

High-Performance Proceedings: ACI International Conference Concrete

Malaysia, 1997

Design and Materials and Recent
Advances in Concrete Technology

V. M. Malhotra



SP-172



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High-Performance Concrete: Design and Materials and Recent Advances in Concrete Technology

**Proceedings
Third CANMET/ACI
International Conference
Kuala Lumpur, Malaysia, 1997**

Editor

V. M. Malhotra



SP - 172

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Printed in the United States of America

Editorial production: Julie Carpenter

Library of Congress catalog card number 97-075035

Cover photo: Cesar Pelli & Associates, reprinted with the permission of Thornton-Tomasetti Engineers, New York.

PREFACE

In December 1991, the American Concrete Institute sponsored an International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design. This was in keeping with the mission of the Institute, and specific on emerging concrete technology in the Pacific Rim region, and to provide technical support for a growing number in ACI International chapters and members. The conference was held in Hong Kong, December 2-6, 1991. More than 80 papers were published in the conference proceedings issued by ACI as SP-128. As a follow-up of the very successful Hong Kong conference, ACI Board of Direction approved a second international conference for 1994. The theme of the conference was High-Performance Concrete and the conference was held in Singapore. The proceedings of the conference were published as SP-149.

The third international conference in this series was held in 1997 in Kuala Lumpur. The theme of the conference was "High-Performance Design and Materials and Recent Advances in Concrete Technology". More than 90 papers from 25 countries were received and reviewed in accordance with the policies of the American Concrete Institute, and 52 were accepted for publication as SP-172. In addition, a number of papers were selected for presentation and for publication as supplemental papers. To all those authors whose submission could not be included in the conference program, the Institute and Conference Organizing Committee extend their appreciation for their interest and hard work. Thanks are extended to members of the review panel who met in Las Vegas in December of 1996 to review the submissions. Without their prompt review and constructive comments, it would not have been possible to publish ACI SP-172 for distribution at the conference. The cooperation of the authors in accepting reviewers' suggestions and revising their manuscripts accordingly is greatly appreciated.

The help and assistance of William R. Tolley and his staff for performing the administrative duties associated with this conference are gratefully acknowledged. Particular thanks are extended to Barbara Cheyne for her devotion to the conference work and for performing tasks sometimes well beyond the call of duty.

V. Mohan Malhotra, Chairman
ACI 1997 International Conference on High-Performance
Concrete: Design and Materials and Recent Advances in
Concrete Technology
Editor, SP-172

CONTENTS

PRECAST CONCRETE COMPOSITE DECK COMPONENTS FOR REHABILITATION OF BRIDGES
by Richard N. White. 1

RESISTANCE OF RICE HUSK ASH CONCRETE TO CARBONATION, ACID ATTACK AND CHLORIDE
ION PENETRATION
by Shuichi Sugita, Qijun Yu, Masami Shoya, Yoichi Tsukinaga and Yasuo Isojima. 29

STRENGTH OF SIFCON IN SHEAR AND TORSION
by S. K. Kaushik and S. P. Singh. 45

THE INFLUENCE OF SILICA FUME AND CURING TEMPERATURE ON THE STRENGTH OF HSC
by Abdullah M. Alshamsi. 57

RESISTANCE OF SILICA FUME CONCRETE TO DEICING SALT SCALING--A REVIEW
by M. H. Zhang, N. Bouzoubaa and V. M. Malhotra. 67

THE USE OF A FINELY-DIVIDED MINING BY-PRODUCT IN RCC FOR DAM CONSTRUCTION
by Richard Gagné, Danielle Martin, Francine A. Blais, Alain Prézeau and Salsutio Guzman. 103

BEHAVIOR OF HIGH-STRENGTH CONCRETE BEAMS
by Richard H. Scott. 119

FACTORS INFLUENCING THERMAL STRESSES IN HPC MEMBERS
by Arshad A. Khan, William D. Cook and Denis Mitchell. 135

DURABILITY OF CONCRETE REPAIRS: CURRENT PROBLEMS AND FUTURE PROSPECTS
by P. H. Emmons, A. M. Vaysburd and J. E. MacDonald. 155

RESIDUAL STRENGTH OF HIGH-PERFORMANCE CONCRETE SUBJECTED TO HIGH
TEMPERATURES
by N. Gowripalan, P. Salonga and C. Dolden. 171

IDENTIFYING THE ALKALI REACTIVITY OF CONCRETE AGGREGATES--TESTS ON AUTOCLAVED
CONCRETE SAMPLES
by Salil K. Roy, Poh Kong Beng and Derek O. Northwood. 193

EFFECTS ON CRACK MEASUREMENT CAUSED BY STEEL REINFORCING BARS by Yiching Lin and Tzonghow Liou.....	209
STATIC AND CYCLIC LOADING OF RC BEAMS UPGRADED BY JACKETING by Hee Kiat Cheong and Niall MacAlevey.....	225
THE DUCTILE BEHAVIOUR INCLUDING FLEXURAL STRENGTH OF HIGH-STRENGTH CONCRETE MEMBERS SUBJECTED TO FLEXURE by Sung-Woo Shin, Suk-Hyung Yoo, Jong-Moon Ahn and Kwang-Soo Lee	247
USE OF HIGH VOLUME FLY ASH IN GROUTING APPLICATIONS by J. Mirza, K. Saleh, V. Roy and M. S. Mirza.....	281
EFFECT OF THE COMBINED USE OF ULTRA-FINE FLY ASH AND SILICA FUME ON STRENGTH OF HPC by Sun Wei, Pan Ganghua and Ding Dajun.....	299
MASS AND HIGH-STRENGTH CONCRETE FOR HIGH-RISE BUILDINGS IN INDONESIA by Hyung-Kyun Byun and Kwang Myong Lee.....	313
THE PETRONAS TWIN TOWERS AND HIGH-PERFORMANCE CONCRETE by Charles H. Thornton, Hamdan Mohamad, Udom Hungspruke, Leonard M. Joseph and Hashimah Hashim.....	329
EFFECT OF CURING AND ADDITION OF POLYPROPYLENE FIBERS ON SELECTED PROPERTIES OF HIGH-STRENGTH CONCRETE by Saud F. Al-Otaibi and Waheeb Al-Khaja.....	351
THE ROLE OF CORROSION INHIBITORS IN HIGH-PERFORMANCE CONCRETE by Theodor A. Bürge.....	375
STUDY ON THE ABILITY TO PASS BETWEEN STEEL REINFORCING BARS OF HIGHLY-FLOWABLE CONCRETE by Hiromi Fujiwara, Shigeyoshi Nagataki, Akito Dozono and Akira Obatake	397
AN EXPERIMENTAL STUDY FOR THE FATIGUE CRACK GROWTH BEHAVIOR OF CONCRETE by Jin Keun Kim and Sang-Hun Han.....	413
CARBON FIBER-REINFORCED POLYMER (CFRP) TENDONS SUBJECTED TO COMBINED AXIAL LOAD AND HARPING by Shuaib H. Ahmad, Carl V. Jerrett and Paul Zia	427
HIGH-PERFORMANCE CONCRETE PLACED UNDER WATER by Kamal Gad Sharobim.....	445

AUSTRALIAN EXPERIENCE WITH HIGH-PERFORMANCE CONCRETE FLOORS AND PAVEMENTS by R. L. Munn, I. I. Dumitru, G. Smorchevsky and A. J. Zdrilic.	465
STUDIES ON HIGH-PERFORMANCE CONCRETE AND ITS APPLICATION by Lin Baoyu, Shan Guoliang and Cai Yuebo.	485
TESTING OF FIBER-REINFORCED STRUCTURAL CONCRETE ELEMENTS by Sidney Mindess, Perry Adebar and Julien Henley.	495
FLEXURAL STRENGTH AND TOUGHNESS OF STEEL FIBER HIGH-STRENGTH CONCRETE by Mahmoud A. Imam.	517
HYBRID EFFECTS OF FIBER-REINFORCED CONCRETE ON FRACTURE TOUGHNESS by Takashi Horiguchi and Kohji Sakai.	535
MINIMUM FLEXURAL REINFORCEMENTS OF HIGH-STRENGTH CONCRETE BEAMS by Faisa F. Wafa and Samir A. Ashour.	549
FLEXURAL BEHAVIOR AND DUCTILITY OF PRESTRESSED BEAMS WITH HIGH-STRENGTH CONCRETE by N. Gowripalan and X. W. Zou.	573
FLEXURAL PERFORMANCE OF STRENGTHENED RC BEAMS REPAIRED WITH POLYMER RESINS by Oan Chul Choi, Young Soo Shin, Gi Suop Hong, Young Kyun Hong, Byung Guk Kim and Long Choi.	601
MIXTURE PROPERTIES OF SELF-COMPACTING, HIGH-PERFORMANCE CONCRETE by Naoki Nagamoto and Kazumasa Ozawa.	623
MECHANICAL PROPERTIES OF SELF-FLOWING CONCRETE by Jin-Kuen Kim and Sang-Hun Han.	637
RHEOLOGICAL PROPERTIES OF SELF-COMPACTING, HIGH-PERFORMANCE CONCRETE by Hoom Kim, Yon-Dong Park, Jaeho Noh, Yongsum Song, Chungho Han and Suckhwa Kang.	653
BEHAVIOR OF FERROCEMENT COMPOSITE COLUMNS IN COMPRESSION by S. K. Kaushik and S. P. Singh.	669
AN EXPERIMENTAL STUDY ON HIGH-STRENGTH CONCRETE COLUMN STRENGTH SUBJECTED TO ECCENTRIC LOADS by Jae-Hoon Lee, Hae-Geun Park and Young-Shik Park.	683
CORE STRENGTHS OF HIGH-STRENGTH CONCRETE by George C. Hoff and Radoslav Elimov.	705

