Proceedings of

JSCE-VIFCEA Joint Seminar on Concrete Engineering in Vietnam

December, 8^{th} - 9^{th}, 2005
Ho Chi Minh City University of Technology

Organized by:

VIFCEA
Viet Nam Federation of Civil Engineering Associations
Ho Chi Minh City Society of Civil Engineers

JSCE
Japan Society of Civil Engineers

HCMUT
Ho Chi Minh City University of Technology

Edited by
KOJI SAKAI and NGUYEN VAN CHANH

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PREFACE

The JSCE (Japan Society of Civil Engineers) - VIFCEA (Vietnam Federation of Civil Engineering Associations) Joint Seminar on Concrete Engineering in Vietnam was held on December 8th – 9th, 2005 in Ho Chi Minh City University of Technology under the agreement of cooperation between JSCE and VIFCEA. The purpose of this seminar is to bring advanced information on concrete in Japan and Vietnam to Vietnamese civil engineers and researchers and to enhance the collaboration between JSCE and VIFCEA.

The proceedings of this seminar contain five papers from JSCE and five papers from VIFCEA. The papers from Japan include the topics on durability design and structural design of concrete structures, roller compacted concrete dam, high strength concrete, and prestressed concrete bridges. The papers from Vietnam include the topics on concrete technology in hot humid climate, high performance concrete, steel fiber reinforced concrete, advanced concrete applied at Vietnam, and potentiality of Vietnam concrete industry.

We hope that the information provided in this seminar will contribute to the construction industry in Vietnam.

This seminar was held with a financial support from the JSCE and VIFCEA. Full acknowledgement should be extended to both organizations for their special consideration. We also would like to express our sincere gratitude to all speakers in this seminar, JSCE Concrete Committee, JSCE International Activities Committee, and Ho Chi Minh City Society of Civil Engineers of VIFCEA for their devoted support. Special gratitude also goes to Ho Chi Minh City University of Technology for their full support.

Koji Sakai (Kagawa University)
Nguyen Van Chanh (HCMC-University of Technology)
Editors
Vietnam - Ho Chi Minh City, December 2005

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THE JSCE DURABILITY DESIGN OF CONCRETE STRUCTURES AND A PROPOSAL FOR VIETNAM CONSTRUCTION INDUSTRY

Koji SAKAI[1]

SUMMARY

Concrete is one of the most important materials employed in public works and building construction projects and a countless number of concrete structures have been constructed worldwide. Although concrete structures are designed on the basis of fundamental performance requirements, the background for design is drastically changing. This paper outlines sustainability issues, describes the present state of durability design, introduces durability design methods of the JSCE Standard Specifications for Concrete Structures, and discusses the future direction in the design of concrete structures.

***Keywords**: Sustainable construction; durability design; JSCE Standard Specification; carbonation; chloride ions ingress; freezing and thawing action; chemical attack; alkali aggregate reaction; environmental design.*

INTRODUCTION

Concrete is one of the most important materials employed in public works and building construction projects. It is estimated that more than a ton of concrete is produced each year for every human being on the planet. The estimated cement consumption of the world in 2002 and 2020 is 1696 and 2132 million tons, respectively[1)]. According to Humphreys et al. (Humphreys et al. 2002), cement consumption is expected to increase almost uniformly until the middle of this century, to reach 4 to 6 billion tons. These circumstances mean that energy consumption, resources depletion, CO_2 emissions, and other environmental impacts resulting from construction activities can no longer be ignored. The conventional design framework for concrete structures is primarily based on safety and currently focused on the aspects of durability. It is obvious that environmental aspects should be also incorporated into the design of concrete structures. From an environmental viewpoint, it can generally be thought that life-extension of a structure is directly related to the reduction of environmental impact. Therefore, the establishment of reasonable durability designs for concrete structures is very important. Although many efforts have been made to evaluate the durability of concrete structures, most of the existing design codes do not provide tools with the exception of the

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JSCE Standard Specification for Concrete Structures (JSCE 2002).

This paper outlines sustainability issues, describes the present state of durability design, introduces durability design methods of the JSCE Standard Specifications for Concrete Structures, and discusses the future direction in the design of concrete structures.

SUSTAINABILITY

Sustainable Development

The true nature of global environmental problems is a result of economic society systems due to the explosion of industrialization since the Industrial Revolution, in which mass production, mass consumption and mass disposal have been pursued. Such systems have caused the destruction of ecological system due to the use of land, natural resource and energy depletion, and water pollution, the emission and diffusion of hazardous substances and greenhouse gases, waste excretions, etc. Mankind has realized that these impacts exceed its allowable limit.

As a fundamental scheme in social economic activities, therefore, a paradigm shift to sustainable development has become significant. The concept of sustainable development was proposed in Brundtland Report (WECD 1987). Sustainable Development was defined as

> "development which meets the needs of the present without compromising the ability of future generations to meet their own needs."

The report described three fundamental aspects: environmental protection, economic growth and social equality. After the publication of this report, a keyword "Sustainable Development" became firmly established as the final target of mankind.

For the sustainable development in the earth, fundamentally we have to prevent the global warming which is thought due to greenhouse gases, such as CO_2. The Kyoto Protocol adopted at the International Conference for the Prevention of Global Warming (COP3) in 1997 required Japan, the U.S.A. and the E.U. to reduce by 2008 -2012 their emission of greenhouse gas by 6, 7 and 8%, respectively, compared to 1990 levels.

Sustainable Construction

Recently, the companies manufacturing general industrial products have increased their concerns for environments. This is due to the fact that they have realized the limitation in their business in which the limit of resources is not considered.

Civil engineering structures and buildings are largely different from general industrial products in the following points:

(1) They are not mass productions.
(2) They have long life span.
(3) They have strong public aspects.

It seems that due to these special features, the concept of environmental design in the life cycle scenario did not emerge.
Construction is one of the biggest industries around world. Construction has major effects on

the global environment because construction is a major consumer of land and raw materials and the operation of building is the biggest energy consumer. The quality and quantity of construction will affect future generations. Therefore, construction industry has a significant role for sustainable development in which the needs of future generation have to be taken into consideration. In other words, sustainable construction has to be considered as a part of sustainable development.

Sustainable construction will be achieved by taking the following factors into consideration:

(1) environmentally friendly construction materials
(2) energy efficiency in buildings
(3) construction and demolition waste management

Construction materials provide the environmental impacts at each stage of the life cycle, such as raw materials extraction, processing, manufacture, distribution, and construction works (on-site materials fabrication, use, and demolition waste). To reduce the environmental impacts, it may be the most important to minimize the amount of virgin materials use. Over-design and under-design should be avoided.

The environmental design system of buildings and structures, in which the selection of materials and structural shape, construction works, maintenance, and demolition/ recycling are included, should be established to minimize the use of resources and energy and to manage construction and demolition waste.

In the conceptual design, in which the owner and designer should agree, environmental aspects for sustainable construction have to be considered in addition to general matters, which include structural concept, location, cost, construction term, and performance requirements.

Concrete is made of cement, water and aggregates. Cement production consumes lots of energy and emits a large amount of CO_2. In addition, aggregate extraction causes natural destruction which includes land use, loss of eco-system, amenity loss etc. The construction and demolish waste are one of the serious problems in construction industry. On the other hand, several industrial by-products, such as blast furnace slag, fly ash, silica fume etc., have been used in concrete. Thus, concrete has a great role for sustainable construction. In other words, sustainable concrete construction has to be considered.

PRESENT STATE OF DURABILITY DESIGN

We have been facing various and sometimes serious durability problems in concrete structures. According to ACI Cement and Concrete Terminology (ACI 116R-00 2000), durability is defined as the ability of concrete to resist weathering action, chemical attack, abrasion, and other conditions of service. The severity of environmental, chemical, and physical attacks on concrete depends on the properties of concrete and its exposure conditions.

The general deterioration phenomena of concrete structures include alkali-aggregate reaction

(Photo 1), freezing and thawing (Photo 2), corrosion of reinforcement in concrete (Photo 3), carbonation of concrete, etc. The alkali-aggregate reaction can be prevented by several countermeasures such as the setting of threshold content of alkali in cement, the utilization of cementitious materials, etc. Freezing and thawing resistance can be secured by introducing an appropriate air-void system in concrete. Concerning the corrosion of reinforcement, it has been understood that concrete cover and its quality are the key and many efforts have been made to evaluate it quantitatively. The carbonation of concrete has a disadvantage in reinforced concrete because the pH of carbonated concrete drops to a value below the passivation threshold of steel. Nevertheless, most of the existing durability design codes do not provide a tool for evaluating the ingress of chloride ions in concrete, the carbonation of concrete due to carbon dioxide, and other deteriorations.

Durability design may be categorized into three levels as follows:

(1) prescriptive design
(2) performance-type design
(3) performance-based design

In the prescriptive design for the durability of concrete, for example, the maximum water cement ratio and minimum cement content are provided depending on the exposure conditions. In the EN206 (BS EN206-1 2000), the recommended limiting values for composition and properties of concrete are provided as indicated in Table-1. In the ACI Building Code Requirements for Structural Concrete (ACI 318-02 2002), total air content for frost-resistant concrete, requirements for special exposure conditions (Table-2), requirements for concrete exposed to deicing chemicals, and requirements for concrete exposed to sulfate-containing solutions (Table-3) are provided for durability. All requirements are stipulated in a prescriptive manner.

It may be said that prescriptive design is based on the simplification of safety side from real performance. However, the background of most provisions is unclear. For example, the water cement ratio for corrosion protection of reinforcement in concrete cannot be easily determined because it is directly related to the concrete cover.

Therefore, it is more reasonable and accurate to consider performance with time. In principle, the required performance should be verified through a direct analysis of time-dependent behavior of a concrete structure under the assumed environmental actions. At present, however, it is difficult to predict the durability performance of a structure throughout the lifespan because of the inadequacy of the models necessary for calculations. Further development of research on numerical approaches will pave the way to the realization of performance-based design.

Under the current situations, what we can do in our codes is to introduce a performance-type design in which principal durability performance is considered with time. The JSCE Standard Specifications for Concrete Structures introduced such a design method for durability design for the first time in 2002.

JSCE STANDARD SPECIFICATION FOR CONCRETE STRUCTURES

Framework

The JSCE Standard Specification stipulates the following fundamental requirements at the design stage of structures:

At the design stage, structural details such as the shape, size, reinforcement arrangement, required properties of concrete and reinforcing material, method of construction (in situ, pre-cast, etc.) and maintenance plan should be decided also taking into account the economic consideration. It should also be ensured that the required performances in terms of serviceability, safety, durability and compatibility with the environment, etc. are satisfied over the service life of the structure.

The JSCE Standard Specifications for Concrete Structures consist of the following four versions:

(1) Structural performance verification
(2) Seismic performance verification
(3) Materials and construction
(4) Maintenance

The procedure for verifying mechanical performance of concrete structures is given in the specification for structural performance verification and the specification for seismic performance verification.

The performance of concrete structures varies over time due to environmental conditions and other factors. The examination on whether such change is in acceptable range is described in the specification for materials and construction.

Once the construction is completed, it is difficult to repair, strengthen or renovate concrete structure, so thorough investigation at the beginning stage of design, accurate prediction for possible problem in service life and future maintenance are of great importance. The specification for maintenance provides basic knowledge for the maintenance of concrete structures.

Figure 1 indicates the framework of the contents to be covered in each specification.

Durability Verification of Concrete Structures

General concept

In the durability verification of concrete structures in the Specifications for Materials and Construction, the following provisions are provided:

(1) The performance of concrete structures shall remain the required performance throughout its designed service life.
(2) This chapter deals with the performance verification for the deterioration of structure on account of carbonation, the ingress of chloride ions, cyclic freezing and thawing action, chemical attack, and alkali aggregate reaction. The chapter also deals with the verifications of water tightness and fire resistance of the structure. It is further recommended that the simultaneous action of two or more mechanisms should be appropriately considered when

required.

It should be noted that this provision requires the performance verification of durability for concrete structures, not for concrete. This means that even if concrete or reinforcements partially deteriorate, there will be no problems when utilized under a certain condition in which the required performance of the concrete structure is satisfied. This is a fundamental concept in the specification, which is completely different from the existing prescriptive durability design methods.

Verification for carbonation

The verification of a structure for carbonation is conducted as follows:

(1) The required performance of concrete structures shall not be impaired by the carbonation of concrete.

(2) Verification for carbonation should be conducted by ensuring that

$$\gamma_i \frac{y_d}{y_{\lim}} \leq 1.0 \qquad (2.2.1)$$

Where,

γ_i: *Factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.*

$y_{\lim}$: *Critical carbonation depth of steel corrosion initiation. In general, it may be obtained from Equation (2.2.2).*

$$y_{\lim} = c - c_k \qquad (2.2.2)$$

Where,

c :*Expected value of cover thickness (mm). In general, it may be taken as the design cover thickness (mm)*

c_k : *Remaining non-carbonated cover thickness (mm). This may be taken as 10mm for structures in a normal environment, and between 10 and 25 mm for structures located in chloride rich environments.*

y_d : *Design value of carbonation depth. In general, it may be obtained from Equation (2.2.3).*

$$y_d = \gamma_{cb} \cdot \alpha_d \sqrt{t} \qquad (2.2.3)$$

Where, α_d :*Design carbonation rate* ($mm/\sqrt{year}$), *which is given as*

$$= \alpha_k \cdot \beta_e \cdot \gamma_c \text{ , where}$$

α_k : *Characteristic value of carbonation rate* ($mm/\sqrt{year}$)

t : *Designed service life of structure (year). Equation (2.2.3) should be used to evaluate the carbonation depth only for a structure whose service lives is less than 100 years.*

β_e : *Coefficient representing the extent of environmental action. It may be taken as 1.0 for environments in which structures are difficult to be dried out or for north-facing surfaces. It may be increased to 1.6 for environments in which structures can be easily dried out or for South-facing surfaces.*

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γ_{cb}: Safety factor to account for the variation in the design value of carbonation depth. Normally it may be taken as 1.15. In the case of high fluidity concrete, it may be taken as 1.1.

γ_c: Factor to account for the material properties of concrete. In general it may be taken as 1.0, but should be taken as 1.3 for upper portions of the structure. However, if there is no difference in the quality of concrete in structure and that of laboratory-cured specimens, the value of 1.0 may be adopted for the whole structure.
(3) When normal Portland cement is used, water to cement ratio lower than 50%, and the thickness of cover concrete not smaller than 30cm, the verification for carbonation may be omitted.

Figure 2 shows the relation between the effective binder ratio and coefficient of carbonation speed. The data consist of different types of binder, including fly ash and blast furnace slag. Although there is large scattering in the data, the following equation was introduced:

$$\alpha_k = -3.57 + 9.0\ W/B$$

where

W/B: water binder ratio

It is obvious that the carbonation of concrete is dependent on the exposure environment of the structure. To consider the effect of exposure environment, environmental factor β_e was introduced. As indicated in Fig. 3, the β_e -values for concrete in a dry environment or when concrete faces south and that in a wet environment or when the concrete faces north are 1.6 and 1.0, respectively. Fig. 4 shows the minimum cover at different water cement ratios and service years in which the corrosion of reinforcing bars due to carbonation of concrete is prevented.

Verification for reinforcement corrosion due to the ingress of chloride ions

The verification of a structure for reinforcement corrosion due to the ingress of chloride ions is conducted as follows:

(1) The required performance of concrete structures shall not be impaired by corrosion of the reinforcement caused by the ingress of chloride ions.
(2) The verification for reinforcement corrosion caused by the ingress of chloride ions should be conducted by ensuring that

$$\gamma_i \frac{C_d}{C_{\lim}} \leq 1.0 \qquad (2.3.1)$$

Where,
γ_i: Factor representing the importance of the structure. In general, it may be taken as 1.0, but should be increased to 1.1 for important structures
$C_{\lim}$: Critical chloride concentration for initiation of steel corrosion. It may be normally taken as 1.2 kg/m^3. However in cases when freezing and thawing action is likely to occur simultaneously, the critical value should be suitably reduced.
C_d:Design value of chloride ion concentration at the depth of reinforcement. It may be

obtained from Equation (2.3.2).

$$C_d = \gamma_{cl} \cdot C_0 \left(1 - erf\left(\frac{0.1 \cdot c}{2\sqrt{D_d \cdot t}}\right)\right) \qquad (2.3.2)$$

C_0:Assumed chloride ion concentration at concrete surface (kg/m^3). Generally, it may be obtained from Table 2.3.1

c: Expected value of concrete cover thickness (mm). In general, the designed cover thickness may be selected.

t: Designed service life of the structure (year). It may be noted that Equation 2.3.2 should be used to evaluate the concentration of chloride ions at the location of the reinforcement only in cases the service life is less than 100 years.

γ_{cl}: Safety factor, to account for the variation in the Design value of the chloride ion concentration at the depth of reinforcement C_d. Normally, it may be set at 1.3, but in the case of high fluidity concrete, a value of 1.1 may be selected.

D_d: Design value of diffusion coefficient of chloride ions into concrete (cm^2/year) It may be obtained from Equation 2.3.3.

$$D_d = \gamma_c \cdot D_k + \left(\frac{w}{l}\right) \cdot \left(\frac{w}{w_a}\right)^2 \cdot D_0 \qquad (2.2.3)$$

γ_c: Factor to account for the material properties of concrete. In general, it may be set at 1.0 but should be taken as 1.3 for upper portions of the structure. However, if there is no difference in the quality of concrete in structure and that of specimens cured in laboratory, this value may be taken as 1.0 even for all portions of the structure.

D_k:Diffusion coefficient of chloride ion in concrete (cm^2/year)

D_0: Constant to express the influence of crack on the movement of chloride ions into concrete (cm^2/year). In general, it may be taken as 200cm^2/year

w: Crack width (mm)as per Section 7.4.4 of the Standard Specifications for Concrete Structures (Volume I - Design).

w_a: Allowable crack width (mm), as per Section 7.4.2 of the Standard Specifications for Concrete Structures (Volume I - Design).

w/l: Ratio of crack width to crack interval as per Section 7.4.5 of the Standard Specifications for Concrete Structures (Volume I - Design)

erf(s) is error function, defined as $erf(s) = \frac{2}{\pi^{1/2}} \int_0^s e^{-\eta^2} d\eta$

Table 2.3.1 Chloride ion concentration at concrete surface C_0 (kg/m^3)

Tidal and splash zone	Distance from the coastline (km)				
	Near coastline	0.1	0.25	0.5	1.0
13.0	9.0	4.5	3.0	2.0	1.5

Considering the effect of elevation above the water surface, an increase of 1m in elevation may be considered to be equivalent to a horizontal distance of 25m. The equivalent C_0 may then be calculated from the above table.

(3) When it is difficult to meet the stipulation in (2) above, other steps to ensure the required durability should be considered. Appropriate countermeasures, such as coating concrete surface with appropriate coating materials, using corrosion resistant reinforcing materials and adopting cathodic protection are recommended under these conditions. In such cases, the

method to be used should be properly evaluated, taking into account the maintenance plan of the structure.
(4) In cases when ingress of chloride ions from the environment is not likely, the reinforcement may be regarded to be protected from chloride-induced corrosion, provided the concentration of chloride ions in fresh concrete does not exceed 0.30 kg/m3. In the case of using prestressing steel material, which is likely to be more susceptible to stress-corrosion, the limit should be suitably reduced.
(5) In cases when deicing agents are used, the durability performance of concrete structures should be especially considered. Further, ingress of chloride ions into concrete should be prevented using water proofing or providing adequate drainage.

In the calculation, a chloride diffusion coefficient must be set as the design value. This means that the actual concrete has a diffusion coefficient for which the designed value is on the safety side. In the specification, the following equation for the prediction of the diffusion coefficient of actual concrete, D_p, were introduced as the predicted values:

(a) When ordinary Portland cement is used

$\log D_p = -3.9\,(W/B)2 + 7.2\,(W/B) - 2.5$

(b) When blast furnace slag and/or silica fume are used

$\log D_p = -3.0\,(W/B)2 + 5.4\,(W/B) - 2.2$

These equations were introduced on the basis of a large amount of data shown in Fig. 5. Large data scattering is also seen in the chloride diffusion coefficients. The reason the JSCE decided to introduce such provisions is very simple. It is essential for us to pursue a direction for evaluating the durability performance of concrete structures. A prototype provision will accelerate research work focused on improving the equations. More extensive work is necessary.

Verification for cyclic freezing and thawing action

The verification of a structure for cyclic freezing and thawing action is conducted as follows:

(1) The required performance of concrete structure shall not be impaired by cyclic freezing and thawing action.
(2) Verification for freezing and thawing action should be conducted by ensuring that

$$\gamma_i \frac{E_{min}}{E_d} \leq 1.0 \qquad (2.4.1)$$

where, γ_i *is a factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.*
E_d *:Design value of relative dynamic modulus of elasticity.*

$$= E_k / \gamma_c$$

E_k *: Characteristic value of the relative dynamic modulus of elasticity.*
γ_c *:Material factor. In general, it may be set at 1.0, but should be taken as 1.3 for upper positions of the structure. However, if there is no difference in the quality of concrete in the structure (in situ) and that of laboratory-cured specimens, this value may be set at 1.0 for all positions.*
E_{min} *: Critical minimum value of relative dynamic modulus of elasticity to ensure required performance of the structure under cyclic freezing and thawing action. In general, it may be obtained from Table 2.4.1.*

Table 2.4.1 Minimum critical level, E_{min}(%), of relative dynamic modulus of elasticity to ensure a satisfactory performance of the structure under cyclic freezing and thawing action

Climate / Section / Exposure of structure	Severe weather conditions or frequent cyclic freezing and thawing action		Not so severe weather conditions, atmospheric temperature rarely drop to below 0°C	
	Thin [2]	General	Thin [2]	General
(1) Immersed in water or often saturated with water [1]	85	70	85	60
(2) Not covered in item (1) above and subjected to normal exposure conditions	70	60	70	60

1) Structures close to the water surface or in contact with water such as waterways, water-tanks, abutments of bridge, bridge piers, retaining walls, tunnel linings, etc. Besides, structures such as slabs, beams etc not close to the water surface but may be exposed to snow, water flow, spray, etc., also belong to this category.
2) Members with thickness less than 20cm may be considered 'thin'.
3) Generally, the verification in section (2) may be omitted in cases when the characteristic value of relative dynamic modulus of elasticity, E_k, is higher than 90.

Verification for chemical attack

The verification of a structure for chemical attack is conducted as follows:

(1) The required performance of concrete structure shall not be impaired by chemical attack
(2) In case the concrete meets the criteria for resistance against chemical attack, the structure may be assumed that its performance will not be impaired on account of chemical attack
(3) In cases when the action of chemical attack is very severe, measures, such as covering the surface of concrete and using corrosion resistant reinforcement, to control the chemical attack should be taken. The effectiveness of such measures should be evaluated using appropriate methods, taking into account the maintenance plan of the structure.

Due to the limited understanding on how the deterioration of concrete under chemical attack results in the degradation of the structure performance, unfortunately, quantitative evaluation has not been realized. Therefore, only conceptual provisions are introduced for the verification of a structure for chemical attack. In an actual verification, accelerated, exposure tests, or any other suitable tests shall be performed on concrete specimens at the conditions as close to the actual conditions as possible.

When it is required that the deterioration of concrete is just within the level that does not affect the required performance of the structure, instead of the verification on the resistance to chemical attack of concrete, it is allowed in the specification to use the following maximum water-cement ratio:
(a) When concrete are in contact with soil or water, which contain 0.2% or more of a sulfate such as SO_4, maximum water cement ratio is 50%.
(b) When deicing salts are used, maximum water cement ratio is 45%.

Verification for alkali-aggregate reaction

The verification of a structure for alkali-aggregate reaction is conducted as follows:

(1) The required performance of concrete structure shall not be impaired by alkali aggregate reaction
(2) In case the concrete meets criteria for resistance against alkali aggregate reaction, the structure may be assumed that its performance will not be impaired on account of alkali aggregate reaction.
(3) The performance of the concrete structure against alkali aggregate reaction may be secured using appropriate surface treatment. In such cases, the effectiveness of the treatment shall be evaluated using appropriate methods, taking into account the maintenance plan of the structure.

The most reliable method to verify the resistance of a structure to alkali aggregate reaction is to cast concrete specimens in the same conditions to the real structure, expose them in similar environmental conditions and confirm the possibility of crack formation. However, considering various involving factors, such as the testing time as well as the required expense, the need to test concrete at various kinds of materials and proportions, etc., this real exposure test is not always feasible.

Therefore, at present the verification for the resistance to alkali aggregate reaction is usually carried out on the basis of the accelerated tests using concrete specimens. The JCI AAR-3 [Test method for evaluation of aggregate reactivity in concrete] is one of such methods that may be used as a reference in carrying out accelerated tests. It has been confirmed from laboratory experiments and field investigations that an expansion level of less than 0.1% at the age of 6 months in the test carried out based on this method, may not cause appreciable degradation in the performance of concrete structures. This level can thus, be used as a standard to decide whether or not the expansion on account of alkali aggregate reaction could be detrimental.

Namely, the verification for resistance to alkali aggregate shall be conducted out by insuring that:

$$\gamma_p \frac{L_p}{L_{\max}} \leq 1.0$$

Where, L_p :Estimated expansion of concrete due to alkali aggregate reaction (%). Generally, this is set equal to the expansion at 6-month exposure in the test in accordance with JCI AAR-3 (Determination of alkali silica reactivity in concrete).

$L_{\max}$:Maximum permissible expansion rate at which concrete still satisfies the required resistance against alkali aggregate reaction. Generally, it may be set at 0.10%.

γ_p : Safety factor to account for the accuracy in determining . Generally it is set in between 1.0 and 1.3. When tests are carried out in accordance with JCI AAR-3, it may be taken to be 1.0

When using aggregates conforming to classification A (non-reactive) as per the test method for evaluation of aggregate reactivity in concrete and restraining alkali aggregate reaction in concrete using appropriate cement, the verification for the resistance to alkali aggregate reaction can be omitted.

PARADIGM SHIFT OF DESIGN FOR CONCRETE STRUCTURES

According to the Gaia hypothesis (Joseph 1993) by meteorologist James E. Lovelock and microbiologist Lynn Margulis, "the earth's climate and atmospheric environment are controlled by animals, plants and microorganisms living on the earth." There is no doubt that the earth has an extremely rare environment in the overwhelmingly vast universe. Its entire equilibrium is thought to be maintained by direct and indirect mutual dependence of various creatures from bacteria to humans. While the atmosphere of the earth in its early days was almost entirely composed of carbon dioxide in the same way as Venus and Mars, it currently consists of 78% nitrogen, 21% oxygen and 0.03% carbon dioxide. It means that this condition has maintained the environment necessary for the survival of life. Human activities, however, began to affect this. It is said that the ancestors of mankind emerged 5 million years ago, and more than 6 billion people currently live on the earth. Such a large number of people consume enormous amounts of resources and energy to secure their living environment, and these amounts are increasing with accelerating speed. This fact is a threat to the "Gaia control" of the global environment. Disorder of the regulating function of the global environment directly and indirectly affects human health, primary production, biodiversity and social assets.

Although the range of environmental issues, which are considered to be the greatest challenge of the 21st century, is wide, measures and actions based on advanced ideas must be taken steadily and responsibly to solve those problems. The field of construction is also no exception.

Current design methods concerning concrete structures have been organized with the focus on "safety." Although "durability" has recently been incorporated in the JSCE Standard Specification as a function of time, it must be said that the environmental viewpoint, which is an essential element of the life-extension of structure and resource efficiency, was extremely weak.

In safety design, the safety of a structure is verified from the relationship between the cross-sectional force determined by the load and structural type, and the design sectional bearing force of an actual member section. In durability design, the durability is verified from the relationship between the performance evaluation index calculated based on a prescribed life cycle and the limit value. The design service life of a structure is determined by considering the service life required of the structure, the maintenance/management method, environmental conditions and the durability and economic efficiency required of the structure.

The act of promoting reduction in the environmental impact in design of a concrete structure, taking global warming and resource efficiency into account, can be considered basically the same as safety and durability design. We have two options. One is not to regulate the reduction of environmental impacts until a serious situation occurs. The other is to introduce a social and economic system including a mechanism for the reduction of environmental impacts. In other words, no matter what the main factor is, to set the limit value, a system that enforces consideration of greenhouse gas emission reduction in design should be introduced. The same can be said of other environmental aspects. If regulation values exist, they can be used as limit values.

Thus, a design method of concrete structures, in which three required performances – structural (safety, serviceability), durability and environmental performance are satisfied at the same time, will be introduced in the future. We may call it "Environmental Design", which is completely different from conventional design systems in the point that the environment is incorporated into design, a paradigm shift of design for concrete structures

At the fib Commission 3, where the author serves as chairman, basic examination of environmental design has been conducted (fib 2004). Development of "Guidelines for environmental design" is currently under way. The Task Group on Environmental Aspect, the Specification Subcommittee, the Concrete Committee of the Japan Society of Civil Engineers, where the author serves as convener, also collected and sorted information on environmental aspects and published "Recommendation on environmental performance verification for concrete structures (Draft) (JSCE 2005)". The Subcommittee on the evaluation of the environmental impact of concrete, the Concrete Committee of the Japan Society of Civil Engineers also presented useful information (JSCE 2004). The author believes that we can achieve global-scale accountability for environmental problems in the field of concrete if we can disseminate environmental design in the near future through these activities, establish an "integrated design" system to cover structure, durability and environment comprehensively.

A movement towards a paradigm shift of design for concrete structures is accelerated

A PROPOSAL FOR VIETNAM CONSTRUCTION INDUSTRY

Vietnam is expected to pursue further advancements by increasingly developing its infrastructure. In other words, the development of infrastructure is most significant for the future of Vietnam. To date, advanced countries have vastly built up their infrastructures; experiencing many problems in the meanwhile. One such problem is the deterioration of concrete structures. In one extreme case, this deterioration resulted in a bridge collapse. At the present stage, it is considered that many concrete structures have potential risks in terms of their durability. This leads to another problem, which is, the percentage of construction budgets dedicated to repair, strengthening and renewal projects is steadily increasing.

In Japan, the construction investment dropped by approximately 40 percent in the past 10 years and a further decline is expected. Such developments in the construction industry will result in the potential loss of construction technology and know-how, which have been accumulated over a long period of time. If there are no new projects, valuable technology will disappear. In a global sense, it will lead to the deterioration of resource and energy efficiency, since developing countries would be unable to utilize the technology accumulated by advanced countries and would be required to build up know-how on the construction technology on their own. Therefore, developing countries would probably repeat the same failure that advanced countries have experienced.

Developing countries are required to establish an infrastructure development strategy based on the assumption that the technology accumulated by advanced countries can be efficiently utilized. Considering that these regions, in which 80 percent of the world's population lives, are pursuing further economic development, it is very significant for them to learn from the "experiences of advanced countries" and reduce global environmental impacts. There is no need to hesitate in introducing highly advanced technology from advanced countries.

In fact, Vietnam does not always have to start from the ground up. In terms of global environmental issues, it is important for Vietnam to be able to use the best available technologies and work to further develop these technologies in manner that is most suitable for Vietnam. The introduction of advanced technology as a project to utilize the Kyoto Protocol CDM is an option. Thus, the concept of infrastructure development in developing countries largely differs from the conventional infrastructure development. The idea of sustainable construction will have much more significance in developing countries than in advanced countries.

With this background in mind, matters that the Vietnam construction industry should take into consideration in the future can be summarized as in following proposals:

(1) Formulation of an appropriate infrastructure development plan with consideration paid to reducing environmental impacts.
(2) Proactive introduction of advanced construction technology.
(3) Consideration of Kyoto Protocol CDM projects.
(4) Promotion of research and development of construction technology.
(5) Development of a quality assessment tests for construction materials.
(6) Development of design standard specifications for structure and durability.
(7) Education of construction workers.
(8) Introduction of an asset management system.
(9) Creation of an international network for researchers and engineers.

CONCLUDING REMARKS

There is no need to mention that concrete structures have contributed to the social and economic activities of human beings. On the other hand, civil engineering and building structures consume enormous resources and emit huge amount of greenhouse gases. Therefore, there is no question about the extreme importance of introducing effective systems to reduce the environmental impact in the field of construction. Durability problems of concrete structures are directly linked to environmental impacts because the shortening of the lifespan will result in the wasteful utilization of limited natural resources. Thus, it will become more and more important to modify the framework of design of concrete structures by incorporating environmental aspects into the current design systems and to improve durability design methods. We are now in the turning point in the history of concrete engineering.

It is believed that there are quite a few things that the industry, academia and government of Japan can contribute to the development of Vietnam's infrastructure. We hope that this joint seminar will trigger the promotion of the national land development of Vietnam through cooperation between our two countries with a focus on global environmental issues.

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Table 1 Recommended limiting values for composition and properties of concrete 〔EN206〕

	Exposure classes																	
	No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environments		
						Sea water			Chloride other than sea water									
	X0	XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XF1	XF2	XF3	XF4	XA1	XA2	XA3
Maximum *w/c*		0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m^3)	—	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	—	—	—	—	—	—	—	—	—	—	—	—	4.0[1)]	4.0[1)]	4.0[1)]	—	—	—
Other requirements												Freeze/thraw resisting aggregates in accordance with the recommendations in prEN 12620:1996					Surfate-resisting cement[2)]	

1) Where the concrete is not air entrained, the performance of concrete should be tested according to ISO FFF-1 in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven.
2) When SO_4 leads to exposure classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate-resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulfate-resisting cement should be used exposure class XA3.

Table 2 Requirements for special exposure conditions [ACI318-02]

Exposure condition	Maximum water-cementitious materials ratio, by weight, normalweight aggregate concrete	Minimum f_c', normalweight and lightweight aggregate concrete, psi
Concrete intended to have low permeability when exposed to water	0.50	4000
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	4500
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources.	0.40	5000

TABLE 3—Requirements for concrete exposed to sulfate-containing solutions [ACI318-02]

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water , ppm	Cement type	Maximum water-cementitious materials ratio, by weight, normalweight aggregate concrete	Minimum f_c', normal-weighty and lightweight aggregate concrete, psi
Negligible	$0.00 \leq SO_4 < 0.10$	$0 \leq SO_4 < 150$	—	—	—
Moderate	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 < 1500$	II,IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4000
Severe	$0.20 \leq SO_4 < 2.00$	$1500 \leq SO_4 < 10{,}000$	V	0.45	4500
Very severe	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzolan	0.45	4500

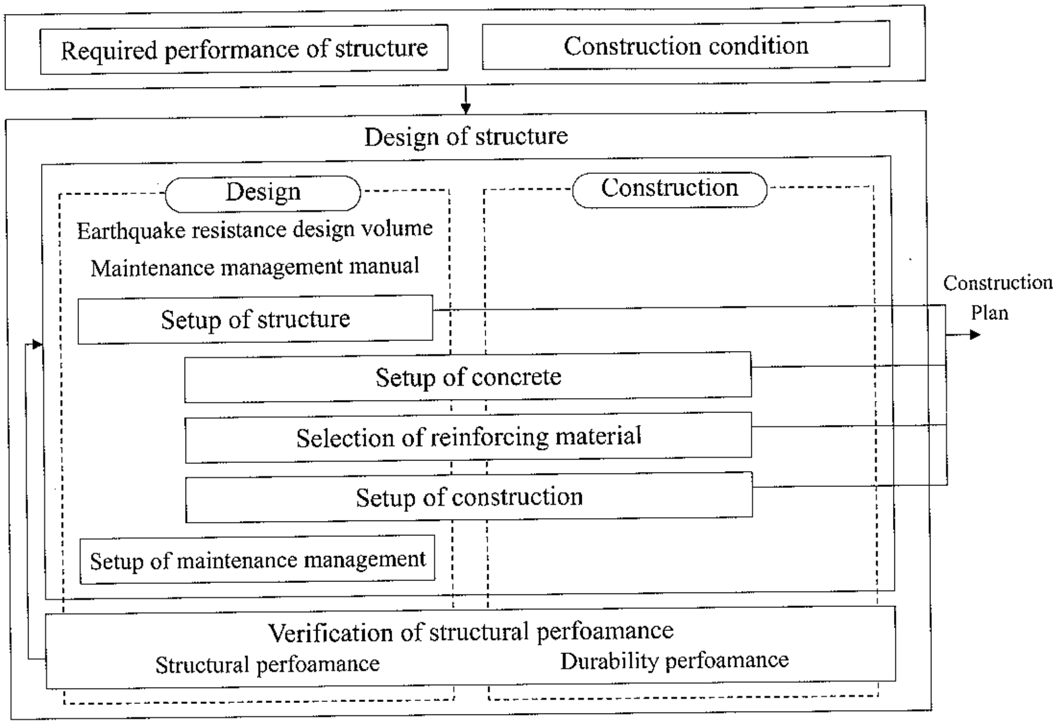


Figure 1 Works at design stage

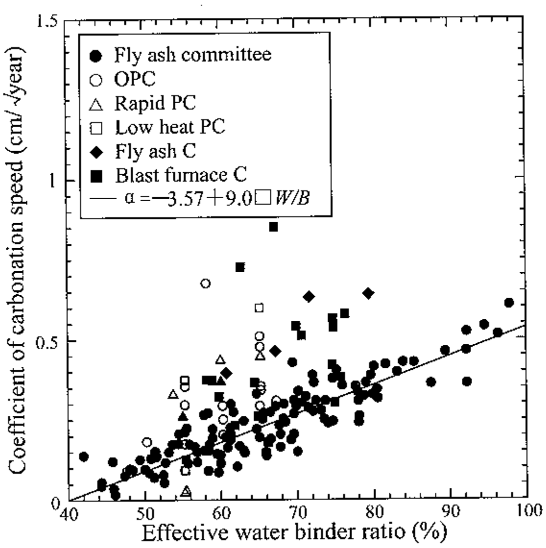


Figure 2 Relation between water binder ratio and carbonation speed

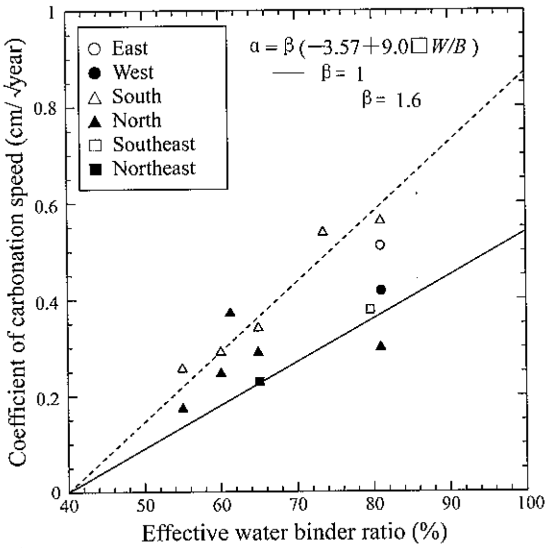


Figure 3 Relation between environment and carbonation speed

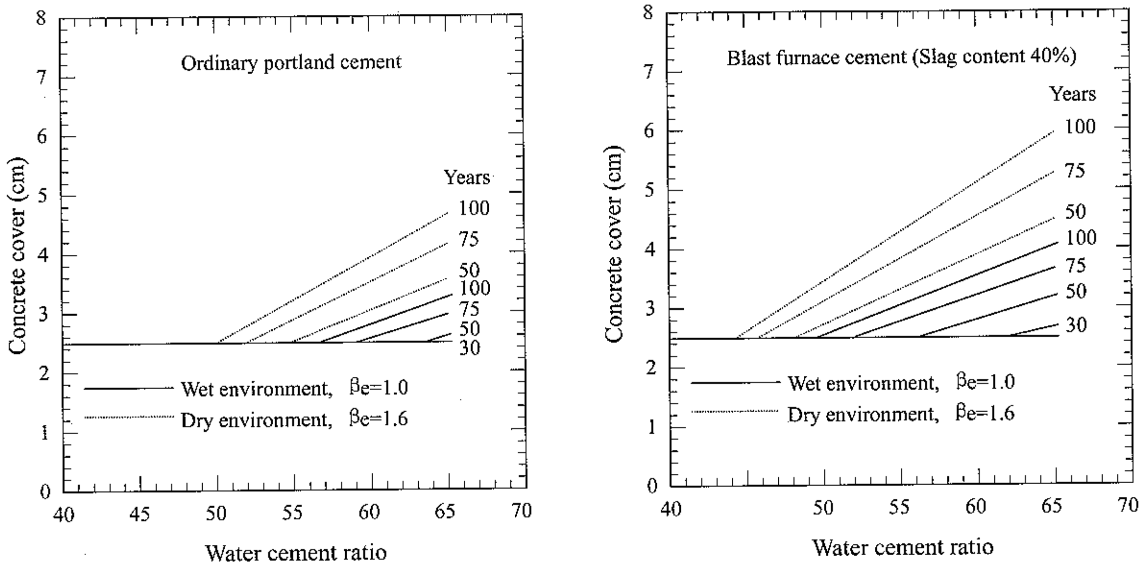


Figure 4 Minimum cover at different water cement ratio and service years

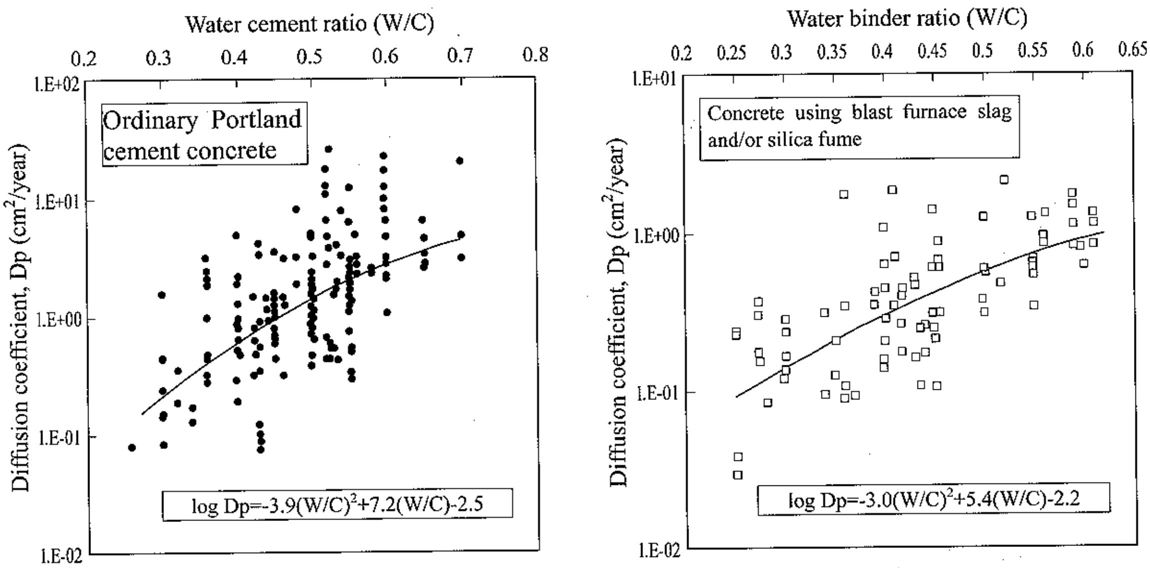
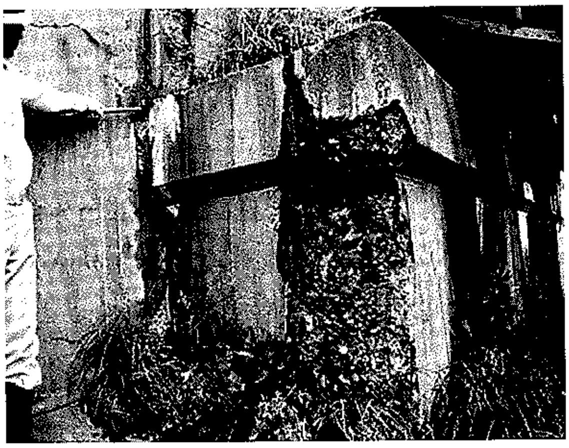


Figure 5 Diffusion coefficient

Photo 1 Damage due to alkali-aggregate



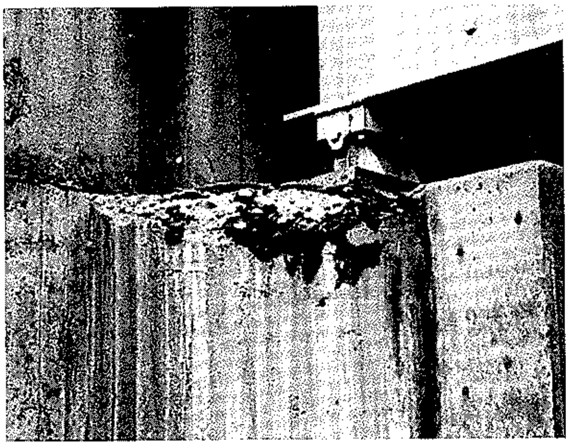
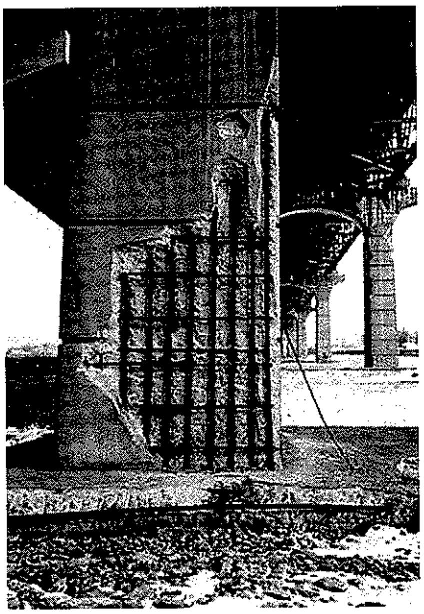


Photo 2 Frost damage



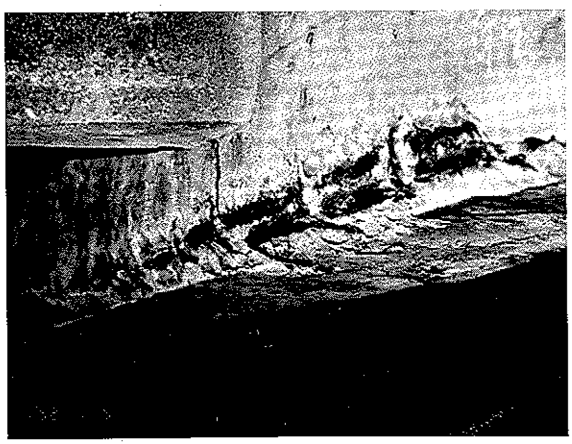


Photo 3 Corrosion of reinforcing bars

SOME PECULIARITIES OF CONCRETE TECHNOLOGY IN THE CONDITIONS OF HOT HUMID CLIMATE OF VIETNAM

Nguyen Tien DICH [1]

SUMMARY

The paper presents some peculiarities of concrete technology in the conditions of Vietnam Hot Humid Climate and working state of concrete structures situated in the local climatic conditions. Some results of the study of the author and colleagues during about 20 recent years in the fields of concrete and concrete technology (concreting, processes of concrete hardening, curing, and deformation of concrete and concrete structures) in the climatic conditions of Vietnam are introduced in the paper.

