sigma'_c \) takes the identical value of 0.26. The density of each samples tested is also shown by way of the relative density, \( Dr \), with the scale indicated on the right hand side of Figure 22. If the average is taken, it can be conclusively mentioned that, for the Jamuna River sand deposited with a relative density smaller than 65\%, it will exhibit contractive behaviour. Therefore the in-situ deposits under such condition will have a potential to develop flow type deformation irrespective of the \( K_c \)-condition, if it is subjected to an external agency for triggering the slide.

\[
\sigma_a = \frac{N}{l} = \gamma' H \cos^2 \alpha \quad \tau_a = \frac{S}{l} = \gamma' H \sin \alpha \cdot \cos \alpha
\]  

\[ (6) \]  

**Figure 22.** Residual strength ratio \( S_{\sigma_0}/\sigma'_c \) versus void ratio-comparison between triaxial compression and extension tests

**SIMPLE METHOD OF ANALYSIS OF THE SLIDE AT JAMUNA RIVER**

**Basic concept**

For the sake of simplicity, let a potential sliding plane be located in parallel to the surface of the submerged slope as illustrated in Figure 23. Then, from the equilibrium of forces amongst the submerged weight of a soil mass and normal and tangential forces \( N \) and \( S \) acting on the potential sliding plane, the stresses \( \sigma_a \) and \( \tau_a \) are obtained as

\[
\sigma_a = \frac{N}{l} = \gamma' H \cos^2 \alpha \quad \tau_a = \frac{S}{l} = \gamma' H \sin \alpha \cdot \cos \alpha
\]

\[ (6) \]
where $\gamma'$ is the submerged unit weight of the soil, $\alpha$ is the angle of the potential sliding plane, and $H$ is the height of the soil mass being considered. Then, given the values of stress components, $\sigma_\alpha$ and $\tau_\alpha$, as above, it is possible to locate a point B in the Mohr diagram as illustrated in Figure 24. The direction of the line $OB$ indicates the angle of obliquity of stress application, $\alpha$, or the angle of stress mobilization. By drawing a half circle through the point B so that it is tangential to the line $OB$, it becomes possible to identify the points of the major and minor principal stresses $\sigma_1$ and $\sigma_3$ on the Mohr diagram. Then, from geometrical considerations, the following relations are obtained.

Figure 23. Forces acting on a soil element above a sliding plane in a submerged slope

Figure 24. Mohr circle to determine $\sigma_1$ and $\sigma_3$ from $\sigma_\alpha$ and $\tau_\alpha$

\[ \sigma_1 = \sigma_\alpha + (\tan \alpha + \frac{1}{\cos \alpha})\tau_\alpha, \quad \sigma_3 = \sigma_\alpha + (\tan \alpha - \frac{1}{\cos \alpha})\tau_\alpha \quad (7) \]

Introducing Eq. (6) into Eq. (7), one obtains

\[ \sigma_1 = \gamma'H(1 + \sin \alpha), \quad \sigma_3 = \gamma'H(1 - \sin \alpha) \quad (8) \]

Thus, the ratio between the minor and major principal stresses is obtained as

\[ K_r = \frac{\sigma_3}{\sigma_1} = \frac{1 - \sin \alpha}{1 + \sin \alpha} \quad (9) \]

The relation of Eq. (9) is displayed in Figure 25. It is known that the majority of natural slopes consisting of relatively soft soils have an angle ranging approximately between
\[ \alpha = 0 \text{ and } \alpha = 30^\circ. \] Thus, the ratio, \( K_c \), between the two principal stresses has a value between 0.3 and 1.0.

![Figure 25. Relation between \( K_c \)-value and angle of slope](image)

Typical pattern of deformation

The typical pattern of undrained deformation of anisotropically consolidated specimens is schematically illustrated in Figure 26 in terms of stress path and stress-strain curve. In Figure 26(a), the abscissa indicates the mean principal effective stress defined by

\[ p' = \frac{\sigma_1' + 2\sigma_3'}{3} \]

and the ordinate represents the shear stress defined by

\[ q = \sigma_1' - \sigma_3'. \]

In Figure 26, point A indicates an initial state of \( K_c \)-consolidation whereupon undrained shear stress application starts. When the specimen is loose, it shows an increase in deviator stress, \( q \), to a point B at peak strength and then a decrease down to a point C corresponding to the state of phase transformation or the quasi-steady state. The bent-over in the stress path takes place at point C and the shear stress increases to a point D where large deformation starts to occur without any change in the effective mean stress \( p' \) and shear stress \( q \). This state is called the steady state. Specimen is loose, the minimum deviator stress is encountered, concomitant with fairly large deformation, at point C where the phase transformation takes place from contractive to dilative behaviour.

Thus, the residual strength is defined by the shear stress \( q_{QS} \) which is mobilized at point C. The residual strength thus defined is called the strength at quasi-steady state. When the sand is very loose, the gain in the deviator stress from point C to D is not achieved. The sample continues to deform with a constant deviator stress. In other words, the quasi-steady state (QSS) becomes coincident with the steady state (SS). In the case of the Jamuna River sand, the samples are seen deforming at a constant deviator stress as apparent from the test data shown in Figures 13 through 15. This may be due to the presence of mica. Thus, in the present study, the quasi-study states will be taken as being identical to the steady state.

As touched upon in the foregoing section, the residual strength ratio can be established by normalizing the residual strength, \( S_{us} \), to the major principal stress \( \sigma_{1c} \) at consolidation, and the ratio \( S_{us} / \sigma_{1c} \) thus determined was found to be independent of the \( K_c \)-condition at the time of consolidation. In unison with this fact, it might be of use if the major principal stress \( \sigma_1' \) is taken, instead of \( p' \), as a variable to represent the state of confinement. The \( q - \sigma_1' \) and \( q - \varepsilon_1 \) curves in this context can be established as...
illustrated in Figure 27 in the fashion similar to those shown in Figure 26. It is to be noted that there is no essential difference between these two methods of representing the undrained behaviour of sand. Thus, from now on, the method of using $\sigma_1'$ as shown in Figure 27 will be adopted in the subsequent pages of this paper.

![Figure 26. Typical stress-path and stress-strain relation for loose sand](image)

![Figure 27. Stress-path and stress-strain diagrams in terms of $\sigma_1'$](image)

**Consideration for failure of underwater slopes in Jamuna River Bridge site**

*Sequence of scenarios to flow failure*

The series of event leading to the flow-type failure under water would be envisioned to consist of a sequence of events as illustrated in Figure 28.

1. The deposit under the level ground was excavated by dredgers to form a slope with an angle $\alpha$, as illustrated in Figure 28(b). At this stage, a soil element under water was brought to a state of anisotropic consolidation with a $\frac{K_c}{\sigma_1'}$ which is evaluated by Eq. (9) through the slope angle $\alpha$.

2. A small magnitude of external agency must have been applied to the soil element to trigger the slip. Although it is not possible to identify a single generic cause as mentioned above, the slip was in fact induced by an additional force $\Delta \sigma_1$ which is deemed to have been applied under an undrained condition. This scenario is illustrated in Figure 28(c).
(3) After the slip had been triggered, the sand mass continued to deform largely, if the sand was deposited sufficiently loose. The feature of soil deformation at this stage is illustrated in Figure 28(d).

Following the slope failures, configuration of underwater slope surface was detected by means of echo sounding. The outcome of these bathymetric surveys is shown in Figures 29 through 34, where the surface of underwater slopes before and after the failure are indicated for typical cross sections in the permanent and temporary slopes. Looking over the configuration of the post-failure slopes, one may envision that the failure was triggered initially at the toe of the slope and followed by flow-type movement of the soil behind it. The profiles of slopes shown in Figures 29 through 34 disclose that the soils have slid down the slope through a considerable distance resulting in thick deposits of debris at the bottom of the dredged channel. Thus, the post-failure surface in the upper part of the submerged slope can be considered to have been the plane where the soil mass had actually slid down. This surface may therefore be considered as the sliding surface on which flow of liquefied sand had taken place. It is apparent that the sliding surface thus determined is inclined with an angle smaller than the original slope angle of 1:3.0 and 1:5.0. However, to simplify calculation, it may be assumed that the moving surface of the slope during flow with large deformation was in parallel with the sliding plane, as schematically illustrated in Figure 23.

- Figure 28. Sequence of events leading to flow failure
  - (a) Initial state
  - (b) After dredging, A
  - (c) Triggering of slip, B
  - (d) Flow failure, C

- Figure 29. Slide on Dec. 3, 1995 in the permanent slope
Figure 30. Slide on Dec. 3, 1995 in the permanent slope

Figure 31. Slide on Dec. 3, 1995 in the permanent slope

Figure 32. Slide on Dec. 3, 1995 in the permanent slope
Triggering mechanism

There are several cases of submarine landslides documented in the literature. One of the large submarine landslides ever reported in details would be the slide which occurred at the rim of the recent fill towards the sea at the Port of Nice in France on October 16, 1979. The fill, much of which was deposited through or in water, was constructed on a deltaic deposit of stratified clayey silt and silty sand. Features and some analysis results are reported by H.B. Seed et al. (1988). Regarding the triggering mechanism, multiple events were cited to have been possible causes for initiating the slide. These are (1) the tidal wave first inducing about 3m lowering of the sea level due presumably to an off-shore landslide, and (2) the increase in an artesian pressure due to rainfalls in the preceding days which occurred at depths of about 40m in the pervious sand layer which is connected with water levels on land. Thus, it was difficult to narrow down the conceivable causes into a single event.

The clarification of the triggering cause for the slides in the Jamuna River channel is also a difficult task but the reasons variously conceived may be summarized as follows. (1) During the process of underwater excavation, the cutting by a dredger might have been carried out so fast that the rapid change in the state of stress did occur creating a highly
undrained condition with the result that a local failure was triggered first and retrogressed into a large slide.

(2) In the course of the excavation conducted stepwise, one step of cutting might have been so large in block that the steep slope locally created did collapse enlarging the zone of slips into the entire slide.

(3) There were natural phenomena frequently occurring in the region of Jamuna River. These include torrential downpour, wave actions and falling water levels. None of these phenomena could be pinpointed as a single seminal cause, but the effects of falling water level are most likely to be one of the reasons triggering the slips.

(4) Besides the externally imposed agencies as cited above, there would be an internal reason. That is the nature of the sand itself containing the mica mineral as mentioned above. The sand in the Jamuna River environment must be deposited in a precarious state narrowly keeping its stability and thus easily susceptible to triggering a failure. This aspect was addressed by Kramer and Seed (1988), who asserted that the sand deposited with a large initial shear stress, that is, the one anisotropically consolidated with a smaller $K_c$-value, the margin of stability is narrow against additional external agency. This fact holds true for many of sand deposits, as exemplified by the test data demonstrated in Figures 13 to 15. It may be seen for example that the sand consolidated with $K_c=0.5$ with the void ratio of 0.875 could collapse if a small deviator stress of $q=10 \text{kPa}$ is applied additionally.

Among several reasons as cited above, there are not strong reasons in support of the above hypotheses 1 and 2, that is, the rapid cutting and overstepping. Once these operations turned out to be undesirable, it is likely that the operators must have changed the way the cutting was performed. Therefore the sliding must have been limited to a certain area throughout the length of the channel. However, the slips did actually occur along the long-stretched zone in the channel. This fact appears to indicate that the hypotheses 1 and 2 are not likely to be the scenario triggering the slips.

