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Feature Article

Initiatives for Building Resilient Foundation Structures

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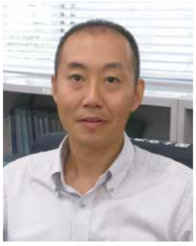
Application of the Partial Factor Design Method for Foundations in the Japanese Specifications for Highway Bridges

by Toshiaki Nanazawa, Ministry of Land, Infrastructure, Transport and Tourism



Toshiaki Nanazawa: After finishing the master's course at the Graduate School of Engineering, Tohoku University, he entered the Ministry of Construction in 1994. He worked at the Center for Advanced Engineering Structural Assessment and Research, PWRI in 2010 and he assumed his current position as Head, Foundation, Tunnel and Substructures Div. of National Institute for Land and Infrastructure Management, MLIT in 2018. His specialized field is structural engineering and foundation engineering. Dr. Eng.

The *Japanese Specifications for Highway Bridges* (JSHB) was revised in 2017. The major revision in this latest version was the introduction of the partial factor design method in the JSHB[1].

Preceding the revision made in 2017, the application of the partial factor design method in the foundation design provided in the JSHB was examined, the content of which is introduced in this article.

Application of the Reliability Design Method for Foundations in the JSHB

In the literature[2], the damage level was judged of 401 bridges located in Sendai, where strong seismic motions were observed from the 2011 Great East Japan Earthquake. As a result of this judgement, it was confirmed that, although the bridges in which damage occurred in the foundation numbered only three of the 401 bridges, large-scale damage occurred in the foundations of all of these three bridges (Fig. 1).

Even when the level of bridge foundation damage from the above great earthquake is compared to damage caused by other earthquakes and scouring damage caused by floods, the same trend can be observed. Why is the number of damaged existing bridge foundations constructed in nonconformity with the latest JSHB few? A main attributable reason for this is that these foundations maintained large safety margin secured by the safety factors and other safety margins at the design stage. Meanwhile, in the existing bridge foundations under certain conditions, enough safety margin provided in the design stage could not be secured, which thus led to the occurrence of bridge damage that exerted an adverse social impact.

Photos 1 and 2 show examples of damage that occurred in the existing bridge substructures. It was confirmed from the results of analysis that the damage and deformation of the bridge substructures could not be prevented only by means of reinforcement and improvement of prescriptions pertaining to design methods, such as increase of design seismic mo-

Fig. 1 Damage Sections and Damage Levels of the Bridges Damaged by the 2011 Great East Japan Earthquake[2]

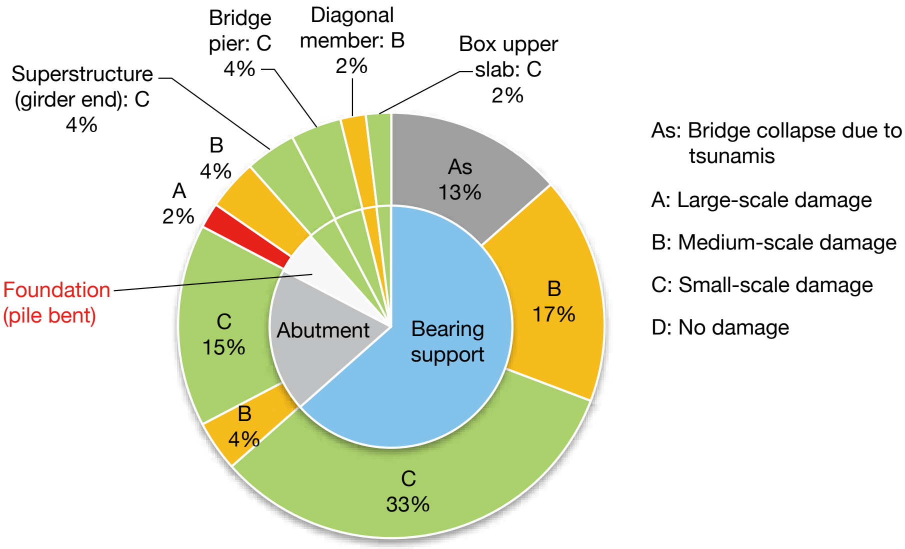


Photo 1 Bridge abutment that collapsed due to the erosion of the slope

Photo 2 Bridge abutment that caused subsidence due to the collapse of the slope during an earthquake

tion, but that further reinforcement and improvement of the prescriptions pertaining to the ground surveys and foundation construction methods were required to prevent the damage and deformation. However, because it is difficult to uniformly reinforce the currently-prevailing prescriptions pertaining to ground surveys and foundation construction methods, it is desirable to change to the design methods that can yield a certain advantage by the use of quantitatively and qualitatively appropriate ground surveys and highly-precise construction control technologies.

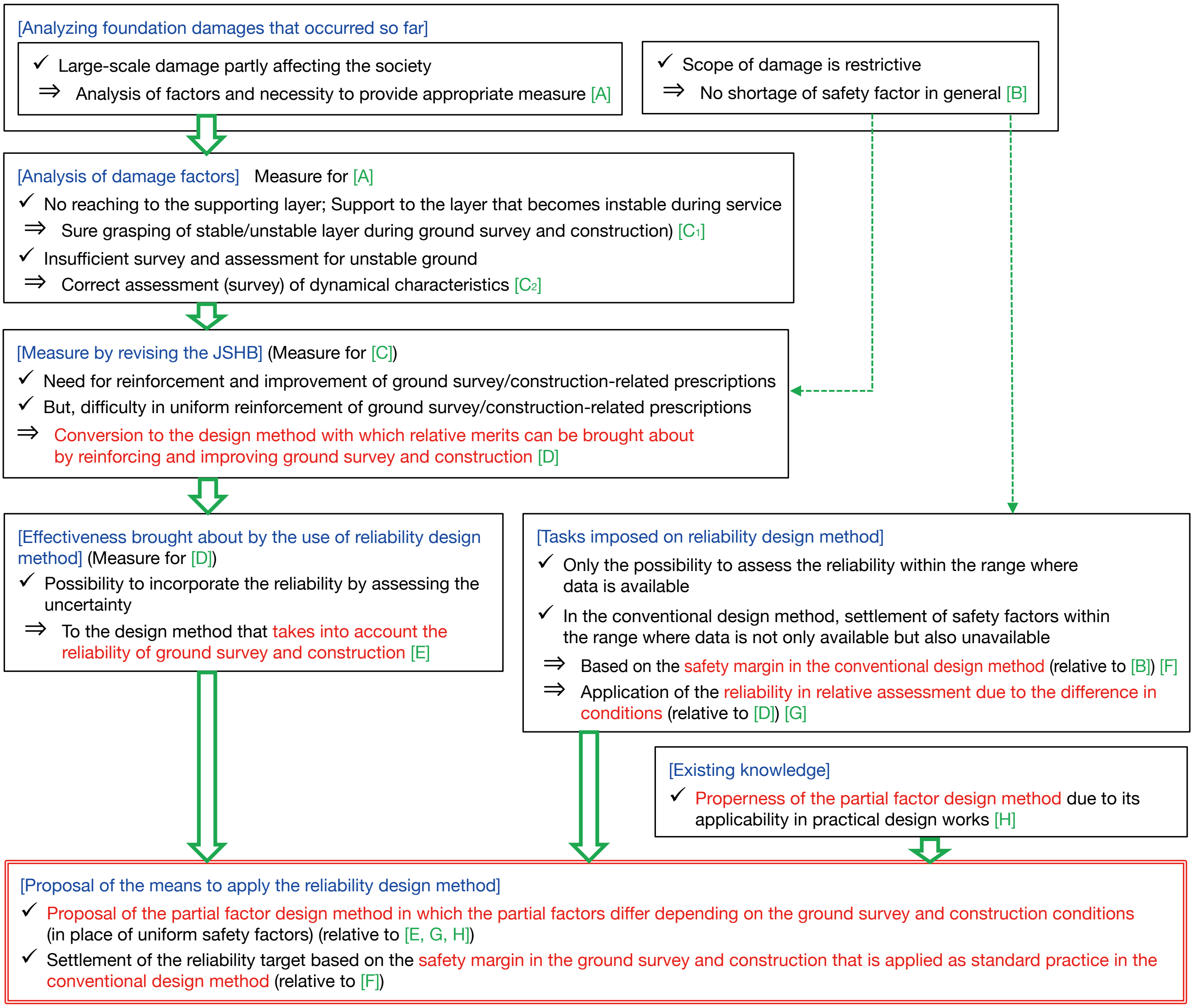
The application of the reliability design method, particularly the partial factor design method, in design specifications brings about high effectiveness, which is summarized in Table 1. As shown in (3) in Table 1, because partial factors conforming to the deviation of each element can be settled, a partial factor design method is likely to be established that can demonstrate advantages brought about by the use of highly precise ground surveys and construction methods. However, it is necessary to pay due attention to the fact that the reliability theory can only be applied in the range where significant statistics are available and where the ease-of-understanding of the design method is properly considered for those engaged in practical design and construction.

Based on the above, the reliability is assessed using statistics relating to the differences in ground conditions and survey methods and the differences in foundation construction methods, and then the partial factors that differ depending on the conditions are settled (refer to Fig. 2). Consequently, it has become possible to make

Table 1 Effectiveness Brought about by the Use of the Partial Factor Design Method

No.	Effectiveness
(1)	Keeping up with international trend
(2)	Appropriate resources allocation to various bridge structures due to the arrangement of reliability
(3)	Possible to settle the rational partial factor that conforms to the deviation of each element
(4)	Promotion of the introduction of new structural materials and members
(5)	Possible to reexamine the partial factors based on data

Fig. 2 Proposal of Partial Factor Design Method for Bridge Foundations with Consideration for the Uncertainty Involved in Ground Surveys and Piling Work



more rational design by the use of higher factor values when highly reliable ground survey and construction method are applied. Meanwhile, in Fig. 2, the symbol shown in green is inserted so that the relation between the task and the corresponding measure can be understood.

Calculation of Partial Factors for the Load-bearing Capacity Design of Pile Foundations

As an example of settling a partial factor, the method to calculate the partial factors applied in verifying the yield bearing capacity versus the axial-direction thrusting force of piles is shown[3]. In order to take into account the difference in uncertainties in the conditions of the resistance side such as structural and ground conditions, the partial factor ϕ relating to the resistance is examined. In this examination, the uncertainty of the load is not taken into account, and the load is treated as a deterministic value. In the assessment of the uncertainty of the ground resistance, the log-normal distribution is applied.

Equation (1) below shows the calculation of the limit value to be applied for retaining the axial-direction thrusting force of piles within the range of the yield bearing capacity of the pile. The pile bearing capacity is verified by means of the comparison between the limit value and the response value to be calculated from the equation. While Equation (1) conforms to the bearing capacity calculation equation in the conventional design method[1], the equation applies the characteristic value of the yield bearing capacity assumed to have a definite relation with the ultimate bearing capacity of pile R_u.

$$R_d = \phi(R_y - W_s) + W_s \cdot W \quad (1)$$

where,

R_d: Limit value of axial-direction thrusting force of pile (kN)
ϕ : Partial factor
R_y: Characteristic value of yield bearing capacity of pile (kN)
W_s: Effective weight of earth substituted by pile (kN)
W: Effective weight of pile and earth inside pile (kN)

Because the response of the pile foundation changes depending on the structural and ground conditions, the uncertain statistic of the response value is calculated by means of the Monte Carlo simulation employing 114 cases of pile foundation models obtained by trial design under diverse structural and ground conditions and based on the conventional design method. On the occasion of the simulation, the uncertainty of both the axial spring constant of a pile K_v and the coefficient of horizontal subgrade reaction k_H are taken into account (Tables 2 and 3). In addition, the uncertainty of the resistance value is as shown in Table 4.

Of the calculation results for the reliability index β for the pile foundation, the calculation result in the event of an earthquake that is predominant in pile foundation design is shown in Fig. 3. Based on the reliability index β thus calculated, the reliability index $\beta^R{}_T$ relating to the target resistance of the pile foundation is settled.

