

Geotechnical, Geological and Earthquake Engineering

Susumu Iai *Editor*

Developments in Earthquake Geotechnics

 Springer

Geotechnical, Geological and Earthquake Engineering

Volume 43

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Editor

Developments in Earthquake Geotechnics

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ISSN 1573-6059 ISSN 1872-4671 (electronic)
Geotechnical, Geological and Earthquake Engineering
ISBN 978-3-319-62068-8 ISBN 978-3-319-62069-5 (eBook)
DOI 10.1007/978-3-319-62069-5

Library of Congress Control Number: 2017954317

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The registered company is Springer International Publishing AG
The registered company address is: Gewerbestrasse 11, 6330 Cham, Switzerland

Preface

The earthquake disasters in Japan and New Zealand in 2011 indicated the urgent need for the latest earthquake geotechnics to be deployed for disaster mitigation. This book aims to provide a timely review and summaries of the most recent advances in earthquake geotechnics. It is unique in its content, including a review of developments in the discipline of earthquake geotechnics over half a century, giving readers of this book a solid grasp of this discipline from its inception up to the most recent developments. The book is based on contributions by 18 leading international experts who met in Kyoto in June 13–15, 2016, to discuss a range of issues related to developments in earthquake geotechnics.

This book addresses comprehensive areas of earthquake geotechnics, including performance-based seismic design, the evolution of geotechnical seismic response analysis from 1964 to 2015, mitigation measures against liquefaction, possible inundations in the Greater Tokyo Area due to tsunamis, and a series of effective stress analyses for the Great East Japan Earthquake in 2011.

The compilation and editing of this book coincided with the second phase of activities of Technical Committee 303 (TC303) “Coastal and River Disaster Mitigation and Rehabilitation” (short name “Floods”) of the ISSMGE for the period 2013–2017, within the broader theme “Impact on Society”. The publication of this book also coincided with the fifth anniversary of the FLIP Consortium, established in 2011, where academicians and practicing engineers significantly contributed to the development, advancement, and use of effective stress analysis. This book reflects the activities of TC303 and the FLIP Consortium.

This book will be of interest to advanced researchers and practicing engineers in earthquake geotechnics.

Kyoto, Japan

Susumu Iai

Contents

1	Performance-Based Seismic Design of Geotechnical Structures . . .	1
	Susumu Iai	
2	Evolution of Geotechnical Seismic Response Analysis from 1964 to 2015	25
	W.D. Liam Finn	
3	Past, Present, and Future Developments in Liquefaction Hazard Analysis	51
	Steven L. Kramer	
4	Countermeasures Against Liquefaction	61
	Junichi Koseki	
5	Liquefaction Induced Downdrag and Dragload from Full-Scale Tests	89
	Kyle M. Rollins, Spencer R. Strand, and J. Erick Hollenbaugh	
6	Whole Life Sensing of Infrastructure	111
	Kenichi Soga	
7	Recent Efforts to Mitigate the Impacts of Earthquake Hazard in Indonesia from Geotechnical Engineering Perspective	131
	Hendriyawan, M. Irsyam, M. Asrurifak, I. Meilano, D.H. Natawidjaja, S. Widiyantoro, A.D. Nugraha, L.M. Sakti, A. Sabaruddin, L. Faisal, P.T. Simatupang, B.M. Hutapea, and T. Afriansyah	
8	Possibility of Four Metre Deep Flooding in Densely Populated Low-Land Area of Tokyo Induced by a Mega-Earthquake	151
	Hideki Ohta, Toshiki Aoyama, Toru Shimizu, Shunichi Hamada, and Masahito Nakamura	

9	One Dimensional Shallow Foundation Macro-element	177
	M.J. Pender, T.B. Algie, R. Salimath, and L.B. Storie	
10	Analysis of Liquefaction During 2011 East Japan Earthquake – Part 1: Seismic Ground Behavior in Tokyo Port at the 2011 Off Pacific Coast of Tohoku Earthquake – An Effective Stress Dynamic Analysis Focusing on the Impact of the Aftershock	201
	Yukio Tamari, Junichi Hyodo, Koji Ichii, Takashi Nakama, and Atsushi Hosoo	
11	Analysis of Liquefaction During 2011 East Japan Earthquake – Part 2: Effective Stress Analysis Considering the Permeability of the Ground ~ Liquefaction at Reclaimed Land in the Main and After Shocks of the 2011 Great East Japan Earthquake	219
	Shun-ichi Sawada, Kazuaki Uemura, Minoru Nobumoto, Makoto Yoshida, Shigeru Sato, Yoshiaki Kageji, Yukio Tamari, Junichi Hyodo, Takashi Nakama, Atsushi Hosoo, and Koji Ichii	
12	Effective Stress Analysis of River Dikes During the 2011 East Japan Earthquake	233
	Naoki Morishima	
13	Effective Stress Analysis of Quay Walls During the 2011 East Japan Earthquake	263
	Soichi Tashiro	
14	Modeling the Axial Behavior of Piles	293
	Junichi Hyodo, Yoshio Shiozaki, Akito Sone, Osamu Ozutsumi, and Koji Ichii	
15	Modelling of Cohesive Soils: Soil Element Behaviors	317
	Tomohiro Nakahara, Kyohei Ueda, and Susumu Iai	
16	Modeling of Cohesive Soils: Consolidation and Seismic Response	341
	Osamu Ozutsumi	
17	Large Deformation (Finite Strain) Analysis: Theory	367
	Kyohei Ueda	
18	Large Deformation (Finite Strain) Analysis: Application	389
	Noriyuki Fujii	
	Index	411

Chapter 1

Performance-Based Seismic Design of Geotechnical Structures

Susumu Iai

Abstract The paper gives an overview of performance-based approach for designing geotechnical structures against earthquakes, including remediation of liquefiable soils. Case histories of implementation and performance of remediation measures during past earthquakes are reviewed. The paper discusses the applicability and limitations of the conventional simplified approach for designing remediation of liquefiable soils and how these limitations can be overcome in the performance-based approach that explicitly considers residual displacements and structural strains beyond elastic limit. The paper also presents recent developments in performance evaluation against extremely long duration earthquake motions such as those during the 2011 East Japan earthquake ($M = 9.0$).

1.1 Introduction

Seismic damage to geotechnical structures is often associated with significant deformation of a soft or liquefiable soil deposit. Designing appropriate balance between structural components and foundation components, including remediation of liquefiable soils, is essential to improve the seismic performance of geotechnical structures. The conventional simplified approach based on pseudo-static force equilibrium provides reasonable guidelines for designing geotechnical structures against earthquake, including remediation of liquefiable soils against moderate earthquake motions. However, there are cases where the conventional simplified approach is not readily applicable. For example, for intense earthquake motions, acceptable residual displacements and structural strains beyond elastic limit become the primary design criteria parameters, which are not evaluated by the conventional simplified approach. There are a number of cases where site specific constraints pose difficulties in implementing the liquefaction remediation measures as suggested by the conventional simplified approach. Objective of this paper is to discuss the applicability and limitations of the conventional simplified approach for designing geotechnical structures, including remediation of liquefiable soils, and

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how these limitations can be overcome in the performance-based approach. The paper also presents recent developments in performance evaluation against extremely long duration earthquake motions such as those during the 2011 East Japan earthquake ($M = 9.0$).

1.2 Standard Procedure

Designing geotechnical structures against earthquakes, including remedial measures against liquefaction, can be classified into two broad categories (Port and Harbour Research Institute 1997; PIANC 2001):

- (a) Soil improvement to reduce the potential of liquefaction at foundation soil;
- (b) Structural solutions to minimize damage, including in the event of liquefaction.

In practice, a combination of these two measures is often adopted.

A flowchart illustrating a standard design procedure for designing geotechnical structures against earthquakes, including liquefaction remediation, is shown in Fig. 1.1. Once the strategy has been determined, it is common practice to select a method first, and then to compare the advantages and disadvantages of different solutions for the particular project. Typically, a number of different solutions will need to be assessed before a final decision can be reached. Sometimes, solutions will be combined to achieve the optimum design. The procedure illustrated in Fig. 1.1 is intended to be used as a guide only; it may often be more efficient to screen out remediation methods that are clearly unsuitable at an early stage based on a rough assessment of the likely area to be treated and any project specific constraints. A key to the selection of the method is often found in the process to try to find out the most vulnerable portion out of the entire soil-structure system, rather than to go through the routine process of checking and evaluating the structural or foundation components one by one out of the whole system.

A combination of two or more remediation methods is often very effective. A typical example might be the combination of a low noise/low vibration method, such as the use of drain, combined with compaction around the improved area, to constrain or confine the overall site. At present, however, formal procedures for selecting the most efficient combination of methods for a specific site have not been developed and the engineer must use judgment and experience to reach the optimum solution.