It is the author’s view that (1) the lowering of water levels and (2) the potentially susceptible nature of the mica-containing sand in the $K_c$-condition are most likely the two major reasons creating conditions about to trigger the failure. Although the scenario is envisaged as above, it was not possible to come up with a single parameter such as a factor of safety to define the triggering mechanism in a quantitative manner. Thus, the discussion on this aspect is out of consideration in the present study.

Analysis of flow slides in Jamuna River

In this section, consideration is given to whether or not the flow type deformation could be induced in the sand deposit in Jamuna River. Thus, the following discussion is related to the flow mode of slide as illustrated in Figure 28(d).

Factor of safety against flow slide

The flow-type failure will be induced in loose sandy deposits, if the magnitude of the residual strength is equal to or smaller than that of the shear stress induced by the gravity force. It is to be mentioned here that, in the flow mode of slide, the gravity-induced stress
would be the main force driving the soil mass to move further downhill. If the soil deposit is in a loose state exhibiting the contractive behaviour with a residual strength which is smaller than the gravity-induced shear stress, then the soil mass would continue to move downwards leading to the follow-type slide. The deviator stress, \( q_o \), applied initially to a soil element at a depth of \( H \) under the submerged slope is evaluated, as follows, with reference to Eq. (8).

\[
q_o = \sigma'_i - \sigma'_c = 2\gamma'H \sin \alpha
\]  

(10)

The residual strength, \( S_{res} \), in the soil after it has been deformed largely is known to depend on the void ratio, but if the largest possible normalized residual strength, \( S_{res} / \sigma'_i = 0.26 \), within the range of contractive behaviour, is taken up for consideration, the residual strength would be estimated with reference to Eqs. (5) and (8) as

\[
S_{res} = \frac{q_o}{2} \cdot \cos \phi_s = 0.26\sigma'_c = 0.26\gamma'H(1 + \sin \alpha)
\]  

(11)

The factor of safety against the flow deformation may be defined as the ratio between the deviator stress \( q_{Qs} \) at the quasi-steady state and the initial deviator stress \( q_o \) as illustrated in Figure 27. With reference to the definition given by Eq. (3), the deviator stress \( q_{Qs} = q_s \) is expressed as \( q_s = 2S_{res} / \cos \phi_s \) and the initial deviator stress is given by \( q_o = \sigma'_i - \sigma'_c = 2\gamma'H \sin \alpha \) from Eq. (8). Thus, the factor of safety against flow is expressed as

\[
F_s = \frac{q_{Qs}}{q_o} = \frac{S_{res}}{\gamma'H \sin \alpha} \cdot \frac{1}{\cos \phi_s}
\]  

(12)

Introducing Eq. (11) into Eq. (12), one obtains,

\[
F_s = 0.26 \cdot \frac{1 + \sin \alpha}{\sin \alpha} \cdot \frac{1}{\cos \phi_s}
\]  

(13)

The factor of safety thus defined is shown plotted in Figure 35 (a) versus the angle of slope \( \alpha \) and also versus the \( K_c \)-value as evaluated by Eq. (9). The individual values of \( F_s \) obtained from the data of the TC-tests with a relative density of about 65% and 60% are indicated in Figure 35(a) by circles and those from the TE-tests are indicated by rectangles. The plot of Figure 21 showing the relation between \( K_c \)-value and \( S_{res} / \sigma'_i \) is reproduced in Figure 36 where the value of void ratio is now indicated for each value of \( S_{res} / \sigma'_i = 0.04, 0.09, 0.15 \) and 0.26.

It is known from the plot of Figure 36 that if the relative density in-situ is assumed to have been \( D_r \approx 60\% \) corresponding to \( e \approx 0.84 \), the residual strength ratio would have been \( S_{res} / \sigma'_i \approx 0.15 \).

For this value, the factor of safety is obtained simply by changing the coefficient in Eq. (13) from 0.26 to 0.15. The relation between \( F_s \) and \( K_c \) for such a case is also shown in Figure 35 (a). The zone enclosed by these two curves is indicated by shaded colour. In the same fashion, the zone corresponding to the residual strength ratio between 0.15 \( (D_r \approx 60\% \) and 0.09 \( (D_r \approx 58\% \) is displayed in Figure 35(b).

The factor of safety corresponding to smaller values of \( S_{res} / \sigma'_i \approx 0.04 \) to 0.09 is shown plotted against \( K_c \)-value in Figure 35(c).
Figure 35. Factor of safety versus $K_c$-value or angle of slope for the Jamuna River sand
Considerations for flow slides in underwater slopes at Jamuna River bridge site

As mentioned in the foregoing section (see Figure 5, the angle of underwater slope in the Jamuna River excavation was 1:3.5 in the upper and lower parts on the permanent slope and 1:5.0 in the middle portion. The slide took place on Dec. 3, 1995 at the cross sections of 1480, 1500, 1550 within the zone of W3 (see Figure 4) and cross section 1800 in W4 on Dec. 15, 1995. These are shown in Figure 29 to 32. The slide on the temporary slope occurred in April and May in 1996. The cross sections for these slides are shown in Figures 33 and 34.

There are no reliable test data available to assess the relative density of the field deposit in the Jamuna River bed, but judging from the nature of fluvial sediments and from the SPT and CPT data, it is envisaged as shown in Figure 9 that the relative density may be around \( D_r = 50\% \) and more likely less than \( D_r = 65\% \). With this assumption in mind, the residual strength of the sand in Jamuna River may be estimated roughly from the diagram of Figure 36 in which the normalized residual strength is indicated in terms of void ratio or relative density.

It might be difficult to assess the normalized residual strength only by way of relative density but if attention is paid to the range in the normalized residual strength assumed in the chart of Figure 35, that is, \( S_{un} / \sigma'_{un} = 0.04 \) to 0.26, this value seems to be in an appropriate range which is acceptable in the light of many data on other sands ever obtained. Under the consideration as above, it may conclusively mentioned that, (1) for the slope with 1:3.5, the factor of safety against flow slide could easily be less than 1.0 for the Jamuna River bed sand with the normalized residual strength less than about which could be most likely the case, and (2) for the slope with 1:5.0 slope, the value of with \( F_s = 1.0 \) could be read off from Figure 35(a) as being about 0.15 which could be the case if there exist loose zones in in-situ deposits.

The outcome of the evaluation as described above is summed up in the diagram of Figure 37 where the lines giving a factor of safety \( F_s = 0.9, 1.0 \) and 1.7 are indicated in terms of slope angle \( \alpha \) and relative density. For reference sake, SPT N-value converted by the empirical relation

\[
N'_i = 25 \left( \frac{D}{100} \right)^2
\]

is demonstrated in the ordinate of Figure 37 with the scale indicated on the right-hand side. Eq. (14) is quoted from the study by Cubrinovski and Ishihara (1999) by assuming that \( e_{\max} - e_{\min} = 0.60 \). The \( N'_i \)-value in Eq. (14) indicates the SPT N-value corresponding to an overburden pressure of 1kg/cm² and also about 80% of the theoretical energy in hammer hitting.

Drawing attention to the line \( F_s = 1.0 \) in Figure 37, one may recognize that for the slope angle between 1:5.0 and 1:3.0, the factor of safety estimated lies in the zone of flow-type instability with \( F_s < 1.0 \) assuming the relative density smaller than about 65%.

In view of the actual conditions in the field at Jamuna River as described in the foregoing sections, the results of the simple analysis summarized in Figure 37 would appear to provide evidences with a reasonable level of credibility for the occurrence of the underwater flow-type slides on the excavated slopes in Jamuna River.
CONCLUSIVE REMARKS

The failure of the underwater slopes during the dredging excavation in the sand bar deposit in Jamuna River in Bangladesh was introduced first and then some speculative reasons for such slips to have occurred were pointed out. To provide a sound basis for interpretation of
failure mechanism, the results of multiple series of triaxial tests in the laboratory were introduced with the framework of the concept in line with the test results on other sands in Japan. The main points derived from the tests are summarized as follows.

1. It is known that the sand initially consolidated with higher degrees of anisotropy, that is, with a smaller $K_c$-value, tends to exhibit more contractive (or collapsible) behaviour when it is deformed largely into a steady-state. To take into account of this effect, it was found more appropriate to select the major initial principal stress, $\sigma_{1c}'$, not the mean initial principal stress, $\tau' = (\sigma_{1c}' + 2\sigma_{2c}')/3$, as an index parameter to express the effects of confinement. Thus, when the residual strength $S_{us}$ at the steady state is normalized to the major principle stress, $\sigma_{1c}'$, it was found that the value of $S_{us}/\sigma_{1c}'$ takes a constant value irrespective of $K_c$-conditions.

2. The normalized residual strength $S_{us}/\sigma_{1c}'$ was found to be expressed uniquely as a function of relative density or void ratio. Within the range of relative density in which the behaviour is contractive, the upper limit of the normalized residual strength for the Jamuna River sand was found to be $S_{us}/\sigma_{1c}' = 0.26$ and this value is encountered when the sand is deposited at a relative density of approximately $D_r = 65\%$. On the basis of the SPT and CPT data, the in-situ relative density at the site of Jamuna River was found to be somewhere around $D_r = 50\%$ which is less than 65\%. This fact implies that the in-situ deposit is within the range of density where the sand exhibits contractive behaviour indicating high potentiality to flow type deformation once the slip is triggered by some other external forces.

3. On the other hand, the slope angle of dredged channel in Jamuna River at the time of sliding is known to have been 1:3.5 ($\approx 16$ degree) and 1:5.0 ($\approx 11.31$ degree). By assuming the flow slides to have occurred on the straight-line sliding plane which is parallel to the surface of the slope, it was possible to assess the magnitude of shear stress and the major principal stress $\sigma_{1c}'$ which was mobilized by gravity force in the sand deposits in the slopes excavated with the angle of 1:3.5 and 1:5.0.

4. By comparing the magnitude of gravity-induced shear stress assessed as above with the value of residual strength evaluated from the results of the laboratory tests, it was found that the gravity-driven shear stress must have become greater than the residual strength available at the riverbed deposit, if the in-situ sand were deposited with a relative density in the range less than 65\%. Thus, the flow-type failure might have occurred.

5. It is to be noticed here that for many other sands in Japan, the upper limit of density showing contractive behaviour is, by and large, smaller than $D_r = 40\%$. This implies that only loosely deposited sand with a relative density less than about 40\% could develop flow type deformation and therefore such a sand is identified potentially more stable in the state of in-situ deposition. In contrast, with a higher value of upper-limit density such as the mica-containing Bangladesh sand, the likelihood is high for in-situ deposits to exist with a relative density less than the upper limit density. Thus, such a sand is to be identified as potentially unstable. In the sense as above, the identification of the upper limit density or threshold density differentiating between conditions of contractive and dilative behaviour of a given sand will pose a important challenge in future development in this area of soil mechanics.
ACKNOWLEDGEMENTS

The study described in this paper was initiated by the author who paid a visit of inspection to the Jamuna River site upon recommendation by Professor W. Van Impe, Past president of ISSMGE. The engineers of HAM-van Oord kindly showed the author around the site. The triaxial tests in the laboratory were carried out under the supervision of Professor Y. Tsukamoto by Mr. Y. Nakayama and T. Shibayama, who were graduate students at the Tokyo University of Science. Professor M. Yoshimine of Tokyo Metropolitan University kindly provided data on the cone penetration tests in Jamuna River site. The author wishes to express his sincere gratitude to the persons cited above.

