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ON THREE-STAGE MITIGATION OF LIQUEFACTION-INDUCED HAZARDS

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ABSTRACT

This paper describes three-stage mitigation of earthquake-induced hazards with special attention to geotechnical and liquefaction problems. Different from the conventional factor-of-safety approach, this approach requires more knowledge on damage mechanisms and choice of an appropriate mitigation measure. By practicing mitigative measures and assessing analytically the effect of mitigation, it will be possible to save cost as well as to achieve safety in a broader range in the public. At the end, geotechnical achievements in the field of post-earthquake emergency action will be described.

Keywords: liquefaction, displacement, mitigation, model test, prediction

1. INTRODUCTION

The recent trends of engineering are oriented towards cost-performance issue. In the field of geotechnical earthquake engineering, the situation is similar, in which emphasis is placed on the effectiveness of any damage mitigation measure as well as their cost. In the traditional idea on liquefaction, it has been aimed to prevent the onset of liquefaction and to thereby achieve safety from a liquefaction hazard. It is important, however, that some structures can maintain their safety or function even if any liquefaction occurs in foundation. A typical example is a lifeline in which an idea of network is valid and, even if one particular pipeline is damaged by liquefaction, remaining pipelines can continue the required service. A road network may be another example. Moreover, the seismic performance of a river dike is considered satisfactory even if a limited extent of subsidence occurs during an earthquake, as long as the dike is restored within a reasonable time. It should be recalled that construction of more resistant structure requires higher cost. It is therefore essential to examine the required level of seismic safety / performance. In this context, a (seismic) performance matrix (Table 1) is often cited in which required levels of earthquake safety are tabulated in accordance with the importance of structures and the level of design earthquakes.

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Table 1. Example of seismic performance matrix.

	Level of design earthquake	
Importance of structure	At least once in life span of structure	Very rare event (return period>hundreds of years)
Very important Standard	Very small damage Small and restorable	Restorable/continued service Avoid collapse (reduced victim

2. REVIEW OF DAMAGE DURING PAST EARTHQUAKES

Firstly, the section compares the conventional kind of earthquake damage. Figures 1 and 2 illustrate the consequence of earthquake in 2005 which hit Kashmir region, northern Pakistan. It can be seen that shaking completely destroyed the shape and function of structures. Most traditional structures, inclusive of natural slopes, have brittle nature which induces total failure once yielding occurs. Restoration requires removal of debris and re-construction which require long time and cost.

Figure 1. Balakot town of Pakistan which was totally destroyed by earthquake in 2005

Figure 2. Slope failure and natural dam near Chakar in Pakistan during the Kashmir earthquake in 2005

There is, however, a situation in which geotechnical structure has an advantage over other kinds of structure. Figure 3 illustrates a quick construction of emergency road after the 1999 ChiChi earthquake in Taiwan. In this earthquake, fault movement destroyed many bridges, and, in the site of this figure, emergency transportation was made possible by constructing this detour road. Thus, earth structure has this advantage of quick construction / restoration. Figure 4 demonstrates a collapse of road embankment. Although the previous section proposed an idea of network for cost-efficient principle, the 2004 Niigata-Chuetsu earthquake destroyed all the roads in the epicentral area, and the idea of network did not work properly. This adverse situation made extremely difficult and delayed the restoration of the area. Thus, those who are engaged in seismic safety issue of public structures should bear in mind the importance of structures in local or regional societal (economic) sense.

Figure 3. Quick construction of detour road after 1999 ChiChi earthquake in Taiwan

Figure 4. Fatal collapse of rural road after 2004 Chuetsu earthquake in Japan

3. ESSENCE OF LIQUEFACTION-INDUCED DAMAGE

Figure 5 illustrates the significant distortion of a gravity quay wall in Kobe Harbor which was induced by a combined effect of strong shaking (inertial force) and development of pore water pressure in the foundation as well as in the backfill. The significant displacement (several meters) and tilting of the wall made the crane get derailed and backfill surface subside. This situation stopped the function of the harbor for many months. Since the idea of quick restoration did not work, the harbor client went to other harbors and hardly came back. Another example of unacceptable deformation due to liquefaction is shown in Figure 6 in which a manhole of sewage system floated. Thus, service to local sanitary environment was intervened by liquefaction.

Figure 5. Heavy distortion of quay wall in

Figure 6. Significant floating of sewage

Kobe Harbor in 1995 manhole at 2004 Tokachi-oki earthquake

Figure 7 illustrates another example of unallowable deformation. The subsidence and slope failure of Yodo River dike in Osaka City made flooding of the back area very likely. Since the main function of river dikes is to prevent flooding, significant subsidence may be fatal, while minor subsidence is not serious and can be restored within a reasonably short time (Figure 8). Thus, the essence of liquefaction-induced damage lies in the extent of residual deformation/displacement and its significance is evaluated in terms of impacts on social and economic issues. Note that liquefaction has hardly killed people except the collapse of tailing dam at El Cobre of Chile in 1960s.

Figure 7. Significant subsidence of Yodo River dike in 1995 (Ministry of Construction)

Figure 8. Minor liquefaction near dike of Tokachi river in 2004. earthquake

4. MITIGATION OF LIQUEFACTION-INDUCED DEFORMATION

While the main aim of liquefaction-mitigation measure is to reduce the residual deformation to an allowable extent, the conventional approach has been more effective. Compaction of sand or installation of drains aimed at preventing onset of liquefaction under design earthquake motion. Past earthquakes verified the reliability of those measures. For example, the central part of Kobe Port Island did not develop liquefaction due to a variety of soil densification measures (Yasuda et al., 1996). For economical densification, dynamic consolidation in terms of free fall of heavy weight was proved useful in Lu-Kang site of Taiwan in 1999 (Figure 9). Gravel drains prevented liquefaction and heavy subsidence in Kushiro Harbor in 1993 (Figure 10).

No liquefaction because of densification

Figure 9. Effects of dynamic compaction in Lu-Kang of Taiwan during 1999 ChiChi earthquake

Figure 10. Minor subsidence in Kushiro Harbor in 1993 in spite of heavy seismic shaking

More difficult aim has emerged in the recent times in which liquefaction-induced damage has to be mitigated in "existing" structures. Although these structures were designed to resist previous design earthquakes, the increased intensity of design earthquake in the recent times requires reinforcement of those structures. Since the ground surface is occupied by existing structures, it is practically impossible to bring in big tamping machines or to cause ground vibration for densification. Figure 11 illustrates ongoing densification of soil under an existing building. This work was carried out by compaction grouting from a basement of the building. Figure 12 shows injection of silicate grout close to an existing harbor structure immediately behind a quay wall.