1.3 Conventional Approach

The design requirements for a specific improvement in the geotechnical structures, including the ground improvement area, will be based on an assessment of the mechanism of damage likely to be induced in the structural components essential to

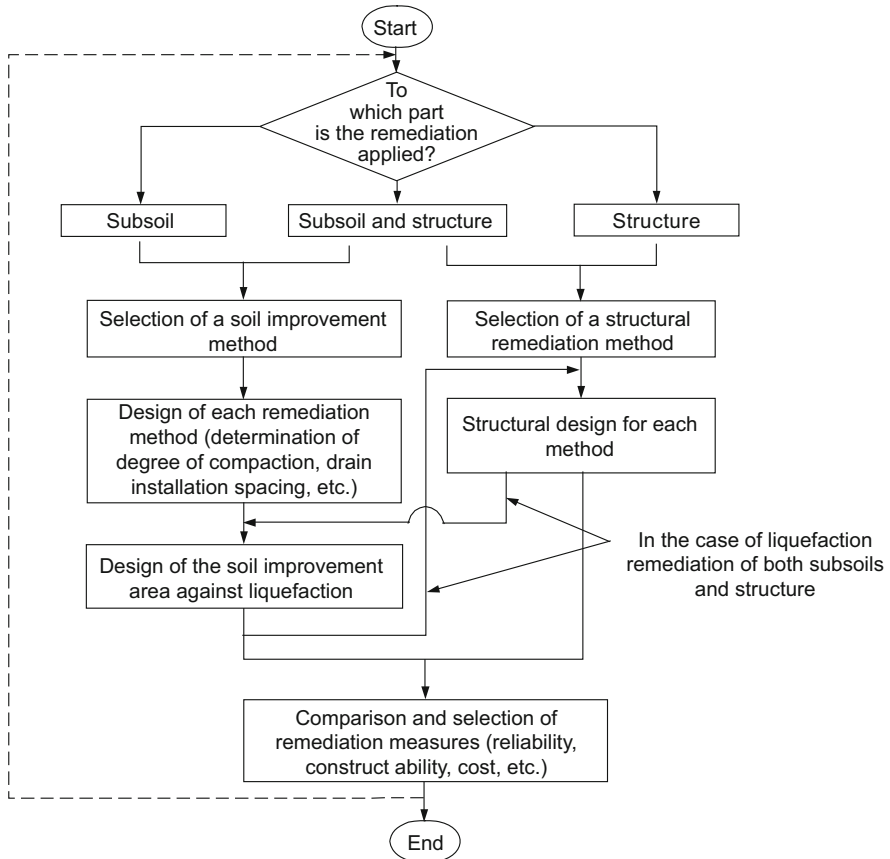


Fig. 1.1 Standard procedure for liquefaction remediation (Port and Harbour Research Institute 1997)

support the soil-structure system or the foundation, often consisting of natural soils by liquefaction. In particular, it is critical to determine whether the damage will be caused primarily by reduction in the shear resistance of the liquefied soil, or by additional external forces due to excessively large displacements. Large displacements of the liquefied subsoil may affect the foundations of existing and new structures, or cause serious damage to lifeline facilities (such as gas, water and telecommunication pipelines and conduits), and special consideration will be necessary in respect of these cases. There are many other cases, however, in which remediation measures against loss of shear resistance will provide benefits against the risk of large displacements; ground compaction being one example.

Generally, even if the soil is predicted to undergo liquefaction over a wide area, it may be possible to limit the area requiring soil improvement to the area that controls the stability of the structure. For example, the zone within the subsoil that contributes predominantly to the stability of spread foundation structures is the part

directly below and immediately around the structure; ground far away from the structure does not contribute to the same extent. A key issue is therefore to establish how far the soil improvement needs to extend laterally from the structure. This can be determined by following the procedure summarized below (Iai et al. 1987, 1991).

The design procedure for a soil improvement area presented below is based around use of the compaction method. The procedure may, however, be adapted for other methods with appropriate modification by considering the appropriate characteristics of the improved soil (such as permeability and cyclic strength/deformation).

1. Propagation of excess pore water pressure into the improved zone

Outside the zone of improved soil, excess pore water pressures will exceed those inside, resulting in a hydraulic gradient driving fluid into the zone of improved soil. This is a complex issue, as the deformation characteristics of dense saturated sand are highly non-linear. As a simplified design procedure, this phenomenon may be addressed as follows:

For excess pore water pressure ratios $u/\sigma'_v < 0.5$, the effect of the excess pore water pressure increase may be ignored because laboratory test data indicate a very small strain generation below this level. For $u/\sigma'_v > 0.5$, however, it is necessary to take into account the effect of excess pore water pressure increase. Shaking table tests and seepage flow analyses suggests that the pore water pressure ratio $u/\sigma'_v > 0.5$ within an area defined by the square ABCD in Fig. 1.2. In this area, the soil shear resistance must be reduced for the purposes of the design. The tests also indicate that an area defined by the triangle ACD exhibited unstable characteristics. This area should therefore be assumed in the design to be liquefiable and treated accordingly.

The exception to this recommendation is when a drain or impermeable sheet or zone has been installed at the perimeter of the improved area in order to shut out the inflow of pore water into the improved area. Under these conditions, the area

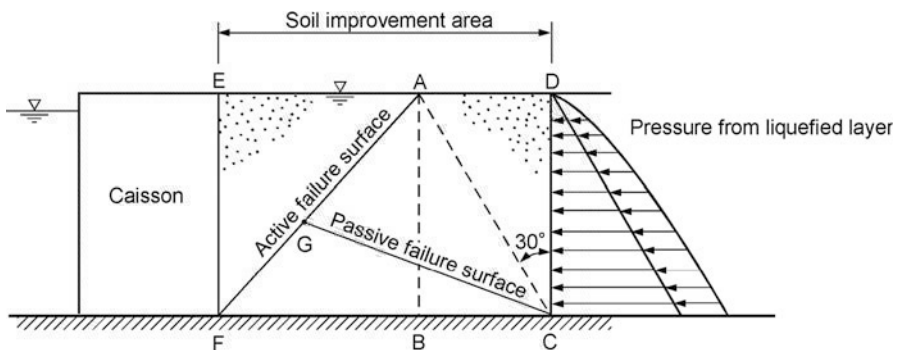


Fig. 1.2 Schematic diagram for investigation of stability with respect to pressures applied from the liquefied sand layer (Iai et al. 1987)

corresponding to the square ABCD need not be included as part of the improvement plan.

2. Pressure applied by the liquefied sand layer

At the boundary between liquefied and non-liquefied ground there is a dynamic force as indicated in Fig. 1.2 and a static pressure corresponding to an earth pressure coefficient $K = 1.0$ which acts on the improved ground due to the liquefaction of the surrounding soils. These forces may greatly exceed the forces acting in the opposite direction from the non-liquefied ground. For a retaining structure backfilled with soil and subject to active static and dynamic lateral earth pressures at EF as shown in Fig. 1.2, the area of soil improvement must be large enough that there is no influence of liquefaction in the active failure zone. To accommodate the net outward force, it is essential to check that sufficient shear resistance can be mobilized along the passive failure surface GC.

3. Loss of shear strength in liquefied sand layer

In the simplified design procedure, unimproved soil of loose or medium relative density should be considered to have negligible shear strength after liquefaction (i.e. the soil is treated as a heavy fluid). Since the shear strength of the improved ground in triangle ACD also cannot be relied upon (see (1) above), then the improvement area should be wide enough to obtain sufficient bearing capacity from the shear resistance along the solid lines EFG and HI in Fig. 1.3.

In practice, lateral pressure from surrounding liquefied sand layers may contribute to the stability of certain structures. Figure 1.3 shows how the dynamic earth pressure may be subtracted from the enhanced static lateral earth pressure (based on an earth pressure coefficient $K = 1.0$) in certain stability calculations. The section on which the pressure from the liquefied sand layer is applied can be assumed to be along the lines GG' or II'.

Although this figure shows only four examples of foundation types, the same principles can be applied to other design arrangements.

4. Design of soil improvement area

Key parameters for any assessment of the area of soil improvement include the angle of internal friction for sands and the anticipated intensity of earthquake shaking. A stability analysis method will also need to be specified. Standard design procedures adopt a simplified, pseudo-static approach to the prediction of earthquake loads and response. Despite this, experience suggests that with appropriate care and selection of parameters, these methods generally provide satisfactory design solutions.