REFERENCES


Keynote and Special Invited Plenary Lectures
This keynote lecture summarises the main topics covered by Eurocode 7 and the interplay with Eurocode 8 and also identifies some topics that need further implementation.

INTRODUCTION

The Commission of the European Communities (CEC) initiated a work in 1975 of establishing a set of harmonised technical rules for the structural and geotechnical design of buildings and civil engineers works based on article 95 of the Treaty. In a first stage would serve as alternative to the national rules applied in the various Member States and in a final stage will replace them.

From 1975 to 1989 the Commission with the help of a Steering Committee with the Representatives of Member States developed the Eurocodes programme.

The Commission, the Member states of the EU and EFTA decided in 1989 based on an agreement between the Commission and CEN to transfer the preparation and the publication of the Eurocodes to CEN.

The Structural Eurocode programme comprises the following standards:

EN 1990 Eurocode 1 – Basis of design
EN 1991 Eurocode 1 – Actions on structures
EN 1992 Eurocode 2 – Design of concrete structures
EN 1993 Eurocode 3 – Design of steel structures
EN 1994 Eurocode 4 – Design of composite steel and concrete structures
EN 1995 Eurocode 5 – Design of timber structures
EN 1996 Eurocode 6 – Design of masonry structures
EN 1997 Eurocode 7 – Geotechnical design
EN 1998 Eurocode 8 – Design of structures for earthquake resistance
EN 1999 Eurocode 9 – Design of aluminium alloy structures

The work performed by the Commission of the European Communities (CEC) in preparing the “Structural Eurocodes” in order to establish a set of harmonised technical rules is impressive. Nevertheless, due to the preparation of these documents by several experts, some provisions of EC8 with the special requirements for seismic geotechnical design that deserve
more consideration will be presented in order to clarify several questions that still remain without answer.

The actual tendency is to prepare unified codes for different regions but keeping the freedom for each country to choose the safety level defined in each National Document of Application. The global safety of factor was substituted by the partial safety factors applied to actions and to the strength of materials.

This keynote lecture summarises the main topics covered by Eurocode 7 and the interplay with Eurocode 8 and also identify some topics that need further implementation.

In dealing with these topics we should never forget the memorable lines of Lao- Tsze, Maxin 64 (550 B.C.): “The journey of a thousand miles begins with one step”.

EUROCODE 7 – GEOTECHNICAL DESIGN

Introduction

The Eurocode 7 (EC7) “Geotechnical Design” gives a general basis for the geotechnical aspects of the design of buildings and civil engineering works. The link between the design requirements in Part 1 and the results of laboratory tests and field investigations run according to standards, codes and other accepted documents is covered by Part 2.

EN 1997 is concerned with the requirements for strength, stability, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

Eurocode 7 - geotechnical design–part 1

The following subjects are dealt with in EN 1997-1 - Geotechnical design:

Section 1: General
Section 2: Basis of Geotechnical Design
Section 3: Geotechnical Data
Section 4: Supervision of Construction, Monitoring and Maintenance
Section 5: Fill, Dewatering, Ground Improvement and Reinforcement
Section 6: Spread Foundations
Section 7: Pile Foundations
Section 8: Anchorages
Section 9: Retaining Structures
Section 10: Hydraulic failure
Section 11: Overall stability
Section 12: Embankments

Design requirements

The following factors shall be considered when determining the geotechnical design requirements:

site conditions with respect to overall stability and ground movements;
nature and size of the structure and its elements, including any special requirements such as the design life;

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conditions with regard to its surroundings (neighbouring structures, traffic, utilities, vegetation, hazardous chemicals, etc.);
ground conditions;
groundwater conditions;
regional seismicity;
influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

Each geotechnical design situation shall be verified that no relevant limit state is exceeded.

Limit states can occur either in the ground or in the structure or by combined failure in the structure and the ground.

Limit states should be verified by one or a combination of the following methods: design by calculation, design by prescriptive measures, design by loads tests and experimental models and observational method.

To establish geotechnical design requirements, three Geotechnical Categories, 1, 2 and 3 are introduced:

Geotechnical Category 1 includes small and relatively simple structures.

Geotechnical Category 2 includes conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions.

Geotechnical Category 3 includes: (i) very large or unusual structures; (ii) structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions; and (iii) structures in highly seismic areas.

Geotechnical design by calculation

Design by calculation involves:
- actions, which may be either imposed loads or imposed displacements, for example from ground movements;
- properties of soils, rocks and other materials;
- geometrical data;
- limiting values of deformations, crack widths, vibrations etc.
- calculation models.

The calculation model may consist of: (i) an analytical model; (ii) a semi-empirical model; (iii) or a numerical model.

Where relevant, it shall be verified that the following limit states are not exceeded: loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance (EQU);

internal failure or excessive deformation of the structure or structural elements, including footings, piles, basement walls, etc., in which the strength of structural materials is significant in providing resistance (STR);

failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO);

loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);

hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).
The selection of characteristic values for geotechnical parameters shall be based on derived values resulting from laboratory and field tests, complemented by well-established experience.

The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

For limit state types STR and GEO in persistent and transient situations, three Design Approaches are outlined. They differ in the way they distribute partial factors between actions, the effects of actions, material properties and resistances. In part, this is due to differing approaches to the way in which allowance is made for uncertainties in modeling the effects of actions and resistances.

In Design Approach 1 partial factors are applied to actions, rather than to the effects of actions and ground parameters,

In Design Approach 2 this approach, partial factors are applied to actions or to the effects of actions and to ground resistances.

In Design Approach 3 partial factors are applied to actions or the effects of actions from the structure and to ground strength parameters.

It shall be verified that a limit state of rupture or excessive deformation will not occur.

Design by prescriptive measures

In design situations where calculation models are not available or not necessary, the exceedance of limit states may be avoided by the use of prescriptive measures. These involve conventional and generally conservative rules in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

Design by load tests and experimental models

When the results of load tests or tests on large or small scale models are used to justify a design, the following features shall be considered and allowed for:
- differences in the ground conditions between the test and the actual construction;
- time effects, especially if the duration of the test is much less than the duration of loading of the actual construction;
- scale effects, especially if small models are used. The effect of stress levels shall be considered, together with the effects of particle size.

Tests may be carried out on a sample of the actual construction or on full scale or smaller scale models.

Observational method

When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as "the observational method", in which the design is reviewed during construction.

The following requirements shall be met before construction is started:
- the limits of behaviour which are acceptable shall be established;
- the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
- a plan of monitoring shall be devised which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
- the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- a plan of contingency actions shall be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.

**Eurocode 7 -part 2**

EN 1997-2 is intended to be used in conjunction with EN 1997-1 and provides rules supplementary to EN 1997-1 related to the:
- planning and reporting of ground investigations;
- general requirements for a number of commonly used laboratory and field tests;
- interpretation and evaluation of test results;
- derivation of values of geotechnical parameters and coefficients.

The field investigation programme shall contain:
- a plan with the locations of the investigation points including the types of investigations,
- the depth of the investigations;
- the type of samples (category, etc) to be taken including specifications on the number and depth at which they are to be taken;
- specifications on the ground water measurement;
- the types of equipment to be used;
- the standards that are to be applied.

The laboratory test programme depends in part on whether comparable experience exists. The extent and quality of comparable experience for the specific soil or rock should be established.

The results of field observations on neighbouring structures, when available, should also be used.

The tests shall be run on specimens representative of the relevant strata. Classification tests shall be used to check whether the samples and test specimens are representative. This can be checked in an iterative way. In a first step classification tests and strength index tests are performed on as many samples as possible to determine the variability of the index properties of a stratum. In a second step the representativeness of strength and compressibility tests can be checked by comparing the results of the classification and strength index tests of the tested sample with entire results of the classification and strength index tests of the stratum.

The flow chart shown below demonstrates the link between design and field and laboratory tests. The design part is covered by EN 1997-1; the parameter values part is covered by EN 1997-2.
EUROCODE 8 – DESIGN OF STRUCTURES FOR EARTHQUAKE RESISTANCE

Introduction

The Eurocode 8 (EC8) “Design of Structures for Earthquake Resistant” deals with the design and construction of buildings and civil engineering works in seismic regions is divided in six Parts.

The Part 1 is divided in 10 sections:

- Section 1 - contains general information;
- Section 2 - contains the basis requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions;
- Section 3 - gives the rules for the representation of seismic actions and their combination with other actions;
- Section 4 - contains general design rules relevant specifically to buildings;
- Section 5 - presents specific rules for concrete buildings;
- Section 6 - gives specific rules for steel buildings;
- Section 7 - contains specific rules for steel-concrete composite buildings;
- Section 8 - presents specific rules for timber buildings;
- Section 9 - gives specific rules for masonry buildings;
- Section 10 - contains fundamental requirements and other relevant aspects for the design and safety related to base isolation.

Further Parts include the following:

- Part 2 contains relevant provisions to bridges.
- Part 3 presents provisions for the seismic strengthening and repair of existing buildings.
- Part 4 gives specific provisions relevant to tanks, silos and pipelines.
Part 5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects.

Part 6 presents specific provisions relevant to towers, masts and chimneys.

In particular the Part 5 of EC8 establishes the requirements, criteria, and rules for siting and foundation soil and complements the rules of Eurocode 7, which do not cover the special requirements of seismic design.

The topics covered by Part 1- Section 1 namely: seismic action, ground conditions and soil investigations, importance categories, importance factors and geotechnical categories and also the topics treated in Part 5 slope stability, potentially liquefiable soils, earth retaining structures, foundation system, topographic aspects are discussed.

Seismic action

The definition of the actions (with the exception of seismic actions) and their combinations is treated in Eurocode 1 “Action on Structures”.

Nevertheless the definition of some terms in EN 1998-1 further clarification of seismic hazard analysis as stressed by Abrahamson (2000) is needed.

In general the national territories are divided by the National Authorities into seismic zones, depending on the local hazard.

In EC 8, in general, the hazard is described in terms of a single parameter, i.e. the value $a_g$ of the effective peak ground acceleration in rock or firm soil called “design ground acceleration” (Figure 1) expressed in terms of: a) the reference seismic action associated with a probability of exceeding ($P_{NCR}$) of 10% in 50 years; or b) a reference return period ($T_{NCR}$) = 475.

These recommended values may be changed by the National Annex of each country (e.g. in UBC (1997) the annual probability of exceedance is 2% in 50 years, or an annual probability of 1/2475).

![Figure 1. Elastic response spectrum (after EC8)](image-url)
where:

- $Se(T)$ elastic response spectrum;
- $T$ vibration period of a linear single-degree-of-freedom system;
- $\alpha_g$ design ground acceleration;
- $T_B, T_C$ limits of the constant spectral acceleration branch;
- $T_D$ value defining the beginning of the constant displacement response range of the spectrum;
- $S$ soil parameter with reference value 1.0 for subsoil class A;
- $\eta$ damping correction factor with reference value 1.0 for 5% viscous damping.

The earthquake motion in EC 8 is represented by the elastic response spectrum defined by 3 components.

It is recommended the use of two types of spectra: type 1 if the earthquake has a surface wave magnitude $M_s$ greater than 5.5 and type 2 in other cases.