Figure 11. Compaction grouting in basement of existing building

Figure 12. Soil improvement by means of silicate grout under existing structure in the vicinity of quay wall

The application of conventional approach such as densification and grouting as described above is often costly near an existing structure. It is more economical and cost efficient to allow liquefaction but to reduce the liquefaction-induced deformation to a reasonably small extent. This point is particularly important in embedded lifelines for which the concerned ground belongs to other agency / public and the lifeline industries are not allowed to modify the soil conditions. Inexpensive mitigation measure is needed for river dikes as well for which the total length is very long, available financial resource is limited, and the allowable deformation is relatively large.

Figure 13 illustrates an example of steel pipe installation in front of an existing bridge pier. The steel pipe wall prevents lateral displacement of soils around the pier and protects it from the significant earth pressure induced by soil movement. Note that the use of steel pipe wall does not try to prevent liquefaction in the surrounding subsoil. It is attempted therein to reduce the ground displacement by installing a rigid wall. The effect of underground wall is supported by the good performance of Yodo River dike in Takami site (Figure 14) which is located to the upstream direction of the heavily damaged dike in Figure 7. To understand the reasons for different seismic performances of dikes in these two areas, Figure 15 illustrates the subsoil conditions as well as the depth of sheet pile walls. Note that the sheet pile walls under the dike were installed not to prevent liquefaction problems but to reduce seepage flow under the dike. The situations in the Takami site are characterized by three features; different soil conditions (deltaic alluvial soil which is different from younger reclaimed soil in Torishima site), the length of sheet pile wall, and the existence of fill on the river side of the dike. Since more aged soil has less possibility of liquefaction, the soil condition may be the cause of different dike performance. Moreover, the thickness of alluvial sand (deltaic deposit) which is prone to liquefaction is approximately 10 meters along the river channel. Thus, the sheet pile in the Takami site reached the bottom of this sandy layer, and was probably able to prevent the lateral displacement of the foundation sand towards the river channel. The mitigative mechanism of underground walls is illustrated in Figure 16 in which the subsidence of the overlying dike is associated with the subsoil movement in the lateral direction. Installation of walls can reduce the lateral soil movement and consequently the subsidence of the overlying embankment.

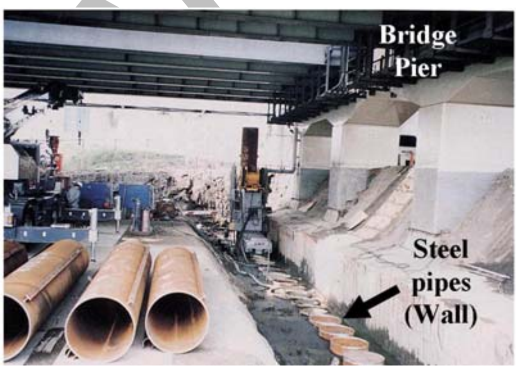


Figure 13. Sheet pile wall as mitigation

Figure 14. Minor distortion of Yodo River in

dike of lateral soil movement (Photo by Prof. S. Yasuda).

Takami site of Osaka City (in contrast with damage in Torishima site in Figure 7).

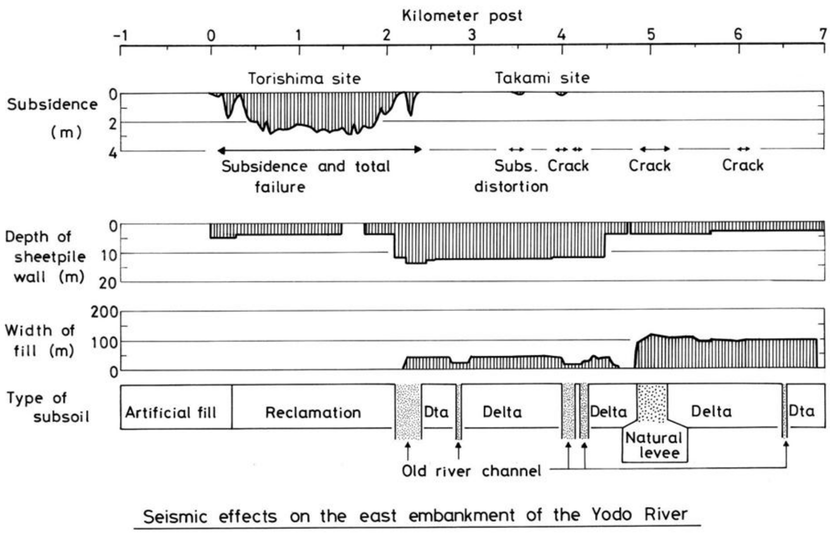


Figure 15. Variation of depth of sheet pile wall and width of river-channel fill along Yodo River

Another reason for different behaviors is the existence of fill on the river side of the dike. Since the Takami site (Figure 14) had a fill which measured 50 m in width, its weight reduced the potential of lateral sliding. This is in good contrast with the dike in Figure 7 that had no such a fill. Figure 17 shows another evidence of wall effects. The foundation of the building on the left side was surrounded by a wall which was constructed upon excavation of the basement. Since this wall existed when the building was shaken by the 1995 Kobe earthquake, the building was not affected by the soil movement caused by a nearby quay wall deformation.

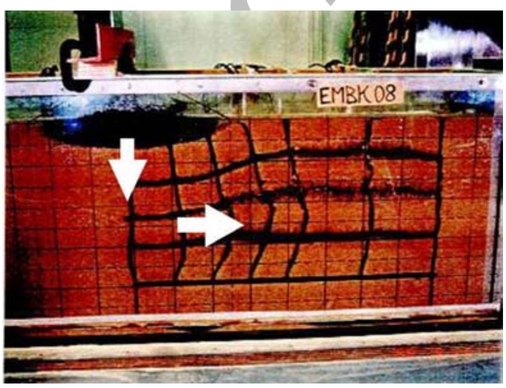


Figure 16. Displacement of liquefied subsoil under embankment model (Mizutani et al., 2001)

Figure 17. Protected building near quay wall in Kobe Harbor (Photo by Prof. J.Koseki)

5. SIGNIFICANCE OF MITIGATION OF GROUND DEFORMATION WHILE ALLOWING ONSET OF LIQUEFACTION

The use of underground wall for reduction (not prevention) of liquefaction-induced damage helps improve cost-performance issue. This philosophy is understood from the view point of three-stage damage mitigation; mitigation at source, on path, and emergency action. Figures 18 and 19 demonstrate examples of mitigation which are practiced in the field of debris and erosion control. Since debris is produced in weathered or unstable mountain slopes, reinforcement and stabilization of slopes is the source-mitigation (Figure 18). This idea, however, cannot be practiced at all the unstable mountain slopes. Therefore, dams are constructed to catch migration of debris in river channels (Figure 19). When river water level rises significantly during heavy rainfall, moreover, warning and/or evacuation order is issued as an emergency measure. This text is attempting to show that these three measures can be conducted in geotechnical earthquake engineering as well. An example is taken of liquefaction problems.

Figure 18. Slope reinforcement to prevent its failure and production of debris (mitigation at source)

Figure 19. Erosion control dam to stop mi-gration of debris to downstream area (mitigation on path).