The soil improvement area required for a typical gravity quay wall based on this approach is shown in Fig. 1.2. For anchored sheet pile walls, the zone of improvement in the vicinity of the anchor wall must be carefully considered. For flexible anchor walls, soil improvement must extend to sufficient depth below the tie rod level to ensure that the adequate capacity is maintained to resist the tie rod force.

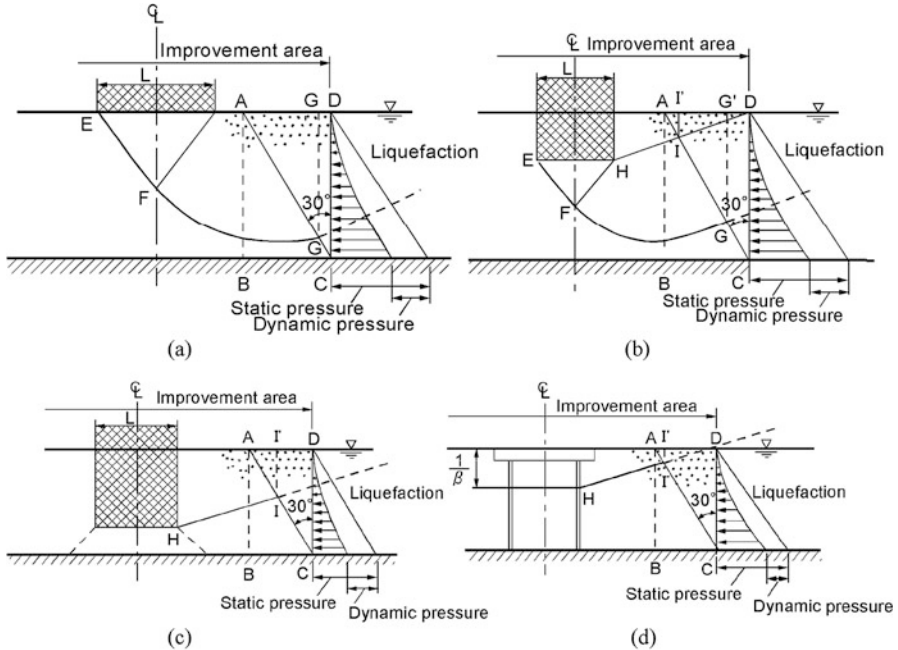


Fig. 1.3 Schematic diagrams for investigation of stability for determining the soil improvement area (Port and Harbour Research Institute 1997; Iai et al. 1987). (a) Shallow foundation. (b) Foundation with shallow embedment. (c) Foundation with deep embedment. (d) Pile foundation

1.4 Performance at Moderate Earthquake Motions

The design procedure based on the conventional simplified approach described in the previous chapter provided a basis for implementing the liquefaction remediation in conventional practice. A number of case histories of implementing liquefaction remediation were gradually increasing. The first opportunity to test the effectiveness and adequateness of the remedial measures against liquefaction was provided during 1993 Kushiro-Oki earthquake of magnitude 7.8 (Iai et al. 1994). Kushiro Port located at 15 km from the epicenter was shaken with a peak horizontal acceleration of 0.47 g. If the spiky wave due to cyclic mobility is filtered out, the effective peak ground acceleration was in the order of 0.3 g.

Many of the quay walls at Kushiro port suffered damage due to liquefaction of backfill sand. The most serious damage to the sheet pile quay walls is shown in Fig. 1.4. This quay wall was of a steel sheet pile type anchored by battered steel piles. The ground consists of loosely deposited backfill sand with a thickness of about 10 m, underlain by medium to dense sand deposit that forms the original ground. Liquefaction at the backfill caused serious deformation in the sheet pile wall as shown by the solid line in this figure. Detailed investigation of the sheet pile wall by diving in the sea revealed that cracks opened in the steel sheet pile wall at an elevation of 4 m below the water level.

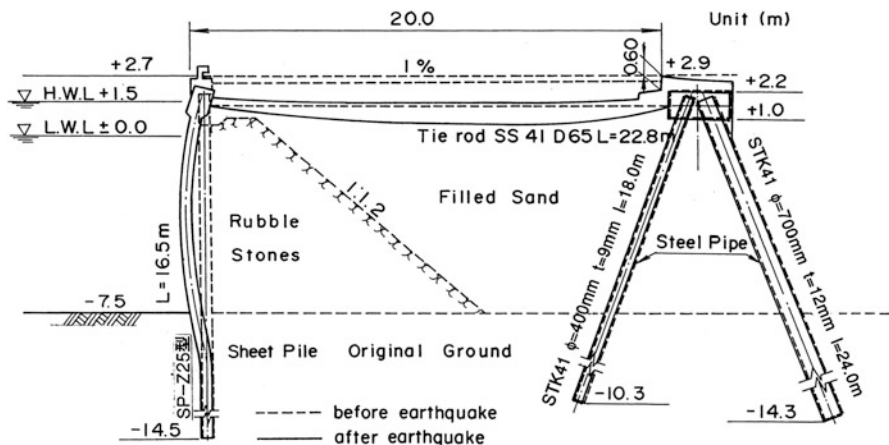


Fig. 1.4 Damaged sheet pile quay wall at South Fishery Wharf, Kushiro Port, during 1993 Kushiro-oki earthquake (Iai et al. 1994)

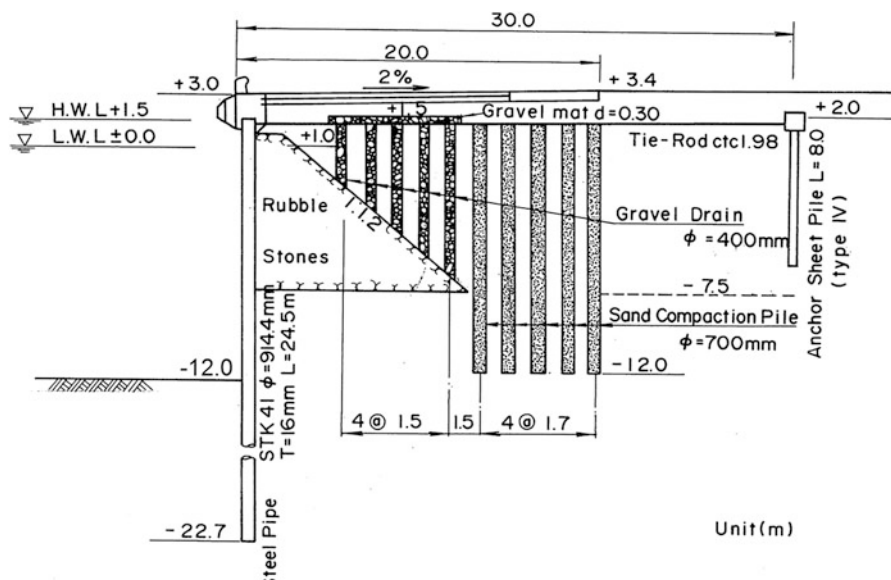


Fig. 1.5 Undamaged sheet pile quay wall at South West Port No.1 Wharf, Kushiro Port, during 1993 Kushiro-oki earthquake (Iai et al. 1994)

In contrast to the damaged sheet pile quay wall, quay walls with compacted backfill sand survived the earthquake without damage. An example, shown in Fig. 1.5, is of a steel pipe pile wall anchored by a steel sheet pile wall with a water depth of 12 m. The ground at this wall originally consisted of fill sand with a thickness of about 10 m, underlain by the original ground of medium to dense sand.

The backfill sand was later treated by a sand compaction pile method as shown in this figure. In order to avoid the affecting the existing steel pipe pile wall during the installation of ground compaction work, gravel drains were installed near the wall as shown in Fig. 1.5.

Despite the earthquake motions with a peak acceleration of 0.3 g level, there was no damage to this quay wall. This case history demonstrated that the measures against liquefaction and design of quay walls according to the procedure based on the conventional simplified approach are sufficient to provide adequate resistance to this level of earthquake motions.

1.5 Performance at Intense Earthquake Motions

1995 Hyogoken-Nambu earthquake with magnitude 7.2 provided a good opportunity to re-evaluate the limitations of the conventional simplified approach of seismic design. Shaken with a peak ground acceleration of 0.5 g, many quay walls in Kobe Port, Japan, suffered serious damage. The damage involves large seaward displacement, settlement and tilt as shown in Fig. 1.6. The damage was cause mainly by deformation in the loosely deposited foundation soil beneath the caisson wall (Inagaki et al. 1996).

In order to evaluate the effects of liquefaction, a series of effective stress analyses were performed. A strain space multiple mechanism model for sand was used for the analysis (Iai et al. 1992). Case-1 is the analysis that simulates the conditions in-situ during the earthquake. In this case, both the foundation soils beneath the caisson wall and the backfill soils were liquefiable decomposed granite.