EVALUATION OF THE ASTM STANDARD CONSOLIDATION REQUIREMENTS FOR PREPARING HIGH-STRENGTH CONCRETE CYLINDERS

by N. J. Carino, G. M. Mullings and W. F. Guthrie. 733

MIX PROPORTIONING AND PROPERTIES OF ROLLER-COMPACTED CONCRETE PAVEMENT USING LIGNITE FLY ASH

by Somnuk Tangtermsirikul and Yukio Aoyagi. 769

ANALYSIS AND OPTIMIZATION OF THE PROPERTIES OF FINE-GRAINED CEMENTLESS CONCRETE BY MEANS OF NATURAL MATHEMATICAL MODELS

by S. I. Pavlenko and L. P. Myshlyaev. 785

DEVELOPMENTS IN THE MANUFACTURE AND USE OF PORTLAND LESTONE CEMENT

by Graeme K. Moir and Steven Kelham. 797

PROPERTIES OF HIGH-PERFORMANCE, HEAT-CURED SLAG CEMENT CONCRETE FOR BRIDGE STRUCTURES

S. L. Mak, G. Foster, G. Chirgwin and D. W. S. Ho. 821

STRESSES OF HIGH-STRENGTH CONCRETE DUE TO AUTOGENOUS SHRINKAGE COMBINED WITH HYDRATION HEAT OF CEMENT

by Ryoichi Sato, Ming Xu and Yang Yang. 837

EFFECT OF TEMPERATURE AND TRIAXIAL COMPRESSION ON ENGINEERING BEHAVIOR OF HIGH-PERFORMANCE CONCRETE

by Xiaojun Li and Gerard Ballivy. 853

CHARACTERISTICS AND APPLICATION OF HIGH-DURABILITY CONCRETE

by Byung-Hwan Oh, WonKi Chung, Seun-Yup Chang and Kyu-Jung Lee. 871

RESISTANCE OF HIGH-QUALITY CONCRETE (HQC) TO CHLORIDE ION PENETRATION

by Tarun R. Naik, Frederick H. Gustin and Shiw S. Singh. 899

THE USE OF FOAMCRETE IN SOUTHERN AFRICA

by E. P. Kearsley and H. F. Mostert. 919

GROUTING OF DRY JOINTS IN HUITES DAM, CHOIX, MEXICO

by William J. Clark, Lois D. Schwarz and Prospero Ortega Moreno
reviewed by Bryant Mather and Scanlon. 935

HIGH-PERFORMANCE CONCRETE INCORPORATING ZEOLITE, FLY ASH AND SILICA FUME

by Sammy Y. N. Chan and Xihuang Ji. 951

BOND BETWEEN HIGH-PERFORMANCE CONCRETE (HPC) AND REINFORCING BARS IN SPLICES OF BEAMS

by Mohammad-Reza Esfahani and B. V. Rangan..... 971

BEHAVIOR OF FIBER-REINFORCED CONCRETE SLABS UNDER LOW VELOCITY PROJECTILE IMPACT

by K. C. G. Ong, P. Paramasivam and M. Basheerkhan..... 993

SP 172-1

Precast Concrete Composite Deck Components for Rehabilitation of Bridges

by Richard N. White

Synopsis: The first part of this paper provides a brief review of the critical issues met in replacing deteriorated bridge decks with precast deck systems, drawing on existing papers published by PCI and ACI. Recommended practices for typical bridge rehabilitation projects are provided.

The second topic focuses on the rehabilitation of the Tappan Zee Bridge over the Hudson River near New York City. Results are presented for an experimental program conducted on a 10 m span full-scale lightweight concrete slab-steel beam composite bridge deck unit. Loading history included 10 million cycles of flexural fatigue loading, followed by a flexural load capacity test. Measured values of capacity and mid-span deflection at this ultimate load level are compared with simplified analytical predictions.

Keywords: Bridge decks; bridges (structures); composite construction; cyclic loads; fatigue; flexural strength; lightweight concrete; precast concrete; reinforced concrete; tests

2 White

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A BRIEF SURVEY OF BRIDGE DECK REPLACEMENT STRATEGIES

Tens of thousands of highway bridges in the U.S. (and elsewhere) are in various stages of deterioration. Bridge decks have a particularly demanding environment. The decks must carry the dynamic, cyclic loading effects of trucks and other traffic, while also being exposed directly to the environmental elements, including thermal effects, rain and snow, freezing and thawing action, and the deleterious effects of de-icing salts in northern climates.

The criteria for successful bridge rehabilitation are quite simple; they include the following (where relative importance varies from bridge to bridge):

- provide a higher level of durability than in the original bridge
- minimize traffic disruption during rehabilitation
- reduce dead load, if possible, to increase load capacity
- provide a reasonable construction cost.

In recent years the use of full depth precast panels has become quite popular for replacing existing bridge decks. A typical application is shown in Fig. 1. A comprehensive review of various applications of both reinforced and prestressed full depth precast decks is given in the state-of-the-art paper by Issa et al (1). A survey of actual in-field performance of these deck systems is provided in (2), and construction procedures for rapid replacement of bridge decks are described in (3). Because of space limitations, only the most important design features and conclusions contained in these papers will be discussed here, as follows:

a. Disruption to traffic must be minimized, particularly on bridges located on major highways. Special construction procedures using

precast deck units are based on having a partial width of the bridge closed to traffic overnight. A relatively short length of the closed deck is removed and replaced the same night, and the full bridge width is available for the morning traffic. The process is repeated until the full length of both halves of the bridge is rehabilitated.

b. Precast deck panels are usually quite short but with high loading effects near the ends of the panels. The necessary complete development of pretensioning strands is not possible, and a fully prestressed system cannot be utilized. Thus it is customary to utilize epoxy coated reinforcing steel for primary load-carrying capabilities in the long direction of the precast panel, often supplemented by a modest level of prestressing to prevent cracking from stresses induced during handling and installation .

c. Composite action of the new deck and the main supporting members is always advantageous and often necessary to provide adequate bridge capacity after rehabilitation. This requires blockouts (openings) in the deck panels for installation of shear connectors for transfer of horizontal shearing forces between the deck and the supporting girders. The top flanges of steel girders are fitted with either welded shear studs or welded channel connectors. The shear connection to the slab is completed by filling each opening with a high-quality, durable concrete (polymer or epoxy concrete if quick curing is required). Typical blockout and shear connections are shown in Fig. 2.

d. Joints between adjacent panels present one of the most vexing obstacles met in achieving a durable precast concrete deck system. The female-to-female joint detail shown in Fig. 3, with a polymer or epoxy concrete gap filler (which can reach strengths of 30 MPa in one hour) appears to give superior performance. The initial gap opening is on the order of 12 mm. Since panels are not perfect in their dimensions, lack-of-fit dimensions of up to 12 mm under or over can be accommodated with this detail. Design, materials, construction, and durability issues for the joint are presented by Gulyas (4). For panels spanning transversely to traffic direction, prestressing is provided in the direction of traffic to further secure good joints and to help resist the shearing and bending actions produced by traffic wheel loads.

e. Most reported cost analyses show that construction costs are slightly higher for precast systems than for cast-in-place slabs, but this cost differential is more than made up in favor of precast systems from the substantial reductions in traffic delays and re-routing as well as in

4 White

construction time savings.

Variations on the full thickness concrete deck panels also find application in bridge rehabilitation, as will be shown in the following discussion of a system using lightweight concrete deck panels precast compositely with shallow steel beams.

No matter which system is used, concrete durability is a critical concern. Bridge decks present a particularly fruitful opportunity for the use of high performance concrete, and the use of precast deck systems offers the advantage of tight quality control and optimal curing conditions to achieve durability.

EXPERIMENTAL STUDY OF PRECAST DECK UNIT FOR THE TAPPAN ZEE BRIDGE, NEW YORK, USA

This second part of the paper summarizes the results of an experimental program conducted at the George Winter Structural Research Laboratory at Cornell University on a full-scale composite lightweight concrete slab-steel wide flange bridge panel, 2.44 m wide and 10.26 m long, supported on a simple span of 9.85 m. The tested section (Fig. 4) was quite similar to a design proposed for a replacement deck on the Tappan Zee Bridge in New York State. The primary reason for conducting the testing program was to qualify this new type of composite construction for use in New York State highway bridge decks.