In the settlement of the reliability index in the case of the difference in ground survey methods, the index β is basically settled that conforms to Case 2 that is most commonly applied in the estimation of the ground deformation modulus (refer to Table 3). In the settlement of the re-

Table 2 Statistics of Uncertainty of the Axial Spring Constant of Pile K_V

Pile installation method	Average	Coefficient of variation
Pile driving method	1.00	0.40
Cast-in-place pile method	1.00	0.50
Bored pile method	1.00	0.45
Pre-boring pile method	1.00	0.35
Steel pipe soil cement pile method	1.00	0.30
Screw pile method	1.00	0.40

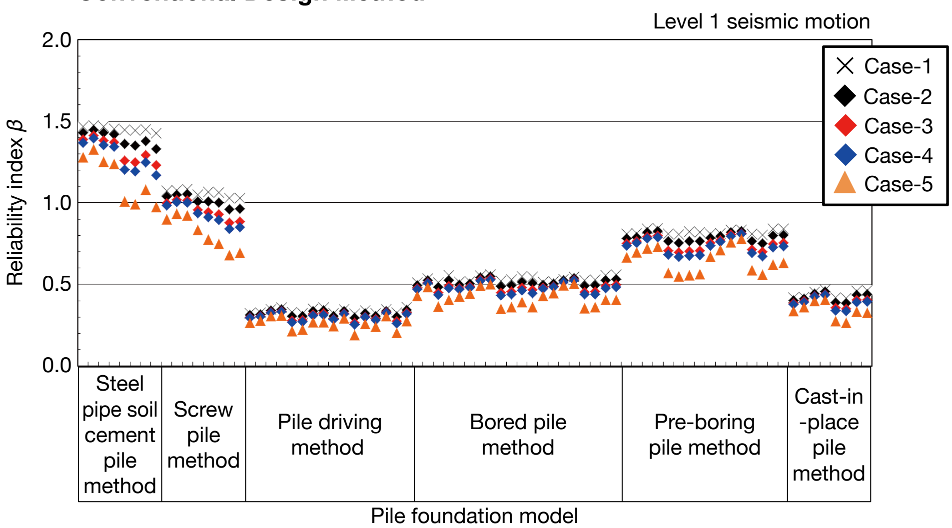
Table 3 Statistics of Uncertainty of the Coefficient of Horizontal Subgrade Reaction K_H

Case	Estimation method for k_H or deformation modulus E_0 used in calculating k_H		Average	Coefficient of variation
Case-1	When obtained from the horizontal loading test for pile		1.0	0.25
Case-2	When obtained by the standard penetration test as well as the laboratory test or borehole horizontal loading test			0.45
Case-3	When obtained only from the standard penetration test	Sandy soil with N value of 5 or more		0.60
Case-4		Cohesive soil with N value of 5 or more		0.70
Case-5		N value of less than 5		1.00

Table 4 Statistics of Uncertainty of the Vertical Bearing Capacity of Piles

Pile installation method	Average	Coefficient of variation
Pile driving method	1.0	0.45
Cast-in-place pile method	1.0	0.40
Bored pile method	1.0	0.35
Pre-boring pile method	1.0	0.25
Steel pipe soil cement pile method	1.0	0.15
Screw pile method	1.0	0.20

Fig. 3 Reliability Index β for Pile Foundations Designed Employing the Conventional Design Method



liability index in the case of the difference in pile installation methods, when confirming the reliability index β in Case 2, the index is divided into the following two groups: a group of relatively small index (β=0.3~0.5) for the pile driving method, cast-in-place pile method and bored pile method (referred to as conventional installation methods) and another group of relatively large index (β=0.7~0.14) for the pre-boring pile method, steel pipe soil cement pile method and screw pile method (referred to as the new installation method). The reason why the index β of the new installation method becomes larger than that of the conventional method is considered to be attributable to upgrading of the installation control method in recently-developed new installation methods, and as a result it has become possible to implement highly-precise installation work.

Settling β of the conventional installation method in Case 2 as the basic value and taking into account the improvement in estimation accuracy brought about by the reexamination of the bearing capacity estimation equation and the deviation due to the difference in ground and pile installation conditions, the reliability index β^R_T relating to the target pile foundation resistance is settled at 0.50.

Then, the partial factor ϕ at the resistance side, which conforms to β^R_T, is calculated. Regarding the yield bearing capacity of the pile subjected to the axial-direction thrusting force, the effect of the difference in the installation method on the yield bearing capacity is more predominant than that of the estimation accuracy for the ground reaction force coefficient on the yield bearing capacity. Table 5 shows the partial factor ϕ that conforms to the difference in the installation method. Table 5 also shows the average ϕ value of the conventional installation method, used as the standard value in settling the reliability index β^R_T.

Meanwhile, due attention should be paid to ensuring that the calculation results shown in the above are not identical with those in the final content of the revised JSHB.

Table 5 Partial Factor Φ by Pile Installation Method

Pile installation method	Φ	
Pile driving method	0.71	0.74 (average value)
Cast-in-place pile method	0.74	
Bored pile method	0.77	
Pre-boring pile method	0.82	—
Steel pipe soil cement pile method	0.88	
Screw pile method	0.85	

Partial Factor Design Method for the Rationalized Design of Foundations

In the conventional design method for bridge foundations, the uniform safety factors have basically been applied. In contrast, the partial factors that differ depending on the pile installation and ground survey methods can be settled by employing the partial factor design method introduced above, which thus allows for securing identical reliability in foundation design regardless of the difference in estimation accuracy. In addition, when highly precise installation and ground survey methods are applied, it will become possible to perform rationalized design, and therefore the application of highly precise installation and ground survey methods are expected to be further promoted.

On top of this, the dissemination of the content shown in this article will bring about such merits—in cases when more highly-precise ground survey and pile installation technologies are developed, it will be possible to settle partial factors that are more advantageous in terms of design compared to current levels, and as a result new possibilities for making more rationalized design will be recognized by those engaged in the development of new technologies that target further improvement of technologies. As an effect induced by these attempts, it is expected for the reliability improvement cycle to attain steady development (refer to Fig. 4).

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1) Japan Road Association: Specifications for Highway Bridges, Part 4 Substructures, JRA (2012), 2012
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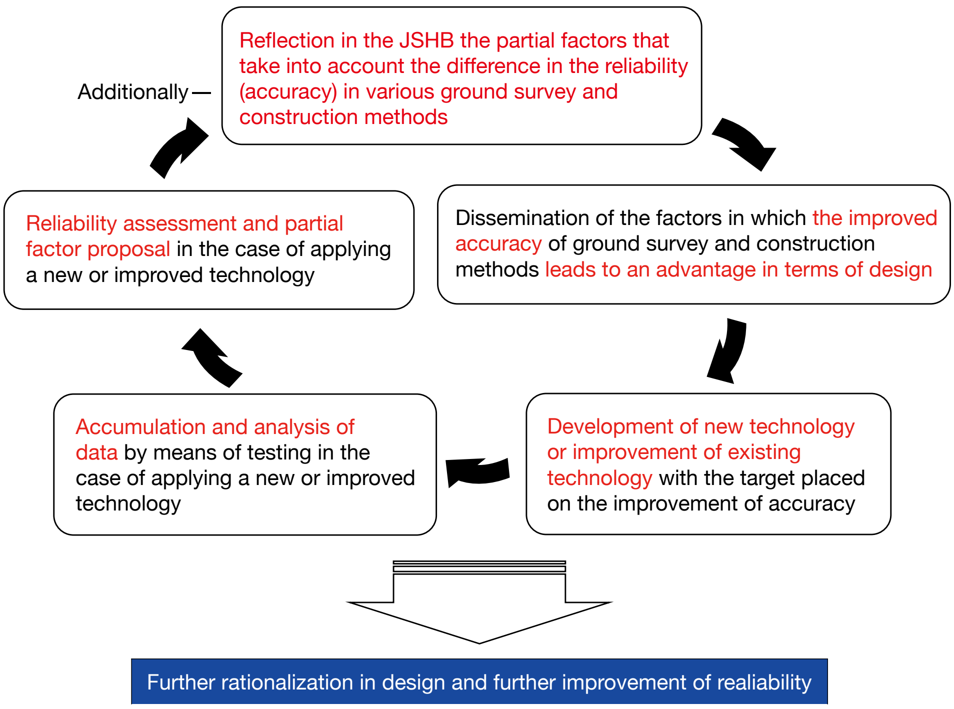
Fig. 4 Cycle for the Technical Development and Estimation Accuracy Improvement Mainly Targeting the Settlement of Reliability-based Partial Factors



Feature Article: Initiatives for Building Resilient Foundation Structures (2)

Revision of Recommendations for Design of Building Foundations and Establishment of Secondary Design Method for Steel Pipe Piles

by Katsuichiro Hijikata, Former Professor at Shibaura Institute of Technology

Katsuichiro Hijikata: After graduating from the School of Engineering, The University of Tokyo, he entered Tokyo Electric Power Company in 1981. Then he was appointed to Professor, School of Architecture, Shibaura Institute of Technology in 2013. He resigned his post as professor in 2021.

The revised version of the *Recommendations for Design of Building Foundations* was published by the Architectural Institute of Japan in November 2019 (hereinafter referred to as the "new *Recommendations*"). This article explains the new *Recommendations* focusing mainly on seismic resistance-related prescriptions for the pile foundation structures provided in the new *Recommendations*.

Further, because the secondary design method for building foundations was newly prescribed in the new *Recommendations*, this article explains the current state of the secondary design method for steel pipe piles being established.

Outline of New Recommendations

• Major Revisions in New Recommendations

The major revisions provided in the new *Recommendations* are shown below:

—It was shown as basic policy that the performance-based design (secondary design) for level 2 seismic motion be made for the foundation structure.

—The design limit value and the required performance, which are applied to verify safety, were made clear as thoroughly as possible. In addition, the performance grade (safety level) of foundation structures that takes into account the importance of respective buildings was proposed.

—In order to promote the practical use of the secondary design, the seismic load and the ground deformation occurring in the event of earthquakes were prescribed. In addition, the "group pile frame model" was proposed as the calculation method for foundation structures.

• Necessity of Secondary Design for Foundations Structures

In the Building Standard Law of Japan, while the prescription of the secondary design for superstructures is provided, that for foundation structures is not provided. Only the prescription—"the foundation structure can be designed only by the use of allowable stress calculation (primary design)"—is provided in Notification No. 1347-2 of the then Ministry of Construction.

The reason why secondary design is not required in designing foundation structures is considered to be attributable to the fact that there have been no examples in Japan in which the collapse or inclination of foundation structures have directly affected human lives. However, the necessity of secondary design of foundation structures has been strongly recognized in recent years due to the following:

—In past earthquake damages, there were many cases in which difficult tasks and huge expenses were required for the recovery and repair of damaged foundation structures. As a result, it has been recognized to be necessary from the viewpoint of asset protection to perform design that can avoid great foundation damage from occurring.

—There exist both private and public important buildings that will require continued usage after earthquakes, and thus it has been desired from the viewpoint of business continuity to take proper countermeasures against level 2 seismic motion that occurs extremely rarely.

Based on the above trends, the Architectural Institute of Japan decided to clearly show the secondary design for foundation structures in the new *Recommendations*, preceding the Building Standard Law of Japan.

• Specific Revision Contents

The content of specific revisions pertaining to the secondary design of foundation structures shown in the new *Recommendations* are introduced below:

—Marginal state value and required performance for design

In the new *Recommendations*, three marginal states were defined, and the required performance that corresponds to these three marginal states were prescribed. Tables 1 and 2 show their specific contents. The figure on the right in Fig. 1 shows the examination items used to confirm the required performance pertaining to pile foundations.