The seismic motion may also be represented by ground acceleration time-histories and related quantities (velocity and displacement). Artificial accelerograms shall match the elastic response spectrum. The number of the accelerograms to be used shall give a stable statistical measure (mean and variance) and a minimum of 3 accelerograms should be used and also some others requirements should be satisfied.

For the computation of permanent ground deformations the use of accelerograms recorded on soil sites in real earthquakes or simulated accelerograms is allowed provided that the samples used are adequately qualified with regard to the seismogenic features of the sources.

For structures with special characteristics spatial models of the seismic action shall be used based on the principles of the elastic response spectra.

**Ground conditions and soil investigations**

For the ground conditions five subsoil classes A, B, C, D and E are considered:

Subsoil class A – rock or other geological formation, including at most 5 m of weaker material at the surface characterised by a shear wave velocity $V_s$ of at least 800 m/s.

Subsoil class B – deposits of very dense sand, gravel or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanics properties with depth shear wave velocity between 360-800 m/s, $N_{SPT}>50$ blows and $c_u > 250$ kPa.

Subsoil class C – deep deposits of dense or medium dense sand, gravel or stiff clays with thickness from several tens to many hundreds of meters characterised by a shear wave velocity from 160 m/s to 360 m/s, $N_{SPT}$ from 15-50 blows and $c_u$ from 70 to 250 kPa.

Subsoil class D – deposits to loose to medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft to firm cohesive soil characterised by a shear wave velocity less than 180 m/s, $N_{SPT}$ less than 15 and $c_u$ less than 70 kPa.

Subsoil class E – a soil profile consisting of a surface alluvium layer with $V_s,30$ values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with $V_s,30>800$ m/s.

Subsoil $S_1$ – deposits consisting - or containing a layer at least 10 m thick - of soft clays/silts with high plasticity index $(PI>40)$ and high water content characterised by a shear wave velocity less than 100 m/s and $c_u$ between 10-20 kPa.

• 44 •
Subsoil $S_2$ – deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A-E or $S_1$.

For the five ground types the recommended values for the parameters $S$, $T_B$, $T_C$, $T_D$, for Type 1 and Type 2 are given in Tables 1 and 2.

Table 1. Values of the parameters describing the type 1 elastic response spectrum

<table>
<thead>
<tr>
<th>Ground type</th>
<th>$S$</th>
<th>$T_A(s)$</th>
<th>$T_C(s)$</th>
<th>$T_D(s)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0</td>
<td>0.15</td>
<td>0.4</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>1.2</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
<tr>
<td>C</td>
<td>1.15</td>
<td>0.20</td>
<td>0.6</td>
<td>2.0</td>
</tr>
<tr>
<td>D</td>
<td>1.35</td>
<td>0.20</td>
<td>0.8</td>
<td>2.0</td>
</tr>
<tr>
<td>E</td>
<td>1.4</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The recommended Type 1 and Type 2 elastic response spectra for ground types A to E are shown in Figures 2 and 3.

Table 2. Values of the parameters describing the Type 2 elastic response spectrum

<table>
<thead>
<tr>
<th>Ground type</th>
<th>$S$</th>
<th>$T_A(s)$</th>
<th>$T_C(s)$</th>
<th>$T_D(s)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0</td>
<td>0.05</td>
<td>0.25</td>
<td>1.2</td>
</tr>
<tr>
<td>B</td>
<td>1.35</td>
<td>0.05</td>
<td>0.25</td>
<td>1.2</td>
</tr>
<tr>
<td>C</td>
<td>1.5</td>
<td>0.10</td>
<td>0.25</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.8</td>
<td>0.10</td>
<td>0.30</td>
<td>1.2</td>
</tr>
<tr>
<td>E</td>
<td>1.6</td>
<td>0.05</td>
<td>0.25</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The recommended values of the parameters for the five ground types A, B, C, D and E for the vertical spectra are shown in Table 3. These values are not applied for ground types $S_1$ and $S_2$.

Table 3. Recommended values of the parameters for the five ground types A, B, C, D and E

<table>
<thead>
<tr>
<th>Spectrum</th>
<th>$\alpha_v/\alpha_g$</th>
<th>$T_A(s)$</th>
<th>$T_C(s)$</th>
<th>$T_D(s)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td>0.9</td>
<td>0.05</td>
<td>0.15</td>
<td>1.0</td>
</tr>
<tr>
<td>Type 2</td>
<td>0.45</td>
<td>0.05</td>
<td>0.15</td>
<td>1.0</td>
</tr>
</tbody>
</table>
The influence of local conditions on site amplification proposed by Seed and Idriss (1982) is shown in Figure 4. The initial response spectra proposed in the pre-standard EC8 based in Seed and Idriss proposal was underestimating the design levels of soft soil sites in contradiction with the observations of the last recorded earthquakes.

Based on records of earthquakes Idriss (1990) has shown that peak accelerations on soft soils have been observed to be larger than on rock sites (Figure 5). The high quality records from very recent earthquakes Northridge (1994), Hyogo-ken-Nambu (1995), Kocaeli (1999), Chi-Chi (1999) and Tottoriken (2000) have confirmed the Idriss (1990) proposal.

Based in strong motions records obtained during Hyogoken-Nanbu earthquake in four vertical arrays sites and using and inverse analysis Kokusho and Matsumuto (1997) have plotted in Figure 6 the maximum horizontal acceleration ratio against maximum base acceleration and proposed the regression equation:

\[ \frac{\text{Acc}_{\text{surface}}}{\text{Acc}_{\text{base}}} = 2.0 \exp(-1.7 \frac{\text{Acc}}{980}) \]  

(1)

This trend with a base in a Pleistocene soil is similar to the Idriss (1990) proposal where the base was in rock.

The downhole arrays are useful: (i) to understand the seismic ground response; and (ii) to calibrate our experimental and mathematical models.

Following the comments proposed by Southern Member States the actual recommended elastic response spectrum of EC8 incorporates the lessons learnt by recent earthquakes.

The soil investigations shall follow the same criteria adopted in non-seismic areas, as defined in EC 7 (Parts 1, 2 and 3).

The soil classification proposed in the pre-standard of EC8, based only on the 3 ground materials and classified by the wave velocities was simpler. The actual ground classification of EC8 follows a classification based on shear wave velocity, on SPT values and on undrained shear strength, similar to UBC (1997) that is shown in Table 4.
Based on the available strong-motion database on equivalent linear and fully nonlinear analyses of response to varying levels and characteristics of excitation Seed et al. (1997) have
proposed for site depending seismic response the Figures 7 and 8, where $A_0$, $A$ and $AB$ are hard to soft rocks, $B$ are deep or medium depth cohesionless or cohesive soils, $C$, $D$ soft soils and $E$ soft soils, high plasticity soils.

Table 4. Ground profile types (after UBC, 1997)

<table>
<thead>
<tr>
<th>Ground profile type</th>
<th>Ground description</th>
<th>Shear wave velocity $V_s$(m/s)</th>
<th>SPT test</th>
<th>Undrained shear strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_A$</td>
<td>Hard rock</td>
<td>1500</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>$S_B$</td>
<td>Rock</td>
<td>760-1500</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>$S_C$</td>
<td>Very dense soil and soft rock</td>
<td>360-760</td>
<td>&gt;50</td>
<td>&gt;100</td>
</tr>
<tr>
<td>$S_D$</td>
<td>Stiff soil</td>
<td>180-360</td>
<td>15−50</td>
<td>50−100</td>
</tr>
<tr>
<td>$S_E$</td>
<td>Soft soil</td>
<td>&lt;180</td>
<td>&lt;15</td>
<td>&lt;50</td>
</tr>
<tr>
<td>$S_F$</td>
<td>Special soils</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Comments: The following comments are pointed: (i) as seismic cone tests have shown good potentialities they should also be recommended; (ii) the EC 8 (Part 5) stress the need for the definition of the variation of shear modulus and damping with strain level, but doesn’t refer to the use of laboratory tests such as cyclic simple shear test, cyclic triaxial test and cyclic torsional test. It is important to stress that a detailed description of laboratory tests for the static characterisation of soils is given in EC 7 Part 2 and the same criteria is not adopted in EC 8 – Part 5.

Figure 7. Proposed site-dependent relationship (after Seed et al., 1997)

Importance categories, importance factors and geotechnical categories

The structures following EC 8 (Part 1.2) are classified in 4 importance categories related with its size, value and importance for the public safety and on the possibility of human losses in case of a collapse.

To each importance category an important factor $\gamma_i$ is assigned. The important factor $\gamma_i = 1.0$ is associated with a design seismic event having a reference return period of $[475]$ years.
The importance categories varying I to IV (with the decreasing of the importance and complexity of the structures) are related with the importance factor $\gamma_i$ assuming the values $[1.4], [1.2], [1.0]$ and [0.8], respectively.

To establish geotechnical design requirements three Geotechnical Categories 1, 2 and 3 were introduced in EC 7 with the highest category related with unusual structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas.

![Figure 8. Proposed site-dependent response spectra, with 5% damping (after Seed et al., 1997) - Reduced image size to fit]

Also it is important to refer that buildings of importance categories $[I, II, III]$ shall generally not be erected in the immediate vicinity of tectonic faults recognised as seismically active in official documents issued by competent national authorities.

Absence of movement in late Quaternary should be used to identify non-active faults for most structures.

It seems that this restriction is not only very difficult to follow for structures such as bridges, tunnels and embankments but conservative due the difficult to identify with reability the surface outbreak of a fault.

Anastapoulos and Gazetas (2006) have proposed a methodology to design structures against major fault ruptures validated through successful Class A predictions of centrifuge model tests and have recommended some changes to EC8 - Part 5.

**Comments:** The following comments are presented: (i) no reference is made for the influence of strong motion data with the near fault factor (confined to distances of less than 10 km from the fault rupture surface) with the increases of the seismic design requirements to be included in building codes; (ii) also no reference is established between the ground motion and the type of the fault such as reverse faulting, strike slip faulting and normal faulting; (iii) EC8 refers to the spatial variation of ground motion but does not present any guidance; (iv) basin edge and other 2D and 3D effects were not incorporated in EC8. The importance of shapes of the boundaries of sedimentary valleys as well as of deeper geologic structures in determining site response was shown from the analysis of records in Northridge and Kobe earthquakes.
SLOPE STABILITY

For the natural or artificial slopes a verification of ground stability to ensure safety or serviceability under the design earthquake should be performed.

The following methods of analysis: (i) dynamic analysis, using finite elements; (ii) rigid block models; and (iii) simplified pseudo – static methods can be used.

For the pseudo–static analyses the following design seismic inertia forces can be taken:

\[ F_H = 0.5 \alpha_{gr} S W / g \] for the horizontal direction \hspace{1cm} (2)

\[ F_V = \pm 0.5 F_H \text{ when the ratio } \alpha_{vg} / \alpha_{gr} \text{ is greater than 0.6} \hspace{1cm} (3) \]

\[ F_V = \pm 0.33 F_H \text{ otherwise} \hspace{1cm} (4) \]

where \( \alpha_{vg} \) is the applicable design ground acceleration in the vertical direction, \( \alpha_{gr} \) is the reference peak ground acceleration for class A ground ratio, \( \gamma_f \) is the importance factor of the structure, \( S \) is the soil parameter and \( W \) is the weight of the sliding mass.