The Tappan Zee Bridge

The 5-km long Tappan Zee Bridge crosses the Hudson River about 10 km above the New Jersey-New York state line, carrying more than 100,000 commuters each day, plus major truck traffic, on Interstate 287. This 41 year old steel truss structure is scheduled to have its entire cast-in-place concrete deck replaced (a \$70 million project), along with repairs to steel members above and below the deck (\$9 million) and additional structural protection against ship collision impact (\$50 million).

According to Mr. Keith Giles, division director of the New York Thruway Authority, "Our goal is to keep all lanes open during peak hours. We have a movable barrier that we can use to make sure there are always four lanes in the commuter direction. Traffic stoppage is not in our game plan". The full rehabilitation project, which also includes a

number of other smaller projects, may take up to 4 years.

The mid-1996 contract signed for the Tappan Zee Bridge rehabilitation calls for replacing a 15.2 m length of a single 4 m wide lane each night, using a 178 mm thick normal density concrete slab acting compositely with galvanized 27 WF beams. The initial installation, which began in early May 1997, involves replacement of a complete lane down the middle of the bridge, closing the lane at 9 pm and re-opening the next morning at 6 am. The penalty for re-opening later than 6 am is \$1000 per minute.

Scope of Experimental Study

Phase I of the experimental program included applying 10 million cycles of loading to simulate the maximum flexural effects of HS 25 truck loading. Fig. 6 shows the panel in position for flexural fatigue testing. Loading was applied through two beams spaced 1.22 m apart and spanning transversely across the 2.44 m width of the panel.

Phase II of the study consisted of a flexural capacity test of the bridge panel. Static loading, applied through a beam system similar to that used in the fatigue loading, was increased gradually up to failure.

Bridge Panel Geometry and Material Properties

Critical cross-sectional dimensions for the bridge panel are shown in Fig. 4(c). The panel consisted of a 127 mm thick lightweight concrete deck supported on four steel wide flange W16X31 beams, with composite action between concrete and steel beams provided by two lines of 19 mm diameter by 76 mm long shear studs spaced uniformly at 165 mm on each beam. The unit was fabricated utilizing the **Inverset** process at the production facilities of The Fort Miller Company in Schuylerville, NY. The **Inverset** process involves supporting the beams at their ends, hanging slab forms from the beams, and placing the concrete deck under the beams (Fig. 5). After the concrete has gained sufficient strength, the entire unit (slab and beams) is inverted to produce a composite system with a high quality (bottom-cast) slab top surface and a modest prestressing action and upward camber resulting from the downward bending action of the fresh soft concrete when it was placed in the form suspended under the steel beams.

The deck slab concrete was made with Type I portland cement, sand, and lightweight aggregate, proportioned to have a unit density not less

6 White

than 1840 kg/m^3 and a 28 day compressive strength of 34 MPa. Measured concrete strength at 139 days was 46.3 MPa (average strength of two 150 by 300 mm cylinders), indicating an approximate 28 day cylinder strength of 40 MPa. Concrete modulus was measured with six tests over a 222 mm gage length on each of two cylinders up to approximately 35% of their compressive strength, giving average values of $E = 22.5 \text{ GPa}$ and 24.3 GPa at 139 days.

Two cylinders were tested just prior to the ultimate load test of the bridge, when the concrete was 1.22 years old. Results were as follows: compressive strength = 60 MPa and 60 MPa, and modulus of elasticity = 24.8 GPa and 21 GPa .

Steel girders were A572 Grade 50 (345 MPa yield), and the slab reinforcement was epoxy coated Grade 60 (414 MPa yield).

Test Set-up and Loading System

Flexural fatigue loading, applied at the rate of 1 cycle every 2 seconds, simulated the flexural effects an HS25 truck loading, including an impact factor of 0.30. The required load, including the weight of the load distribution system, was 236 kN. Load was applied with an MTS 245 kN capacity hydraulic actuator and was divided into equal loads with a three-level loading system as shown in Fig. 6. The net target actuator loading was 231 kN.

It should be noted that no attempt was made to simulate the absolute maximum end reactions and shear forces that would be produced by HS25 loading, nor were the local effects of concentrated wheel loads on the slab simulated by the loading. A parallel experimental program at the University of Oklahoma investigated the punching shear strength of an identically designed slab-beam unit.

The four steel beams of the bridge panel were supported at each end on W 5X16 steel beams resting on concrete pedestals as shown in Fig. 6. 13 mm thick neoprene pads of the type typically used for bridges -- 55-65 hardness, 10 MPa tensile strength and a temperature range of -29°C to 77°C -- were placed between the steel beams and the transverse supporting beams to help equalize loadings and to permit longitudinal motion of the beams during load application. At one end of the panel, each beam was restrained against longitudinal motion.

Instrumentation

49 channels of instrumentation were used -- 19 channels of displacement and load transducers to measure center span and end support deflections, slip of the concrete deck with respect to the top flanges of the steel beams, and load magnitude and displacement of the hydraulic actuator, and 30 electric resistance strain gages to measure strains on the steel beams, in the concrete in the plane of the reinforcing bars, and on the top and bottom surfaces of the concrete deck. All transducers and gages were connected to a computer-controlled data acquisition system which provided rapid data reading and ample capacity for digital storage.

Cyclic Loading Test Results

Deflections and Stiffness of Panel -- Maximum mid-span deflection was nearly constant at about 15 mm throughout the 10 million load cycles. The corresponding flexural stiffnesses of the bridge panel (applied load divided by net mid-span deflection) are plotted in Fig. 7. The general trend in response was a slight increase in stiffness up to about 2 million cycles, a small decrease from 2 million to 3.5 million cycles, and nearly constant stiffness during the remaining load cycles. The stiffness after 10 million cycles, 15.5 kN/mm, decreased 3.2% from the value of 16.0 kN/mm measured after 100,000 cycles.

Potential residual (permanent) deflections accumulated during the cyclic loading testing over a period spanning many months were measured with a surveyor's level. Measurements taken before (5/21/93) and after (8/2/93) the 2 million load cycles indicate that the residual net deflection at midspan was 0.15 mm. Similar measurements taken on 9/9/93 after additional cycling showed an additional 0.05 mm residual displacement. Readings taken after 8.5 million cycles (on 3/10/94) and after 10 million cycles (on 5/12/94) showed total residual displacements of 0.79 mm and 1.17 mm, respectively.

Recovery of displacement as a function of time after unloading was measured after 8.5 million load cycles, but results are not reported here because measurement reliability was questionable -- the measured small values of displacement were highly sensitive to temperature changes in the laboratory environment. Thus there is no credible data available on the issue of possible creep effects from the cumulative effects of having live load applied to the bridge for so many cycles. However, given that the residual displacement did not exceed 1.17 mm, cumulative live load creep effects were negligible.



8 White

Strains and Stresses in Beams and Slab; Fatigue Strength of Concrete -- Measured strains produced by a static load of 231 kN applied at the conclusion of two, six, and ten million load cycles for each of the four beam locations are plotted in Fig. 8. The beams are identified as E1, I1, I2, and E2, where E and I denote exterior and interior beams, respectively. Strain measurements were made with gages bonded to (a) the top of the concrete slab, (b) the top of the top flange of the steel beam, and (c) the bottom of the bottom flange of the steel beam. The strain values increased slightly during the progression of static loading cycles executed every 500,000 load cycles, which is consistent with the measured displacement data.

Midspan flexural stresses produced by the HS25 (+ impact) moment were about 103 MPa tension in the bottom flanges of the four steel beams, and about 5.5 MPa compression in the top of the concrete slab immediately above the beams.

Although fatigue strength of concrete tends to increase when loading is stopped for taking strain and displacement readings, as was done here, it is believed that this effect does not provide any appreciable enhancement of fatigue strength. Countering this strength enhancement effect is the fact that actual passage of trucks on the real bridge would not be nearly as rapid as the basic loading regime used in this laboratory study, where many years of truck traffic were simulated in about one year's time.

Composite Action Between Slab and Steel Beams -- Potential slip between the concrete deck and the top flange of the steel beams was measured at both ends of all four beams. With the exception of the right ends of beams I1, I2, and E2, measured slips did not exceed 0.008 mm, which is the same order of magnitude as the sensitivity of the gages (noise level threshold). There was no discernible trend of increasing slip with increasing number of load cycles. Thus essentially full composite action was maintained during the 10 million load cycles.

Nondestructive Investigation of Concrete Slab Condition - The concrete deck slab was analyzed with a commercially available impulse-echo flaw detection system. Three sets of data were taken -- before loading began, after 2 million load cycles, and after 10 million load cycles. This analysis was done by the developer of the equipment, Cornell faculty member Dr. Mary Sansalone, and two structural engineering graduate students. The impulse-echo equipment was used to detect possible slab cracking or delamination, to investigate potential changes

High-Performance Concrete 9

in the quality of contact between concrete and steel beam flange at the deck-girder interfaces, and to reveal any other potential flaws. The three sets of data gave essentially identical results, indicating no separations of the deck from the top flanges of the steel beams and no measurable internal laminar cracking.

