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Civil Engineering

Port and Harbor Facilities

Building Constraction

Concrete-filled Steel Tube Columns

Revision of Technical Standards for Port and Harbour Facilities in Japan

by Dr. Hiroshi Yokota
Port and Airport Research Institute

Background

The Technical Standards for Port and Harbour Facilities has been used as the national standard based on Article 56-2 of Port Act in Japan when a harbor facility is newly planned, constructed, maintained, rehabilitated, or upgraded. The origin of the Standards can be dated back to 1950 when the official design manual for harbor facilities was firstly published. Before the publication, design and execution of harbor facilities had been undertaken based on knowledge and experience of individual engineers. In 1979, the Standards was systematized under the Port Act and subjected to two major revisions in 1989 and 1999, respectively. It has been playing a very essential role in assuring the reliability and quality of harbor facilities. The third major revision work is now under way to come into force in April 2007, which is mentioned in this article.

Please note that some parts in this paper are subject to change before publication.

The current version of the Standards is constituted based on the prescriptive specification format in which standardized methods of design calculations are specified. On the other hand, the design method is now shifted from the prescriptive specification format to the performance-based format. This trend was triggered by the publication of ISO 2394 "General principles on reliability for structures" in 1998. In line with the trend, national design standards and specifications covering civil infrastructure should principally be revised into performance-based statements. Under these circumstances, the Japanese Ministry of Land, Infrastructure and Transport summarized a short report entitled "Basis of Structural Design for Buildings and Public Works" in October 2002. The revision of the Standards has been undertaken to fulfill the basis given in the report.

Points of Revision

The key points of the revision are summarized as follows:

1) Performance-based design concept is introduced.
2) Reliability-based concept is applied as a standard basis for verifying compliance to performance requirements.
3) Compatibility certification system is established.
4) Maintenance and rehabilitation are linked with initial design.
5) Seismic ground motion and seismic performance verification are updated.
6) Beach, cargo handling equipment, etc. are newly covered.

Hierarchy of Performance Verification

Within the performance-based design concept, the Standards only specifies the performance requirements to facilities to be designed but the methods of verification are not specifically provided. The concepts of performance for harbor facilities are classified into (1) objectives, (2) performance requirements, (3) performance criteria, and (4) verification, of which hierarchy is shown in Fig. 1. Performance requirements and performance criteria are specified in the ministerial ordinance and announcements, while the methods of verification are exemplified in references for users' convenience.

Depending on the requirements, safety,

Fig. 1 Hierarchy of the Concept of Performances

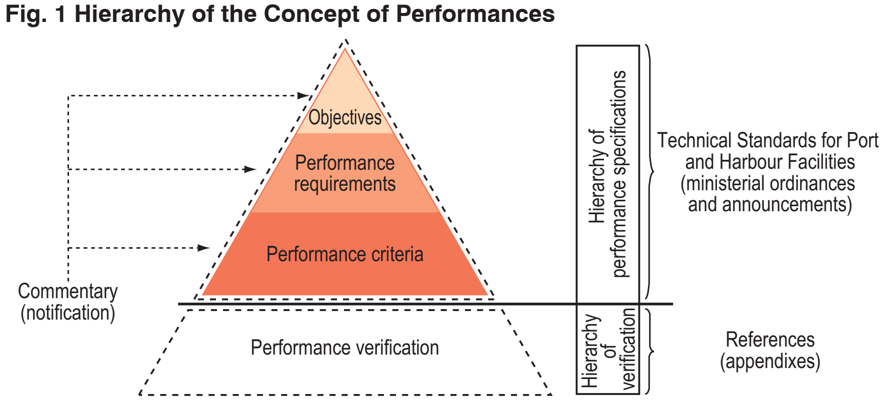
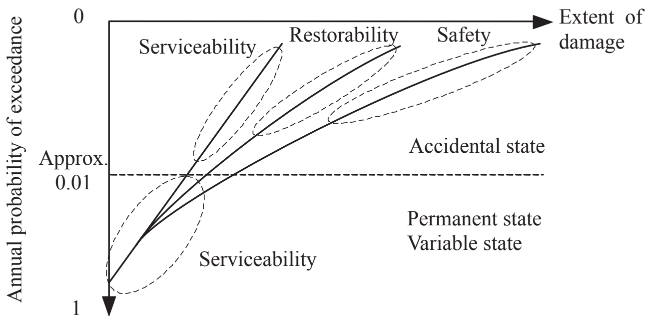


Table 1 Design Working Life

Design working life	Examples
1-5 years	Temporary structure
25 years	Replaceable structural member
50 years	General structure
100 years or longer	Critical structure, important structure

Fig. 2 Performance Requirements under Specific Actions



serviceability, or restorability should be ensured during the design working life. The general design working life is firstly specified clearly as presented in Table 1. Safety and serviceability are relating to human life in and around the structure and the functions of the structure, respectively. Restorability is the performance that, if required, continued use of the facility is feasible against foreseeable actions by restoration using technologies available within reasonable ranges of cost and time. Other requirements such as landscape, impact on environment, economic efficiency, etc. are not directly selected as the targets of verification because they are not so easy to verify with quantitative indices. Durability is taken into account as the change in performance with time.

The generally recommended methods for performance verification are listed in Table 2. It is preferable to quantitatively evaluate the structural performance using probabilistic approach rather than deterministic approach. Therefore, the concept of reliability design is applied as a standard basis for verifying compliance to performance requirements. The reliability of structures subjected to known or foreseeable types of actions is considered in relation to the performance of the structure throughout its design working life. Based on the level-1 reliability design, partial safety factors corresponding to the performance requirements are quantified depending on the failure probability of structural systems as shown in Fig. 2. Sets of partial safety factors for the verification of open-type wharf are listed in Table 3 for examples.

Compatibility Certification

Performance-based design concept will become a basic rule for design work so that engineers can obtain greatly expanded flexibility during design, execution, and maintenance. To approve the correctness of adopted verification methods, the framework for the certification of their compatibility with those recommended in the Standards should become very important. Therefore, as shown in Fig. 3, for facilities having significant roles to public safety and interests, the designed outputs shall be certified by a registered certification body if the design method was not approved as a standard method.

Maintenance and Rehabilitation

The life of harbor facility is rather long and designed today must be expected to meet demands during its design working life that cannot be foreseen. Strategic maintenance is the only way to be taken after commencement of service for avoiding heavy deterioration and loss of structural performance as a consequence. For this purpose, a comprehensive maintenance work is eagerly required to be implemented to ensure structural performance over their required levels during their design working life.

To ensure the reliability during the long working life of the facility, the levels of initial design and maintenance should be linked well as shown in Fig. 4. The maintenance strategies are categorized as maintenance strategy A: high durability to avoid any loss of structural performance below the required level, maintenance strategy B: frequent maintenance and prevention work are expected to keep structural performance over threshold levels (preventive maintenance), and maintenance strategy

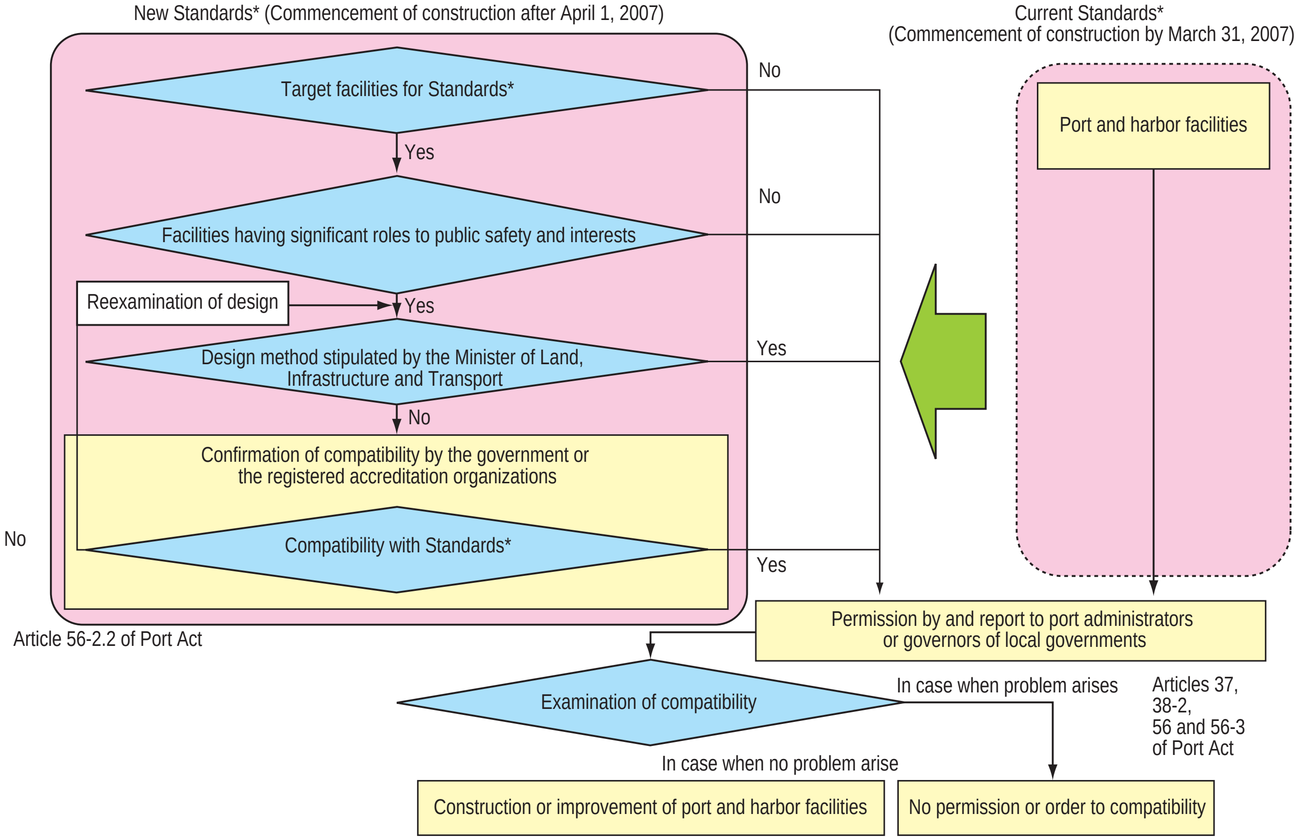
Table 2 Recommended Methods of Performance Verification

State	Principal action	Verification method
Permanent Variable	Self-weight, earth pressure, hydrostatic pressure, load, wave, wind, etc.	Reliability design (partial safety factor etc.)
		Model experiment, field test, etc.
	L1 ground motion	Reliability design (partial safety factor etc.)
		Numerical simulation (nonlinear analysis taking into account ground-structure interaction
		Model experiment, field test, etc.
Accidental	L2 ground motion, tsunami, collision, accidental wave, fire, etc.	Numerical simulation (expressing displacement, local failure, etc.)
		Model experiment, field test, etc.