Pseudo-static method shall not be used for soils that develop high pore water pressure or significant degradation of stiffness under cyclic loading.

The serviceability limit state condition may be checked using simplified analyses with a rigid block sliding for the computation of the permanent displacement.

A modified Newmark model to compute displacements of natural slopes that includes pore pressure generation, time interval, computation of cycles involved in the cycle degradation and the computed degradation path of the slope critical acceleration was proposed by Biondi and Maugeri.

For a saturated soil in zones where \( \alpha_{gr} S > 0.15 \) it is important to incorporate the strength degradation and pore pressure increase due to cyclic loading.

Dense sands with strong dilatant effects do not exhibit reduction of shear strength.

Kramer and Paulsen (2004) have proposed a model to assess the deformations of reinforced slopes that incorporates the yielding of reinforced zone, the failure and tension of the reinforcements. The model was calibrated by shaking table and centrifuge tests.

ISSMGE TC4 Manual (1999) presents methods for rock slopes stability based on the hardness of rock and characteristics of faults proposed by Kanagawa Prefectural Government (Japan) and Mora and Vahrson.

Comments: The following items deserve more consideration: i) some guidelines to assess the residual strength of the soil; ii) some guidelines to assess the rock slopes stability.

Two mitigation methods namely anchors and soft layers were applied to study the case of Aegion slope. The following conclusions were obtained (Stamatopoulos, 2005). (i) With the use of anchors the whole body connected with anchors will move with less total and differential acceleration. There is a need to optimise the anchors inclination and length. (ii) The use of soft barrier will allow a decrease of acceleration and consequently a reduction of displacement.

POTENTIALLY LIQUEFIABLE SOILS

Following 4.1.3. (2)-Part5-EC8 “An evaluation of the liquefaction susceptibility shall be made when the foundations soils include extended layers or thick lenses of loose sand, with or without silt/clay fines, beneath the water level, and when such level is close to the ground surface”.

\[ \cdot 50 \cdot \]
Soil investigations should include SPT or CPT tests and grain size distribution (Idriss and Boulanger, 2004). Normalisation of the overburden effects can be performed by multiplying SPT or CPT value by the factor \((100/\sigma'_vo)^{1/2}\) where \(\sigma'_vo\) (kPa) is the effective overburden pressure. This normalisation factor should be taken not smaller than 0.5 and not greater than 2.

The seismic shear stress \(\tau_e\) can be estimated from the simplified expression:

\[
\tau_e = 0.65 \alpha_{gr} \gamma_f S \sigma_{vo}
\]  

(5)

where \(\alpha_{gr}\) is the design ground acceleration ratio, \(\gamma_f\) is the importance factor, \(S\) is the soil parameter and \(\sigma_{vo}\) is the total overburden pressure. This expression should not be applied for depths larger than 20 m. The shear level should be multiplied by a safety factor of \([1.25]\).

The magnitude correction factors in EC8 follow the proposal of Ambraseys (1988) and are different from the NCEER (1997) factors. A comparison between the different proposals is shown in Table 5.

Table 5. Magnitude scaling factors

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>5.5</td>
<td>1.43</td>
<td>2.20</td>
<td>2.86</td>
</tr>
<tr>
<td>6.0</td>
<td>1.32</td>
<td>1.76</td>
<td>2.20</td>
</tr>
<tr>
<td>6.5</td>
<td>1.19</td>
<td>1.44</td>
<td>1.69</td>
</tr>
<tr>
<td>7.0</td>
<td>1.08</td>
<td>1.19</td>
<td>1.30</td>
</tr>
<tr>
<td>7.5</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>8.0</td>
<td>0.94</td>
<td>0.84</td>
<td>0.67</td>
</tr>
<tr>
<td>8.5</td>
<td>0.89</td>
<td>0.72</td>
<td>0.44</td>
</tr>
</tbody>
</table>

A new proposal with a summary of different authors presented by Seed et al. (2001) is shown in Figure 9.

![Figure 9. Recommendations for correlations with magnitude (after Seed et al., 2001)](image-url)
Empirical liquefaction charts are given with seismic shear wave velocities versus SPT values to assess liquefaction. A comparison between NCEER (1997) and EC8 proposal for pre-standard is shown in Figure 10. It is important to refer that the proposal for EC8 is based on the results of Roberston et al. (1992) and the proposal of NCEER (1997) incorporates very recent results.

Figure 10. Liquefaction potential assessment by NCEER (1997) and EC8 (pre-standard)

However the EC8 standard version considers that these correlations are still under development and need the assistance of a specialist.

The importance of this topic has increased and the assessment of liquefaction resistance from shear wave crosshole tomography was proposed by Furuta and Yamamoto (2000).

A new proposal presented by Cetin et al. (2001) is shown in Figure 11 considered advanced in relation with the previous ones, as integrates: (i) data of recent earthquakes; (ii) corrections due the existence of fines; (iii) experience related a better interpretation of SPT test; (iv) local effects; (v) cases histories related more than 200 earthquakes; (v) Baysiana theory.

Bray et al. (2004) have shown that the chinese criteria proposed by Seed and Idriss (1982) was not reliable for the analysis of silty sands liquefaction and have proposed the plasticity index.

The topic related with the assessment of post liquefaction strength is not treated in EC8, but it seems that the following variables are important: fabric or type of compaction, direction of loading, void ratio and initial effective confining stress (Byrne and Beaty, 1999).

A relationship between SPT N value and residual strength was proposed by Seed and Harder (1990) from direct testing and field experience (Figure 12).

Ishihara et al. (1990) have proposed a relation of normalized residual strength and SPT tests, based on laboratory tests compared with data from back-analysis of actual failure cases (Figure 13). Also Ishihara et al. (1990) by assembling records of earthquake caused failures
in embankments, tailings dams, and river dykes have proposed the relation of Figure 14, in terms of the normalized residual strength plotted versus CPT value.

Alba (2004) has proposed Bingham model, based in triaxial tests of large samples, to simulate residual strength of liquefied sands.

Figure 11. Probabilistic approach for liquefaction analysis (after Cetin et al., 2001)

Figure 12. Relationship between (N1) 60 and undrained residual strength (after Seed and Harder, 1990)

Figure 13. Relation of normalized residual strength and SPT tests (after Ishihara et al., 1990)

The susceptibility of foundations soils to densification and to excessive settlements is referred in EC8, but the assessment of expected liquefaction - induced deformation deserves more consideration.

By combination of cyclic shear stress ratio and normalized SPT N-values Tokimatsu and Seed (1987) have proposed relationships with shear strain (Figure 15).
To assess the settlement of the ground due to the liquefaction of sand deposits based on the knowledge of the safety factor against liquefaction and the relative density converted to the value of $N_1$ a chart (Figure 16) was proposed by Ishihara (1993).

Following EC8 ground improvement against liquefaction should compact the soil or use drainage to reduce the pore water pressure. The use of pile foundations should be considered with caution due to the large forces induced in the piles by the liquefiable layers and the difficulties to determine the location and thickness of these layers.
Two categories of remedial measures against liquefaction were proposed: (i) Solutions aiming at withstanding liquefaction - Confinement wall: stiff walls anchored in a non-liquefied layer (or a bedrock) to avoid lateral spreading in case of liquefaction; Soil reinforcement - transfer of loads to a non-liquefiable layer. (ii) Solutions to avoid liquefaction: - Soil densification: compaction grouting to minimise the liquefaction potential; - Dewatering: to lower the water table in order to minimise the risk of liquefaction; - Drainage to facilitate the dissipation of pore pressure; - Fine grouting: to increase the soil cohesion.


Following EC8 ground improvement against liquefaction should compact the soil or use drainage to reduce the pore water pressure. The use of pile foundations should be considered with caution due the large forces induced in the piles by the liquefiable layers and the difficulties to determine the location and thickness of these layers. The remedial measures against liquefaction can be classified in two categories (TC4 ISSMGE, 2001; INA, 2001): (i) the prevention of liquefaction; and (ii) the reduction of damage to facilities due to liquefaction.

The measures to prevent of occurrence of liquefaction include the improvement of soil properties or improvement of conditions for stress, deformation and pore water pressure. In practice a combination of these two methods is adopted.

The measures to reduce liquefaction induced damage to facilities include (1) to maintain stability by reinforcing structure: reinforcement of pile foundation and reinforcement of soil deformation with sheet pile and underground wall; (2) to relieve external force by softening or modifying structure: adjusting of bulk unit weight, anchorage of buried structures, flattening embankments.

In NEMISREF Project the following criteria for selection was used (Evers, 2005): (i) Potential efficiency; (ii) Technical feasibility; (iii) Impact on structure and environmental; (iv) Cost-effectiveness; (v) Innovation.

Two methods were selected: (i) Soil grouting using calcifying bacteria; (ii) confinement wall.

Related with calcifying bacteria the objective of soil consolidation is to create a cementation between the grains of soil skeleton increasing the cohesion.

With confinement wall even if partial liquefaction could occur the final deformations will be controlled.

The improvement of soil properties, to prevent soil liquefaction, by soil cementation and solidification is performed by deep mix method (Port Harbour Research Institute, 1997), so within this framework the use of bacteria technique is innovative.

The structural strengthening is performed by pile foundation and sheet pile (INA, 2001) and so the confining wall can be considered innovative.

The proposed methods of remediation have an additional advantage minimizing the effects on existing structures during soil improvement.
**Comments:** From the analyses of this section it seems that the following items deserve more clarification (Sêco e Pinto, 1999):

i) It is important to quantify the values of extended layers or thick lenses of loose sand;
i) What is the meaning of “......when such level is close to the ground surface”? What depth? What is the maximum depth liquefaction can occur?
i) No recommendation is presented to compute seismic shear stress $\tau_e$ for depths larger than 20 m;
v) The use of Becker hammer and geophysical tests to assess the liquefaction of gravely materials should be stressed;
v) The recommended multiplied factor CM for earthquake magnitudes different from 7.5 deserves more explanation. It is important to refer that the well known correlation proposed by Seed et al (1984) for cyclic stress ratio versus $N_1$ (60) to assess liquefaction and adopted in Annex B of EC8 – Part 5 use different correction factor for earthquake magnitudes different from 7.5;

vi) No reference is given for the residual strength of soil.

**FOUNDATION SYSTEM**

In general for the Soil-Structure Interaction (SSI) the design engineers ignore the kinematic component, considering a fixed base analysis of the structure, due the following reasons: (i) in some cases the kinematic interaction may be neglected; (ii) aseismic building codes, with a few exceptions e.g. Eurocode 8 do not refer it; (iii) kinematic interaction effects are more difficult to assess than inertial forces (Sêco e Pinto, 2003).

There is strong evidence that slender tall structures, structures founded in very soft soils and structures with deep foundations the SSI plays and important role.

The Eurocode 8 states:” Bending moments developing due to kinematic interaction shall be computed only when two or more of the following conditions occur simultaneously: (i) the subsoil profile is of class $D$, $S_1$ or $S_2$, and contains consecutive layers with sharply differing stiffness;(ii) the zone is of moderate or high seismicity, $\alpha>0.10$;(iii) the supported structure is of important category I or II.

The stability of footings for the ultimate state limit design criteria shall be analysed against failure by sliding and against bearing capacity failure.

For shallow foundations under seismic loads failure can not be defined for situations when safety factor becomes less than 1, but is related with permanent irrecoverable displacements.