Model tests on the mitigative effects of underground walls have been undertaken on a variety of situations. The first testing concerns the subsidence of embankment or river dike which rests on liquefiable subsoil. Figure 20 illustrates a 30-G centrifugal model in which two sheet pile walls were installed under the foot of the slopes. The bottoms of the walls were fixed at the base in order to prevent translation and rotation. This reproduces the reality in which walls penetrate into an unliquefiable soil layer. As illustrated in Figure 16, those walls were intended to decrease the lateral displacement of liquefied subsoil, reducing consequently the subsidence of the surface embankment. Figure 21 shows the appearance of the model after subsidence. The walls developed their bending stiffness to reduce lateral displacement of subsoil. As the subsidence of the embankment shows, walls cannot perfectly prevent the subsidence, because soil movement between two walls (Figure 22) cannot be prevented and is still able to induce subsidence.

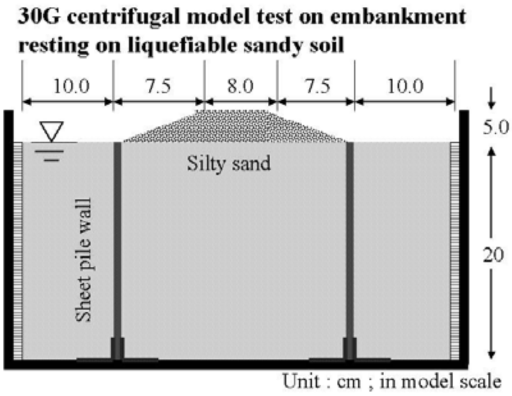


Figure 20. Centrifugal model test on mitigation of subsidence of embankment

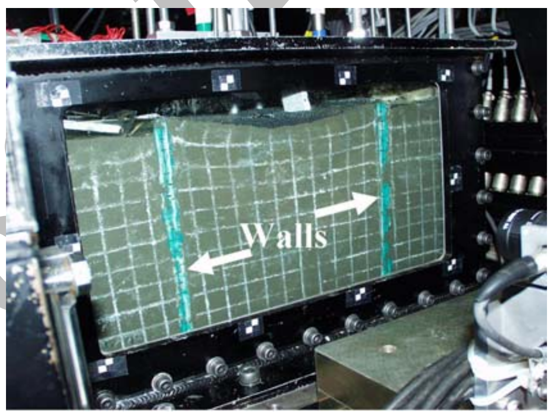


Figure 21. Embankment model after subsidence (30-G model)

Figure 23 illustrates the time history of base shaking in the test on embankment. Excitation took place three times with decreasing the amplitude in order to reproduce the effects of aftershocks in reality and also to study the effects of continued shaking on deformation of liquefied sandy ground. Accordingly, the time history of subsidence in Figure 24 consists of three stages of development. Deformation of subsoil thus occurred only when shaking was ongoing. While two kinds of underground walls were employed in the present tests (sheet pile wall and compacted sand wall), both measures were able to reduce the subsidence.

Grouting can construct an underground near an existing structure as well. Figure 25 shows the injection of silicate liquid slowly and uniformly into sandy ground. This procedure does not employ any big equipment and does not cause vibration. Figure 26 demonstrates an excavated shape of grouted sand. This effect of uniform seeping is permanent without weathering and is not poisonous to human health either.

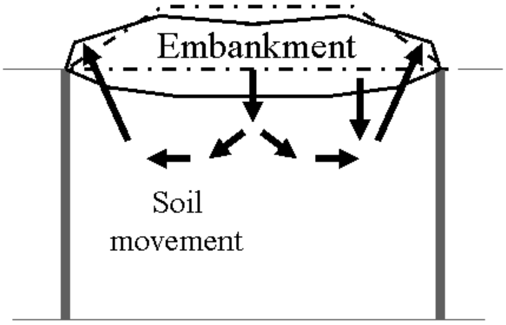


Figure 22. Subsidence of embankment induced by soil migration between two underground walls.

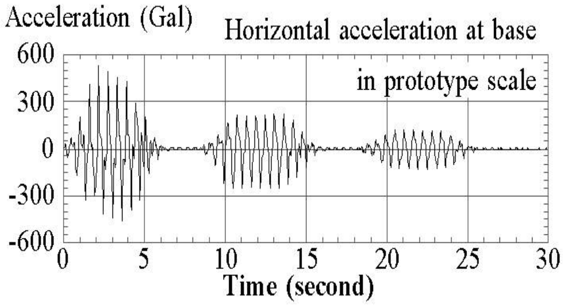


Figure 23. Time history of base acceleration in test of Figure 19

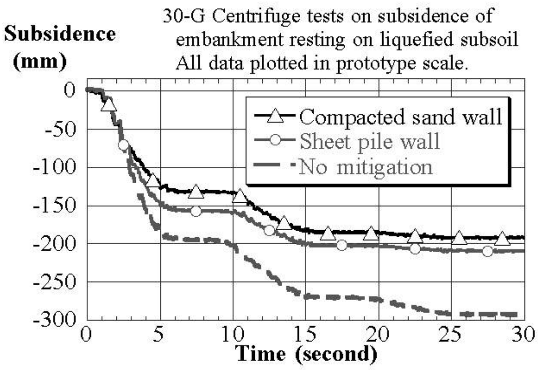


Figure 24. Time history of subsidence at bottom of surface embankment.

Figure 25. Seeping of silicate liquid into sandy ground

Towhata and Kabashima (2001) conducted triaxial shear tests to examine the improved behaviour of grouted sand. Triaxial compression and extension tests in drained manner (Figure 27) revealed that rigidity of sand was improved by this grouting. Figure 28 further shows the increased liquefaction resistance of grouted sand. Although cost is high, this measure is able to improve sand under difficult situation where ground deformation or vibration is not allowed by existing structures. To examine the mitigation of subsidence by this grouting, 1-G model tests were conducted. Figure 29 illustrates a model of liquid storage tank with underground walls made by grouting. Base shaking with 500 Gal and 10 Hz with 20 cycles caused different extent of subsidence in accordance with the type of walls (Figure 30). Although grouting of entire subsoil exhibits the best performance, grouting with limited width achieved satisfactory results as well.

Figure 26. Excavation of subsoil grouted by seeping of silicate liquid.