Table 3 Partial Safety Factors for Verification Subjected to L1 Ground Motion

	HR	IR	NR
Target reliability index	3.65	2.67	2.19
Target probability of failure	1.3×10^{-4}	3.8×10^{-3}	1.4×10^{-2}
Partial safety factor, γ			
Yield strength	1.00	1.00	1.00
Ground reaction coefficient	0.66	0.72	0.80
Seismic coefficient	1.68	1.36	1.23
External load	1.00	1.00	1.00
Structural analysis factor	1.00	1.00	1.00

Fig. 3 Flow of Compatibility Certification



* Technical Standards for Port and Harbour Facilities

Fig. 4 Linkage between Design and Maintenance Standards

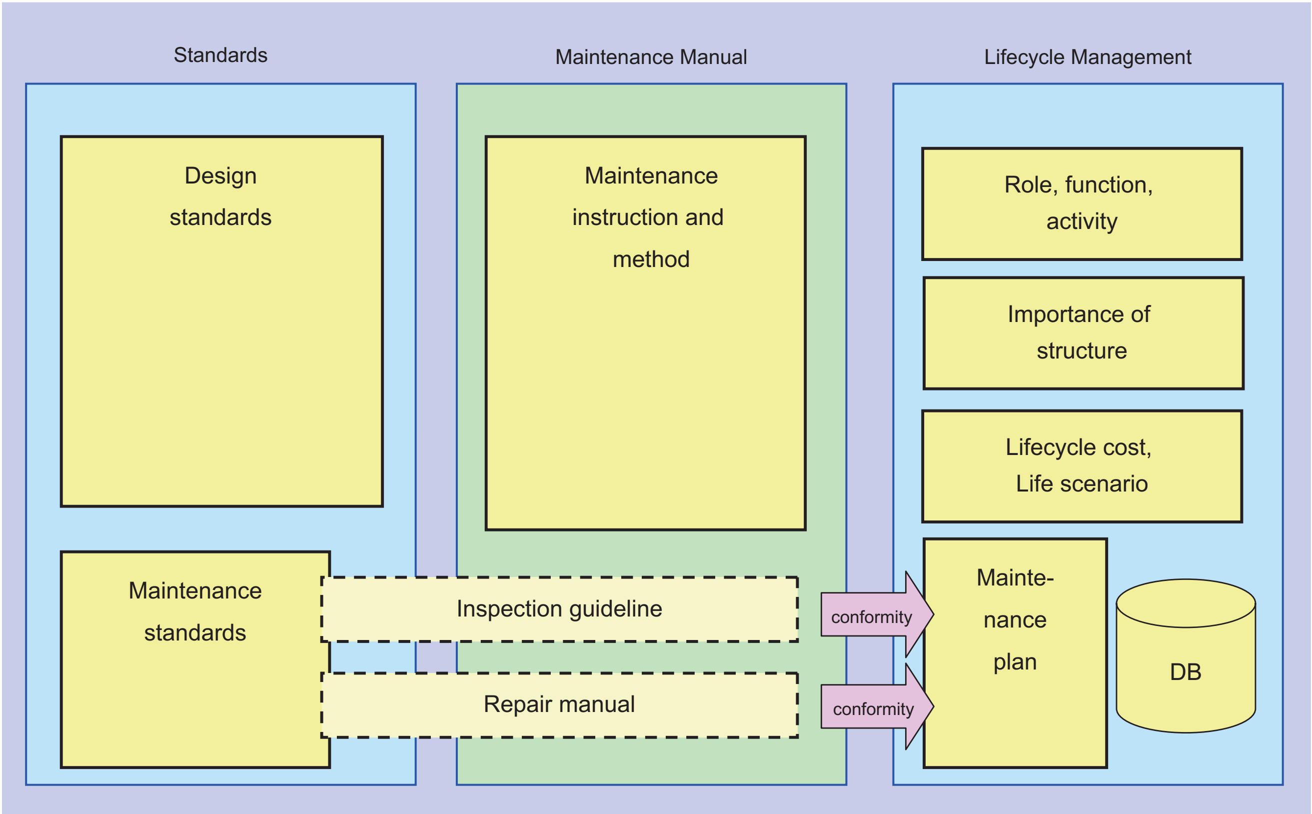
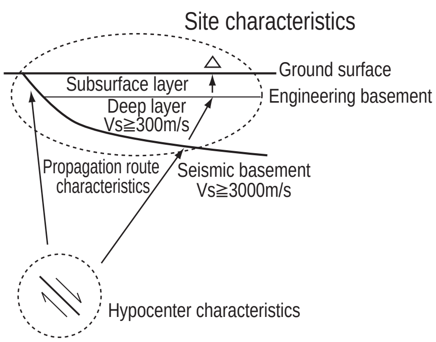


Fig. 5 Seismic Actions



C: a couple of major rehabilitation work is expected to recover structural performance to earlier levels (corrective maintenance).

According to the selected maintenance strategy, the verification for the loss in structural performance over time should be carried out. For example, when a facility made of steel is categorized in maintenance strategy A, the facility should be highly protected against corrosion by covering its surfaces with sufficiently durable materials such as titan, super-stainless steel, etc. During the service life, the facility should be thoroughly and regularly inspected. In case of maintenance strategy B, the corrosion protection systems are installed with medium durable materials and they will be replaced regularly based on the predicted timing.

Seismic Performance Verification

Seismic performance of facility has been verified by the seismic coefficient method based on the regional seismic coefficient. However, seismic actions are influenced by seismic source property, propagation path property, and site property as shown in Fig. 5. Furthermore, seismic response of facility is governed by not only seismic wave but also frequency characteristics. Therefore, the seismic action for the seismic performance verification should be determined taking into account these characteristics of seismic waves and structure-seismic interactions.

The Standards, therefore, abolishes regional seismic coefficients but introduces firm ground motions as time histories of acceleration. Two levels of earthquake ground motions, Level 1 and Level 2, are specified depending on the likelihood of earthquakes. Level 1 ground motion (L1) is likely to occur once or twice during the design working life (normally a return period of 75 years). Level 2 (L2) is the maximum credible earthquake ground motion, which is an infrequent rare event, but when this happens, it is excessively intense. In the Standards, L1 is considered as a variable action, while L2 is taken into account as an accidental action.

This two level approach attempts basically to ensure a specified level of serviceability for L1 and prescribe the extent of seismic damage for L2. For ship mooring facilities, the following four classes are defined depending on the performance requirements during and after earthquake.

- High seismic resistant facility (HR)
  For transport of emergency materials after earthquakes (HR-E)
  For international sea container terminals (HR-C)
- Intermediate seismic resistant facility (IR)
- Normal seismic resistant facility (NR)
  Facilities other than high seismic resistant facilities

The performance requirements matrices of these four class-categorized facilities are presented in Table 4. Serviceability is basically required against L1 ground motion in all facilities so that they may be continuously used without any rehabilitation work. For example, in case of an open-type wharf, serviceability should be ensured if design axial force of steel pile does not exceed the resistance of ground failure, stress of steel pile does not exceed the stress limit, and resulting forces in concrete deck do not exceed the respective capacities. Against L2 ground motion, the performance requirements vary with the class of facility. A facility for transport of emergency materials (HR-E), serviceability is also required in order to accommodate vessels for transporting relief supplies smoothly. For international sea container terminals (HR-C), restorability is required against L2 ground motion because the facility may be resumed to use within a few weeks after light restoration. An intermediate seismic resistant facility requires safety against

Table 4 Seismic Performance Matrix

Category	L1	L2
HR-E	Serviceability	Serviceability
HR-C	Serviceability	Restorability
IR	Serviceability	Safety
NR	Serviceability	Not recommended

L2 ground motion. Safety can be ensured unless two hinges are formed in one pile simultaneously. The overall level of acceptable damage is quantitatively defined (damage criteria) using indices such as values of displacements, stresses, or ductility factors. The residual horizontal displacement is recommended as a certain value taking into account the individual requirements.

Future Prospects

As every engineer understands, performance-based design is very rational and is a promising approach for constructing civil infrastructure. The revised version of the Standards seems to bring drastic changes to users. The most essential point of this approach lies in its verification technology. Highly reliable and widely applicable analysis methods are now being developed. Furthermore, some partial safety factors were preliminarily determined based on the limited numbers of variation in design parameters or code calibration. The values of these factors may be changed to more reasonable ones with acquiring experiences and studies. We should keep making efforts to determine the design related properties with high accuracy, such as forces, actions, strengths, and so on for future revision.

References

1) ISO: ISO 2394 General principles on reliability for structures, 1998
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3) International Navigation Association: Seismic Design Guideline for Port Structures, Balkema, 2001
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Contribution of Japanese ODA to Port Development in Asian Countries

by Akira Moriki
Director, Research Division II
The Overseas Coastal Area Development Institute of Japan (OCDI)

Preface

In order to support the industrial and economic development and self-sustainability of developing nations, the Japanese government provides these countries with official development assistance (ODA). The following outlines the use of Japanese ODA to support the development of port facilities in Asia and, at the same time, introduces typical examples of port development projects using steel structures.

Yen Loans for Asian Countries in Port Sector

Japan's use of ODA to improve the social infrastructure of developing nations takes the form of "loan aid" (yen loans). Conventionally, the nations targeted for financing through ODA have consisted of the developing nations of Asia that have close economic and political relations with Japan.

The support of port development with yen loans started in Indonesia and Malaysia and then gradually grew to include, mainly, other ASEAN nations. Entering the 1980s, support for China began and expanded to include Thailand, the Philippines, and Sri Lanka; total financing in the 1980s surpassed ¥300 billion. In the 1990s, the support for China increased and support for Vietnam started. Total financing in the 1990s amounted to about ¥270 billion, a slightly lower amount than in the 1980s. After 2000, support has been extended mainly to Vietnam and Cambodia.

Of the total financing provided, by nation, from 1950 to 2004, China accounted for 36%, while Indonesia, the Philippines, Vietnam, and Sri Lanka tallied about 13% each (Table 1).

Table 1 Accumulated Yen Loan Financing for Port Development by Nation (1950~2004)

(Nominal price base)

Nation	Accumulated amount (US$ million)	Share (%)
China	1,815	36.2
Korea	42	0.8
India	77	1.5
Pakistan	34	0.7
Philippines	626	12.5
Vietnam	659	13.1
Malaysia	42	0.8
Indonesia	686	13.6
Thailand	287	5.7
Sri Lanka	665	13.2
Other nations	84	1.7
Total	5,017	100.0

Use of Steel Structures in Port Facilities in ODA Projects

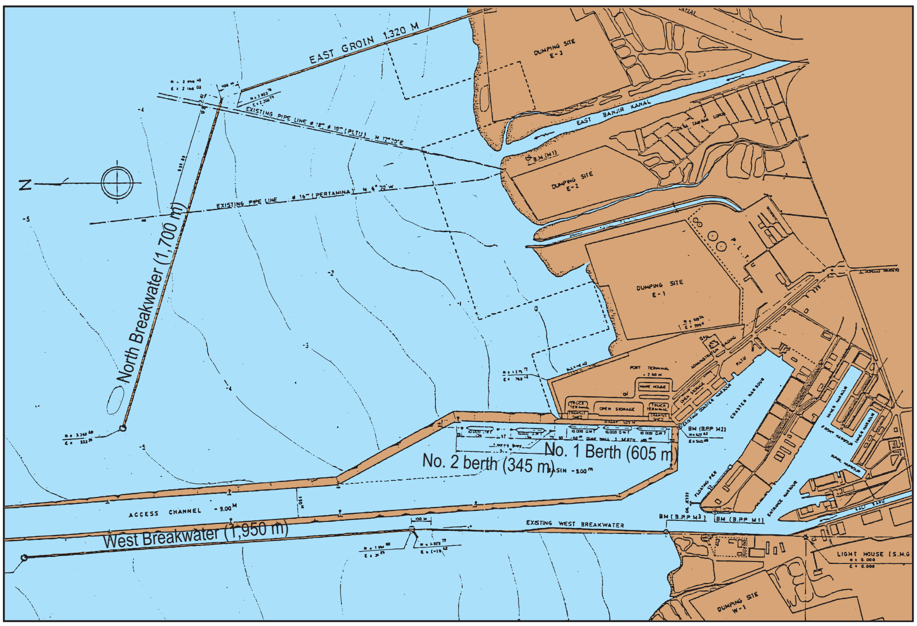
Because of the unavailability of publicly released data that clearly describe the structural types of port facilities developed with ODA support, I asked about the structures of some entities engaged in ODA projects. According to the results thus far obtained, ODA projects that include the use of steel structures for piers and other major facilities are relatively few in number. This is attributed to the fact that the construction cost of steel structures (piers, etc.) is generally higher than that of gravity-type structures (caissons, etc.). Accordingly, steel structures are applied only in cases where gravity-type structures are not technically feasible.