PECULIARITY OF THE PROCESS OF CONCRETE HARDENING IN THE CONDITIONS OF HOT HUMID CLIMATE

During the process of concrete hardening under the action of Hot and Humid Climate (HHC) there have a series of happened physical processes, such as: processes of water loss and plastic deformation, formation of porous structure and initial structure of concrete, appearance of surface cracks and so on [1-4]. Hereinafter is introduction of some the results of studying them in Vietnam

Process of water loss of concrete

Studying shows that under the action of HHC conditions (such as: solar radiation, wind, rain, air temperature and humidity) concrete loses its mixing water right after finishing the surface of concrete structures (Figure1). The process of water loss of concrete has the following characteristics:

- Beginning water loss right after finishing of surface of structures
- Loss of 40-60 % initial mixing water after 4-6 h of concrete hardening
- Duration of water loss is 4-6 first days of hardening (Table 1)
- Formation of porous structure (Figure 2)
- Causing plastic deformation of concrete

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The factors that influence on the process of water loss of concrete may be listed as intensity of solar radiation, air humidity, wind velocity, climatic season and so on.
The study shows that concrete may loses 60-63% of initial water at the day having strong sun light, while at the day of low sun light- loses only 29-52% of water.

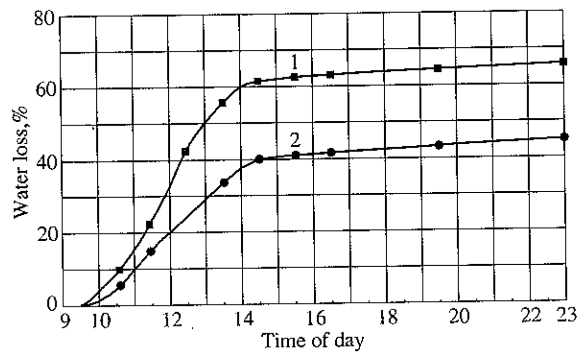
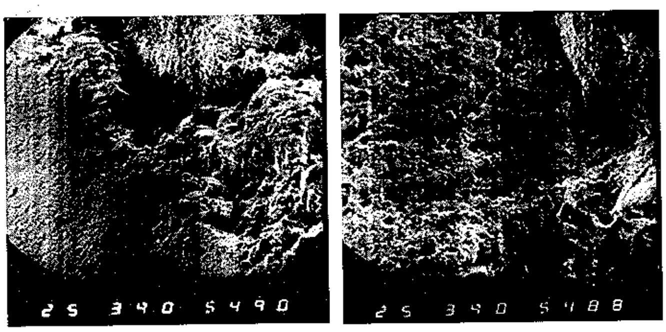


Figure 1- Process of water loss of concrete

Table 1 Duration of water loss of concrete in HHC

N/X	Water loss of concrete after drying days under sunlight								Mh
	1	2	3	4	5	6	7	8	
0.65	60.7	73.3	74.0	76.1	77.4	77.8	78.0	78.5	$25m^{-1}$
0.55	50.8	59.5	62.2	64.6	64.6	68.1	68.6	66.2	
0.45	46.8	50.7	51.3	52.5	52.6	52.9	52.9	55.8	



a) Without moist curing b) With moist curing
Figure 2 Porous structure of concrete due to water loss

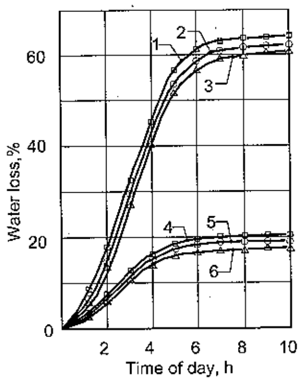
At the day in summer with air humidity 40-60% concrete (with W/C= 0.37-0.54) loses 40-60% of water depending of W/C ratio, while with the air humidity 86-93%- loses only 32-37% of water.

Figure 3 shows that after 6h of hardening at the day in summer concrete loses 40-60% of initial water, while in winter- loses only about 20% of water. It may be explained that the high solar radiation and air temperature in summer strengthened process of water loss of concrete. It means that covering concrete surface for prevention of water evaporation is necessary in summer.

Process of plastic deformation

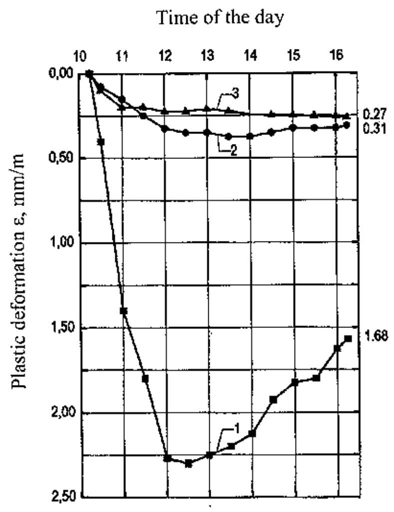
Plastic deformation is the process of volume change of concrete (shrinkage or expansion) when it has no strength or is being in plastic state.

Water loss process causes a negative pressure in concrete body that makes changing its volume. In the HHC conditions the volume change of concrete consists of plastic shrinkage and expansion (Figure 4).



1,2,3- In summer, W/C = 0,57; 0,46; 0,37
4,5,6- In winter, W/C = 0,76; 0,61; 0,46

Figure 3 Water loss of concrete according to time of seasons



1 - Specimens with $M_{exp} = 30^{m-1}$.
2 - Specimens with $M_{exp} = 0$.
3- Specimens in the room conditions. ($t = 31\ ^0C$, $\varphi = 98 \div 100\%$, $M_{exp} = 30^{m-1}$).
The weather: $t = 33 \div 35\ ^0C$.
$\varphi = 62 \div 75\%$;
$v = 0.1$ m/s
$Q_{max} = 762$ kcal/m^2.h.

Figure 4 The change of plastic deformation of concrete at time of day

The process of plastic deformation of concrete in the conditions of HHC has the following characteristics:

- It take place right at the first minutes of concrete hardening;
- Including shrinkage and expansion;
- Maximum value of plastic shrinkage may reach 3-4mm/m, and plastic expansion- 0.5-0.8mm/m depending on climatic conditions;

- Process of plastic deformation consists of 5 periods: Beginning of shrinkage; rapid shrinkage; slow shrinkage; expansion and then stabilization.
- Plastic deformation may cause some surface cracks in concrete structures (Figure 5).

Although plastic deformation takes place only for some first hours of concrete hardening it may reduce its remarkable value of compressive strength: After 4 hours situated under sun light at a day in summer concrete loses 20-30% compressive strength at the age of 28 days (R_{28}), depending on maximum value of plastic deformation ε_{max} (Table 2).
The above aspects show that curing concrete right after concreting in the conditions of HHC is very importance.

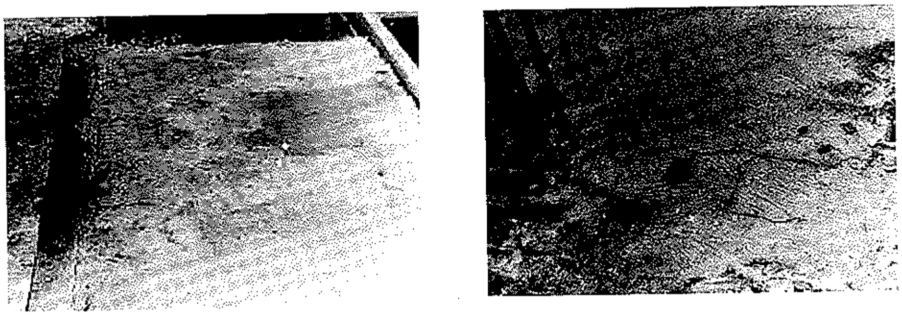


Figure 5 Surface cracks of concrete structures due to plastic deformation

Table 2 Influence of plastic deformation on 28 days' strength, R_{28}, of concrete

W/C	After 2.25h under sun light			After 4h under sun light		
	ε_{max}, mm/m	Water loss W, %	Compressive strength, % R_{28}	ε_{max}, mm/m	Water loss W, %	Compressive strength, % R_{28}
0.750	0.73	46	137	0.60	57	80
0.625	1.20	30	100	0.80	59	75
0.460	2.90	29	77.7	2.60	53	70

Moist curing of concrete in HHC conditions

Moist curing of concrete is an importance stage of concreting process in Vietnam. Without it concrete may lose a part of strength at the age of 28 days. Table 3 shows that in summer, concrete may lose 9-35% of its compressive strength at the age of 28 days, depending on W/C ratio and Module of exposure M_{exp} of concrete [1].

Table 3 The loss of compressive strength of concrete on missing of moist curing

W/C	Mexp, m^{-1}	Loss of compressive strength of concrete at the age of 28 days on missing of moist curing, % of the designed strength		
		In May	In June	In July
0.68	10	9	13	12
	30	14	21	16
0.55	10	-	16	14
	30	23	33	27
0.44	10	-	15	15
	30	-	35	29

Basis of moist curing process

Moist curing process consists of 2 periods: The initial and continuous periods.

The initial period is the duration of time of caring concrete by covering with some wet material or an membrane for preventing water of concrete from evaporation to atmosphere. The duration of this period is about 4-7h depending on the seasons in concreting area.

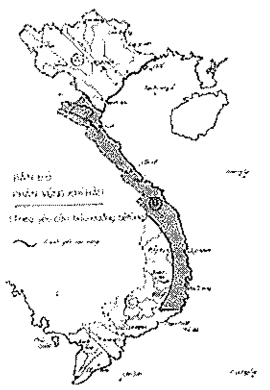
The continuous period is begun right after finishing of initial one and prolongs for some first days of concrete hardening. During this period concrete surface should be kept in wet state by continuously sprinkling water on it. The continuous period is characterized by the so call **Critical strength** and **Essential time** of curing. These are the key parameters of moist curing process. The study of curing concrete in Vietnam was made by author and colleagues for a long time for determination of those parameters. And then a map of curing of concrete was established.

Zone map of curing for concrete

Since the weather is different for seasons and areas of the country the study of curing processes was made in various areas and seasons for some years from the North to the South of Vietnam. After treating data of the study the critical strength and the essential time were grouped into 3 zones of curing (A, B and C) according to characters of geography location. The zone map of curing was established for concrete technology in Vietnam. Every zone is characterized by two seasons with various values of Rcr and Tess (Table 4). From the Table 4 we can see that the Critical strength and the Essential time are increased according to the seasons Summer and Dry from the North to the South (50-55%; 55-60%; 70%) and (3; 4; 6 days) correlatively. On the contrary, they are decreased according to the seasons Winter and Wet from the North to the South of Vietnam. Practices in Vietnam show that the zone map of curing is good suitable to climatic conditions in Vietnam.

The results of study were used for development of the Vietnamese standard TCVN 5592:1991[5]. This standard provides the requirements of moist curing concrete according to Rcr and Tess for every season of 3 zones of curing.

Table 4 Characteristics of zone map of curing according to Critical strength and the Essential time of concrete

Zone map of curing	**Zones**	**Seasons**	**Months**	**Rcr, %R_{28d}**	**Tess, days**
	A	Summer Winter	IV – IX X - III	50-55 40-45	3 4
	B	Dry Wet	II – VII VIII - I	55-60 35-40	4 2
	C	Dry Wet	V – XI XII - IV	70 30	6 1

Concreting of massive concrete structures in HHC

Conditions for considering a massive structure

There are two following parameters used for considering a structure as a massive one in the conditions of HHC of Vietnam [6]:

- The concrete height of structure $h > 2m$
- The minimum dimension of structure $a_{min} > 2m$

Conditions causing cracks in mass concrete

There are two following parameters that are considered as conditions of causing cracks in mass concrete due to effect of cement hydration process:

- The temperature differential between parts of concrete structure $\Delta T > 20^oC$. This is the necessary condition.
- The Module of temperature differential of concrete structure $M_T \geq 50^oC/m$. This is the sufficient condition.

Without the condition $\Delta T > 20^oC$ cracks will not be occurred in concrete body. But with only one this condition concrete may have or have not cracks.

With both the two conditions cracks must be occurred in concrete body.

M_T is the temperature differential between two points in concrete situated in a distance of 1m.

$$M_T = \Delta T / a$$

Where a is the distance of the two points in concrete body, m.

In critical conditions we have:

$$M_T = 20/a = 50$$
$$a = 20/50 = 0.4m$$

It means that in massive concrete structures only 0.4m around concrete body runs the risk of being cracked. The inside part of concrete is considered as a safety one called a safety core.

By a special software we can make a picture of safety core of mass concrete as showed in Figure 6.

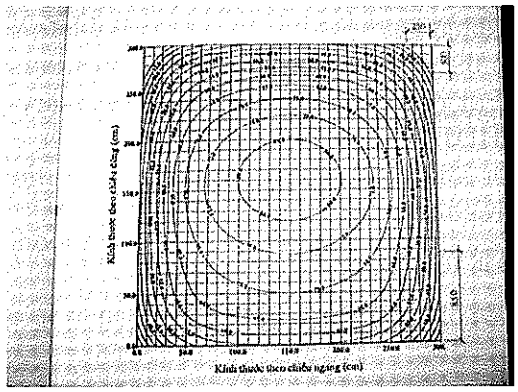


Figure 6 Safety core of mass concrete

Method of preventing cracks

The method of preventing cracks in mass concrete is to do some measures so that the ΔT is allays less than 20°C.

Some measures are as follows:

- Reducing the heat generated by cement, such as:
  - To limit the rate of heat generation of cement in concrete;
  - To use low heat cement;
  - To reduce temperature of fresh concrete (to lower temperature of concrete Materials and water, and to cover fresh concrete from solar radiation)
- Limiting ΔT, such as:
  - To cover concrete body by heat insulation materials;
  - To transfer the heat of mass concrete from interior to surround atmosphere;
  - To devise concrete body into some smaller parts that will not be a mass concrete.

The national standard TCXDVN 305: 2004

The standard [7] was developed by the author consists of following main parts:

- Conditions to be called as a mass concrete structure;
- Conditions causing cracks in concrete body;
- Measures for prevention of cracks in concrete;
- Requirements of design and construction/concreting of massive concrete structures.

Deformation of concrete structures in HHC of Vietnam

Features causing cracks in concrete structures

The figure 7 shows the continue deformation of a concrete roof for one year in the conditions of HHC. The Δ on the figure is the value of deformation restrained by foundation layer which causes tensile stress in concrete structures. If tensile stress is greater than tensile strength of concrete, it cracks.

The results of author's study show the following sutuations:

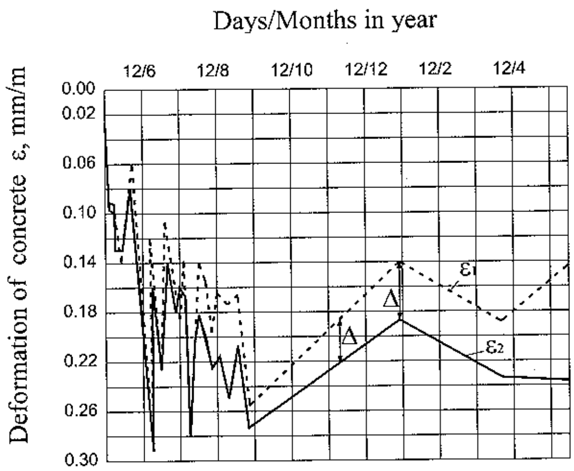
if $\Delta < 0{,}1$mm/m, concrete does not crack;

if $\Delta = 0{,}1 \div 0{,}2$mm/m, concrete may crack or not depending on properties of concrete and its drying rate;

if $\Delta > 0{,}2$mm/m, concrete cracks.

In the conditions of HHC of Vietnam cracks due to restrained deformation often appear in rather long structures and in the structures having a composition or curved form restraining their deformation (Figure 8). For prevention cracks, it is necessary to place some so call Hot Humid Deformation Joints- HHDJ.

The results of study of author and colleagues allow to define the spacing and appropriate places of HHDJ in concrete structures.



ε_1- Restrained deformation of concrete roof cemented with foundation layer.

ε_2- Free deformation of concrete roof.

Δ- Value of deformation restrained by foundation layer.

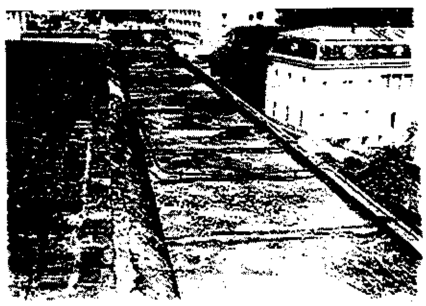
Figure 7 Deformation of concrete roof in HHC conditions

Placing HHDJ for structures

There are two types of HHDJ: Expansion and Contraction Joints. Spacing of these joints are defined as follows:

* *For expansion joints:*

L_{max} = 6m to 9m spacing for open-air structures without reinforcement or having only constructive reinforcement, that are situated under the direct action of hot and humid climatic conditions (such as watertight concrete layer on the roof; road pavements; platforms,..);



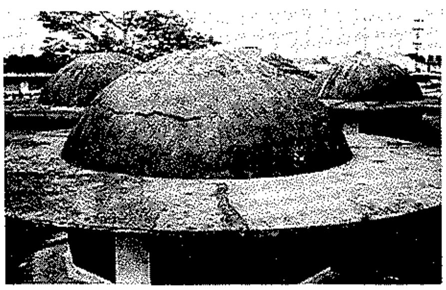


Figure 7 Cracks in concrete structures situated under action of HHC

L_{max} = 9m to 18m spacing for structures having no reinforcement or having only constructive reinforcement that are protected from direct sunlight;

L_{max} = 35m spacing for reinforced concrete structures that are situated under direct sunlight (such as concrete roof; external walls);

L_{max} = 50m spacing for reinforced concrete structures that are protected from direct sunlight (such as concrete roof having heat protective layer; internal walls).

* *For contraction joints:*

lmax = 6m to 9m spacing for any reinforced concrete structures being under the direct action of climatic conditions;

lmax = half of cupola or dome height for structures with form of small cupola or dome being under the direct action of climatic conditions.

Figure 8 shows an example of placing of HHDJ in a concrete road wall in Hanoi with L_{max} = 30m and l_{max} = 9-10m.

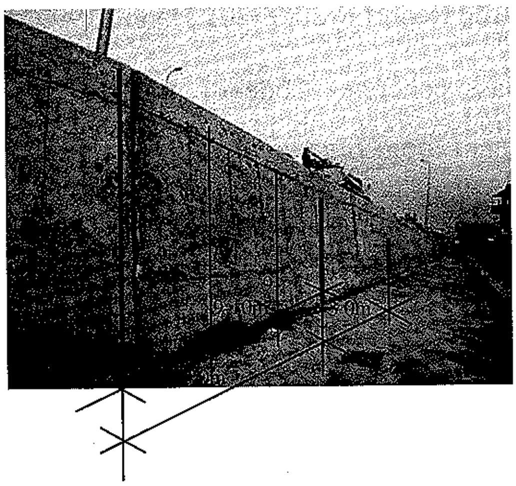


Figure 8 Placing expansion and contraction joints to a concrete road wall in Hanoi

The National standard TCXDVN 313: 2004

The results of author's study in the field of prevention of cracks in concrete structures working under action of HHC were used for development of the National standard TCXDVN 313:2004 [8] which consists of the following main aspects:

- Prevention of surface cracks of concrete structures;
- Prevention of structural cracks of concrete; and
- Placing of HHDJ in concrete structures.

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ROLLER COMPACTED CONCRETE DAM AND UTILIZATION OF FLY ASH IN JAPAN

Kimitaka UJI[1]

SUMMARY

RCC Dams have been constructed in order to reduce the construction period and construction cost due to the decrease of cementitious materials. Fly ash is important material for RCC Dam, and usually replaced from 20 to 30 % for cement. But, in other countries, replacement ratio more than 50 % is adopted. Also, it becomes difficult to use the fly ash with adequate qualities. In this paper, the properties of RCD in Japan is described compared with RCC Dam in other countries, and the standard specification of JIS for fly ash and the effect of fly ash out of standards on concrete properties is discussed.

Keywords: *Roller compacted concrete dam; RCD; fly ash; cementitious material; cement content; replacement ratio of fly ash; ignition loss; characteristics of fly ash concrete.*

INTRODUCTION

More than 250 RCC Dams have been constructed in 40 countries. And the dam volume becomes large steadily based on the actual results and the development of construction technique and mix proportions.

In Japan, a term of "RCD (Roller Compacted Dam)" is used for RCC (Roller Compacted Concrete) Dam. RCD have been expected almost same performance as the dam constructed by traditional method for permeability and so on. The purpose of use of RCC method is the reduction of construction period and the reduction of construction cost due to the decrease of cementitious materials. There are some differences between RCC method and RCD methods. For instance, the thickness of 1 lift is 300 mm for RCC Dam and 750-1,000 mm for RCD.

Fly ash is important material for RCC Dam, and usually replaced from 20 to 30 % for cement in Japan. But, replacement ratio more than 50 % is sometimes adopted in other countries. It becomes difficult to use the fly ash with adequate qualities. Therefore, it is necessary to clear the characteristics of concrete including fly ash out of standards.

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In this paper, the features of RCD in Japan are described, and the effects of the 4 fly ash classified to 4 types in Japan Industrial Standard (JIS) and the fly ash out of specification on concrete properties are discussed.

RCC Dam (RCD) in Japan

RCC Dam (in Japan, usually used "RCD") was firstly examined at Ohkawa Dam in 1976, and Shimajigawa Dam was actually constructed by RCD method in 1980, following Tamagawa Dam in 1987. At those times, the cement content was large compared with Dams of other countries.

Willow Creek Dam with 52m-hight was constructed in 1982 by RCC method in USA. Cement of 47 kg and fly ash of 19 kg, total cementitious materials: 66 kg, were used for 1 m^3. But, large leak of water occurred, so that cement grouting was adopted. Copperfield Dam with height of 40 m was constructed in Australia (1984). Binder of 110 kg (Cement: 80 kg, fly ash: 30 kg) was used for surface concrete, and a little leak was observed. On the other hand, Upper Stillwater Dam was constructed in 1987, using concrete with cement content of 252 kg/m^3.

Table 1 shows the volumes and heights of about 150 Dams constructed by RCC method from 1981 to 1999. Average volume of dam except the data of Japan is 200,000 m^3, average height is 46 m, and average volume of RCC is 150,000 m^3. Then, in USA having 23 Dams, those values are 140,000 m^3, 36 m, 130,000 m^3, respectively. From these data, it can be seen that RCC Dams in USA are comparatively small. Maximum RCC dam in USA is Upper Stillwater Dam, which has volume of 1,280,000 m^3, height of 91 m, and RCC volume of 1,130,000 m^3. And the volume of RCC Dams in Spain is similar to that in USA.

In Japan, average dam volume is 550,000 m^3, average height is 87 m, and average RCC volume is 350,000 m^3. The volume of RCD Dam is larger than the average volume of other countries. Maximum dam volume is 1,930,000 m^3 of Miyagase Dam (completion in 1997), and the height is 156 m of Urayama Dam (completion in 1999) and Miyagase Dam. Scale of RCC in China is similar to that of Japan.

Typical RCD Dams in Japan are mentioned in Table 2. Shimajigawa Dam of which height is 89 m has volume of 317,000 m^3 and RCC volume of 165,000 m^3. Tamagawa Dam has a height of 100 m, dam volume and RCC volume are 1,090,000 m3 and 750,000 m^3, respectively.

At the end of 2002 year, 251 RCC Dams were completed in 39 countries, the number of RCC Dams are China: 45, Japan: 42, USA: 36, Brazil: 29, Spain: 21. Average height in recent years is 80 m and dam volume is 600,000 m^3. Recently, RCC Dam becomes large compared with RCC Dam at the beginning. Also, Sankyo Dam has being constructed in China. This project started in 1993 and will be completed in 2009. The concrete volume will be 15,000,000 m^3.

The change of cementitious materials content for RCC Dam is shown in Fig. 1. At 1950', cementitious material content of 180 kg/m^3 was used for block construction method, i.e., traditional concrete gravity dam. Cementitious materials content decreases 100 kg/m^3 for 60 years due to the improvement of technique on quality control and construction machine. Consequently, in RCC Dam, total volume of cementitious materials became about 120 kg/m^3.

At the beginning of construction by RCC method, lean mixture was adopted, but at present, rich mixture becomes sometimes to be used. Table 3 shows the normal mix proportions. Rate of rich mixture RCC (binder: 150kg/m^3 <) is 47.4%, and the rate of fly ash to cement (F/C) is 0.53. Medium mixture (100-150 kg/m^3) and lean mixture (<100 kg/m^3) of RCC are 18.6% and 16.1 %, respectively. The rate of RCD is 15.4 % and F/C is 0.30. F/C of rich mixture RCC is high, and cement is 91 kg/m^3, fly ash is 103 kg/m^3. In RCD concrete, cement content is 86 kg/m^3, and fly ash is 37 kg/m^3. Both cement contents of rich mixture RCC and RCD concrete are similar each other.

The thickness of each lift for compaction of RCD concrete differs from that of RCC Dam. In Table 4 and Fig. 2, it is obvious that the 1 lift of RCC Dam is mainly 300 mm and that of RCD is from 750 to 1,000 mm. Mixed concrete are spread with bulldozer for 3 or 4 thin layers, then they are compacted about 10 times by vibratory roller. Shimajigawa Dam and others, which were constructed in 1980', have a thickness of 500 mm for 1 lift. The thickness of 1 lift becomes 750 mm in 1990' and recently 1,000 mm on the economical viewpoint.

Fig. 3 shows the relationship between the heights of RCD Dam and the rate of monthly average construction height. Rate of construction of RCD Dam, which is adopted in Japan, is slow in comparison with RCC Dam method. But, in cases of Dams having volume more than 190,000 m^3 in Fig. 4, the rate of construction by RCD method is similar to that of RCC method in USA or China.

The sloped layer method shown in Fig. 5 was developed, and employed on the construction of Tannur Dam and Lajeado Dam. This method was enabled to continuously place concrete by adjusting the slope degree in consideration of the capacity of placing machines and the concrete supply.

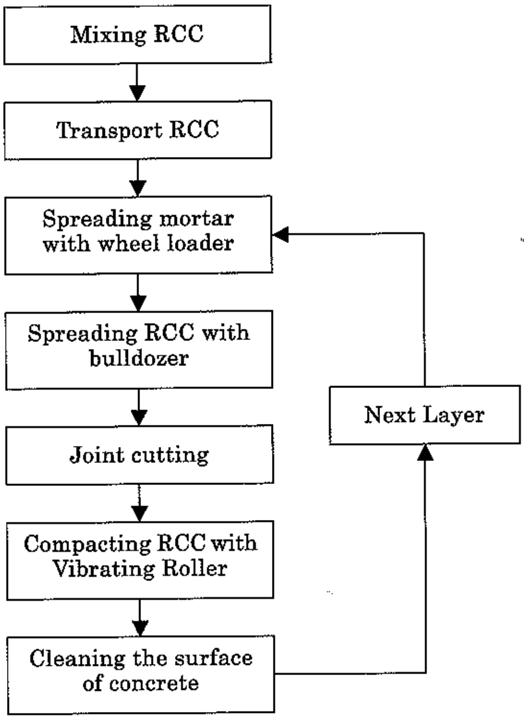
The purpose of use of RCC method is the reduction of construction period and the reduction of construction cost due to less cementitious materials content. Generally, it can be thought that RCD Dam actualizes the reduction of 9 % of construction cost and 13 % of construction period.

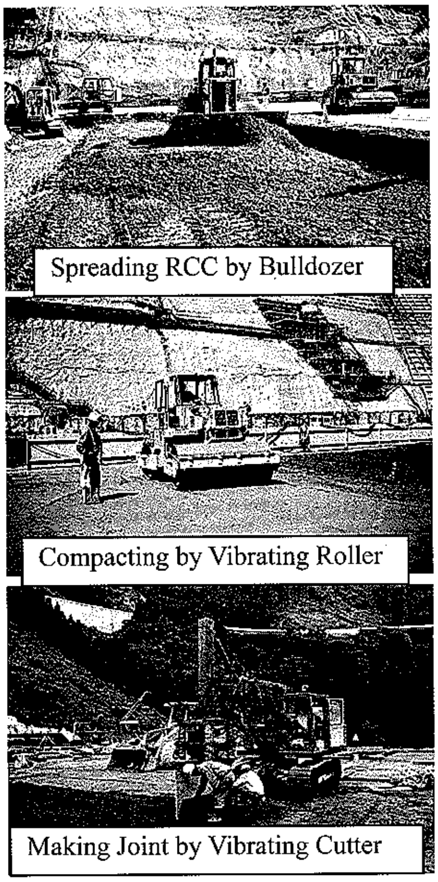
Fig. 6 shows the mix proportions and the position of segmentation for Kazunogawa Dam. For inner concrete, 3 mix proportions were adopted, and the replacement ratio of fly ash was 30 %. Mix proportions are shown in Table 5. The content of cement and fly ash for RCD were from 130 kg/m^3 to 110 kg/m^3. Maximum size of coarse aggregate was 120 mm. Consistency was examined by VC value (Vibrating Compaction Value) using the testing machine shown in Photo 1, and procedure was described below. The VC value was specified to 20 ± 10 sec.

Procedure of VC value;
i) Packing RCD screened by sieve of 40 mm into mold (diameter of 24 cm, height of 20 cm)
ii) Vibrating RCD concrete
iii) Measuring mortar rising time

Comparison of RCC Dam method and traditional method (for gravity dam) is shown in Table 6.

RCC construction process is shown as follows;





Spreading RCC by Bulldozer

Compacting by Vibrating Roller

Making Joint by Vibrating Cutter

Chubetsu Dam is a combination of a concrete gravity dam and rockfill dam of 86.0 m and 78.5 m dam height respectively, as shown in Phote-2 and Figs. 7 and 8. The riverbed around the dam site is very wide at about 600 m, and unconsolidated river deposits are distributed there thickly. As a result of a comparative study on various types of dam and combinations, it was determined to adopt a concrete gravity dam for the foundation with bedrock in a 290 m portion of crest length on the left bank where the thickness of the gravel deposits was as shallow as 5-10 m. A rockfill dam with a center core was chosen for the foundation based on the low permeability of the fully consolidated lower river deposits in the 595 m of crest length on the right bank. Mix proportions for concrete gravity dam constructed by RCD method is shown in Table 7. Maximum size of coarse aggregate was 150 mm. The content of the cement and fly ash for RCD was 120 kg/m3, and replacement ratio of fly ash was 0.30. The VC value of 10$\pm$5 sec was specified.

Utilization of fly ash

Japan Industrial Standard for fly ash was revised in 1999, and 4 types of fly ash was classified in JIS A 6201-1999 for the utilization of coal ash. Table 8 shows the quality of 4 types of fly ash (F I, II, III, IV). Generally, F II is used for cementitious material. F I is used for high quality concrete because of high specific surface and low ignition loss. Quality of F III and F IV is low, but these are specified for environmental protection and effectively use of resource. 4 types of fly ash are classified due to maximum ignition loss and specific surface. Ignition loss of F II is specified to be less than 5%, and minimum specific surface is 2,500 g/cm^2. Maximum of ignition loss of F III is 8.0 %, and minimum specific surface of F IV is 1,500 g/m^2. Chemical ingredients of fly ash are shown in Table 9. Also, the ignition loss of fly ash ranges from less than 1% to 17%.

As shown in Fig. 9, the generation capacity by coal-fired power in Japan is 13 %, and 30 million kw are generated by 37 power plants. As shown in Tables 10-12, production of fly ash in 2003 was 9.9 million tons, including 7.5 million tons from electric power industry and 2.4 million tons from general industry. And 6.1 million tons (81.7%) and 2.3 million tons (95.1%) were used, respectively. Fly ash is used for cement and concrete, civil engineering, building construction, agriculture/forestry and others. 6.33 million tons (76%) were used for cementitious materials.

Because of its spherical particles and pozzolanic effect, fly ash improved from coal ash has the following characteristics for use in concrete.
(a) Long-term strength increase
(b) Low dry shrinkage
(c) Control alkali aggregate reaction
(d) Low heat from hydration
(e) Water-tightness

The properties of concrete including F II type of fly ash are well known. But, there is little data for other types of fly ash. In this paper, the effect of ignition loss on the properties of concrete using fly ash. The values of ignition loss range to the extent of 17%. The relationship between pozzolanic activity index and ignition loss is shown in Fig. 10. Strength development of mortar containing the fly ash with high ignition loss does not differ from mortar using F II type fly ash. On the other hand, as shown in Fig. 11, flow value ratios decrease with increasing of ignition loss, therefore, workability of concrete with high ignition loss may lower. Fig. 12 shows the relationship between ignition loss and the required content of chemical admixture. Increase of ignition loss affects the quantity of chemical admixture for adequate workability.

Relationship between compressive strength and Young's modulus is shown in Fig. 13. Concrete has 9 kinds of fly ash with different ignition loss and replacement ratio from 0 to 45 %. Compressive strength has a proper relation with Young's modulus. Fig. 14 shows the relation between compressive strength and tensile strength under the conditions of 5 kinds of fly ash and replacement of ratio from 0 to 30 %. It can be thought that the ignition loss rarely affect the properties of hardened concrete.

CONCLUDING REMARKS

RCC Dam method was adopted from 1980, and the number of RCC Dams is increasing steadily. The reduction of construction cost can be achieved by RCC method, because of the shortening of construction period and the reduction of cementitious materials content. Cementitious materials content in RCC method decreases 100 kg/m3 compared with traditional method due to improvement of technique on quality control and construction machine. Also, the reduction of cement content is effective for prevention of thermal cracking. Thickness and velocity of lift for RCD differ from RCC Dam method. For instance, thickness of lift is 750-1,000mm for RCD, and that of RCC Dam is 300mm. Rational and economical construction method must be further investigated.

Utilization of fly ash is very important for effective use of resource. Fly ash with high ignition loss rarely affects the properties of hardened concrete, but the increase of chemical admixture for the adequate workability of fresh concrete. More investigation is desirable for fly ash with low quality.

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Table 1 Volume and height and RCC Dams (1981-1999)

National	Number of Data		Volume of Dam(m^3)	Volume of RCC(m^3)	Height of Dam (m)	Crest Length (m)
USA	23	Ave.	142,130	128,304	36	295
		Max.	1,281,000 (Upper Stillwater Dam,1987)	1,125,000 (Upper Stillwater Dam,1987)	91	815
		Min.	3,000	3,000	17	46
China	34	Ave.	468,088	303,353	71	345
		Max.	1,970,000 (Guanyinge Dam,1995)	1,240,000 (Guanyinge Dam,1990)	128	1,040
		Min.	23,000	17,000	25	93
Spain	21	Ave.	165,571	148,000	47	273
		Max.	1,016,000	980,000	99	835
		Min.	5,000	4,000	16	75
Average	116	Ave.	204,972	151,960	46	285
Japan	30	Ave.	552,490	345,000	87	304
		Max.	1,927,300 (Miyagase Dam,1997)	1,341,515 (Urayama Dam,1995)	156	445
		Min.	193,579	97,255	48	218

Table 2 Typical RCC Dam (RCD) in Japan

Dam	Volume of Dam (m^3)	Volume of RCC (m^3)	Height of Dam (m)	Completion Year
Shimajigawa	317,000	165,000	89	1981.3
Tamagawa	1,092,000	754,450	100	1990.12
Miyagase	1,927,300	1,303,844	156	1997.3
Urayama	1,709,387	1,341,515	156	199.3
Kazunogawa	622,000	315,000	105	2001.3
Chubetsu	777,000	336,000	86	2005

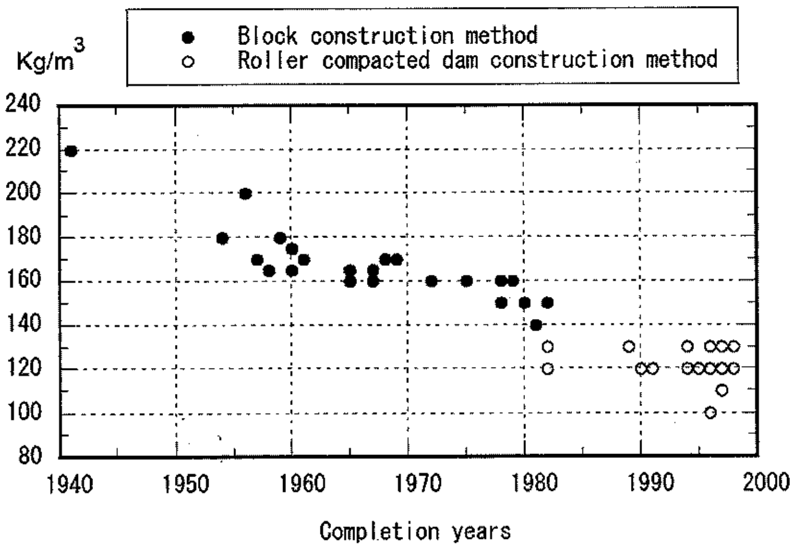

Fig. 1 Change of cementitious materials content

Table 3 Mix proportions for RCC Dam

Mix Proportion	Binder	Rate of use	C(kg)	F(kg)	W(kg)	F/(C+F)	W/B
Rich mixture RCC	150kg<	47.4%	91	103	107	0.53	0.55
Medium mixture RCC	100-150kg	18.6%	72	44	116	0.38	1.00
Lean mixture RCC	<100kg	16.1%	69	11	123	0.14	1.54
RCD		15.4%	86	37	94	0.30	0.76
Others		1.4%					

Table 4 Height of lift for RCC Dam

Mixer	Revolving-paddle	40.5%
	Tilting	20.7%
	Continuous	26.5%
Height of Lift	<250mm	5.3%
	300mm(Normal)	64.4%
	350-500mm	5.3%
	500-1000mm(RCD)	18.1%
	1000mm<	6.8%

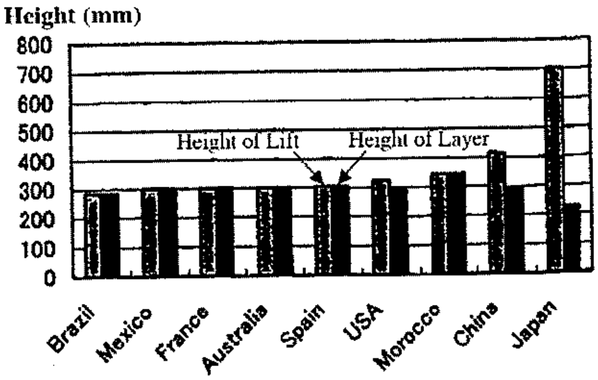

Fig. 2 Height of lift and layer

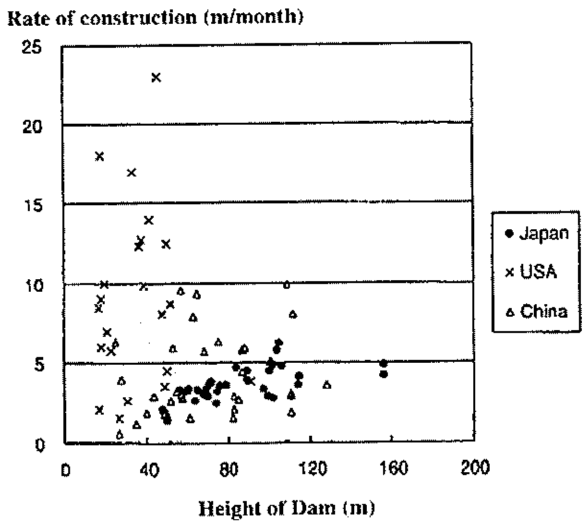

Fig. 3 Relation between the rate of construction and height of dam

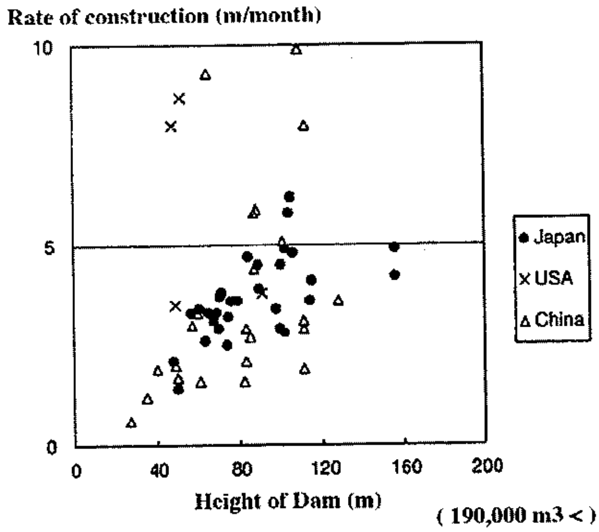

Fig. 4 Relation between the rate of construction and height of dam (more than 190,000 m^3)

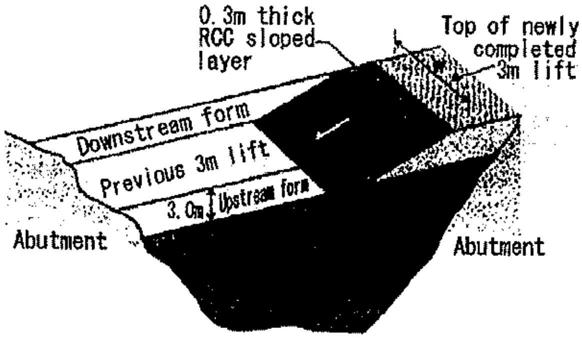

Fig. 5 Sloped layer method

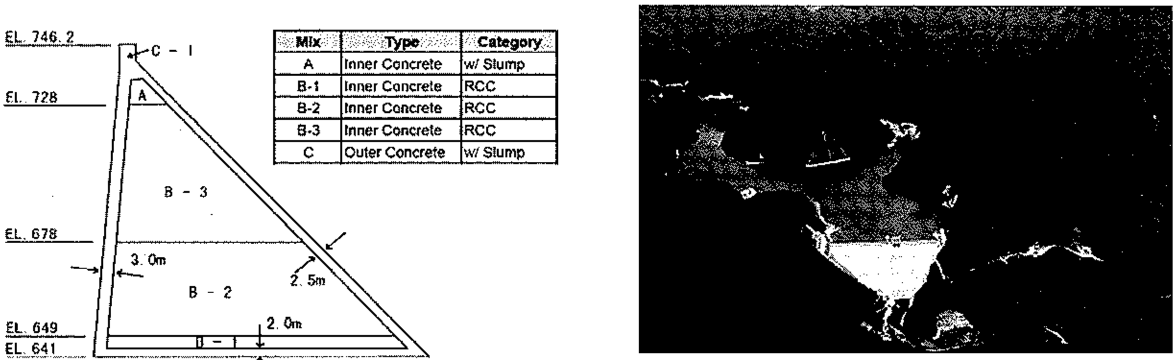

Fig. 6 Segmention of Kazunogawa Dam

Table 5 Mix proportion for Kazunogawa Dam

Type of Concrete	Mix No.	Max Size of Coarse Aggregate (mm)	VC-Value (sec) Slump Value (cm)	Air (%)	W/(C+F) (%)	FA Replacement F/(C+F) (%)	Fine Percent s/a (%)
RCC	B-1	120	20±10	1.5±1	69.2	30	28
	B-2				75.0		
	B-3				82.7		
Slump Concrete	A	120	3±1	3±1	71.4	30	25
	C-1				49.0		22