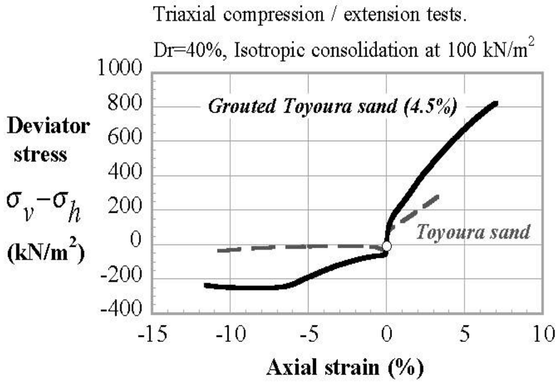


Figure 27. Stress-strain behavior of sand improved by seeping grout

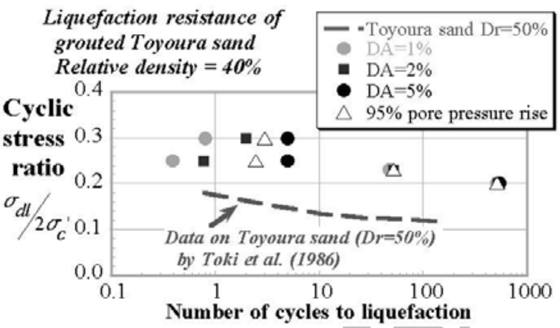


Figure 28. Liquefaction resistance of sand improved by seeping grout.

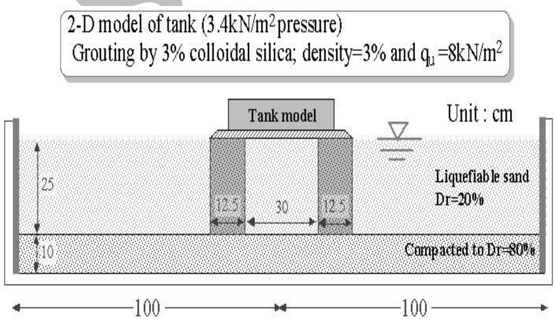


Figure 29. Configuration of 1-G model on subsidence of tank.

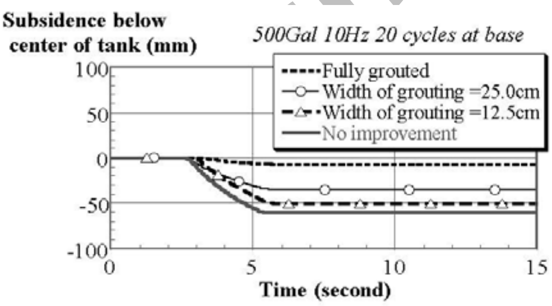


Figure 30. Time history of subsidence of tank model with and without grouted walls.

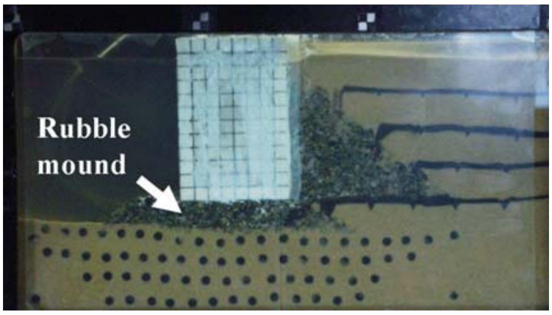


Figure 31. Deformed shape of gravity quay wall model without mitigation(30-G centrifugal test)

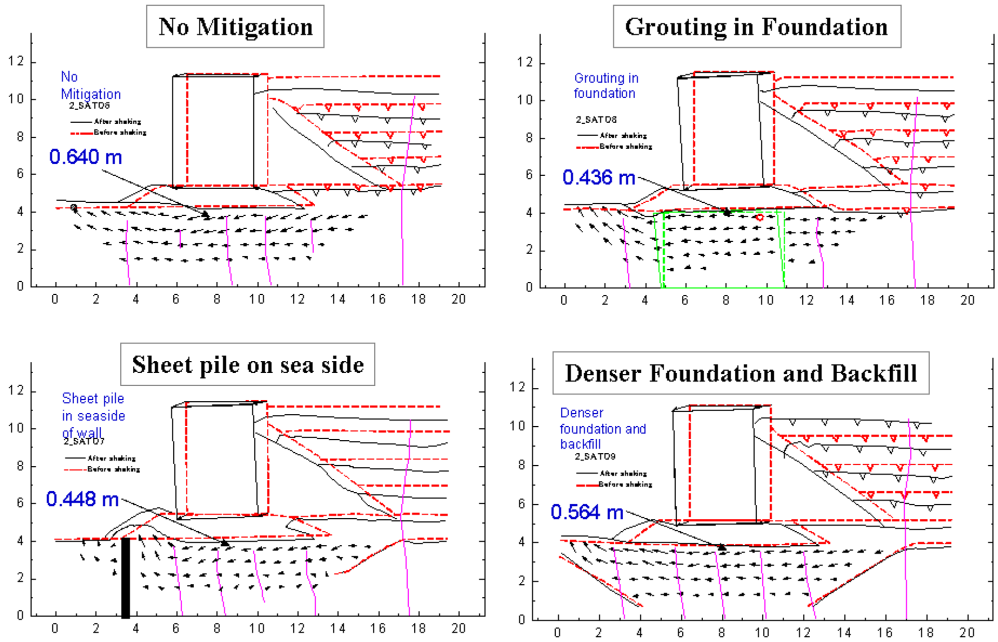


Figure 32. Deformation of foundation sand under gravity quay wall obtained by 30G centrifugal tests)

It seems that grouting can be applied to seismic reinforcement of a gravity quay wall. This study concerns a quay wall which rests on soft marine clay. Since the bearing capacity of this soft soil is not sufficient to bear a heavy quay wall, this clay is replaced by sandy materials. Thus, liquefaction or at least development of pore pressure and loss of shear resistance is possible during strong earthquakes upon strong shaking. Softening of the foundation sand immediately induces large distortion of the overlying quay wall and, in the case of Kobe Harbor (Figure 5), the service of the harbor stopped for a long time and clients were lost. Note that there is another cause of the large distortion which is the dynamic earth pressure exerted by the backfill soil behind the wall.

A series of shaking table tests were conducted in a 50-G centrifugal environment. Base shaking was of 430 Gal in amplitude, 2 Hz in frequency, and 20 seconds in duration time; all expressed in the prototype scale. Figure 31 indicates the distortion of a model without mitigation. The black dots under the quay wall are markers that indicate displacement of the sand replacement. After shaking, the foundation sand moved out towards the sea and the quay wall model translated and tilted in the same direction. Thereinafter three more tests were carried out in order to examine the effects of several mitigative measures (Figure 32). The first mitigation was the seeping grout under the quay wall. The second one was a use of sheet pile wall in front of the wall which was intended to prevent lateral displacement of the foundation sand. The third mitigation was the densification of both foundation and backfill. Although this measure is not of practical value because it is too costly, it was examined for comparison purposes.

The consequent deformation in the foundation was observed by using marker locations and was illustrated in Figure 32. It is indicated by the displacement numbers in the figure that all the three mitigative measures reduced the lateral displacement at the top of the foundation sand. This mitigative effect was particularly important in the case of sheet pile wall and grouting. The displacement at the top of the quay wall was, however, similar in all tests. This is because shear deformation in the rubble mound (see Figure 31) was greater and the tilting of the quay wall body was greater as well when foundation sand did not deform very much (Figure 33). Probably the stabilized foundation sand increased the inertial effects and dynamic earth pressure behind the wall.