The seismic codes recommend to check the following inequality:

$$S_d < R_d$$

where $S_d$ is the seismic design action and $R_d$ the system design resistance.

In the inequality (3) partial safety factors shall be included following the recommendations of Eurocode 8.

Theoretical and experimental studies to provide bearing capacity solutions to include the effect of soil inertia forces led to the inequality (Pecker, 1997):

$$\phi(N,V,M,F) < 0$$

where $\phi=0$ defines the equation of the bounding surface (Figure 17).
The combination of the loading lying the outside the surface corresponds to an unstable situation and the combination lying inside the bounding surface corresponds to a potentially stable situation.

Piles and piers shall be designed to resist the following action effects: (i) inertia forces from the superstructure; and (ii) kinematic forces resulting from the deformation of the surrounding soil due the propagation of seismic waves.

The complete solution is a 3D analysis very time demanding and it is not adequate for design purposes. The decomposition of the problem in steps is shown in Figure 18 and implies (Gazetas and Mylonakis, 1998): (i) the kinematic interaction involving the response of the base acceleration of the system considering the mass of superstructure equal to zero; (ii) the inertial interaction that involves the computation of the dynamic impedances at the foundation level and the dynamic response of the superstructure.

For the computation of internal forces along the pile, as well as the deflection and rotation at the pile head, both discrete (based in Winkler Spring model) or continuum models can be used (Finn and Fujita, 2004).

The lateral resistance of soil layers susceptible to liquefaction shall be neglected.

In general the linear behaviour is assumed for the soil.

The nonlinear systems are more general and the term nonlinearities include the geometric and material nonlinearities (Pecker and Pender, 2000).

The engineering approach considers two sub-domains (Figure 19): (i) a far field domain where the non-linearities are negligible; (ii) a near field domain in the neighbouring of the foundation where the effects of the geometrical and material nonlinearities are concentrated.

The following effects shall be included: (i) flexural stiffness of the pile; (ii) soil reactions along the pile; (iii) pile–group effects; and (iv) the connection between pile and structure.

The use of inclined piles is not recommended to absorb the lateral loads of the soils. If inclined piles are used they must be designed to support axial as well bending loads.

Piles shall be designed to remain elastic, if this is not possible potential plastic hinging shall be considered for: (i) a region of depth 2d (d-diameter of the pile) from the pile cap; (ii) a region of ±2d from any interface between two layers with different shear stiffness (ratio of shear moduli > 6).
Evidence has shown that soil confinement increases pile ductility capacity and increases pile plastic hinge length. Piles have shown the capability to retain much of their axial and lateral capacity even after cracking and experienced ductility levels up to 2.5 (Gerolymos and Gazetas, 2006).

The investigation methods for pile foundation damage are: direct visual inspection, the use of borehole camera inspection and pile integrity test. The ground deformation can be investigated by visual survey and GPS survey (Matsui et al. 1997).

Comments: The following topics deserve more consideration:

i) The influence of pile cap;

ii) The moment rotation capacity of pile footing;

iii) The incorporation of the non linear behaviour of the materials in the methods of analysis;

iv) The instrumentation of the piles for design purposes;

v) Some guidelines about group effects, as there are significant different opinions on the influence of group effects related with the number of piles, spacing, direction of loads, soil types and construction methods of piles.

For the evaluation of mitigation methods a preliminary analysis of the following solutions was performed (Evers, 2005): (i) Stiffening solutions - hard layer, reinforced concrete walls, soil stiffening at foundation level and inclined piles; (ii) Soft material barriers - soft layer, expanded polystyrene (EPS) walls, air-water balloons and soft caisson; (iii) oscillators.

For the criteria of selection the following factors were used: Potential efficiency, technical feasibility, impact on structure and environment, cost-effectiveness and innovation.
From this analysis the following two mitigation methods: i) soil stiffening (inclined micro-piles) and ii) deformable soft barriers (soft caisson) were selected.

**EARTH RETAINING STRUCTURES**

The methods of analyses of an earth-retaining structure shall incorporate: (i) the non-linear behaviour of the soil; (ii) the inertia effects associated with the masses of the soil; (iii) the hydrodynamic effects generated with by the presence; (iv) the compatibility between deformations of the soil, wall and the tiebacks.

For the pseudo–static analysis of rotating structures the seismic coefficients can be taken as:

\[
\begin{align*}
    k_h &= \alpha_{gr} \gamma f S / g \cdot r \\
    k_v &= \pm 0.5 k_h \text{ when the ratio } \alpha_{vgr} \gamma f / \alpha_{gr} \text{ is greater than 0.6} \\
    k_v &= \pm 0.33 k_h \text{ otherwise}
\end{align*}
\]

where \(\alpha_{gr}\) is the reference peak ground acceleration for class A ground, \(S\) is the soil parameter, \(\gamma f\) is the importance factor of the structure and the factor \(r\) takes the values listed in Table 6.

<table>
<thead>
<tr>
<th>Type of retaining structure</th>
<th>(r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free gravity walls that can accept a displacement (dr \leq 300 \alpha S) (mm)</td>
<td>2</td>
</tr>
<tr>
<td>As above with (dr \leq 200 \alpha S) (mm)</td>
<td>1.5</td>
</tr>
<tr>
<td>Flexural r.c. walls, anchored or braced walls, r.c. walls founded on vertical piles,</td>
<td></td>
</tr>
<tr>
<td>restrained basement walls and bridge abutments</td>
<td>1.0</td>
</tr>
</tbody>
</table>

For saturated cohesionless soils susceptible to develop high pore pressure the \(r\) factor should not be taken larger than 1.0, and the safety factor against liquefaction should not be less than 2.

The point of application of the force due to dynamic earth pressure shall be assumed to lie at midheight of the wall and for walls which are free to rotate about their toe it is appropriate to consider the dynamic force acting at the same point as the static force.

For a soil permeability coefficient less than \(5 \times 10^{-4}\) m/s the pore pressure is not free to move and the soil will behave as an undrained situation, during the occurrence of seismic action.

The earth pressure coefficient can be computed from the Mononobe and Okabe formula.

The point of application of the force due to the hydrodynamic water pressure lies at a depth below the top of the saturated layer equal to 60% of the height of such layer.

The pressure distributions on the wall due to the static and the dynamic action shall be assumed to act with an inclination with respect to the normal to the wall not greater than \((2/3) \phi\) for the active state and equal to zero for the passive state.

The stability of soil foundation shall be assessed for the following conditions: (i) overall stability; and (ii) local soil failure.

The anchoring system (tiebacks and anchors) provided behind walls and bulkheads shall have enough strength to assure equilibrium of the critical soil wedge under seismic conditions, as well as a sufficient capacity to adapt to the seismic deformations of the soil.
The EC8 does not refer to the behaviour of reinforced walls. The behaviour of these structures during recent earthquakes suggests that these types of structures are well suited for seismically active regions (Sitar et al., 1997).

The EC8 only refers the condition of walls to slide, but it is important to stress the rocking of large concrete gravity walls under earthquake loading (Sêco e Pinto, 1995).

For an embedded retaining structure characterised by a ductile behaviour, it can be anticipated that the equivalent value of the acceleration to use in a pseudo-static calculation, as if it were constant in time, should be significant smaller than the expected peak acceleration (Anastassapoulos, 2004).

Comments: From the analyses of this section it seems that the following items deserve additional consideration:

(i) Design methods for the computation of permanent displacements that allow the couple computation of rotation and translation movements should be referred;
(ii) For retaining walls of medium heights (greater than 6 m) the computed displacements are larger than the values listed by EC 8 (Wu and Prakash, 2001);
(iii) The permanent displacements should be related with the height of the wall;
(iv) The good behaviour of geogrid – reinforced soil retaining walls in comparison with reinforced concrete cantilever retaining walls, during the occurrence of earthquakes, should be stressed.

TOPOGRAPHIC AMPLIFICATION FACTORS

For the stability verification of ground slopes EC8 recommends simplified amplification factors for the seismic action to incorporate the topographic effects. Such factors should be applied for slopes with height greater than 30 m.

The following recommendations are given:

(i) for slopes angles less than 15° the topography effects can be neglected;
(ii) for isolated cliffs and slopes a value of $S \geq 1.2$ should be used;
(iii) for slopes angles > 30° a value of $S \geq 1.4$ should be used and $S \geq 1.2$ for smaller slope angles;
(iv) in the presence of a looser surface layer more than 5 m thick, the smallest value given in (ii) and (iii) shall be used increased by at least 20%.

No reference is made for 2 D models or 3 D models and for the frequency range amplifications observed in 2 D and 3 D models.

However Paolucci (2005) have pointed that amplification factors for 2D analyses are of the same range of EC8, but for 3D analyses the values are 25% higher.

To assess the topographic amplification is important to separate from the site amplification. Also topographic amplification varies with the frequency content of the earthquake (Pitilakis et al., 2005).

One recent example is related with the topographic amplification occurred in the coastal bluffs of the Pacific Palisades during the January 17, 1994 Northridge earthquake. The slopes with 40 to 60 m height and steep between 45 to 60 degrees failure.

Parametric studies conducted by Idriss (1968) on 27 and 45 degrees clay slopes using finite element method have shown that the magnitude of peak surface acceleration was greater at
the crest surface of the slope than at points lower on the slope, but comparing the peak ground acceleration at the crest to that at some distance behind the crest in some cases the acceleration at the crest was much greater, in other case cases there was little difference. The natural period of the soil column behind the crest of a slope was responsible for much more amplification of the input motion than the slope geometry.

Ashford et al (1997) concluded that topographic effects can be normalized as a function of the ratio of the slope height and wave length of the motion and the trend is shown in Figure 20.

![Figure 20. Amplification effects of steep slopes](image)

**INTERACTION WITH OTHER SEISMIC CODES**

The continuous process of elaborating codes and standards that incorporates the lessons learned by earthquakes has a significant effect for the design and construction in seismic areas. As an example due the consequences of Kobe earthquakes new improvements have been implemented for the assessment of liquefaction in Japanese codes (Yasuda, 1999).

The actual tendency in almost every region of the world is to elaborate unified codes. This was the main purpose of Eurocodes.

In United States, nevertheless the long tradition of existing different codes in the states, due the different geographic regions of the U. S., where the western region in comparison with eastern region faces the largest levels of probabilistic seismic risk, efforts to elaborate an uniform code incorporating the UBC (Uniform Building Code) with NEHRP (National Seismic Hazard Reduction Program) in IBC (International Building Code), not an international code, are undertaken (Seed and Moss, 1999).

In general all geotechnical codes use factored parameters for loads, resistance and strength to harmonize with structural practice. But the way of doing is different, for instance the Eurocode uses partial safety factors for loads and strength (Cuellar, 1999) and in the New Zealand code the LRFD (load and resistance factored design) is used (Pender, 1999).
It is important to stress that Eurocodes represent a significant step forward and allows various Nationally Determined Parameters (NDP) which should be confirmed or modified in the National Annexes.

FINAL REMARKS

The work performed by the Commission of the European Communities (CEC) in preparing the “Structural Eurocodes” in order to establish a set of harmonised technical rules is impressive. However we feel that some topics deserve more consideration.