—Seismic load for design

In the new *Recommendations*, the seismic load due to level 2 seismic motion, which works on the pile foundation, was prescribed as shown in the figure on the left in Fig. 1. Of these seismic loads, the inertia force and overturning moment of superstructure were prescribed using the exist-

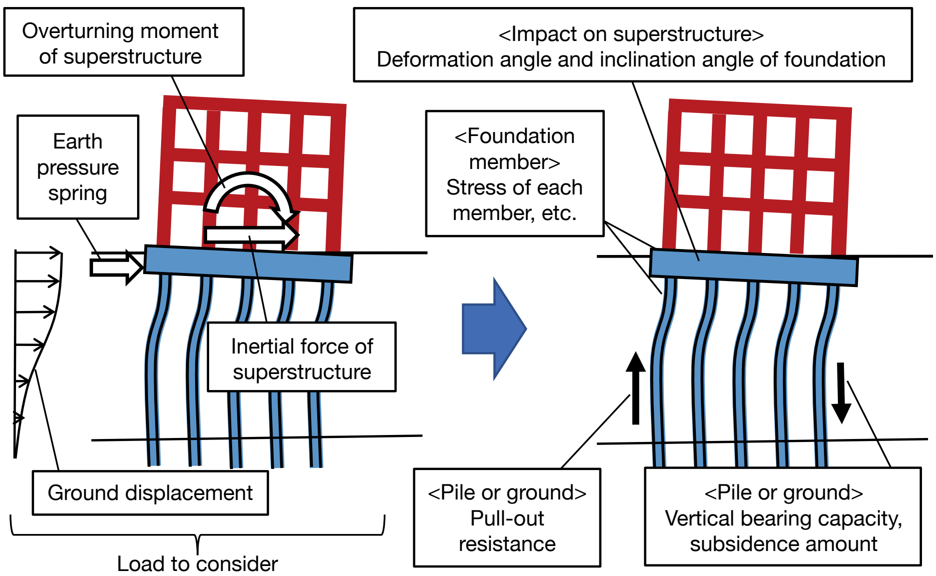
Table 1 Definition of Marginal States of Foundations

Marginal states	Definition
Usage limit state	A state in which the superstructure begins to have inadequate usability due to subsidence and displacement of the ground and foundation members.
Damage limit state	A state in which repair/reinforcement of the superstructure or foundation member begins to be required due to subsidence/deformation of the ground/foundation member.
Ultimate limit state	A state in which the superstructure cannot be supported due to destruction or deformation of the ground/foundation member, or a state in which repair/reinforcement of the foundation member begins to become extremely difficult.

Table 2 Required Performances for Marginal States

Marginal states	Impact on superstructure	Foundation	Ground
Usage limit state	There is no problem with usability and durability.	There is no problem with durability. No harmful cracks occur.	No harmful subsidence or deformation occurs during use.
Damage limit state	Does not cause excessive inclination or damage that requires structural repair/reinforcement.	No damage that requires structural repair or reinforcement.	Excessive subsidence and residual deformation do not occur.
Ultimate limit state	Does not fall or collapse	Does not cause brittle fracture. In addition, the limit of deformation performance is reached and the yield strength is not reduced.	The ground (improved ground) does not lose its normal force

Fig. 1 Examination Items and Seismic Loads for Confirming Required Performances



ing evaluation equations in the Building Standard Law of Japan and other guidelines. As regards the ground displacement and the earth pressure spring, a simple evaluation equation having a form that does not need detailed analysis was newly prescribed.

—Calculation method for stress working on piles

In the new *Recommendations*, in order to calculate the stress working on piles, the "group pile frame model" was proposed (refer to Fig. 2). This analysis method is a static analysis model in which the seismic motion is substituted for the equivalent seismic load, and incremental analysis is carried out by taking into account the nonlinearity of the ground and pile. In the analysis, one row of group piles is taken out, and incremental analysis is carried out using this row as a two-dimensional model (row pile model). The pile circumferential ground is substituted for the equivalent horizontal ground spring, and the ground displacement is input via the horizontal ground spring (response displacement method). Further, the foundation slab is set as rigid, and the condition of pile tip is horizontal roller in which the subsidence of the pile tip is not allowed.

Future Tasks Involved in Secondary Design Method for Steel Pipe Piles

The secondary design method for foundation structures was shown in the new *Recommendations*. However, in order to promote the practical calculation of the secondary design method targeting steel pipe piles, the following three tasks remain to be solved:

—As regards the M-ϕ relationship of steel pipe piles, its revision is underway at the Architectural Institute of Japan, and thus it will be necessary to start a regular discussion on the M-ϕ relationship after grasping the revision results. More specifically, in order to promote regular discussion, it will be necessary to follow the *Strength and Deformation Capacity of Foundation Members* planned to be published in fiscal 2021.

—While the calculation method for the pile stress of group pile frame model has been shown, the specific analysis code has not yet been prepared.

—The effect of secondary design on steel pipe piles has not yet been grasped.

As regards the first task, the review is underway at the Architectural Institute of Japan, which is proceeding as scheduled. As regards the second and third tasks, the Research Working Group on the Secondary Design of Steel Pipe Pile Foundations was established within the Japanese Society of Steel Construction in 2019, where examinations are underway.

• Improvement of Group Pile Frame Model

The analysis program exclusively used

Fig. 2 Overview of Group Pile Frame Model

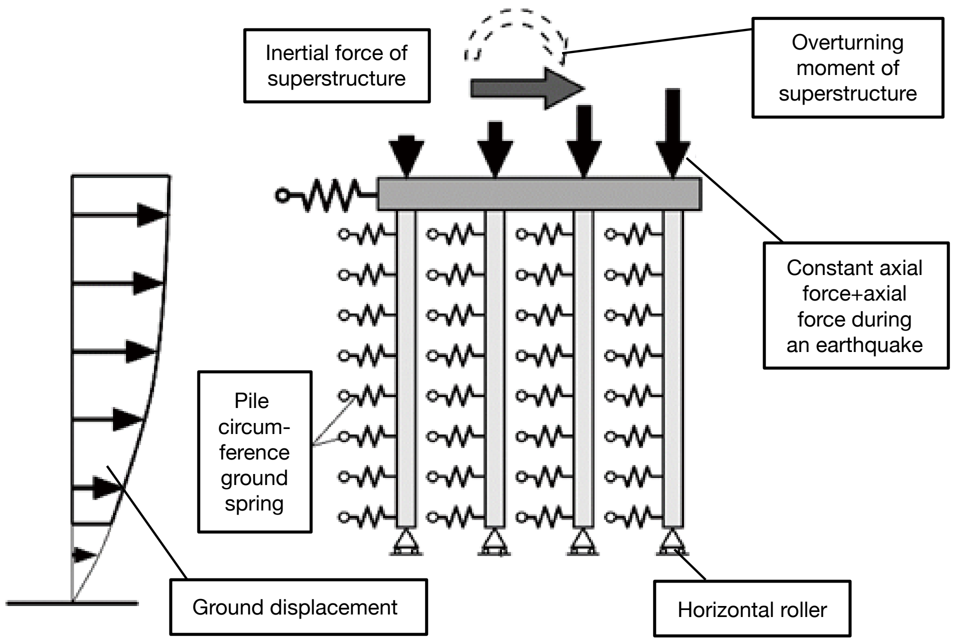
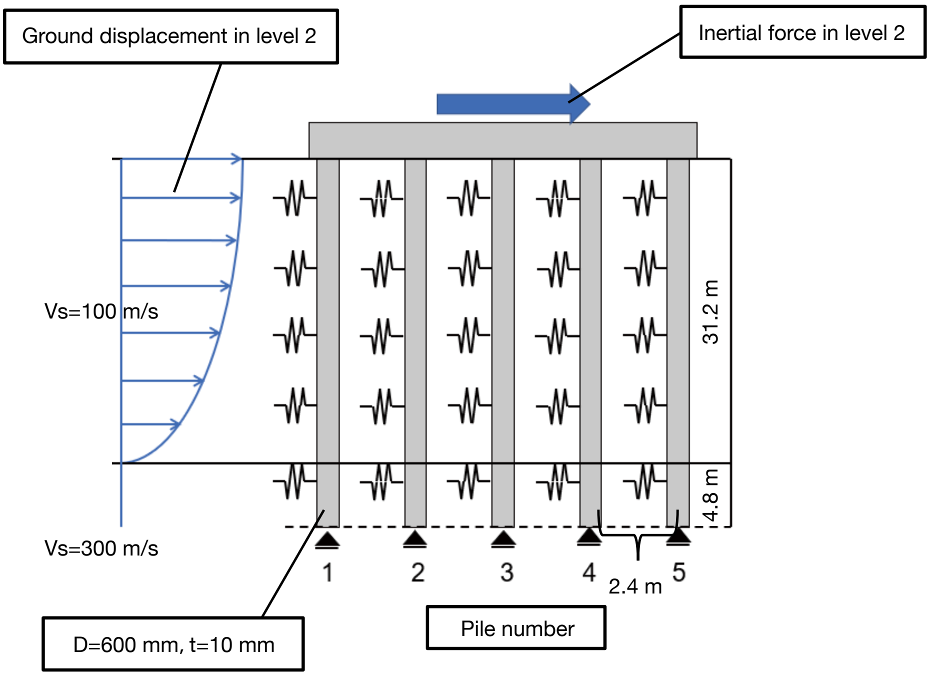


Fig. 3 Analysis Conditions



for the group pile frame model proposed in the new *Recommendations* and in common use has not yet been prepared, and improvement has been called for. To cope with such a situation, the Hijikata Laboratory of the Shibaura Institute of Technology has developed a calculation code (Group Pile EASY-PILE) for use for the group pile frame model. This code has been developed based on the original program of the Hijikata Laboratory, to which diverse functions have been added.

The code has been subjected to verification by the Research Working Group and has been made open free of charge since October 2019. The code is ready for use by everyone.

• Grasping the Effect of Secondary Design on Pile Foundations

While the application of secondary design has not been required for the design of pile foundations in Japan, secondary design has been applied in the construction of important buildings as the need arises. In conventional secondary design, the ultimate state of piles is commonly evaluated by the use of the ultimate strength, and the pile head plastic hinge that can be allowed to be formed in the group pile has been limited in one place. On the other hand, in the new *Recommendations*, it is prescribed that the ultimate state of steel pipe piles can be evaluated even by the use of ultimate deformation, and thus it has become possible to form the plastic hinge at multiple places on the pile heads of group piles.

However, examples of calculations employing the group pile frame model proposed in the new *Recommendations* are less available, and examples of examinations in which the pile structure is evaluated by the use of ultimate deformation are also less available. Given such a situation, the Research Working Group has promoted stress calculations pertaining to steel pipe piles under various conditions to accumulate related knowledge. Examples of examination results by the Research Working Group are introduced below:

Fig. 3 shows example of examination results targeting group piles. Table 3 shows the specification of steel pipe piles used in the examination, and Table 4 the ground conditions. In the examination, the constant load working on one pile was set at 627.5 kN, the design inertia force working on the fifth row of piles 1,500 kN, and the design overturning moment 32,500 kN. The M-ϕ relationship (moment curvature) of steel pipe piles was settled in conformity with the *Recommendations for Plastic Design of Steel Structures* of the Architectural Institute of Japan. The calculation was carried out up to the stage of surpassing the design load, and the point where a certain pile of steel pipe piles reached the limit deformation (limit curvature in this occasion) was set as the end point, and then the horizontal strength of the pile slab at this end point was calculated.

Fig. 4 shows the M-ϕ relationship of the pile head in the limit state, and Fig. 5 the hinge formation state. As seen in the figures, when the steel pipe pile is evaluated in terms of limit deformation, plural hinges are formed on the pile head, and as a result it was confirmed that the horizontal strength of the foundation slab was increased by 36% compared to the case in which the pile is evaluated in terms of limit strength (the hinge is allowed to form at one point).