Meanwhile, another hurdle is encountered when the use of Japanese steel products is required for an ODA project. According to the ODA Guidelines agreed to with the OECD (the main objective of the guidelines is to restrain states from providing ODAs aimed at supporting enterprises of the donor nation), while it is allowed to provide grant aid on "tied" condition (that obliges the recipient to procure needed materials from the donor nation), loan aid should, in principle, be provided as an "untied" condition. In the case of untied condition, the materials used in ODA projects are to be competitively procured through international bidding. This has made it difficult for Japanese makers to successfully compete in bidding with local makers and to find markets for those materials.

However, there are cases where loans can be provided on a "tied" condition when agreed to by the recipient nation under certain restrictions. This is the case, for example, when a loan with specially relaxed terms (i.e. interest rate, deferment period, repayment period, etc.) obliges the recipient nation to purchase necessary materials from the donor nation while also allowing the recipient nation the freedom to choose between this loan and other "untied" loans on general terms. Among examples of similar "tied" loans are "special yen loans" that were implemented as a countermeasure against the currency crisis in Asia.

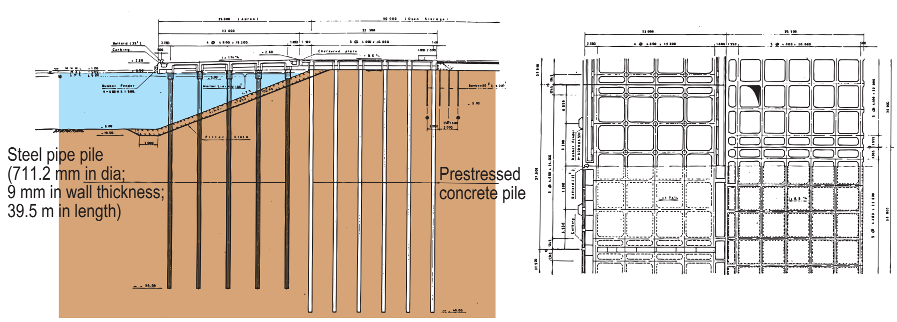
In addition, the Special Terms for Economic Partnership (STEP) scheme was introduced in July 2002. This approach is associated with "tied" yen loans that aim to promote "assistance that shows the face of the donor nation, Japan," through technology transfers to developing nations that make use of Japan's excellent technology and know-how. This approach is applicable, for example, when local technical conditions call for the use of specialty steel products that only Japan can produce. As a means of promoting the use of Japanese steel products in the future, STEP-based ODA projects are regarded as having great potential.

Fig. 1 General Layout of Semarang Port



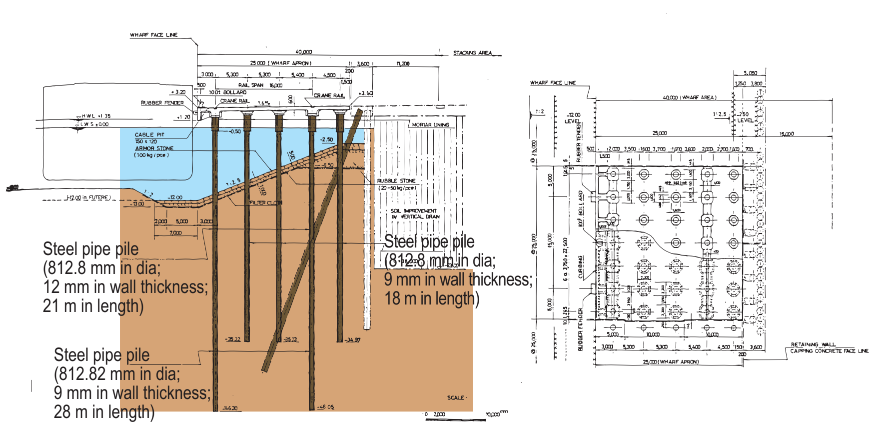
Source: Urgent Reinforcement of Semarang Port, Design Report 1985

Fig. 2 Section and Plan of No. 1 Berth at Semarang Port



Source: Urgent Reinforcement of Semarang Port, Design Report 1985

Fig. 3 Section and Plan of No. 2 Berth at Semarang Port



Source: Urgent Reinforcement of Semarang Port, Design Report 1985

Steel Structures Used in the Port Facilities of Asian ODA Projects

Introduced below are three examples of steel structures used in Asian port facilities and funded as ODA projects.

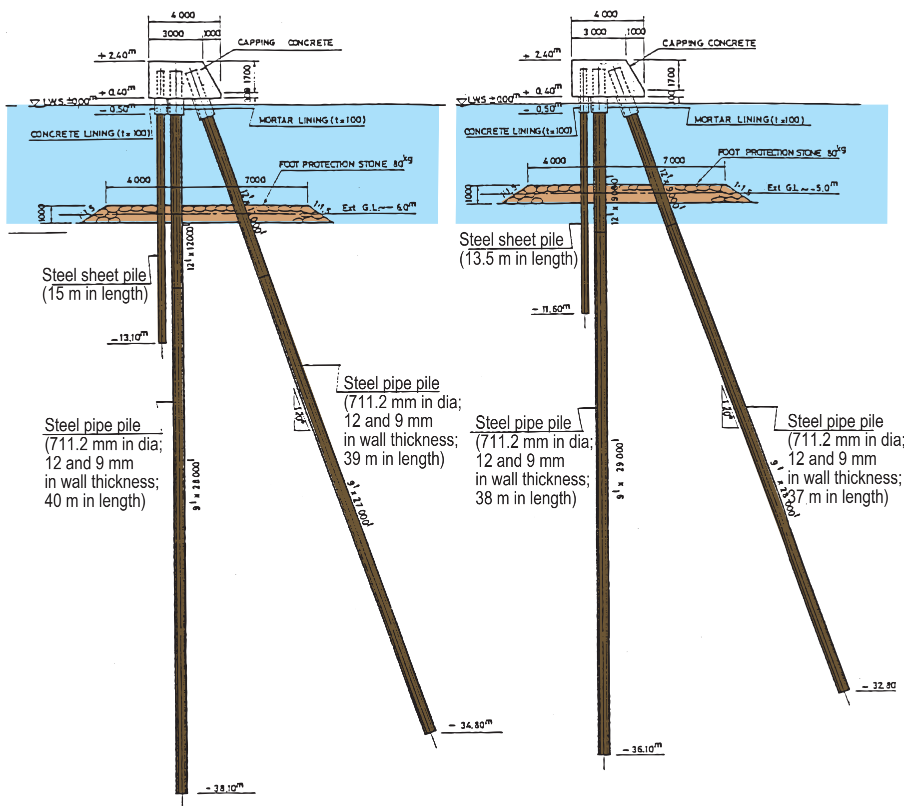
● Development Project for Semarang Port in Indonesia (First Phase)

An ODA contract was concluded in 1981 with the following conditions: yen loan contract, total amount of ¥17.3 billion, 2.5% annual interest rate, 29-year repayment period (deferred for 10 years), and partial untied procurement. Then, in 1991, an additional yen loan contract was concluded for the second phase of the project.

At Semarang Port located on the north coast of Java, Indonesia, the following facilities are being built to facilitate the mooring of 10,000 DWT-class vessels and to meet the needs of container handling: a quay with 2 berths (No. 1 berth with a water depth of 12 m and a length of 605 m for handling general cargoes; No. 2 berth with a water depth of 12 m and a length of 345 m for container handling); 2 breakwaters (north breakwater with a length of 1,700 m and west breakwater with a length of 1,950 m); and anchorage for related sea routes (temporary depth of 9 m).

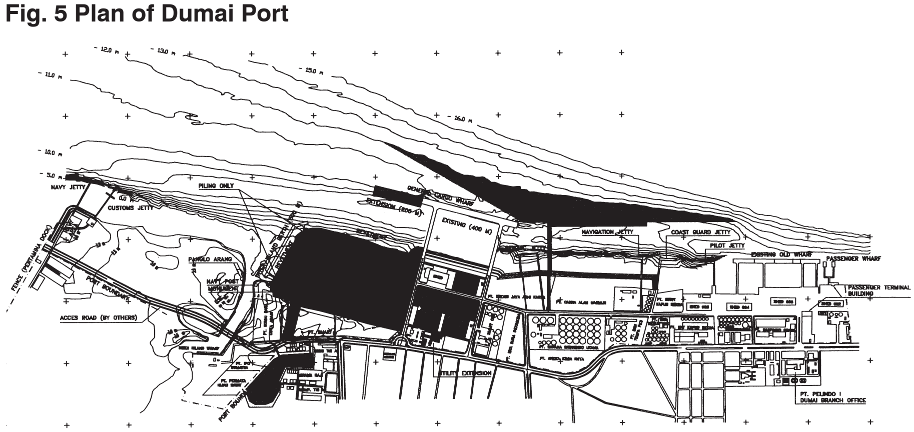
At the breakwater, a structure was adopted that uses steel pipe piles (combined

Fig. 4 Section of Breakwater at Semarang Port



Source: Urgent Reinforcement of Semarang Port, Design Report 1985

Fig. 5 Plan of Dumai Port



Source: Dumai Port Development Project, 2002

use of vertical and oblique piles) to support a steel sheet pile continuous wall; and, at the quay, a pier employing steel pipe piles was adopted. Because the loan was partially tied, it is considered feasible that Japanese steel products have been procured. However, because the project was begun more than 20 years ago, the details of such procurements are unknown. (Refer to Figs. 1~4)

● Development Project for Dumai Port in Indonesia (Second Phase)

An ODA contract was concluded in 1998 with the following conditions: yen loan contract, total amount of ¥3.8 billion, 2.7% annual interest rate, 30-year repayment period (deferred for 10 years), and general untied procurement. In 1989, a yen loan contract was concluded for the first-phase project; the current contract is for the second-phase project.

Dumai Port, located 130 km north of Pekanbaru, the metropolitan center of Riau state, Sumatra, is the largest cargo handling port in the state and is positioned as one of Indonesia's major ports. During the five years since the contract was concluded, cargo-handling volume at the port increased at an annual rate of 32.2% for general cargo and 64.2% for palm oil. Growth of the latter was due to increasing shipments of palm oil produced in the states of Riau and North Sumatra and to other factors. However, because improvements to port facilities have not kept pace with the increasing demand, loading and unloading operations have grown inefficient.

In the current project, the wharf for general cargo will be expanded from 400 m to 600 m and two berths for loading/unloading palm oil will be added. The resulting improvements and expansion will stimulate the local economy by making it possible to meet the need for greater cargo handling, allowing efficient port operations, and promoting the streamlined distribution of goods.

The piers adopted for both the general cargo wharf and the palm oil berth will be supported by steel pipe piles (combined use of vertical and oblique piles). Because of the untied procurement, it is supposed that steel materials have been obtained from local mills. (Refer to Figs. 5~7)

● Mindanao Container Terminal Project in the Philippines

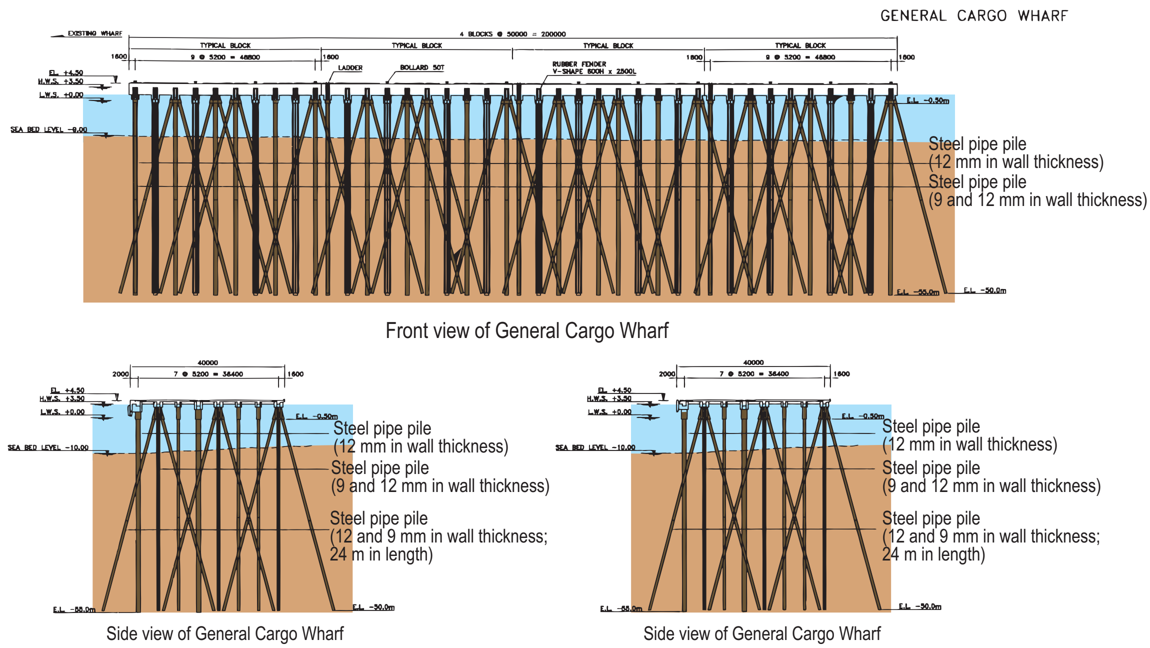
An ODA contract was concluded in 2000 with the following conditions: yen loan, total amount of ¥8.3 billion, special yen loan, 1.0% annual interest rate, 40-year repayment period (deferred for 10 years), and tied procurement.