Earthquakes are very complex and dangerous natural phenomena, which occurs primarily in known seismic zones, although severe earthquakes have also occurred outside these zones in areas considered being geologically stable. As a result, regulatory agencies became more stringent in their requirements for demonstration of adequate seismic stability and design engineers responded by developing new and more convincing design approaches than had previously used. Thus the past years have seen a major change in interest and attitude towards this aspect of design.

The lessons learned from recent earthquakes such as: Northridge (1994), Kobe (1995), Umbria-Marche (1997), Kocaeli (1999), Athens (1999), Chi-Chi (1999) and Bhuj (2001) have provided important observational data related with the seismic behavior of geotechnical structures.

The need of cost effective methods to upgrade buildings by developing new specific foundations techniques is a major problem. So the objective of reducing the earthquake motion transferred to the structure through the foundation by developing innovative constructive techniques for soil improvement and soil reinforcement is getting increase attention.

One very important question to be discussed is: (i) how detailed a seismic code must be; (ii) what is the time consuming to establish a set of harmonised technical rules for the design and construction works? (iii) How to improve the relations between the users: relevant authorities, clients and designers? and (iv) how to implement in practice that codes may not cover in detail every possible design situation and it may require specialised engineering judgement and experience? It is hoped that the contributions to be presented by CEN members, in the next years, will help to clarify several questions that still remain without answer.

In dealing with this subject we should always have in mind:

All for Love
“Errors, like straws, upon the surface
flow;
He who would search for pearls must
dive below”.

(John Dryden)

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LIQUEFACTION MITIGATION OF SAND DEPOSITS BY
GRANULAR PILES- AN OVERVIEW

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Soil liquefaction and associated ground failures have been a major source of damage during
the past earthquakes. The risk of liquefaction and associated ground deformation can be
reduced by various ground-improvement methods including the stone column (gravel drain)
technique. Ground improvement by rammed granular piles (RGP) is considered one of the
most reliable of these methods. This paper summarizes different mechanisms involved in the
effective function of granular piles as a ground treatment method for liquefaction mitigation.
Various mechanisms like: Drainage, Reinforcement, Storage, Dilation and Densification
effects are briefly discussed. Generation and dissipation of the pore pressures in the granular
pile reinforced ground under different earthquake conditions are quantified considering these
different mechanisms. Granular piles are proved to be very effective for liquefaction
mitigation.

INTRODUCTION

Ground improvement techniques are commonly employed to mitigate liquefaction hazards.
Most common methods to improve the engineering properties of the soils can be classified as
densification, reinforcement, grouting/mixing and drainage. Out of the various ground
treatment methods, (Rammed) Granular Drains/Piles (RGP) are the most widely-preferred
alternative all over the world, due to technical feasibility, low energy utilization and cost
effectiveness. Provision of gravel drains/granular piles/stone columns is the most commonly
adopted ground treatment methodology for liquefaction mitigation which has proved its
effectiveness in many instances (Mitchell and Wentz, 1991). RGP improve the ground by
reinforcement, densification of the surrounding soil and by providing drainage. Different
mechanisms operate in the function of gravel drains/granular piles in liquefaction mitigation.
These mechanisms can be stated as Drainage, Storage, Dilation, Densification and
Reinforcement. The following sections discuss each of these mechanisms in detail.
GRANULAR PILES, INSTALLATION, FUNCTIONS, MECHANISMS

Columnar inclusions such as stone columns/granular piles, sand compaction piles, lime or cement columns, etc., have been used as a ground improvement technique to increase bearing capacity, reduce settlement, increase the time rate of consolidation, improve stability and resistance to liquefaction of soft ground. Various techniques of installation have been conceived for various types of columnar inclusions in a wide variety of soils such as loose sandy to soft compressible soils depending on technical ability, efficiency and local conditions. Ground improvement by means of granular piles/stone columns/geo-piers, which is associated with partial substitution of the in-situ soil, originated in sixties. Stone columns generally use gravel or crushed stone as backfill. Effect of method of installation, cased and uncased holes, number of lifts and magnitude of compactive energy per lift given to granular piles and pile spacing were discussed by Madhav and Thiruselvam (1988). Numerous publications (e.g. Barksdale and Bachus, 1983; Munfakh et al., 1987; Baez and Martin, 1992; and Lopez and Hayden, 1992; Brennan and Madabhushi, 2002) describe the use of stone columns for ground reinforcement and their potential to mitigate the liquefaction. Liquefied and Non-liquefied subsoil conditions of two reclaimed islands in Kobe City after the 1995 Hyogoken-Nambu earthquake were investigated by Yasuda et al. (1996) who identified that subsoils treated with sand compaction piles or rod (vibro) compaction did not liquefy and nor subside even though the earthquake shaking was very strong. Ground treated by granular piles provide increased bearing capacity, significant reduction in settlement, free drainage, increase of liquefaction resistance, etc. Granular piles are installed by vibro-compaction, vibro-replacement, cased bore hole (rammed stone columns/RGP) or by simple auger boring methods (Datye and Nagaraju 1981, Balaam and Booker 1981). RGP are installed into the ground by partial or full displacement methods and by ramming in stages, using a heavy falling weight, within a ‘pre-bored casing’ or ‘driven closed end casing’, retracting the casing pipe stepwise. In the latter case, driving of closed end tube itself densifies the surrounding soil. Ramming of granular piles further densifies and reinforces the ground. Stone columns/granular piles or sand compaction piles are generally composed of compacted gravel, crushed stone or sand. Theoretical background, analysis, design aspects and installation techniques were being developed since 1970s by various researchers and practicing engineers all over the world (Hughes and Withers, 1974; Datye and Nagaraju, 1981; Engelhardt and Golding, 1975; Madhav et al. 1979, etc.).

A possible method of stabilizing a soil deposit, susceptible to liquefaction, is to install a system of gravel or rock drains so that pore-water pressures generated by cyclic loading may be dissipated almost as fast as they are generated (Seed and Booker, 1977). Granular pile inclusions improve the deformation properties of the ambient soil. Granular piles help in mitigating earthquake induced liquefaction effects through one or more of these functions.

1. Granular piles function as drains and permit rapid dissipation of earthquake induced pore pressures by virtue of their high permeability with the additional advantage that they tend to dilate as they get sheared during an earthquake event.

2. Granular piles densify and reinforce the in-situ soil;
3. Granular piles, installed in a very dense state, are not prone to liquefaction and replace a significant quantity of in-situ liquefiable soil;

4. Granular piles modify the nature of earthquake experienced by the in-situ soil;

DRAINAGE MECHANISM OF GRAVEL DRAINS

In many cases, the installation of drainage system offers an attractive and economical procedure for stabilizing an otherwise potentially liquefiable sand deposit. Adopting ground treatment by gravel drains as a method to determine the stability of potentially liquefiable sand deposits was initiated originally by Seed and Booker (1977). In fact, better field performance of gravel drains/stone columns may be directly attributable to their capacity to dissipate pore-water pressures because of their higher permeability. Due to the installation of gravel drains, the generated porewater pressure due to repeated loading may be dissipated almost as fast as they are generated. Seed and Booker (1977) applied the one-dimensional theory of porewater pressure generation and dissipation developed by Seed et al. (1975) to the analysis of columnar gravel drains under a variety of earthquake conditions and proposed a simple radial consolidation analytical model.

In cases of high liquefaction potential the installation of columnar gravel drains may well provide an efficient method for preventing the development of excessively high pore water pressures. Generally, the horizontal permeability of a sand/gravel is several times greater than its vertical permeability. As such the spacing between vertical drains can be made less than the distance required for water to drain vertically to a free surface. Hence, the drainage mechanism by gravel drains/stone columns is attributed to their high permeability and the ability to reduce the drainage path significantly so that the developed excess pore water pressures are dissipated almost as fast as they are generated.

For flow into a gravel drain, assuming pure radial flow, and constant coefficients of permeability ($k_b$) and volume compressibility ($m_v$), the governing equation for the phenomenon can be written as (Seed and Booker, 1977):

$$\frac{k_b}{\gamma_m m_v} \left( \frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) = \frac{\partial u}{\partial t} - \frac{\partial u}{\partial r} \frac{\partial N}{\partial r}$$

Seed and Booker (1977) presented the charts (Figure 1), for the variation of maximum pore pressure ratio against time during an earthquake for different area ratios and other earthquake parameters, to aid in the design of gravel drains for liquefaction mitigation. These charts are based on the assumption that the drains are infinitely permeable so that the excess pore water pressure in the drain is effectively zero and the coefficient of permeability of the ambient soil is constant.

Tokimatsu and Yoshimi (1980), Sasaki and Taniguchi (1982) and Onoue (1988), report results similar to those of Seed and Booker (1977) taking into consideration additional factors such as well resistance (finite permeability of gravel drain) and drain slenderness ratio (slenderness ratio: $L/r$, where $L$ is the length, and $r$ is the radius of the gravel drain). Iai and Koizumi (1986), Onoue et al. (1987) and Iai et al. (1988), present design procedures for gravel drains derived analytically and verified by model or in situ tests. All of the above studies considered only the drainage effect of gravel drains (Adalier and Elgamal 2004).
Poorooshasb et al. (2000) demonstrated the effectiveness of inclusion of stone columns in reducing the risk of liquefaction of very loose to loose sandy and silty sand layers using the concept of equivalent permeability. Based on the radial consolidation theory as applicable to granular pile treated ground, Poorooshasb et al. (2000) propose an equivalent coefficient permeability, \( k_{eq} \), for the treated soil in terms of the permeability, \( k_{untr} \), of untreated ground, as

\[
k_{eq} = \frac{k_{untr} t_{50} \text{ (for untreated ground)}}{t_{50} \text{ (for treated ground)}}
\]

where \( t_{50} \) values for the untreated and granular pile treated ground are the times for 50% degree of consolidation based on one dimensional and radial consolidation theories respectively. The ratio, \( r (= k_{eq}/k_{untr}) \) is derived (Figure 2) in terms of the spacing ratio, \( n (= S/d) \) where \( S \) and \( d \) are respectively the spacing and diameter of granular piles. The results presented are for triangular arrangement of drains but can easily be modified for square arrangement.

Thus, the principal mechanism of granular drains/stone columns in mitigating the liquefaction potential is their drainage effect.

![Figure 1. Effect of drain diameter and drain spacing on maximum pore pressure ratio (after Seed and Booker, 1977)](image1.png)

![Figure 2. Equivalent permeability concept (after Poorooshasb et al., 2000)](image2.png)

**STORAGE EFFECT, GRAVEL DRAINS**

Generally, in most practical cases, the water level is at some depth below ground surface. During an earthquake, excess pore water pressure is dissipated by water discharging into the vertical drains, causing a rise in the water level within the drains. This rise in water level results in an increased “static” excess pore pressure (or backpressure) inside the drain elements, which in turn increases the excess pore pressure in the soil deposit. A reservoir (natural or man-made) with additional storage capacity above the original water table provides a buffer storage capacity and thus reduces the rate of rise of water level, thus minimizing backpressure effect in the drain (Pestana et al. 1997).