	Unit Weight(kg/m^3)						
	W	C	F	C+F	S	G	Ad(%)
B-1	90	91	39	130	629	1648	0.325
B-2	90	84	36	120	632	1652	0.300
B-3	91	77	33	110	634	1660	0.275
A	100	98	42	140	546	1683	0.350
C-1	98	140	60	200	469	1707	0.500

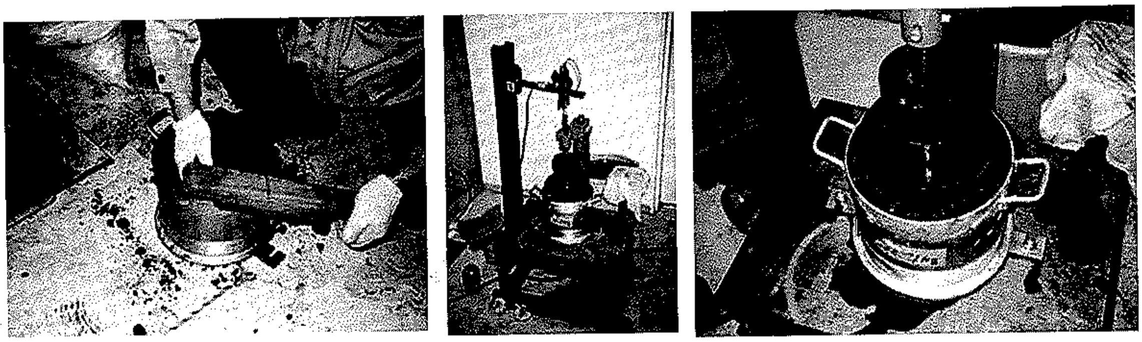

Photo 1 VC value testing machine

Table 6 Comparison of traditional method and RCC method

Items	Traditional Method	RCC Method
Concrete Proportion	C=160kg/m^3 Slump=4cm	C=120kg/m^3 Slump=0cm VC value=20±10sec
Transport	Crane	Crane, Dump Truck
Placing	Block Placing Lift Height=1.5m Width=15m	Layer Placing Lift Height=0.75m The area is around 3,000m^2
Spreading	Direct Flow Out	Bulldozer
Compacting	Stick Vibrator	Vibrating Roller
Placing interval	6 days	3 days

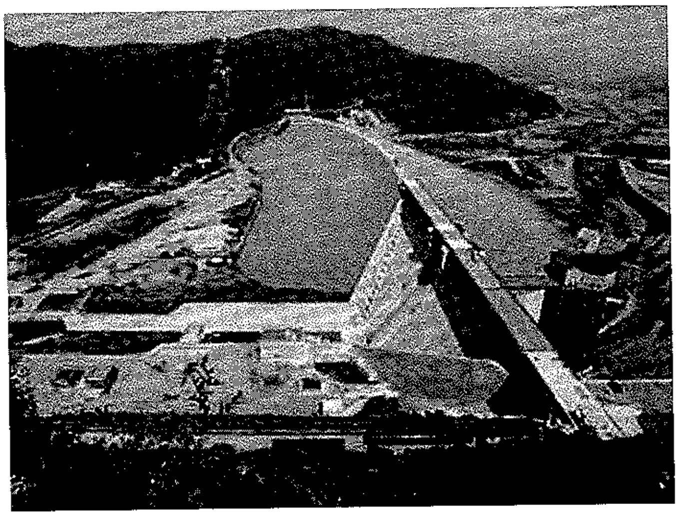

Photo 2 Chubetsu Dam

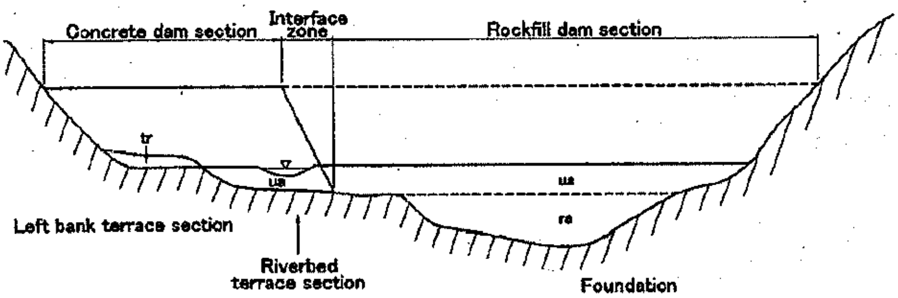

Fig. 7 Conceptual arrangement of composite type dam

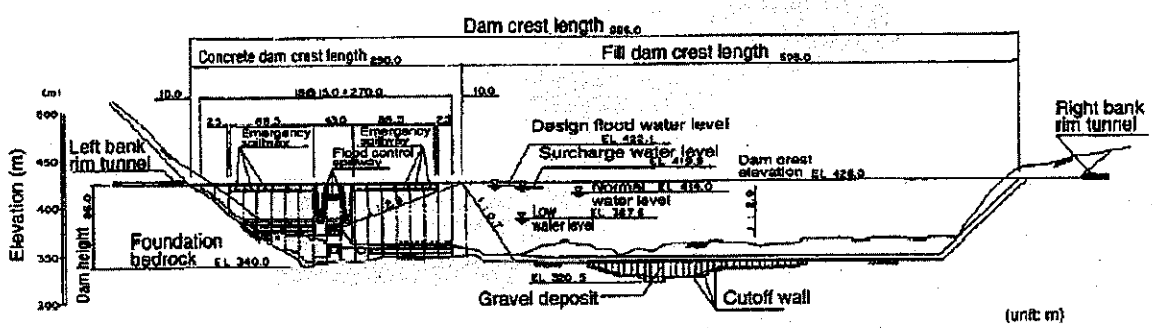

Fig. 8 Upstream face of the dam

Table 7 Mix proportions for Chubetsu dam

	Gmax	Slump VC	Air (%)	W/(C+F) (%)	s/a (%)	Unit weight(kg/m^3) W	C+F	S	G	Ad
Outside	150	3±1 cm	3±1	43.2	24	95	220	499	1575	0.55
RCD	150	10±5sec	1.5±1	63.3	28	76	120	632	1625	0.30
RC	40	8±2.5 cm	5±1	50.7	43	142	280	793	1056	0.70

Table 8 Specifications for fly ash

Properties	FA I	FA II	FA III	FA IV
Min Silicon Dioxide(%)	45			
Max Moisture Content(%)	1.0			
Max Ignition Loss(%)	3.0	5.0	8.0	5.0
Min Density(g/cm^3)	1.95			
Fineness				
Passing 45μm(%)	10	40	40	70
Specific Surface(g/cm^2)	5,000	2,500	2,500	1,500
Flow Value Ratio	105	95	85	75
Pozzolanic Activity Index				
28days(%)	90	80	80	60
91days(%)	100	90	90	70

Table 9　Chemical ingredients of fly ash

	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO
Fly Ash	40.1 - 74.4	15.7 - 35.2	1.4 - 17.5	0.2 - 7.4	0.3 - 10.1
Crinker Ash	51.6 - 64.0	17.3 - 26.9	4.2 - 10.9	1.0 - 2.6	2.3 - 8.8
Cement	22.2	5.1	3.2	1.2	65.3

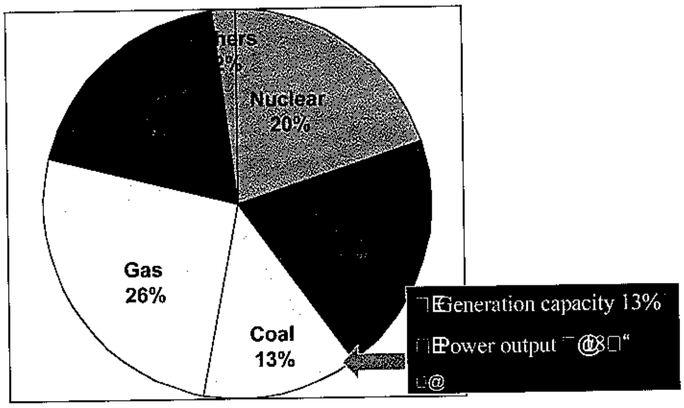

Fig. 9　Generation capacity by coal-fired power

Table 10　Production of fly ash

Year		Production		
		Total	Used	Landfilled
2003	Electric power industry	7.5 million tons (75.8%)	6.1 million tons (81.7%)	1.4 million tons (18.3%)
	General industry	2.4 million tons (24.2%)	2.3 million tons (95.1%)	0.1 million tons (4.9%)
	Total	9.9 million tons (100%)	8.4 million tons (84.9%)	1.5 million tons (15.1%)

Table 11　Utilization of fly ash (unit: million tons)

Year	Situation of Utilization	Electric power industry		General industry		Total	
		Quantity	%	Quantity	%	Quantity	%
2003	Cement&concrete	4.60	75	1.73	76	6.33	76*
	Civil engineering	0.58	10	0.24	11	0.82	10
	Building construction	0.23	4	0.17	7	0.40	5
	Agriculture/forestry&fisheries	0.06	1	0.11	5	0.17	2
	Others	0.63	10	0.03	2	0.66	8
	Total	6.11	100	2.28	100	8.38	100

*Cement material 70%, Cement additive 4%, Concrete admixture 2%

Table 12 Applications of fly ash

Category	Applications	Used in 2003 (million tons)
Cement&concrete	Cement materials, materials to be blended with cement, concrete admixtures, etc.	6.33
Civil engineering	Groud-improving materials, road base course materials, asphalt filler materials, concrete products for civil engineering applications, coal mine restoration, etc.	0.82
Building construction	Building boards, artificial lightweight aggregate, concrete products for building applications, etc.	0.40
Agriculture/forestry&fisheries	Fertilizer, soil conditioners, deicing agents, etc.	0.17
Others	Wastewater treatment chemicals, iron manufacture, etc.	0.66
	Total	8.38

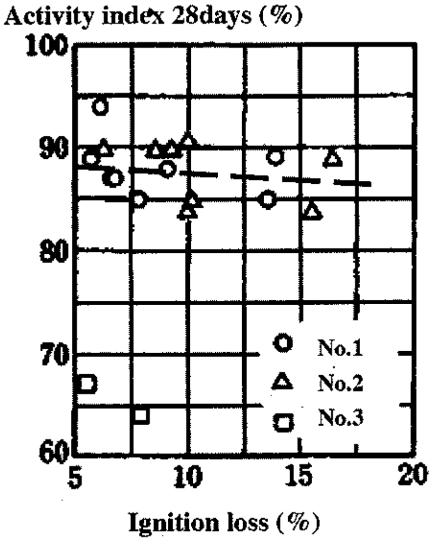

Fig. 10 Relationship between activity index and ignition loss

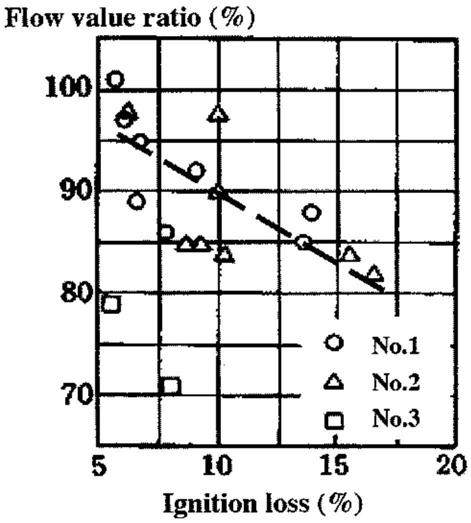

Fig. 11 Relationship between flow value ratio and ignition loss

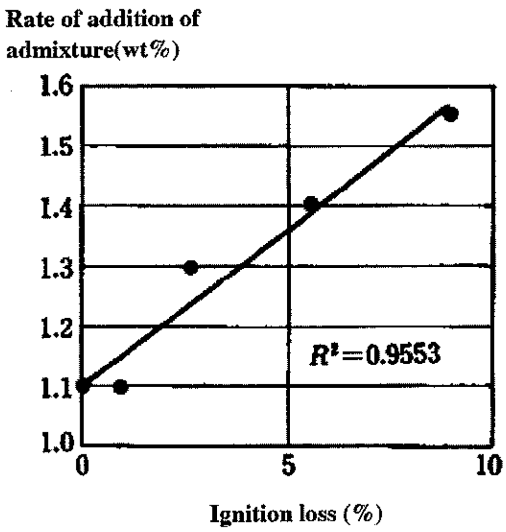

Fig. 12 Relationship between rate of addition of admixture and ignition loss

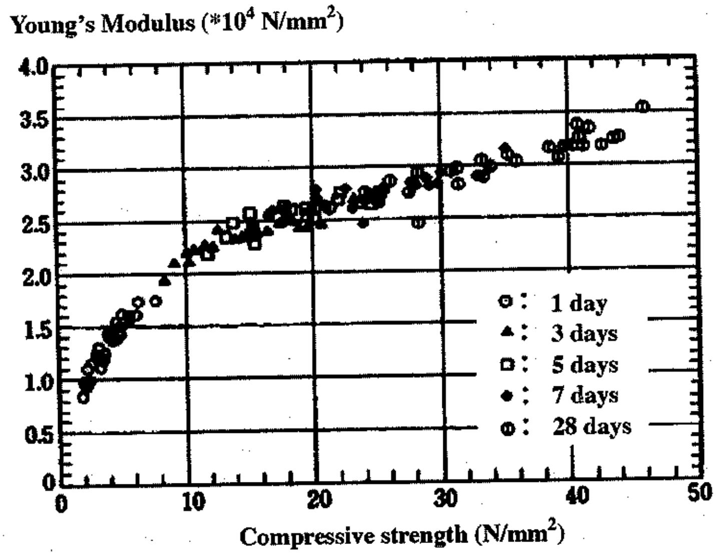

Fig. 13 Relationship between compressive strength and Young's modulus

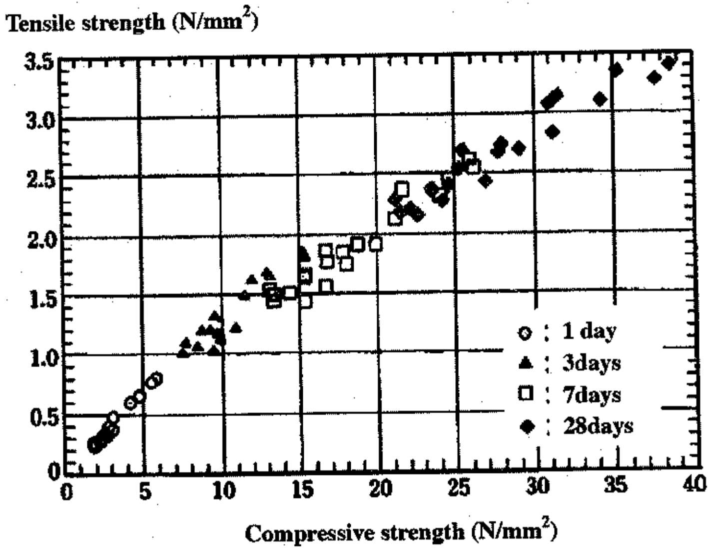

Fig. 14 Relationship between compressive strength and tensile strength

PERFORMANCE-BASED DESIGN OF CONCRETE STRUCTURES
— JSCE STANDARD SPECIFICATIONS FOR CONCRETE STRUCTURES ON STRUCTURAL PERFORMANCE VERIFICATION —

Hiroshi YOKOTA[1]

SUMMARY

The worldwide trend for structural design is shifting from the conventional prescriptive specified design format to the performance-based design format. The JSCE Standard Specifications for Concrete Structures started to embrace the performance-based design in 1995, which was almost completed in their 2002 versions. This paper outlines the present state of structural design of concrete structures focusing on the verification of safety and serviceability as per the JSCE Standard Specifications for Concrete Structures.

Keywords: *Performance-based design, concrete structure, JSCE Standard Specification; safety, serviceability, fatigue.*

INTRODUCTION

This paper briefly introduces the general procedures of structural design of reinforced concrete and prestressed concrete structures as per the JSCE Standard Specifications for Concrete Structures-2002, Structural Performance Verification. The Concrete Committee of JSCE has been actively conducting extensive research and investigation in Japan since its establishment in 1928. The committee considers the preparation and revision of the JSCE Standard Specifications for Concrete Structures as the most important task. The JSCE Standard Specifications are held in high regard by practicing engineers and have not only served as model codes for design, construction, and maintenance of concrete structures but also have been a major driving force in the development of new technologies.

Meeting the change in design format worldwide (ISO, 1998), the JSCE Standard Specifications were shifted from the specification-based format to performance-based format in 1995. The present version published in 2002 introduces many new concepts and methods for the first time in the world. This is a clear indication of the high level of competence and technology available in the country though various issues are still not completely understood and need further investigation. The JSCE Standard Specifications are nominated in ISO

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19338 (2003) as performance-based design codes deemed to satisfy the requirements in performance-based design standard.

In the JSCE Standard Specifications, the following three performance requirements are considered: safety, serviceability, and restorability. Since restorability can be taken particularly at the verification against earthquake actions, the restorability is referred to in the Standard Specifications-2002, Seismic Performance Verification. The general procedure for verifying mechanical performance of concrete structures is given in the JSCE Standard Specifications for Structural Performance Verification (hereinafter called "the Specification"). The performance of concrete structures varies over time due to environmental conditions and other factors. The examination on whether such change is in acceptable range is described in the JSCE Standard Specifications for Materials and Construction. Once the construction is completed, it is difficult to repair, strengthen, or renovate concrete structure; therefore, thorough investigation at the beginning stage of design, accurate prediction for possible problem in service life and future maintenance are of great importance. As recommended verification methods, those with the limit state design methodology are provided; ultimate limit states and fatigue limit state for safety and serviceability limit states.

Table 1 lists the table of contents of the Specification.

Table 1 Table of contents

Chapter 1	General
Chapter 2	Basis of Design
Chapter 3	Design Values for Materials
Chapter 4	Load
Chapter 5	Structural Analysis
Chapter 6	Verification of Structural Safety
Chapter 7	Verification of Serviceability
Chapter 8	Verification of Fatigue Resistance
Chapter 9	General Structural Details
Chapter 10	Prestressed Concrete
Chapter 11	Composite Steel and Concrete Structure
Chapter 12	Design of Members
Chapter 13	Strut-and-Tie Model

SCOPE

Figure 1 gives a schematic representation of the various steps – design, construction planning, fabrication and erection, construction in practice, and maintenance – involved in the construction and management of concrete structures. At each stage, the work is carried out in a manner that all requirements specified in any upstream stage have to be satisfied.

The Specification provides the standard method of verification for safety and serviceability of structures in the design stage of all concrete structures, including those made with plain and reinforced concrete, prestressed concrete and steel-concrete composites, as well as the structural details as the prerequisite of verification. In cases where adequate structural performance can be confirmed using prototype experiments assuming design loads, scaled model experiments, or numerical analyses of which accuracy and applicability have been already ensured, the procedure for verification of structural performance as specified in the

Specification may not be followed.

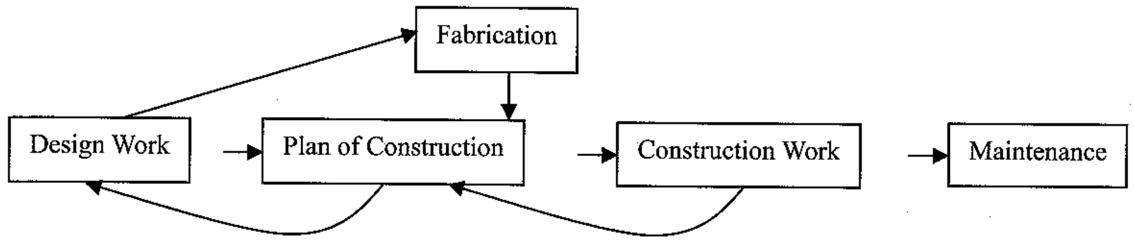

Figure 1 Flow of work

BASIS OF DESIGN

General

At the stage of design of the structure, the following issues are to be determined with due consideration of overall economy.

a) structural details including the shapes and sizes of members,
b) details of reinforcement,
c) specifications of concrete and reinforcement to be used,
d) construction method – cast-in-place or precast, and
e) maintenance method

Throughout the design life of the structure, appropriate steps are taken to ensure that the performance requirements are satisfied in respect to safety and serviceability. The other requirements such as environmental suitability, aesthetics, etc. may be taken into account. Verification methods for the structural characteristics are described in the Specification.

Design Life

For design of a structure, it is necessary to indicate the design life with wide consideration to the purpose of the structure, economically determined period over which the structure is required to be in service, environmental conditions in the neighborhood of the structure, and demanded durability of the structure. In general, the longer design life is specified, the higher durability and fatigue resistance performance will be required.

Prerequisite of Verification

The methods of performance verification described in the Specification are based on mechanics of structures and materials, which generally have some conditions such as deformational consistency of concrete and reinforcement and local states of stresses. Thus, general requirements for structural detailing are specified to realize these assumptions. Otherwise, accuracy of the verification reduces and scope for application is restricted.

For the verification of safety and serviceability of a structure, variation of performance over time during the design life of the structure should be considered. Therefore, it is necessary to assume safety margin by estimating possible deviation of member dimensions, accuracy of bar arrangement and varying mechanical properties of materials.

Principles of Verification

In principle, performance verification of a structure is carried out in such a way that limit states according to performance requirements of the structure are determined in regard to the entire structure or each constituent member during construction and design life, and that the structure or structural member with the structural details such as shape, size, bar arrangement is examined not to reach the limit state. The limit states are classified into the ultimate limit state, serviceability limit state, and fatigue limit state. The examination at the limit states should be carried out using the characteristic values of material strengths and loads and the safety factors.

As a principle of the Specification, performance requirements of a structure should be set clearly at first. Thereafter, an equivalent limit state corresponding to each requirement should be specified. When a structure or a part of it reaches a certain limit state, its serviceability may suddenly lose or it may even reach failure in some cases. Then, the structure cannot hold its function nor meet the requirements because of various defects. In such cases, performance verification of the structure may be done by examining the limit states. In setting a limit state, an index representing the state of a structure, member or material should be selected, and then a limit value set in accordance with the required performance. Verification is done by examining if a calculated response under given actions exceeds the limit values or not. The limit values should be set taking into account reliability of the analysis method and the model employed in calculating the response value. In the case that a total structural system is not simple, consisting of several structures, a process with a possibility that the structural system does not meet the requirements is selected. Required performance is set for each constituent structure, and thereafter, limit state may be set for each element in each structure.

The ultimate limit state is associated with the load carrying capacity of the structure or member. An examination for the failure of the member should be carried out by comparing the member force and its capacity by selecting appropriate cross sections. The serviceability limit state is associated with normal use or durability of the structure. The fatigue limit state is associated with the fatigue failure of the structure or member subjected to repeated loads. Although the fatigue limit state is somewhere included in the ultimate limit state, the Specification defines separately.

Safety Factors

Five partial safety factors are introduced including material factor, γ_m, load factor, γ_f, structural analysis factor, γ_a, member factor, γ_b, and structure factor, γ_i. Material factor, γ_m is determined considering the unfavorable deviations of material strengths from the characteristic values, the differences of material properties between test specimens and actual structures, effects of material properties on the specific limit states, and time dependent variations of material properties. Load factor, γ_f is determined considering unfavorable deviations of loads from the characteristic values, uncertainty in evaluation of loads, effect of nature of loads on the limit states, and variations of environmental actions. Structural analysis factor, γ_a is determined considering the uncertainty of computational accuracy in determination of member forces through structural analysis. Member factors, γ_b is determined considering the uncertainties in computation of capacities of members, seriousness of dimensional error of members, and the importance of members on the entire structure when the member reaches a certain limit state. The value of the member factor is determined corresponding to each equation for member capacities. Structural factor, γ_i is determined considering the importance of the structure, as determined by the social impact when the structure would reach the limit state.

The value of safety factors should be given for each limit state, and they are not necessarily the same for different limit states. Although each safety factor covers the uncertainty of individual event separately, the influence of each factor may be considered collectively. The standard values for safety factors that may be used when verifying durability and applying the inspection system in accordance with the JSCE Standard Specifications for Materials and Construction are listed in Table 2.

Table 2 Standard values for safety factors

	Material factor, γ_m		Member factor, γ_b	Structural analysis factor, γ_a	Load factor, γ_f	Structure factor, γ_i
	concrete, γ_c	steel, γ_s				
Ultimate limit state	1.3	1.0 or 1.05	1.1 ~ 1.3	1.0	1.0 ~ 1.2	1.0 ~ 1.2
Serviceability limit state	1.0	1.0	1.0	1.0	1.0	1.0
Fatigue limit state	1.3	1.05	1.1 ~ 1.1	1.0	1.0	1.0 ~ 1.1

DESIGN VALUE FOR MATERIALS

Quality of concrete or steel is represented by not only compressive or tensile strength but also other material properties such as other strengths, modulus of elasticity or deformation characteristics, thermal characteristics, durability and water tightness. The characteristic value for material strength, f_k is determined taking into account the variation in tested values, such that most of the tested values exceed this characteristic value. The design strength of material, f_d is obtained by dividing the characteristic value for material strength, f_k by a material factor, γ_m. When a specified value for material strength, f_n is determined independently from its characteristic value, the value of f_k is obtained by multiplying the specified value by a material modification factor, ρ_m.

Various types of concrete are used in concrete structures. Appropriate type and quality of the concrete are necessary to be applied for concrete used in structures or members, in consideration of its purpose for use, environmental condition, design life, construction condition and so on. Reinforcing bars, prestressing steel and rolled sections of structural steel used in composite steel-concrete construction are the different steels used in concrete structures. In accordance with needs in structural performance verification, quality of concrete is represented by not only compressive strength but also quantities for various material properties. Material properties can be classified into mechanical properties, such as strength and deformation characteristics, physical properties, chemical properties, and so on. Strength characteristics are expressed by strengths under static and fatigue loadings in compression, tension, bond, etc. Deformation characteristics are expressed by time-independent quantities such as modulus of elasticity and Poisson's ratio, and time-dependent quantities such as creep coefficient and shrinkage strain.

Durability of concrete is considered to be its resistance to time-dependent deterioration resulting from various actions, such as weather, intrusion of chemicals and erosion by chemicals. For the durability of reinforced concrete, resistance to corrosion of reinforcing steel over time is an additional concern. In regard to steel corrosion, durability performance verification has been carried out using the resistance to carbonation and chloride ion intrusion of concrete as indices.

Verification for safety of reinforced concrete may generally be carried out by assuming concrete to be a completely brittle material under tension. However, since performance verification for members, in which the occurrence and propagation of cracking governs, cannot be carried out in a rational manner, it may be necessary to take into consideration a fracture process zone at the crack tip where micro cracks accumulate. In a fracture process zone, which is located between the elastic zone having no cracks and the completely cracked portion, the transferred tensile stress decreases as the crack width; that is, the total width of minute cracks in the zone, increases. This is the so-called tension softening. A tension softening curve expresses the relationship between the transferred stress and the crack width, and the area below the curve corresponds to the fracture energy, which is equal to the energy required to form a unit area of completely opened crack. It is reported that by incorporating the tension softening properties in analysis, the fracture phenomenon in concrete associated with propagation of cracks can be understood, and the size effect in the apparent strength of members can be rationally explained.

An idealized curve given in Figure 2 may be used for the tension softening part. For tension softening properties of concrete, fracture energy, G_F (N/m) for normal concrete may be obtained using Equation 1.

$$G_F = 10(d_{max})^{1/3} f'^{1/3}_{ck} \tag{1}$$

where, d_{max}: maximum size of aggregate (mm)
f'_{ck}: characteristic compressive strength (N/mm^2)

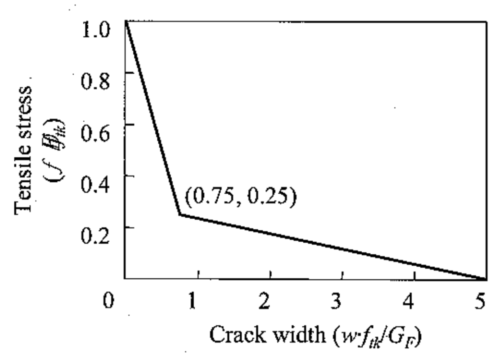

Figure 2 Tension softening curve

LOAD

Structures should be designed for appropriate combinations of loads likely to act during the construction stage and design life of the structure according to the limit states being considered. Design loads are obtained by multiplying a characteristic value of load by appropriate load factor. Combinations of design loads are determined according to the limit states as shown in Table 3.

Permanent loads act continuously on the structure of which variation is rare and in magnitude is negligible compared to the average magnitude. Dead load, load produced by prestressing, shrinkage and creep of concrete, earth pressure, and water pressure should, in principle, be considered as permanent loads. Variable load is the load that varies frequently or continuously and is such that the variations in the magnitude cannot be neglected compared to the average load. Load having varied value such as temperature, wind and snow is considered as variable loads. Accidental load rarely during the design life, but has serious consequences when it occurs. The accidental loads include earthquake, collision, the effect of strong wind, and so on.

Appropriate combinations of loads are selected from permanent load, main variable load, subsidiary variable load, and accidental load according to the limit states to be considered during construction and during the design life of the structure.

Table 3 Combinations of design loads

Limit state	Combination to be considered
Ultimate limit state	Permanent load + main variable load + subsidiary variable load
	Permanent load + accidental load + subsidiary variable load
Serviceability limit state	Permanent load + variable load
Fatigue limit state	Permanent load + variable load

STRUCTURAL ANALYSIS

Appropriate tools for analysis are used in consideration of factors such as the geometry of structures, support conditions, states of loads and the limit states to be examined. Appropriate structural analysis using reliable and accurate models is carried out by using these tools to calculate response values such as member forces, deflection and crack width under design loads. Structures may be analyzed assuming them to be made of simplified elements such as slabs, beams, frames, arches, shells and their combinations. Loads may be modeled in a manner to give equivalent loads to be on the safe side. In the process, load distributions may be simplified or dynamic loads may be replaced by static loads conservatively.

Member forces such as flexural moment, shear force, axial force and torsional moment corresponding to the limit states, are computed based on appropriate analytical theories. Moment of inertia for the member computed on the basis of the gross cross section, neglecting the presence of reinforcing bars, may be used for linear analysis of structures or to estimate the natural period of a structure. Linear analysis may be used for computing member forces for ultimate limit state and serviceability limit state.

STRUCTURAL PERFORMANCE VERIFICATION

Verification for Safety

General

Safety of structure is maintained as long as it or its members do not fail. In the case of a statically highly indeterminate structure, its safety may not be immediately lost even when some of the members reach the ultimate limit state and become incapable of carrying load. In cases where partial failure of members is permitted but the overall safety of the structure still needs to be maintained even after partial failure of some of the members, the nonlinear and the post-failure behavior of members should be appropriately taken into consideration at the time of safety verification as in seismic performance verification.

Basis of verification

It is to be verified that the concrete structure meets the required safety performance during its design life. Safety of a structure is verified by confirming that (a) the ultimate limit state for failure of cross section for any of the members, and, (b) the ultimate limit state for rigid body stability for the structure, is not reached.

Examination of the ultimate limit state for failure of cross section is carried out by confirming that Equation 2 is satisfied; that is, the value obtained by multiplying the ratio of design member force, S_d to design capacity of member cross section, R_d by structure factor γ_i is not greater than 1.0.

$$\gamma_i S_d / R_d \leq 1.0 \tag{2}$$

The design capacity of cross section, R_d can be obtained from Equation 3, by dividing computed member capacity (considering its cross section, design strength of materials, f_d, etc.), $R(f_d)$ by member factor, γ_b.

$$R_d = R(f_d) / \gamma_b \tag{3}$$

The design member force, S_d is obtained from Equation 4 as the sum of member forces, S, computed using the design loads, F_d multiplied by structural analysis factor, γ_a.

$$S_d = \Sigma \gamma_a S(F_d) \tag{4}$$

Examination of ultimate limit states for displacement, deformation, formation of collapse mechanism and others, and, examination of the ultimate limit state of a member or structure using non-linear analysis without considering member forces, is carried out using methods whose applicability to the structure and accuracy have been previously established.

For design of members, generally, the ultimate limit state for failure of member cross-sections is examined for safety verification and other ultimate limit states are not taken into consideration so frequently. For members subjected to one of the member forces among flexural moment, axial load, shear force and torsional moment, verification of safety against failure of member cross sections is carried out as shown in Figure 3.

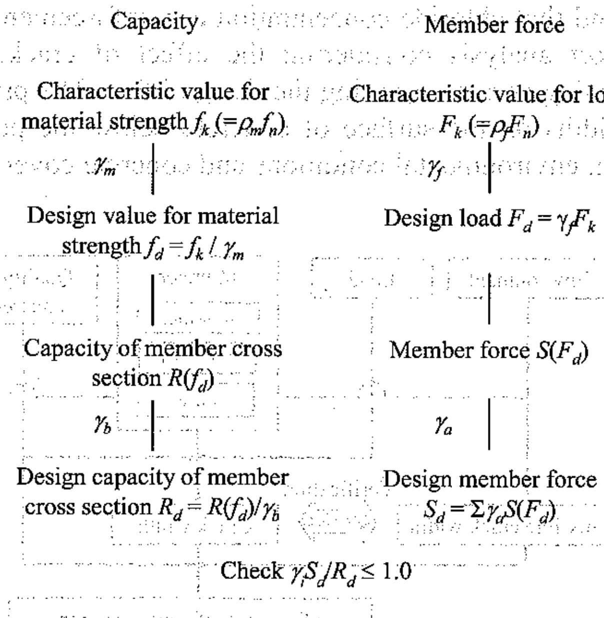

Figure 3 Check for ultimate limit state for failure of cross-section

Verification for Serviceability

General

Structures or members are required to preserve sufficient functions suitable for the purpose of their usage during their design life, such as comfort, water-tightness, appearance, durability during their design life, and others. The verification for serviceability of structure should, in principle, be carried out by ensuring that a concrete member or structure does not reach the serviceability limit state under the design load.

The Specification provides standard methods to verify serviceability of structures based on the assumption that they satisfy required durability and constructability specified in the JSCE Standard Specifications for Materials and Construction, and therefore, material deterioration during the design life is negligible. In case that the material degradation is inevitable, its influences are required to be considered in verification of structural performance by appropriate methods. The limit states of crack width from a viewpoint of durability of structures are presented.

Examination for cracking

Examination using appropriate methods is be carried out to ensure that cracking in concrete does not impair the function, durability, and appearance of the structure. Examination of cracks for durability should, in principle, be carried out by controlling the width of crack at the concrete surface to the extent that required performance of the structure is not impaired by corrosion of reinforcement due to chloride ingress or carbonation of concrete during design life under the given environmental conditions.

Performance of concrete in the concrete cover to protect reinforcement from corrosion due to chloride ingress is achieved by not only controlling crack width but also providing good quality of concrete. Based on this fact, examination of serviceability limit of cracks for durability should be made, in principle, by confirming both that crack width is smaller than the permissible width and that chloride concentration at reinforcement in concrete, predicted by the chloride transport analysis considering the effect of crack, does not exceed the threshold value for initiating corrosion during the design life. The principle of crack control is to keep the crack widths on the surface of concrete below the permissible crack width, which in turn depends on environmental conditions and concrete cover as shown in Figure 4.

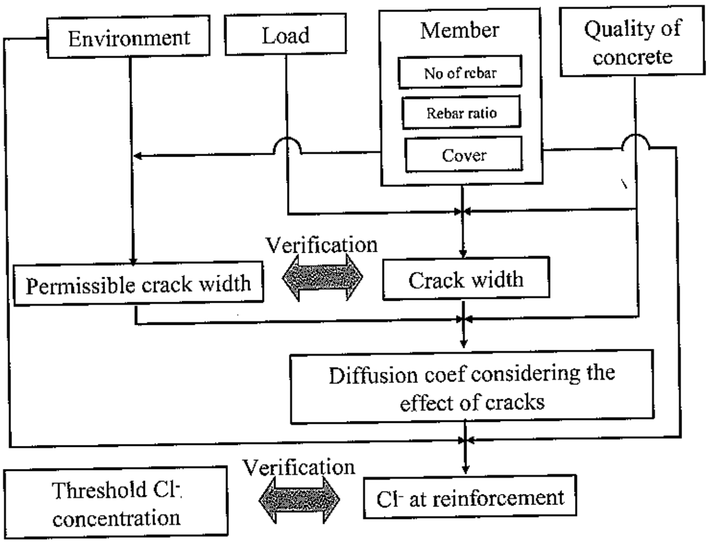

Figure 4 Link between structural performance and durability

Permissible crack width

The functions, importance, service life and purpose of use of structure, environmental and loading conditions of the structure, effects of axial loads, type and stress condition of the reinforcement, concrete cover, scatter in crack widths, etc. need to be taken into account in determining the permissible crack widths. In general, the permissible crack width for corrosion of reinforcement may be determined depending on the environmental conditions, concrete cover and type of reinforcement, as given in Table 4. However, the concrete cover c used there should not exceed 100 mm. Durability of concrete structures is greatly affected by corrosion of reinforcement. Depending upon the extent to which they affect the corrosion, environmental conditions have been classified into three categories as summarized in Table 5.

Corrosion of reinforcement in concrete does not depend only on crack width. Hence, the permissible crack widths indicated in this clause are minimum requirements to avoid risk of corrosion of reinforcement. For structures under "corrosive" or "severely corrosive" environments not only should the crack widths be kept to levels smaller than the permissible width, but also an examination for the concentration of chloride ions at the position of reinforcement should be carried out.

Table 4 Permissible crack width, w_a

	Environmental conditions for reinforcement corrosion		
	Normal	Corrosive	Severely corrosive
Deformed bars and plain bars	$0.005c$	$0.004c$	$0.0035c$
Prestressing steel	$0.004c$	---	---

Table 5 Classification of environmental conditions for reinforcement corrosion

Normal environment	Normal outdoor environment with ordinary conditions without any airborne salt, underground, etc.
Corrosive environment	1. In comparison to the normal environment, environment with more frequent cyclic drying and wetting, and underground environment below the level of underground water containing especially corrosive (or detrimental) substances, which may cause harmful corrosion of reinforcement. 2. Environment of marine structures submerged in seawater, or structures not exposed to severe marine environment, etc.
Severely corrosive environment	1. Environment in which reinforcement is subjected to detrimental influences considerably. 2. Environment of marine structures subjected to tides, splash, or exposed to severe ocean winds, etc.

Examination for flexural cracks

For structures without flexural cracks, examination of flexural cracks is not required in the examination of the serviceability limit state. However, in cases when there is a high risk of cracks, such as thermal cracks, which are not generally considered in the examination of the serviceability limit state, an examination using appropriate methods should be carried out to determine if such cracks may be potential causes for future problems. Examination for flexural cracks should be made, by ensuring that the crack width, w calculated using Equation 5 is not greater than the permissible crack width, w_a given in Table 4.

$$w = 1.1 k_1 k_2 k_3 \{4c + 0.7(c_s - \phi)\} \left[\frac{\sigma_{se}}{E_s} \left(\text{or } \frac{\sigma_{pe}}{E_p} \right) + \varepsilon'_{csd} \right] \tag{5}$$

where, k_1: a constant to take into account the effect of surface geometry of reinforcement on crack width. It may be taken to be 1.0 for deformed bars, 1.3 for plain bars and prestressing steel.

k_2: a constant to take into account the effect of concrete quality on crack width.

$$k_2 = \frac{15}{f'_c + 20} + 0.7 \tag{6}$$

f'_c: compressive strength of concrete (N/mm^2). In general, it may be taken to be equal to the design compressive strength.

k_3: a constant to take into account the effect of multiple layers of tensile reinforcement on crack width.

$$k_3 = \frac{5(n+2)}{7n+8} \tag{7}$$

n: number of the layers of tensile reinforcement.

c: concrete cover (mm).

c_s: center-to-center distance of tensile reinforcements (mm).

ϕ: diameter of tensile reinforcement (mm).

ε'_{csd}: compressive strain for evaluation of increment of crack width due to shrinkage and creep of concrete (150×10^{-6} in general concrete and 100×10^{-6} for high strength concrete)

$\sigma_{se(pe)}$: increment of stress of reinforcement (prestressing steel) from the state in which concrete stress at the portion of reinforcement is zero (N/mm^2).

$E_{s(p)}$: Young's modulus of reinforcement (prestressing bar)

Flexural cracking in reinforced and prestressed concrete is affected by various factors. According to the previous studies, the main factors are the types of reinforcement, increase of stress in reinforcement, concrete cover, effective cross sectional area of concrete, diameter of reinforcement, ratio of reinforcement, number of layers of reinforcement, surface geometry of reinforcement, quality of concrete, magnitude of prestress, and so on. Equation 5 has been formulated on the basis of the existing equations for prediction of crack width and results of recent studies.

Spacing of flexural cracks is affected by the bond between the reinforcement and the concrete. Coefficient k_1 is a constant to represent the effect of surface geometry of reinforcement, which is one of the bond factors affecting crack width. Coefficient k_2 is a constant to represent the effect of changes in bonding characteristics between the reinforcement and the concrete due to changes of concrete quality on crack width. It has been reported that concrete with little material segregation and having a dense pore structure, possesses not only high resistance to cracking but also reduces crack width due to good bonding. Coefficient k_3 is a constant to represent the effect of the reinforcement in the second and higher layers in members with multi layers of reinforcement on the surface crack width.

Examination for displacement and deformation

Displacements and deformations, in general, are related to maintaining functions and serviceability for safety and comfort with moving traffic, preventing damages due to excessive displacements and deformations, and maintaining esthetics of structures. Considering the purpose of use of a structure, enough stiffness and appropriate camber should be provided, and support need to be selected adequately. It is advisable to examine the influences of gasp kinks between members and expansions/shortenings of members if necessary. For structures resting directly on the ground, it is advisable to examine the serviceability limit state of vertical support using an appropriate method, in cases when such an examination is specially required.

There are two types of displacement and deformation. One is short-term displacement and deformation caused instantaneously at the time of application of load. The other is additional displacement and deformation caused by shrinkage and creep of concrete due to permanent loads. Long-term displacement and deformation are defined as the sum of the short-term and the additional displacement and deformation. In cases when precise computation of displacement and deformation are not required, displacement and deformation of cracked reinforced and cracked prestressed concrete members, may be computed using the moment of inertia of a gross cross section assuming no flexural crack. In computation of short-term or long-term displacement and deformation, for cases when reduction of stiffness due to flexural cracking and the influence of creep and shrinkage are taken into account, the effective flexural stiffness is used.

Verification of Fatigue Resistance

Examination for performance of a structure in fatigue is carried out when the ratio of variable loads to total loads is large, or the structure is subjected to a large number of loading cycles. Fatigue failure of a material in a structure will directly affect the safety of the structure. This specification provides the standard of verification of safety of a structure relating to fatigue failure of the material in the structure.

Safety of a beam under fatigue load is examined for flexural moment and shear force, while safety of a slab under fatigue load is examined for flexural moment and punching shear. For a column, examination of safety under fatigue loading is not required in general. However, when the applied flexural moment or axial tensile force is large, examination for fatigue is carried out in a manner similar to that for beams.

GENERAL STRUCTURAL DETAILS

The Specification covers the following as general structural details:

(1) concrete cover
(2) clear distance of reinforcement
(3) bend configurations of reinforcement
(4) development and splice of reinforcement
(5) beveling
(6) additional reinforcement for exposed surfaces
(7) additional reinforcement for concentrated reactions and for openings
(8) construction joints and expansion joints
(9) water-tight structures
(10) drainage and water proofing
(11) protection of concrete surface
(12) haunches

The descriptions in structural details sometimes contradict the spirit of performance-based design because almost all the statements are prescriptive specifications. Regarding the structural details, it can be understood that these are indispensable prerequisites for performance verification as per the Specification.