The existing ports in Mindanao, such as Cagayan de Oro, lack container-handling facilities and pose a bottleneck to economic development there. Because of this, the current project will provide a container wharf equipped with gantry cranes and other advanced equipment adequate for handling current container volume as well as expected future growth. Further, a new base port in north Mindanao that adjoins Cagayan de Oro will be constructed in

Tagoloan. The project aims at improving the social infrastructure of northern Mindanao and developing the local economy.

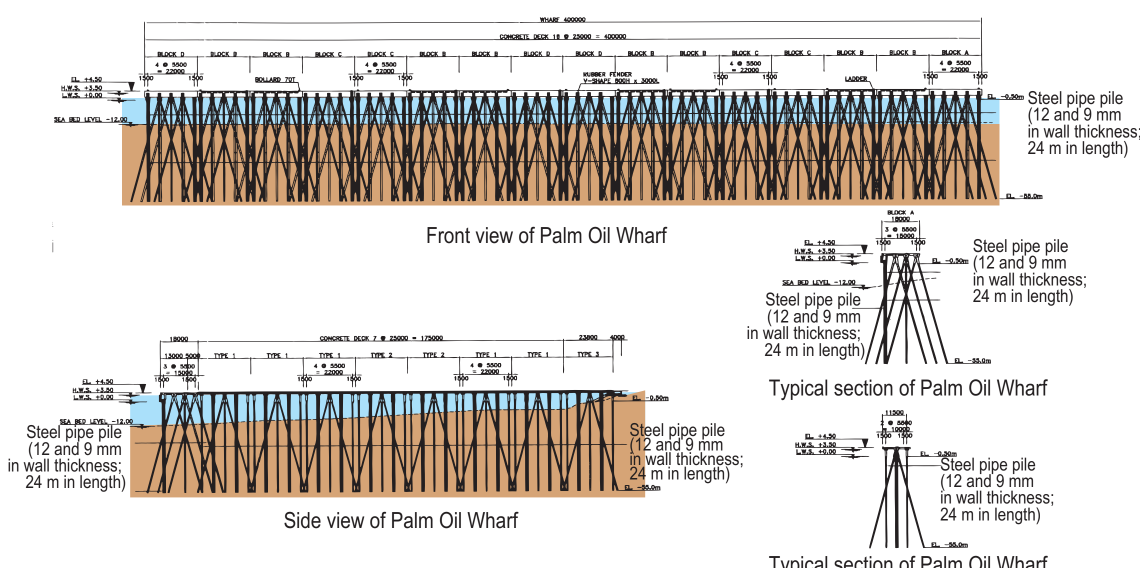
The project plan calls for the construction of a container wharf with a water depth of 12 m and a length of 300 m; the wharf will employ a pier constructed of steel pipe sheet piles. The structural plan was not available, but an aggregate table showing the steel products used was obtained and is attached below (Table 2). Because of the tied procurement, Japanese steel products have been used in the construction.

Fig. 6 Front and Side Views of General Cargo Wharf at Dumai Port



Source: Dumai Port Development Project, 2002

Fig. 7 Front and Side Views of Palm Oil Wharf and Trestle at Dumai Port



Source: Dumai Port Development Project, 2002

Acknowledgment

The yen loan projects were prepared by referring to publicly available documents of the Japan Bank for International Cooperation (JBIC), a financing organization; the project plan was obtained from publicly available documents of Kawasaki Steel Corporation (currently JFE Steel Corporation), one of the companies that received orders for the project. The structural plan and aggregate table for Semarang Port were provided by Japan Port Consultant (JPC) and those for Dumai Port and the Mindanao container wharf were obtained from Pacific Consultant International (PCI). Both of these companies are engaged in engineering services. I wish to thank all these organizations for their generous cooperation in the preparation of the text.

Table 2 Steel Pipe and Sheet Piles Used for Container Berthing Structures of Mindanao Container Terminal

(Unit: tons)

Steel piles	Weight
Pipe pile (1,200 mm in dia.; SKY400, L/T connector)	
Wall thickness: 6 mm	2,722.50
Wall thickness: 14 mm	1,820.87
Wall thickness: 12 mm	2,310.43
Pipe pile (1,000 mm in dia.; 14 mm in wall thickness; SKY400, L/T connector)	51.68
Pipe pile (700 mm in dia.; 12 mm in wall thickness; SKY400, L/T connector)	131.38
Pipe pile (600 mm in dia.; 14 mm in wall thickness; SKY400)	42.42
Sheet pile (U-shape; FSP-IV)	123.59
Total	7,202.87

Concrete-filled Steel Tube Columns

—Guidelines for Dynamic Seismic Design of Steel Frames Using CFT Columns—

by Dr. Shosuke Morino, Vice-President, Mie University, Dr. Akihiko Kawano, Professor, Kyushu University, and Dr. Jun Kawaguchi, Associate Professor, Mie University

Shosuke Morino: After graduating from the Kyoto University in 1965, he obtained Ph. D from the Lehigh University in 1970 and the Kyoto University in 1985. He became Vice-President, Mie University in 2004.

Akihiko Kawano: After graduating from the Kyushu University in 1974, he obtained Ph. D from the Kyushu University in 1989. He became Professor, Kyushu University in 2001.

Jun Kawaguchi: After graduating from the Graduate School of Mie University in 1990, he obtained Ph. D from the Mie University in 2000. He became Associate Professor, Mie University in 2004.

The Research Group on Seismic Design of Steel Frames Employing Concrete-filled Steel Tube (CFT) Columns (chaired by Dr. Shosuke Morino, Vice-President of Mie University) was established within the Japanese Society of Steel Construction with the aim of conducting a three-year study starting from 2002. The major aim of this group was to develop a seismic design method based on dynamic nonlinear response analysis applicable to steel-frame structures employing CFT columns, mainly for buildings with heights of 60 m or less.

During the course of its research, the Working Group selected input seismic waves, analytical approaches, and analytical models. It also prepared assessment criteria using target performances and responses and established standards for improved response systems. The successful result of this effort was the publication of *Guidelines for Dynamic Seismic Design of Steel Frames Using CFT Columns*, a manual for design that provides practical examples of calculations based on the above. An outline of *Guidelines* is introduced below.

Features and Major Contents of *Guidelines*

CFT steel-frame structures are regularly used in the construction of medium- and high-rise buildings (Photo 1). Most of the design work for these buildings employs nonlinear time-history response analysis, and based on response results, this analysis contributes toward the optimization of stiffness, strength, seismic resistance, and other seismic design factors. Further, this analytical approach is used to explain the seismic resistance of buildings to project owners and is made possible by the recent widespread availability of low-cost, high-performance computers.

However, the seismic design method generally used for buildings with heights of 60 m or less is based on ultimate horizontal strength and response limit strength calculation methods. These design methods follow a static methodology that does not directly examine information that depends on dynamic repetitive effects, such as the accumulated plastic deformation of each structural part and the deformation concentration on specified floors, even if such information is important to the design.

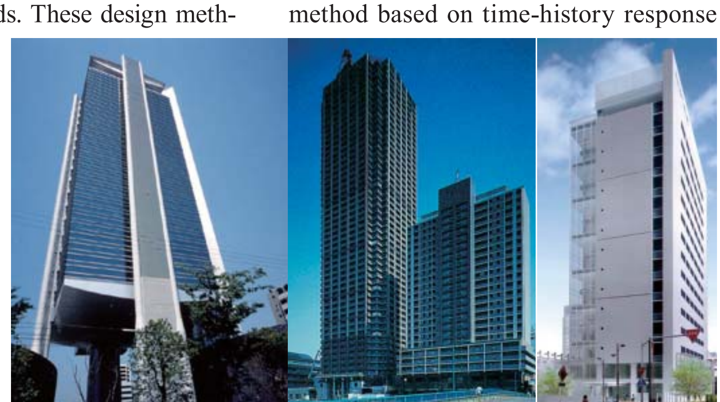


Photo 1 Examples of medium- and high-rise buildings constructed using CFT structures

It is to correct this situation that a design method based on time-history response analysis is used, even though certain problems remain unsolved pertaining to the selection and preparation of appropriate input seismic waves; the selection of restoring forces and the hysteretic characteristics of structural members and floors; and the technical level required for a response assessment. Accordingly, this design method has not been applied to general buildings.

To remedy the situation, extensive studies have already been made of these problems, and it is now thought that the design method based on time-history response analysis has reached the stage where it can be practically applied to CFT steel-frame structures, but only to those with comparatively moderate nonlinearity in their restoring force characteristics.

Major features of *Guidelines* are as follows:

1) The design method based on time-history response analysis used in *Guidelines* adopts the framework of performance-based seismic design; in addition, the method meets required performances while, at the same time, establishing rational structural specifications by appropriately assessing a structure's dynamic behaviors during earthquakes and by making multi-level inspections of building functions, repair, and safety.
2) The design method can establish a matrix of required performances agreed to by the project owner and the designer and can work out seismic resistance parameters that are numerically treatable.
3) A concrete value for seismic resistance is given according to a building's performance requirements and the performances retained by the structural members and frame; and a criterion is used that is compared to the response value.
4) Regarding the performances retained by CFT members, the critical range for functions, repair, and safety is defined based on experimental results.
5) In cases when the response value does not satisfy the required value or when the marginal degree is excessive, a concrete design procedure is proposed so that performance controls can be carried out in a way that will realize an optimized design.

Application Range of *Guidelines*

The application range of *Guidelines* extends from steel moment-resistant frames using CFT columns for buildings with heights of 60 m or less to frames with vibration-damping devices. However, excluded from this range are braces that rapidly lose strength due to bend buckling, etc. A plane model can be applied in the analysis, but in this case it is necessary to pay attention to reductions in column and beam strength due to the distortional deformation of buildings and bidirectional input.

Outline of Seismic Design Method Based on Dynamic Analysis and Performance Control

In Japan, in addition to the introduction of static simple design methods, such as performance-based design, the response limit strength calculation method and the energy method that assess the response displacement of structural systems are also being proposed. The current situation, however, shows that most practical designs still rely on conventional static allowable stress design. In Japan, in the design of high-rise buildings greater than 60 m, dynamic analysis is concurrently used to assess the quantitative level of damage to frames and seismic-resistant elements. The recent revision of the Building Standard Law authorizes the use of dynamic analysis even in the design of buildings lower than 60 m, resulting in a steady increase in the use of dynamic analysis in practical design.

However, dynamic analysis as it is currently being implemented has the strong characteristic of serving as a tool to check performance (for example, whether story drift angles are controlled to within 1/100 or less) and is not used for design work that facilitates the optimization of structural specifications by directly utilizing analytical results. Further, the dynamic analytical model that is applied is generally a simple one, and a multi-degree-of-freedom springs and masses model is frequently adopted whereby each story of the building is replaced with an equivalent shear spring.