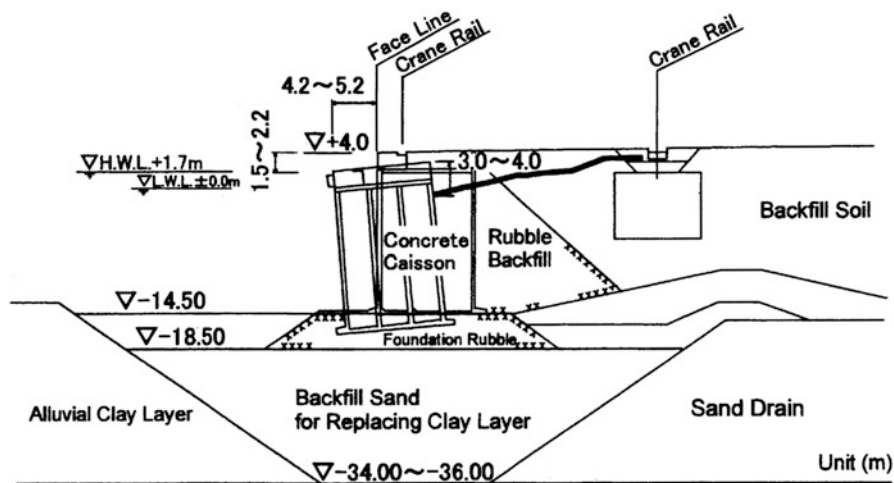


Fig. 1.6 Damage to a caisson quay wall at Kobe Port during 1995 Hyogoken-Nambu earthquake (Inagaki et al. n.d.)

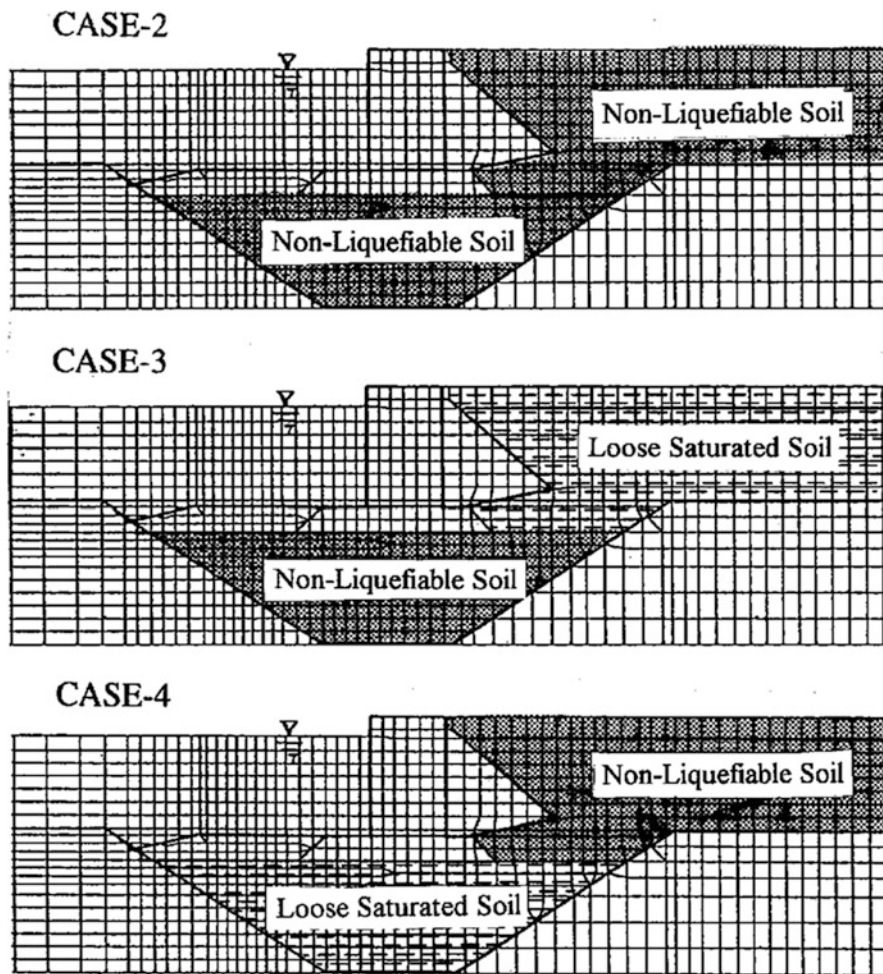


Fig. 1.7 Conditions assumed for parametric study, Cases 2 through 4 (Iai et al. 1992)

Cases-2 through 4 are defined by the extent of the non-liquefiable soil relative to the caisson wall as shown in Fig. 1.7.

Case-1 analysis was successful in simulating the damage to a caisson type quay wall as shown in Fig. 1.8. The major results of the parameter study are summarized in Table 1.1. These results indicate that the deformation of the gravity wall may be reduced up to about one half of that at the earthquake if the excess pore water pressure increase was prevented in the subsoil as in Case-2. In particular, horizontal displacements of Cases-3 and 4 are 0.5(=2.1-1.6) and 0.9(=2.5-1.6)m larger than that of Case-2, suggesting that the effect of the pore water pressure increase in the foundation soil beneath the caisson wall (Case-4) is about twice as that of the backfill (Case-3).

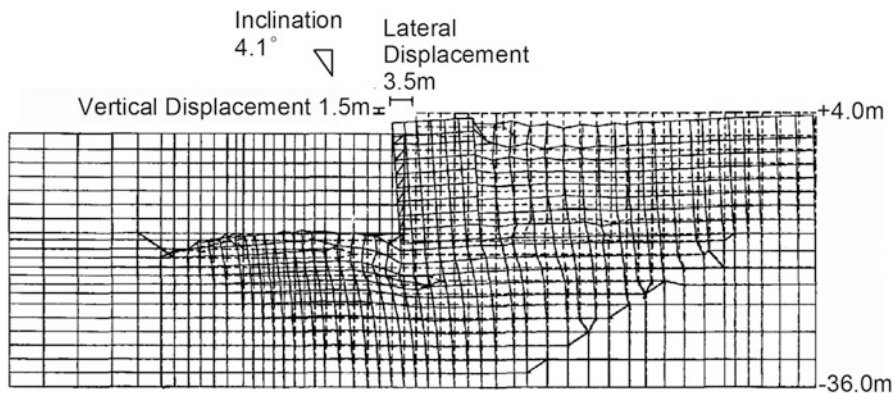


Fig. 1.8 Computed deformation of a gravity quay wall, Case-1 (Iai et al. 1998)

Table 1.1 Major results of parametric study for gravity quay wall (Iai et al. 1998)

Residual displacements of caisson			
Case	Horizontal (m)	Vertical (m)	Tilt (degrees)
Case 1	3.5	1.5	4.1
Case 2	1.6	0.6	2.4
Case 3	2.1	0.7	3.1
Case 4	2.5	1.1	2.2

These results were compared with the performance of the quay walls at Port Island (phase II). One (PC-13) was constructed on a loose deposited foundation similar to that shown in Fig. 1.6, whereas others (PC-14 and -15) were constructed on a foundation improved by the sand compaction pile (SCP) method as shown in Fig. 1.9. Although these quay walls were constructed along a straight face line, displacements of the quay walls PC-14 and -15 were about 2.5 m and 0.3 m in horizontal and vertical directions whereas those at PC-13 were about 3.5 m and 1.5 m in horizontal and vertical directions. The analysis conditions of Case-3, having non-liquefiable foundation, approximate the performance of the quay walls PC-14 and 15. The analysis condition of Case-1, having loose deposited foundation, approximates the performance of the quay wall PC-13. The results for the parameter study of Cases-1 and 3 discussed above can explain the difference between the quay walls constructed on SCP and a loose deposited foundation (Iai et al. 1998).

For variations in the peak acceleration of the earthquake motion input at the base, the horizontal residual displacement at the top of the caisson wall was computed as shown in Fig. 1.10. These response curves constitute the basis for performance-based design described in the next section.