Flexural Capacity Testing Program

Loading System and Modifications to Instrumentation -- The bridge panel was moved to a stronger reaction frame consisting of two hold-downs anchored into the laboratory floor, with a transverse reaction beam spanning between the two hold-downs. Load was applied with a 1780 kN capacity Enerpac hydraulic ram acting between the hold-down system and a (stronger) load distribution system similar to that used for the fatigue testing program.

Load was applied in a load-control mode in increments of 90 kN for the first 623 kN and was then reduced to 45 kN increments as yielding was approached. At higher load levels, there was significant reduction of load at the end of each increment when loading was stopped to permit inspection.

Several changes were made to the instrumentation system used for the fatigue testing program, including: (a) beam deflection at mid-span was measured for one beam only, using a DCDT for displacements up to 100 mm, and an engineering scale to get approximate values of larger deflections, and (b) potential slip between the top flanges of the steel beams and the concrete slab was measured at 10 locations -- both ends of the four beams, and at 1.83 m and 3.66 m from the south end of Beam E1.

Load-Deflection Response; Panel Stiffness -- Mid-span displacement vs. applied load is plotted in Fig. 9. Response was linear to a load of about 490 kN (about 2.1 times flexural design load). At higher loads, gradual yielding of the steel beams and nonlinear action in the concrete resulted in increasing nonlinearity. At a load of about 1245 kN, problems with the loading system required an unloading. As shown in Fig. 9, the bridge panel unloaded with essentially the same stiffness as the original linear stiffness. The load system was repaired and reloading faithfully followed the unloading path. Reduction of stiffness is plotted in Fig. 10. Just before failure, with the load in excess of 1335 kN, and with a midspan deflection of 203 mm ($L/50$), the panel maintained about 40% of its original stiffness.

10 White

Strain Magnitudes and Distributions -- The bottom fiber strains in the exterior beam showed nonlinearity after a load of about 490 kN. Bottom fiber interior beam strains remained linear to higher load levels, approaching 670 kN, and then increased sharply because the exterior beams had yielded first and could not "take up" much additional load as the yielding interior beams tried to shed load.

The top fiber steel beam strain plots reached a maximum value of approximately 0.00013 to 0.00015 at a load of about 755 kN. At higher loads, strain increments became tensile, reducing the amount of compression until strains eventually reached zero at an applied load in excess of 1110 kN. This state of zero strain corresponded to a neutral axis located at the bottom of the slab thickness. At higher loads, tensile strains increased rapidly as the neutral axis continued to migrate upward.

Mid-span strain distributions at three different load levels (890, 1110, and 1245 kN) are plotted in Fig. 11. It can be seen that the assumption of plane sections remaining plane is satisfied very well even when the bridge panel had experienced severe plastic strains. The two strain values measured on the bottom flange were quite different at high loads because the steel beams were fabricated with an appreciable sweep in the bottom flange, which tended to straighten during plastification.

Concrete strains were measured on the top surface of the slab at three locations -- over the exterior beam E1, the interior beam I1, and at the center of the deck (Fig. 12). Response was linear up to a load of about 534 kN, with gradually increasing nonlinearity at higher loads. Maximum strains were about 0.0025 at a load level of about 1245 kN, the last opportunity to record reliable strain readings.

Behavior Near Ultimate; Failure Mode -- At the end of the test, with the load peaking at 1380 kN and the midspan deflection exceeding 200 mm, concrete strains above the girders were about 3500 microstrain and the average steel strain in the bottom flanges was over 10,000 microstrain (about 5 times the yield strain of the steel). The plane section assumption was still satisfied reasonably well at this advanced stage of behavior.

The failure mode was a "classical" underreinforced flexural mode, as had been predicted by a simple "back-of-the-envelope" analysis which gave estimated values of 1200 kN peak load with mid-span deflection = 180 mm. Continued yielding of the steel beams resulted in upward migration of the neutral axis and a more rapid increase in the concrete

compression strain levels: Failure occurred by sudden crushing of the top fibers of the slab near the center of the panel span (Fig. 13).

Composite Action During Test to Failure -- Slips in 3 of the 4 gages at the south end were quite consistent, showing a slip of about 0.038 mm. at a load of 890 kN, and reasonably linear response all the way to failure. Slip values at the north end were less consistent; all but Beam E2 values were comparable to the south end slips. There was cracking in the north end region of the beam E2 which may have loosened the gage mounting and perhaps degraded the composite action slightly.

Concrete Cracking -- Minor cracking became visible along the edges of the slab near the ends of the bridge panel, and on the slab top surface in the outside 300 mm width of the slab at a load of 467 kN. There were about 6 visible top surface cracks at each end of the deck panel. Crack widths were less than 0.05 mm. Very few additional cracks occurred until just before failure. A final set of impact-echo measurements showed little differences from the pre-test readings described earlier in this paper, thus indicating negligible lami- nar cracking and delamination during the ultimate load test.

SUMMARY AND CONCLUSIONS

A 9.85 m span lightweight concrete ***Inverset*** bridge panel system was loaded with (a) 10 million cycles of loading that produced a mid-span bending moment equivalent to that of an HS25 truck loading, including AASHTO-specified impact effects, and (b) a gradually increasing load up to failure of the panel. The fatigue loading rate was one cycle every 2 seconds. Absolute maximum shear effects and local wheel loading effects were not simulated with this loading system.

The panel sustained 10 million load cycles with no apparent damage and with small increases in peak values of flexural strain (as measured during static loading cycles applied between each set of 500,000 dynamically applied fatigue cycles). Flexural stiffness decreased about 3% during the 10 million load cycles. There was no discernible pattern of slip between the beam top flanges and the concrete slab.

In the ultimate load test, the panel remained essentially linear up to about 2.5 times the flexural design load. The panel carried an ultimate load of 1380 kN at a midspan deflection exceeding 200 mm, approxi-

12 White

mately L/50. This failure loading produced a maximum midspan bending moment about 6 times that of an HS-25 loading including 30% impact. The failure mode was gradual flexural, with considerable yielding action in the steel beams, and associated large deflections, prior to reaching critical strain in the top fibers of the slab. Seven of the ten measured slips between the beam top flanges and concrete deck were less than 0.04 mm at a load of 890 kN, and there was no indication of any significant degradation in the composite action between steel beams and concrete slab.

ACKNOWLEDGEMENTS

The advice and assistance provided by Peter J. Smith, Vice President for Engineering, The Ft. Miller Co. Inc., and by Stanley J. Grossman, Grossman and Keith Engineering Co. and developer of the *Inverset* system, are very much appreciated. Timothy Bond, technical manager of the George Winter Laboratory was responsible for the instrumentation and fatigue loading test. Prof. Mary Sansalone and her graduate students conducted the impact-echo NDT studies of the concrete slab.

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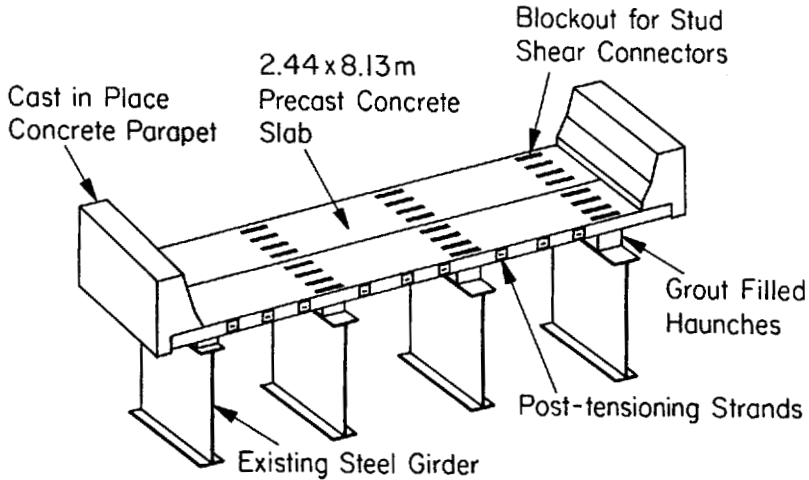


Fig. 1—Typical section of precast concrete full-depth slab decking--bridge 02200, Connecticut Department of Transportation

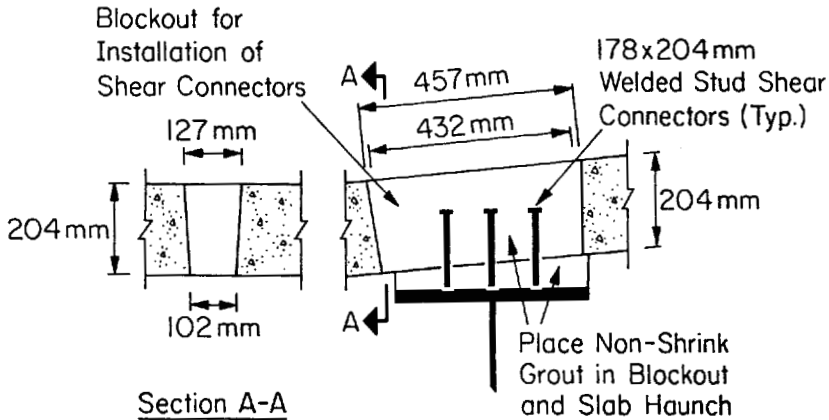


Fig. 2—Typical shear connector blockout detail--bridge 02200, Connecticut Department of Transportation