Table 3 Specifications of Steel Pipe Piles

Length	Diameter	Thickness	Yield stress	Young's modulus	Pile spacing ratio
36 m	600 mm	10 mm (same for each position)	357.5 N/mm²	205000 N/mm²	4.0

Table 4 Ground Conditions Used for Analysis

Condition	Surface ground	Deep ground
Ground characteristics:	Sandy ground	Sandy ground
Depth	0~31.2 m	31.2~36 m
Unit volume mass	1.8×10^3 kg/m³	1.8×10^3 kg/m³
Friction angle	21.3°	46.6°
Shear wave velocity	100 m/s	300 m/s
Poisson's ratio	0.33	0.33

Fig. 4 M-Φ Relationship of Pile Heads

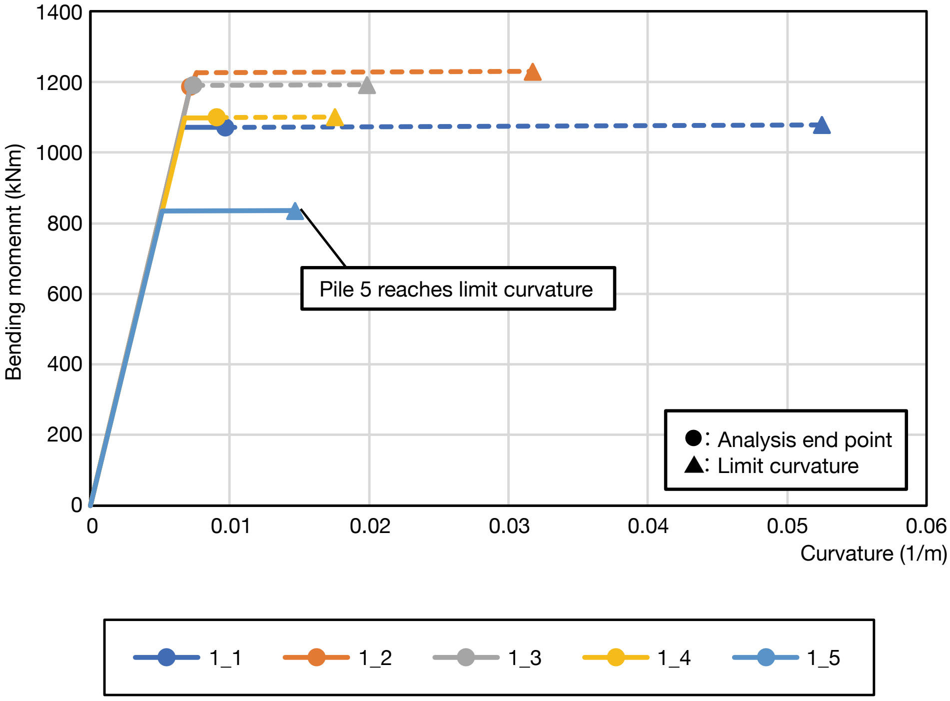
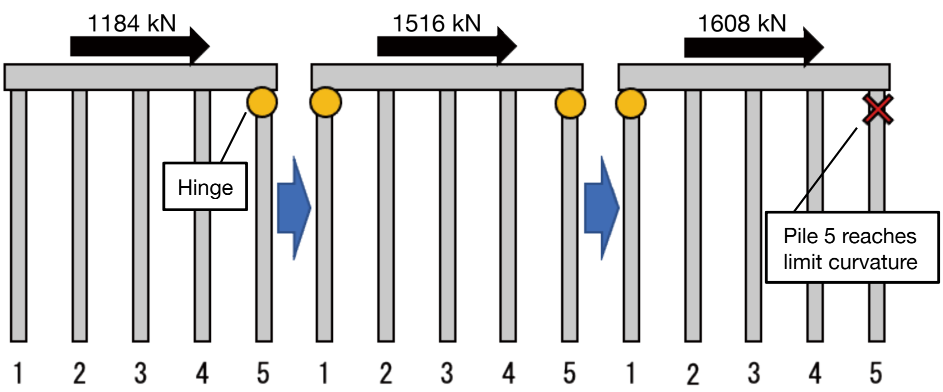


Fig. 5 Hinge Formation State



Refined Secondary Design Method

In this article, the revision content of the *Recommendations for Design of Building Foundations* was introduced. Further, examples of trial calculation for the secondary design method for steel pipe piles were shown. In the trial calculation, it was confirmed that, when steel pipe piles are evaluated in terms of limit deformation (limit curvature), the strength is significantly increased compared to the case in which the strength is evaluated by the use of the existing calculation method and that the design and application of steel pipe piles can be expected to be rationalized.

Expectations are high for the steady brush-up of the secondary design method for steel pipe piles so as to surely establish the secondary design method for steel pipe piles.

Development of Seismic Retrofitting Technology for Existing Bridge Abutment Foundations Using Large-Scale Shaking Table Test

by Michio Ohsumi and Yong Yang, Public Works Research Institute

Michio Ohsumi: After completing the master's course at the Graduate School of Engineering, The University of Tokyo, he entered the Ministry of Construction in 1996. He currently serves as Chief Researcher at the Center for Advanced Engineering Structural Assessment and Research, Public Works Research Institute. His specialized field is seismic engineering.

Yong Yang: After completing the doctor's course at the Graduate School of Engineering, The University of Tokyo, he served as Project Researcher from 2016 to 2017, Institute of Industrial Science, The University of Tokyo. He currently serves as Research Specialist at the Public Works Research Institute.

Toward Highly Resilient Highway Networks

Ground liquefaction can cause the lateral flow acting on the bridge foundations. In the Kobe earthquake in 1995, it was confirmed that the liquefaction lateral flow could lead to destructive damage to existing bridge structures. It is necessary to develop the seismic retrofitting technology for existing bridges located on liquefiable ground to improve the highway network resilience. To attain this goal, at the Center for Advanced Engineering Structural Assessment and Research (CAESAR) of the Public Works Research Institute, the experimental researches had been carried out from 2014 to 2019.

In the first series of experiment research, the dynamic centrifuge test with 1/60 scale[1] was conducted. In the second series, the shaking table test with 1/10 scale[2] was conducted. In the third series, the shaking table test with 1/4.5 scale was carried out.

This article describes the result of the seismic retrofitting effect of the large-scale shaking table test in the third series of experiment. In addition, the estimating model of the earth pressure due to liquefaction established based on the series of experiment results, which is the important consideration issue in the retrofitting design of existing bridges, is also introduced.

Shaking Table Test with 1/4.5 Scale

• Bridge Abutment Models

The retrofitting method shown in Fig.1 was adopted in the shaking table test. This proposal was made through joint research with the Japanese Association for Steel Pile Piles (JASPP). This retrofitting method can avoid traffic hindrance because the retrofitting work is only carried out at the side of bridge abutment. For the steel pipe sheet pile wall used in this retrofitting method, the retrofitting difficulty is expected to be decreased, especially in the case when the heavy construction machine cannot be used due to narrow construction yard.

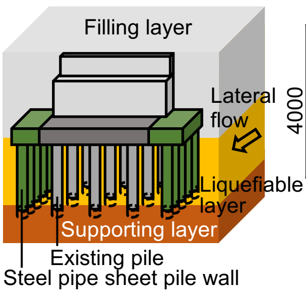
Fig. 2 shows the outline of experiment models with 1/4.5 scale. As shown in Fig. 2(a), model 1 (without retrofitting) and model 2 (with retrofitting) were set in the same soil tank. Model 1 was designed according to the former standard[3]. The existing piles of two models were made of reinforced concrete material with a diameter of 101.6 mm. The steel pipes used for sheet pile wall in the model 2 had a diameter of 100 mm. As shown in Fig. 2(b), the filling layer had a thickness of 1780 mm and the liquefiable layer had a thickness of 2200 mm.

Photo 1 shows the shaking table with nickname 'E-Defense', which is owned by National Research Institute for Earth Science and Disaster Resilience (NIED). The soil tank was set on the shaking table.

• Seismic Retrofitting Effect

Under the seismic ground motion (2-I-I-3)[4] with amplitude-adjusted waves observed in the Tohoku earthquake, the maximum response of tensile strain of existing piles and steel pipes is shown in Fig. 3. In the model 1 without retrofitting, the maximum tensile strain of the middle and front existing piles at the

Fig. 1 Proposal of Retrofitting Method



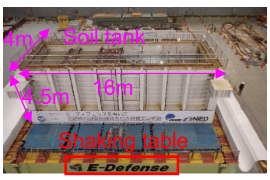


Photo 1 Soil tank and E-Defense shaking table

Fig. 2 Outline of Experiment Models (unit: mm)

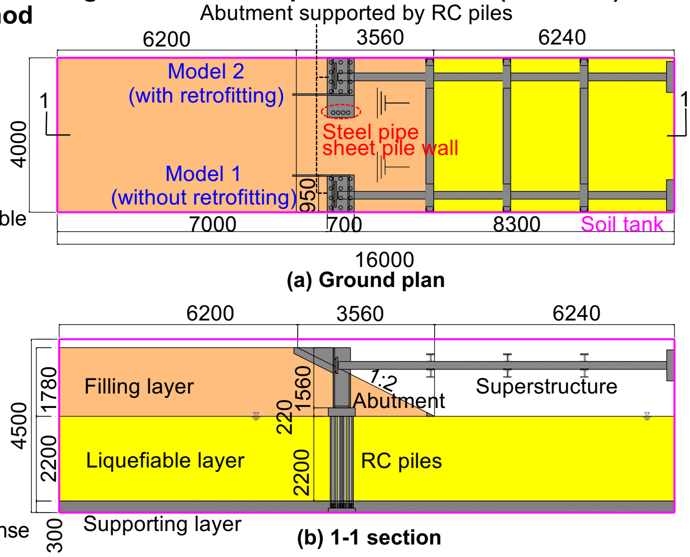
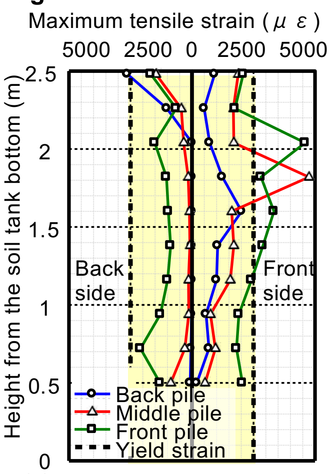
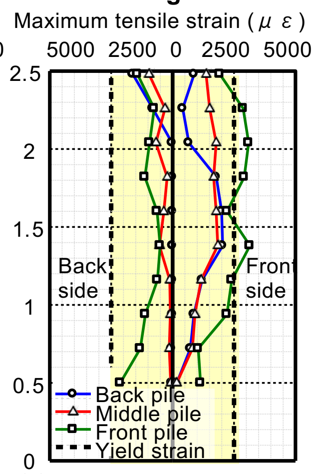


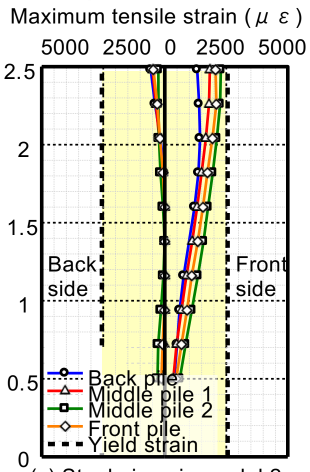
Fig. 3 Maximum Tensile Strain of Existing Piles and Steel Pipes



(a) Existing piles in model 1 without retrofitting



(b) Existing piles in model 2 with retrofitting



(c) Steel pipes in model 2

Fig. 4 Coefficient for Depth Effect (C_1)

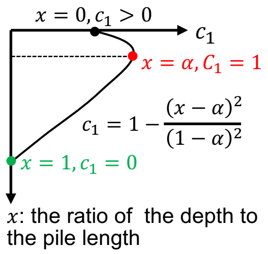


Fig. 5 Coefficient for Pile Arrangement Effect (C_2)

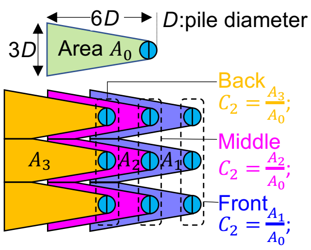
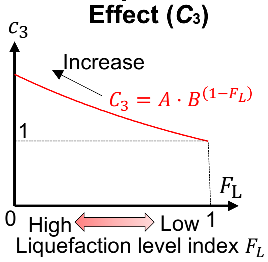


Fig. 6 Coefficient for Liquefaction Effect (C_3)



front side was obviously more than yield strain. However, in the model 2 with retrofitting, the maximum tensile strain of the middle and front existing piles was obviously decreased at the depth about 2 m and the maximum tensile strain of the middle existing pile was less than yield strain. The maximum tensile strain of steel pipes of sheet pile wall was less than yield strain, showing that the sheet pile wall can remain in elastic status to provide enough capacity.

Further information about the experimental results can be found in the reference[5].

Estimating Model of Earth Pressure

The earth pressure (*EP*) acting on the pile foundation in the liquefiable layer is modeled as $EP=C_1 \cdot C_2 \cdot C_3 \cdot q$. The parameter q is the overburden pressure due to filling layer. The coefficients C_1, C_2, and C_3 are for the effect of depth, pile arrangement, and ground liquefaction, respectively.