Several new mechanical devices have recently been invented for anchorage and splice in reinforcement. As for anchorage, the following methods are included: (a) bars such as high strength bars, ultra-large-diameter bars, and threaded bars, not stipulated in the present standards, that are embedded into concrete with sufficient development length, (b) embedment of bars having their ends made into a special shape, (c) use of special jigs such as steel plates, nuts or other hardware attached to the end of the bar, to transmit stresses to the concrete, or bar end anchorage connected with the steel frame by a weld or mechanically, and (d) by reinforcing the concrete around bars to increase the bond strength, and thereby shortening the required development length.

When one of these methods is used, the performance of embedment is verified depending on the type of the structure and members, the loading conditions, and other factors: (a) static strength, (b) resistance to repeated high stresses, (c) resistance to fatigue to high cycle, (d) reliability of execution and other conditions, and (e) other characteristics such as performance under low-temperature, etc. The examination for the performance of anchorages of reinforcement is generally carried out by the pullout test of reinforcement or the loading test using beam or column specimens.

There should be no harmful cracking in the anchorage zone under loads up to the levels corresponding the serviceability limit state in the steel reinforcement. The concrete around anchored reinforcement should not deform under loads up to levels corresponding to those that cause yielding in the reinforcement. The concrete around the anchored reinforcement should not fail under loads up to the levels corresponding to the breaking strength of the reinforcement. Under repetitive loading, the length of pullout of reinforcement, the width of cracks in the concrete anchorage zone, and looseness at the concrete/steel reinforcement interface should be within the specified limits. The required performance of reinforcement anchored in concrete varies depending on the purpose and the location of the anchorage. Thus, it is not reasonable to specify a single representative index for the performance.

PRESTRESSED CONCRETE

Use of prestressed concrete enables us to improve crack characteristics at the serviceability limit state and to reduce the required cross sectional area with use of high strength steel. As far as treatment of the prestressing force introduced to concrete members for the purpose of design calculations is concerned, it may, in general, be treated as a load in the consideration of the serviceability limit state. At the ultimate limit state, only the indeterminate force may be considered as the effect of the prestressing force is included in calculating the ultimate strength of the cross-section.

Prestressed concrete classified into general prestressed concrete (PC) and prestressed reinforced concrete (PRC) structures. In PC no cracking is allowed at the serviceability limit state, and the stress of tension-side fringe of concrete is controlled through introduction of prestressing forces. In PRC, cracking is allowed at the serviceability limit, and the crack width can be controlled by a suitable provision of deformed reinforcing bars and introduction of prestressing forces. However, the limit state of the stress of tension-side fringe is determined variously because of the circumstances, function or purpose of structure, or other reasons. In particular for PRC structure, the restraint of reinforcing bars by creep and shrinkage of concrete is considered. If required, a more rigorous analysis that also takes into account the effect of cracking may be performed, considerably widening the scope of application of such structure.

In recent year, many defects of prestressed concrete structures due to corrosion of prestressing steel have been reported. Since corrosion of prestressing steel reduces the safety or serviceability of prestressed concrete structure, it should not occur during the design life.

For post-tensioned structures, grouting is carried out for 2 reasons: one is to protect the prestressing tendon against corrosion, and the other is to establish a bond between the prestressing tendon and the surrounding concrete. Therefore, if grouting is incomplete, initiation of corrosion in the prestressing tendon, or concentration of cracks, or in the worst case, a much reduced failure strength due to breaking of the prestressing tendon, may be feared. As quality of grouting considerably affects occurrence of corrosion in prestressing tendons, it should be ensured that grout fills all the voids in the sheath.

DESIGN OF MEMBERS

The chapter of "Design of Members" specifies distinctive aspects of structural members. It covers beams including deep beams and corbels, columns, rigid frames, arches, planar members including slabs and footings, shell, wall, and precast concrete.

In this paper, among various specifications, the shear capacity of deep beam is particularly mentioned. Provision specifies a conservatively simplified equation for computation of the shear capacity of deep beams, in such a way that it gives the same value as obtained in the equation for the shear capacity of ordinary beam members. Since contribution of shear reinforcement is smaller than that of the slender beam, it was ignored for standing the safe side. However, given that the shear reinforcement is adequately provided, it has recently been revealed by the experiment that the contribution to the shear strength can be expected. Therefore, the shear reinforcement is taken into account to calculate the shear capacity of deep beam in the latest version of the Specification as presented in Equation 8.

$$V_{ydd} = V_{cdd} + \phi V_{sd} \tag{8}$$
$$\phi = -0.17 + 0.3(a_v/d) + 0.33/p_{wb} \leq 1.0$$

Where,
V_{cdd}: design shear capacity without shear reinforcement
V_{sd}: design shear capacity of shear reinforcement
a_v: distance between edge of support and loaded point
d: effective depth of member at loaded point
p_{wb}: ratio of shear reinforcement (%)

STRUT-AND-TIE MODEL

General

The strut-and-tie model is a useful tool to estimate the resisting capacity of a structure and how the internal forces of a structure flow. The model was first introduced in the Specification. A reasonable, safe design can be achieved for structural members, such as beams, columns, or slabs, of which design and construction procedures have been well established by following the provisions mentioned earlier. On the other hand, ultimate structural capacity of members having abrupt changes in sections, corners of framed structures, and/or openings cannot be well examined by using the standard provisions. In such cases, it is desirable to perform proof experiments or nonlinear analyses with sufficient accuracy based on assumptions of geometry and material properties. However, if the resisting mechanism for the design load is predetermined, the design for determining the details of reinforcement and the properties of materials used can be accordingly carried out with the strut-and-tie model. Therefore, the strut-and-tie model may be applied to examine the ultimate limit state of structures having discontinuity regions in which the flow of internal forces changes significantly.

Description of the Model

In the strut-and-tie model, structures or members are discretely modeled into an assembly of one-dimensional struts and ties as well as nodes connecting these struts and ties (see example as shown in Figure 5). The ultimate capacity corresponding to the assumed flow of forces is calculated based on the static equilibrium conditions and the individual strength of the struts and ties. The tie is normally modeled on the basis of the resultant force from a layer of reinforcing steel. The strut represents the resultant of either a uniform compressive stress field or a fan-shaped compressive stress field. The node represents a certain amount of concrete volume, in which struts either intersect with ties or are deviated by ties.

Since the strut-and-tie model does not strictly consider the compatibility condition in deformation when deciding the location of struts and ties, it should be ensured that the compatibility of deformations is adequately maintained by providing sufficient deformation ability to the structural members. This ensures the transfer of forces according to the resisting mechanism assumed in the ultimate limit state. The Specification specifies strength of ties, strength of struts, and strength of nodes and anchorages of reinforcing bars.

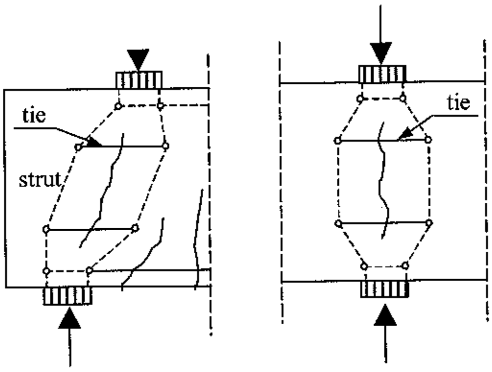

Figure 5 An example of strut-and-tie model

CONCLUDING REMARKS

The updating work on the Specification has just started. It becomes more important to modify the framework of design of concrete structures by linking structural performance aspects with durability aspects and environmental issues. We will be making best efforts to provide concrete engineers, designers, etc. with sufficient backgrounds of concrete structures as well as manuals for practical use.

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THE DEVELOPMENT OF DURABLE HIGH PERFORMANCE CONCRETE IN VIETNAM

Nguyen HUNG [1] and Nguyen Van CHANH [2]

SUMMARY

The recent development in the field of high-performance concrete (HPC) have been a giant step in making concrete a high-tech material with enhanced characteristics and durability. They have even led to it being a more ecological material in the sense that the component admixtures, aggregates and water are fully used to produce a material with a longer life cycle.
Achieving high-performance is not done with a casual approach; all ingredients must be carefully selected. High-performance concrete is very sensitive to plastic and autogenously shrinkage, so that their use demands an immediate water curing. High-performance concrete is definitely more durable the usual concrete. Its increased use will be more often linked to its durability than its high strength. Durability will become a key issue because we will become more and more concerned with sustainable development.
In this paper, the basic characteristics of high-performance concretes are described and discussed.

Keywords: *High-performance concrete, component, characteristic, durability.*

GENERAL INFORMATION

The high performance concrete concept if translated into technical terms for cement-based composites means:

- such a consistency in the fresh mix that its workability, flowability, mobility, compactability, pumpability and finishability ensure good results of execution without much effort from workers or an excessive expense of energy.
- excellent behaviour of materials in their hardened state, i.e. strength and deformations satisfying standard requirements imposed by the applications.
- relatively high strength at an early age.

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- acceptable behaviour in the long-term, i.e. durability adequate to requirements during the forecast life of the structure, low maintenance costs and relative facility of repair works.
- Good aspect of the structure during its service life, i.e. without visible cracks, voids and spellings, excessive defections, etc.

The concepts of high strength and high performance concrete varied over time. The compressive strength of ordinary concrete increased from about 15 MPa to 40 MPa. At present, high performance concretes (HPC) of $f_{28} \geq 60$ MPa and very high performance concretes (VHPC) with $f_{28} \leq 120$ MPa are conventionally distinguished.
In the performance approach to concretes the main questions which are formulated by on parties involved in the construction process are answered – the contractor who takes care of production, transportation, casting and curing of the fresh mix, the investor, the owner of the structure and the general public. All of them are interested in a low general cost for the structure, its long-term serviceability and safety. The users are less aware of various physical and chemical properties, but the overall performance is of primary importance. The scope of application of HPC and VHPC is determined by their improved properties with respects to ordinary concretes and by the technical and economic advantages which may be obtained as a result.

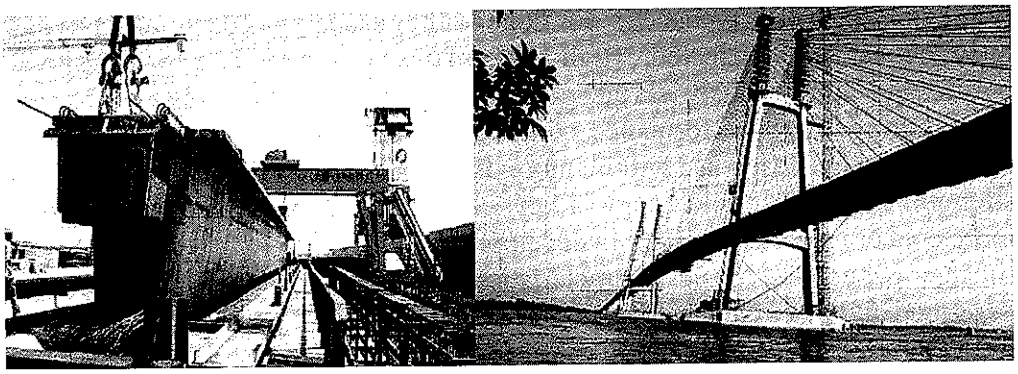

Photo 1: Viet Nam My Thuan bridge was built by high performance concrete

A COMPOSITE MATERIAL

High Performance Concrete are made with basically the same components as ordinary concretes but without the application of special technologies. The composition of HPC is characterized with respect to ordinary concretes by:

- Increased fraction of fine and very fine grains;
- Lower w/c ratio and use of superplasticizers (also known in this context as High-Range Water-Reducing Admixtures-HRWRA);
- Smaller fraction of coarse aggregate and smaller maximum grain dimension.

In the design of HPC composition two main groups of problems need to be solved in a well coordinated way. The first one covers the chemical and physical properties of all component, their compatibility and synergisms, together with their particular roles in the material structures. The second group of problems concerns the feasibility of using the designed material in local conditions and at a reasonable cost. Usually, the required strength is also attained when these problems are solved.

The examples of the composition of HPC and VHPC presented in Table 1 show the features listed above.

Table 1 Examples of high and very high-performance concretes

High-performance concretes								Very high-performance concretes	
		(1) (kg)		(2) (kg)		(3) (kg)		(4) (kg)	(5) (kg)
Gravel	12.5/20	852	10/20	698	5/16	955			
Gravel	5/12.5	267	6/14	465	2.5/6.3	217	2.5/10	1076	1094
Sand	0/5	765	0.1/2.5	738	0/2.5	934	0/5	753.9	772.6
Cement		425		425		425	cement	502.6	506.6
Water		150		160		160	Water	115.6	115.4
Superplast		6.4		8.5		12.8	Superlast	12.1	21.1
Retarded		1.7		1.7		1.7	Silica fume	50.2	50.7
Slump(mm)		180 - 250		110 - 190		200 - 210	Slump(mm)	230	230
		(MPa)		(MPa)		(MPa)		(MPa)	(Mpa)
fc_1		17.8		36.2		57.7	fc_{28}	120.7	120
fc_2		60.6		68.3		57.7			
fc_{28}		74		75.9		67.2			
fc_{90}		82.5		81.5			d (kg/m^3)	2510	2 560
ft_{28} (split)		5.3		4.5			E (MPa)	49 751	50 314
w/c		0.35		0.38		0.38	w/c	0.21	0.21

Portland cement

Ordinary Portland cement of good quality may be used for HPC. A high content of tricalcium silicate C_3S and bicalcium silicate C_2S (alite and belite) is favoured together with a low content of tricalcium aluminate C_3A. Because of the low values of water/cement ratio required, a relatively high amount of Portland cement is used 400 kg/m^3 and more. The amount of cement may be reduced by blending with order micro-fillers like fly ash or high purity silica with grains of about 4μm in diameter.

A Portland cement of rather fine grain is preferred, but not too fine, so as to avoid excessive acceleration of all processes. The compatibility with other components should be verified as well as low shrinkage and heat from hydration.

Admixtures and additives

Two main groups of admixtures are considered to be very important component of HPC – superplasticizers which improve the workability of the fresh mix with low values of water/cement ratio, and micro-fillers to increase the density of the hardened material.
There are several kinds of superplasticizers available. They belong to two main groups' sulphonated melamine-formaldehyde condensates and sulphonated naphthalene-formaldehyde condensates. Their action is explained by the absorption of polyanions on the surface of cement grains and by the generation of negative potential which eliminates the attraction and coagulation of the grains. As a result, a decrease of internal friction is obtained and the workability expressed for example by slump of 180 – 250 mm is ensured for 1 – 1.5 hours. The correct selection of a superplasticizers for other mix components is important. Usually superplasticizers are added as 2 – 4% of cement mass, according to the producer's prescription. With higher dosages some delay may occur in hydration and hardening together with the apparent early setting of the fresh mix.

When Portland cement is partly replaced by fly ash or blast-furnace slag, then better flowability of the fresh mix is usually obtained and lower dosages of superplasticizers are possible. This is quite profitable from the economical viewpoint: superplasticizer is an important part of cost of the concrete components.
The compatibility of superplasticizers with cement as well as their efficiency, duration of the fresh mix fluidity, sensitivity to ambient temperature and other factors should be verified by experiments executed in local conditions.
Fly ash, silica fume and other silica products are used as micro-fillers. They are furnished as powder or slurry with grains of one or two orders smaller than Portland cement. They enter into the voids between the cement grains and by acting as water-redactors enhance the efficiency of the superplasticizers. By their addition, the contacts between aggregate grains of bleeding and segregation of the fresh mix during transportation and casting.
The pozzolanic properties of silica fume result in a slow hydration process and in more efficient gel development. Silica fume considerably improves the performance of the binder phase and increases its bonding action with the aggregate and reinforcement. The highly porous interface is the weakest element in the structure of an ordinary concrete and its strengthening is decisive for HPC.
The application of silica fume is not necessary for concrete of compressive strength up to 60 MPa, but it is considered as a compulsory component of VHPC. However, in HPC and ordinary concrete silica fume also improves their density and durability.
According to the optimum content of silica fume is 7 – 15 % of cement mass. A higher dosage may increase brittleness and influence unfavourably the total cost of the final composite material.

W/ c ratio

The reduced amount of water and low value of the w/c ratio are necessary for the high strength and low porosity which characterize HPC. The excellent workability of fresh mix required is ensured by admixtures. W/c ratio remains between 0.25 and 0.30 and between 0.30 and 0.35 for HPC, probably for higher percentage of hydration of Portland cement. Also, lower dosage of superplasticizer might be possible for required workability.
Because of the pozzolanic properties of microfillers, it is common to also calculate the water to cement and microfiller ratio: w/(c+m).

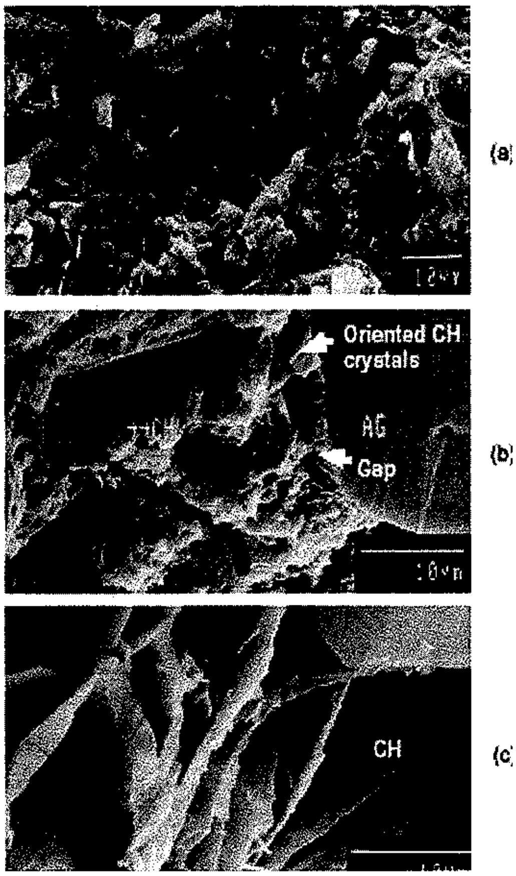

Figure 1. Microstructure of high water/cement ratio concrete: (a) high porosity and heterogeneity of the matrix, (b) orientated crystal of Ca(OH)2 on aggregate, (c) CH crystal [2]

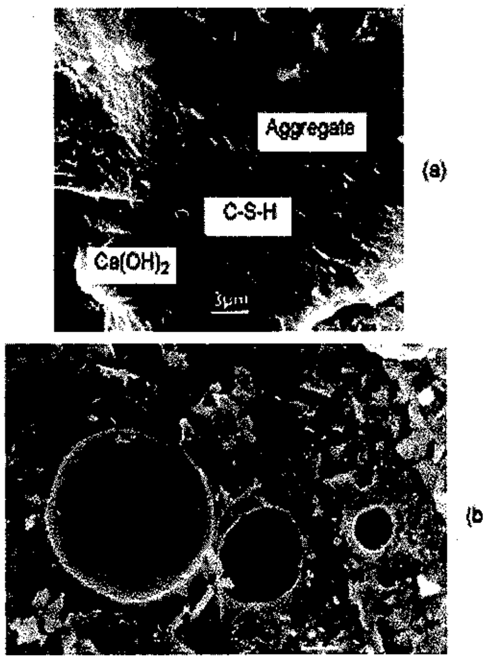

Figure 2. Microstructure of a high performance concrete: low porosity and homogeneity of the matrix [2]

Aggregate

The properties of aggregate are important and for HPC they should satisfy several different requirements. First of all, good workability must be ensured and for that reason continuous sieve distributions are preferred.

Maximum grain diameter should be limited in order to improve workability and to reduce discontinuities and stress concentrations. According to the grains should not exceed 25 mm, 20 mm is indicated and certain authors even propose 10 mm as aximum grain diameter.

Grains of spheroidal shape are preferred and natural gravel is considered better than crushed aggregate for improved workability and lower requirement.

The appropriate sieve distribution and packing of aggregate grains, together with other particles, in the mix is the direct and classic way to ensure the high density of the hardened concrete. In practice, the composition of grains obtained from quarries should be improved, but such modifications can be expensive. In other situations, the correct proportions of grain of different diameter are furnished on request.

The quality of sand seems to play a smaller role and requirement as for ordinary concrete apply. Continuous sieve distribution is also favorable and mineralogical composition should be similar to that of the coarse aggregate.

TECHNOLOGY

The methods of production of HPC are basically the same as in the case of ordinary concretes. A high quality of components and their prescribed proportions should be maintained in the fresh mix. The quality of components and of the final product should be controlled at all stages of execution.

The main reason where by HPC is accepted and fully supported by contractors and investors is the excellent workability of the fresh mix obtained by the appropriate application of superplasticizers which are either added to the water before mixing or, when the mix is transported over a longer distance are added in two portions – before mixing; and after transportation, directly before casting. The facility to fill the moulds or forms by the fresh mix and its pumpability is ensured when sufficient slump of the Abrams cone and the coherence of the mix are maintained during the full amount of time required for both transportation and casting.
The slump of the Abrams cone is not the best measure of the consistency of HPC and other methods are available. However, these methods are not generally approved and standardized and specialized equipment varies in different countries. That is why the Abrams cone is still used.

If vibration of the fresh mix is necessary, then the characteristics of the equipment used should be adapted with regard to its frequency and amplitude.
When a micro-fillers is used for HPC like silica fume or fly ash, the prescribed way of adding it to the mix and the correct sequence of adding other components should be carefully executed. The good dispersion of silica fume is particularly important because its effects result from combined physical and chemical mechanisms.

The cure of HPC after casting is different than in the case of ordinary concretes. There is no need to maintain high humidity over a long time period because of the high rate of increase of strength. In order to avoid microcracking on the surface, it is advised that any evaporation of water during the first few hours should be prevented, e.g. by perfect sealing. Furthermore, to prevent stress induction due to the characteristic ability of HPC for self-desiccation, curing in water is not advised, because non-uniform swelling may occur due to the characteristic ability of HPC for self-desiccation, curing in water is not advised because non-uniform swelling may occur due to the low permeability of HPC. The absence of bleeding in HPC is the reason why the humidity of the fresh mix should be maintained immediately following casting. Sufficient curing should allow the chemical mechanism to develop its full effect.

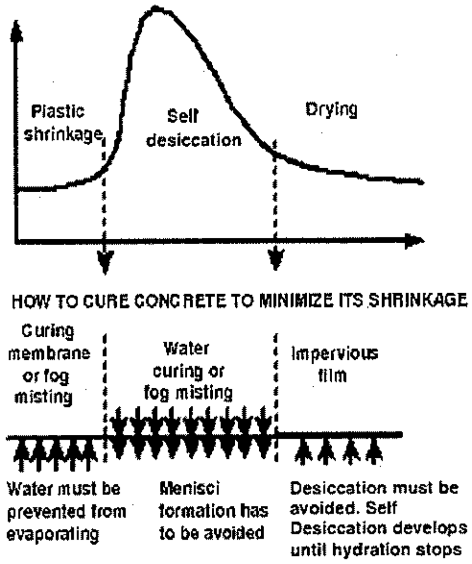

Figure 3 The most appropriate curing regimes during the course of the hydration reaction [3]

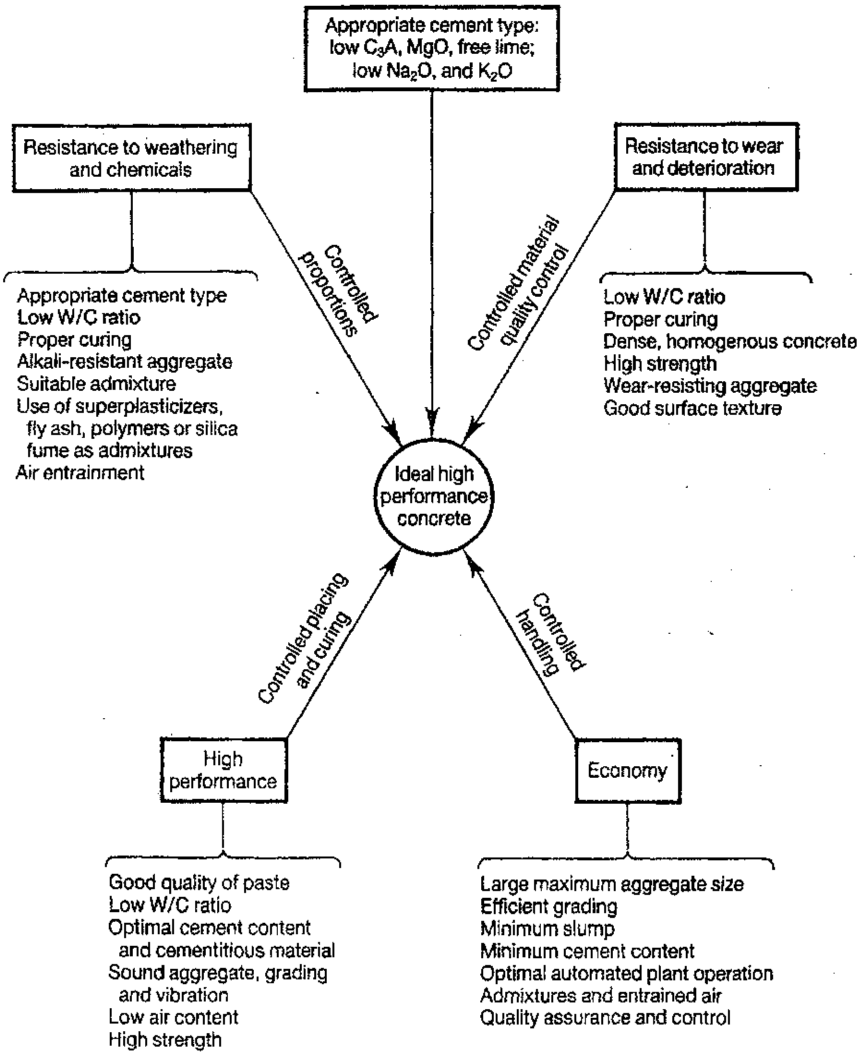

Figure 4 Principal parameters for high performance concrete [4]

The high rate of hardening and relatively high early strength after one, there or seven days enables the moulds and scaffolding to be removed earlier than in the case of ordinary concretes. That is an advantage which is used for quick re-use of moulds or for the reduction of traffic closures on structures being repaired. The fresh mix is also only exposed for a short time to possible adverse ambient factors.

In view of the large variety of possible compositions of HPC and of ambient conditions, the hardening process may develop differently depending on circumstances. That is why a detailed method of cure and the time delay for demoulding should be established following verification and observations in situ.

DURABILITY

Based on years of experience with ordinary concrete, we can safely assume that high performance concrete is more durable than ordinary concrete. Indeed, the experience gained with ordinary concrete has taught us that concrete durability is mainly governed by concrete permeability and the harshness of the environment.
It is easy to asses the harshness of any environment with respect to high performance concrete because hydrated cement paste is essentially porous material that contains some freezable water. Assessment involves simply examining how the environment affects each of these characteristics.
The durability of HPC are related to their composition and structure, characterized by a dense and strong matrix with good bonding to the aggregate grains and by an absence of excessive pores and other inhomogeneities.

The most important property of HPC is its improved durability thanks to the increased impermeability and homogeneity of the material's structure. Hydration products are in a higher degree amorphe than in ordinary concretes and capillar pores of 0.1 – 1.0 μm are considerably reduced or eliminated. Thus the material's resistance to different climatic actions and corrosive factors is enhanced. Increase capability to adopt alien (Cl^-, Na^+, K^+) and improve impermeability decrease the diffusion of chlorides improve resistance against the long-term action of sulphates and against alkali-aggregate reaction and related swelling.

The carbonation process in HPC is basically reduced because CO_2 cannot penetrate its dense structure. Accelerated tests executed showed practically no traces of carbonation. When silica fume is added as admixture for VHPC, the pH of concrete may decrease thus creating conditions more favourable for corrosion of steel-reinforcement. However, in that case the electrical resistance of the concrete is enhanced and consequently it will slow down the corrosion process.

The shrinkage of HPC develops differently than in ordinary concrete. A higher rate of autogenously shrinkage may induce additional stresses when free displacements are constrained at an early age. In contrast, drying shrinkage is less important because of the smaller amount of free water and the lower permeability of the matrix. The total strain due to shrinkage depends on the size of the elements and on the curing conditions, however, in most cases smaller values than for ordinary concretes were observed.

So-called plastic shrinkage may cause cracks in external layers of elements which appear immediately after casting. Such effects are related to local desiccation because of the dense structure of HPC and the low w/c ratio pore water does not migrate from the internal core of elements and the phenomenon of bleeding is absent. The only remedy is abundant moisturing of HPC at a very young age.

The evolution of heat during hydration of HPC should be considered carefully. The amount of heat depends on cement content and on the degree and rate of its hydration. The admixture of silica fume and low water/cement ratio result in a smaller degree of hydration and lower heat evolution. It is expected, therefore, that a smaller amount of heat is produced in HPC. However, appropriate measures should be prepared if necessary to lower the temperature of components, to evacuate heat and to allow for possible displacements, etc. furthermore, it should be bond in mind that for the appropriate action of silica fume lower temperatures are favourable. More research is needed in that area to quantify the effects of different material composition, methods of execution and ambient conditions.

On the other hand, it is not always simple to assess how easily aggressive agents will penetrate concrete. For example, water flow through a 0.70 w/c concrete is easy to measure, but water flow almost stops in a 0.40 w/c concrete, regardless of thickness of the sample and the amount of pressure applied. The gas permeability is also difficult to measure.

Despite all the criticism leveled at it, the so-called "Rapid chloride-ion permeability test" gives a fair idea of the interconnectivity of the fine pores in concrete that are too fine to allow water flow. Experience has revealed good correlation between the water permeability and rapid chloride ion permeability for concrete specimens with aggregate w/c greater than 0.40. Chloride-ion permeability is expressed in coulombs, which corresponds to the total amount of electrical charge that passes during the 6-hour test through the concrete sample when subjected to a potential difference of 50 volts.

When the rapid chloride-ion permeability test is performed on concrete samples with lower w/c, the number of coulombs passing through the sample decreases. It is easy to achieve a chloride-ion permeability of less than 1000 coulombs for high performance concrete containing about 10% silica fume and having w/c around 0.30. Much lower chloride-ion permeability values can be achieved if the w/c is reduced below 0.25. Values as low as 150 coulombs have been reported, far lower than the 5000 to 6000 coulombs reported for ordinary concrete.

The rapid chloride-ion test reveals that the connectivity of the pore system decreases drastically as the w/c decreases, making the migration of aggressive ions or gas more difficult in high-performance concrete than in its plain counterpart. The author believes that this is the best indication that the service life of high-performance concrete should exceed that of ordinary concrete cannot be readily extrapolated to include high-performance concrete. However, it can be said that some high-performance concrete structures will outlast the average life span of a human being in developed countries.

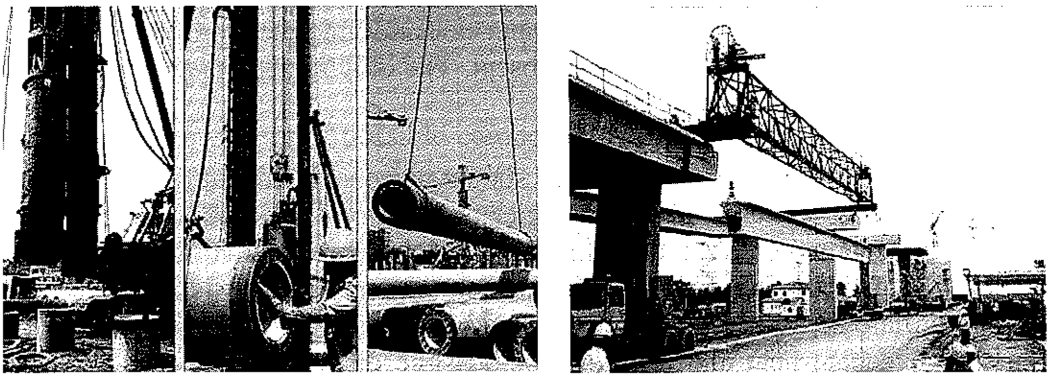

Photo 2 High performance concrete applied in construction at Viet Nam

ECONOMIC CONSIDERATIONS AND FURTHER DEVELOPMENT OF HPC

The economic questions concerning the application of cement-based composites have considerable importance for their development in various building and civil engineering structures.

The cost of HPC is not particularly high. On the contrary commercially available concrete of nearly 100 MPa costs significantly less that 3.5 times the price of ordinary 28 MPa concrete. Thus, more strength is obtained per unit cost and per unit mass, together with more stillness per unit cost and lower specific creep.

Furthermore, the specific cost of the material, calculated after the cost of components and execution, is only a small part of the total cost which also comprises maintenance costs of various kinds, including repairs, possible replacement, breaks in normal exploitation. Using high performance materials, smaller dimensions of structures are necessary and lower maintenance cost may be expected.

In the economic calculations the cost of increased control of all components and of high performance materials in subsequent stages of execution should be partly compensated by gains due to the lower mean values of their required properties. When variations of the material's properties are reduced, then lower mean values may be designed and that aspect again allows a decrease in cost. Methods to evaluate the increase durability and other advantages of HPC are needed. These should be based on rational provision of repair and maintenance expenses during the lifetime of the designed structure. Without such calculations the investor will not be convinced that in many situations the use of HPC is the optimum solution.

Research in the field of optimization of the structure and composition of materials, is also aimed at improving the economic advantages of HPC.

Questions of economy determine the fields of application of cement matrix composites and that is the reason why ordinary concretes are still used and will continue to be used in the future for traditional plain and reinforced structures where high strength and improved durability are not necessarily required. Advanced composite materials are needed for special outstanding structures or for their specific regions, sometimes called "hot-points" like joints and nodes or for elements likely to particularly exposed to destruction.

CONCLUDING REMARKS

Notwithstanding multiple applications of HPC and VHPC, there are several important problems which should be studies further and in order that solutions may be found. These are:

- Optimization of mix composition for given requirements and conditions;
- New kinds of admixtures, microfillers and dispersed reinforcement;
- Application of lightweight aggregate;
- Thermal effects related to hydration of Portland cement;
- Possibility of decreasing the content of Portland cement
- Mechanical behaviour in various conditions, etc.

An effort is needed to further develop both structure designs with the application of HPC. Further work should be aimed at preparation of appropriate design codes and quantitative recommendations. Without any doubt, the trend towards improving the performance of concrete-like composites will also be developed in the further, corresponding to new needs in building and civil engineering structures.

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THE LATEST TECHNOLOGIES OF PRESTRESSED CONCRETE BRIDGES IN JAPAN

Tamio YOSHIOKA[1]

SUMMARY

For more than 50 years prestressed concrete is one of the most important construction materials in not only Japan but also all over the world especially in the field of bridge construction. Since the first prestressed concrete bridge was constructed in 1952, tremendous prestressed concrete bridges have been constructed in Japan. In this paper the latest technologies, an extradosed bridge, a cable stayed bridge, a stress-ribbon bridge and truss bridges of the prestressed concrete and steel-concrete composite structure in Japan are introduced.

Keywords: *Prestressed concrete bridge; extradosed bridge; cable stayed bridge, stress-ribbon bridge; composite bridge; truss bridge*

A STEEL-CONCRETE COMPOSITE EXTRADOSED BRIDGE KISO & IBI RIVER BRIDGE

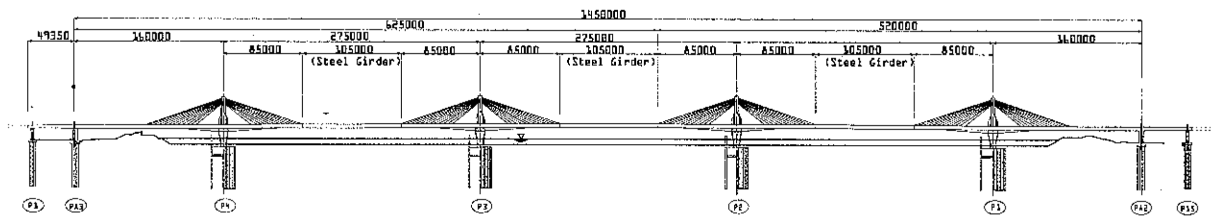

Fig.1 Side view of KISO River Bridge

Location : KISO & IBI River Bridges are located near NAGOYA city, 370 km west of Tokyo. These are a part of the New MEISHIN Expressway between NAGOYA and KOBE city.

Outline of the bridge : The KISO and IBI River Bridge are 1,145m (=160+3@275+160m) and 1,397m (=154+4@271.5+157m) long respectively. Both bridges are 33m wide with six traffic lanes. The depth of the concrete girder varies from 7m at the supports to 4m at the standard section. The depth of the steel girder is uniformly 4m. The height of the pylon is 30m.

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The external tendons (stay cables) are longitudinally arranged at the center of the section as a single-plane suspension system.

Structural characteristics : The KISO and IBI River Bridges are prestressed concrete-steel composite, five and six span continuous, extradosed box girder bridges respectively. In the extradosed bridge the tendons are placed externally with very large eccentricity and they are anchored at the pylon and post-tensioned from the girder side. Regarding the differences between the extradosed and cable stayed bridge the height of the pylon and the depth of the girder of this new type of bridge are shorter and higher than those of the counter one respectively. Very costly post-tensioning is executed once as external tendons in the extradosed bridge, while the post-tensioning stay cables (adjustment of deflection) must be done many times in the cable stayed bridge.
In order to reduce the self weight of bridge the steel girders of around 100m long are employed at the central section of middle spans. This renders the span much longer.

Construction : For the concrete sections the segmental free cantilever construction was adopted. Precast concrete box segments were pre-fabricated in casting yards, 10-15km away from the construction site, and transported to the bridge position with a barge. The column capital segment, whose weight is approximately 400 tons, was erected from a floating crane barge. Cantilever erection of precast concrete box girders excepting the column capital segments is executed with erection noses. The strength of concrete is 60 N/mm^2. The short-line-match-cast technology was employed, in which the side surface of a already cast concrete is used as a formwork for a next segment. The number of all segments is 360.
After the completion of concrete sections the steel girder, which was manufactured in a factory (approximately 2,000 tons), were transported with a barge and lifted into the final position from the reaction girders installed at the end of main concrete girders already in place. The steel girder was fixed tightly to the concrete girders to close the span.

Photos : Photo 1-1 and 1-2 show the completed bridge and the bridge under construction after the completion of the concrete sections respectively. The erection noses for the concrete segmental girders are seen at the end of the girder in Photo 1-2. In photo 1-3 the central steel girder is under erection. Photo 1-4 shows the short-line-match-casting and photo 1-5 the view of the prefabrication casting yards.

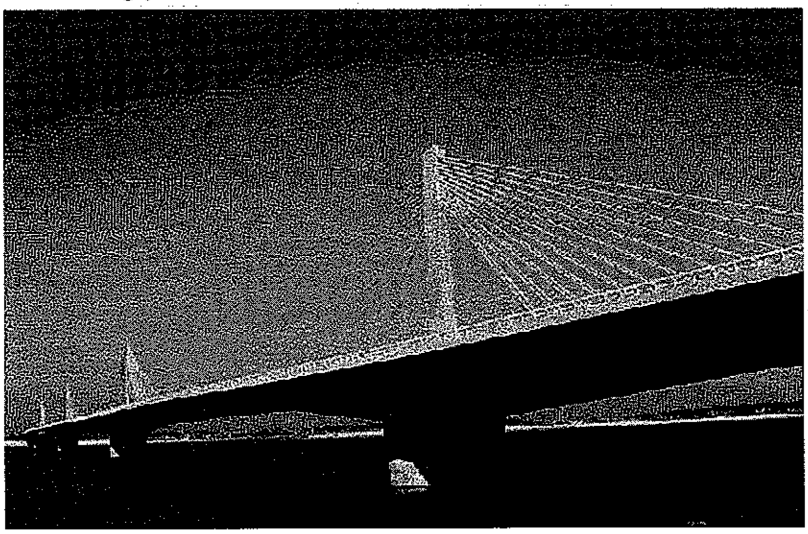

Photo 1-1 Completed KISO River Bridge

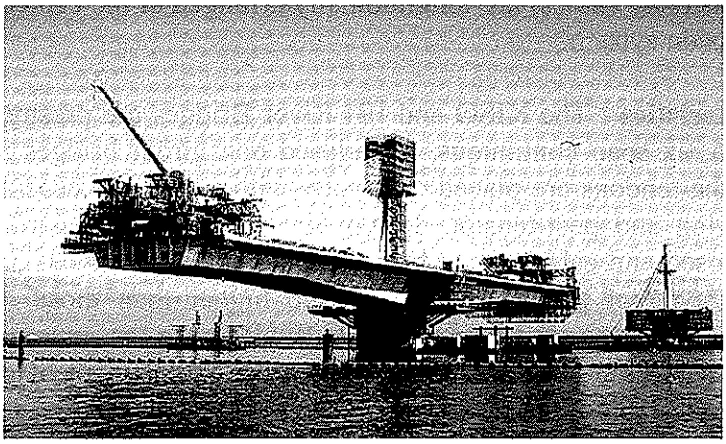

Photo 1-2 After the completion of concrete

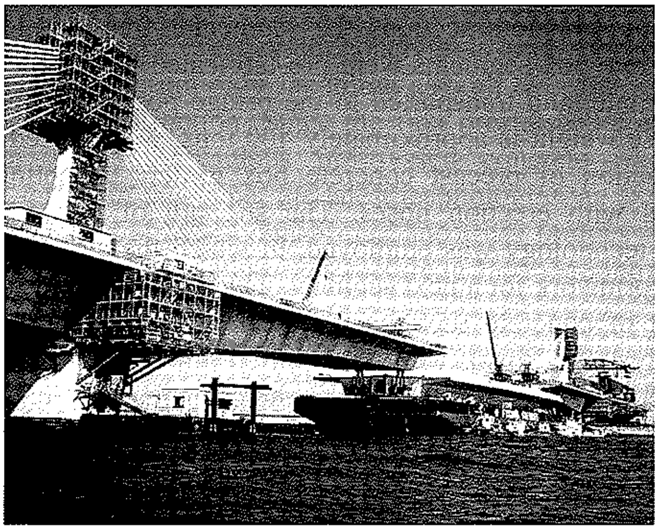

Photo 1-3 Erection of the central steel girder

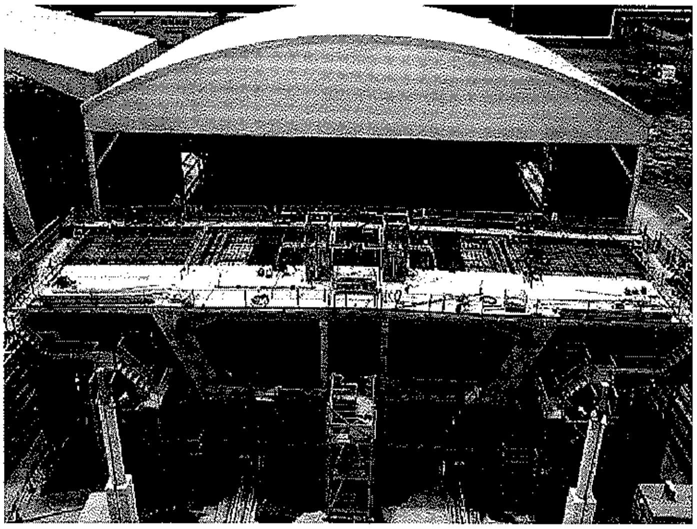

Photo 1-4 Short-line-match-casting

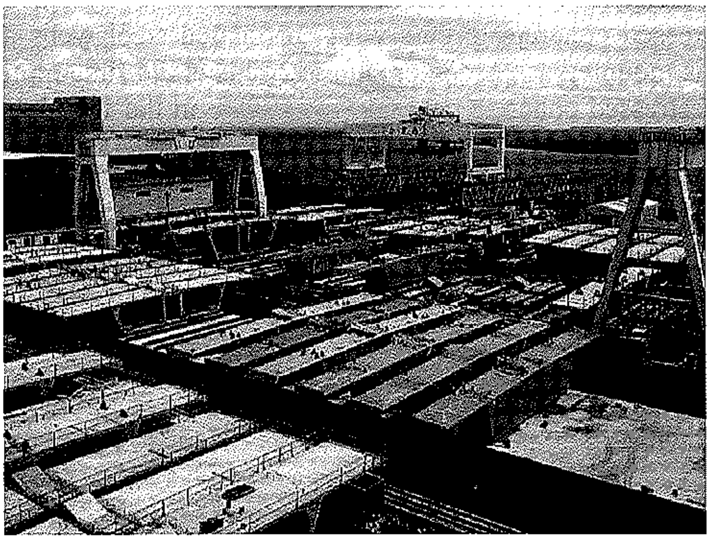

Photo 1-5 Prefabrication casting yard

A HYBRID CABLE STAYED BRIDGE
YAHAGI-GAWA BRIDGE

Location : The YAHAGI-GAWA bridge, a part of the New TOMEI Expressway, crosses YAHAGI-GAWA River at 50km east of NAGOYA city. This bridge is located near the place of EXPO 2005 AICHI.