The case history of seismic performance of quay walls during 1995 Hyogoken-Nambu earthquake provided a number of valuable lessons. In particular, the case history and analysis described above suggest that (1) liquefaction remediation is

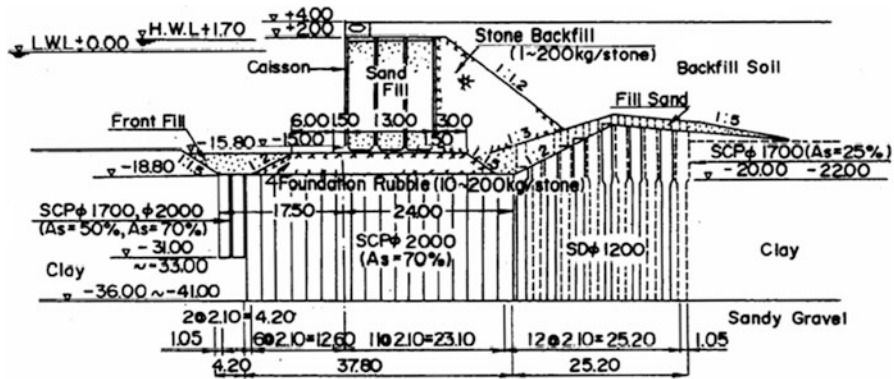


Fig. 1.9 Cross section of a quay wall in Kobe Port improved with sand compaction piles (SCP) (Inagaki et al. n.d.; Iai et al. 1998)

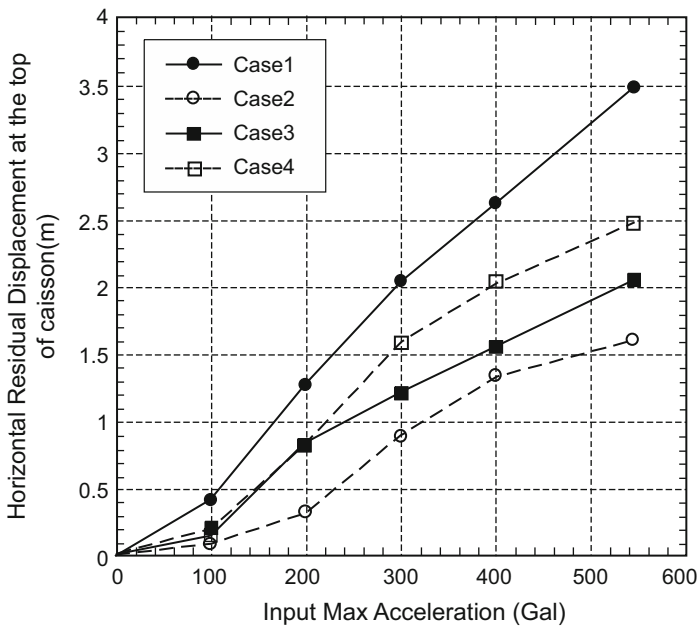


Fig. 1.10 Effects of input acceleration levels on horizontal residual displacement (PIANC 2001)

effective for improving performance of quay walls even for intense earthquake motions, (2) some residual deformation may be acceptable, (3) deformations in ground and foundation soils and the corresponding structural deformation and stress states are key design parameters, and (4) conventional simplified limit equilibrium-based methods are not well suited to evaluating these parameters. All of these issues

indicate that there is a strong need to develop performance-based approach for liquefaction remediation (PIANC 2001).

1.6 Toward Performance-Based Design

The goal of the performance-based design is to overcome the limitations present in conventional simplified seismic design. The conventional seismic design is based on providing capacity to resist a design seismic force, but it does not provide information on the performance of a structure when the limit of the force-balance is exceeded. If we demand that limit equilibrium not be exceeded in conventional design for the relatively high intensity ground motions associated with a very rare seismic event, the construction/retrofitting cost will most likely be too high. If force-balance design is based on a more frequent seismic event, then it is difficult to estimate the seismic performance of the structure when subjected to ground motions that are greater than those used in design.

In performance-based design, appropriate levels of design earthquake motions must be defined and corresponding acceptable levels of structural damage must be clearly identified. Two levels of earthquake motions are typically used as design reference motions, defined as follows:

Level 1 (L1): the level of earthquake motions that are likely to occur during the life-span of the structure;

Level 2 (L2): the level of earthquake motions associated with infrequent rare events, that typically involve very strong ground shaking.

The acceptable level of damage is specified according to the specific needs of the users/owners of the facilities and may be defined on the basis of the acceptable level of structural and operational damage given in Table 1.2. The structural damage category in this table is directly related to the amount of work needed to restore the full functional capacity of the structure and is often referred to as direct loss due to earthquakes. The operational damage category is related to the amount of work needed to restore full or partial serviceability. Economic losses associated with the loss of serviceability are often referred to as indirect losses. In addition to the fundamental functions of servicing sea transport, the functions of port structures may include protection of human life and property, functioning as an emergency base for transportation, and as protection from environmental threats from spilling hazardous materials such as oils. If applicable, the effects on these issues should be considered in defining the acceptable level of damage in addition to those shown in Table 1.2.

Once the design earthquake levels and acceptable damage levels have been properly defined, the required performance of a structure may be specified by the appropriate performance grade S, A, B, or C defined in Table 1.3. In performance-based design, a structure is designed to meet these performance grades.

Table 1.2 Acceptable level of damage in performance-based design^a

Acceptable level of damage	Structural	Operational
Degree I: Serviceable	Minor or no damage	Little or no loss of serviceability
Degree II: Repairable	Controlled damage ^b	Short-term loss of serviceability ^c
Degree III: Near collapse	Extensive damage in near collapse	Long-term or complete loss of serviceability
Degree IV: Collapse ^d	Complete loss of structure	Complete loss of serviceability

^aConsiderations: protection of human life and property, functions as an emergency base for transportation, and protection from environmental threats from spilling hazardous materials such as oils, if applicable, should be considered in defining the damage criteria in addition to those shown in this table

^bWith limited inelastic response and/or residual time for repairs

^cStructure out of service for short to moderate duration

^dWithout significant effects on surroundings

Table 1.3 Performance grades S, A, B, and C

Performance grade	Design earthquake	
	Level 1(L1)	Level 2(L2)
Grade S	Degree I: Serviceable	Degree I: Serviceable
Grade A	Degree I: Serviceable	Degree II: Repairable
Grade B	Degree I: Serviceable	Degree III: Near collapse
Grade C	Degree II: Repairable	Degree IV: Collapse

The principal steps taken in performance-based design are shown in the flow-chart in Fig. 1.11:

1. Choose a performance grade from S, A, B, or C: This step is typically done by referring to Tables 1.2 and 1.3 and selecting the damage level consistent with the needs of the users/owners. Another procedure for choosing a performance grade is to base the grade on the importance of the structure. Degrees of importance are defined in most seismic codes and standards. If applicable, a performance grade other than those of S, A, B, or C may be introduced to meet specific needs of the users/owners.
2. Define damage criteria: Specify the level of acceptable damage in engineering parameters such as displacements, limit stress states, or ductility factors.
3. Evaluate seismic performance of a structure: Evaluation is typically done by comparing the response parameters from a seismic analysis of the structure with the damage criteria. If the results of the analysis do not meet the damage criteria, the proposed design or existing structure should be modified. Soil improvement including remediation measures against liquefaction may be necessary at this stage. Details of liquefaction remediation can be found in the publication of the Port and Harbour Research Institute (Port and Harbour Research Institute 1997).