• Coefficient for the Depth Effect (C_1)

To accurately evaluate the earth pressure due to liquefiable ground, the coefficient C_1 is set to reflect the distribution character of earth pressure on pile. As shown in Fig. 4, when the ratio x of the depth to the pile length is equal to 0, the result of C_1 is more than 0, meaning the earth pressure occurring at the pile top; when x is equal to parameter α, C_1 is equal to 1, corresponding to the maximum of earth pressure; when x is equal to 1, C_1 is equal to 0, meaning there is no earth pressure occurring at the pile bottom.

The parameter α calculated by fitting the results of three series of the experiments is 0.35.

• Coefficient for the Pile Arrangement Effect (C_2)

It was measured that the earth pressure on pile at the back side of abutment was obviously more than that of other piles. Thus, the coefficient C_2 is introduced to consider the pile arrangement effect. As shown in Fig. 5, the areas A_0 and $A_{1\sim3}$ mean the magnitude of the earth pressure on piles. The coefficient C_2 is defined as the ratio of these areas of piles in pile group to that of single pile.

• Coefficient for the Liquefaction Level Effect (C_3)

The earth pressure also depends on the liquefaction level. The coefficient C_3 is introduced to reflect the liquefaction level effect. As shown in Fig. 6, C_3 is effective when the liquefaction level index F_L is less than 1.

By fitting the experimental results, the parameters A and B in the formula of C_3 were obtained 1.0 and 2.0, respectively.

Further information about the evaluation model of the earth pressure due to liquefaction can be found in the reference[6].

◆◆◆

The authors hope that the above research achievements related to the seismic retrofitting technology for existing bridge can contribute to build the resilient highway network to reduce the earthquake-induced loss in the future.

Acknowledgment

The research had been supported by Council for Science, Technology and Innovation (CSTI), Cross-ministerial Strategic Innovation Promotion Program (SIP), "Enhancement of societal resiliency against natural disasters" (Funding agency: JST). We offer our thanks to those of the Council for Science, Technology and Innovation who had supported this research project.

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Feature Article: Initiatives for Building Resilient Foundation Structures (4)

Reformation Design of Existing Port and Harbor Steel Structures

—Effect of Modeling in Numerical Analysis of Seismic Response—

by Eiji Kohama, National Institute of Maritime, Port and Aviation Technology

Eiji Kohama: After finishing the doctoral course at the Faculty of Engineering, Graduate School of Hokkaido University, he entered Port and Harbor Research Institute of the Ministry of Transport in 2000, and then served as senior researcher at the Port and Airport Research Institute. He assumed his current position as head at Earthquake and Structural Dynamics Group, National Institute of Maritime, Port and Aviation Technology in 2016. His specialized field is geotechnical earthquake engineering.

A study was made noticing the reformation design of port and harbor structures by means of new installation of steel sheet piles in front of the existing sheet pile quay wall with vertical pile anchorage.

As regards the reformation design, it is accepted as a safety-side design that the existing structure is not taken into account at the stage of analysis but that analysis is made by means of modeling only the newly-installed structure, and as a result design employing such analysis methods is increasingly being applied. Meanwhile, in respect to this, some concerns arise—the safety side in design may be excessively assessed and there are many unclear aspects involved in the original construction process of existing structures and the effect of the subsequent reformation process on existing structures.

Then, examinations were made of the effect of the difference in dealing with existing structures in the event of analysis on the seismic response analysis result, employing a 2D FEM effective stress analysis program (FLIP).

Seismic Response Analysis

Fig. 1 shows the quay targeted for seismic response analysis. Fig. 2 shows the construction of the existing sheet pile quay wall with vertical pile anchorage and subsequent reformation processes to increase the quay depth by the use of sheet pile structure with coupled-pile anchorage. Processes 1-3 shown in the figure cover the construction process for the existing quay structure, and processes 4-7 show the process to increase the quay depth by the use of the new sheet pile structure.

In order to examine the effect of the existing structure and the subsequent reformation process on the seismic response analysis, a comparison study was made by means of modeling of three cases shown in Table 1.

- In Case 2, the seismic response was analyzed by taking into account both the existing sheet pile quay and the new sheet pile structure and all the processes shown in Fig. 2 in the dead weight analysis prior to the seismic response analysis.
- In Case 3, the seismic response was analyzed by taking into account the existing sheet pile quay but by taking into account only the process to construct the new pile structure in the dead weight analysis, and analysis was made assuming the simultaneous installation of both the existing and new sheet pile structures (refer to Fig. 3).
- In Case 1, the seismic response was analyzed by ignoring the existing sheet pile quay structure, by taking into account only the new sheet pile structure installation process even in the dead weight analysis that simulates the reformation process, and without taking into account the existing sheet pile structure.

Fig. 4 shows the results of seismic response analysis employing input seismic motions. As a result, the residual displacement in Case 2 where both the existing sheet pile structure and the new sheet pile structure are taken into account was about half that in Case 1 where the existing sheet pile structure was ignored and in Case 3 where the existing structure installation process was ignored, and further the residual displacement in Case 1 was nearly similar to that in Case 3 (Table 2). The bending moment of newly-installed sheet pile in Case 2 was larger than those in Cases 1 and 3, and the bending moment in Case 1 was nearly similar to that in Case 3 (Fig. 5).

Useful Knowledge for Reformation Design

When noticing the ending point of the dead

Fig. 1 Quay Targeted for Study

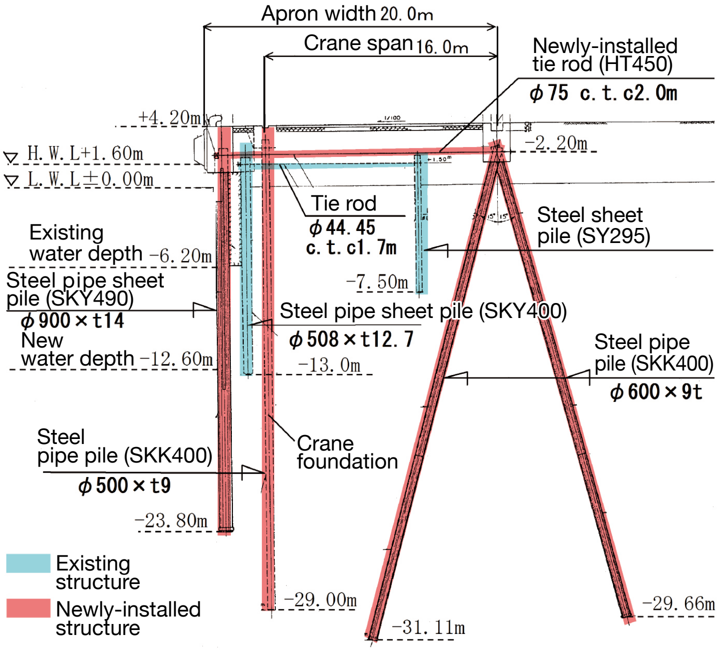


Table 1 Modeling Cases

Case	Modeling	Process
1	Modeling only of newly-installed sheet pile structure	4
2	Modeling of newly-installed and existing sheet pile structures; Construction process is taken into account	7
3	Modeling of newly-installed and existing sheet pile structures; Existing sheet pile structure construction process is not taken into account	4

weight analysis prior to the seismic response analysis, the tension force working on the newly-installed tie rod was small in Case 2, and the tension force in Case 1 was nearly similar to that in Case 3 (Table 3). Further, the distribution of the maximum shear strain in Case 1 resembled that in Case 3, and their distribution trend differed from the trend in Case 2 (Fig. 6).

Therefore, the seismic response analysis results in Case 3 where the existing pile structure was ignored were similar to those in Case 1 where the existing structure installation was ignored, and it is considered that the tension force basically did not work on the existing pile structure and that the existing sheet pile structure basically did not contribute to the behavior of new pile structure during working of seismic motions in Case 3.

In the case of reforming an existing sheet pile quay wall to a new sheet pile quay wall, the following fact has become clear from this study—in modelling not only the new structure but also the existing structure, in cases when detailed construction processes are not taken into account, it is impossible to estimate the effect of the existing structure on the reformed structure.

Reference

1) Iai, S., Matsunaga, Y. and Kameoka, T.: Strain space plasticity model for cyclic mobility, Soils and Foundations, Vol. 32, No. 2, pp. 1-15, 1992.

Fig. 2 Processes to Reform Existing Quay (Processes Taken into Account in Dead Weight Analysis and Seismic Response Analysis in Case 2)

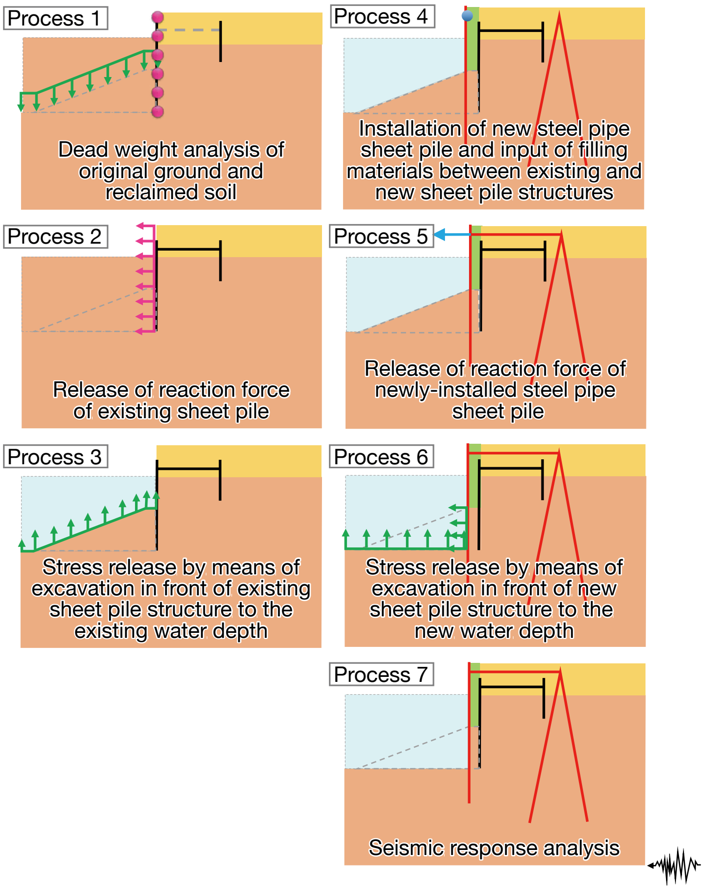


Fig. 3 Elimination of Existing Quay Construction Process (Processes Taken into Account in Dead Weight Analysis and Seismic Response Analysis in Case 3)

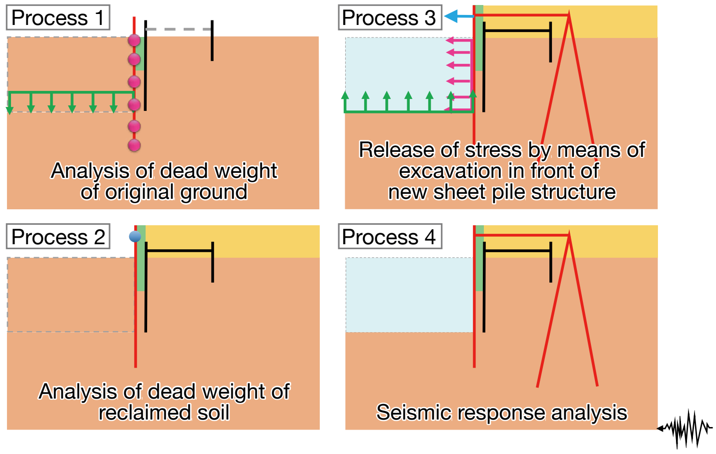


Table 3 Tension Force Working on Tie Rod at the Dead Weight Analysis Ending Stage

Case	Tension force working on new tie rod	Tension force working on existing tie rod
1	405.0 kN	—
2	117.1 kN	132.9 kN
3	351.7 kN	29.2 kN