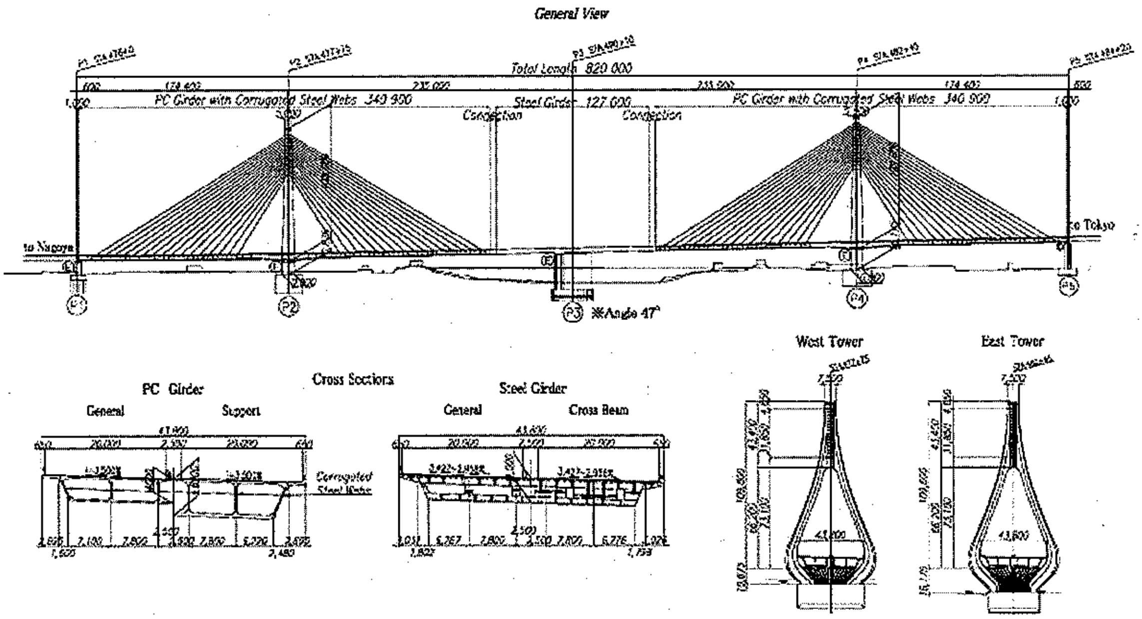

Fig.2 Side view and sections of YAHAGI-GAWA Bridge

Outline of the bridge : The bridge length is 820m (=175+2@235+175m) and the width is 43.8m with eight traffic lanes. The height of pylon is 109.6m from the bearing level. The depth of girder varies from 6m at the pylon to 4m at the standard section.

Structural characteristics : YAHAGI-GAWA bridge is a hybrid cable stayed bridge composed of prestressed concrete girders with corrugated steel webs and a steel girder of 127m long, which is mounted at the center support. This is the first application of corrugated steel web to the prestressed concrete cable stayed bridge in the world. The single plane stay cables suspend the bridge composed of five-cell box girders of 43.8m wide. In order to avoid very complicated reinforcing around the anchorages of stay cables at the girder ends prefabricated steel plate anchor beams, which are embedded in the upper and lower decks, are employed.
Because of aesthetic reason the pylon has a curved shape, simulating a drop of water. Since this complicated shape causes large forces in the pylon, steel shell structures are embedded in the concrete pylon as reinforcements instead of conventional re-bars. Against large shear forces set up at the connection between the pylon and column horizontal prestressing tendons, which are curved downward at the end of tendons, are placed to counteract the shear forces.

Construction : The pylon was divided in four sections and the each section was executed in different scaffolding systems suitable to the section. For instance at the middle part, where the pylon has two columns, a climbing scaffolding system was adapted.
The superstructure was constructed in free cantilever method using a traveler. All steel members of corrugated webs, diaphragms and anchor beams, significant re-bars and the formworks for the upper deck were prefabricated as a unit on the ground under the side span and transported to the traveler. The rest of reinforcement of re-bars, external longitudinal prestressing tendons inside the box girder and transverse tendons embedded in the upper deck were placed in the position and concreting the lower and upper decks were followed. This prefabrication allowed the very rapid construction to take place and the bridge was completed within the planned term of construction.
The steel girder of 127m long and 4,250 tons mounted at the center support was erected with the free cantilever method, balancing the weight of girders of both sides and closed to the concrete sections.

Photos : Photo 2-1 shows the completed bridge. The climbing scaffolding system for the pylon can be seen in Photo 2-2. Photo 2-3 show the free cantilever erection with the traveler.

Photo 2-1 the completed YAHAGI-GAWA Bridge

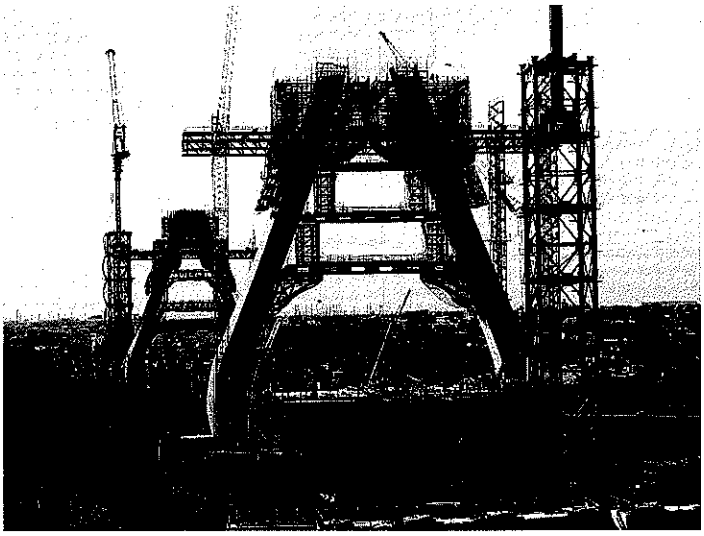

Photo 2-2 Climbing scaffolding for the pylon

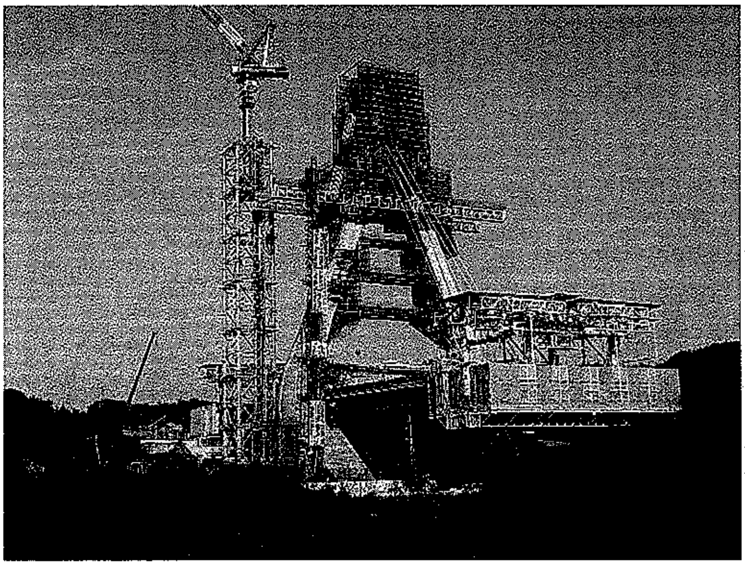

Photo 2-3 Free cantilever erection with the traveler

A COMPOSITE STEEL TRUSS BRIDGE WITH CONCRETE SLABS SHITSUMI-OHASHI BRIDGE

Location : SHITSUMI-OHASHI Bridge is located in a mountain area, 300km west of OSAKA city. It crosses a artificial lake of SHITSUMI dam.

Outline of the bridge : The bridge length is 280m (=65+75+60+45+35m) long and 10.75m wide. Girder depth varies from 6.5m to 2.5m.

Structural characteristics : The bridge is a 5-span continuous composite truss bridge with concrete upper and lower slabs and steel pipe webs. It consists of a composite truss structure from abutment A1 to pier P3 and a conventional prestressed concrete box girder structure for the remaining part to abutment A2. As shown in Fig.3-2, the joint has a shear key and the whole junction is embedded in the concrete slab. The performance was verified by full-size

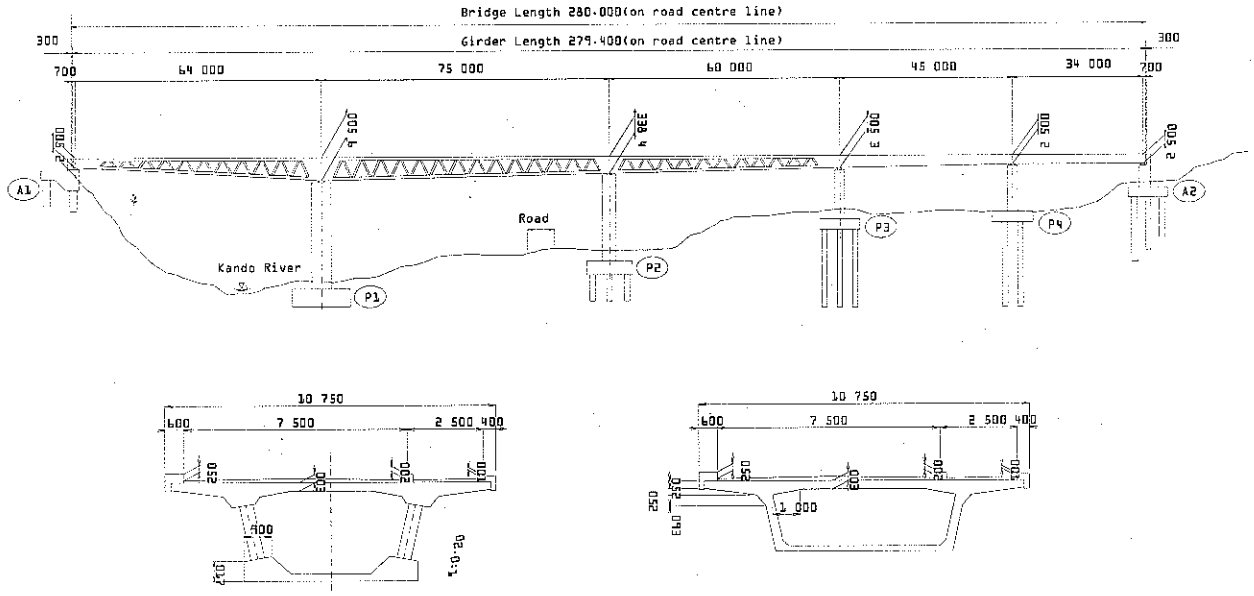

Fig.3-1 Side view and sections

load bearing test and fatigue test. The compressive force to be transmitted to the steel pipe is transmitted to the concrete filling inside the compression pipe through the round steel ribs welded to the inside of the steel pipe, and the force is transmitted to the tension diagonal member through the shear key welded only on the tension diagonal member side. The structure is the same on the upper and lower slab sides.

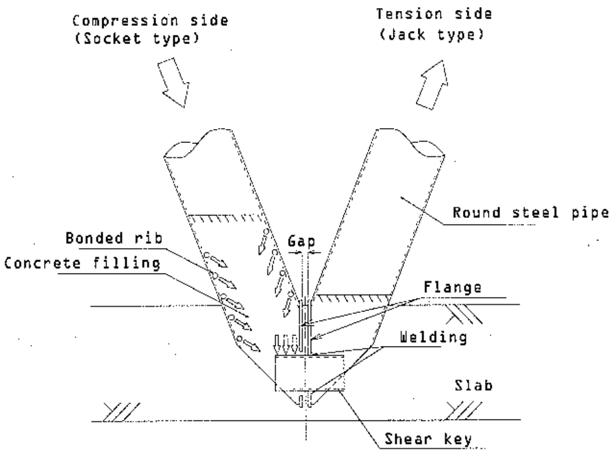

Fig.3-2 The joint structure

Half of the steel truss member (steel pipe) is embedded at the both ends of the column head concrete web. In order to transmit the force from the truss member to the concrete web dowels, studs and steel bars are welded at the embedded surface of the steel pipe. The steel pipe is filled with concrete to reduce not only stress concentration at the bottom of the truss member but also the influence of temperature change between the steel pipe and concrete web.

Construction : A pylon was placed at the pier P1 to suspend the girders temporally and free cantilever construction was executed from pier P1 to abutment A1 and pier P2 using a traveler of the capacity of 3,500kNm. The segment length was 5m and 10 segments were erected on each side. The other spans were in-situ concreted on the shoring.

Photos : Photo 3-1 shows the completed bridge. Photo 3-2 and 3-3 show the joint and the bridge under erection respectively.

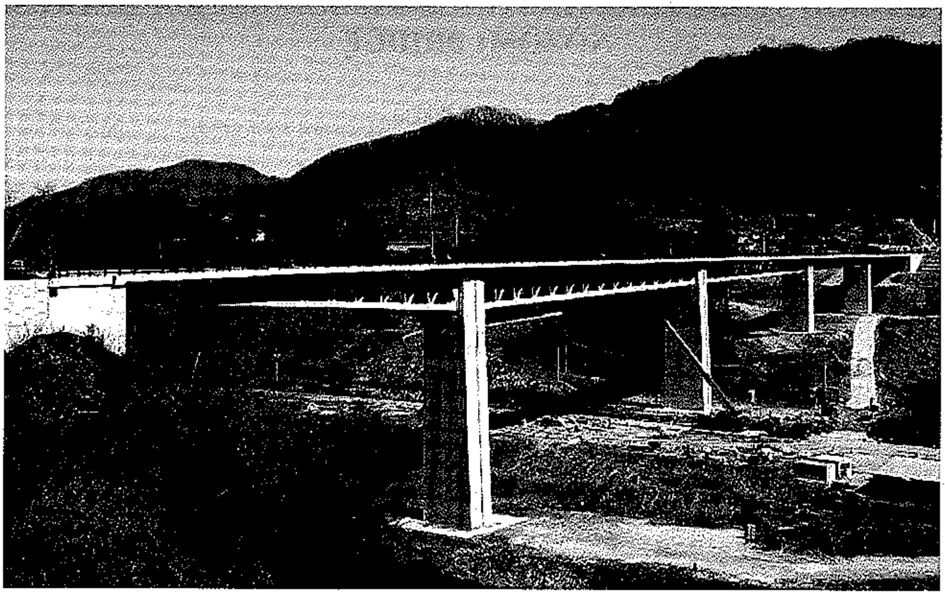

Photo 3-1 the completed bridge SHITSUMI-OHASHU Bridge

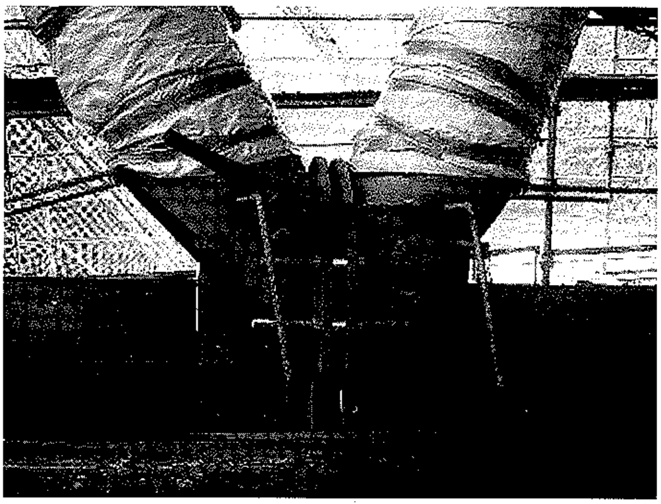

Photo 3-2 joint

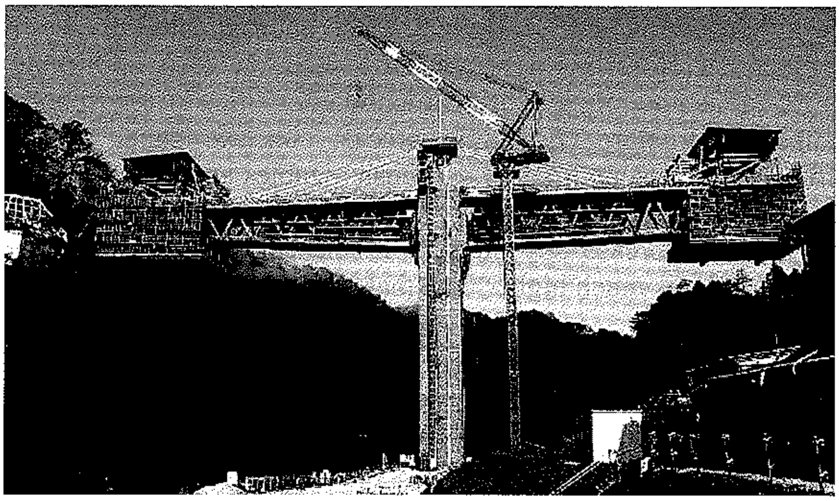

Photo 3-3 the bridge under erection

A HYBRID STRUCTURE OF STRESS-RIBBON DECK AND TRUSS
NOZOMI BRIDGE

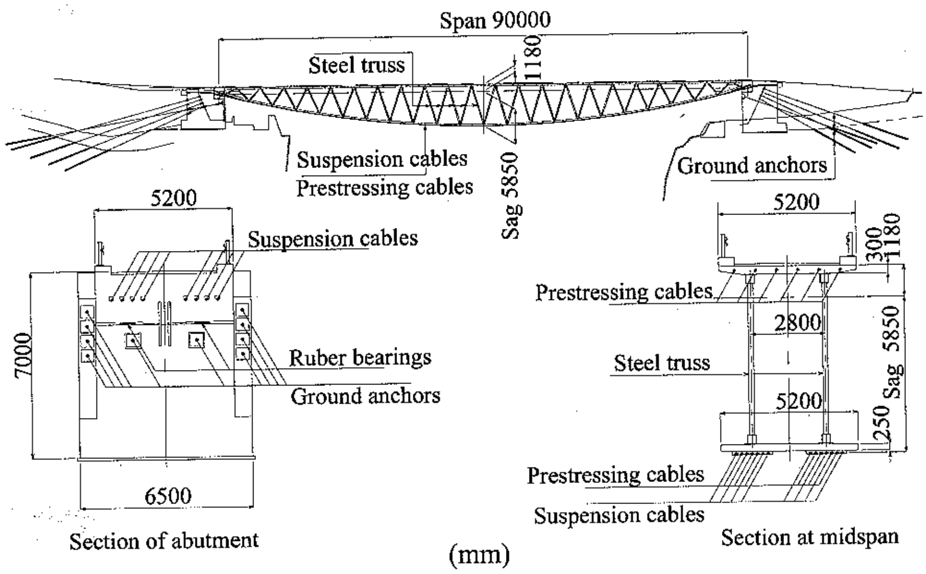

Fig. 4-1 Side view and sections

Location : NOZOMI Bridge is located in front of MARUYAMA Dam over KISO River, 50km north-northeast from NAGOYA city.

Outline of the bridge : The bridge is 90m long and 5.2m wide. Unlike a stress-ribbon bridge, which is generally used as a pedestrian bridge, this is a roadway bridge, although this looks like a stress-ribbon bridge. Since the bridge was planed to provide a access road to the dam construction site, very heavy traffics were expected to pass frequently through the bridge.

Structural characteristics : The bridge is a hybrid structure consisting of a stress-ribbon deck and truss chords consisting of diagonal steel pipes and concrete lower deck. This hybrid bridge has advantages over the stress-ribbon deck bridge since the former exerts much less horizontal force in suspension cables and has higher flexural stiffness than those in the latter. Studies show that the maximum horizontal reactions at the abutment and the deflection at the mid-span due to live load are significantly reduced to approximately the half of those in the stress-ribbon bridge. Therefore this new type of hybrid bridge is applicable to a roadway bridge. Nozomi Bridge is not only a hybrid structure combining stress-ribbon deck and truss, but also a composite structure combining precast concrete panel and steel pipe. The self-weight of truss girder is supported by suspension cables and does not set up any stresses in members of the truss chords. And the surface and the traffic loads are supported by the truss girder and do not increase any stresses in the suspension cable since the flexural stiffness of the truss girder is much higher than that of the suspension cable.

Construction : This bridge has advantages over the truss bridge because the hybrid bridge can be constructed in a similar way of the stress-ribbon bridge without costly false works and erection equipments. Fig.4-2 shows the construction procedure of this bridge.

Photos : Photo 4-1 and 4-2 show the completed bridge and the erection of truss chords.

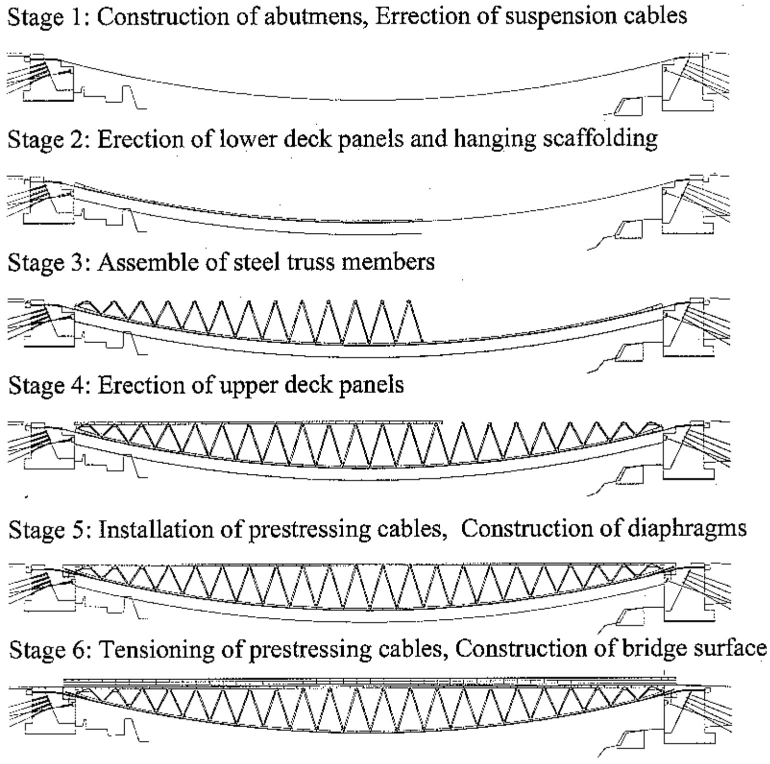

Fig. 4-2 Construction procedure

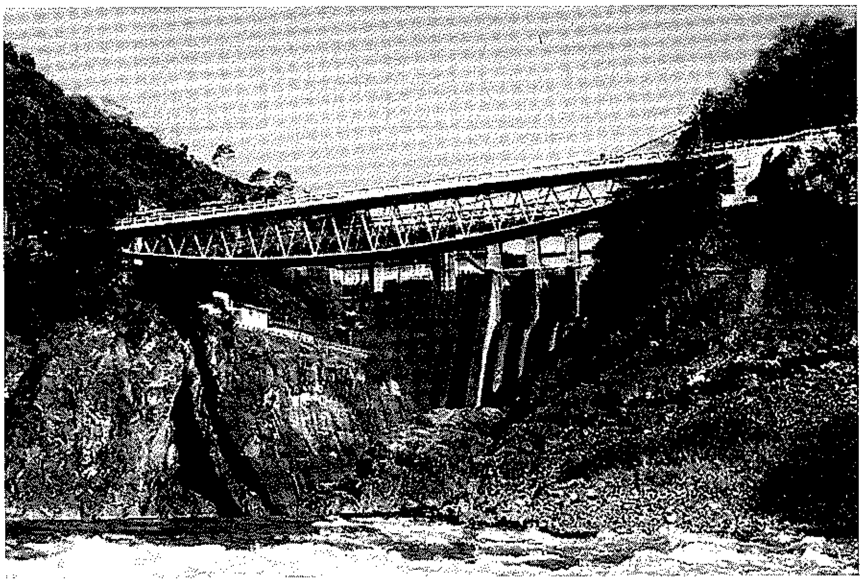

Photo 4-1 Completed NOZOMI Bridge

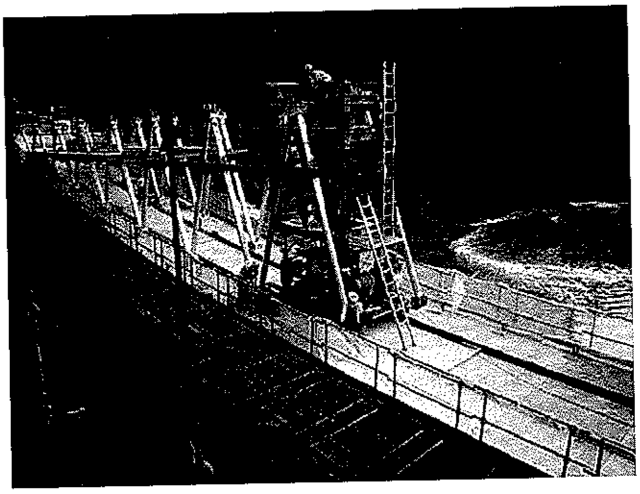

Fig.4-2 Erection of truss chords

A STRESS-RIBBON BRIDGE WITH EXTERNAL TENDONS MORINO-WAKUWAKU BRIDGE

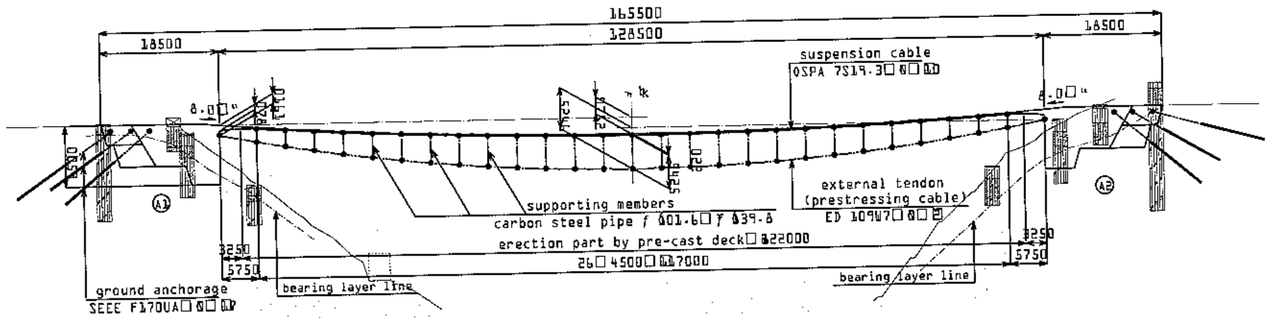

Fig.5-1 Side view

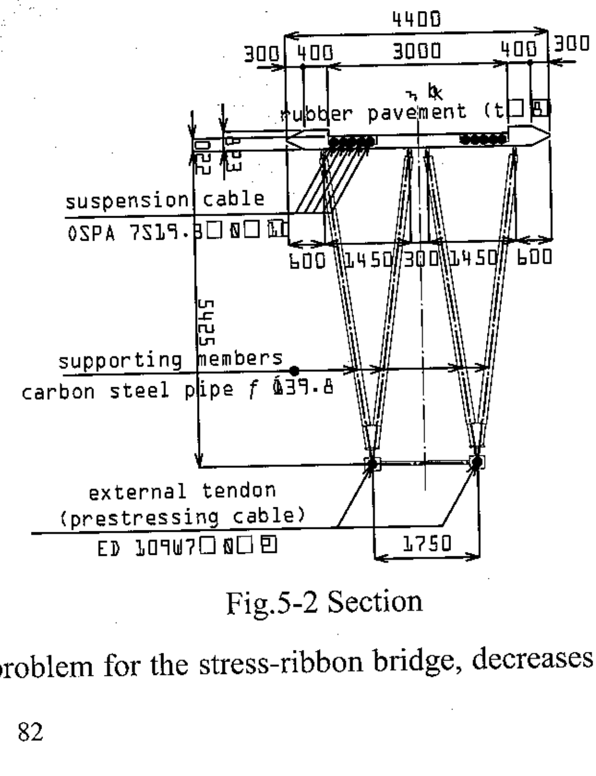

Fig.5-2 Section

Location : MORINO-WAKUWAKU Bridge is located in a park, 200km north of TOKYO.

Outline of the bridge : This bridge is a pedestrian bridge and the first stress-ribbon bridge with external tendons in the world. The bridge is 128.5m long in span and 4.4m wide. The depth of the deck is 22cm.

Structural characteristics : The bridge consists of the conventional stress-ribbon deck and external tendons which are connected to the concrete deck with supporting members (carbon steel pipe). The horizontal force acting on substructures, which is a very troublesome problem for the stress-ribbon bridge, decreases to about 70% by setting larger sag of external cables than that of the concrete deck. The flutter vibration in this bridge is generated with higher wing velocity than that in the conventional stress-ribbon bridge.

Construction : A prefabricated unit consisting of a pre-cast concrete deck, supporting members and a hanging scaffolding was erected with a crane and it was slid on the suspension cable embedded in the concrete deck. After the external tendons were placed in the position they were tensioned to the desired force.

Photos : Photo 5-1 shows the completed bridge and photo 5-2 during sliding erection.

Photo 5-1 Completed MORINO-WAKUWAKU Bridge

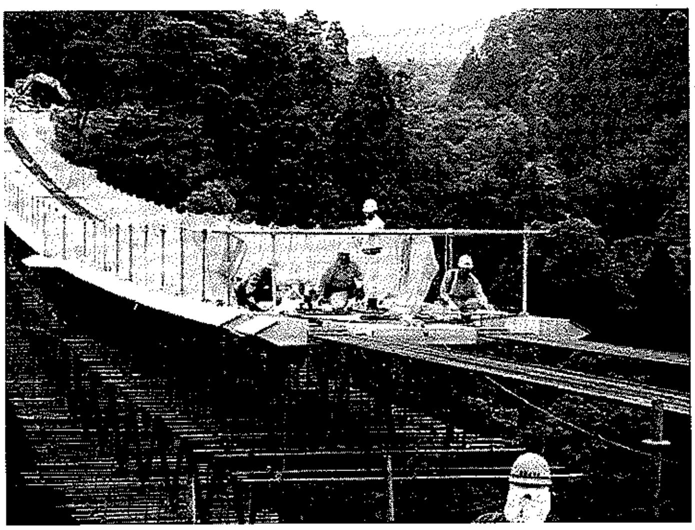

Photo 5-2 Sliding erection

A PRESTRESSED CONCRETE TRUSS BRIDGE
KAMAN-TANI BRIDGE

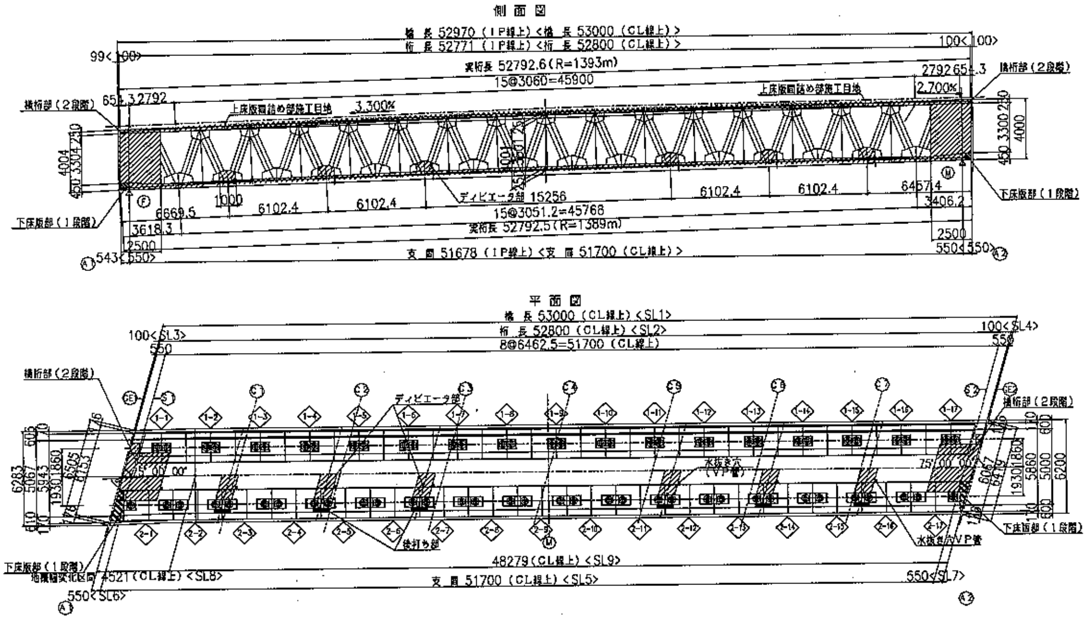

Fig.6-1 Side view and plan

Location : The bridge is located in SHIKOKU Island, 150km southwest of OSAKA city.

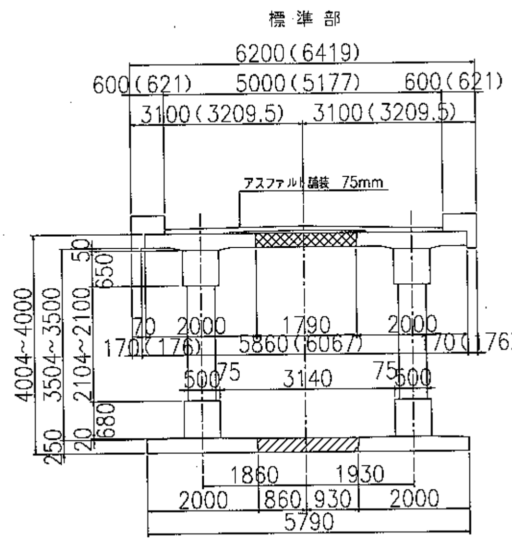

Fig.6-2 Section

Outline of the bridge : KAMAN-TANI Bridge is a prestressed concrete truss bridge, 53m long and 6.2m wide. There are four prestressed concrete railway truss bridges in Japan, while this is the first roadway prestressed concrete truss bridge.

Structural characteristics : Both the diagonal concrete members and the lower concrete deck and the whole structure are internally and externally post-tensioned to counteract tensile stresses set up mainly due to the dead load and live load respectively. Furthermore the joints, diaphragms and deviators are in part prestressed to counteract tensile stresses set up due to stress concentration. The design interests are in the principal tensile stress at the serviceability limit state and the shear resistance at the ultimate limit state in the joint.

Construction : The bridge was constructed in the segmental construction strategy, in which the bridge was divided into two main girders. Fig.6-3 shows a precast segment composed of both the upper and lower deck, two diagonal members and joints. The concrete strength of both the deck and joint and the diagonal member are 50 and 60N/mm^2 respectively. The diagonal members were manufactured in a concrete pile factory for the centrifugal compaction. Five segments were match-cast in a line sequentially. After separating the segments the last one was moved to the first position and the same way was repeated.

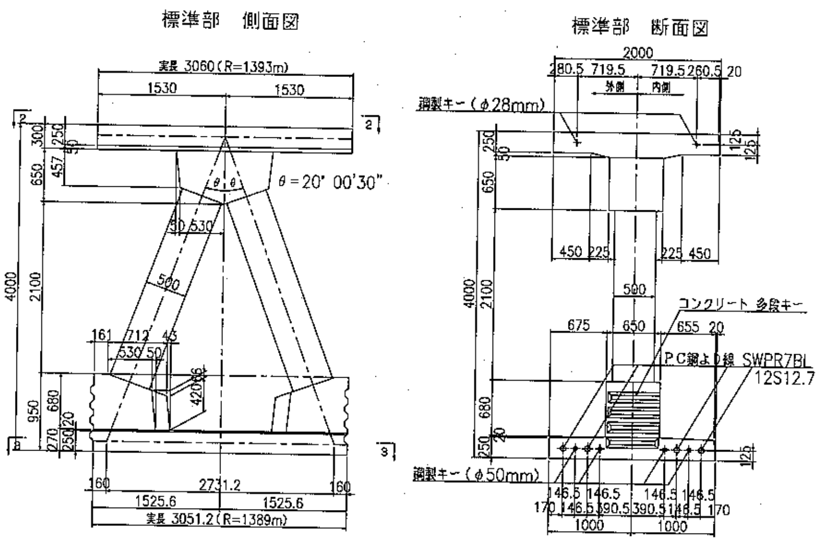

Fig.6-3 Precast segment

Segments were transported to the site, assembled and post-tensioned to put segments together. Two main girders were erected with a launching girder, joints between two main girders were in-situ concreted and the bridge was externally post-tensioned.

Photos : Photo 6-1, 6-2 and 6-3 show the completed bridge, segment prefabrication and erection of a main girder.

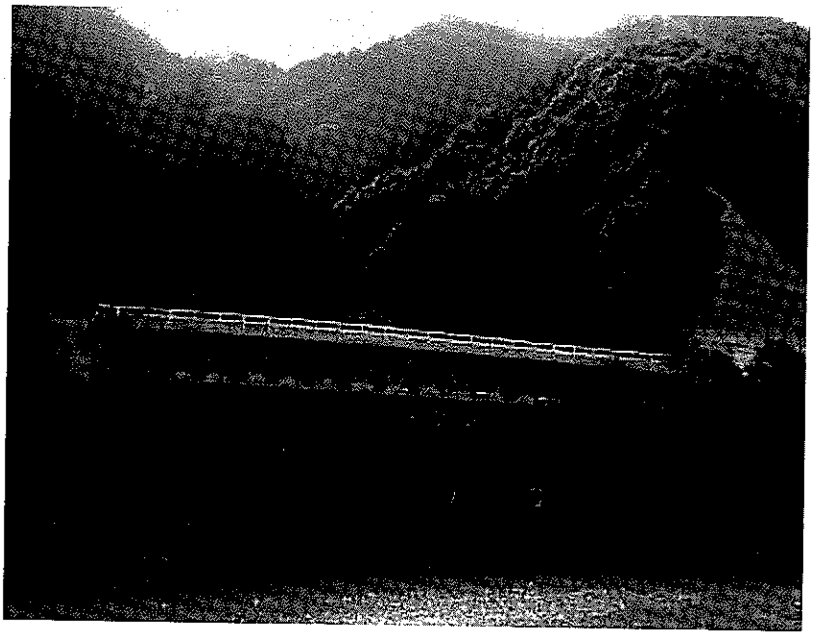

Photo 6-1 Completed KAMAN-TANI Bridge

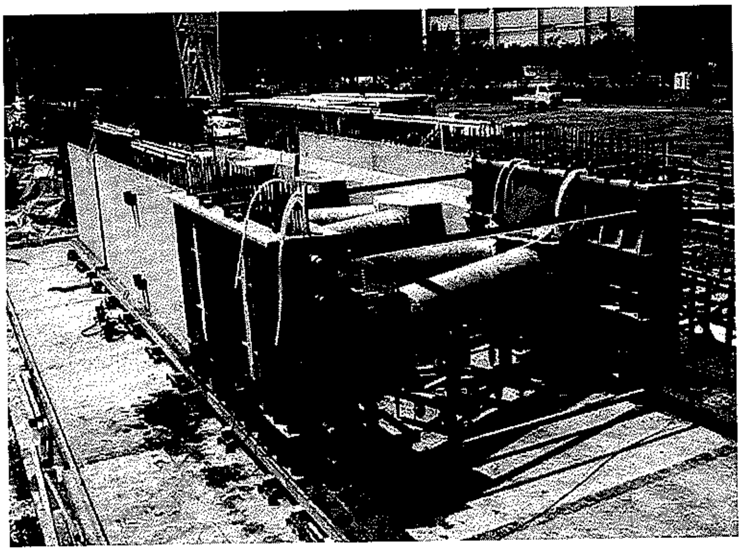

Photo 6-2 Segment fabrication

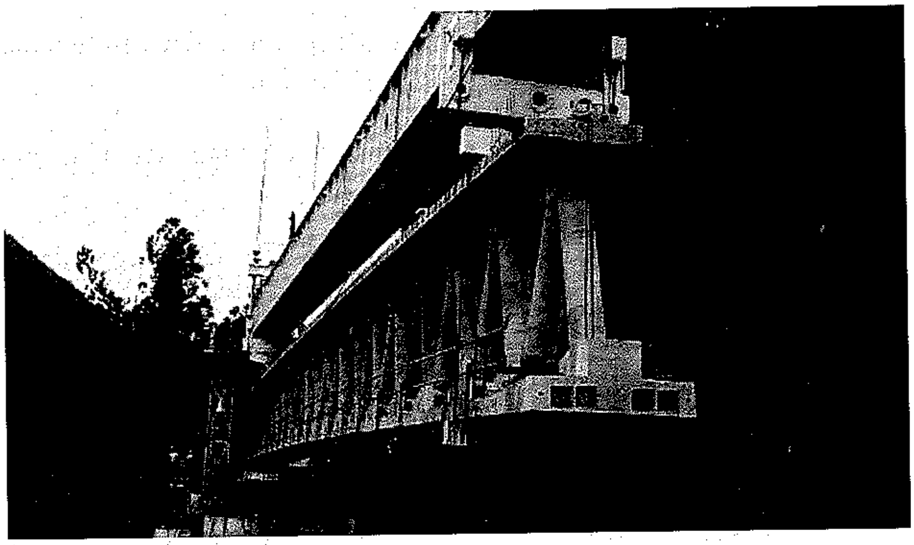

Photo 6-3 Under erection

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STATE-OF-THE-ART REPORT ON HIGH-STRENGTH CONCRETE IN JAPAN —RECENT DEVELOPMENTS AND APPLICATIONS—

Toru KAWAI[1]

SUMMARY

Research and development on high-strength concrete have continuously been done for more than 40 years. This paper describes the past successful technology developments on materials to obtain the workable high-strength concrete and introduces recent enhanced performances as to strength, durability and fire resistance of high-strength concrete. Furthermore, this paper reports some applications of high-strength concrete to structural columns and CFT structures of high-rise buildings and an application of high-strength steel fiber reinforced concrete to a pre-stressed concrete bridge.

Keywords: *High-strength concrete; superplasticizer; silicafume; durability; water-binder ratio; high-rise building; fire resistance; spalling; steel fiber; synthetic fiber; fiber reinforced concrete; CFT; precast block*

INTRODUCTION

Realizing higher strength concrete has been a target or a dream for researchers and engineers engaged in construction industry. In the early 1960's, a superplasticizer was invented in Japan. By the inclusion of the superplasticizer, high-strength concrete could be realized by reducing W/C to under 30%. The high-strength concrete was however applied only to the factory products because it had a large loss in slump. Recent successful researches and developments on materials and construction methods have led to the cast-in place high-strength concrete with good workability, the strength of more than 150 MPa and higher durability. The high-strength concretes have been applied to a lot of high-rise buildings or diaphragm walls. The high-strength concrete however has two defects. One is the occurrence of thermal cracks due to the heat of hydration of a large amount of cement content. To overcome this, many types of low heat cements were developed and utilized. The other is the poor resistance to spalling during fire attack. Inclusion of some synthetic fibers is much effective for reducing the spalling of high-strength concretes.

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Furthermore, high-strength steel fiber reinforced concrete (referred to as "RPC") based on a densest packing theory with heat curing was investigated to exhibit compressive strength of more than 200 MPa with great ductility.
Including above mentioned, this report describes recent developments and applications of high-strength concrete in Japan.

DEFINITIONS OF HIGH STRENGTH CONCRETE

ACI committee presents [1] that in the 1960's, 7500 psi (52 MPa) concrete was considered high-strength concrete and in the 1970's, 9000 psi (62 MPa) concrete was considered the same. The committee also recognized that the definition of the high-strength concrete varies on a geographical basis. In regions where 9000 psi (62 MPa) concrete is already being produced commercially, high-strength concrete might be in the range of 12,000 to 15,000psi (83 to 103 MPa) strength concrete.

Meanwhile, Japan Society of Civil Engineers [2] presents that the high-strength concrete is defined as the concrete which has the design strength of 60 – 100 MPa. Architectural Institute of Japan [3] presents that the high strength concrete is defined as the concrete which has the design strength of more than 36 MPa. JIS A 5308 "Ready-mixed concrete" prescribes that the high strength concrete is defined as the concrete which has the nominal strength of 50 or 60 MPa. Definition of the high-strength concrete has changed over the years and has depended on the country and the organization as mentioned above.

According to a lot of results from the research and the technical reports on high-strength concrete, the characteristics of the concrete in the fresh state and hardened state have a tendency to change on the boundary where the strength is around 60-80 MPa. In this paper, taking above mentioned into account, high-strength concrete is tentatively defined as the concrete having the strength of more than 60 MPa.

TECHNOLOGY DEVELOPMENT

A lot of technology developments which have been done to realize the high strength concrete in terms of material, mix proportion and construction method are introduced here.