14 White

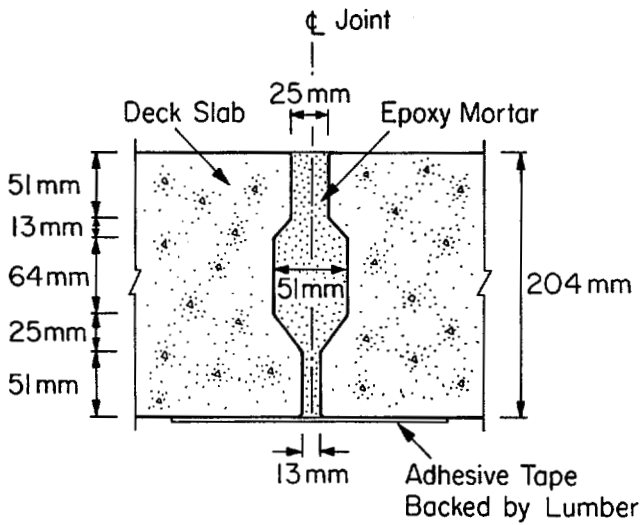


Fig. 3—Transverse joint between precast slabs-- New York State Thruway Authority

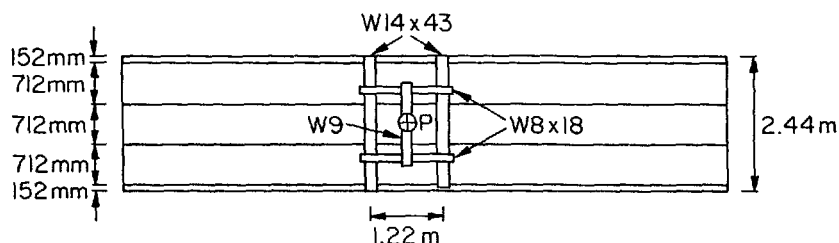


Fig. 4A—Composite deck unit geometry; plan view--bridge with loading elements

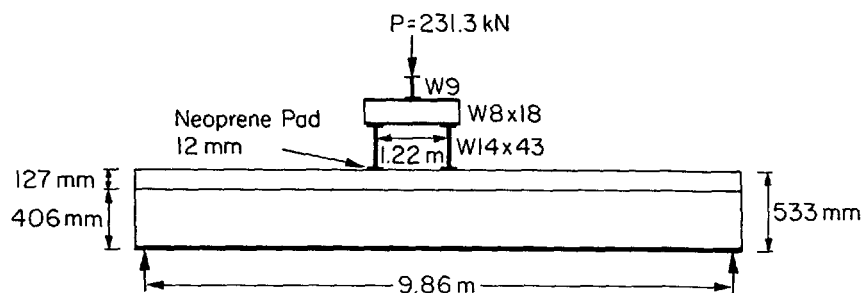


Fig. 4B—Composite deck unit geometry; elevation view--load applied through 3-level spreader assembly

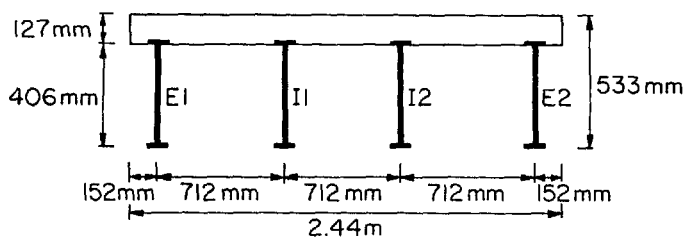


Fig. 4C—Composite deck unit geometry; cross-section through deck and steel beams

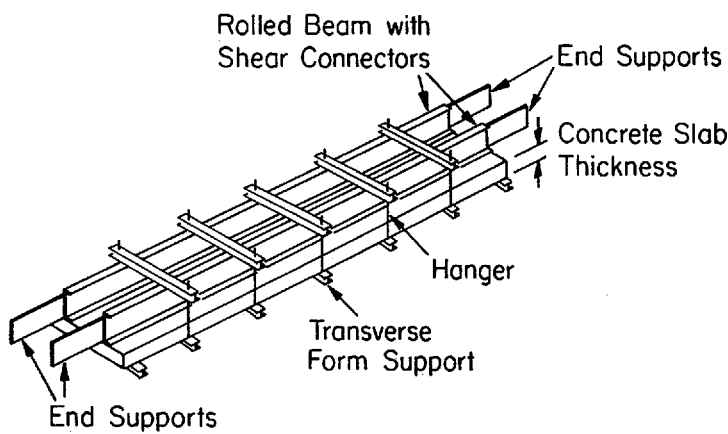


Fig. 5—*Inverted* precast composite deck construction, with forms hung from beams to support slab

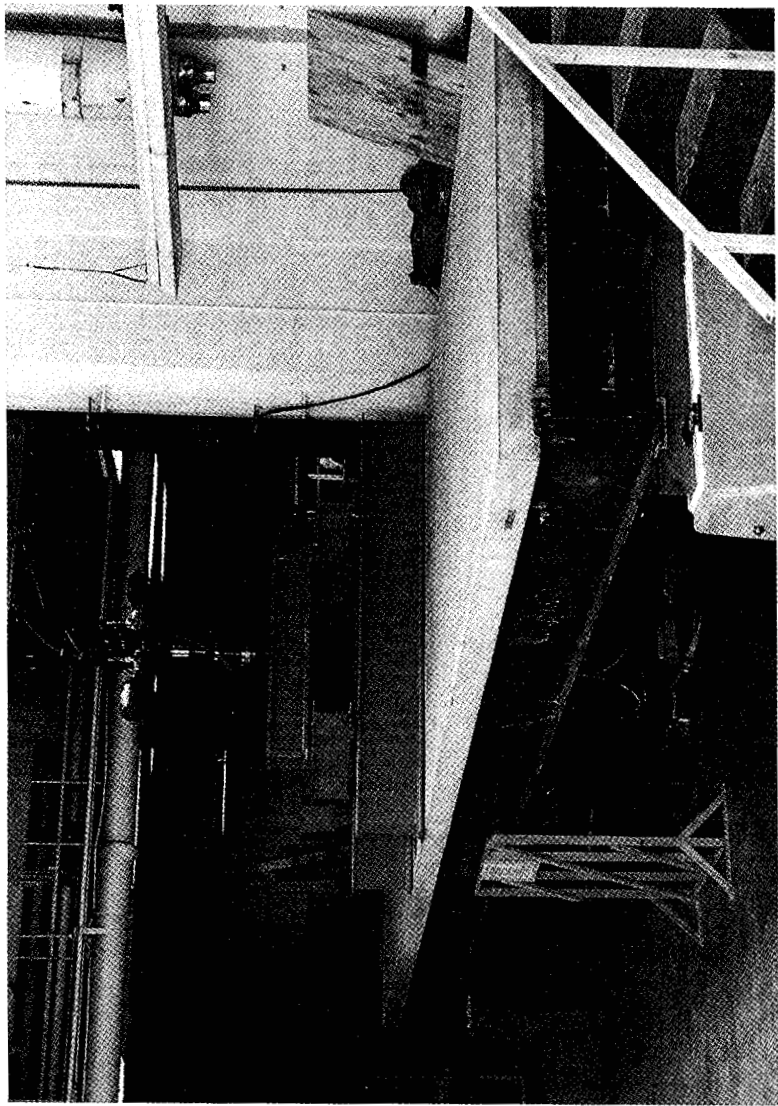


Fig. 6—*Inverset* bridge panel in position for cyclic loading

18 White

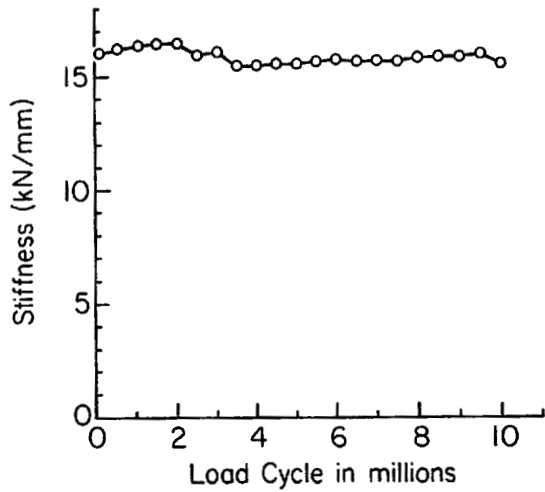


Fig. 7—Mid-span stiffness vs. load cycle, 0 to 10 million cycles

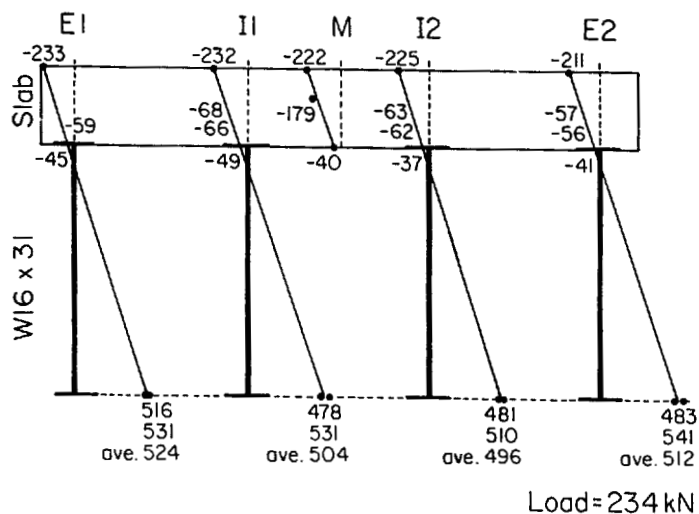


Fig. 8A—Mid-span strain distributions at 2, 6, and 10 million load cycles; cycle 2 million