Fig. 5 Distribution of Bending Moment in Newly-installed Steel Pipe Sheet Pile

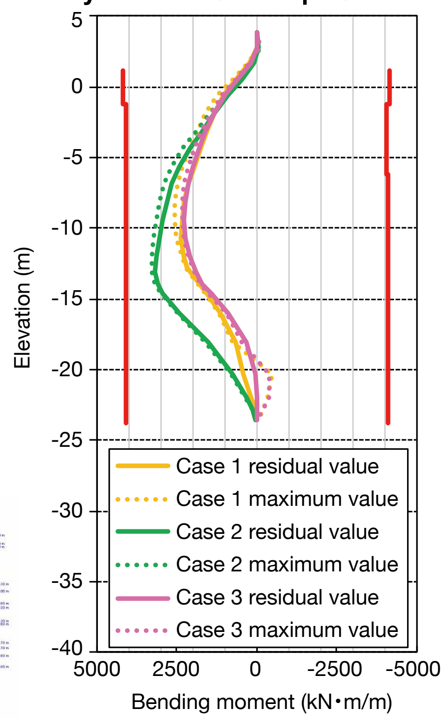


Table 2 Residual Displacement of Sheet Piles in Seismic Response Analysis

Case	Residual displacement at Pile Crest
1	1,372 mm
2	738 mm
3	1,353 mm

Fig. 4 Input Seismic Motion

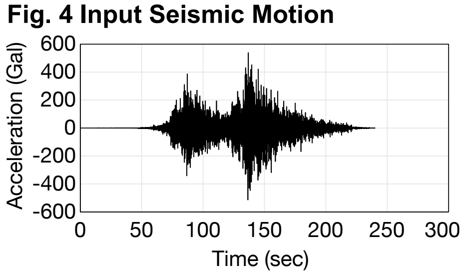
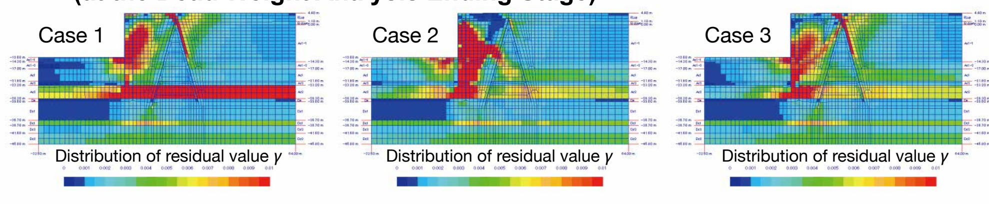


Fig. 6 Distribution of Maximum Shear Strain γ_{max} (at the Dead Weight Analysis Ending Stage)



Performance of Steel Sheet Piles for the Stability of River Embankments under Earthquakes

by Jun Otani, Professor, Kumamoto University and Kiyonobu Kasama, Associate Professor, Kyushu University

Jun Otani: He is Trustee and Vice President at Kumamoto University. He served as research fellow at Scrips Institute of Oceanography, University of California, USA in 1987 and received Ph.D at University of Houston, USA (Civil Engineering) in 1990. His research fields cover soil mechanics, geotechnical engineering and application of X-ray CT.

Kiyonobu Kasama: He is an associate professor at the department of civil engineering, Kyushu university. He holds Ph.D from Kyushu University, where he has been teaching and conducting research for 20 years. He received the Outstanding Paper Award for Young Researchers of the Japanese Geotechnical Society and the society of material science.

Introduction

Steel sheet piles have been used mainly for the purpose of temporary works such as excavation and pre-construction structures. However, recently those application fields have been expanded to the permanent structures such as foundations. PFS (Partial Floating Sheet pile) method as shown in Fig. 1 is the one of this technique which was originally developed as the countermeasure methods for subsidence of surrounding soft ground due to river embankment constructions.

Recently more wide application fields have been expected for the steel sheet pile method and this is the objective of Technical Committee (TC) under the International Press-in Association. In this article, the activities of the WG (Working Group No.1 chaired by Prof. Kasama) in the TC which is the performance of the steel sheet piles under 2016 Kumamoto Earthquake in Japan are briefly summarized.

Steel Sheet Piles at the Site

Fig. 2 shows the location and purpose of the steel sheet pile construction method

Fig. 1 PFS Method

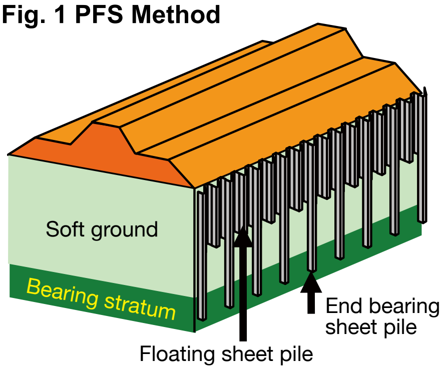
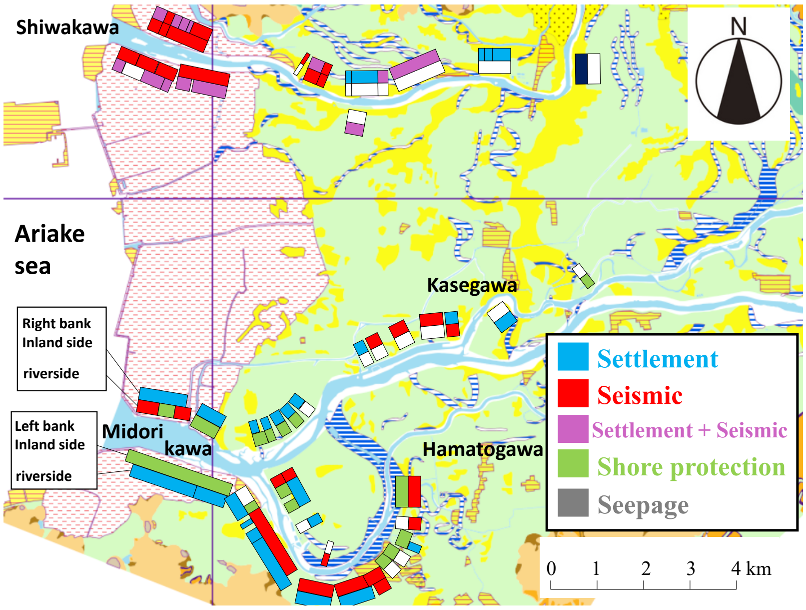
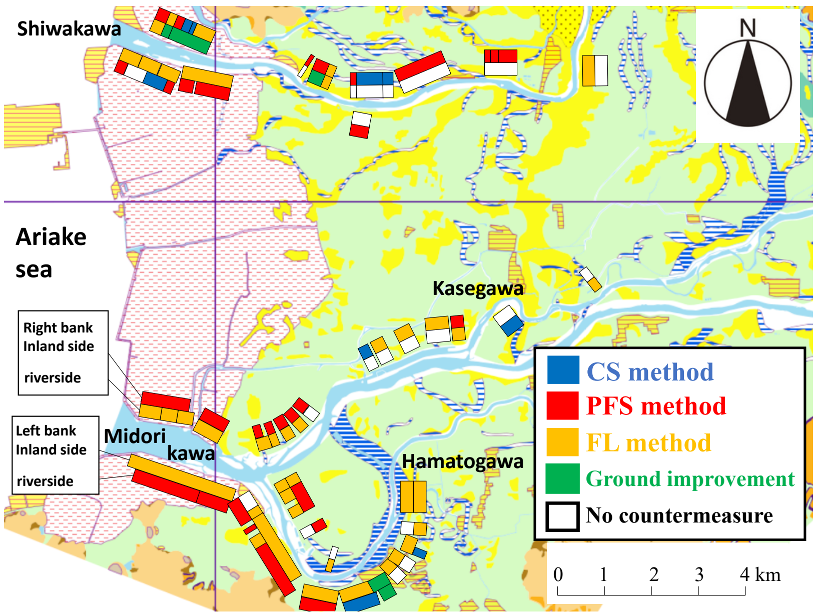


Fig. 2 Location of Steel Sheet Pile Method



(a) Purpose of countermeasures



(b) Type of steel sheet pile method

in the Kumamoto Plain, Japan. This area is located at Kumamoto Prefecture in the middle of Kyushu Island. There are two main rivers called Shira River and Midori River including two sub-rivers called Kase River and Hamatogawa River. In this figure, the left and right sides of the river are indicated in two colors, respectively, and the color of the side near the bank indicates the front side of the river (outside of the bank) and the side farther from the bank indicates the back side of the river (inside of the bank).

In Shirakawa River, the inner side of the embankment near the house was reinforced for the purpose of only subsidence or both subsidence and earthquake resistance, while the outer side of the embankment was reinforced only for earthquake resistance. A reinforcement of the embankment was carried out on the inner side of the Midori and Hamato Rivers to prevent subsidence, and on the outer side of the embankment, reinforcement of the embankment was carried out for the purpose of constructing a revetment using earthquake resistance measures or sheet pile method. Most of the river embankments were reinforced by the PFS and FL (floating sheet pile) methods on the inner and outer sides of the embankment, respectively.

Table 1 shows the statistics of the sheet pile lengths used for each method in the rivers. The column of PFS method in this table shows the sheet pile lengths of both end bearing and floating sheet piles used and their sheet pile length ratios. The average sheet pile lengths for end bearing (CS method) and floating method (FL method) are 34.2 m and 14.6 m, respectively, indicating that the end bearing sheet pile is about twice as long as the FL method. In the meantime, the average end bearing and floating sheet pile lengths of the PFS method are 38.7 m and 25.5 m, respectively.

Performance of Steel Sheet Piles under Earthquake

Fig. 3 shows the probability density distributions of the subsidence of each type of embankment. The legend in the figures shows the type of sheet piling method for the inner side of the embankment. The subsidence of the no countermeasure section is measured in total of 551 points at the sites and is distributed over a wide range of -1.28 m to 1.56 m, whereas the subsidence of the section reinforced by the various steel sheet piling methods is concentrated in the range of -0.08 to 0.39 m.

In order to investigate the effect of various methods and combinations of methods on subsidence control, Table 2 summarizes the statistics of subsidence caused by earthquakes. When the inner side of the embankment was reinforced by the PFS method, the average subsidence of the embankment reinforced by the FL method (outside of the embankment), ground improvement (outside of the embankment), and no countermeasure (outside of the embankment) was 0.11 m, 0.08 m, and 0.04 m, respectively, which is a small value. When the embankment was reinforced by the FL method, the mean subsidence was slightly higher than 0.16 m and 0.13 m for either the inner or outer side of the embankment, respectively.

In addition, the subsidence of the embankment reinforced on the outside of the embankment tended to be larger than that of the other methods, for example, 0.39 m was observed at the point where the embankment was reinforced by the FL method. The average subsidence of the combination of FL reinforcement and ground improvement was 0.08 m and 0.09 m, respectively.

Conclusions

In recent years, there are a large number of natural disasters such as heavy rains and earthquakes, and the sheet piles can be an effective countermeasure such as the stability of the river embankments. A construction technique called press-in method has also accelerated the application of the sheet pile method. Finally, it is hoped that more wide varieties of the steel sheet piles should be used.