Two major remarkable developments of admixtures for high strength concrete are an invention of superplasticizer and a use of silicafume[4,5]. Dr. Hattori developed formaldehyde condensates of beta-naphthalene sulfonates with the primary aim of significantly reducing the water demand of concrete to produce high-strength concrete[6]. Water reductions of up to 30 percent were achieved with the use of this superplasticizer called Mighty 150. This admixture was introduced into the Japanese concrete industry as a nominal name of "high-range water-reducing admixture" in the early 1960's. Since then, it has considerably contributed to produce the high-strength concrete. In Germany, Dr. Aignesberger and his colleagues developed the melamine based superplasticizer having nealy the same performance as the beta-naphthalene based one[7]. These two chemical admixtures however have one common defect that the loss in slump is considerably large. To solve this problem, new technology of slump control with a reactive polymeric dispersant[8] and a steric hinderance theory were studied. As a result, "an air-entraining and high-range water-reducing admixture" was also developed in Japan. This admixture is mixed in the concrete at the ready-mixed concrete plant and the concrete mixture is transported to the job-site to be placed because the loss in slump is relatively small. Since its introduction, this admixture has dramatically increased in use for achieving high-strength concrete and self-compacting concrete.

Under the circumstances, air–entraining and high-range water-reducing admixture was prescribed into JIS A 6204 (Chemical admixtures for concrete). High-strength concrete shows a small yield stress by the inclusion of the chemical admixture but it shows a high plastic viscosity due to a low water-cement (binder) ratio in terms of rheological aspect. This high plastic viscosity generally makes it difficult to achieve the easy placement. A new type of polycarboxylate based air-entraining and high-range water-reducing admixture has been developed. The admixture imparts the high-strength concrete with high deformability and reduces its plastic viscosity in the period immediately after initial mixing until placement, even when the water-binder ratio is 20% or below.

Silicafume is a by-product from the ferro alloys industry. It consists of extremely fine amorphous silica particles. It must be always used to achieve the concrete with the strength of more than 80 MPa. Replacement of the cement by silicafume increases the strength of the concrete and enhances the durability of the concrete[9]. The reasons for this interesting property may be attributed to a microfiller effect and a pozzolanic reaction of silicafume in the cement based products. The particles of silicafume are a 100 times smaller than the cement grains. A mean diameter is approximately 0.1 μ m. Threfore, it is said that the particle of silicafume is smaller than that of smoke of cigarette. The microfiller effect means that silicafume particles are easily introduced into the space between cement grains, thus reducing the space available for water and producing dense structure of hydration products. The pozzolanic reaction means that silicafume particles react chemically with calcium hydoxide to produce well crystalliszed CSH gel and to enhance durability.

Cement content of mix proportions of high-strength concrete is extremely higher compared with normal strength concrete. Binary cement and trinary cement which contain cement and mineral admixtures are commonly used to prevent the thermal cracks due to heat of hydration. Ground granulated blast furnace slag and flyash are sometimes utilized as mineral admixtures. Also, low-heat or moderate-heat portland cement containing relatively high amount of belite is often used to prevent thermal cracks and to reduce an autogenous shrinkage. These properties are desirable for the massive high-strength concrete.

High-strength concrete is not able to be mixed up by the tilting mixer because of the relatively high plastic viscosity. Long mixing for 2 to 4 min. with the revolving-paddle mixer is needed to get the workable high-strength concrete.

HIGH STRENGTH CONCRETE FOR CAST IN- PLACEMENT

Information on principal performances such as compressive and tensile strengths, Young's modulus, rate of carbonation, drying shrinkage and freeze-thaw durability of high-strength concrete with the strength up to around 100 MPa have already been published[10,11]. For these several years, a lot of progressive researches and developments have been conducted to the stage that we can use the high-strength concrete with a water-binder ratio of 20% or below and a design strength of 120 MPa or greater. In this section, principal performances of high-strength concrete with the water-binder ratios down to 12% are shown[12].

Strength

The mix design of the concrete is firstly outlined. Table 1 shows the mix proportions. The binding material was low-heat portland cement as specified in JIS R 5201 with 10% (by mass) replacement by silicafume powder. The water content and the designed air content were 150kg/m^3 and 2.0% respectively in all high-strength concretes. The water binder ratio was varied, with values of 12, 15, 18 and 22%.(These mixtures are referred to as LS12, LS15, LS18 and LS22, respectively.) Two kinds of air-entraining and high-range water-reducing admixtures were used: One was a conventional admixture SPC based on a polycarboxylate polymer with polyethylene oxide and the other was a new admixture SPN based on a polycarboxylate polymer with a new monomer. SPN was developed to impart higher deformability and reduce plastic viscosity of the high strength concrete compared with SPC even if water-binder ratios are under 20%. For comparison, normal strength concrete made from ordinary portland cement, containing an AE water-reducing agent, was prepared (OP55).

Table 1 Mix proportions

No	W/B (%)	s/a (%)	Unit content (kg/m^3)						SP	AEA	Slump-flow (mm)	Slump (cm)	Air (%)
			W	C	LC	SF	S	G	Dosage (B×%)				
LS12	12.0	23.8			1125	125	254		4.0		430		2.4
LS15	15.0	35.7	150	0	900	100	463	836	2.0	0	700	-	1.7
LS18	18.0	41.9			750	83	603		1.5		700		1.8
LS22	22.0	46.6			614	68	729		1.2		675		2.2
OP55	55.0	47.0	176	320	320	0	836	948	0	0.25	-	19.0	4.5

Figure 1 shows the relationship between W/B and the dosages of SPN and SPC to attain the required flowability. Flowability is expressed by slumpflow instead of slump because high flowability (low yield stress) is indispensable for high-strength concrete to achieve the easy placement. The difference between the dosages becomes greater as W/B ratio becomes lower. Even at the concrete with W/B =14%, an SPN dosage of 3.0% was just sufficient. This result pointed out that SPN exhibits much higher dispersibility than SPC and that SPN is more suitable for high strength concrete.

Figure 2 shows the compressive strength. Test specimens were cured not only by standard curing but also by simple adiabatic curing. A simple adiabatic curing means that specimen were formed and cured under sealed conditions in a heat-insulation formwork described in Fig. 3 for 7 days and then were continued to be cured under the condition of in air at 20 ℃ and RH of 60%. The specimens by simple adiabatic curing showed higher strength than those by standard curing at the age of 7 days. Figure 4 shows the temperature hysteresis of specimens cured by simple adiabatic curing. Compared with standard curing, temperatures of the specimens cured were considerably higher during 7days. These date explain the difference of the strengths at the age of 7days. On the other hand, the specimens by standard curing and those by the simple adiabatic curing showed nearly the same strength at the age of 56 days. This corresponds to the common understanding that the higher the early curing temperature, the lower the increase in compressive strength later on. The highest compressive strength of the standard cured specimen reached 170 MPa by LS18. LS 15 and LS12 exhibited only a little bit lower strengths. From these results, compressive strength of the concrete with water-binder ratios of under 20% can reach more than 150 MPa.

Durability

Figure 5 shows the result of freeze-thaw tests up to 300 cycles. Any drastic reduction in relative dynamic modulus of elasticity could not be measured in the specimens of the high strength concrete. It can be concluded that in the high strength range, even if the air content is approximately 2%, excellent freeze-thaw resistance is achieved. The OP55 specimens exhibited good freeze-thaw resistance because it contained 4.5% air entrainment. The mass change was smaller for every W/B compared with OP55. No scaling occurred in the specimens.

Figure 6 shows the changes in length and mass up to the age of 1 year in the drying shrinkage tests. Both of change ratios of high-strength concretes were much lower than those of OP55. As W/B ratio reduces, both of changes become smaller at all ages. These are explained by the fact that the free water content in the hardened concrete reduces, as W/B ratio reduces.

Figure 7 shows the test results of the chloride ion permeation into LS15 and OP55. At depths between 0 to 2 cm from the surface, chloride ion permeation in LS15 is considerably lower than that in OP55. Chloride ion permeation in OP55 is increasing, but that in LS15 is rarely increasing as the number of cycles increases. JSCE [2] introduces the equations for the prediction of the diffusion coefficient of the concrete which are depending on the water-cement ratios. From these results and considerations, durability of high strength concrete is enhanced in terms of permeability.

Figure 8 shows the total pore volume calculated from the measurement of distribution of pore diameters in specimens. The total pore volumes at all ages are noticeably less in high-strength concrete than in OP55. It is natural to understand that the high-strength concretes have much denser structures in the hardened state than the normal strength concrete. Almost no change in total pore volume of high-strength concrete was seen in the period between 28 days and 6 months. This means that the most of the hydration of cement occurs in the high-strength concrete during the early age. Because all of the water-binder ratios of the high-strength concretes discussed here are below 23% which is the ratio theoretically required for complete hydration of cement. On the other hand, total pore volume of OP55 decreases in the period between 28 days and 6 months. The hydration of cement gradually occurs after 28 days in the normal strength concrete.

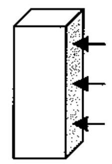
Fire resistance[13]

Figure 9 shows RC specimens with four different strengths after fire resistance test. The degree of the spalling becomes severer, as the design strength becomes higher. It is generally thought from the tests that spalling during fire attack frequently occurs in the high-strength concrete with strength of more than 60 MPa. It becomes serious problems for building structures and sometimes for concrete linings of shield tunnel. Figure 10 shows RC columns made from high-strength concrete after fire resistance tests. The RC column made from plain high-strength concrete was heavily damaged by spalling as shown in Fig.10 (a). On the other hand, the RC column made from high-strength concrete containing polypropylene (hereafter, referred to as "PP") fibers of 0.33% by volume had no spalling as shown in Fig.10 (b). Effectiveness of PP fibers for reduction in spalling was confirmed by this test. PP fibers are known as effective material for the reduction in spalling. The mechanism of spalling reduction is explained on the assumption that PP fibers in concrete disappear at high temperature leaving tubular pores in the concrete through which the high-pressure vapor in the high temperature concrete flows out easily. According to this assumption, in order to clarify the effect of the diameter and length of PP fibers on the reduction in spalling, fire resistance tests on concrete specimens were carried out. Table 2 shows the mix proportion and Table 3 shows experimental specifications.

Table 2 Mix proportion and properties of concrete for small specimens

W/C (%)	s/a (%)	Water (kg/m^3)	Cement (kg/m^3)	Compressive strength (MPa)	Moisture content (%)
25.0	42.1	165	660	113	4.5

Table 3 Experimental specifications

Fiber			Dimensions of specimen (mm)	Heating condition
Diameter(mm)	Length(mm)	Volume (%)		
12, 18, 30, 48	5, 10, 20	0.10	Rectangular prism 100×100×400	 :heat flow
100	12	0.10		
-	-	0		
48	10	0.05, 0.10, 0.30		
100	12	0.05, 0.10, 0.30		

The results are shown in Fig.11. According to the figure, PP fibers with smaller diameter and longer length are desirable for the reduction in spalling. It can be seen from Fig.12 that the higher the volume of fibers in concrete, the less the spalling. Moreover, it is confirmed from Fig.12 that the smaller diameter and the longer length are advantageous for reducing the spalling.

Next, a lot of fire resistance tests on high-strength concrete containing PP fibers were also carried out to identify the relationships between water-cement ratio and degree of spalling using reinforced concrete elements with different configurations. According to the results, 'spalling reduction ratio' α which is an index of the reduction in spalling by PP fibers was calculated. Figure 13 shows the relationship between calculated spalling reduction ratio and volume percentage of fibers in concrete, and Figure 14 shows the relationship between calculated spalling reduction ratio and W/C ratio of concrete. Each relationship seems to have a linear relation and a regression equation is defined as Eq.1.

$$\alpha = V_f \times \left(43.9 \times \frac{W}{C} - 7.43 \right) \quad (1)$$

where α : spalling reduction ratio

V_f: volume percentage of fibers in unit volume of concrete, (vol%)

From the equation (1), it is pointed out that the degree of spalling depends on volume of PP fibers and water-cement ratio of the concrete.

Applications To High-Rise Buildings

Applications of high strength concrete to RC columns

The application of high-strength concrete with strength of more than 60 MPa to columns of high-rise buildings started in North America in 1970's[14]. Meanwhile, the application of the concrete that possesses a design strength of 100 MPa or more to columns of high-rise buildings is increasing in Japan and now we have more than 20 applications. Through the use of high-strength concrete floor areas can be increased because it allows the sectional area of building columns to be reduced. In this section, one of the construction records of high-strength concrete applied to a high-rise building with the strength exceeding 150 MPa is reported[15].

The building was a 45-story apartment building in Tokyo as shown in Fig.15. High-strength concrete was applied to the columns from the first to the third floors. Total volume of the high-strength concrete was 700 m^3. Table 4 shows the mix proportions. The compressive strength required from the structural design was 130 MPa and the W/B ratio was 18%. The mix proportion was determined from a lot of results of the field tests conducted in winter and in summer using mock-up columns. The binder consisted of 70% ordinary portland cement, 20% blast-furnace slag and 10% silica fume, and it had been premixed at the cement factory as a trinary cement for high-strength concrete. Compressive strength of cores taken from the mock-up column (800×800mm) made with the concrete using the same binder is described in Fig.16. This binder was proven to develop a high compressive strength up to an age of 10 years. Crushed andesite stone and sand were selected as aggregates for high-strength. A polycarboxylate-type air-entraining and high-range water-reducing admixture was used. Synthetic fibers for preventing spalling in case of a fire were added.

Table 4 Mix proportions

Fc	W/B	Unit content(kg/m^3)						Slumpflow	Air
(MPa)	(%)	W	B	S	G	Fiber(vol%)	SP	(mm)	(%)
130	18	150	834	647	835	0.22	12.9	650	1.0

(Note)Requirements: Water content (Not more than 169kg/m^3), Slumpflow(600-700mm), Air content(0.5-2.0%)

The concrete was primarily conveyed with a concrete bucket within the construction site as shown in Fig.17 while being partially pumped. At the jobsite, quality control tests were carried out as to air content, slumpflow, water content and compressive strength as shown in Fig. 18. All the results of water content, slumpflow and air content satisfied the requirements. The test results of compressive strength are shown in Fig. 19. The average compressive strength of standard-cured specimens at the age of 56 days exceeded 150 MPa, with the coefficient of variation being as low as 2.0%. During construction, two mock-up columns were constructed with the same high-strength concrete on different days. The top and bottom ends were insulated to correspond the temperature conditions of the mock-up columns to those of the actual columns. The temperature histories near the center of the mock-up columns are shown in Fig. 20. The maximum temperatures of the first and second mock-up columns were 57°C and 62°C, respectively. Fig. 21 shows the compressive strength of the drilled cores. The strengths in both columns satisfied the requirement of 150 MPa at the age of 56 days.

Application of high strength concrete to CFT

Concrete-filled tubular (referred to as "CFT") structure was firstly adopted in 1990's in Japan. The development of the high-strength self-compacting concrete made it possible to fill the concrete completely into a steel tubular column having a number of diaphragms inside. CFT structure has rapidly been prevailing in Japan with the strength requirements for concrete becoming more demanding. In this section, an application record of CFT structure with high-strength concrete to a high-rise building is reported[16].

Building B is a 36-story high-rise office building located in Tokyo as shown in Fig.22. Columns up to 90m.above ground were designed in CFT structure. The main columns were square with a cross-sectional size of 800 by 800 mm as shown in Fig.23. Square columns were strengthened with internal diaphragms, each having a round opening in the center with a diameter of 200 to 400mm, through which concrete passed. Concrete was pumped from the ground (see Fig.24) to the allowable level at which the lateral pressure was expected to reach the permissible stress of the welds. Concrete buckets were used for filling concrete in the upper parts of square columns above the pumped level (see Fig. 25).
Circular columns with a diameter of 800 mm were partially used. Since circular columns were found resistant to the lateral pressure by calculation, concrete was placed into circular columns by pumping from the ground to a height of 90 m in a single lift.

Building C is a 47-story high-rise condominium located in Tokyo as shown in Fig.26. The columns at from the first to twelfth floor were designed in CFT structure using concrete with a design strength of 100 MPa. The cross-section of the main columns was a square measuring 900 by 900 mm. Table 5 shows the mix proportions of the high-strength concrete with water binder ratio of about 20% and slumpflow of 650mm. The binder was 70% ordinary portland cement, 20% blast-furnace slag and 10% silica fume.

Table 5 Mix proportions

Plant	Fc (MPa)	W/B (%)	Unit content (kg/m^3)					Slumpflow (mm)	Air (%)
			W	B	S	G	SP		
A	100	20	155	775	634	862	13.6	650	2.0
B	100	21	150	714	676	894	14.2	650	2.0

In advance of an actual construction, a trial placement of the concrete into a mock-up column of the actual size was conducted. The concrete was placed with a bucket. Sections of the mock-up column are shown in Fig. 27. The mock-up column was successfully filled with concrete. Figure 28 shows a section of the mock-up column cut by a wire saw. This experiment revealed that the high strength concrete with a high viscosity was filled in all corners of the column to the bottom, including those under diaphragms, without any problem. Figure 29 shows the results of quality control. The average of the compressive strengths of specimens by standard curing was 135 MPa, exceeding the required strength.

Application of fire resistance concrete

PP fibers are known as effective material for the reduction in spalling. Newly developed polyacetal fibers (referred to as "PA"), which are more effective to resistance to spalling, are being used instead of PP fibers. An application of PA fibers is introduced here.

Building D is a 40-story high-rise condominium located in Tokyo as shown in Fig.30.

High-strength concrete was applied to the columns from the first to the eighth floor and the number of the columns was 294. The compressive strength required from the structural design was 80-120 MPa. High-strength concrete with a strength of 120 MPa was applied to columns at the first floor. The cross-section of the columns was a square measuring 1000 by 1000 mm. Table 6 shows the mix proportions of the high strength concrete with a water binder ratio of 15.6% and slumpflow of 650mm. The binder was 90% low-heat portland cement and 10% silica fume.

Table 6 Mix proportions

Fc (MPa)	W/B (%)	Unit content(kg/m^3)						Slumpflow (mm)	Air (%)
		W	B	S	G	Fiber(vol%)	SP		
120	15.6	145	929	541	875	0.267	12.9	650	2.0

PA fibers (see Fig.31) have been used instead of PP fibers at this project. Figure 32 shows the melting performance of PP and PA fibers. PA fibers melt away at lower temperatures than PP fibers. In advance of an actual construction, fire resistance tests based on mock-up RC columns made with high-strength concrete were conducted to confirm the anti-spalling performance of the two fibers. Figure 33 shows the appearances of the RC columns after fire resistance test. The column containing no fiber suffered heavy damage from spalling. On the other hand, the columns containing fibers had resistance to spalling. But the degree of resistance depended on the kind of fiber and the dosage of the fiber. According to the result, the column containing PA fibers had higher resistance to spalling than that containing PP fibers even if the dosage of PA fibers was smaller than that of PP fibers.

As the strength of concrete increases, the degree of spalling during fire attack becomes severe. Therefore, PP fibers are applied to the concrete with strength of 80-100 MPa and PA fibers are applied to the concrete with strength of 120 MPa or more. Required qualities of the high-strength concrete have been confirmed and the work is now being successfully conducted (see Figs. 34 and 35).

HIGH STRENGTH STEEL FIBER REINFORCED CONCRETE

In Europe, high-performance steel fiber reinforced cementitious composite referred to as Reactive Powder Concrete (RPC) was developed[18]. Steam curing at 90℃ and densest packing design enable to produce precast concrete having high-performance and ultra high-strength of around 200 MPa. The actual applications of RPC have been done by around 35 projects in the world. Recently some applications have been done in Japan and Recommendations for the concrete[19] was established by JSCE. In this section, main features as to mix proportion, properties in fresh and hardened states, mechanical properties, durability and construction method of the first application to pre-stressing concrete bridge in Japan is reported.

Table 7 shows the mix proportions. Water-cement ratio is around 23% and water-binder ratio is 12%. Cement, quartz and silicafume particles are well balanced to obtain the densest packing described in Fig.36. This packing design achieves high-strength and high durability. RPC shows good flowability (self-leveling property) to be cast into the thin mold or into the complicated shaped mold. It takes 8-14min. to be mixed to get the specified flowability even by the revolving-blade mixer.

Table 7 Mix proportions

Steel fiber			Unit content(kg/m^3)					Flow value (mm)
Diameter (mm)	Length (mm)	Volume (%)	W	C	Grain (quartz, sand etc.)	Steel fiber	SP	
0.2	15	2	180	774	1523	157	22(liquid)	240-260

The compressive strength of RPC is 200-240 MPa and the tensile strength is around 9 MPa after 90℃ heat curing for 48 hours. Figure 37 shows a relationship between typical load bending stress and deflection of a beam. The first initial crack stress (25-30 MPa) mainly depends on the flexural toughness of mortar matrix and the ultimate flexural stress (40-45 MPa) depends on the bridging effect of steel fibers. RPC has very high energy absorption capabilities. The high tensile strength combined with enough ductility makes conventional reinforcement unnecessary. The 15mm long steel fibers inside of the mortar matrix act as reinforcement to resist for tensile stress.

RPC forms a dense matrix structure due to the densest packing design and pozzolanic reaction. Test results present that RPC has an excellent freeze-thaw resistance and an excellent abrasion resistance. The microstructure of the RPC is so dense that it is difficult to determine the diffusion coefficient of chloride ion. The diffusion coefficient determined by special methods by the use of EPMA analysis or electrical permeation test method ranged between 1.9 and 2.2 $\times 10^{-10}$ cm^2/sec. This value is approximately 1/100 of that of conventional high-strength concrete. Figure 38 compares the calculated chloride profiles within the specimens exposed to a surface chloride content of 3.0kg/m^3. In the case of RPC, the depth at which the chloride concentration reaches the threshold value for corrosion (1.2kg/m^3) is approximately 1/10 of that of conventional high-strength concrete.

RPC premix, water and a chemical admixture were mixed as a primary mixing. After checking the flow value, steel fibers were added to the mixture and it was mixed as a secondary mixing for 7 min. The flow value was checked again (see Fig.39). Mixed RPC was placed through a tremie attached to the hopper outlet to manufacture the precast blocks described in Fig.40. After placing, the block was cured by sheets to prevent water evaporation as a primary curing. For secondary curing, 90℃ steam curing was conducted for 48 hours in a house. The rate of rising and dropping temperature were controlled to15℃/h and 7 to10℃/h, respectively to prevent the cracks due to temperature difference in the blocks. Transverse wet jointing of blocks was conducted in the plant prior to installation(see Fig.41). After blocks were fixed on both sides, RPC was poured from the bottom slab to the top slab. The joints were heat-cured by electrical heaters and insulation. The specified strength was achieved as temperature between 70 and 90℃ was maintained at joints.

CONCLUDING REMARKS

More than 40 years have passed since a high-strength concrete was realized. The high-strength concrete itself has been enhanced in terms of performance and placeability through a lot of useful developments to the stage that now many applications have been done in north America, Europe, Japan and other countries. In the near future, the high-strength concrete will be prevailing and increasing in use all over the world.

Meanwhile, the fact is that to acquire the much higher strength of the concrete has been one of the dreams and is also one of the incentives to research and development for researchers and engineers. Many reliable data as to the relationship between the total pore volume and compressive strength exist. Extrapolating the date, the compressive strength can theoretically reach 700 MPa as the total pore volume of the concrete will be reduced to be in the vicinity of zero. Research and development on high-strength concrete making full use of more advanced technologies will continue until the strength of the concrete reaches this limit.

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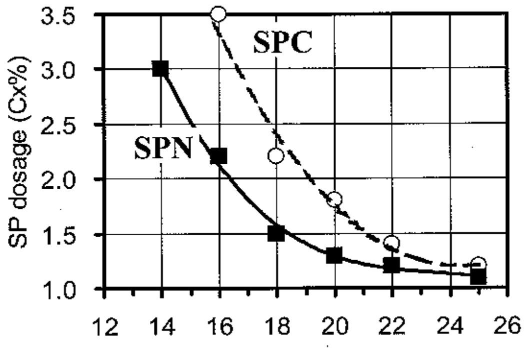


Fig. 1 Comparison of SP dosage

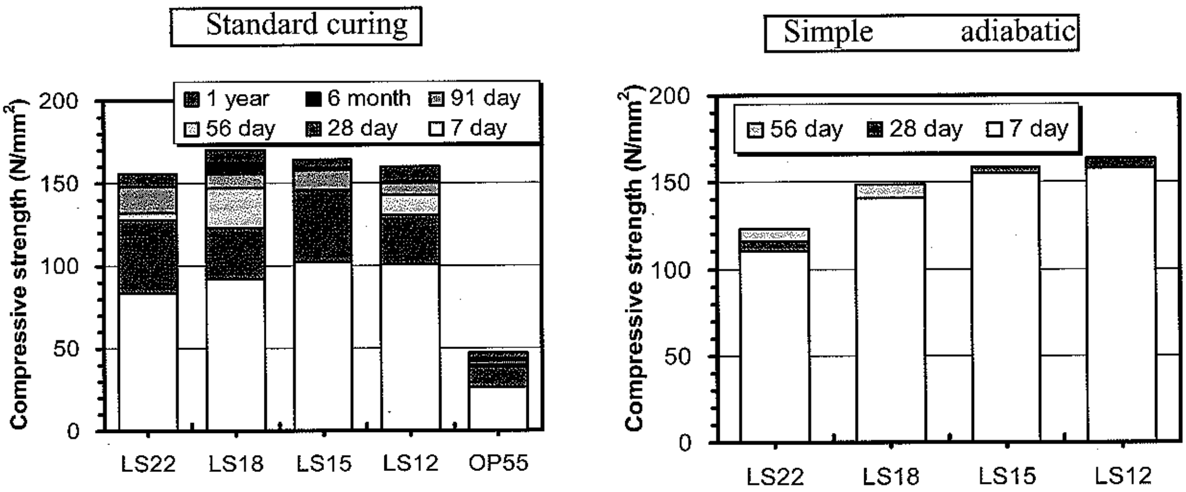


Fig. 2 Compressive strength

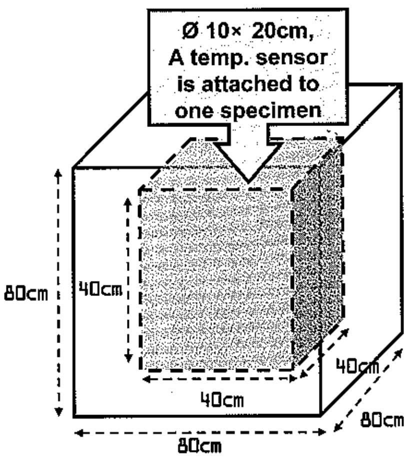


Fig. 3 Diagram of simple adiabatic curing method

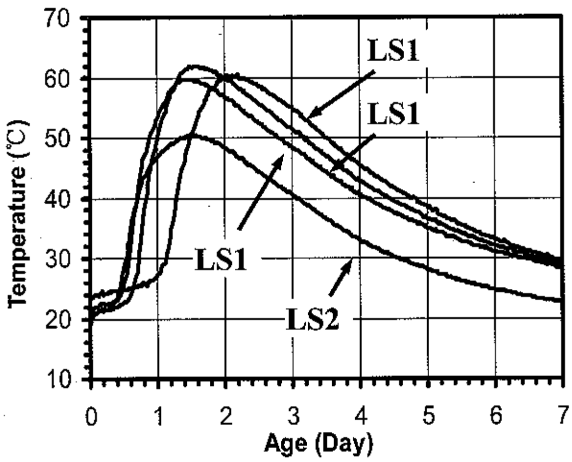


Fig. 4 Temperature hysteresis of specimens cured by simple adiabatic curing

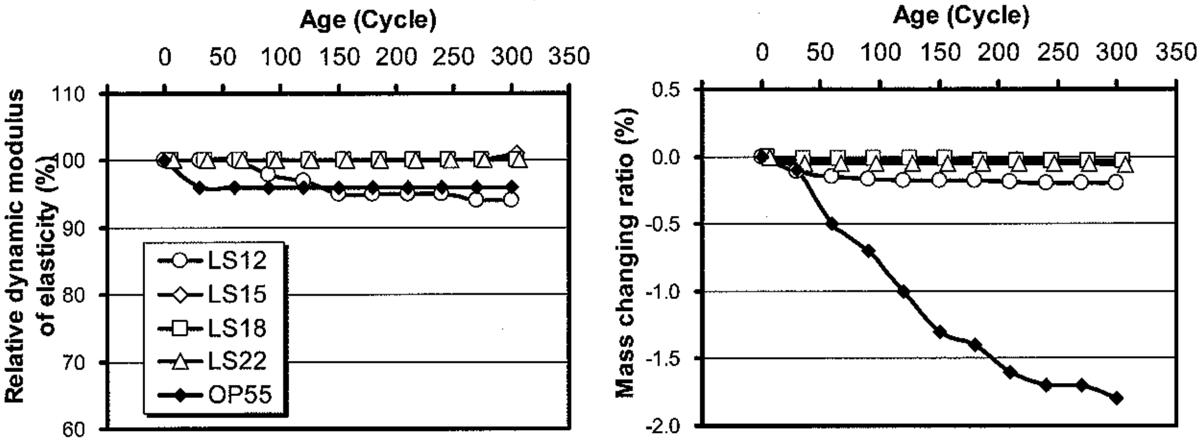


Fig. 5 Freeze-thaw test

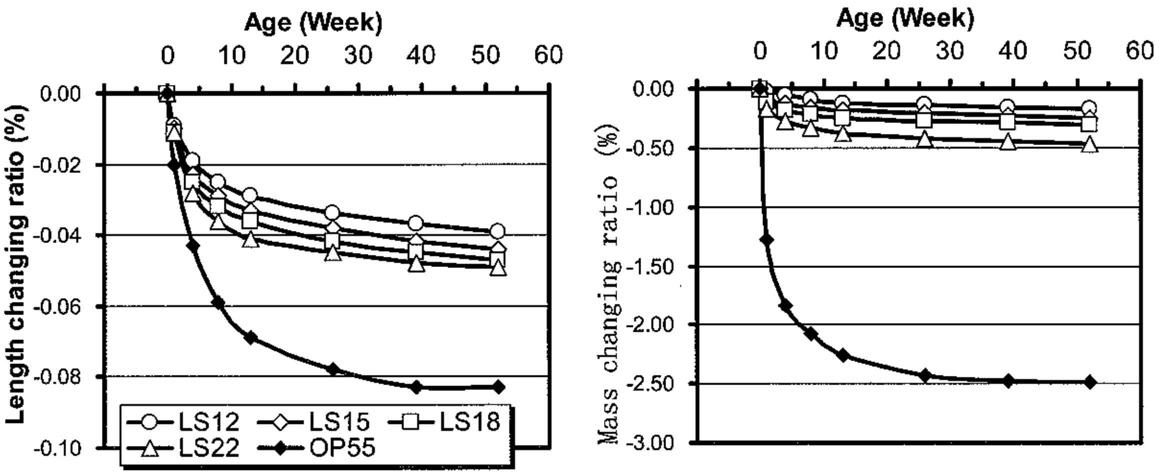


Fig. 6 Changes in length and mass

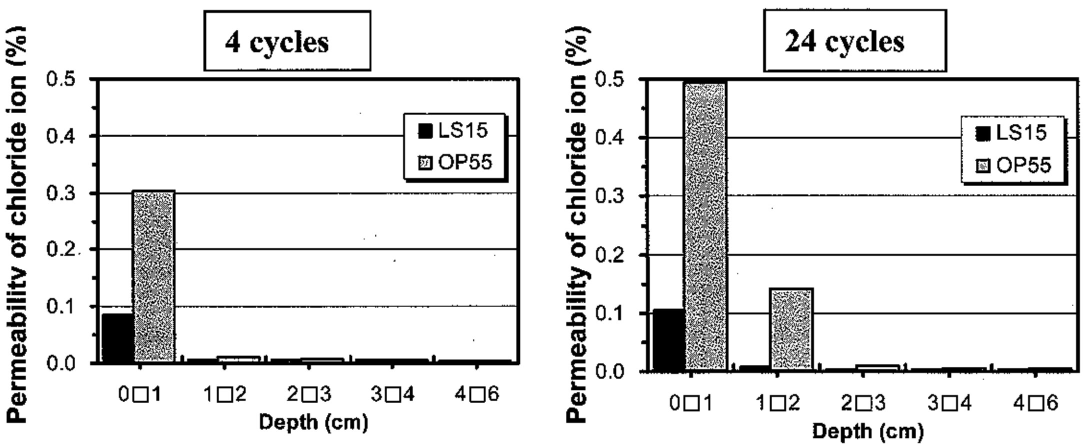


Fig. 7 Chloride ion permeability

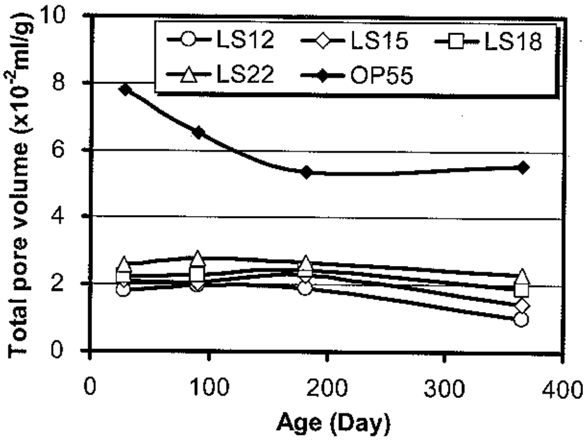


Fig. 8 Total pore volume

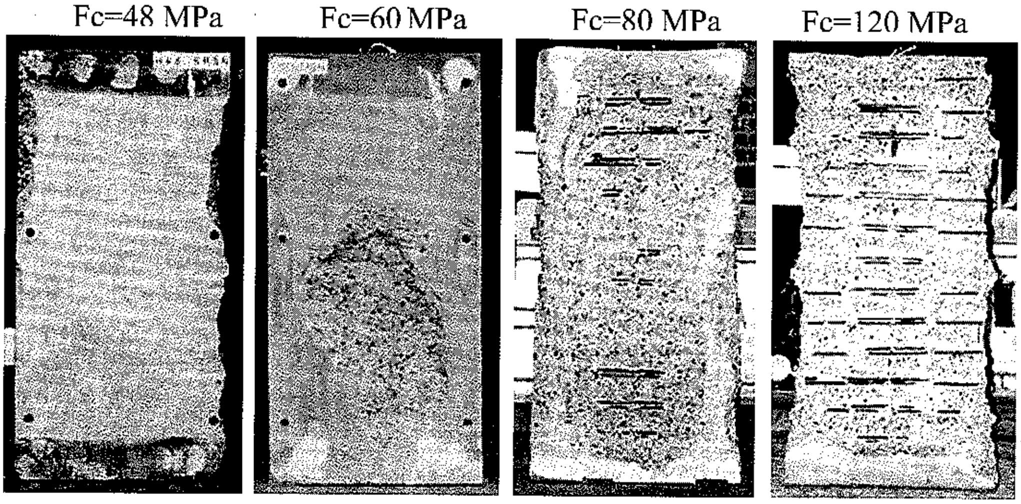


Fig. 9 RC specimens after fire resistance test

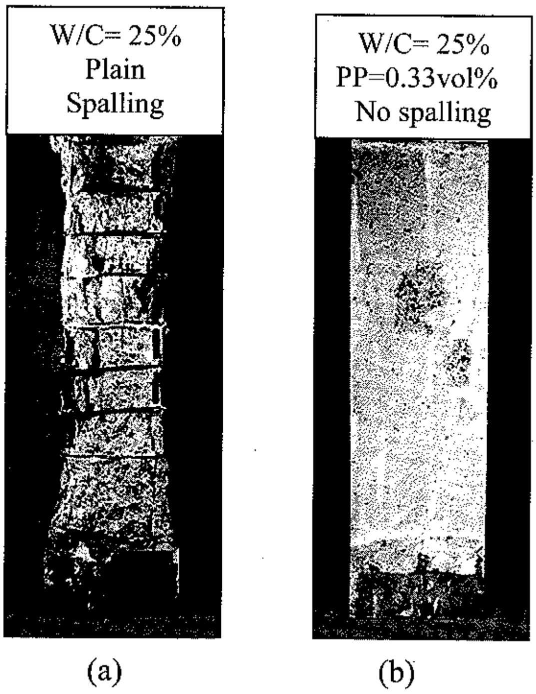


Fig. 10 RC columns after fire resistance test

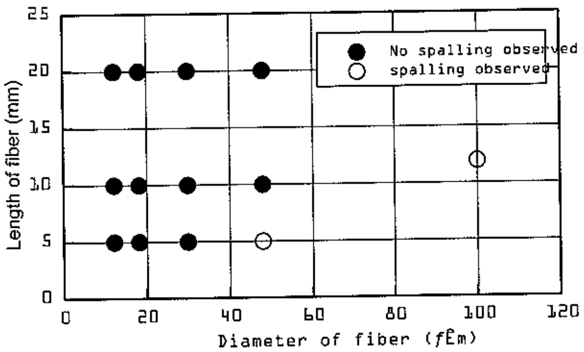


Fig. 11 Influence of diameter and lengths of synthetic fibers on spalling reduction

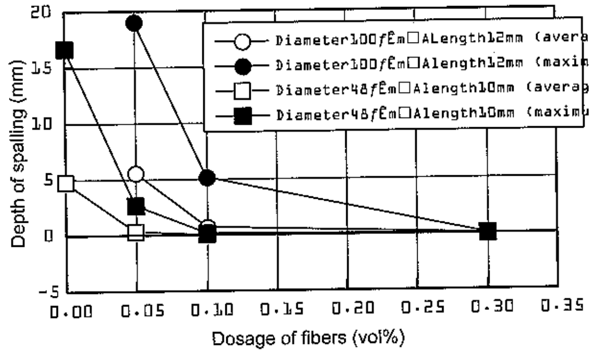


Fig.12 Relationships between volume of synthetic fibers in concrete and spalling reduction

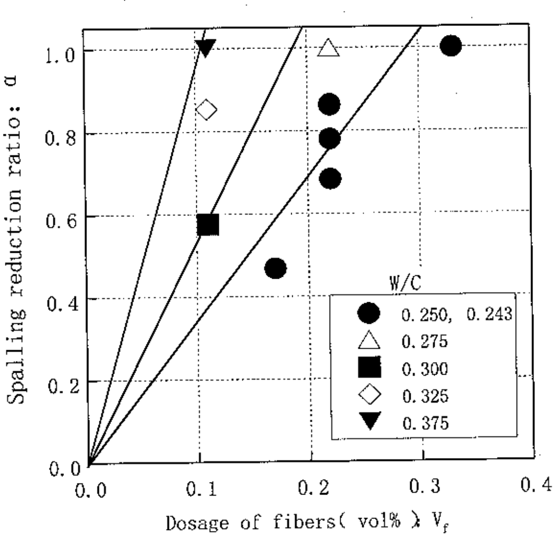


Fig. 13 Volume percentage of fibers. vs. spalling reduction ratio

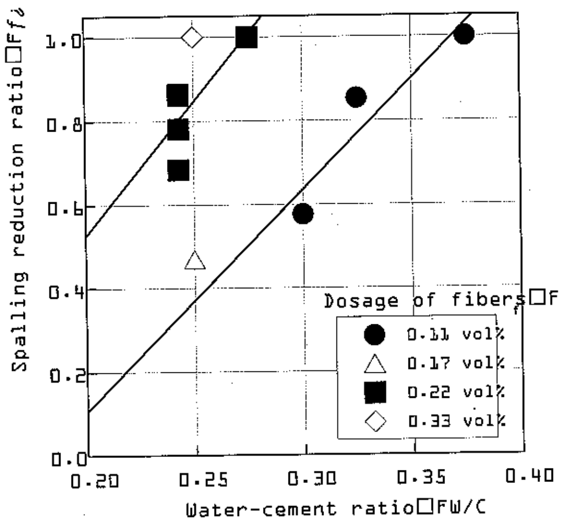


Fig. 14 W/C ratio vs. spalling reduction ratio

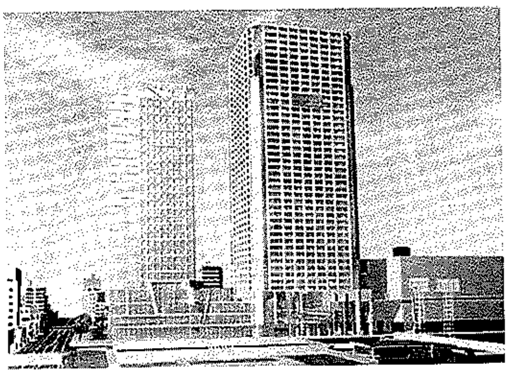


Fig. 15 The high-rise building

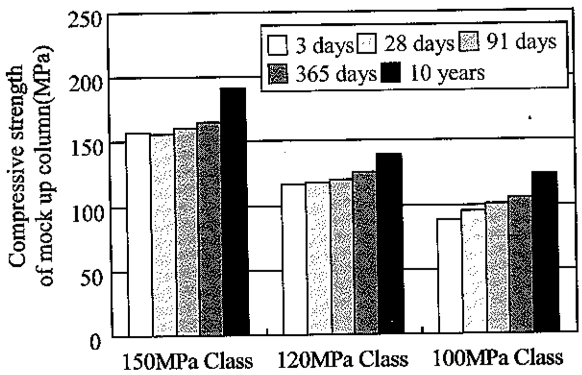


Fig. 16 Long-term compressive strength of mock-up columns using developing binder

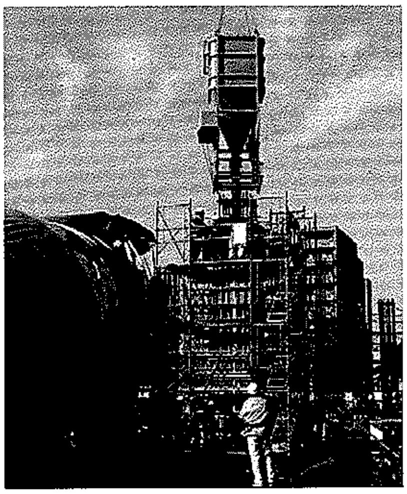


Fig. 17 Placement with a bucket

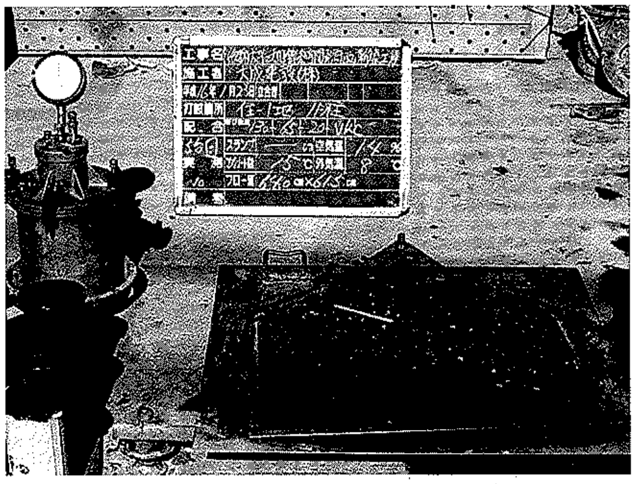


Fig. 18 Quality control test

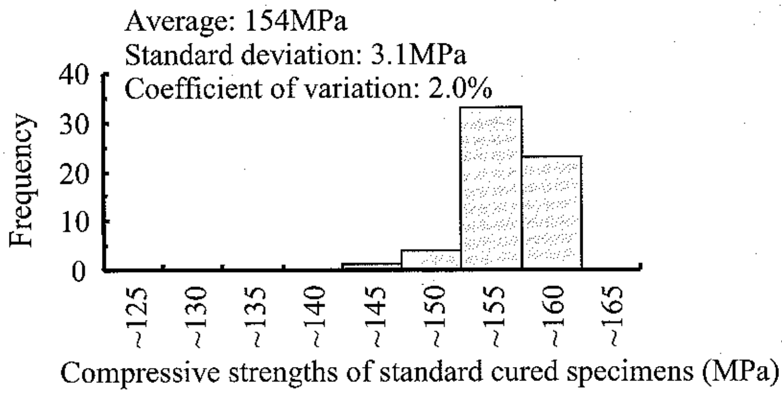


Fig. 19 Results of compressive strength

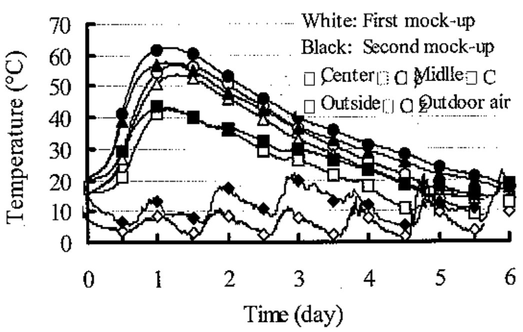


Fig. 20 The temperature histories of the mock-up columns.