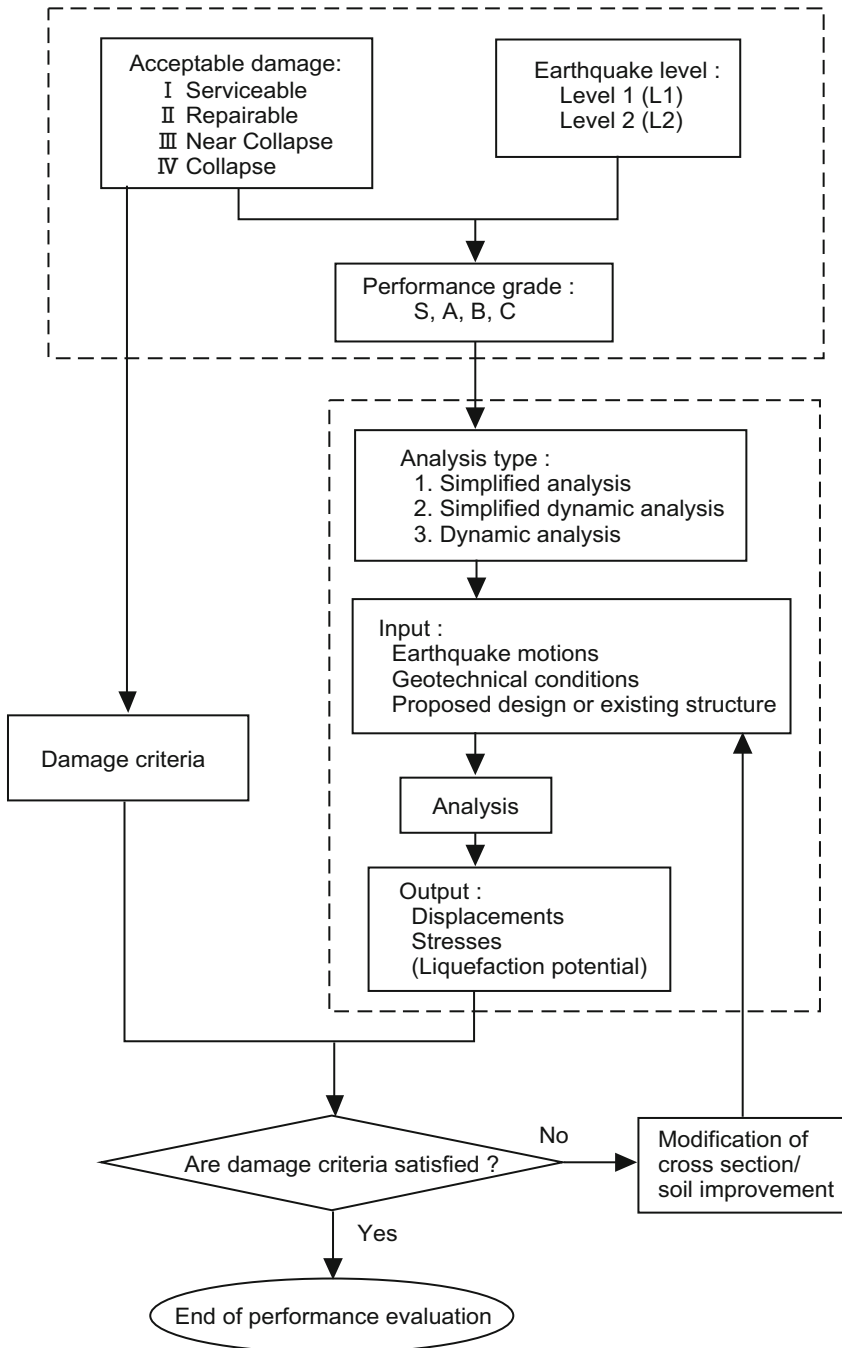


Fig. 1.11 Flow chart for performance evaluation (PIANC 2001)

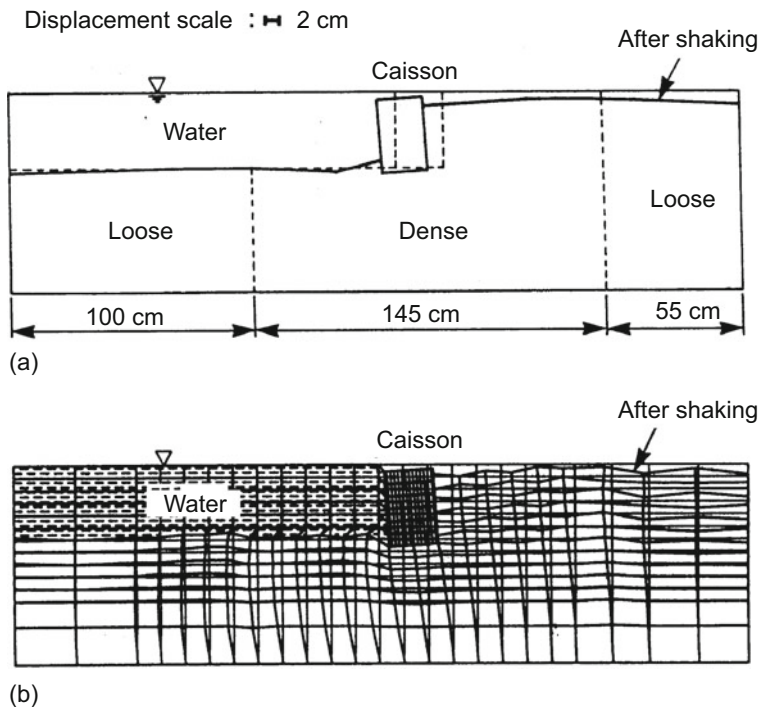


Fig. 1.12 Model test and computed results of residual deformation of a caisson quay wall (Iai 1994). (a). Shaking table tests. (b) Effective stress analysis

More comprehensive discussions on the performance-based design of port structures can be found in a publication by a working group on seismic effects for port structures, International Navigation Association (PIANC 2001).

Design charts useful for determining the area of ground improvement within the context of performance-based design were obtained in early 1990s. An example for a caisson quay wall is shown in Figs. 1.12 and 1.13, where based on a series of shaking table tests and effective stress analysis (Fig. 1.12) by varying the extent of ground improvement area (Fig. 1.13a), the area of ground improvement is shown in normalized design chart (Fig. 1.13b) (Iai et al. 1998).

Another example is the degree of uplift of a buried structure due to liquefaction as shown in Figs. 1.14 and 1.15 (Iai and Matsunaga 1991). Similar work was also performed on a sheet pile quay wall (Port and Harbour Research Institute 1997; Iai and Matsunaga 1990; McCullough and Dickenson 1998).

In the framework of performance-based design, an example of the liquefaction remediation for a gravity quay wall may be given using the design chart shown in Fig. 1.13 as follows. In this example, let us assume that the quay wall was given the performance grade A with seismic criteria over L1 ($PGA = 250 \text{ cm/s}^2$) and L2 ($PGA = 400 \text{ cm/s}^2$) earthquake motions specified by the residual displacements (d/H) of less than 1 % for maintaining full operation and 4 % for allowing quick

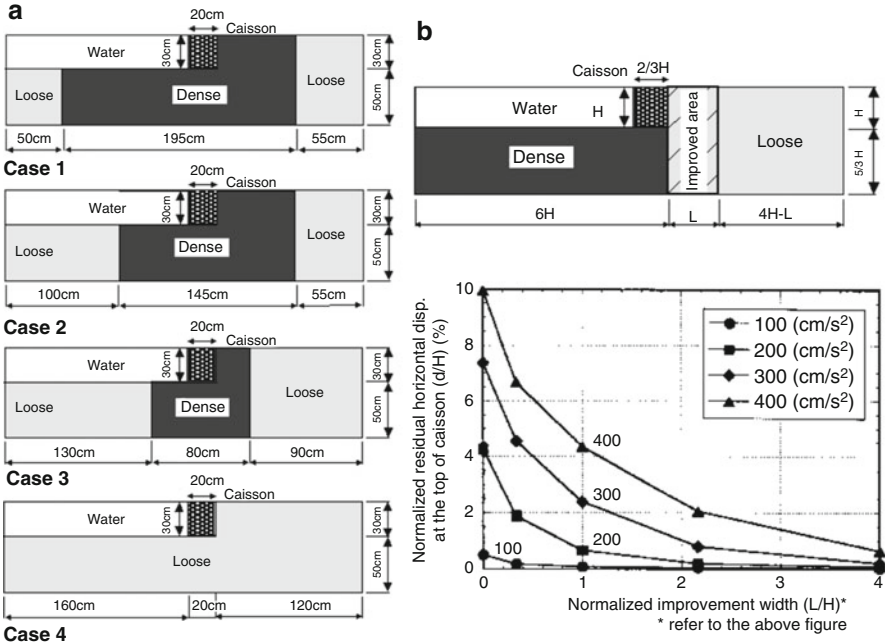


Fig. 1.13 Design chart for ground improvement area (Iai 1994). (a) Variation of cross section. (b) Width of compaction area vs. residual horizontal displacement

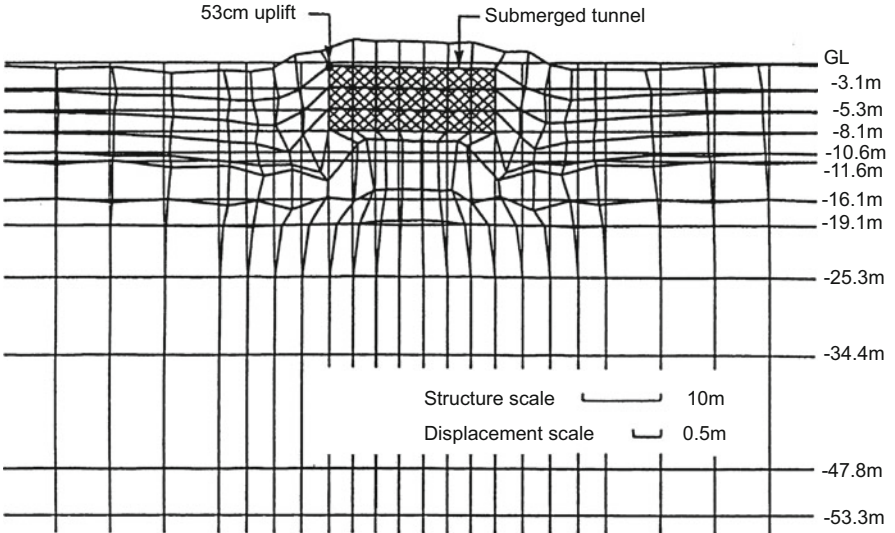


Fig. 1.14 Example of computed uplift of a buried structure in liquefied soil (Iai and Matsunaga 1990)