Table 1 Statistics of Sheet Pile Length

	No.	Mean (m)	Mode (m)	*COV**	Min (m)	Max (m)
CS method	35	34.2	37	0.24	14	42
FL method	121	14.6	15	0.41	8	30
PFS method						
End bearing	99	38.7	40.5	0.13	28	53
Floating	99	25.5	25.5	0.22	11.5	36.5
Ratio**	99	0.66	0.86	0.20	0.27	0.90

* The coefficient of variation
** The ratio of floating sheet pile and end supporting sheet pile

Fig. 3 Probability Density Function of Settlements

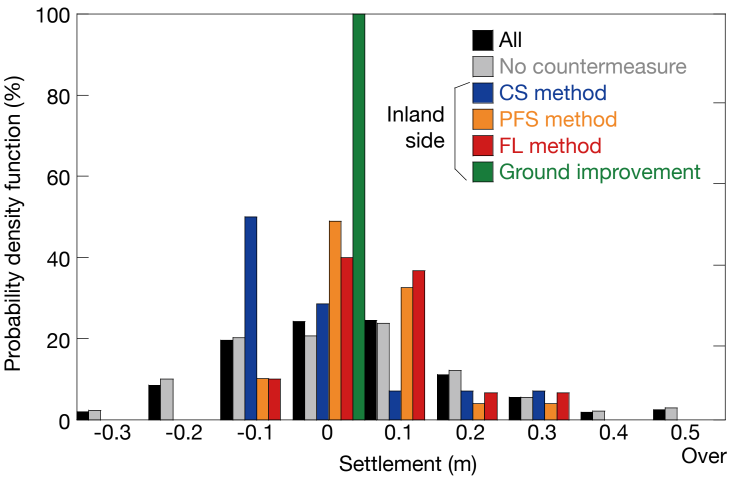


Table 2 Statistics of Seismic Settlement

Inland side	River side	No.	Mean (m)	*COV**	Max (m)	Min (m)
All		645	0.10	2.24	1.56	-1.28
No countermeasure		551	0.10	2.36	1.56	-1.28
Countermeasure		64	0.09	1.06	0.39	-0.08
CS method	FL method	4	0.15	1.44	0.39	-0.08
	GI**	2	0.03	1.41	0.07	0.00
	No	8	0.01	5.06	0.13	-0.07
PFS method	FL method	29	0.11	0.84	0.38	0.00
	GI**	3	0.08	1.09	0.15	-0.02
	No	17	0.04	0.90	0.14	-0.01
FL method	FL method	4	0.08	1.11	0.15	-0.04
	GI**	3	0.09	0.73	0.16	0.05
	No	8	0.16	0.39	0.26	0.08
No	FL method	15	0.13	1.00	0.39	-0.06
GI**	GI**	1	0.05	—	0.05	0.05

* The coefficient of variation
** Ground improvement

Towards the Realization of Resilience Innovation for 2035

by Haruo Hayashi
National Research Institute for Earth Science and Disaster Resilience

Haruo Hayashi: He finished the doctoral course at the Psychology Department, University of California, Los Angeles in 1983. He became Professor, Disaster Prevention Research Institute, Kyoto University in 1996 and assumed his current position as President, National Research Institute for Earth Science and Disaster Resilience in 2015. His specialized field is social psychology and risk management.

Can Japan Hurdle National Crisis-level Disasters in the First Half of the 21st Century?

It is forecast that a series of national crisis-level natural disasters will occur in Japan in the first half of the 21st century. To cope with such a situation, it will be necessary for Japan to hurdle the fateful crisis triggered by lingering predicaments that will follow these national crisis-level natural disasters. (Refer to Fig. 1)

The major factor attributable to the occurrence of such national crisis-level disasters is two great earthquakes forecast to occur in the near future (refer to Fig. 2): One is the Nankai Trough earthquake that has occurred every century since the seventh century and is forecast to occur before and after 2035 (refer to Fig. 3). The other is the inland earth-

Fig. 1 Existential Risk for Japan in the First Half of 21st Century

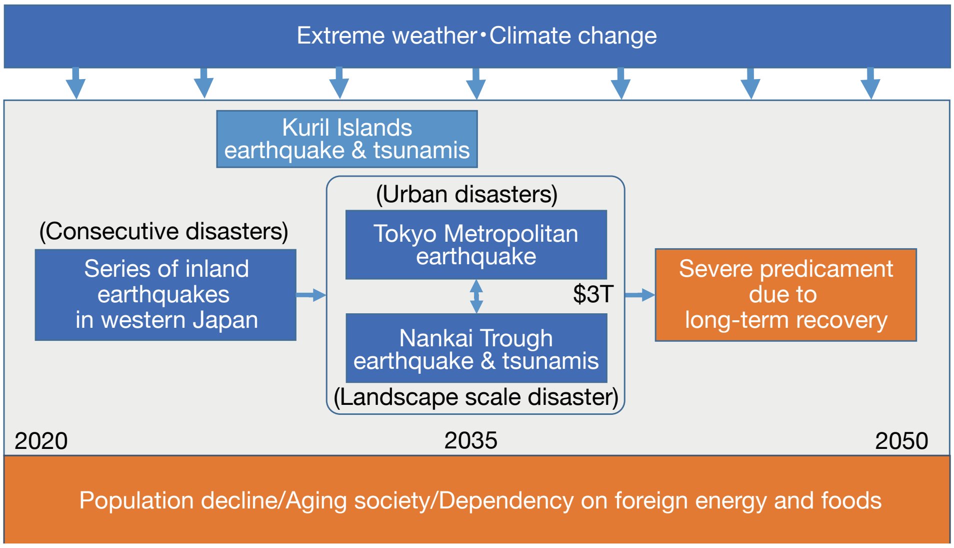


Fig. 2 Expected National Crises

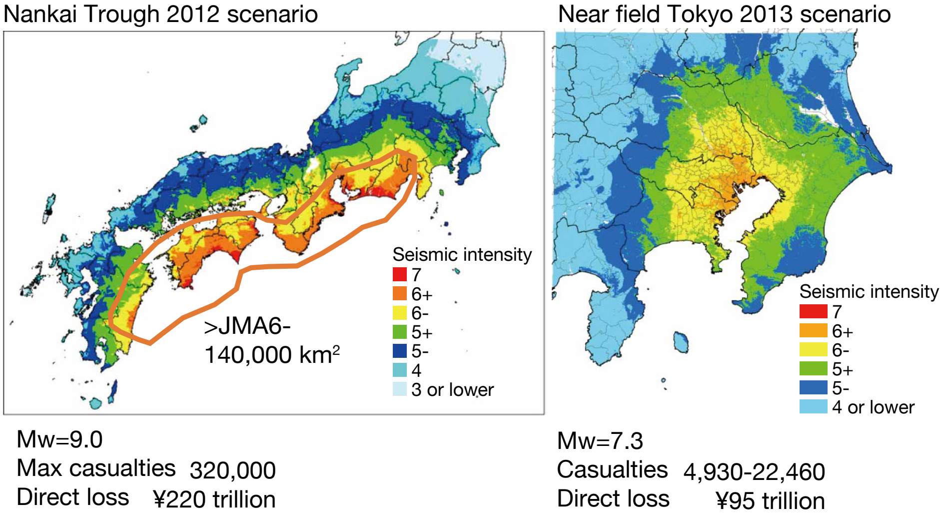
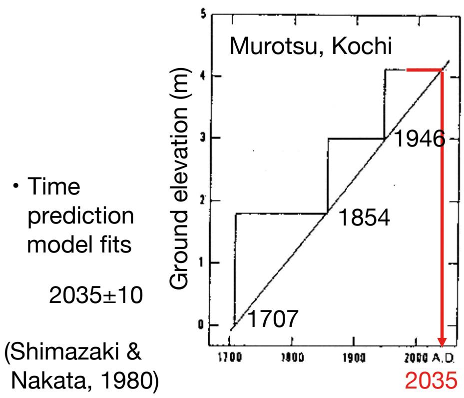


Fig. 3 When Would Be Next Nankai?



quake that is feared to occur in the Tokyo Metropolitan area nearly simultaneously with the Nankai Trough earthquake. In addition, it will be necessary to take into account the serious damage caused by a series of inland earthquakes that are forecast to frequently occur centering on the western Japan area during that period.

Serious damage brought about by a series of such earthquake disasters will require restoration and reconstruction processes that will span a long time. Further, it is forecast that such restoration and reconstruction scenarios will be made worse due to increasing extreme climate events and ongoing global warming.

The reason why I dare to call these disasters national crisis-level disaster is that Japan has certainly experienced notable political changes after the occurrence of the following four most recent Nankai Trough earthquakes. In the Keicho earthquake of 1605, political power shifted from the Toyotomi administration to the Tokugawa government, in the Hoei earthquake of 1707 from the Tokugawa head family government to the Kishu Tokugawa family government, in the Ansei earthquakes of 1854 from the Tokugawa government to the Meiji government, and in the Showa earthquakes of 1944 and 1946 from the Empire of Japan to Japan of today.

The monetary damage caused by the largest-scale natural disasters Japan has experienced in the postwar period amounted to 10 trillion yen for the Great Hanshin Earthquake of 1995 and 17 trillion yen for the Great East Japan Earthquake of 2011. For the reconstruction from these earthquakes, a sum of 17 trillion yen was required for the Great Hanshin Earthquake, and a sum of 32 trillion yen has thus far been required for the Great East Japan Earthquake. The direct damage from the national crisis-level disasters forecast to occur in the future is estimated to surpass 300 trillion yen in the worst case, and when indirect damage is also added, the huge amount of 1,250 trillion yen will be required to attain full restoration according to trial calculations by the Society of Civil Engineers (refer to Fig. 4).

Portugal was once a superpower that divided the world into two, but the country's fortunes declined steadily starting with the Lisbon earthquake of 1755 and has shown no notable recovery even in recent times. In order for the Nankai Trough earthquakes in the 21st century not to play a triggering role of bringing about the decline of Japan, it will be necessary to establish by 2035 a society with redundant "resilience" that can hurdle such national crisis-level disasters.

What Is Disaster Resilience?

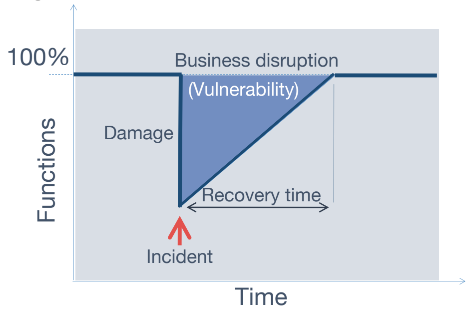
Disaster resilience can be modelled as shown in Fig. 5. Due to damage caused by disasters, there are cases in which the system partially or entirely loses its function. When the lost function of the system may be recovered by starting recovery system after a certain period of time, there arises a triangle of business disruptions. In other words, this triangle can be explained as the vulnerability that the system possesses. How to minimize this triangle area depends on the improvement of disaster resilience.

To attain the above goal, there are two independent strategies—improving prevention capabilities to lessen the damage and preparedness capabilities to speed the recovery. The best strategy is the combined use of these two capabilities to improve the comprehensive capabilities of the strategy (refer to Fig. 6). Specifically, the following four operations are cited in order to improve disaster resilience (refer to Fig. 7):

Fig. 4 Huge Damage Is Expected

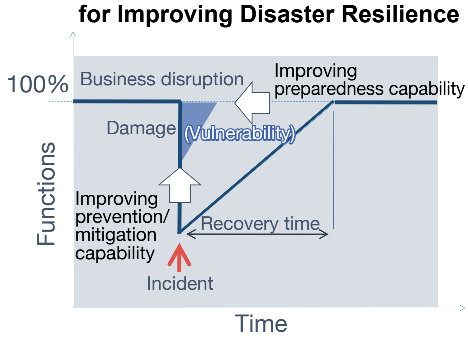
Earthquake	Nankai Trough (Tōkai, Tōnankai, Nankai)		Tokyo Near Field		Tohoku 2011	Kobe 1995
	2012 scenario	2003 scenario	2013 scenario	2005 scenario		
Magnitude	M9.0	M8.7	M7.3	M 7.3	M 9.0	M 7.3
Killed/missing	80,000 – 320,000	24,000	5,000 – 22,500	11,000	19,294	6,434
Injured	257,000 – 623,000	300,000	90,000 – 120,000	240,000	6,100	44,000
Buildings–collapsed	627,000 – 1,346,000	450,000		200,000	126,500	105,000
Buildings–heavy damage	----	----	----	------	227,600	144,400
Buildings–burned	50,000 – 750,000	90,000	38,000 – 412,000	650,000	---	7,400
Evacuees (max)	----	6,000,000	7,200,000	7,500,000	480,000	320,000
Direct damage (¥ trillion)	220	81	95	112	17	10

Fig. 5 What Is Disaster Resilience



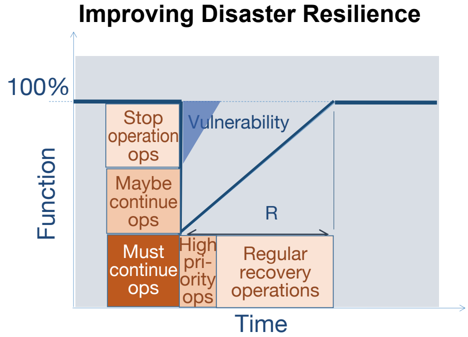
Adapted from MCEER's Resilience Framework

Fig. 6 An Integrated Approach Is a Key for Improving Disaster Resilience



Adapted from MCEER's Resilience Framework

Fig. 7 Four Operations Prioritized for Improving Disaster Resilience



Adapted from MCEER model on Lifeline

- Firstly, functions that are not completely lost are identified and the prevention effort is concentrated on maintaining the business continuity capabilities played by these functions. Taking the electricity supply as an example—In the Great East Japan Earthquake of 2011, Fukushima No. 1 nuclear power plant had lost its function due to tsunamis, which greatly affected social activities, showing how basic function maintenance is important.
- Secondly, continued investment is directed for items conducive to enhancing prevention capabilities. In other words, it is important to clearly show those items for which function loss occurs in the event of a disaster. This concept can easily be understood in the following example: While investment in the maintenance of an electricity transmission network is preferably continued, the electricity supply network is identified as an item whose function may be lost in the event of a disaster, provided that certain operations should be preferably taken for its early recovery.
- Thirdly, available restoration resources are concentrated on those items that should be given top priority for restoration. In this regard, there are many cases in which restoration resources are preferentially allocated to schools and hospitals that have a highly public nature, as well as the industrial infrastructure that supports employment in the local area.
- Lastly, restoration is to be achieved as early as possible in a broad way.