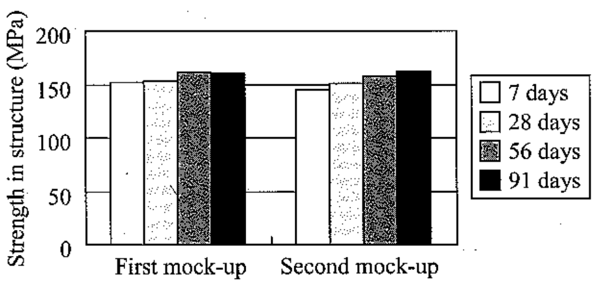


Fig. 21 Compressive strength of concrete in structure.

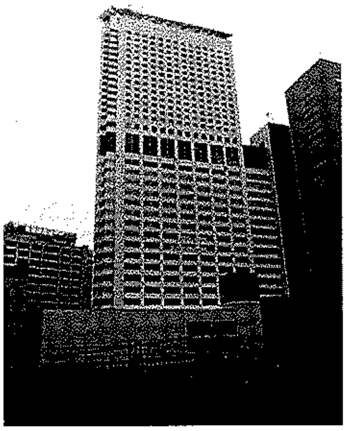


Fig. 22 Office building

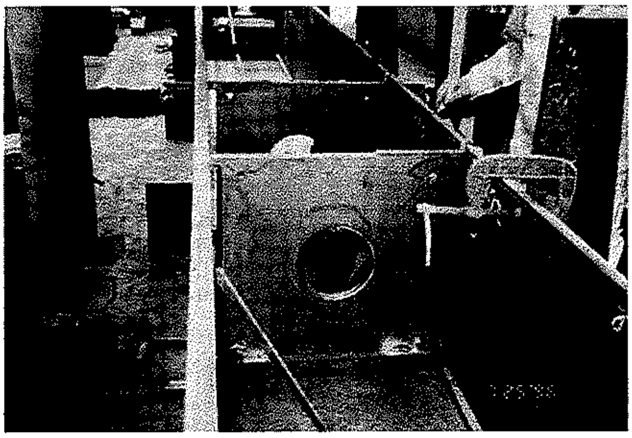


Fig. 23 Diaphragms in a steel tubular column

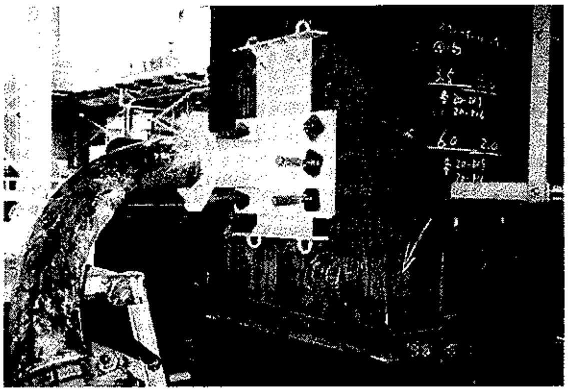


Fig. 24 Placing with a concrete pump

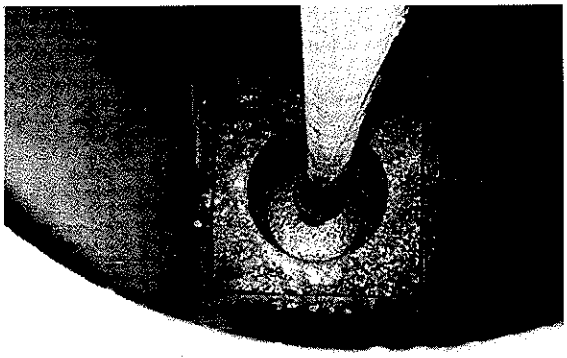


Fig. 25 Placing with a concrete bucket

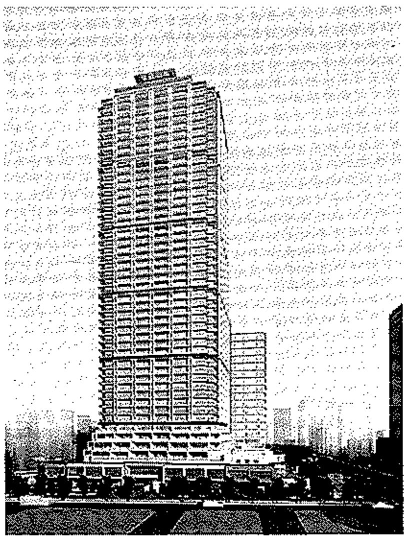


Fig. 26 Condominium

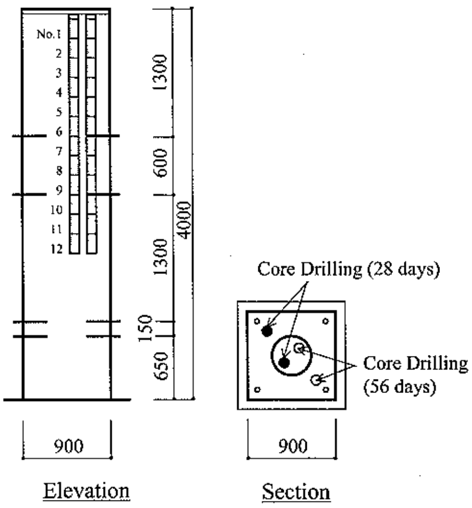


Fig. 27 Sections of mock-up column

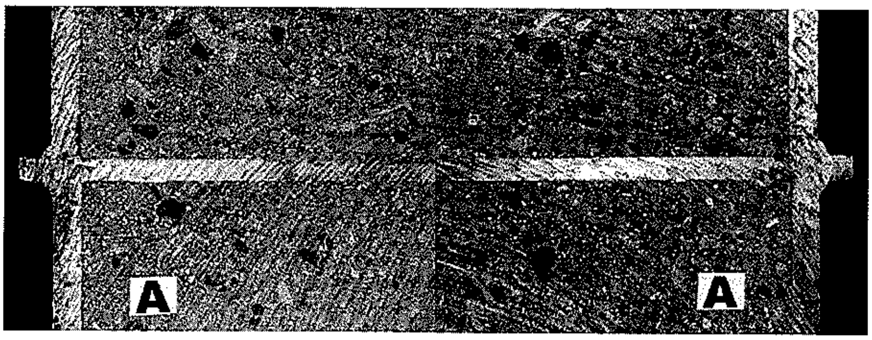


Fig. 28 A cut section of mock-up column

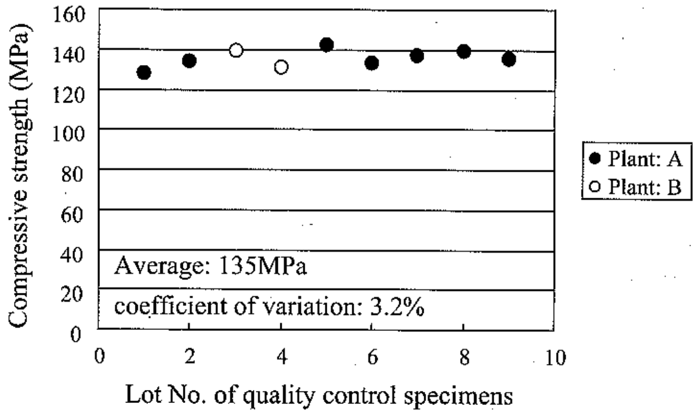


Fig. 29 Results of quality control of 100 MPa concrete

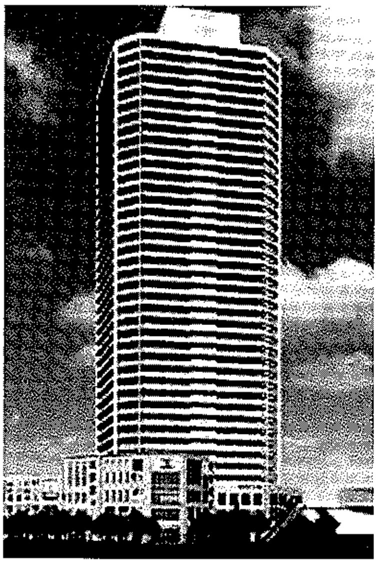


Fig. 30 Condominium

Fig. 31 Polyacetal fibers

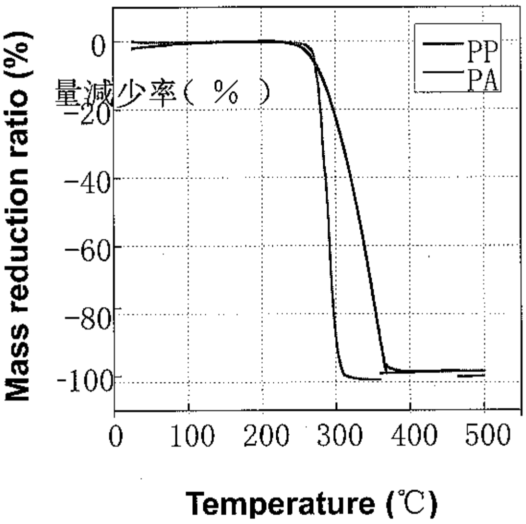
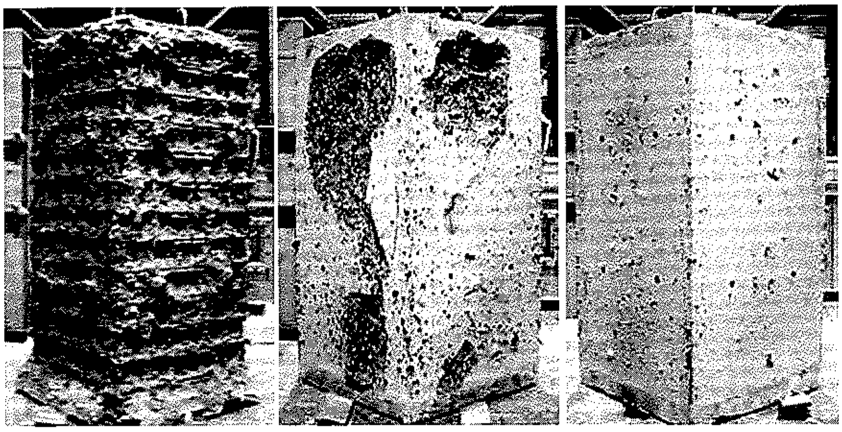


Fig. 32　Melting performance of fibers



Plain　　PP 0.3vol%　　PA 0.2 vol%

Fig. 33　Columns after fire resistance test

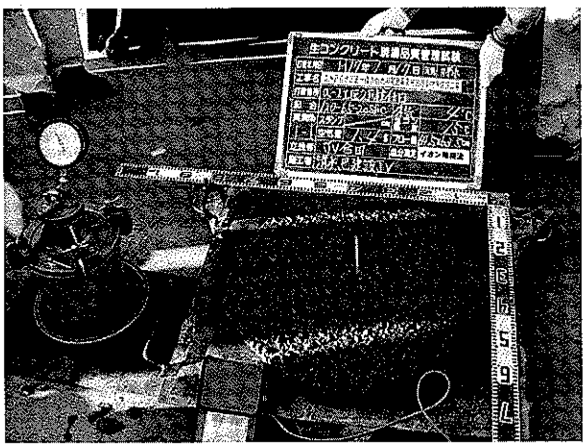


Fig. 34　Quality control tests

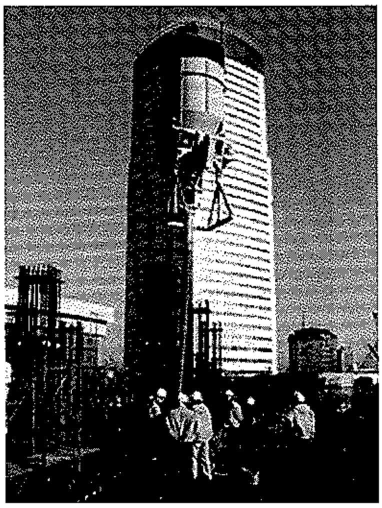


Fig. 35　Placement with a bucket

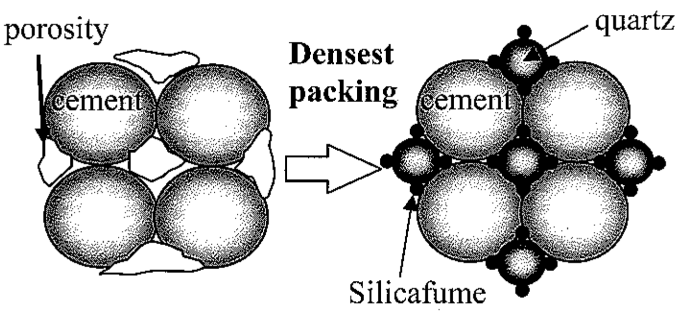


Fig. 36 Packing mix design

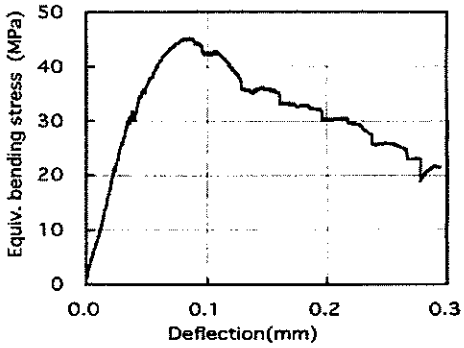


Fig. 37 Flexural behavior

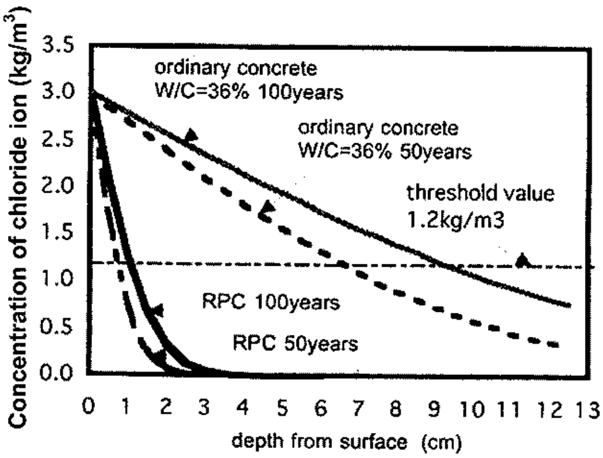


Fig. 38 Chloride profiles

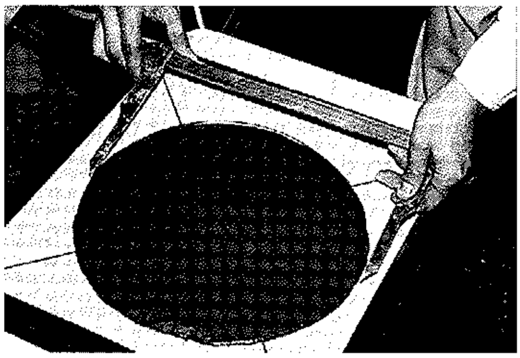


Fig. 39 Measurement of flow value

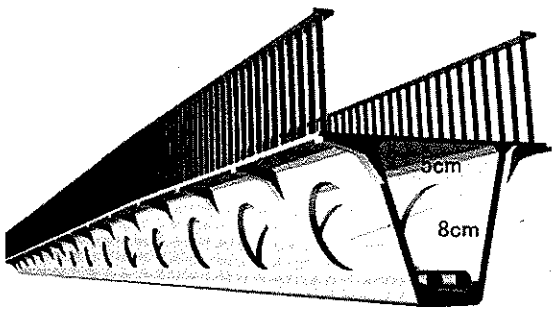


Fig. 40 Precast block

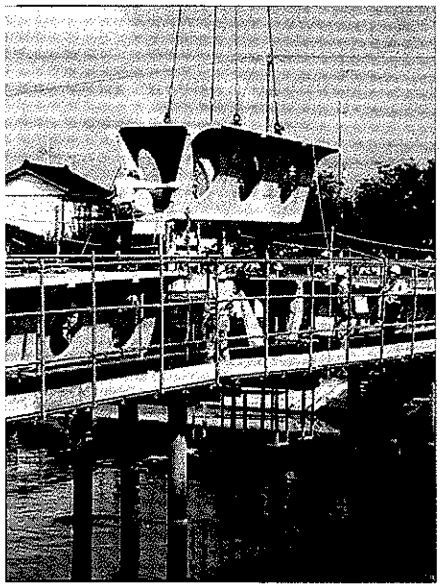


Fig. 41 Installation of block

STEEL FIBER REINFORCED CONCRETE

Nguyen Van CHANH [1]

SUMMARY

It is now well established that one of the important properties of steel fibre reinforced concrete (SFRC) is its superior resistance to cracking and crack propagation. As a result of this ability to arrest cracks, fibre composites possess increased extensibility and tensile strength, both at first crack and at ultimate, particular under flexural loading; and the fibres are able to hold the matrix together even after extensive cracking. The net result of all these is to impart to the fibre composite pronounced post – cracking ductility which is unheard of in ordinary concrete. The transformation from a brittle to a ductile type of material would increase substantially the energy absorption characteristics of the fibre composite and its ability to withstand repeatedly applied, shock or impact loading.
In this paper, the mechanic properties, technologies, and applications of SFRC are discussed.

Keywords: *Steel fiber, concrete, properties, crack, ductility, technology.*

INTRODUCTION

Fibre reinforced concrete (FRC) may be defined as a composite materials made with Portland cement, aggregate, and incorporating discrete discontinuous fibres.
Now, why would we wish to add such fibres to concrete? Plain, unreinforced concrete is a brittle material, with a low tensile strength and a low strain capcity. The role of randomly distributes discontinuous fibres is to bridge across the cracks that develop provides some post-cracking "ductility". If the fibres are sufficiently strong, sufficiently bonded to material, and permit the FRC to carry significant stresses over a relatively large strain capacity in the post-cracking stage.

There are, of course, other (and probably cheaper) ways of increasing the strength of concrete. The real contribution of the fibres is to increase the toughness of the concrete (defined as some function of the area under the load vs. deflection curve), under any type of loading. That

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is, the fibres tend to increase the strain at peak load, and provide a great deal of energy absorption in post-peak portion of the load vs. deflection curve.

When the fibre reinforcement is in the form of short discrete fibres, they act effectively as rigid inclusions in the concrete matrix. Physically, they have thus the same order of magnitude as aggregate inclusions; steel fibre reinforcement cannot therefore be regarded as a direct replacement of longitudinal reinforcement in reinforced and prestressed structural members. However, because of the inherent material properties of fibre concrete, the presence of fibres in the body of the concrete or the provision of a tensile skin of fibre concrete can be expected to improve the resistance of conventionally reinforced structural members to cracking, deflection and other serviceability conditions.

The fibre reinforcement may be used in the form of three – dimensionally randomly distributed fibres throughout the structural member when the added advantages of the fibre to shear resistance and crack control can be further utilised . On the other hand, the fibre concrete may also be used as a tensile skin to cover the steel reinforcement when a more efficient two – dimensional orientation of the fibres could be obtained.

MIX DESIGN OF SFRC

As with any other type of concrete, the mix proportions for SFRC depend upon the requirements for a particular job, in terms of strength, workability, and so on. Several procedures for proportioning SFRC mixes are available, which emphasize the workability of the resulting mix. However, there are some considerations that are particular to SFRC.
In general, SFRC mixes contain higher cement contents and higher ratios of fine to coarse aggregate than do ordinary concretes, and so the mix design procedures the apply to conventional concrete may not be entirely applicable to SFRC. Commonly, to reduce the quantity of cement, up to 35% of the cement may be replaced with fly ash. In addition, to improve the workability of higher fibre volume mixes, water reducing admixtures and, in particular, superlasticizers are often used, in conjunction with air entrainment. The range of proportions for normal weight SFRC is shown in table 1.
For steel fibre reinforced shotcrete, different considerations apply, with most mix designs being arrived at empirically. Typical mix designs for steel fibre shotcrete are given in table 2.

A particular fibre type, orientation and percentage of fibers, the workability of the mix decreased as the size and quantity of aggregate particles greater than 5 mm increased; the presence of aggregate particles less than 5 mm in size had little effect on the compacting characteristics of the mix. Figure 1 shows the effects of maximum aggregate size on workability.
The second factor which has a major effect on workability is the aspect ratio (*l/d*) of the fibres. The workability decreases with increasing aspect ratio, as shown in figure 2, in practice it is very difficult to achieve a uniform mix if the aspect ratio is greater than about 100.

Table 1 Range of proportions for normal weight fibre reinforce Concrete [6]

Property	Mortar	9.5mm Maximum aggregate size	19 mm Maximum aggregate size
Cement (kg/m^3)	415-710	355-590	300-535
w/c ratio	0.3-0.45	0.35-0.45	0.4-0.5
Fine/coarse aggregate(%)	100	45-60	45-55
Entrained air (%)	7-10	4-7	4-6
Fibre content (%) by volume			
smooth steel	1-2	0.9-1.8	0.8-1.6
deformed steel	0.5-1.0	0.4-0.9	0.3-0.8

Table 2 Typical steel fibre reinforce shotcrete mixes [7]

Property	Fine aggregate mixture (Kg/m^3)	9.5mm Aggregate mixture (Kg/m^3)
Cement	446-559	445
Blended sand (<6.35mm)[a]	1438-1679	697-880
9.5mm aggregate		700-875
Steel fibres[b, c]	35-157	39-150
Accelerator	Varies	Varies
w/c ratio	0.40-0.45	0.40-0.45

- [a] The sand contained about 5% moisture
- [b] 1% steel fibres by volume = 78.6kg/m^3
- [c] Since fibre rebound is generally greater than aggregate rebound, there is usually a smaller percentage of fibres in the shotcrete in place

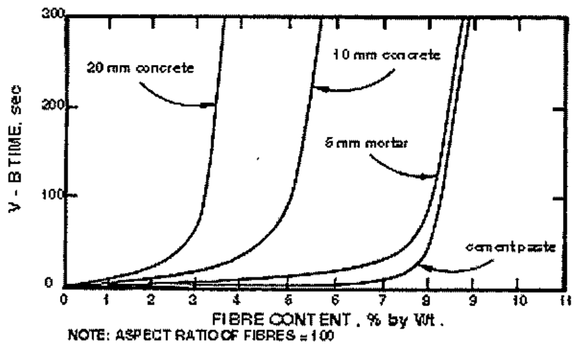

Figure 1 Workability versus fibre content for Matrices with different maximum aggregate sizes [8]

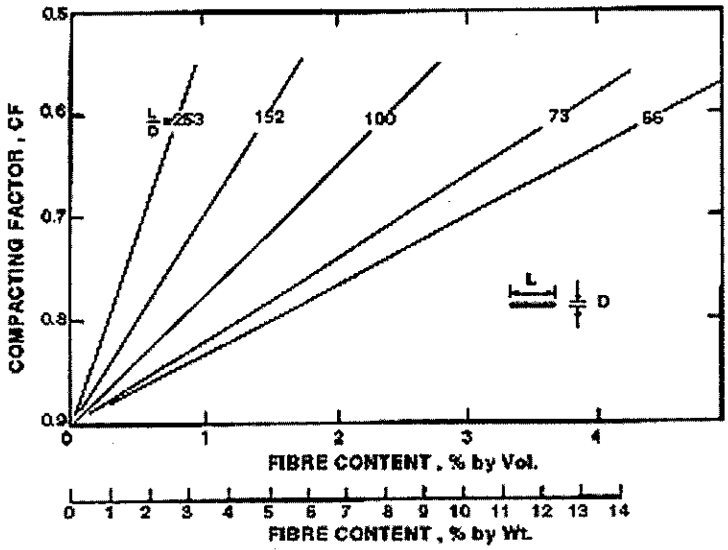

Figure 2 Effect of fibre aspect ratio on the workability of concrete, as measured by the compacting factor [8]

TECHNOLOGY FOR PRODUCING SFRC

SFRC can, in general, be produced using conventional concrete practice, though there are obviously some important differences. The basic problem is to introduce a sufficient volume of uniformly dispersed to achieve the desired improvements in mechanical behaviour, while retaining sufficient workability in the fresh mix to permit proper mixing, placing and finishing. The performance of the hardened concrete is enhanced more by fibres with a higher aspect ratio, since this improves the fibre-matrix bond. On the other hand, a high aspect ratio adversely affects the workability of the fresh mix. In general, the problems of both workability and uniform distribution increase with increasing fibre length and volume.

One of the chief difficulties in obtaining a uniform fibre distribution is the tendency for steel fibres to ball or clump together. Clumping may be caused by a number of factors:

i The fibres may already be clumped together before they are added to the mix; normal mixing action will not break down these clumps.
ii Fibres may be added too quickly to allow them to disperse in the mixer.
iii Too high a volume of fibres may be added.
iv The mixer itself may be too worn or inefficient to disperse the fibres.
v Introducing the fibres to the mixer before the other concrete ingredients will cause them to clump together.

In view of this, care must be taken in the mixing procedures. Most commonly, when using a transit mix truck or revolving drum mixer, the fibres should be added last to the wet concrete. The concrete alone, typically, should have a slump of 50-75 mm greater than the desired slump of the SFRC. Of course, the fibres should be added free of clumps, usually by first passing them through an appropriate screen. Once the fibres are all in the mixer, about 30-40 revolutions at mixing speed should properly disperse the fibres. Alternatively, the fibres may be added to the fine aggregate on a conveyor belt during the addition of aggregate to the

concrete mix. The use of collated fibres held together by a water-soluble sizing which dissolves during mixing largely eliminates the problem of clumping.
SFRC can be placed adequately using normal concrete equipment. It appears to be very stiff because the fibres tend to inhibit flow; however when vibrated, the material will flow readily into the forms. It should be noted that water should be added to SFRC mixes to improve the workability only with great care, since above a w/c ratio of about 0.5, additional water may increase the slump of the SFRC without increasing its workability and place ability under vibration. The finishing operations with SFRC are essentially the same as for ordinary concrete, thought perhaps more care must be taken regarding workmanship.

STATIC MECHANICAL PROPERTIES

Compressive Strength

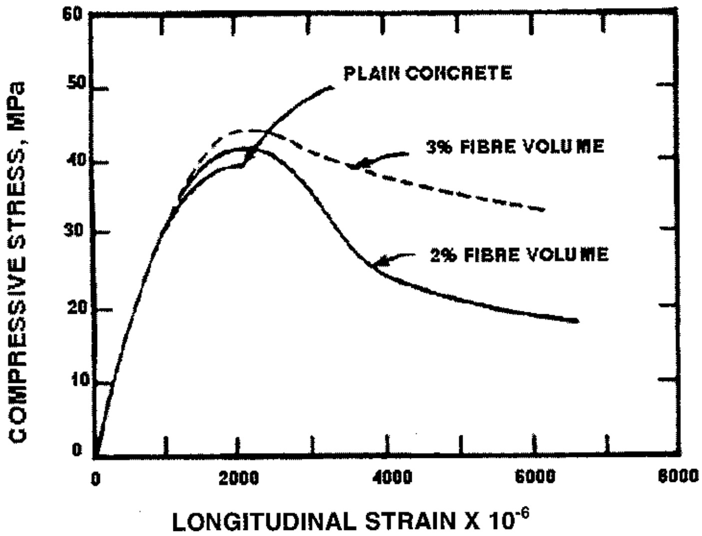

Figure 3 Stress-Strain curves in compression for SFRC [9]

Fibres do little to enhance the static compressive strength of concrete, with increases in strength ranging from essentially nil to perhaps 25%. Even in members which contain conventional reinforcement in addition to the steel fibres, the fibres have little effect on compressive strength. However, the fibres do substantially increase the post-cracking ductility, or energy absorption of the material.
This is shown graphically in the compressive stress-strain curves of SFRC in figure 3

Tensile Strength

Fibres aligned in the direction of the tensile stress may bring about very large increases in direct tensile strength, as high as 133% for 5% of smooth, straight steel fibres. However, for more or less randomly distributed fibres, the increase in strength is much smaller, ranging from as little as no increase in some instances to perhaps 60%, with many investigations indicating intermediate values, as shown in figure 4. Splitting-tension test of SFRC show similar result. Thus, adding fibres merely to increase the direct tensile strength is probably not worthwhile. However, as in compression, steel fibres do lead to major increases in the post-cracking behaviour or toughness of the composites.

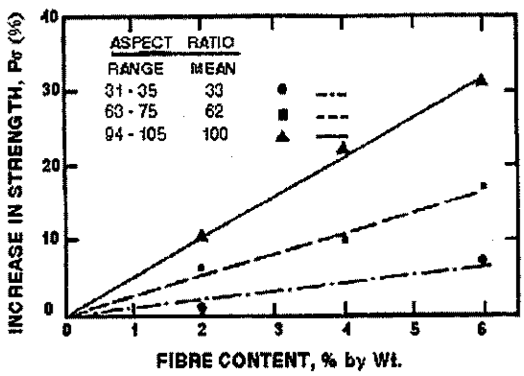

Figure 4 Influence of fibre content on tensile strength [9]

Flexural Strength

Steel fibres are generally found to have aggregate much greater effect on the flexural strength of SFRC than on either the compressive or tensile strength, with increases of more than 100% having been reported. The increases in flexural strength is particularly sensitive, not only to the fibre volume, but also to the aspect ratio of the fibres, with higher aspect ratio leading to larger strength increases. Figure 5 describes the fibre effect in terms of the combined parameter W*l*/*d*, where *l*/*d* is the aspect ratio and W is the weight percent of fibres. It should be noted that for W*l*/*d* > 600, the mix characteristics tended to be quite unsatisfactory. Deformed fibres show the same types of increases at lower volumes, because of their improved bond characteristics.

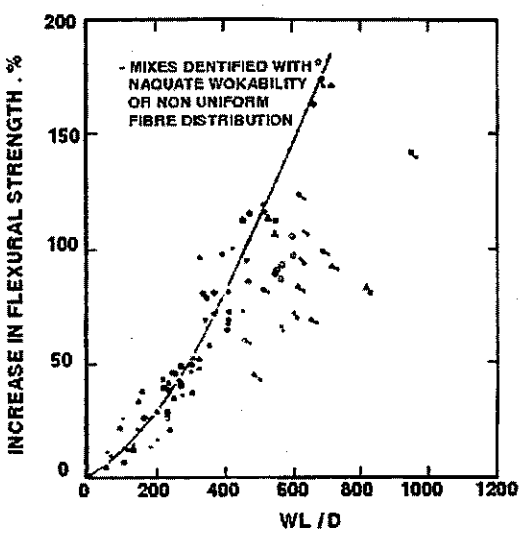

Figure 5 The effect of Wl/d on the flexural strength of mortar and concrete [9]

As was indicated previously, fibres are added to concrete not to improve the strength, but primarily to improve the toughness, or energy absorption capacity. Commonly, the flexural toughness is defined as the area under the complete load-deflection curve in flexure; this is sometimes referred to as the total energy to fracture. Alternatively, the toughness may be defined as the area under the load-deflection curve out to some particular deflection, or out to the point at which the load has fallen back to some fixed percentage of the peak load. Probably the most commonly used measure of toughness is the toughness index proposed by Johnston and incorporated into ASTM C1018. As is the case with flexural strength, flexural toughness also increases at the parameter W*l/d* increases, as show in figure 6.

The load-deflection curves for different types and volumes of steel fibres can vary enormously, as was shown previously in figure 7. For all of the empirical measures of toughness, fibres with better bond characteristics (i.e. deformed fibres, or fibres with greater aspect ratio) give higher toughness values than do smooth, straight fibres at the same volume concentrations.

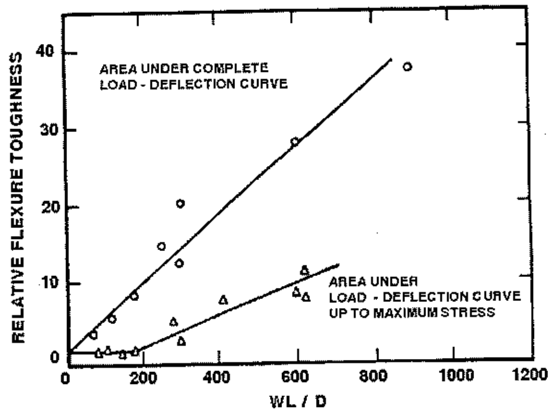

Figure 6 The effect of Wl/d on the flexural toughness of SFRC, based on data from [9].

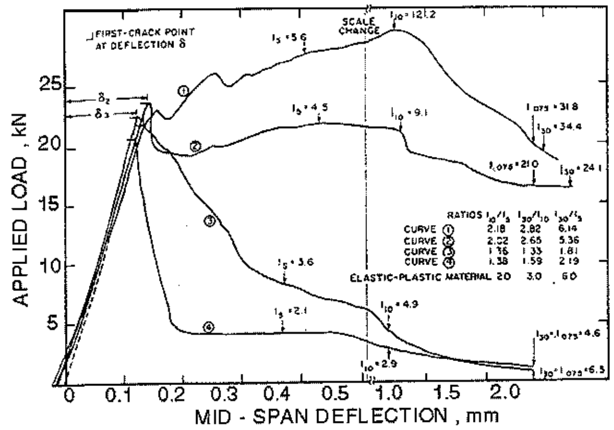

Figure 7 A range of load – deflection curves obtained in the testing of steel fibre reinforced concrete [10]

STRUCTURAL USE OF SFRC

As recommended by ACI Committee 544, 'when used in structural applications, steel fibre reinforced concrete should only be used in a supplementary role to inhibit cracking, to improve resistance to impact or dynamic loading, and to resist material disintegration. In structural members where flexural or tensile loads will occur ..... the reinforcing steel must be capable of supporting the total tensile load'. Thus, while there are a number of techniques for predicting the strength of beams reinforced only with steel fibres, there are no predictive equations for large SFRC beams, since these would be expected to contain conventional reinforcing bars as well. An extensive guide to design considerations for SFRC has recently been published by the American Concrete Institute. In this section, the use of SFRC will be discussed primarily in structural members which also contain conventional reinforcement.

For beams containing both fibres and continuous reinforcing bars, the situation is complex, since the fibres act in two ways:

(1) They permit the tensile strength of the SFRC to be used in design, because the matrix will no longer lose its load-carrying capacity at first crack; and

(2) They improve the bond between the matrix and the reinforcing bars by inhibiting the growth of cracks emanating form the deformations (lugs) on the bars.

However, it is the improved tensile strength of SFRC that is mostly considered in the beam analysis, since the improvements in bond strength are much more difficult to quantify.
Steel fibres have been shown to increase the ultimate moment and ultimate deflection of conventionally reinforced beams; the higher the tensile stress due to the fibres, the higher the ultimate moment.

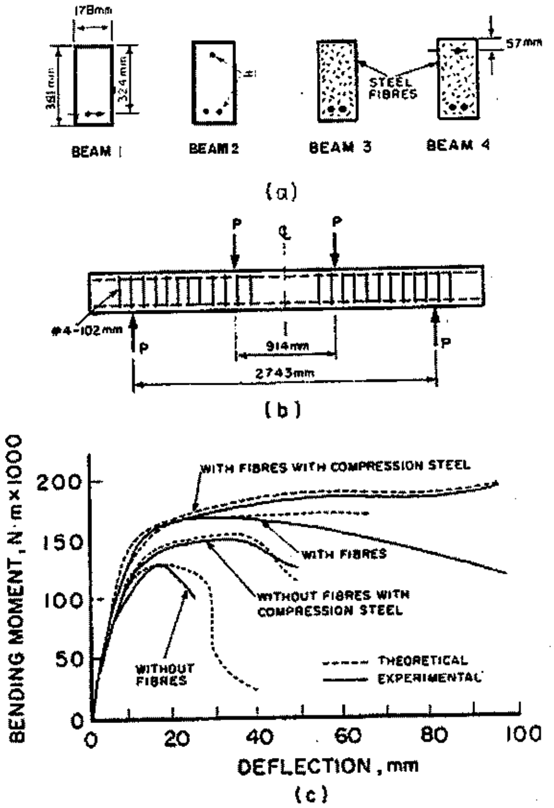

Figure 8 Experimental moment versus deflection curves for SFRC beams [11]

APPLICATION OF SFRC

The uses of SFRC over the past thirty years have been so varied and so widespread, that it is difficult to categorize them.The most common applications are pavements, tunnel linings, pavements and slabs, shotcrete and now shotcrete also containing silica fume, airport pavements, bridge deck slab repairs, and so on. There has also been some recent experimental work on roller-compacted concrete (RCC) reinforced with steel fibres. The list is endless, apparently limited only by the ingenuity of the engineers involved. The fibres themselves are, unfortunately, relatively expensive; a 1% steel fibre addition will approximately double the material costs of the concrete, and this has tended to limit the use of SFRC to special applications.

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ADVANCED CONCRETE APPLIED AT VIETNAM

Tran Ba VIET [1]

INTRODUCTION

In the recent years, the advanced types of concrete applied in Vietnam include: High Performance Concrete (HPC), Self - Compacting Concrete (SCC), Steel Fiber Concrete, Roller Compacted Concrete (RCC) and Concrete for Marine structures. In addition in Vietnam, Rice Husk Ash (RHA) has also been studied to be used for production of these types of concrete. In this paper, the author presents the main results of the research and application of the above mentioned types of concrete and RHA in construction industry.

HPC – HIGH PERFORMANCE CONCRETE

This type of concrete is recognized as a high performance concrete type in addition to high physical- mechanical properties. HPC has been developed since the availability of high rank - water reduced admixture in the construction market. We have studied producing high - slump concrete with grade of 100MPa using either PC 40 or PCB 40. In fact, we have successfully produced precast elements of 80MPa: the tube piles, and in-place structural elements of 60MPa were also cast. Concrete with strength up to 60MPa has been used for precast elements, columns and beams of the lower stories of high-rise buildings, for water supply piping system, bridge girders and abutments/piers, as well as for elements that are the load-bearing ones in addition to waterproofing. For the time being, concrete with the strength of 60MPa can be easily produced at the commercial ready – mixed concrete stations (plants) in the routine production conditions. Crushed lime stone can be used as coarse aggregates for production of 60MPa strength concrete. With higher concrete strengths, granite, basalt should be used. Floated coal ash, lime stone powder, or activated pulverized coal ash can be used to replace a portion of cement and to increase the fine amount. Granular coal ash, the waste discharged from the metallurgical industry, can be imported from Japan to be used for replacing a portion of cement in concrete. Concretes with high slump, from 14-26cm, were produced. To produce concretes with the strength equal to and more than 60MPa, SF and RHA should be used. Studying on HPC is helpful for production of Steel Fiber Concrete, anti-corrosion concrete for Marine structures and for Self- Compacting Concrete.

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SELF – COMPACTING CONCRETE

SCC has been studied since the availability of new-generation super-plasticizer of polycaboxylate Natri in the Vietnam construction market together with the presence of MBT and SIKA firms. Commercial names of the two admixture families are Glenium and Viscosity. The two mentioned firms have commenced providing their clients with the aids to the research and application of these admixtures. At present, the cacboxylate-based admixtures supplied by other firms such as Grace, Stondhard, and suppliers from China (Nikkang) are available in Vietnam construction market.
However, to legalize the use of SCC in Vietnam, the Vietnam Institute for Building Science and Technology (IBST) has studied the production of SCC, using the two mentioned super plasticizers for the concrete of different grades of strength: 30, 40, 50, 60, 70, 80 MPa. Both cement types PC and PCB with either Pha Lai thermo-power plant floated coal ash, crushed sand, or lime stone powder as a filler can be used.
From results obtained, the procedure for proportioning and construction of SCC has been developed by IBST and it was issued by the Vietnam Ministry of Construction (MOC).
At present, the wide application of this concrete is limited due to the high price of the admixture that makes the price unit of SCC higher than that of normal concrete. The price of the admixture at present in Vietnam is VND 60,000/litre and it makes the price unit of concrete increase about VND 200,000/m^3 than normal concrete. For the time being, SCC has been applied to casting bridge girders, structural elements with dense reinforcement, facing concrete such as that used for bridge desk waterproofing.
Based on research results on SCC, SCC with scattered fiber reinforcement for the facing and for the elements that are load-bearing ones in addition to waterproofing, such as the lining of Hai Van tunnel and other structures, has also been studied.
The disadvantage of the new generation super plasticizers is the high slump loss, therefore, the use of concrete in the hot weather condition or the use of ready-mixed concrete is not reasonable. This is the question that should be continuously studied. The use of SF (Silica fume) or RHA (Rice husk ash) for replacing a portion of cement in concrete has also been studied and a good result was obtained. SF and RHA have resulted in a strength ratio that meets the required standard.

SCATTERED FIBER REINFORCED CONCRETE (SFRC)

At present, two types of this concrete, PPFC (polypropylene fiber concrete) and SFC (scattered steel fiber reinforced concrete), are being applied. Research and application results of these two concrete types to different fields such as the facing of waterproofing structure, precast sewers, concrete elements that are the load-bearing ones in addition to waterproofing (such as tunnel lining, architectural concrete, decorating concrete,..) are available.
Some research results have been developed to be a Ph.D. thesis. The prosperity of application of scattered fiber reinforced concrete (SFRC) to the Vietnam construction field is very high. PPFC with grade of 30 has been applied to the facing of the steps of My Dinh National Stadium. Another applied research on PPFC for decorating concrete has also been developed in a testing scale.

SFC using RADMIX steel fiber has been applied to the elements that are load-bearing ones in addition to waterproofing, such as the lining of Hai Van tunnel and other defense structures.
At present, IBST can produce SFC with grade up to 100 and flexural strength of 250daN/cm^2 whose slump is 15cm.
Some types of steel fiber such as cold rolled fibers, hot–rolled/laminated ones with different lengths and diameters have been considered to be used in different construction conditions under the action of the Northern and Southern climate conditions of Vietnam.
The properties such as: ultimate curing strength, draying shrinkage, plastic shrinkage, impact-resistant ability, anti-cracking ability, corrosion resistance, train-stress in the Northern and Southern climate conditions of Vietnam have also been initially studied. The studies will be presented in a Ph.D. thesis and another three MEng. theses.

ROLLER COMPACTED CONCRETE (RCC)

RCC has been studying for the last 3 years. Though the research result obtained is limited, particularly the production technology, quality control, testing and construction equipment, it is recognized that RCC will be widely applied to the construction of water works in the coming years in Vietnam.
In fact, the 67 m high PlayKrong hydro-power plant's dam and the 56m high dam of Dinh Binh water irrigation work were constructed with RCC. A 2,000m^3 test block of RCC at PlayKrong and another 3,000m^3 test block of RCC were cast at Son La hydro-power plant to check the proportions (grading parameters) of RCC as well as the construction technology. Testing data on the strength of the cast block, waterproofing grade of concrete in the direction perpendicular with the casting surface are met. However, the permeable parameter along the casting surface, the bonding regarded as the tensioning resistance (anti-sliding resistance) between the slip formed layers have not been tested yet. The grading of aggregates should be adjusted, particularly the maximum size of aggregates. So do the technological parameters, particularly the rolling rate, which depends much on the grading. The thermal problem and problem of thermal stress have not also been controlled.
In the near future, the following dams are expected to be constructed with RCC: Ban Ve, A Vuong, Se San 4Dong Nai 3, Dong Nai 4, Song Tranh 2, Ban Chat, Huoi Quang and, the largest one is the dam of Son La hydro-power plant, which is 135m high with some 3,3 million of m^3 of concrete.
Based on the research results, we are now developing the construction and acceptance standards for RCC and these standards will be soon submitted to the Ministry of Construction for approval.
A research fellow has developed his research result and turned it out to be a Ph.D. thesis. The strong development of RCC in Vietnam is due to two main causes: a lower cost and a shorter duration of construction.
With regard to RCC studies, the following main questions are left undone: the bond between the lifts, the calculation of anti-sliding factor for the lifts, the permeable parameter along the casting surface, the quality control at the construction site, particularly the difference of the bond of different areas of the top lift; the control and construction of the bedding concrete layer (in this concrete, the amount of paste is more than the normal one); the reduction of permeable pressure through the bottom lift and the contraction joints of the top lift; the

analysis and control of thermal stress of RCC dams; the calculation and inspection of the monolithic nature of RCC dams in terms of bearing and waterproofing capacity, particularly the dam of Son La hydro-power plant, which is 135 m high with some 3,3 million of m^3 of concrete and built in an activated earthquake zone; the calculation and selection of mixing system of equipment, transportation and construction equipment; the construction measures in the flooding seasons. These are the urgent (burning) issues that must be studied prior to the construction of the Son La hydro-power plant dam, which is located on the upstream of the Red river, directly affecting the Hoa Binh hydro-power plant as well as the Red river delta, in addition to Hanoi, the capital of Vietnam. Vietnam cannot solve these questions itself and it needs to be consulted by the experienced foreign consultants such as the Japanese and Chinese ones.