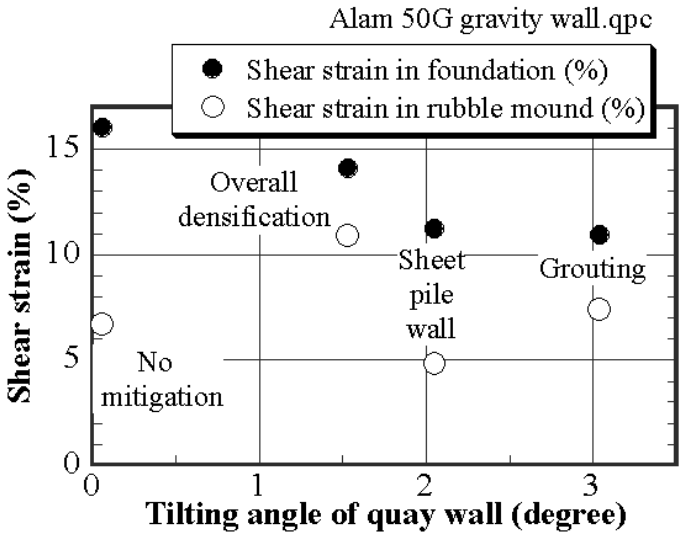


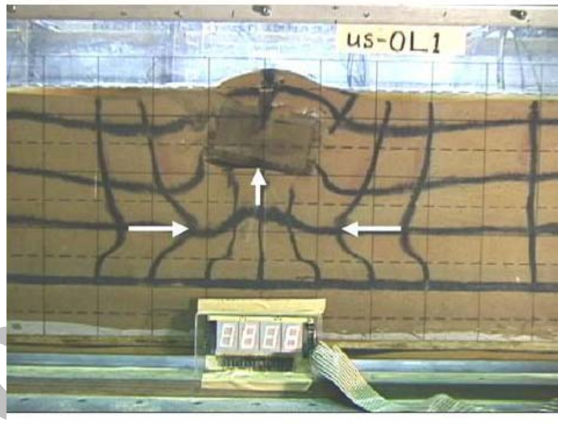
Figure 33. Effects of mitigative measures on deformation of quay wall model.

The last example of an underground wall concerns floating of an embedded structure. In addition to floating of sewage manhole (Figure 6), a bigger water treatment tank floated during the 1964 Niigata earthquake and also in 2004 Niigata-Chuetsu earthquake. In this regard, the author carried out 1-G shaking model tests on mitigation of a bigger structure such as underground parking lot, shopping mall, subway, and highway. Figure 34 illustrates the configuration of the model. The configuration after shaking (Figure 34b) clearly indicates that the floating was accompanied by the inward movement of liquefied subsoil under the structure and that the structure was pushed upwards by this sand movement. It is therefore inferred that floating may be mitigated by preventing soil movement by installation of underground walls beside the structure.

Figure 35 illustrates the appearance of a model with sheet pile walls on both sides. Although floating still occurred, its magnitude was drastically reduced. Figure 36 compares the effects of three mitigative measures on the magnitude of floating. In addition to an ordinary sheet pile wall, tests were run on walls made by densification of soil and special sheet pile wall with drainage pipe (Figure 37). It is found there that floating is reduced to 10% or less by installing suitable mitigation measure.

By comparing Figures 24, 30, and 36 it is detected that mitigation of floating is more effective than that of subsidence of tank and embankment. The reason for this is illustrated in Figure 38. When liquefied soil tries to come in towards the underground structure, sheet pile walls are pushed inward as well. While bending stiffness is the source of resistance of walls against soil movement, the stiffness of walls is further increased by the rigidity of the underground structure. In case of subsidence, on the contrary, sheet pile walls are simply pushed outwards and there is no such additional stiffness. It seems thus promising to install walls around existing structures in order to reduce floating.

(a) prior to shaking



(b) after shaking and liquefaction

Figure 34. 1-G model test on floating of underground structure

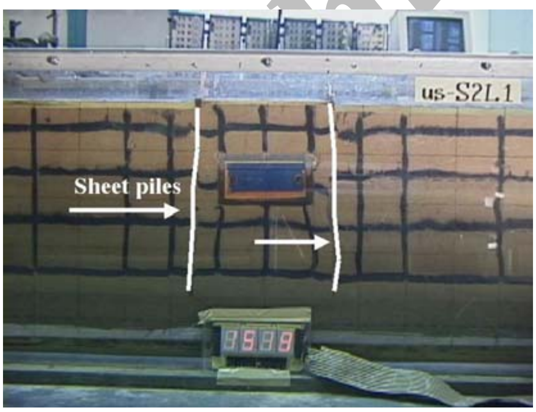


Figure 35. Reduced floating of underground structure model after liquefaction

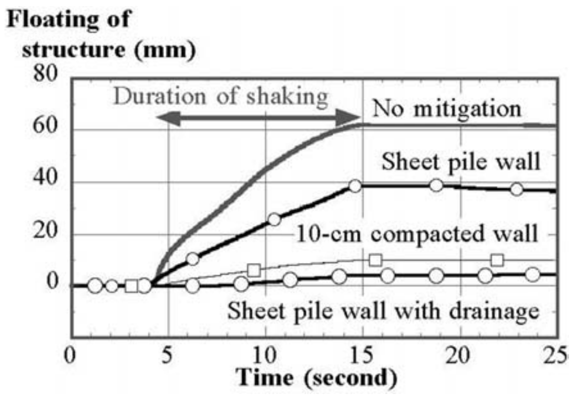


Figure 36. Comparison of floating of underground structure models with and without mitigation

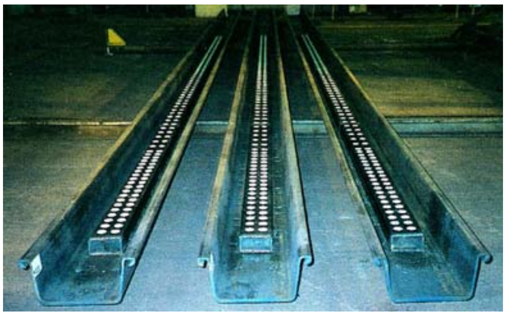


Figure 37. Sheet pile wall with drainage pipe

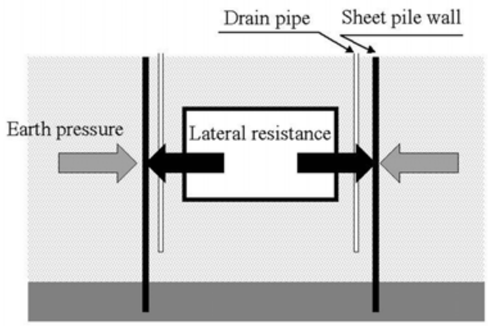


Figure 38. Cause of effective mitigation in floating of underground structure

6. DECISION MAKING ON ALLOWABLE SEISMIC DISPLACEMENT AND DEFORMATION

There is a trend in the recent times that design principles shift from the conventional factor-of-safety principle to the performance-based principle. One important reason for this situation is that stronger accelerations are recorded during recent earthquakes (Figure 39) and the limited soil strength cannot achieve factor of safety greater than 1.0 under the increasing design earthquake load. It is therefore aimed to allow the factor of safety less than 1.0 but still keep the residual deformation within a small extent.