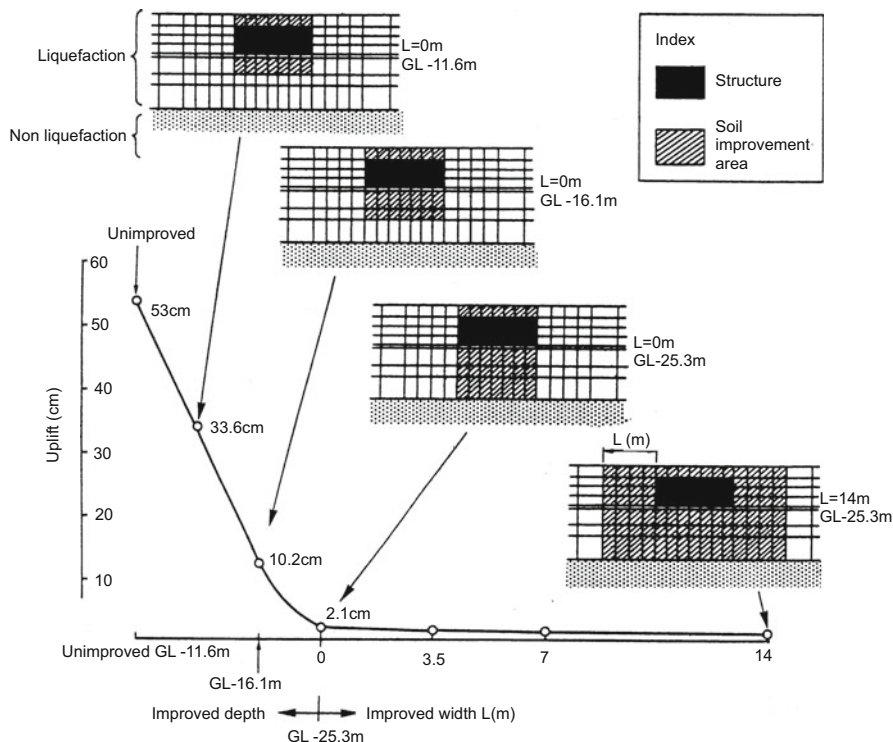


Fig. 1.15 Uplift and improved depth of buried structures (Port and Harbour Research Institute 1997)

recovery of operation. By referring to Fig. 1.13, the L1 criterion gives the area $L/H = 1.7$ whereas the L2 criterion gives the area of $L/H = 1.2$. Thus, the improvement area required to meet these performance requirements is $L/H = 1.7$. In this particular example, stronger L2 excitation does not govern the final design that was highly influenced by a high performance requirement for L1.

Performance-based approach can readily be applicable to other types of structures, including pile-supported wharves (shown in Figs. 1.16 and 1.17), that may require the structural damage criteria with respect to piles and a deck as well as displacements of dykes or retaining structures. Details can be found in the guidelines by PIANC (2001).

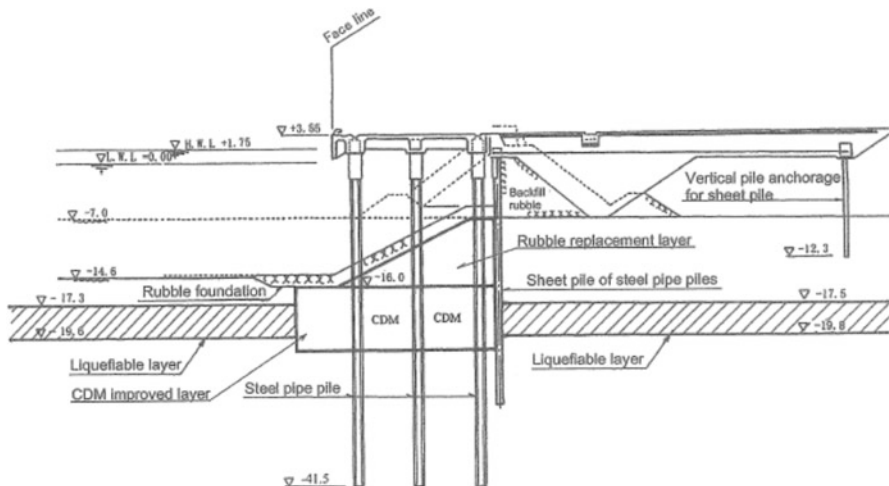


Fig. 1.16 Example of ground improvement for a pile-supported wharf (PIANC 2001)

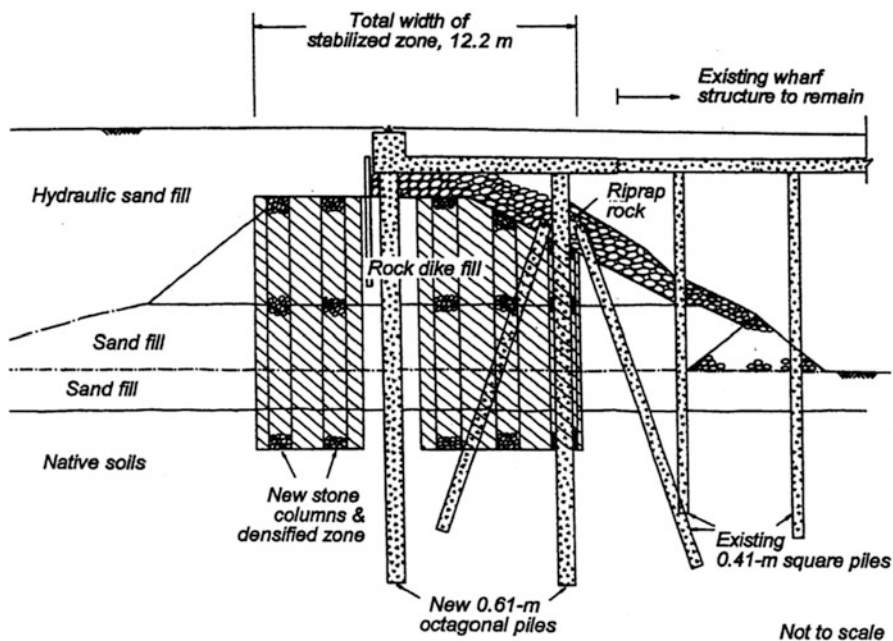


Fig. 1.17 Example of ground improvement for a pile-supported wharf (Egan et al. 1992)

1.7 Seismic Performance with Effects of Partial Drainage During Earthquake Motions of Extremely Long Duration

Duration of the earthquake motions that have been discussed in the preceding sections is about 30 s or less so that undrained behavior of sand provides reasonable representation of the seismic performance of geotechnical structures. For a longer duration of earthquake motions such as those recorded during 2011 East Japan earthquake of magnitude 9.0, the duration of the earthquake motion is in the order of 200 s and the effect of partial drainage during this duration will likely to be significant. Thus, the effective stress analysis allowing dissipation and redistribution of excess pore water pressures during and after earthquake should become the primary analysis procedure to appropriately evaluate the seismic performance of geotechnical structures.

A typical example of the damage to a sheet pile quay wall during 2011 East Japan earthquake is shown in Fig. 1.18. In this example, the sheet pile wall was subject to the earthquake motion with $PGA = 0.28\text{ g}$ shown in Fig. 1.19, and displaced 200 mm toward the sea with back fill settlements in 100 mm immediately behind the sheet pile wall. The settlement at back fill extends further inland and resulted in 300 mm differential settlement behind the anchor with coupled piles.