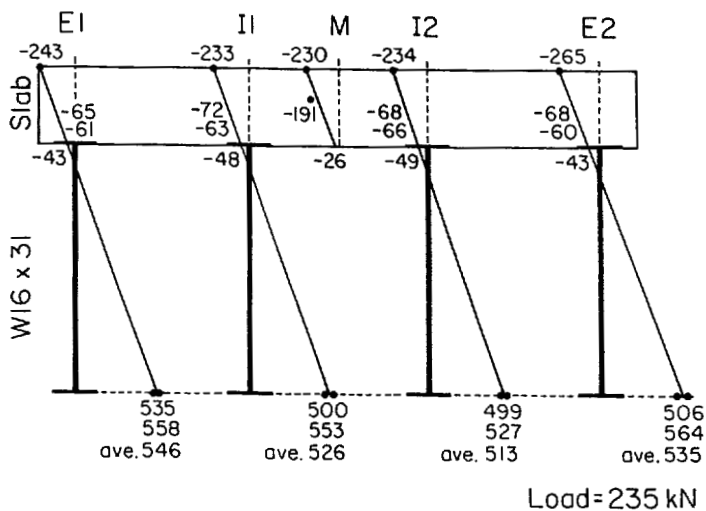


Fig. 8B—Mid-span strain distributions at 2, 6, and 10 million load cycles; cycle 6 million

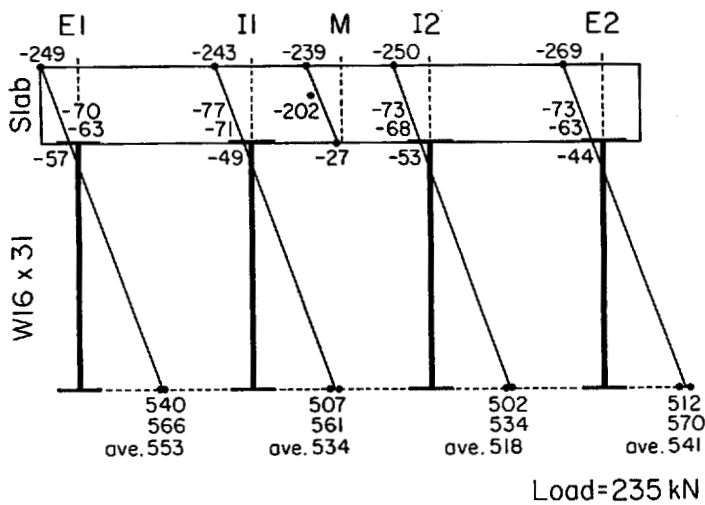


Fig. 8C—Mid-span strain distributions at 2, 6, and 10 million load cycles; cycle 10 million

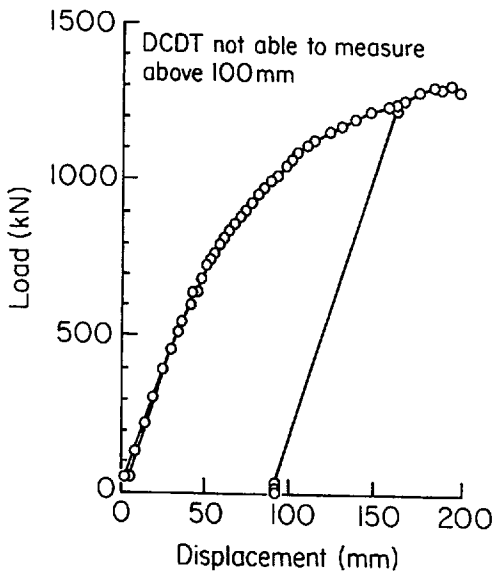


Fig. 9—Load vs. mid-span displacement, ultimate load test

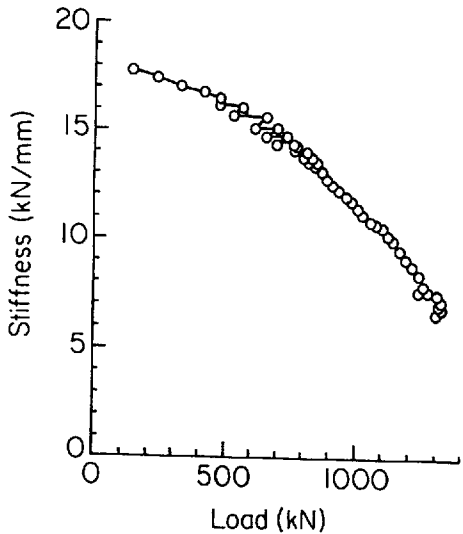


Fig. 10—Mid-span stiffness vs. load, ultimate load test

22 White

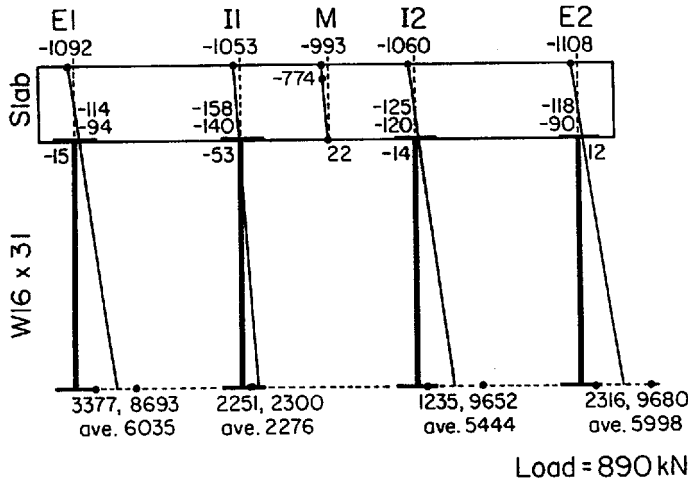


Fig. 11A—Mid-span strain distributions at loads near ultimate capacity; load = 890 kN

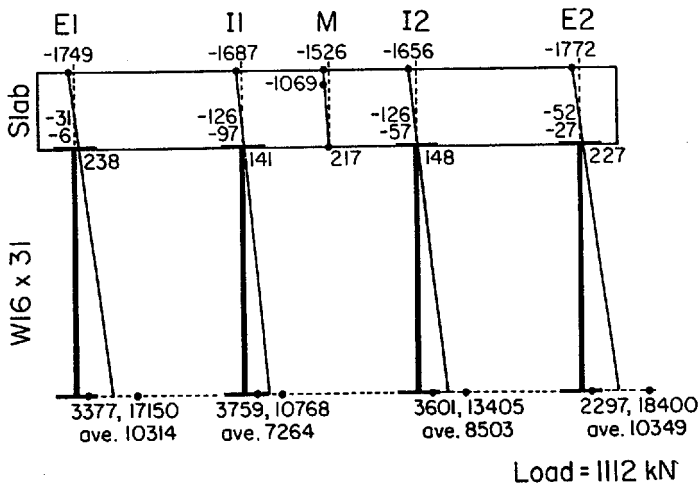


Fig. 11B—Mid-span strain distributions at loads near ultimate capacity; load = 1112 kN

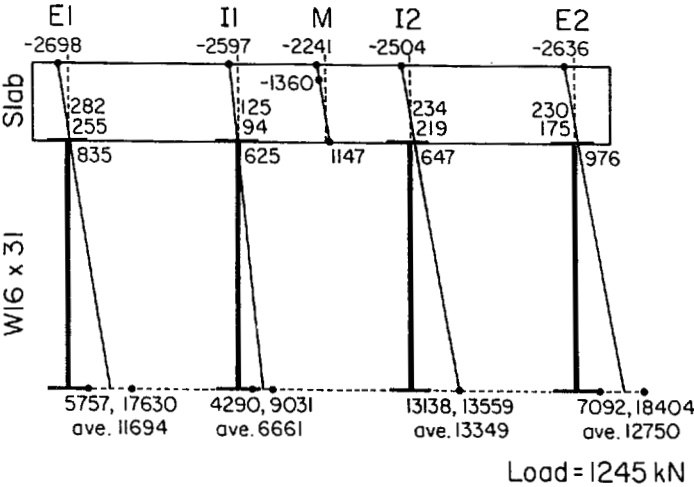


Fig. 11C—Mid-span strain distributions at loads near ultimate capacity; load = 1245 kN