The design method proposed in *Guidelines* is based on dynamic analysis that can more precisely assess the behavior of buildings during earthquakes, while at the same time allowing the practical use of rational seismic design of the performance assessment type.

A concrete seismic design flow is as shown in Fig. 1. The order of elements inherent in the current design method is outlined below, referring to Fig. 1.

● Premises for Design

The premises include the conditions given in the preceding stage of structural design, and indicate the information on the application of the building, the site plan and, further, the basic plan and elevation. The prerequisites necessary for a concrete structural plan are given at the initial design stage.

● Establishing Required Building Performances (Seismic Resistance Grade)

At this stage, the concrete performances required to start the performance design of a building are set up. Based on discussions between the project owner and the designer, the required performances for the target building are decided, and a seismic resistance grade is established. Table 1 shows an example of the matrix for required performances, which is used in setting up

Fig. 1 Flow of Seismic Design Based on Dynamic Analysis in *Guidelines*

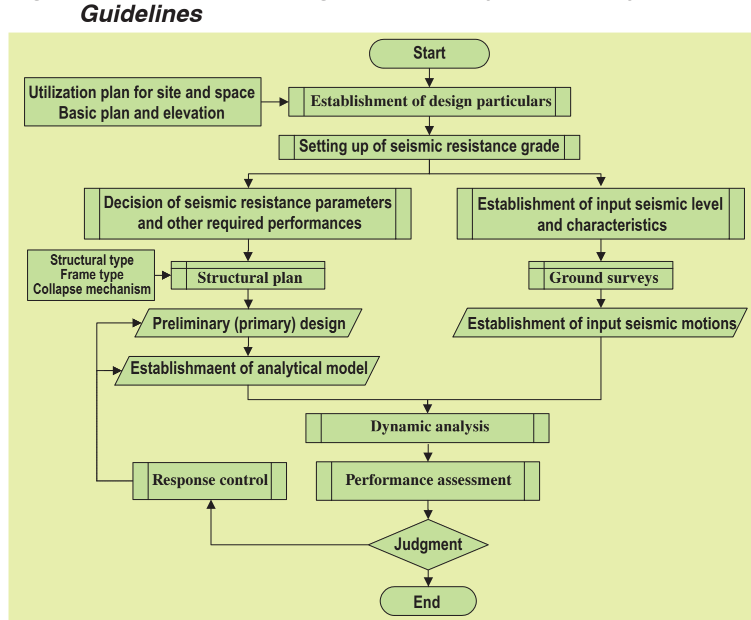


Table 1 Example of Matrix for Seismic Resistance Grades and Required Performances

Application of buildings	Seismic resistance grade socially required
Standard, general-use buildings (the level required in Building Standard Law)	Standard grade
Important facilities, facilities used by many and unspecified persons and facilities to store dangerous articles (fire station, hospital, high-rise building, computer network base, semiconductor plant, etc.)	High grade
Facilities for which function retention is expected during great disasters and higher safety is required (disaster prevention base, nuclear power plant, chemical plant dealing with high-level dangerous articles, etc.)	Special grade

Seismic resistance grade	Required performance to bear the seismic load with return period of about several tenth years		Required performance to bear the seismic load with return period of about several hundred years	
	Function, disaster level, repair	Engineering quantity that constitutes the criteria	Function, disaster level, repair	Required value *Req* as seismic resistance parameter
Standard grade	Function retaining No disaster Repair unnecessary	Maximum story drift angle ≦ 1/200 (Easing to 1/120 is allowable) No plastic deformation	Protection of human life, securement of limited functions Medium to serious damage Medium- to large-scale repair	Maximum story drift angle ≦ R_d Maximum deformation angle ≦ R_u
High grade			Securement of specified function Small damage Small-scale repair	Maximum story drift angle ≦ R_d/1.5 Maximum deformation angle ≦ R_u/1.5
Special grade			Securement of main functions Slight damage Slight repair	Maximum story drift angle ≦ R_d/2.0 Maximum deformation angle ≦ R_u/2.0

Note: Rd: Limit of story drift angle; Ru: Limit of member deformation

the seismic resistance grade. Based on the matrix, the seismic performance factors are determined from engineering quantity that can be numerically treatable, such as the maximum story drift angle, maximum deformation of members, and the accumulated plastic deformation, for which quantitatively required values are set up.

● Structural Plan

Here, decisions are made regarding structural type, the use or nonuse of energy-absorbing devices, and other issues. Concurrently, decisions based on the given plan and elevation are also made regarding the arrangement of columns and beams, as well as dampers and other seismic members. Also determined at this stage are the basic collapse mechanism, which is deeply related to the seismic resistance of a building, and the conditions required for guaranteeing the specified collapse mechanism (column to beam strength ratio, etc.). It is common to determine these conditions so that local collapse mechanism is prevented.

● Preliminary Design

In order to implement dynamic analysis, it is necessary to determine the assumed cross sections of the members, dampers, etc. in the preceding stage of dynamic analysis (see Fig. 2). This design procedure is called the preliminary design.

At this stage, it is necessary to determine the shape and dimensions of the members, the specifications for the connections, the characteristic and energy-absorbing performances of the dampers, etc. Fig. 3 shows the criteria for determining the appropriate strength of hysteretic dampers due to stiffness of the dampers. Here, it is desirable to put the the ratio of strength carried by the damper to the total retained horizontal strength β close to the optimized value β_{opt}, depending on the stiffness ratio k of the damper to the frame. In cases when the β on each story exceeds the appropriate range, it is recommended to redesign the hysteretic dampers.

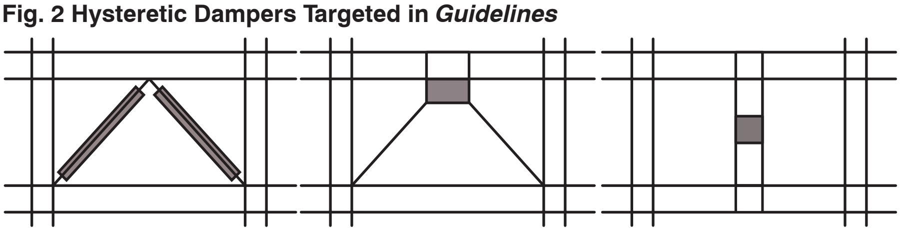
When dynamic analysis is implemented, the validity of decisions regarding the cross sections can be checked. And, by determining the assumed cross sections in advance by taking into account the response behavior during medium- and large-scale earthquakes, it is possible not only to make designs that are more rational but also to reduce design changes after dynamic analysis.

● Establishment of an Analytical Model

At this stage, the basic model required for dynamic analysis is established. Further, the frame model (fishbone model, frame model, etc., as shown in Fig. 4), models of the members, connections, and other structural elements, and the damping characteristics are established. Also selected is the software to be used for analysis. *Guidelines* provides outlines and descriptions of the advantages, disadvantages, and other features of each model and recommends that the springs and masses model not be used because of its imprecision in analyzing large deformation.

In *Guidelines*, the fishbone model or the frame model, shown in Fig. 4, is adopted.

Fig. 2 Hysteretic Dampers Targeted in *Guidelines*



(a) Buckling-restrained brace (b) Brace supporting shear panel (c) Stud shear panel

Fig. 3 Appropriate Strength Ratio of Hysteretic-type Dampers

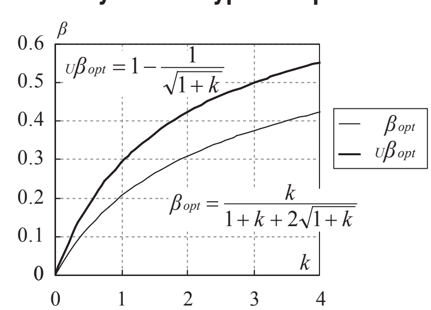


Fig. 4 Frame Models with Hysteretic Dampers (left: frame model; right: fishbone model)

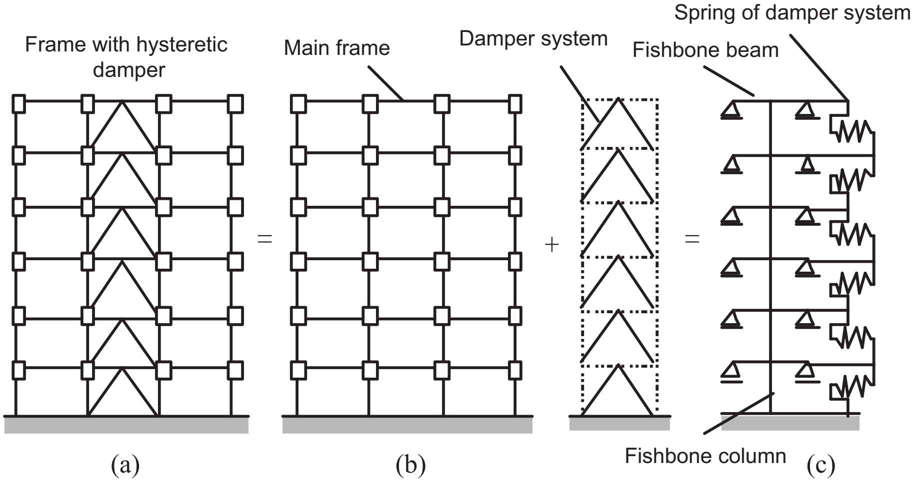


Fig. 5 Example of Yield Function of CFT Cross Section (400×15 square column)

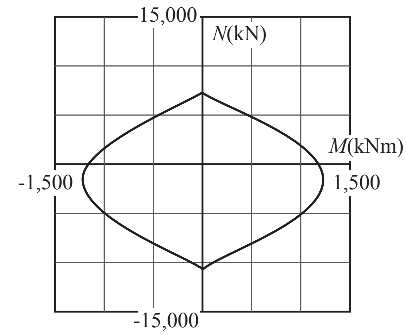
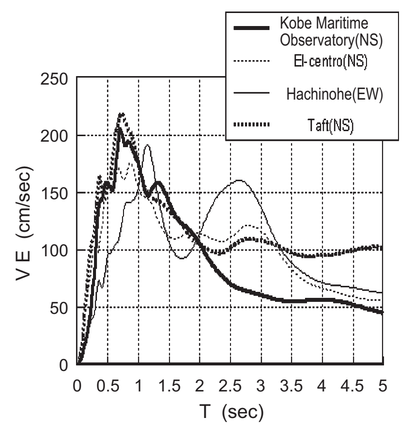


Fig. 6 Example of Energy Spectrum Employing Input Seismic Motions Obtained by Standardizing Maximum Velocity at 50 cm/sec



In preparing the analytical frame model, it is proposed that the frame be modeled using the generalized hardening plastic hinges analytical method, which allows for consideration of geometrical nonlinearity and material nonlinearity. In the model, the yield function shown in Fig. 5 and the plastic stiffness matrix of the CFT cross sections are newly derived.

● Survey of Ground and Faults, and the Establishment of Input Seismic Motion

In conducting design work, it is necessary to appropriately establish the nature and strength of the input seismic motion by taking into account information on a building's site conditions; that is, seismic hazard, faults, and ground conditions. *Guidelines* indicates the standard method for preparing the necessary number of input seismic waves, the input seismic motion level, the response spectrum, and simulated seismic waves, and a method for selecting the observed seismic waves.

On the other hand, in cases when seismic waves conforming to a building's site conditions cannot be set up, seismic waves set at 50 cm/sec or more are to be adopted. This value was obtained by standardizing seismic waves by means of their maximum velocity. Fig. 6 shows an example of an energy spectrum employing input seismic waves obtained by standardizing the maximum velocity at 50 cm/sec. Referring to these waves confirms whether or not the spectrum value in the vicinity of the primary natural period of a building is sufficient.