The effective stress analysis of this sheet pile was performed by multiple step analysis of initial gravity analysis by closely following the sequence of construction of the quay wall (Tashiro et al. 2017). After the multiple step analysis of initial

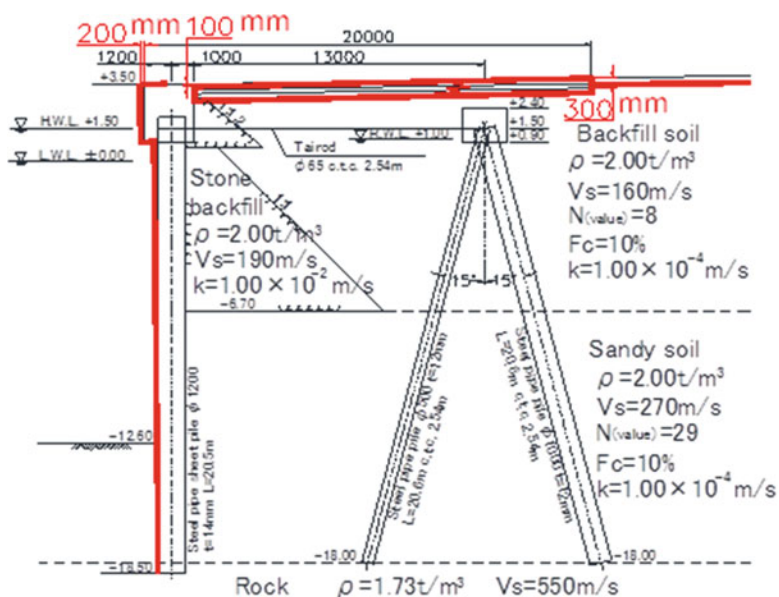


Fig. 1.18 Damage to a sheet pile quay wall subject to earthquake motion of extremely long duration (2011 East Japan Earthquake (M = 9.0), Soma Port, No.2 Wharf (-12 m))

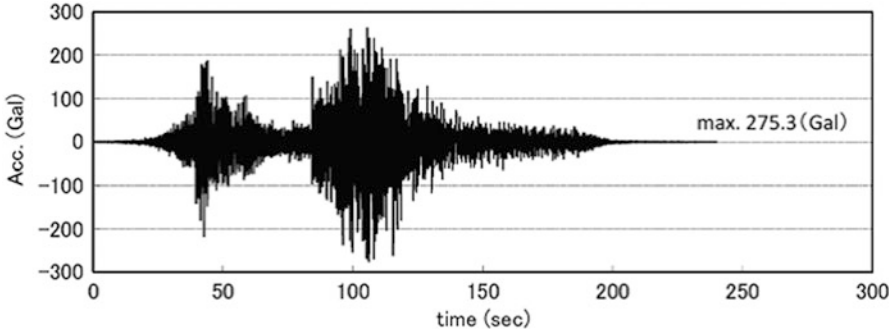


Fig. 1.19 Input earthquake motion at Soma Port during 2011 East Japan Earthquake

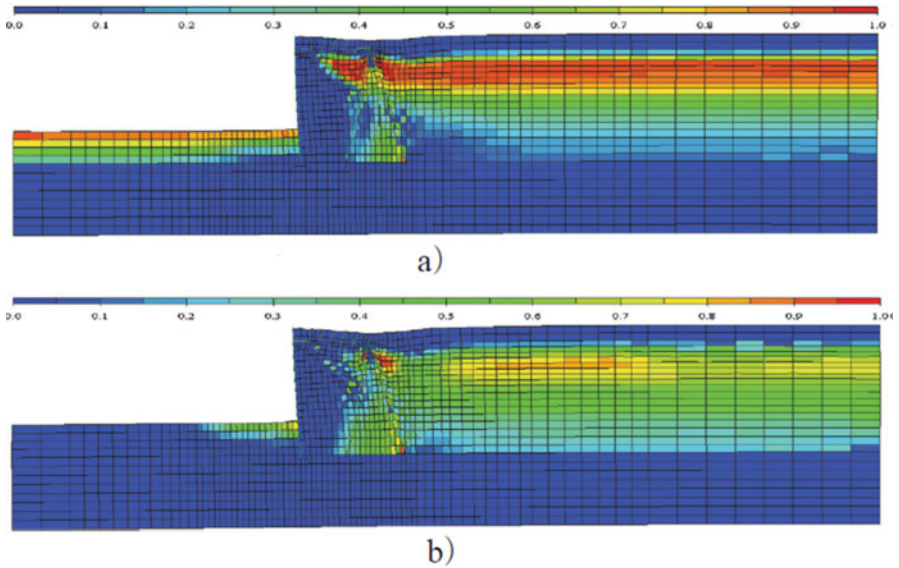


Fig. 1.20 Results of the effective stress analysis of a sheet pile quay wall (Displacement scale: enlarged by five times, Color contour: excess pore water pressure ratio). (a) $t = 126$ s (immediately after primary portion of shaking). (b) $t = 240$ s (partially dissipated state of excess pore water pressure)

gravity analysis, the dynamic effective stress analyses were performed in partially drained and undrained conditions using Cocktail glass model (Iai et al. 2011) for partially drained (Case 1) and undrained analyses (Case 6) and Multiple spring model (Iai et al. 1992) for undrained analysis (Case 5). For further details of the analyses, refer to (Tashiro et al. 2017). The computed results of the sheet pile quay wall in terms of global deformation and excess pore water pressure ratios are shown in Fig. 1.20. The upper half of backfill soil liquefied and the sheet pile wall displaced toward the sea, resulted in settlements at the ground surface inclining toward inland.

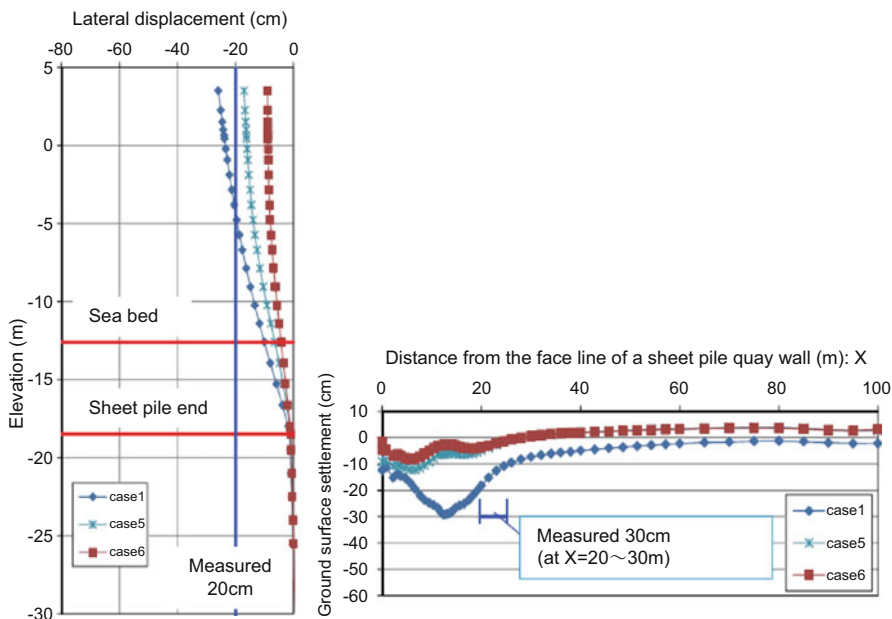


Fig. 1.21 Computed and measured lateral and vertical displacements of a sheet pile quay wall (*Right*: Lateral displacement of a sheet pile wall, *Left*: vertical displacement at the ground surface behind the face line of a sheet pile quay wall) (Case 1: Cocktail glass model (Iai et al. 2011), partial drainage, coefficient of permeability $k = 1 \times 10^{-4}$ m/s; Case 5: Multi spring model (Iai et al. 1992), undrained condition; Case 6: Cocktail glass model (Iai et al. 2011), undrained condition)

The computed and measured displacements at the sheet pile and the ground surface are shown in Fig.1.21. These results indicate that the analyses with partially drained and undrained conditions all capture the order of magnitude lateral displacements of the sheet pile quay wall. However, the partially drained analysis (Case-1) captures the order of magnitude vertical displacements at the ground surface much better than the undrained analyses (Cases-5 and 6).

Consequently, seismic performance of geotechnical structures subject to earthquake motions of long duration can be more precisely evaluated by partially drained analysis, allowing the dissipation and redistribution of excess pore water pressures during and after earthquakes.

1.8 Conclusions

Seismic damage to geotechnical structures is often associated with significant deformation of a soft or liquefiable soil deposit; hence, if the potential for liquefaction exists, implementing appropriate remediation measures against liquefaction is an effective approach to significantly improve seismic performance. The

conventional simplified approach based on pseudo-static force equilibrium provides reasonable guidelines for designing remediation of liquefiable soils against moderate earthquake motions. However, there are cases where the conventional simplified approach is not readily applicable. For example, for intense earthquake motions, acceptable displacements and structural stresses become the primary design criteria, which are not evaluated by the conventional simplified approach. There are a number of cases where site specific constraints pose difficulties in implementing the liquefaction remediation measures as suggested by the conventional simplified approach. In order to meet these situations, performance-based approach provides a reasonable framework to work on for adequate liquefaction remediation. Case histories of performance and examples of design described in the paper may be useful to apply these conclusions in practice.

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