24 White

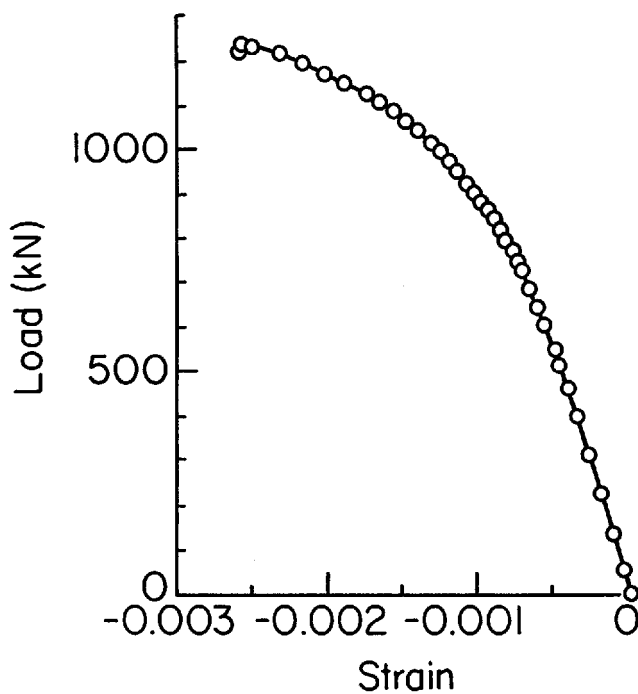


Fig. 12A—Mid-span top fiber concrete strains vs. load; above exterior beam E1

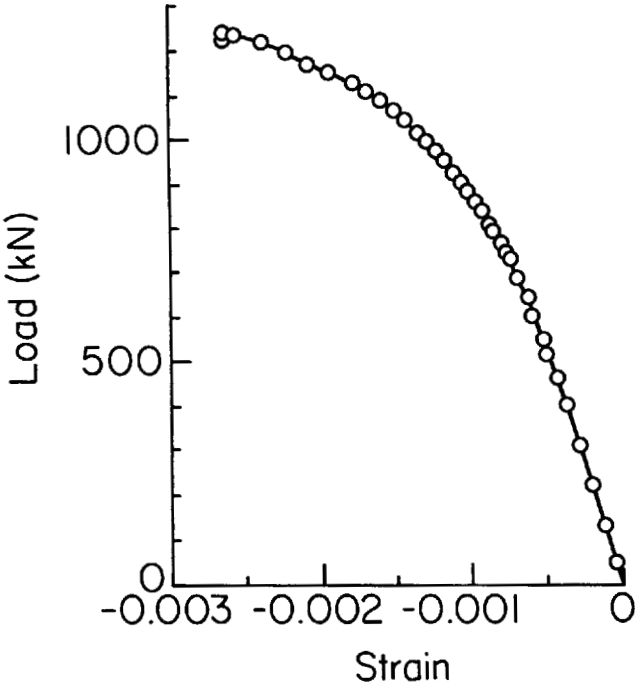


Fig. 12B—Mid-span top fiber concrete strains vs. load; above interior beam I1

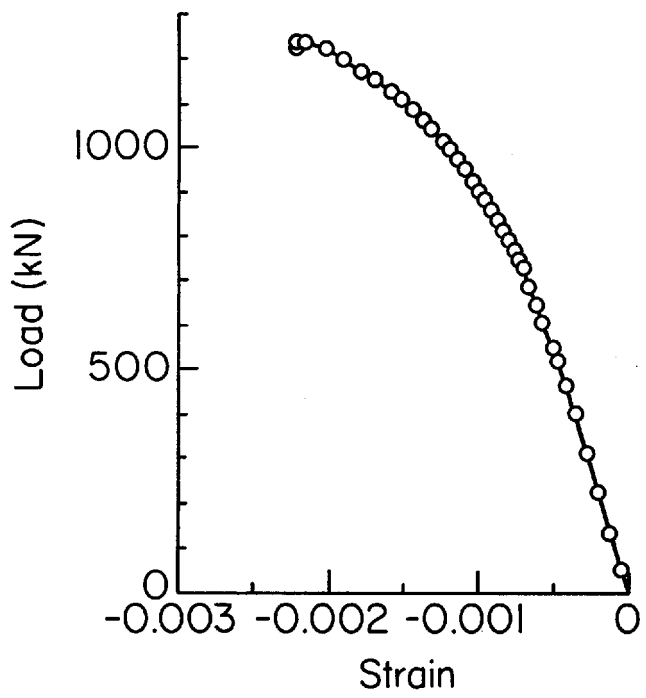


Fig. 12C—Mid-span top fiber concrete strains vs. load; center of deck

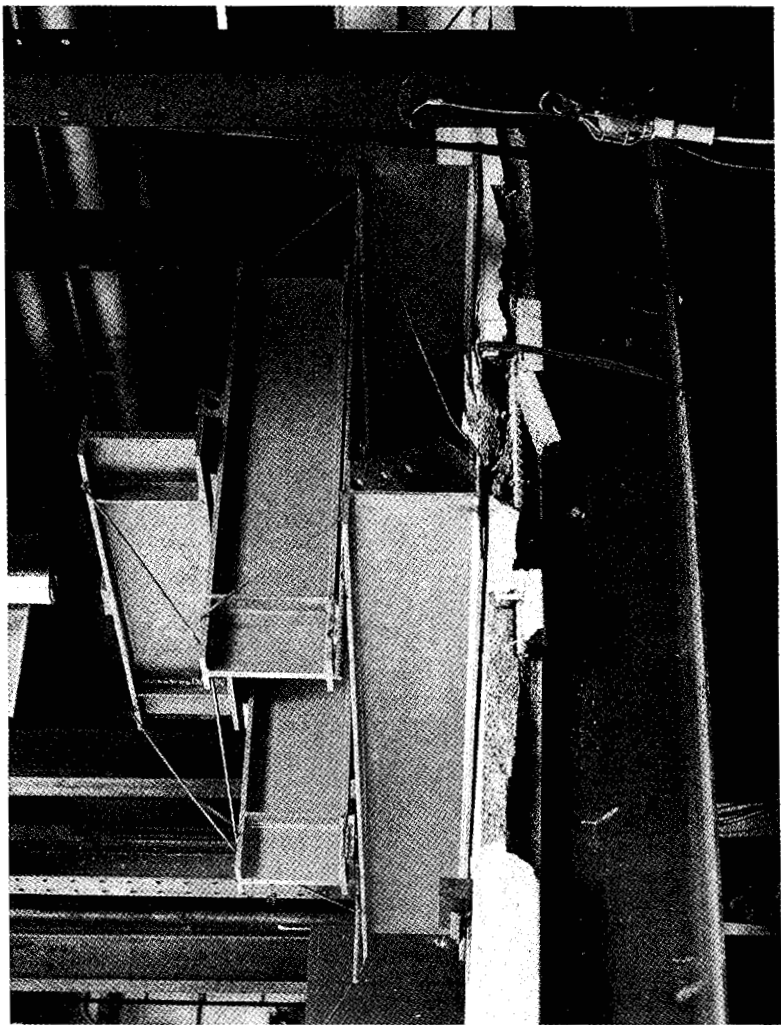


Fig. 13—Concrete failure zone at mid-span of bridge deck, ultimate load test

SP 172-2

The Resistance of Rice Husk Ash Concrete to Carbonation, Acid Attack and Chloride Ion Penetration

by Shuichi Sugita, Qijun Yu, Masami Shoya, Yoichi
Tsukinaga and Yasuo Isojima

Synopsis: It was found that highly reactive rice-husk ash (RHA) with more than 40 m²/g BET specific surface could be obtained by the two-step burning method. In this study, concrete specimens with different RHA blending ratios were prepared at different water-cement ratios, then cured in water, 2% HCl solution, and an accelerated carbonating chamber respectively. The results show that after the addition of RHA to the cement not only the strength of concrete is enhanced greatly but also the losses of mass and strength of the concrete cured in the HCl solution and the carbonation rate of concrete were decreased. It was also observed that the resistance of concrete to the penetration of water, air, and chloride-ion can be much improved by using RHA.

Keywords: Acid resistance; carbonation; gels; penetration tests; pore size distribution; strength

30 Sugita et al.

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Yu Qijun, Dr. of Engineering, an Associate Professor, Wuhan University of Technology, P. R. China. Now he works as a visiting scholar in Hachinohe Institute of Technology. His studies has been mainly on cement and concrete chemistry. Recently he also shows much interest in the reuse of industrial wastes and the durability of concrete.

Masami Shoya, ACI member, graduated from Hokkaido University, is a Professor at the Dept. of Civil Engineering, Hachinohe Institute of Technology. He has many papers on the durability and shrinkage of concrete. Now he is investigating the methods for assessing the deterioration of concrete.

Yoichi Tsukinaga is an Associate Professor at the Dept. of Architectural Engineering, Hachinohe Institute of Technology. He has been engaged in the research on the durability of concrete and non-destructive testing.

Yasuo Isojima is a technician. Since 1981 he has been doing research work with Prof. Sugita and Prof. Shoya.