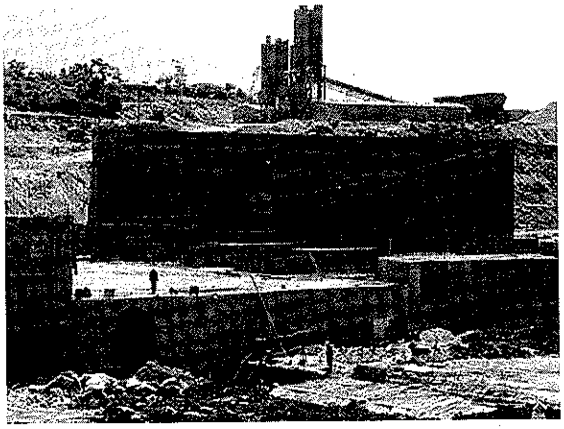

Photo 1 RCC at Dam PlayKrong
(Kontum Province, Vietnam)

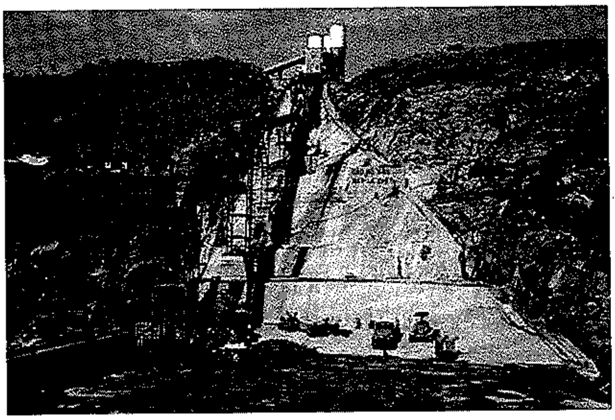

Photo 2 RCC at Dam Dinh Binh
(Binh Dinh Province, Vietnam)

WITH REGARD TO CONCRETE FOR MARINE STRUCTURES

In Vietnam, there is more than 3,000 km of coastal length. Thus, to study on concrete for marine structure is an urgent need. It has been studying for more than 20 years and now, many results on this filed are available. The Vietnam Institute for Building Science and Technology (IBST) has also conducted a great programme for more than 5 years from which, many specifications/guidelines on design, construction and acceptance for marine concrete structures have been developed by IBST and approved by Vietnam MOC to be issued as a legal basis for construction and acceptance of marine reinforced concrete structures.

One of the important matter that should be paid much attention with regard to the actual investigation and research result is the zoning of marine climates and zoning of sea water action on the corrosion of reinforced concrete structures. The research results showed that: there was no sulphate-corroded trace in marine concrete structure; Thus, it means that steel corrosion will makes concrete deteriorated before sulphate-corroded phenomenon can be identified. Based on this result, the main orientation of anti-corrosion is to prevent steel corrosion caused by ion Cl^-. This solution can be performed by increasing the concrete cover, increasing the density of concrete, the waterproofing ability, adding corrosion inhibitors such as calcium nitrite to concrete during the construction process; or by covering reinforcement steel with an anti-corrosion coating and a coating for concrete. The research result obtained in Vietnam is similar to that gained by the Japanese colleagues at the PARIS. However this concept is different from our traditional one. In the above anti-corrosion solutions, the solution of increasing concrete quality by using HPC with RHA, in addition to the use of calcium nitrite as a corrosion inhibitor is often preferred when constructing new marine structures.

Research results have been being applied in building new structures, repairing the old ones, such as the adjustment of the thickness of concrete cover for the Bai Chay bridge and many other marine concrete structures in Vietnam.

WITH REGARD TO RESEARCH AND PRODUCTION OF RHA (RICE HUSK ASH)

To produce HPC, SFC, anti-corrosion concrete for marine concrete structures, silica fume (SF) is required to be used as a super-fine mineral admixture. In Vietnam, this is the exported admixtures with a very high cost due to the transportation expenses.

We have studied the production, and equipment required to activate rice husk as rice husk ash (RHA) according to Metha – Pitt technology. The strength activation of RHA produced according to this technology is similar to that condensed SF, but with a very cheap price due to the advantage of using rice hush much available in an agricultural country as Vietnam. At the same time, we have studied to develop the standard on strongly-activated mineral admixtures of RHA and SF used for concrete and mortar. The said standard was issued by Vietnam MOC, creating favorable conditions for wide application of RHA to the production of the above mentioned concrete. RHA produced by us can be exported to other countries, including Japan, for production of HPC.

CONCLUSION

Herein above is some brief information about the research, production and application of various types of advanced concrete in Vietnam in the recent years. The development of construction field, particularly the development of the infrastructure with a rate of over 16%/year has made the demand of using advanced concrete urgent. Mastering of the production technology, applied design and technological equipment is the task of technology transfer. There are many other questions relating to advanced concretes that should be understood and clarified. Therefore, the cooperation, the aid in training, research and application of advanced concretes are needed.

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THE EFFECT OF TRIETHANOLAMINE AND LIMESTONE POWDER ON STRENGTH DEVELOPMENT AND FORMATION OF HARDENED PORTLAND CEMENT STRUCTURE

Nguyen Nhu QUY (1) and Nguyen Trong LAM (2)

SUMMARY

The presence of finely ground limestone powder in concrete becomes more and more popular due to its ability to upgrade several properties of concrete. When the limestone powder(LP) is used in concrete in combination with triethanolamine (TEA) as an accelerator the hydration and hardening process of Portland cement is likely changed. This study reported the results of the authors' research on the effect of TEA and limestone powder on strength development of concrete with varying water-cement ratio using statistic design of experiments. The results of SEM and X-ray examination reveal that TEA has a rather profound influence on the concrete strength development as well as the formation of hardened Portland cement structure.

Keywords: *Accelerator, Fine filler, Strength development, Hardened cement structure.*

INTRODUCTION

TEA is a surface active substance which is absorbed on surface of cement particles and cement hydrated products. TEA enables the solution of some metallic ion like Fe^{3+} and Al^{3+} thus increases the activity of C_4AF compound and inhibits the formation of $Fe(OH)_3$ and $Al(OH)_3$ on surface of cement particle. This action can facilitate the hydration rate of silicate and alumilate phases in cement particle. Besides, TEA can also reduce to some extort the surface tension of water that enables the cement powder wetting and dissolving highly reactive cement compounds. In the cement paste ground limestone powder acts as fine filler and effects the formation and growth of cement hydration products. $CaCO_3$ particles act as embryos of crystallization which accelerates the hardening process as well as the formation of $Ca(OH)_2$ crystals. An attempt has been made to combine TEA and ground limestone powder together to form a chemical admixture in terms of an effective non-corroding accelerator

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which doesn't adverse 28 day compressive strength. In this research the statistic design of experiments was used to investigate the influence of TEA concentration, ground limestone powder addition rate and water-cement ration on the concrete strength development as well as the formation of hardened cement paste structure.

MATERIALS AND METHOD OF TESTING

Materials

In this research Portland cement PC-40 produced by Chin-Fon cement plant was used. This cement is similar to cement Type I according to ASTM - C150. Oxide composition of cement is given in table 1. Normal river sand and a crushed stone were used as aggregate, their physical properties are shown in table 2 and table 3.

Table 1 Oxide composition of cement and ground limestone

Oxide	Cement	Ground limestone powder
SiO_2	21,8	0,18
Al_2O_3	6,0	0,30
Fe_2O_3	3,3	0,12
CaO	64,3	54,20
MgO	0,80	0,91
MnO	0,02	-
TiO_2	0,25	-
K_2O	0,74	0,09
Na_2O	0,08	0,06
SO_3	2,4	0,03
Cl^-	0,008	-
Fr.CaO	0,8	-
L.O.I	1,0	43,9

TEA of density $\rho = 1{,}124$ g/ml and ground limestone powder with total content of $CaCO_3$ and $MgCO_3$ equal to 99,6%, fineness as 4,1% retaining on 0,08 mm sieve were used. The oxide composition of ground limestone powder is listed in table 1.

Table 2 Particle gradation of aggregate

Aggregate type	Percentage passing through sieve size (mm)									
	40	20	10	5,0	2,5	1,25	0,63	0,315	0,14	< 0,14
Sand	-	-	-	100	88	76	54	20	1,9	-
Gravel	-	100	30	0	-	-	-	-	-	-

Table 3 Physical properties of aggregate

Aggregate type	Specific gravity (g/cm^3)	Water absorption, (%)	Fineness modulus	Dry-rodded unit weight, (g/cm^3)	Void ration, (%)
Sand	2,65	0,4	2,5	1,63	38,5
Gravel	2,7	0,2	-	1,62	40

Method of testing

Testing has been done according to the following sequence.

+ Building plan of experiments.
+ Concrete mix proportioning followed plan of experiments.
+ Study the influence of TEA and LP on cement setting time.
+ Study the influence of TEA and LP on cement hardened paste structure.
+ Study the influence of TEA and LP on concrete strength development.

The concrete mix proportions of control specimens and the optimum specimens are listed in table 4. In this paper only most successful specimens are show, as a result of statistic design of experiments with two level three factorial control composite design method.

Table 4 Concrete mix proportions

Type of specimens	W/C	Dosage of TEA (ml/l)	Rate of LP addition (%)	Unit content, kg/m^3			
				Cement	Sand	Gavel	Water
Control	0,5	-	-	431	706	975	216
Test	0,5	1,2	7,0	400	706	975	216

The consistency of fresh concrete mix is approximate 7,0 cm.

RESULT AND DISCUSSION

Influence of TEA and LP on cement setting time

Test results show that initial and final setting time of sample containing LP are shortened, but the time interval remains nearly unchanged (see figure 1). In case of samples containing TEA the initial set is delayed but the final set remains the same as for control sample (see figure 2). The combination of TEA and LP has strong effect on setting characteristics of paste. Both initial and final sets are delayed but setting time interval is shortened considerable (figure 3).

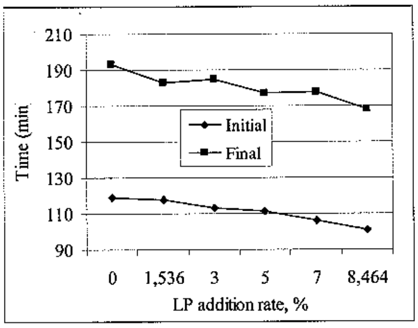


Figure1 With LP

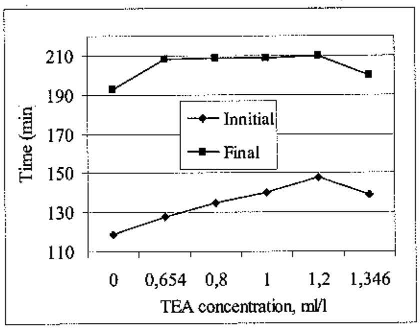


Figure2 With TEA

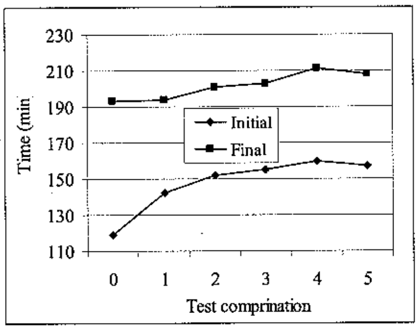


Figure3 With both TEA and LP

Influence of TEA and LP on the concrete strength development

Test results show that the presence of TEA and LP in concrete mix accelerates its strength development (see table 5). At 3 day age the compressive strength, increases about 26% over that of control specimens. At 7 day age the increase in the compressive strength reaches up to 18%. At 28 day age the compressive strength of both tested and control specimens is nearly the same unlike the case when LP is used as fine filler alone where the 28 day compressive strength of concrete is reduced by about 10 to 20%.

Table 5 The compressive strength development of concrete

Type of specimens	W/C	Dosage of TEA(ml/l)	Rate of LP addition	Slump (cm)	Compressive strength MPa, at the age of		
					3 days	7 days	28 days
Control	0.5	-	-	7.0	28.7	36.0	47.1
Test	0.5	1.2	7.0	7.0	36.2	42.5	48.1

Influence of TEA and LP on the microstructure of hardened cement paste

The microstructure of hardened cement paste with and without TEA and LP was examined by XPD and SEM. The amount of $Ca(OH)_2$ which is represented by peak with d-values 4.92, 2.63 and 1.93A°.

In the samples with TEA and LP is much less than that in the control samples. This may gives an evidence of the formation of hydrates with high $CaO:SiO_2$ ratio. By 28 days the amount of $Ca(OH)_2$ crystal becomes larger in the sample with TEA and LP (see figure4, figure5, figure6 and figure7).

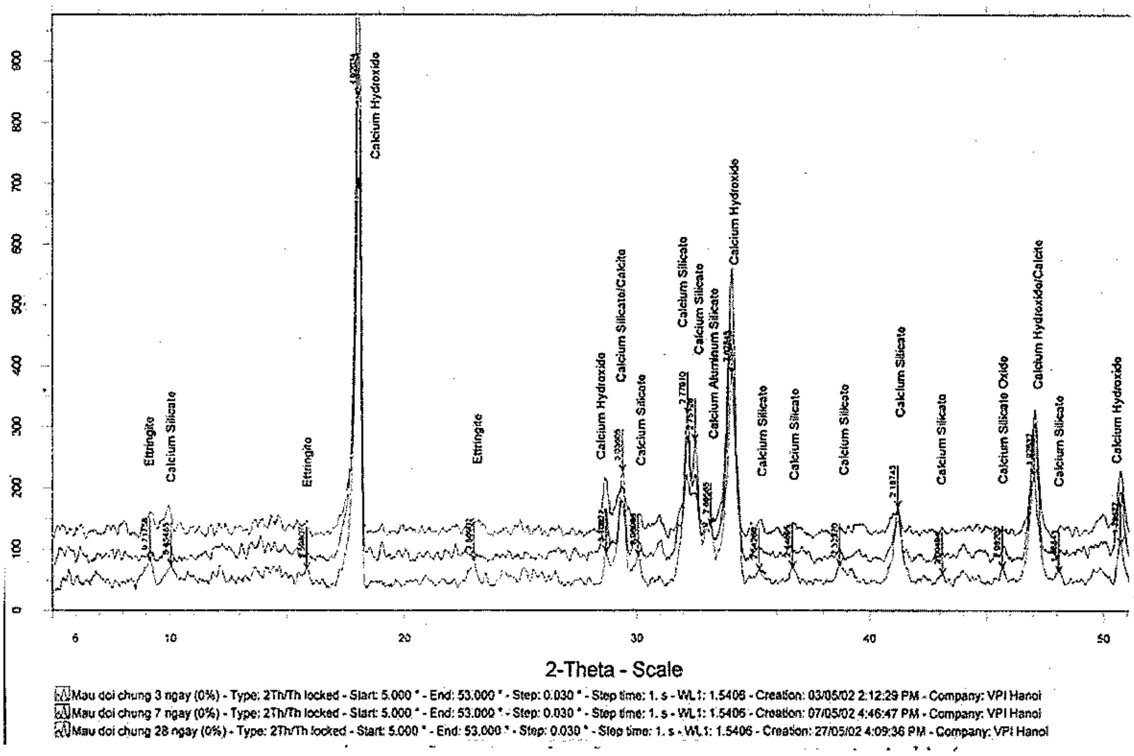


Figure 4 XPD pattern of cement paste without TEA and LP at 3, 7 and 28 days

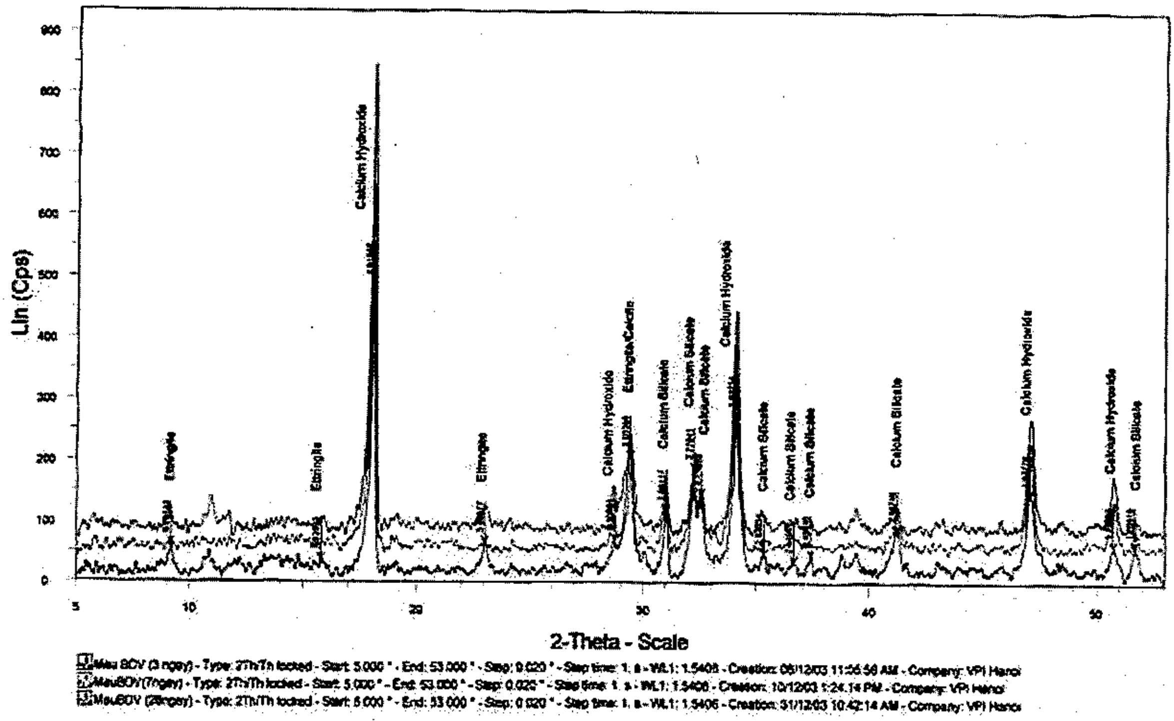


Figure 5 XPD pattern of cement paste without TEA and LP at 3, 7 and 28 days

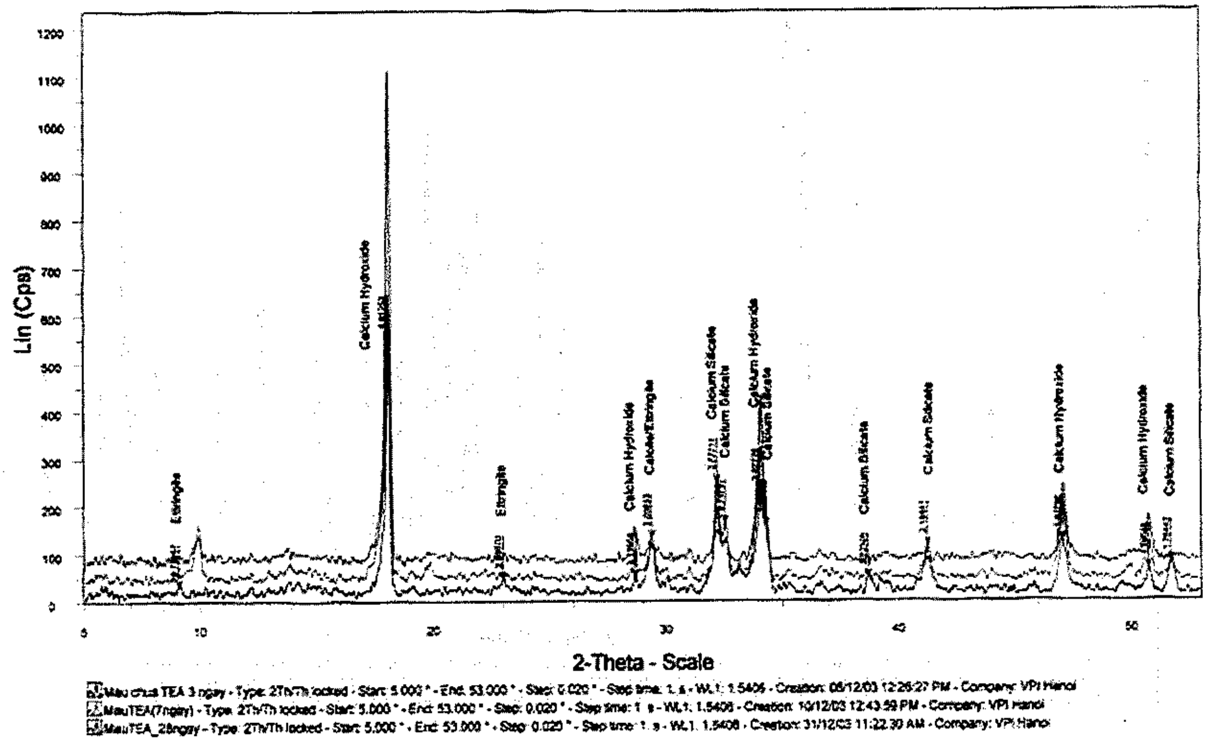


Figure 6 XPD pattern of cement paste with TEA at 3, 7 and 28 days

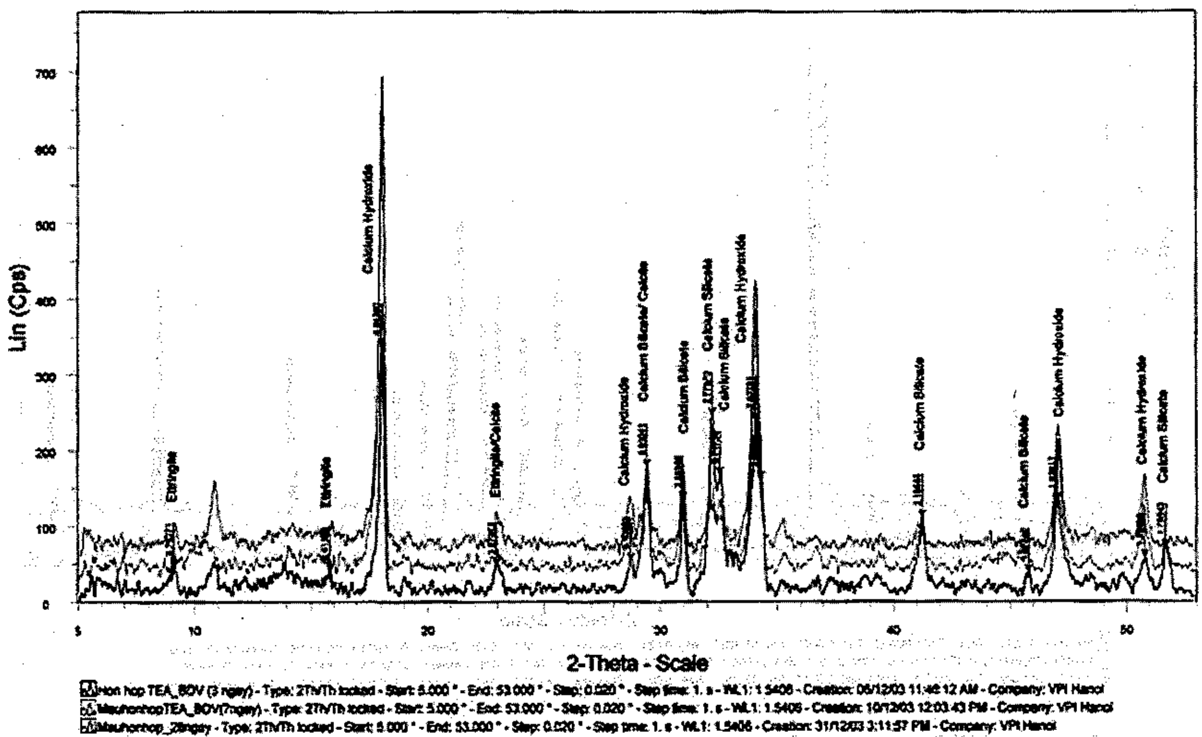
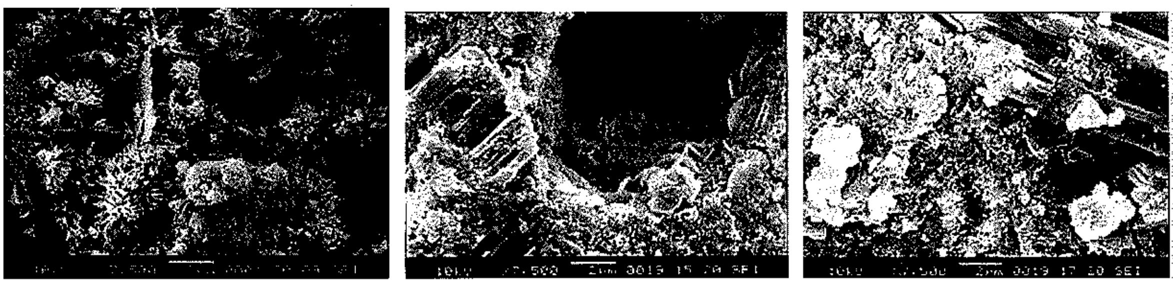


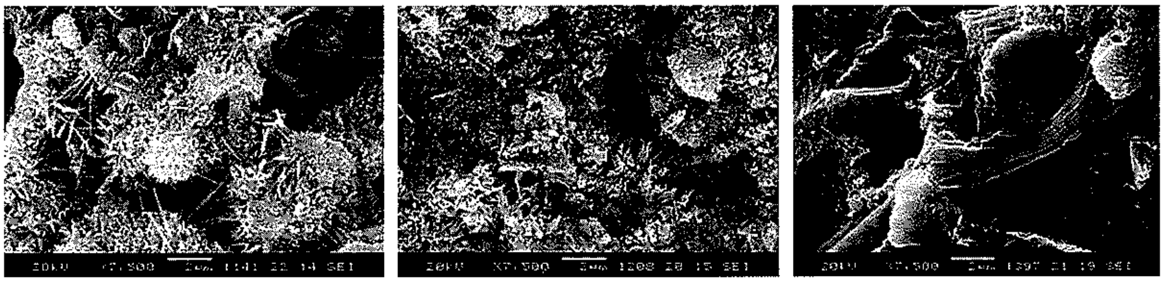
Figure 7 XPD pattern of cement paste with TEA and LP at 3, 7 and 28 days

The microphotographs (see figure8, figure9, figure10 and figure11) show clear difference in microstructure of hardened cement paste with and without TEA and LP.



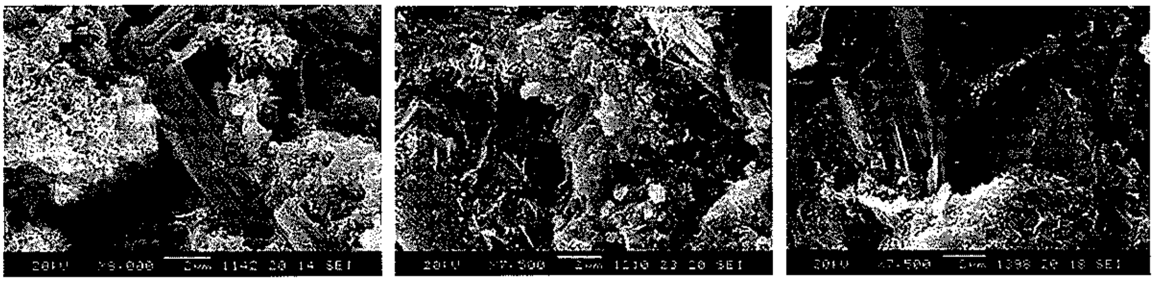
At 3 days At 7 days At 28 days

Figure 8 SEM image of cement paste without TEA and LP at 3, 7 & 28 days



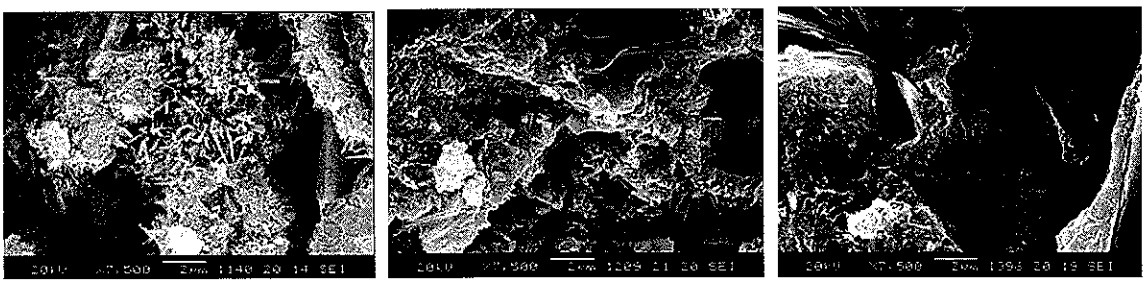
At 3 days At 7 days At 28 days

Figure 9 SEM image of cement paste with LP at 3, 7 & 28 days



At 3 days At 7 days At 28 days

Figure 10 SEM image of cement paste with TEA at 3, 7 & 28 days



At 3 days At 7 days At 28 days

Figure 11 SEM image of cement paste with TEA and LP at 3, 7 & 28 days

After 3 days hydration the sample containing 7% LP show a large quantity of Ettringite of coarse crystallization distribute in large pores. These ettringite crystals may increase the density of hardened cement paste. In the sample with TEA a large amount of hydrates filling up the space between cement particles. These hydrates crystallized in form of coarse flakes. In the samples with both TEA and LP apart from a large quantify of hydrates the ettringite crystals are also presented. In these samples the coarse hydrates in form of plats are also observed. In later period of hydration the differences in the hardened cement microstructure can hardly be observe more over the hardened cement paste with TEA and LP is seem to be denser.

CONCLUSIONS

Within tested material combination the following conclusion can be drawled:

1. The presence of TEA and LP shortens the time interval between initial and final setting time of cement paste.
2. With dosage of 7% LP and TEA concentration of 1.2 ml/l the combination admixture accelerates the concrete strength development in early stage hydration and eliminates the strength reduction of concrete at 28 days of hydration which is common for concrete with LP addition alone.
3. XPD and SEM examination show visible change in Ettringite crystals' morphology and the combination o TEA and LP accelerates cement hydration thus increases its density

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THE INTRODUCTION OF CEMENT AND CONCRETE TECHNOLOGY IN VIETNAM AND JAPAN

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SUMMARY

It is widely recognized that cement and concrete technology is developing and concrete production amount and batching plant increases year after year in Vietnam. On the other hand, Japan is one of advanced country on cement and concrete technology. This paper introduces the present conditions of cement and concrete technology comparing in Vietnam and in Japan, to put it concretely, materials for concrete such as cement, aggregate and admixture, concrete properties and ready-mixed concrete.

***Keywords**: Cement, Concrete, Ready-mixed concrete, Standard, Aggregate, Admixture, Vietnam, Japan*

INTRODUCTION

This paper introduces technical conditions of cement and concrete, comparing in Vietnam and in Japan. Nghi Son Cement Corporation is a Japanese-Vietnamese joint company, so it is easy to collect useful technical information in both countries through our general work. And we hope this information will provide fundamental understandings on technical conditions of cement and concrete in both Vietnam and Japan.

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CEMENT

There are some cement standards in both countries. Concerning quality of cement, “Portland Cement – TCVN 2682” and “Portland Blended Cements – TCVN 6260” are specified in Vietnam. And “Portland Cement – JIS R 5210” is specified in Japan. The comparison of these standards is shown in table 1.

Table 1 Comparison of cement standards in Vietnam and Japan

		Vietnam					Japan		
		PC30	PC40	PC50	PCB30	PCB40	N	M	H
Chemical composition (%)	LOI	≤5.0					≤3.0		
	IR	≤1.5							
	MgO	≤5.0					≤5.0		
	SO_3	≤3.5			≤3.5		≤3.0		≤3.5
	R_2O						≤0.75		
Fineness	Blain(m^2/g)	≥2700		≥2800	≥2700		≥2500	≥2500	≥3300
	0.08mm	≤1.5		≤1.2	≤1.2				
Setting time	Intial (min)	≥45			≥45		≥60		≥45
	Final	≤6h15m			≤1.0h		≤1.0h		
Soundness		≤1.0			≤1.0		≤1.0		
Compressive strength (N/mm^2)	1 day								≥10
	3 days	≥16	≥21	≥31	≥14	≥18	≥12.5	≥7.5	≥20
	7 days						≥22.5	≥15	≥32.5
	28 days	≥30	≥40	≥50	≥30	≥40	≥42.5	≥32.5	≥47.5

N: Normal portland cement (which is equivalent to ASTM Type I and PC 40)
M: Moderate heat portland cement (which is equivalent to ASTM Type II)
H: High early strength cement (which is equivalent to ASTM Type III)

Except for these typical cements, Low Heat Portland Cement, Ultra High Early Strength Portland Cement and Sulfate Resisting Cement are specified in JIS R 5210. Further, Blast Furnace Slag Cement which is popular in Japan is specified in other standard, JIS R 5211.

In Vietnam, PCB cement especially PCB40 is widely used. On the other hand, the PC cement production comes to decrease year by year. The type of cement product is shown in Fig 1.

Typical property of cement and difference of cement situation between in Vietnam and in Japan is shown in Table 2 and Table 3 respectively.

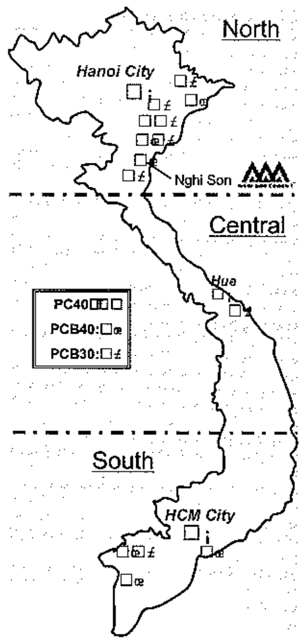

Figure 1 Type of Cement Product in Vietnam

We have natural resource of good quality as raw materials for cement in North Vietnam. But the qualities of cement are various even if they are same type, PCB40, and depend on the product type and cement company, because additive type and content against cement varies. This condition is clear from table 2.

Table 2 Typical Properties of PCB40 Cement in Vietnam

	Ig.loss (%)	insol (%)	SO_3 (%)	R_2O (%)	Blaine (cm^2/g)	Compressive Strength(N/mm^2)		
						3days	7days	28days
Cement in Vietnam	1.2 - 6.5	2 - 13	1.7 - 2.4	0.4 - 1.1	3,200 - 3,900	21 - 33	30 - 46	41 - 54
Nghi Son Cement	2.9	3.3	1.91	0.71	3,620	30.3	43.1	52.1

Table 3 Difference of Cement Situation between in Vietnam and in Japan

	Vietnam	Japan
Portland Cement	Portland Cement (PC30, PC40, PC50) Portland Blended Cement (PCB30, PCB40)	Normal portland cement High early strength portland cement Ultra high early strength portland cement Moderate heat portland cement Low heat portland cement Sulfate resisting cement
Additive Content	No Additive - PC, <20 - PCB (<40 in case of active -PCB)	<5
Typical Additive	Limestone, Basalt, Natural Pozzolan	Blast Furnace Slag, Fly Ash, Limestone, Silica
Quality	Depends on Product or Company	Equivalent in case of same type
Other Cement	Pozzolanic Portland Cement Blast - Furnace Portland Cement White Portland Cement Sulfate Resisting Portland Cement Low Heat Portland Cement	Portlan Blast - Furnace Slag Cement Portland Fly Ash Cement Eco-Cement Portland Pozzolan Cement (Silica Cement)
Bag Percentage	>90%	<5%

OTHER MATERIAL FOR CONCRETE

Aggregate

<Vietnam>

Quality control of aggregate is scarce, so it is difficult to grasp the quality of aggregate. But especially in South area, fineness modulus of fine aggregate becomes smaller and that affects unit water of concrete increased. Some skilled companies attempts to develop and use grinding stone for adjustment of fineness modulus.

<Japan>

Resource of aggregate become exhausted gradually, so utilization technology for aggregate has advanced. (such as artificial aggregate made from blast furnace slag, fly ash, recycled glass and so on). Moreover concrete using recycled aggregate is also developed. It is said that quality control of aggregate is more severe than that in Vietnam. Fineness modulus of fine aggregate varies depending on area.

Admixture

<Vietnam>
Admixture such as Blast Furnace Slag and Fly Ash is hardly used in concrete, because their quality is low compared with foreign country. However lately seminar and study on admixture for concrete is found, so there is some possibility that admixture for concrete will become popular. Blast Furnace Portland Cement is specified in TCVN although it isn't used for concrete in Vietnam.

<Japan>
Blast furnace slag is widely applied not as admixture for concrete but as material for pre-mixed slag cement. Fly ash is applied to dam concrete such as RCC which is called RCD in Japan. Blast furnace slag and fly ash is specified in JIS as both pre-mixed cement and admixture for concrete. Moreover, silica fume for high strength and/or durable concrete, expansive additive and so on is specified in JIS.

Chemical Admixture

<Vietnam>
Generally, many batching plants usually use chemical admixture for concrete, but in case of house construction except for supply from batching plant, waterproofing admixture is merely used into concrete at some site. Most popular type of chemical admixture for concrete is water reducing and set retarding type which complies with ASTM C494 Type D. And air entraining agent is rarely used except for some big project.

<Japan>
Chemical admixture for concrete is specified in JIS A 6204. According to this standard, compressive strength ratio, length change ratio and deference of setting time against no use of chemical admixture shall be tested. The resistance against freezing and thawing shall be evaluated, too, because there is freezing and thawing action in Japan. Most popular type is AE and water-reducing type. High-range water reducing and air-entraining admixture was developed and added in JIS A 6204 recently.

The condition of other materials for concrete except for cement is shown as below (See Table 3).

Table 3 The Condition of Other Materials for Concrete (except for cement)

		Vietnam	Japan
Aggregate	Quality Control	Scarce	Severe
	Fineness Modulus	Small in South area	Depending on area
Admixture	Blast Furnace Slag	not applied yet	Pre-Mixed Cement
	Fly Ash	Rarely	Dam Concrete, Pre-Mixed Cement
	Others	None	Silica Fume, Expansive Additive and so on
Chemical Admixture	Use	Batching Plant only	Most Concrete
	Popular Type	Water-Reducing and Set-Retarding Type	AE and Water-Reducing Type
	Standard	ASTM C494	JIS A 6204

CONCRETE

An example of concrete test results in Vietnam is shown in Table 4.
Typical fine aggregates in South Vietnam were used for concrete. According to test results, unit water of concrete is higher than that of concrete in Japan, mainly because of fine aggregate quality. However, it is not clear because many factors such as following items affect the fresh concrete and hardened concrete properties.

- Temperature (Curing) : 27degrees in Vietnam 20degrees in Japan
- Air Content : Non-AE in Vietnam AE Concrete in Japan
- Mixer : Tilting Type is popular in Vietnam Forced Type in Japan
- Mixing Time
- Quality of Water for Concrete
- Coarse Aggregate Shape
- Test Methods and so on

Table 4 An Example of Concrete Test Results in Vietnam

Fine Aggregate	F.M.	W/C (%)	Unit Weight (kg/m^3)		Slump (cm)	Compressive Strength (N/mm^2)		
			Water	Cement		3days	7days	28days
Fine Sand	1.89	55.1	193	350	16	21.4	28.3	38.4
Good Quality	2.78	52.6	184	350	15	27.4	35.1	48.3
Fine Sand + 60% Grinding Stone	2.91	50.3	176	350	15	30.6	39.3	53.0

Example of test results of compressive strength is shown in figure 2.

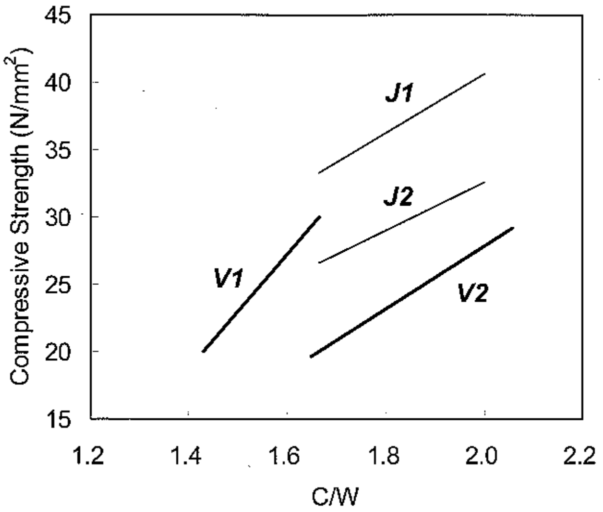

Figure 2 Examples of Relationship between C/W and Compressive Strength

V1: In case of using fine aggregate with low fineness modulus in Vietnam

V2: In case of using normal quality of fine aggregate in Vietnam

J1: In case of using high quality of aggregate in Japan

J2: In case of using normal quality of aggregate at BP in Japan

According to this figure, in case fineness modulus of fine aggregate in Vietnam is small (V1), unit water and unit cement increases. As a result, although drying shrinkage shall increase, concrete strength shall be higher than that using normal quality of fine aggregate (V2) because amount of aggregate decreases. In case of using high quality of aggregate in Japan, concrete strength is higher (J1), however actual relationship between C/W and compressive strength at average batching plant is located around the line of J2.

For reference, there are some consultant companies to evaluate concrete properties in Vietnam, but it is rather difficult for them to execute special test such as evaluation of durability and thermal property, so we have little data of durability and thermal properties.

READY-MIXED CONCRETE

Ready-mixed concrete conditions are much different each other as below.

<Japan>

- Production amount of ready mixed concrete — 120 million-m^3/year
- Number of ready-mixed concrete company — around 3,800
- Number of ready-mixed concrete batching plant — around 4,300

Production amount of ready mixed concrete in the last 10 years is shown in Fig 3. It is well known that production amount and ready-mixed concrete company decreases year after year. Regarding standard, “Ready-mixed concrete” is specified in JIS A 5308.

<Vietnam>

It is widely recognized that production amount and batching plant increases year after year. It is expected that number of batching plant including concrete product plant is only around 100 in Vietnam.

As opposed to Japan, there is no effective standard or guideline against batching plant in Vietnam. So, it is supposed that one of important theme will be to set up an effective standard or guideline for ready-mixed concrete, including facilities and method for quality control.

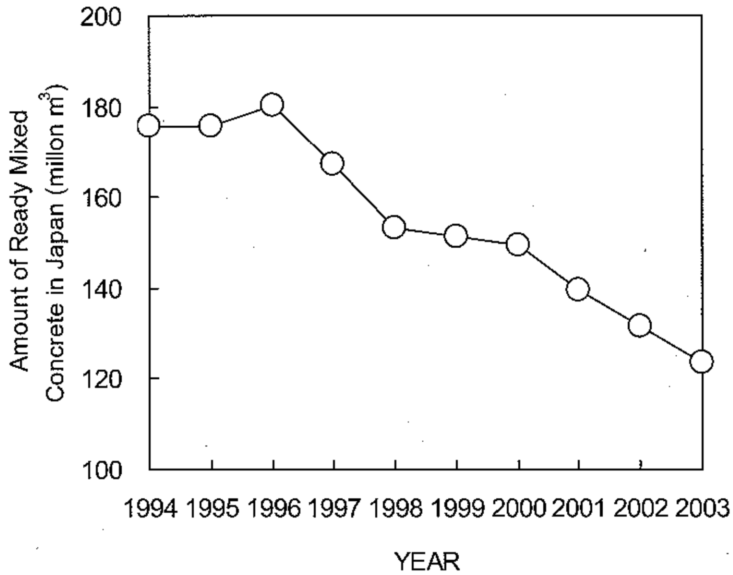

Figure 3 Production Amount of Ready Mixed Concrete in Japan

The some main items which are specified in JIS and actual conditions of batching plant in Vietnam are shown in Table 5.

Table 5 Comparison of Conditions for Ready-Mixed Concrete

	Japan (JIS A 5308)	Vietnam (Actual Condition at General BP)
Strength (N/mm^2)	18 - 60	20,25,30 are popular. (Actually some BP can produce more than 60.)
Type of Concrete	Normal, Light Weight, Paving and High Strength	Some Special Concretes are specified in TCVN or TCXDVN
Slump (cm)	2.5 - 21	Depending on Construction
Slump Flow (cm)	50, 60	None
Air Content (%)	4.5% except for Light weight concrete	Non-AE except for big project
Storage of Material	decided in detail	Depending on Batching Plant (or Construction)
Precision for Measuring		
Quality Control		

CONCLUDING REMARKS

Based on above information, we can grasp the difference of conditions on cement and concrete technology between in Vietnam and in Japan. The concrete condition in Vietnam, especially ready-mixed concrete (batching plant), is far from that in Japan. It's a pity that there are some actions of adding water after mixing, no measure of surface moisture and so on here and there at some batching plant now. So, it will be important not only to make effective guideline but also to make use of technical system effectively in Vietnam. And we hope this seminar including our information will help technology of each country advanced, especially in Vietnam, because concrete technology is making rapid progress in Vietnam now.

The author is deeply indebted to special support arrangement by VIFCEA, JSCE and Ho Chi Minh City University of Technology, and will make effort to contribute technology development on cement and concrete in both Vietnam and Japan in future.

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