The effect of storage capacity on vertical drain performance in liquefiable sand deposits is presented by Pestana et al. (1998) by considering a two-layer soil system with a very low
permeability clay overlying the liquefiable sand layer as shown in the Figure 3. Figure 4 shows average pore pressure ratios, $R_u(z_{\text{max}})$, versus cycle ratio, $r_N$ for perfect drain with and without the storage capacity. Here the perfect drain implied the drain with out any well resistance as that of the Seed and Booker (1977) model. If the water table in the system is not at the ground surface, there is room within the drain for water to rise that represents the storage capacity. In the figure for all storage cases, the water level in the drain was initially at 1 m depth. $R_u$ is the average pore pressure ratio at depth, $z$, in the liquefiable layer. As the upper soil layer acts as a barrier to vertical flow, the maximum value of $R_u$, $R_u(z_{\text{max}})$, occurs at the interface between the upper and lower soil layers. The cyclic ratio, $r_N$, is the ratio of the number of uniform stress cycles in an earthquake versus the number of uniform stress cycles required to cause liquefaction in the soil layer in question under undrained conditions. The figure shows that at a small drain spacing ($s/d=4$), the rise in the water level in the drain forces a steady rise in the pore pressure ratios in the drain compared to the case with no storage. Similarly, in the other cases ($s/d=6, 8$ and $10$), the effect of storage is to increase the maximum pore pressure ratios.

Pestana et al. (1997, 1998) analysed the use of the providing the reservoir in reducing the depth of water levels within the drain and also to minimise the drain resistance to flow in to the drain. The reservoir may be formed by using a composite drainage product consisting of a perforated geopipe surrounded by a filler fabric to prevent clogging of the orifices (Pestana et al. 1998). This is achieved by a technique of auguring a hole larger that the diameter of the drain near the ground surface and filling it with crushed rock or gravel. This creates a reservoir for flow rising out of the drain during an earthquake, and at the same time, provides a means for reducing the rise in water level with in the drain as the same volume of water is now spread out over a larger area (Figure 5) as the equivalent area of the reservoir is larger that that of the drain.
DILATION EFFECT, GRAVEL DRAINS

It has been generally recognized that the susceptibility of a given soils to liquefaction is determined to a high degree by its void ratio or relative density. In any given earthquake loose sands are very likely to liquefy but the same granular material in a denser condition would not. For example, in 1964 Niigata earthquake, liquefaction was extensive where the relative density of the sand was about 50% or less, but not in areas where the relative density exceeded about 70% (Seed and Idriss, 1971).

Shearing of dense dilative soils may generate some small positive pore pressures at small strains. However, at larger strains, the pore pressures decrease and become negative as the soil grains move up over one another, tending to cause an increase in soil volume (dilation). For dense, saturated sands sheared without drainage, the tendency for dilation or volume increase results in generation of negative pore pressure, with consequent increases in the effective stress and the shear strength of the granular material. The response of saturated sand under undrained triaxial conditions (Leonards, 1962) is shown in Figure 6. While positive pore pressures are generated in loose sands, generation of very high negative pore pressures can be observed due to suppression of the tendency for dilation in medium dense and dense sands. Figure 7 (Vaid et al., 1981) is a typical example of the volume change behaviour of granular material under drained conditions in a simple shear test at different vertical stress conditions. While initially loose samples undergo volume decrease, dense samples experience volume increase (dilation) during shearing. The rate of dilation increases with relative density. The dilation angle is one single parameter which can be readily obtained from both laboratory (drained triaxial or simple shear) and in situ (self-boring pressuremeter) tests, which can give a measure of the liquefaction resistance.

Dilation effect on the drainage function of granular piles was studied by Madhav and Arlekar (2000) by extending the Seed and Booker model (1977). It is shown that the dilation effect on pore pressure dissipation by granular piles for the range of parameters considered was marginal (Figure 8). Granular piles installed in loose sand deposits are often compacted at a relatively high density. Seismic forces which tend to generate positive pore pressures in these deposits cause an opposite effect of dilation in the dense granular piles. During the seismic event, negative pore pressures that tend to get generated therein increase
the gradient and permit rapid rates of drainage than otherwise. Madhav and Arlekar (2000) quantify this effect of dilating granular pile in mitigating liquefaction damage. The variation of maximum pore pressure ratio, $W_{\text{max}}$, with normalized time for different rates of dilation is depicted in Figure 9. The effect of dilation rates in reducing the peak values or the pore pressure ratio can be clearly seen in the figure.

![Figure 6. Response of saturated sand under undrained triaxial test conditions (after Leonards 1962)](image1)

![Figure 7. Volume change behavior for granular material under drained conditions (after Vaid et al. 1981)](image2)

![Figure 8. Effect of dilation of granular material on pore pressure generation and dissipation](image3)

![Figure 9. Maximum pore pressure versus time: Effect of dilation and $T_{\text{bd}}$](image4)

**DENSIFICATION EFFECT OF GRAVEL DRAINS**

One of the chief benefits of ground treatment with granular piles is the densification of in situ ground. The densification effect can be easily but indirectly quantified by in situ tests. The effect of densification is manifested through an increase in the coefficient of earth
pressure at rest and in the values of modulus of deformation of the soil (Ohbayashi et al. 1999). The densification is maximum close to the SCP and least at the farthest point, i.e. at the center of the grid points. Tsukamoto et al. (2000) examined the changes in the state of stress due to static sand compaction pile penetration in densifying loose to medium dense soils and presented improved or modified SPT $N$ values (after treatment) in terms of replacement ratio, and initial SPT $N$ (before treatment) values of the soil.

Ramming action of the falling weight tends to ram the stone into sides of the hole, densifying while reinforcing the ground in addition to compacting the stone column substantially. Thus, densification and reinforcement effects cause modifications in the properties of the in situ soil. This densification and modification effects on the soil parameters of the ground are not uniform over the entire zone of surrounding ground but are functions of the distance from the point of densification. During the process of installation of RGP, the soil adjacent to and in the vicinity of the point of treatment gets densified most. This densification effect decreases with the distance from the point of densification. Densification by RGP causes increase in deformation moduli and decrease in the coefficients of permeability and of volume change.

Murali Krishna et al. (2006) incorporated the densification effect of granular piles, with respect to the variation of flow parameters from the centre of the granular pile, in the analysis of pore pressure generation and dissipation that was originally developed by Seed and Booker (1977). The modified form of the governing Eq. 1 with the inclusion of densification is:

$$\frac{k_s(r)}{\gamma_{w} \cdot m_1(r)} \left( \frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) + \frac{1}{\gamma_{w} \cdot m_1(r)} \frac{\partial (k_s(r))}{\partial r} \frac{\partial u}{\partial r} = \frac{\partial u}{\partial t} - \frac{\partial u}{\partial N} \frac{\partial N}{\partial t}$$

(2)

Murali Krishna et al. (2006) studied the densification effect with respect to the coefficients of permeability and volume change at the near and at the farthest ends of the granular pile, individually and together, on maximum pore pressure variations during an earthquake event. Figures 10 and 11 show the densification effect on maximum pore pressure ratio with respect to coefficient of volume change at the near and farthest ends respectively. $R_{ma}$ and $R_{mb}$ are normalized coefficients of volume change due to densification at the near and farthest ends respectively. Figures show that densification effect minimises the pore pressure ratios. Figures 12 and 13 show the densification effect on maximum pore pressure ratio with respect to both the coefficients of permeability and volume change at the near and farthest ends respectively.

Madhav and Murali Krishna (2007) combined both the densification and dilation effects and incorporated them in the analysis of pore pressure generation and dissipation. They also verified the effect of type of variation with distance on maximum pore pressure ratios and concluded that the pore pressures ratios are not sensitive to the type of variation of permeability with distance. Figure 14 shows the effect of densification with respect to flow parameters at the near end in addition to the dilation effect. It is seen from the figure that the dilation effect reduces the negative effect of the densification effect.
Figure 10. Effect of $R_{ma}$ on $W_{max}$

Figure 11. Effect of $R_{mb}$ on $W_{max}$

Figure 12. Effect of $R_{ma}$ and $R_{fa}$ on $W_{max}$

Figure 13. Effect of $R_{mb}$ and $R_{fb}$ on $W_{max}$

Figure 14. Effect of densification with respect to $R_{fa}$ & $R_{ma}$ and dilation on $W_{max}$

Densification effect on the coefficient of volume change is positive in that the maximum induced pore water pressure ratios get reduced and is sensitive to the type of variation.
considered as pore pressure ratios are lesser for the exponential variation. Densification effect, on the coefficient of permeability alone or in addition to effect on coefficient of volume change, increases the maximum pore water pressure ratios giving a negative effect. The pore pressures ratios are not sensitive to the type of variation of permeability with distance. Densification effect on both coefficients of permeability and volume change result in a either slightly negative or positive effect depending on the degree of densification.

Dilation effect generates negative pore water pressures in the granular piles. The negative pore pressures generated in a dilating gravel drain reduce potential liquefaction induced pore pressures by permitting faster rates of dissipation, and hence enhance liquefaction mitigation. The negative effect of the densification (reduction in permeability) is offset by the dilation effect thus proving the effectiveness of granular piles in liquefaction mitigation. It is recommended that the densification and dilation effects should be considered while designing granular pile/stone column treatment for liquefaction mitigation.

**REINFORCEMENT EFFECT**

Granular piles, installed in to a very dense state, are not prone to liquefaction and replace a significant quantity of in situ liquefiable soil and reinforce the same. The very high deformation modulus and stiffness of the granular pile material provide reinforcement for the in situ soil and offer another mechanism to mitigate liquefaction. Stone columns installed in loose sands are effective in damage control as they are installed to a relative density (more rigid/stiff) well above the density of the surrounding soil and hence increase the ability of the layer to withstand the earthquake forces more efficiently.

Poorooshasb et al. (2006) and Noorzad et al. (2007) demonstrated the reinforcement effect of stone columns while analysing their performance during an earthquake. They proposed that the seismic load imposed on the soil is shared between the stone column and the surrounding ground and stone column carries the major load. The geometry of the stone column installation is shown in the Figure 15 while the free body diagram (Figure 16) illustrates the load distribution as a result of reinforcement.

![Figure 15. Definition Sketch for reinforcement effect (after Poorooshasb et al., 2006)](image15.png)

![Figure 16. FBD of the Unit Cell (after Poorooshasb et al., 2006)](image16.png)
Figure 17 shows the performance of the sand layer with void ratio at the critical state (0.9) without and with stone column against the 1989 Loma Prieta earthquake acceleration-time history. It is seen from the figure that the increase in columns diameter did have a dramatic effect on the performance of system measured in terms of the development of the excess pore water pressure and the deformations get reduced significantly.

Noorzad et al. (2007) presented the performance of partially penetrating stone columns during earthquake on similar lines. Figure 18 shows the performance of stone columns with various lengths as a liquefaction hazard mitigation countermeasure. Here the unreinforced sand layer is 7.5 m deep and at critical state with void ratio of 0.9. The stone columns of 750 mm diameter and of different lengths are installed at a pitch of 2.5 m. It is seen from the figure that the effect of column length is obvious in mitigating liquefaction.
SUMMARY

Liquefaction is the most hazardous damage during an earthquake. Among various remedial measures available, installation of gravel drains/Granular piles is the most widely adopted method for liquefaction mitigation. Granular piles provide drainage facility, densify the surrounding soil and reinforce the in-situ ground. Different mechanisms take place in the functioning of the stone columns during the process of liquefaction mitigation. The major fundamental mechanism is drainage by which developed excess pore water pressures get dissipated almost as fast as they get generated. The other mechanisms include storage, dilation, densification and reinforcement. While studying the performance of the gravel drain system all the mechanisms along with their mechanics are to be considered.

REFERENCES