● Dynamic Analysis

Dynamic analysis is implemented and the response values needed to assess performance is extracted from the analytical results. In assessing the response values for multiple numbers of seismic waves, the stochastic model is adopted in order to rationally incorporate the indeterminacy of seismic responses. In cases when the response values are not stochastically treated, the maximum value of the analytical results obtained from multiple responses is adopted. Based on the above analysis, the level of damage to the frame, members, and other structural elements is to be assessed.

● Performance Assessment, Fulfillment of Required Performances, and Judgment of Fulfillment Level

Based on the response value *Res* of the seismic performance factors obtained from dynamic analysis, various required performances are calculated and a performance assessment is conducted. That is, whether the required performances are satisfied is confirmed by judging whether or not the response value *Res* of the seismic performance factors satisfies the retained performance *Ru*. At the same time, the level to which the required performances are satisfied is judged by comparing *Res* and *Ru*.

In *Guidelines*, the critical range of functions, repair, and safety pertaining to the ultimate strength of CFT members is defined based on experimental results. Fig. 7 shows the results of an examination of the elasto-plastic behavior, strength and deformation capacity, energy dissipation capacity, and other structural performances of a CFT frame subjected to vertical load and cyclic horizontal force. The test specimen was a three-story frame composed of square CFT columns and H-shape beams.

In the figure, R_d=1/50 (rad) is shown using ▲ and ▼—the required value for the maximum story drift angle of a building with a standard seismic resistance grade to bear the load of an earthquake with a return period of about several hundred years. Further, the point of $R_d/2$ of a building having a special-class seismic resistance grade is shown using △ and ▽. From the figure, it is known that, in this case too, the strength of the frame reaches its maximum at a story drift angle of R_d=1/50 (rad) and that the building conditions are stable.

● Response Control

In cases when the required performances are judged not to have been satisfactorily met, the cross sections of the members are revised, or, as occasion demands, the structural plan is reexamined. Revisions and examinations such as these constitute performance control, which suggests changes to the practical cross sections in conformity with the required performances. The revisions of the cross sections of members, dynamic analysis, and performance control are to be repeated until conformation is made on whether or not the required performances have been met to a satisfactory degree.

Outline of Design Examples

In *Guidelines*, examples of the dynamic design method using CFT columns are introduced for the following three steel-frame structures.

1) Moment resistant frame (MRF)
2) MRF with hysteretic dampers
3) MRF of the column yielding allowable type

The plane configuration, floor height, and other specifications of the respective frames were prepared using identical frames; and the standard seismic resistance grade was assumed in the design of these frame structures. Fig. 8 shows the standard floor plan and the Y-direction framing elevation of the MRF with hysteretic dampers.

For the MRF, response revisions were examined assuming that the seismic resistance grade is raised. For the MFR with hysteretic dampers, buckling restrained braces were arranged around the core, thereby leading to smaller girder cross sections than in the MRF. For the MRF with hysteretic dampers, a trial design was made that allows yielding of the columns by fully utilizing the high performance of CFT columns.

Fig. 9 shows the seismic motion adopted and the height-direction distribution of responses to the story drift angle of the respective trial design examples. The story drift angle=1/80, the design target, is satisfied in each of the respective designs, which indicates that the current method enables the design of CFT frames that are more rational and higher in seismic safety than possible with the conventional static design method.

*****

The above presents the framework of a seismic design method based on non-linear, time-history response analysis that aims to more rationally and in a more optimal manner secure the functionality of CFT frame structures subjected to strong earthquakes. Also introduced is an outline of *Guidelines for Dynamic Seismic Design of Steel Frames Using CFT Columns*, which indicates the basic data regarding input seismic motion, CFT frame models, assessment criteria, and practical measures for improving response. The usefulness of *Guidelines* was further indicated using design examples. The Japanese version of *Guidelines* was published in August 2006 by the Japanese Society of Steel Construction.

Fig. 7 Examples of Deformation Capacity of CFT Frame

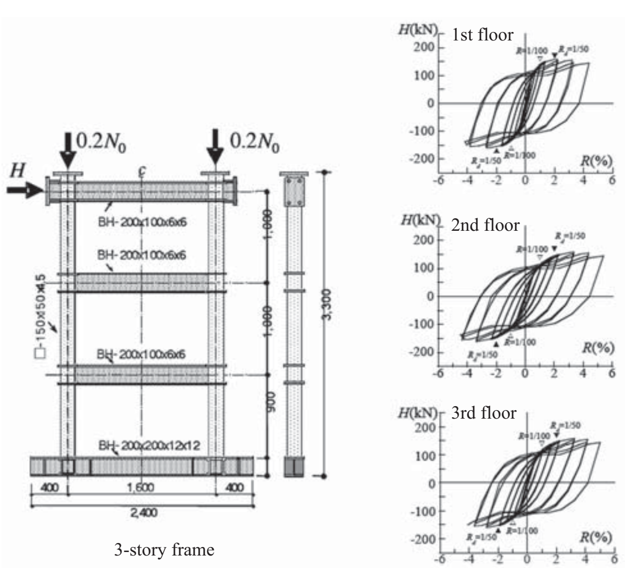


Fig. 8 Particulars and Configuration of Frame with Hysteretic Damper of Trial Design Examples

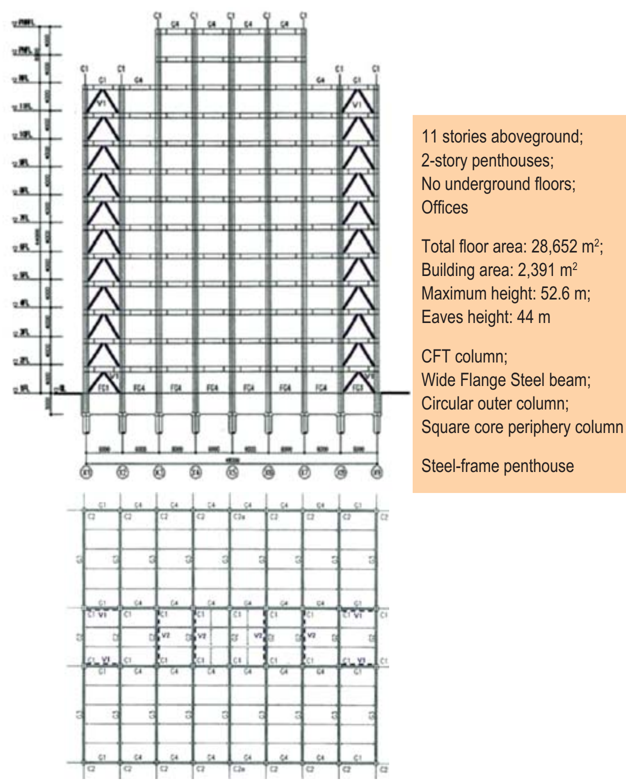
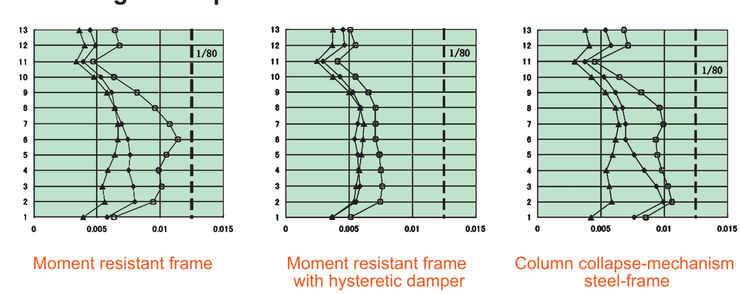


Fig. 9 Height-direction Distribution of Responses to Story Drift Angles and Seismic Motions Used in Respective Trial Design Examples



Name of earthquake	Maximum velocity	Maximum acceleration
EL CENTLO 1940 (NS)	50cm/sec	511gal
HACHINOHE 1968 (EW)	50cm/sec	255gal
JMA KOBE 1995 (NS)	50cm/sec	470gal

Steel-structure Harbor Facilities in Asia (Two-part Series: 1)

Subic Bay Port Development Project

—13.7 m Deep Container Berth Supported by Steel Pipe Piles—

by Isao Michishita
Contractor's Representative (Project Manager)
Penta-Shimizu-Toa Joint Venture

Project Background

Subic Bay in north-west Luzon of the Republic of the Philippines, well known as a former Navy Base of the United States of America, is now under development to rebuild the bay area as a new commercial container port. After US Navy returned the Subic Bay in 1992, the Subic Bay Metropolitan Authority (SBMA) was founded by the Philippine government to exercise overall control over the redevelopment of the Subic Bay.

A feasibility study was entrusted to the Japan International Cooperation Agency (JICA) in 1998 for the evaluation of potentiality of the Subic Bay as a new container port. The study report estimated that the potential quantities of container and non-container cargos at the Subic Bay Port would be 420,000 TEU or 1.24 million tons in 2010 and 720,000 TEU or 1.79 million tons in 2020, respectively. In response to this estimation, two 280-m long container berths with 13.7-m depth at wharf front suitable for a 2,000 TEU capacity container cargo vessel will be required by 2007 and an additional berth of the same scale by 2015 for container cargo handling. For non-container cargos, the existing wharves, such as the Marine Terminal at NSD District and Boton Wharf, will have to be required to be expanded and rehabilitated.

Photo 1 New Container Terminal at Cubi Point

After a basic and detail engineering design by the Japan-based Pacific Consultant International (PCI) that followed the master plan concluded in the feasibility study report, the Subic Bay Port Development Project was commenced in May 2004 under a special yen loan scheme of the Japan Bank for International Cooperation (JBIC). As of June 2006 as shown in Photos 1 and 2, the progress of the project was approximately 76%.

Photo 2 Marine Terminal (NSD District) — Marginal pier to the existing wharf

Preparation of Steel Pipe Piles

The design concept allowed steel pipe piles as the supporting foundation of reinforced concrete superstructures for both the New Container Terminal and a marginal pier at the Marine Terminal. The majority of the piles were imported from Japan, with their maximum length of 17 m due to