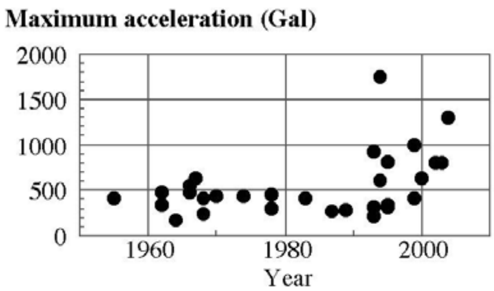


Figure 39. Maximum accelerations which have been recorded during past earthquakes

This seismic performance-based design requires the value of allowable deformation to be decided. It is however difficult to decide this deformation directly. This difficulty comes from the fact that efforts for restoration of damaged geotechnical structure such as fill are not much different whether the deformation is 30 cm or 40 cm. Furthermore, when a road embankment has two lanes for example, the function of the road is still maintained if one of the lanes survives an earthquake and the other part is damaged; one lane can be used for emergency traffic. The above remark implies that the magnitude of allowable seismic deformation depends not only on the engineering issues such as strain or yielding but also on the social issues. In this respect, a special study was conducted by a research committee of JSCE by inquiring questions to engineers and administrators who had hard time in restoration of geotechnical structures which were damaged by big earthquakes in 1990s. For details, refer to Towhata (2005).

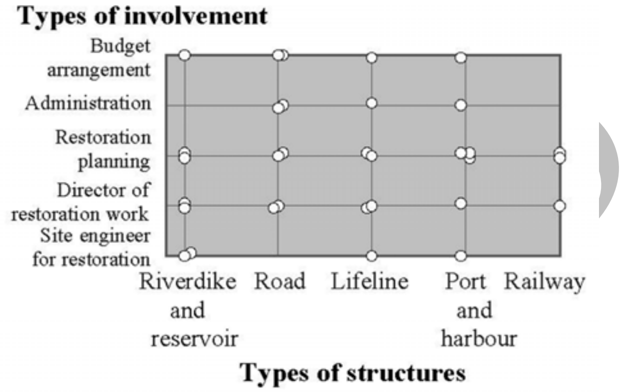


Figure 40. Distribution of interviewed people according to job and kind of concerned structures

The essence of the inquiry study was as what follows;

1) The questions were asked to those who worked for restoration of a variety of structures (Figure 40). However, clients and passengers of damaged facilities were not included because their opinions are often too conservative (e.g., transportation service should be maintained even after a very strong earthquake in order to help family get together quickly) and requires unnecessarily resistant design.
2) The experience of hard time concerning not only the on-site restoration work but also budget arrangement and administration gives those people reasonable idea about the allowable extent of damage.

Accordingly, one of the questions on what factor governs the extent of allowable deformation was answered as summarized in Table 2. The majority of people answered that human life is the most important issue, considering that design should aim to protect people's life as the most important issue. This is certainly agreeable. However, damage of geotechnical structures hardly kills people (most victims during earthquakes are killed by collapse of houses and sometimes landslide). It is therefore reasonable to pay attention to the second important issue that is "negative effects to the public."

Examples of negative effects are such as long-term closure of a bridge and its approach road as well as no supply of water and gas. A more serious negative effect was caused by the heavy distortion of quay walls in Kobe Harbor which made clients go to other harbors without return and the local maritime economy was substantially affected. On the other hand, Table 2 shows that restoration cost is least important according to the idea of the inquired people. This idea seems to be typical in public sectors.

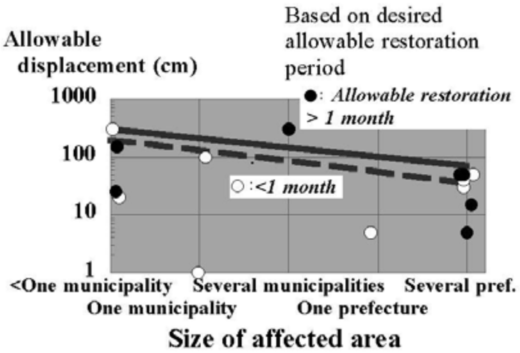


Figure 41. Relationship between allowable displacement and size of area to be affected in terms of allowable restoration period.

Table 2. Factors that affect the allowable displacement.

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

Examples of negative effects as described above suggest that negative effects depend not on the extent of deformation but more directly on time issues; pending of service for a long time creates many problems in the public. Another important aspect is the size of affected population or economy. Negative effects to the whole nation are more serious than those to a small village. In this regard, the present study takes into account the size factor in terms of the size of the affected area. Certainly this is the first attempt and more detailed study is needed in future by using population or economy.

By considering thus that the negative effects to the public consist of time and aerial size, Figure 41 was obtained. In this figure, people's opinion on the allowable time for restoration (allowable time without service) is summarized in relation with the allowable displacement and size of affected area. To make the idea clear, Figure 41 classified the allowable restoration time into two groups; longer or shorter than one month. This classification is related with the importance of structures (shorter time for important structures) and also the type of design earthquake (longer restoration time for very rare earthquakes).

Figure 41 is intended to help decide the allowable displacement / deformation of geotechnical structures in a rational way. As stated before, direct decision on the allowable deformation is not easy. It is on the contrary easy to decide the restoration "time" such as one week or three months. It is very possible to decide the size of an area which would be affected by damage in concerned structure as well. For example, closure of a national important highway would affect the whole nation, while stopping of local gas supply affects a much smaller area. By combining these aerial and time factors, Figure 41 gives the allowable displacement. In other words, the earthquake-induced deformation should remain within a limit for which the restoration period is shorter than the target restoration time. This basic principle is easy to be followed in practice, although detailed relationship between displacement and restoration time varies with types of structures and other situations.

7. MECHANICAL PROPERTIES OF LIQUEFIED SAND

Performance-based design on seismic behavior of geotechnical structures requires two components to be established. One is the development of a practical method for calculating the earthquake-induced deformation. The design is considered to be satisfactory, if the calculated displacement satisfies the desired level; for example, see Figure 41. The other is an engineering method by which the displacement is reduced to a required level. Since the latter was fully discussed in the previous section, the following section will address the calculation. This section, in particular, introduces the deformation characteristics of liquefied sand under very low effective stress. This knowledge is essential in calculation of liquefaction-induced displacement.