How to improve disaster resilience is none other than how to hurdle disasters with the main target of minimizing loss of function of the system as a whole by means of the comprehensive design of all operations involved in disaster prevention.

Three Capabilities Composing Disaster Resilience

In order to improve disaster resilience, it is imperative to improve prediction capabilities, prevention capabilities and response capabilities in a well-balanced manner (refer to Fig. 8). The starting point is to improve prediction capabilities, and this is the issue involved in risk assessment. We are surrounded by diverse kinds of hazards, but we do not have the resources to manage all of these hazards. Accordingly, we are to manage in turns respective serious hazards in which the product of the hazard probability by the scale of the effect in the event of a hazard is large.

As regards serious risks, in order to suppress the occurrence of damage, we are to make efforts to improve prevention capabilities. Hand washing, gargling and mask wearing are effective as a countermeasure against COVID-19, but these countermeasures cannot be applied as a prevention measure for earthquake disasters. As can be seen from this, comprehensive prevention measures that are effective against any kinds of hazards are not available. Prevention measures are prepared for each respective hazard, and high professional expertise is required to be provided to manage each hazard. In this way, prevention capabilities are divided vertically into divisions and thus it has become difficult to obtain mutual understanding between hazards.

As far as disaster prevention capabilities are concerned, the prevention capabilities of Japan, especially as applied to various types of structures, are accepted as world class, but their further improvement will require a vast amount of funding and time. Given this situation, it seems difficult for prevention capabilities to achieve outstanding improvement until national crisis-level disasters occur around 2035.

So, it is the improvement of response capabilities that offer high expectations in dealing the above issue. The main aim of response capabilities is to recover from damage on the condition that damages occur in the event of a disaster, and its noteworthy feature lies in that response means does not change even if any damage caused by any kind of hazard would occur. The social response to COVID-19 is basically similar to that to the damage caused by earthquakes, strong winds and floods. Therefore, the improvement of response capabilities for frequent climate-induced disasters leads to the improvement of response capabilities for earthquake-induced disasters.

How to Improve Response Capabilities for Natural Disasters

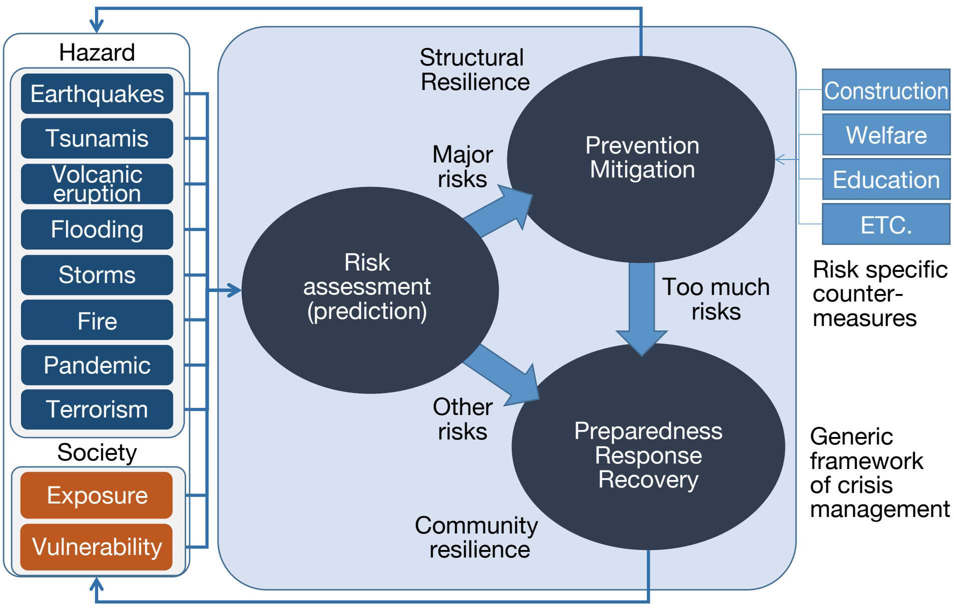
Response and prevention capabilities for disasters are improved by hazards. However, in order to improve response capabilities, it is necessary to know the consequences. For the improvement of response capabilities, the inconvenient effects brought about by disaster are arranged one by one, and then a response means is worked out to dispel each inconvenient effect.

In order to put into effect an effective disaster response means, due consideration is to be paid to the following five tasks (refer to Fig. 9):

The first task is the establishment of common operation picture for the situation awareness (refer to Fig.10). Disasters produce a new reality or situation following rapid environmental changes. It is necessary for those concerned to rapidly and correctly recognize the new situation. This is an urgent task in terms of time.

Second is the implementation of disaster response measures. There are three

Fig. 8 Three Components of Disaster Resilience



kinds of tasks in which the peak periods after the occurrence of a disaster differ from each other. The priority task during the first period of 100 hours just after the occurrence of the disaster is to protect human lives. The second task to be mainly promoted during the subsequent 1,000 hours is to restore the social flow. Normally, society works with the support of the flow of people, goods, money, and information. It is the disaster that suspends this normal flow, and therefore the social flow must be rapidly restored to a normal state within this duration, and at the same time operations to temporarily support lives until this is restored also enter into the peak period. After the passing of 1,000 hours, the third task is caried out mainly targeting the restoration of collapsed towns and the reconstruction of the daily lives of victims. While the timing in which these three tasks enter into their peak stages differs from each other, due attention should be paid to the following: in cases when these three tasks are not implemented in parallel, there arises fears that the response has been missed.

In order to effectively implement these four tasks, the fifth task must be implemented—that of planning the restoration measures and promoting logistics support throughout the disaster restoration period (refer to Fig.11).

Fig. 9 Five Tasks for Effective Disaster Response

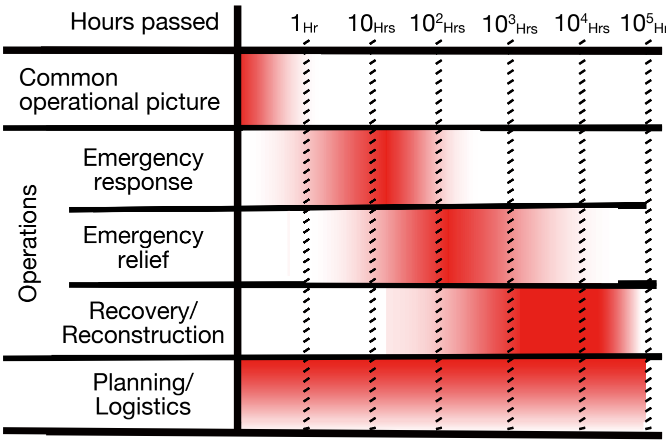


Fig. 10 Common Operational Picture Using GIS

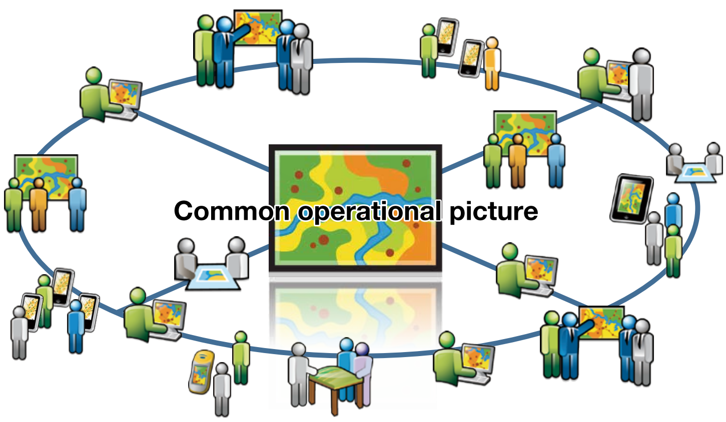


Fig. 11 SIP4D (Shared Information Platform for Disaster Management)

Information “pipeline” to connect between the disaster site and research has maximized the impacts

2020 AJSI Webinar: Energy-Efficient and Environmental Transition towards Sustainable Steel Industry

The Japan Iron and Steel Federation (JISF) concluded a memorandum with the ASEAN Iron and Steel Council in fostering interaction with regard to environment, standardization and trade in May 2014. Since then, in the field of the environment, with the cooperation of the Ministry of Economy, Trade and Industry, JISF has started up a public-private collaborative platform called the ASEAN-Japan Steel Initiative (AJSI), through which JISF has extended support for energy efficiency and environmental protection in the ASEAN steel industry.

On December 14, 2020, as a link of AJSI, JISF held the online seminar "2020 AJSI Webinar: Energy-Efficient and Environmental Transition towards Sustainable Steel Industry" targeting Indonesia, Singapore, Thailand, the Philippines, Vietnam, Malaysia and Myanmar, to which a total of around 200 persons from the government, steelmakers and others in those nations participated.

For the ASEAN steel industry which has been impacted by the COVID-19 as with other nations, the webinar covered examples of short-term energy efficient and environmental protection measures that are currently in high need and simple in practical use. In addition, the webinar provided information about medium- to long-term energy efficient and environment initiatives and trends. In particular, because there are many steelmakers in ASEAN that operate electric furnaces and are engaged solely in rolling, examples of small-scale technologies and low-cost operation improvements were positively taken up, which acquired extremely high assessment from the participants and the South East Asia Iron and Steel Institute as well.

JISF, jointly with the Japanese government, is determined to continue to supply support for energy efficiency and environmental protection to the ASEAN steel industry through the activities of AJSI in the future.

Technical Report of Long-term Exposure Tests for Construction Materials

The Committee on Overseas Market Promotion of JISF has recently published an English-version technical report titled "Durability Assessment of Various Kinds of Construction Materials by Means of Long-term Exposure Tests."

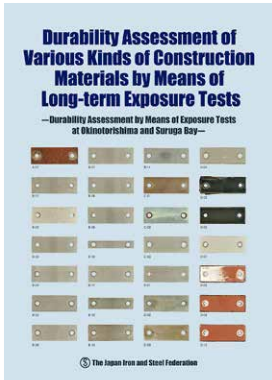
The Research Group on Corrosion Protection and Durability of Offshore Steel Structures of JISF, jointly with the Public Works Research Institute, a public research organ, had conducted long-term exposure tests for 28 types of construction materials since 1982 at two different testing sites at Okinotorishima (19 years of exposure) and the Marine Engineering Research Facility in Suruga Bay (24 years of exposure) to assess the durability of these construction materials.

Okinotorishima is located at the southernmost tip of Japan in a tropical zone where temperatures and humidity are high, and radiation is also high. Further the tidal current and wave height are also high and the island is constantly subjected to seawater splashing. Thus, the corrosion environment at Okinotorishima is far stricter than that in the peripheral sea areas of Japan.

Meanwhile, in order to assess the long-term durability in the peripheral sea areas of Japan, the offshore atmospheric exposure tests were conducted employing identical construction materials in Suruga Bay located on the main island of Japan in parallel with testing at Okinotorishima.

The report makes a comparison of exposure test results between these two different corrosion environments and provides useful data pertaining to the durability of various types of steel and other metallic materials and coated/sprayed/lined/painted corrosion-resistant structural members to be applied in tropical and other severe offshore environments.

The full text of the report and its attachments (reference photos) are published on the JISF website: https://www.jisf.or.jp/en/activity/sc-reports/index.html



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