Fig. 1 New Container Terminal—Plan, Section and Sub-soil Data

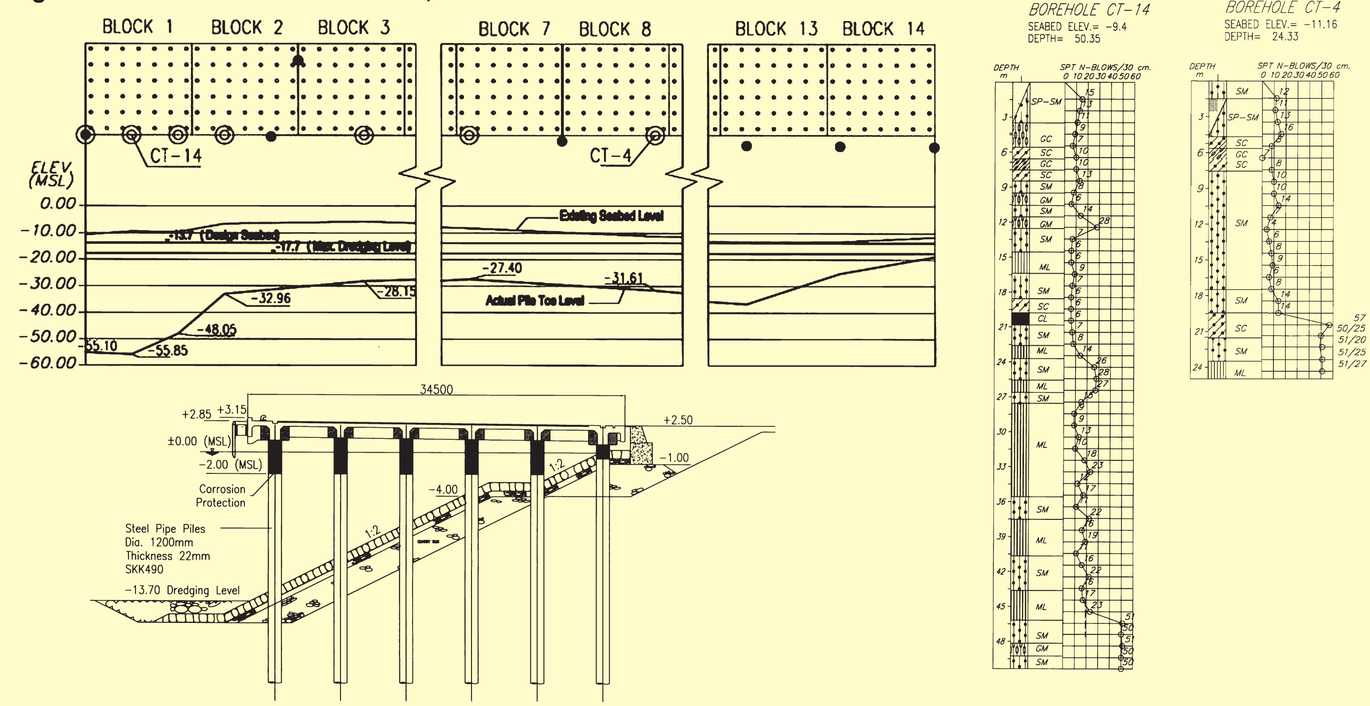
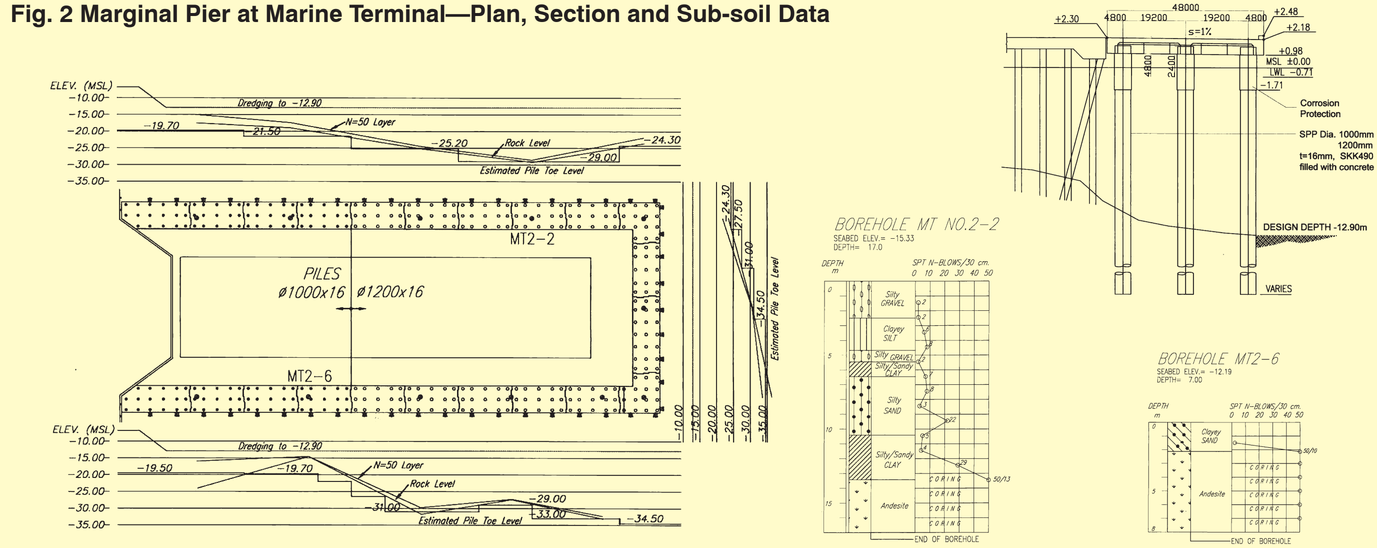


Fig. 2 Marginal Pier at Marine Terminal—Plan, Section and Sub-soil Data



space restriction of shipping cargo vessels. The piles were joined at the project site using two sets of semi-automated welding machines to prepare piles to the required full length. For the piles whose required full length was longer than 42 m, an offshore splice welding was duly employed due to limitation in leader height of the piling barge.

Driving Piles at New Container Terminal

The detail design for the New Container Terminal adopted the vertical configuration of steel piles as shown in Fig. 1. A total of 588 piles are 1,200 mm in outer diameter and 22 mm in wall thickness (JIS SKK490). The design working load of those piles is 2,200 to 3,500 KN for a long term and 1,600 to 4,500 KN for a short term to cater quay gantry cranes and container handling load.

A soil investigation conducted prior to pile driving reported a complicated sub-soil condition. While the estimated foundation strata of the piles (over 50 of SPT N value) was relatively stable at the depth of –30 to –40 m MSL at wharf block No. 3 to No. 14, the strata was deepened to –56 m MSL at wharf block No. 1 and No. 2. Due to this complexity of the foundation strata, the offshore splice welding for joining piles was unavoidable. A piling barge was utilized for offshore piling works as shown in Photo 3.

Pre-drilled Piles in Marginal Pier at Marine Terminal

The number of the piles for the marginal pier at the Marine Terminal is 340 (JIS SKK 490), whereas it is 126 for 1,000 mm

Photo 3 Pile driving at New Container Terminal

Photo 4 Pre-drilling at Marine Terminal

Photo 5 Static load test (Kentledge)

in outer diameter and 214 for 1,200 mm in outer diameter. The wall thickness of the piles is 16 mm as shown in Fig. 2.

It had been recognized prior to piling that there would be the shallow outcrop of andesite rock strata at the Marine Terminal. The design of the marginal pier was proposed not to employ battered piles to eliminate risks of shallowly socketed tension piles but to adopt only vertical piles with infill concrete securing rigidity of the pile for resistance against excess lateral movement of the wharf structure during vessel berthing and earthquake.

Pre-drilling was required for the vertical piles at the area where the shallow outcrop of the rock strata is observed to meet the minimum required depth of pile toe level –19.5 m MSL. A floating pontoon equipped with an auger machine was used to drill the rock strata as shown in Photo 4.

Pile Driving Criteria and Static Load Testing of Piles

Piling works have been carried out using a 10-ton hydraulic hammer mounted on the piling barge. Two piling barges were in charge for the works which lasted for 12 months for the New Container Terminal and 6 months for the marginal pier at the Marine Terminal including pre-drilling.

The final set of pile driving was controlled by Hiley's Formula which practically provides the bearing capacity of the driven pile. Penetration and rebound of the pile for the final 10 blows of hammering were recorded for this purpose. The average final penetration of the pile was in the range of 3-6 mm/blow when the pile driving ceased.

Static load tests were conducted adopting so-called Kentledge Method to verify the bearing capacity of the pile as shown in Photo 5. Pile dynamic analysis was conducted to the selected piles to measure the bearing capacity and skin friction of the piles as well, in comparison with the pile set criteria by Hiley's Formula and static load test results.

Corrosion Protection System for Steel Pipe Piles

All piles in both the New Container Terminal and the marginal pier at the Marine Terminal are protected against corrosion by means of 4 layers of protection system as shown in Fig. 3. The corrosion protection is applied at the wave splash zone up to –2.0 m MSL.

Fig. 3 Corrosion Protection System

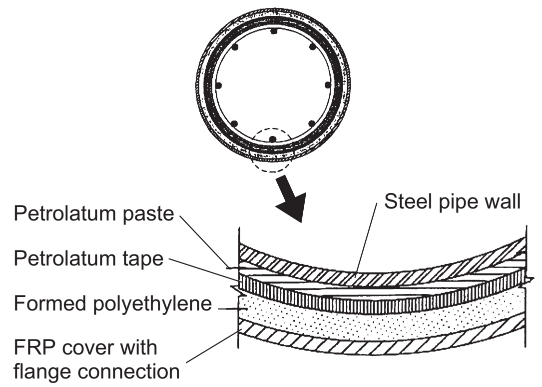


Photo 6 Quay gantry crane being unloaded

Other Steel Products: Quay Gantry Crane

There is another major steel-made equipment, quay gantry crane (QGC), under scope of this project. In total, four units of QGCs are to be designed, fabricated, delivered and commissioned. QGC has a so-called goose neck type boom with telescopic spreader due to the aviatory height restriction instituted by the Subic Bay International Airport, which is located next to the New Container Terminal. Maximum lifting capacity is 53 tons and total weight of the QGC is approximately 800 tons. To date, two units of QGCs have been delivered to the site as shown in Photo 6.

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The project is now in its final stage: a first half of the New Container Terminal and the marginal pier at the Marine Terminal will be completed by the first quarter of 2007 and the whole project including a second half of the New Container Terminal, access road, rehabilitation of existing wharves and port administration building will be completed by the second quarter of 2007.

Recent Technical Papers

STEEL CONSTRUCTION ENGINEERING

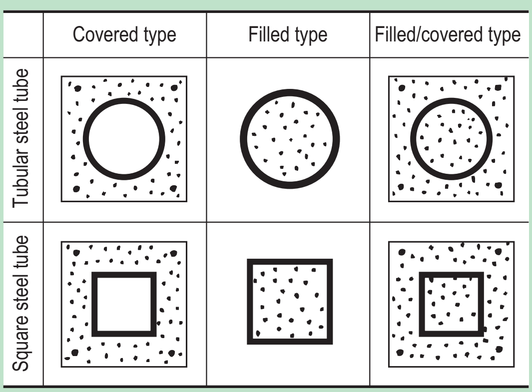
● **Structural Performance and Design of Concrete-filled Steel Tubular Structures**

Chiaki Matsui (Dr., Professor, Dept. of Architecture, University of Kyushu)

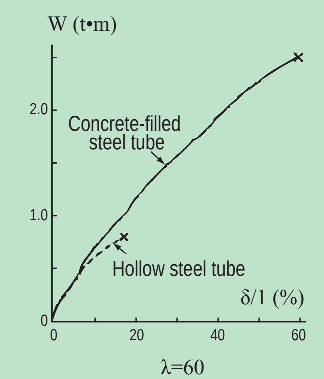
ABSTRACT: The present situations of design and construction of concrete-filled steel tubular structures in Japan are described. An outline of research works conducted at the Kyushu University, such as the elasto-plastic behaviors of frames, the ultimate strength and deformation capacity of members, the limiting values of width (diameter)-to-thickness ratio of cross sections, the stability and strength of slender columns, and the behaviors of truss frames, are discussed. The design strength of columns based on several foreign design standards is compared with the Japanese design strength.

(No. 2, Vol. 1, June 1994)

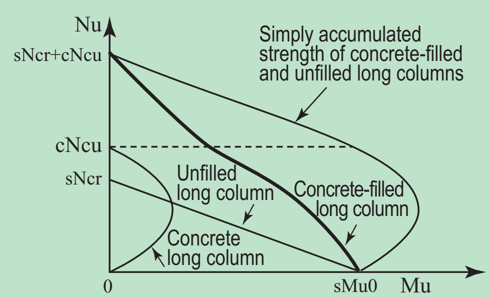
Types of Cross Section of Steel Pile Concrete



Energy Absorbing Capacity



Accumulated Strength of Long Columns



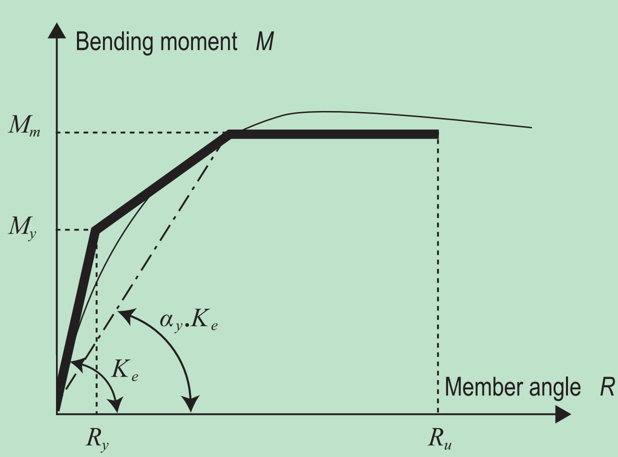
● **Evaluation of Deformation Capacity of Concrete-filled Steel Tubular Slender Columns**

Toshiaki Fujimoto (Dr., Technical Research Laboratory, Ando Corporation)

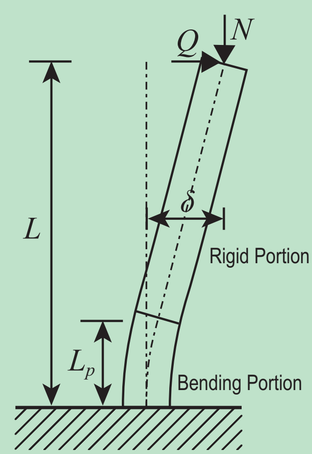
ABSTRACT: In order to evaluate the deformation capacity of a concrete-filled steel tubular slender column, investigations of a previous restoring force characteristic model and a evaluating equation of deformation capacity were conducted. Firstly, the application scope of the previous model and the evaluating equation were clarified. Consequently, it was understood that the restoring force characteristic model could be applied to CFT slender columns. However, it was thought that the evaluating equation of deformation capacity was inapplicable to CFT slender columns. Then, the evaluating method of deformation capacity of CFT slender columns was proposed based on the previous restoring force characteristic model and a evaluating equation.