While there is a variety of idea on the mechanical properties of liquefied sand, the author makes use of the idea of (Bingham) viscous modeling. This idea originally comes from model tests by Kogai et al. (2000) in which the drag force to pull a pipe in liquefied ground increases linearly with the velocity (Figure 42). Similar observation has been made from lateral load exerted by flow of liquefied sand on pile (Sesov et al., 2004).

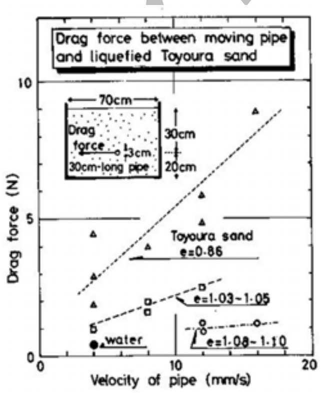


Figure 42. Rate dependency of drag force stepwise quick loading of deviator stress embedded pipe

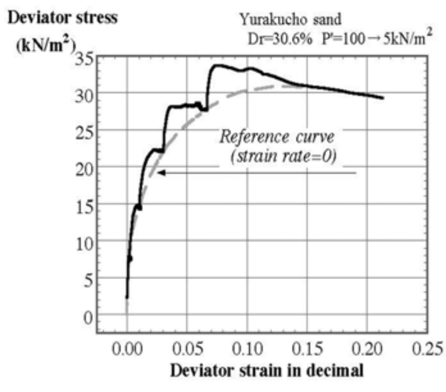


Figure 43. Triaxial compression test with in drained manner (Gallage et al., 2005)

Study of stress-strain (or strain rate) relationship of liquefied sand by model tests has a limitation that the state of stress and strain is not uniform in a model ground. This is an essential shortcoming because behavior of soil is highly dependent on the nonlinear stress-strain states. To obtain more accurate knowledge, triaxial shear tests were carried out under low effective stress in drained manners.

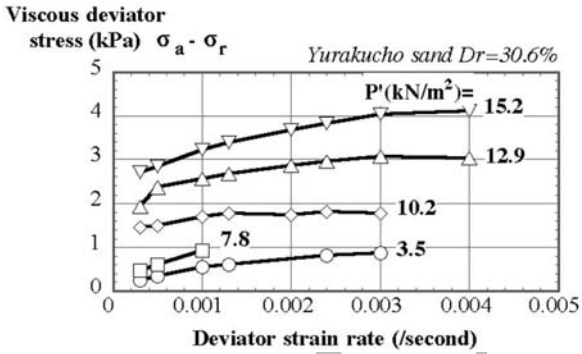


Figure 44. Bingham nature of liquefied sand under low effective stress.

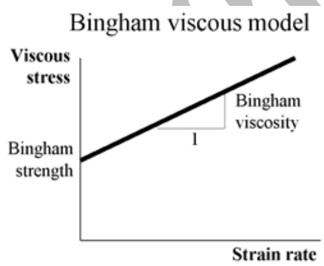


Figure 45. Schematic diagram of Bingham viscous model

In the test of Figure 43, a loose sandy specimen was consolidated under 100 kPa, and the effective stress was reduced to 5 kPa by introducing high back pressure (pore water pressure controlled externally) or running undrained cyclic loading. This procedure reproduces the process of liquefaction in reality. A drainage valve of a triaxial apparatus was then opened and triaxial compression was conducted while maintaining constant the pore water pressure and the lateral effective stress (σ'_3). In the stress-strain diagram of Figure 43, the deviatoric stress, $\sigma_1 - \sigma_3$, was increased by steps followed by constant value. This way of loading was intended to measure creep deformation of sand and thereby to obtain the viscous parameters of sand. The reference curve in the figure was obtained by connecting the stress-strain state at the end of creep. Therefore, this curve indicates rate-independent and frictional nature of sand, and the difference between the measured and reference curves gives the rate-dependent nature.

Figure 44 depicts the viscous component of stress which changes with strain rate. Although there is no proportionality, a linear relationship is reasonable between strain rate and stress. Therefore, it was decided to model this behavior by Bingham viscous model which is expressed by strength and viscous coefficient (Figure 45). These Bingham viscosity parameters were read from experimental data and were plotted in Figures 46 and 47 against the mean effective stress. Since triaxial specimens could not maintain their shape and stability under zero effective stress, test results under low but still positive effective stress were collected, plotted, and were extrapolated towards zero stress in order to assess the nature of liquefied sand. It was found accordingly that the Bingham strength vanishes in completely liquefied state, changing sand to a Newtonian viscous liquid, and that the viscosity coefficient lies in the range of 30 to 100kPa.sec.

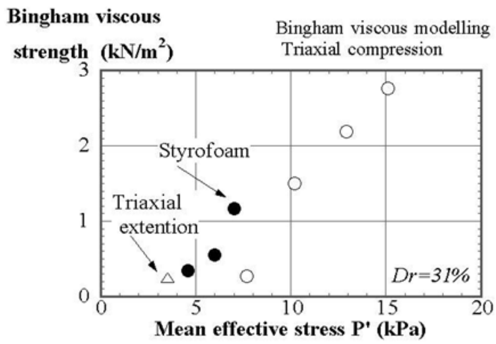


Figure 46. Bingham strength of liquefied sand under low effective stress.

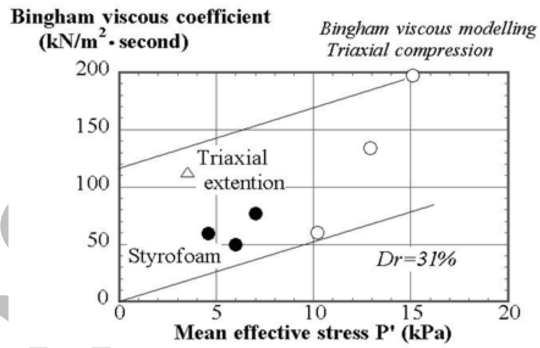


Figure 47. Bingham viscous coefficient of sand changing with effective stress.

8. CALCULATION OF LIQUEFACTION-INDUCED DEFORMATION OF GEOTECHNICAL STRUCTURES

By using the obtained viscous parameters (Bingham strength=zero), a few calculation was conducted. For the analytical details, refer to Towhata et al. (1999). Firstly, the centrifugal test on subsidence of embankment resting on liquefied subsoil (Figure 20) was calculated in Figure 48. The mitigative effects of sheet pile walls are included here. The calculated time history of subsidence seems to be reasonable. Noteworthy is that this calculation requires only a limited number of data such as thickness of liquefiable soil layer and viscosity together with unit weight of soil. The second analysis concerned the floating of an underground structure (Figures 34 and 35). An analysis on the case without mitigation (Figure 34) was conducted by varying the viscosity and the best-fitting coefficient of viscosity was found to be 10kPa.sec (Figure 49) By using this, the case with sheet pile walls was analyzed in Figure 50. Although the agreement is not yet satisfactory, the mitigative effects were somehow reproduced by the analysis.