(No. 43, Vol. 11, September 2004)

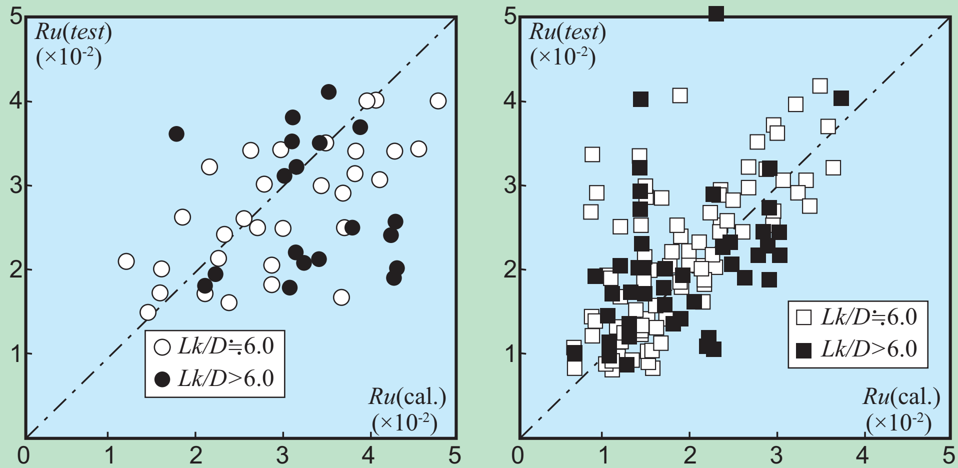
Restoring Force Model



Assumed Model



Comparison of Critical Member Angles



(a) Circular steel tube　　(b) Square steel tube

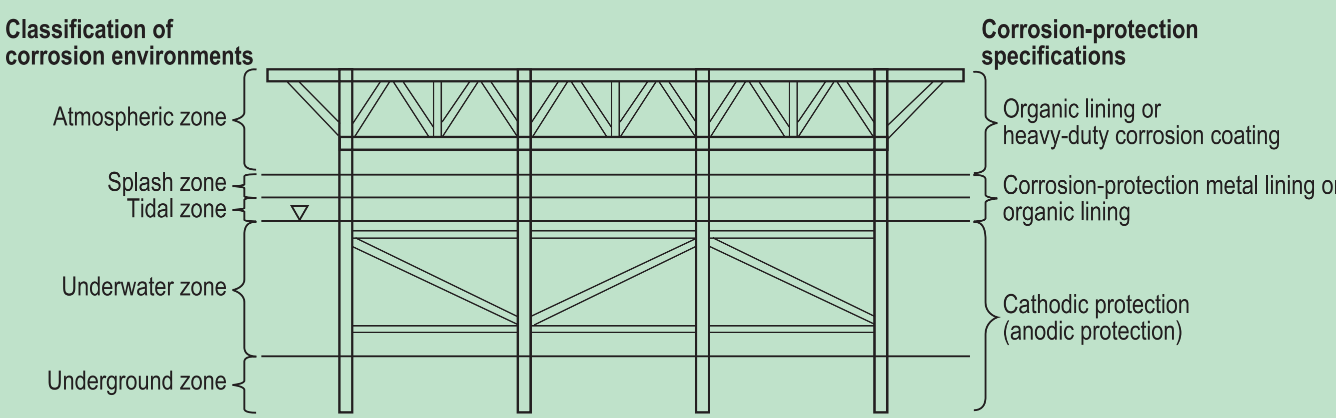
- **Proposal for Corrosion-protective Measures for Coastal Steel Structure**

Hiroyuki Horikawa (Dr., Technical Research Laboratory, JFE Engineering Corporation)
Masaki Yoshikawa (Dr., JFE Engineering Corporation)
Akihiro Tamada (Ph.D., JFE Engineering Corporation)
Mitsuyuki Hashimoto (Ph.D., JFE Engineering Corporation)

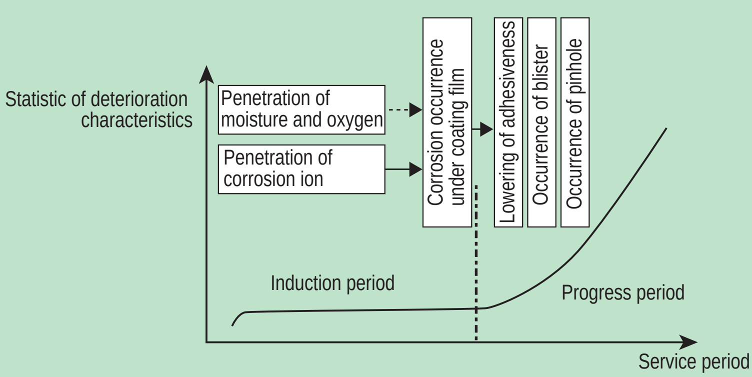
ABSTRACT: This paper proposes a new maintenance method for corrosion protection of costal steel structures. Conventionally, steel structures have been maintained following the regulations and rules based on the past precious experiences and knowledge. However, the reduction of public investment to the social infrastructures encourages more effective maintenance than that of the conventional methods. Authors developed a new and rational maintenance method for coastal steel structures to meet such demand. Especially, preventing the corrosion failure is important for coastal steel structures under severe corrosion environment of ocean. Application of quantitative probability evaluation and extreme value statistics to the maintenance method was investigated in this paper.

(No. 47, Vol. 12, September 2005)

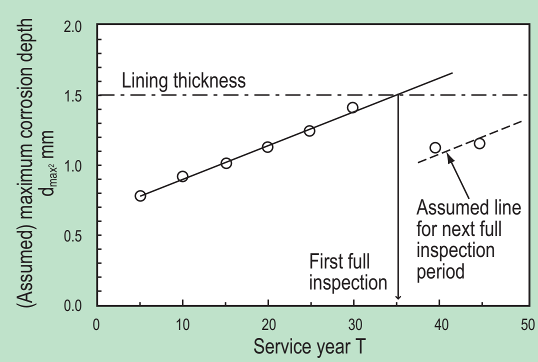
Outline of Harbor Structure



Schematic Diagram for Assumed Deterioration of Organic Lining



Method to Decide Full Inspection Period

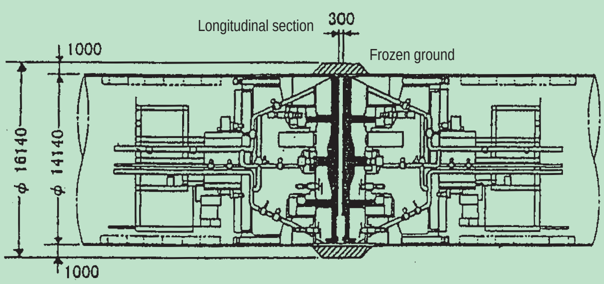


● **Welding in Construction of TOKYO WAN AQUA-LINE (Welding under Low Temperature of Underground Shield Tunnel)**

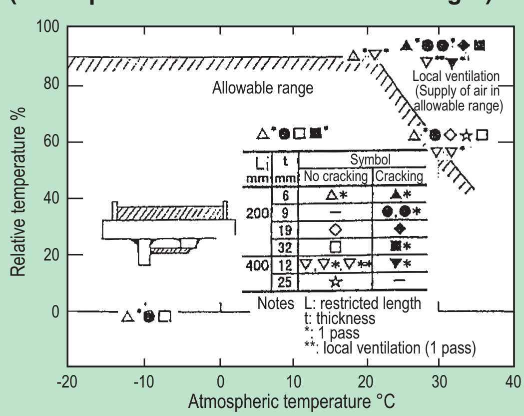
Yasumasa Nakanishi (Dr., Ishikawajima-Harima Heavy Industries Co., Ltd.)
Jun Ishii (Ph.D., Ishikawajima-Harima Heavy Industries Co., Ltd.)
Tsunayoshi Funasaki (Trans-Tokyo Bay Highway Corporation)
Daizo Tanaka (Shimizu Corporation)

ABSTRACT: For the construction of TOKYO WAN AQUA-LINE (Trans-Tokyo Bay Highway), the slurry shield method was employed. Because of the long length of the tunnel, the shield machines from both sides were joined under the ground. In this process, the ground freezing method was used as an assistant method and slurry around shield machine was frozen at –30ºC. Because the machines were also cooled, it was difficult to employ pre- and/or post-heating for prevention of cold cracking. In this study, the welding procedure for water proof and reinforcement welding without pre-and/or post-heating was examined by experiments to obtain enough joint strength. The propriety of welding design was also confirmed through the mock-up test. The actual underground connection was finished and TOKYO WAN AQUA-LINE is now under service. (No. 25, Vol. 7, March 2000)

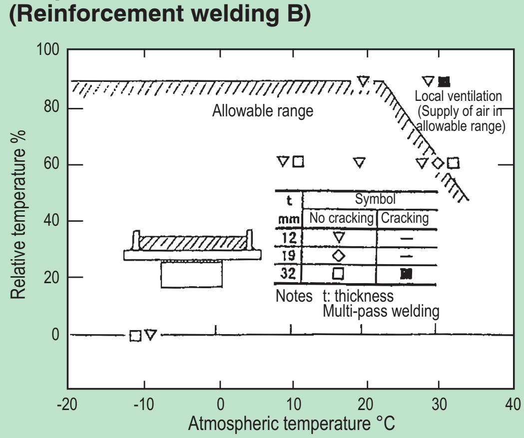
Ground Freezing Method for Underground Connection of Shield Tunnels of TOKYO WAN AQUA-LINE



Test Results for Weld Cracking under Low Temperature (Water proof and reinforcement welding A)



Test Results for Weld Cracking under Low Temperature (Reinforcement welding B)



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CONTENTS

COVER

In the Subic Bay Port development project in the Philippines, container berths supported by steel pipe piles are being constructed. (For details, see pages 14 to 16.)

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The Japan Iron and Steel Federation
3-2-10, Nihonbashi Kayabacho, Chuo-ku, Tokyo 103-0025, Japan
Phone: 81-3-3669-4815 Fax: 81-3-3667-0245
Chairman: Akio Mimura
URL http://www.jisf.or.jp
Japanese Society of Steel Construction
Yotsuya Mitsubishi Bldg. 9th Fl., 3-2-1 Yotsuya, Shinjuku-ku, Tokyo 160-0004, Japan
Phone: 81-3-5919-1535 Fax: 81-3-5919-1536
President: Akira Chihaya
URL http://www.jssc.or.jp

Editorial Group
JISF/JSSC Joint Editing Group for Steel Construction Today & Tomorrow
Editor-in-Chief: Takeshi Katayama

The Japan Iron and Steel Federation
● **Beijing Office**
Rm. 609, Jongtai Tower, 24 Jianguomenwai-Street, Chaoyang-District. Beijing. 100022, China
Phone/Fax: (010) 6515-6678
Phone: (010) 6515-6699 (Ext. 30221)