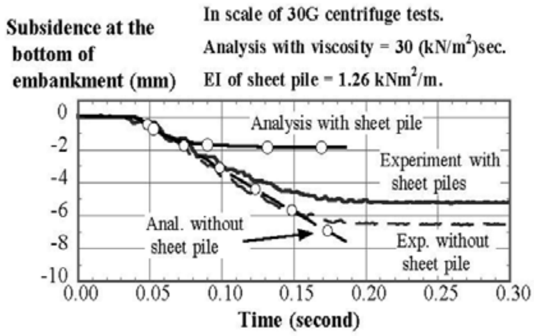


Figure 48. Viscous analysis on subsidence of embankment resting on liquefied subsoil

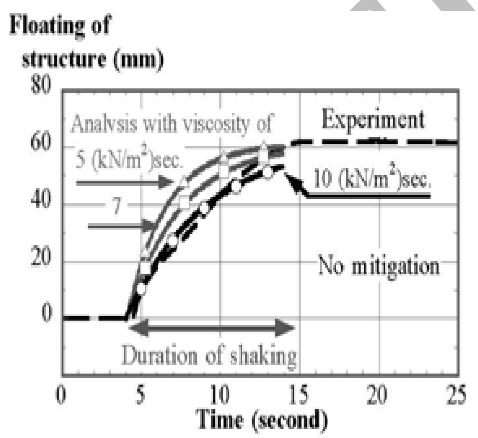


Figure 49. Viscous analysis on floating without sheet pile wall

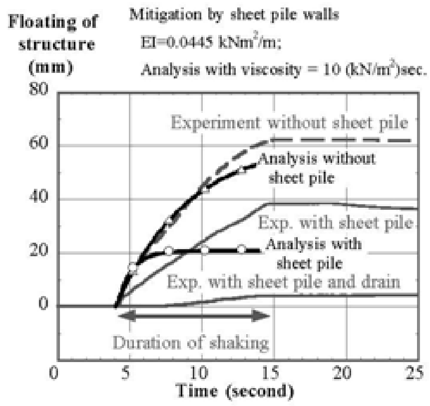


Figure 50. Viscous analysis on floating with mitigation by sheet pile walls

9. EMERGENCY ACTION

Tokyo Gas Company has been installing an emergency action system which monitors intensity of earthquake motion through wireless, makes judgment on significance of pipeline damage, if strong earthquake motion is recorded, and stops gas supply to probably damaged area in order to avoid further damage due to leakage of gas. This safety measure works immediately after a strong earthquake, which is remarkably quicker than conventional human inspection. Being named SUPREME (Shimizu et al., 2006), the emergency safety system monitors the maximum acceleration, Amax, and the spectrum intensity, SI, which is defined by

$$SI \equiv \frac{1}{2.4}\int_{0.1}^{2.5}\left(\text{Velocity response spectrum}\right)dT \qquad (1)$$

In which T stands for the natural period of a structure and the critical damping ratio is 20%. There is an empirical correlation between SI and the number of gas pipeline damage per km (Figure 51).

In addition to being close to the maximum ground velocity, the SI value can help assess the maximum ground displacement, Dmax, during an earthquake (Towhata et al., 1996);

$$D_{max} \approx 2SI^2 / A_{max} \qquad (2)$$

The validity of this equation is verified in Figure 52. Based on model tests and analyses of earthquake motion records, it was decided by Towhata et al. (1996) to use a displacement mode in Figure 53, and, by using the assessed displacement, the thickness of liquefied soil is quickly assessed by

$$H = \left(\pi D_{max}\right) / \left(2 \times 0.01875\right) \qquad (3)$$

Among many examples to validate this formula, the case of Kobe Port Island is illustrated in Figure 54. By deploying 3,700 monitoring stations, SUPREME is expected to help avoid emergency problems due to gas leakage.

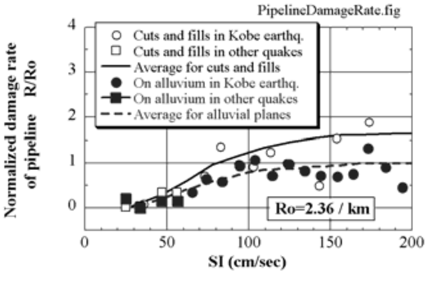


Figure 51. Relationship between number of damage of low-pressure gas pipeline and SI value (after Shimizu et al., 2006).

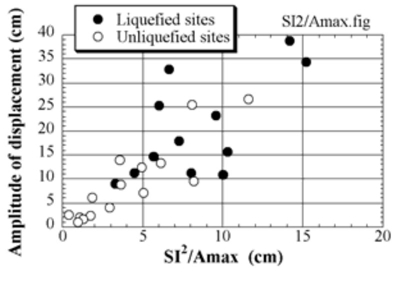


Figure 52. Empirical correlation between ground displacement, SI, and Amax (Towhata et al., 1996)

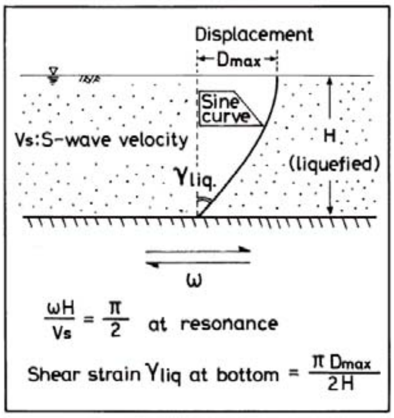


Figure 53. Mode of displacement in liquefied soil

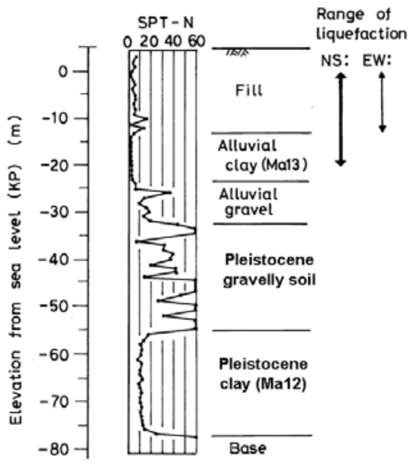


Figure 54. Assessed thickness of liquefaction in Kobe Port Island

10. CONCLUSIONS

This paper describes developments of earthquake geotechnical engineering with emphasis on liquefaction problems. In addition to preventing problems, it is also important to mitigate the extent of damage. This philosophy is otherwise called performance-based approach and is probably able to reduce cost without sacrificing the public safety and convenience.

The performance-based principle requires more detailed knowledge on earthquake motion and soil behavior undergoing earthquake loading. It will thereby be possible to improve the public safety in the field where the conventional approach was not able to function properly due to financial problems. It is further important that post-earthquake emergency action can achieve more goals which the stress-strain approach cannot. The three kinds of safety approach as described above constitute a three-stage safety measure which as a whole achieves more cost-effective extent of public safety during earthquakes.

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