INTRODUCTION

It is well known that some industrial by-products such as fly ash, blast-furnace slag and silica fume are the most common pozzolanic materials used in the cement and concrete industry. Recently investigations on the production of the RHA with high activity and the possible application of it in concrete have been made by many researchers (1, 2), all the results of which reveal that the use of RHA in concrete has a promising future. RHA, being similar to silica fume in SiO_2 content, contains 90-95% SiO_2 and 1-5% carbon residue, but the silica in it can be amorphous or crystalline in form according to the manner in which it is burned. When over-burning occurs, the amorphous SiO_2 in it will be changed to cristobalite, quartz, and tridymite (3); when it is burnt at low temperatures much unburnt carbon will exist in it. In the both cases the pozzolanic activity of RHA will be decreased. Although RHA of high quality has been prepared by many researchers in laboratories, now it seems to be very difficult to get active and homogeneous RHA in an industrial scale. How to produce active RHA in quantity and make it possible that RHA can be widely used in concrete as one kind of effective pozzolanic material is a practical problem. Based on the two-step burning method, the authors designed two different burning systems for

producing highly active RHA in quantity, i.e. the continuous system and the batch system as shown in Fig. 1 and Fig. 2. The two-step burning method means that by this method the whole process of burning RHA should be divided into two separated steps: one is to drive off the organic materials and water in the husk and carbonize the husk at a relatively low temperature under a reducing atmosphere, the other is to burn the carbonized husk to ash at a proper temperature in a given time. Experimental results showed the RHA produced either by the continuous system or the batch system was very active: with the substitution of it for normal portland cement in mortar or concrete, from 10 to 50 by mass percentage, the strength of mortar or concrete could be increased considerably (4).

Besides the well-known strengthening effect on concrete can RHA improve the durability of concrete? As the resistance of concrete to acid attack and carbonation plays a role in the durability of concrete and the resistance of concrete to penetration is also closely related to the durability of concrete, in the present study the following properties of RHA blended concrete were investigated: (1) compressive and flexural strength, (2) strength and mass variation of the concrete specimens cured in 2% HCl solution, (3) resistance to carbonation, and (4) resistance to water, air, and chloride-ion penetration. In all the experiments normal portland cement concrete was used as the control.

RAW MATERIALS

For lowering the production cost the RHA used in this study was burned in a batch furnace, then it was ground for 60 minutes in a ball mill with the addition of 0.025% grinding agent. Fig. 3 shows the silica in the RHA is mainly amorphous. Its chemical composition and physical properties are presented in Table 1. By the method proposed by Luxan et al. (5) it has been found that the variation in electrical conductivity of the saturated $\text{Ca}(\text{OH})_2$ solution added with the RHA is 3.16 mS/cm, which shows the RHA has a very good pozzolanic activity (4, 5). The cement and aggregates used in the experiments were: (1) Sumitomo normal portland cement, (2) river sand, which was used as fine aggregate, and (3) crushed stone with the nominal maximum size of 25 mm, used as coarse aggregate. The properties of the aggregates are listed in Table 2, the physical properties and chemical composition of the cement are shown in Table 3.

In previous work it was observed that the water requirement of RHA blended concrete was high. In order to control the air volume and slump of all the fresh concrete with different RHA addition to be $5 \pm 1\%$ and 12 ± 3 cm an air-entraining admixture (Vinsol resin) and NL-1450 solution which is a water reducer mainly consisting of one kind of highly polymerized aromatic sulposalt were used.

EXPERIMENTAL AND RESULTS

The Effect of RHA Addition on the Strength of Concrete

The mixture proportions of the concrete with or without RHA addition are presented in Table 4. According to JIS A 1132 concrete cylinders of 100×200 mm for compressive strength were prepared at: (1) RHA blending ratios of 0, 10, 20, 30, and 50 by mass percentage, (2) water-cement ratios of 65, 75, and 80% and then cured in the water at 20 ± 1 °C for 28 days. From Fig. 4 it can be found that: (1) with the addition of RHA the compressive strength of concrete is increased, (2) at the RHA blending ratio of 20% the concrete attains the maximum increment in strength, and (3) the larger the w/c the greater the increasing ratio of strength, for instance, in the case of the w/c being 80% and RHA blending ratio being 20% the strength of concrete is increased by about 68% compared with that of the control concrete.

The pore distribution and total cumulative pore volume of the tested concretes were determined by using a porosimeter (PASCAL 240). The results are shown in Table 5.

The Resistance of RHA Blended Concrete to Carbonation

Concrete cylinders of 100×200 mm were made at RHA blending ratios of 0, 10, and 30% and water-cement ratios of 75% and 80% (Table 4). After curing in water for 28 days the cylinders were dried at 60 ± 2 °C for 24 hr and then dried in the room of 20 ± 2 °C and R.H.<40% for one week. Subsequently 3 slices of 100×50 mm were taken out from the middle portion of each of the 100×200 mm cylinders. Finally the slices, with their top and bottom surfaces being tightly sealed with epoxy but with their outside surfaces being exposed (Fig.5), were cured for 2, 4, and 8 weeks respectively in an accelerated carbonating chamber in which the conditions of 30°C, 60% R.H., and 5% CO₂ were set.

After curing to a definite age three slices with the same mixture proportion were taken out from the chamber. Each was divided into halves along the direction of its diameter. At first the area of the carbonated portion was measured by a computer coordinating area-curveimeter (by spraying 1% phenolphthalein solution upon the freshly exposed surface to distinguish the carbonated portion from the uncarbonated one), then the carbonation depth of the slice was obtained by dividing the area by the width of the carbonated portion. The carbonation depth of concrete was determined by taking an average over the carbonation depth of the three slices (Fig. 5). From Fig. 6 it can be found that the carbonation depth of concrete is decreased with the increase in the amount of RHA. The more the added RHA the less the carbonation depth of concrete. But after curing for 8 weeks in the chamber the specimens, either with or without RHA addition, have been wholly carbonated.

The Resistance of RHA Blended Concrete to the Attack by 2% HCl Solution

Concrete specimens of $100 \times 100 \times 400$ mm and 100×200 mm, with w/c = 65%, 75%, and 80%, were used to investigate the resistance of RHA blended concrete to the attack by 2% HCl solution. After curing in the water at $20 \pm 1^\circ\text{C}$ for 4 weeks the specimens were dried in the room of $20 \pm 2^\circ\text{C}$ and R.H. < 40% for one week, then the surface of every specimen was partially sealed with paraffin wax, as shown in Fig. 7. Finally they were cured in water and in 2% HCl solution ($\text{pH} = 0.80 \pm 0.02$), respectively, for 4 and 13 weeks. The HCl solution was renewed every 3 days. The variation in compressive and flexural strength of the concrete cured in the HCl solution with RHA blending ratio is presented in Fig. 8, from which it can be found that both the compressive and the flexural strength of all the RHA blended concretes are higher than those of the control concrete. Fig. 9 shows that after curing in the HCl solution both RHA blended concrete and the control concrete have a loss of mass, and the loss of mass is increased with the curing time. But the mass loss of RHA blended concrete is much lower than that of the control concrete, at the same time it is decreased with the increment of the amount of added RHA. Fig. 10 are the photos of the tested concrete specimens cured in the solution for 4 weeks, from which it can be found the uncovered portion, where was in direct contact with the HCl solution, of the concrete specimens without RHA addition has been seriously damaged. On the contrary, the surface of the specimens prepared with RHA addition, where was also exposed to the HCl solution, remains nearly unchanged.

The Resistance of RHA Blended Concrete to Penetration

Concrete cylinders of 100×200 mm with 0, 10, and 30% RHA addition were prepared at water-cement ratios of 55%, 65%, and 75%. After curing in water for 28 days each of them was cut into halves with the size being about 100×100 mm, then they were dried in the room of $20 \pm 2^\circ\text{C}$ and 60% R.H. for one week. Afterwards the following experiments were conducted on them: (1) air permeability, (2) water permeability, and (3) chloride-ion penetration. In the last experiment the specimens of 100×100 mm were immersed into 13% NaCl solution at 20°C for one week, then they were removed to the same room to be dried for one week. After six immersion-drying cycles the average penetration depth of chloride-ion was determined with the area-curvimeter by spraying 0.1% fluorescein sodium solution and 2% AgNO_3 solution on the exposed surface, obtained by cutting the specimen into halves along the direction of its diameter, to display the chloride-ion penetrating region. Fig. 11-13 shows that in the investigated range of RHA addition the more the added RHA, the less the coefficient of air permeability and water permeability, and the less the chloride-ion penetration depth. Moreover at high water-cement ratios, for instance at a w/c of 65% or 75%, the coefficient of air permeability and the coefficient of water permeability of RHA blended concrete are much smaller than those of the concrete without RHA addition. Above results show that the resistance of concrete, especially the concrete with high water-cement ratios, to the penetration of air, water, and chloride-ion can be improved greatly by adding RHA to it.

The Product of the Reaction of RHA with Saturated $\text{Ca}(\text{OH})_2$ Solution

Portlandite often exists in hardened cement because the liquid phase of hydrating cement is saturated with Ca^{2+} and OH^- ions. But Sugita found that the amount of $\text{Ca}(\text{OH})_2$ in the hardened cement paste and concrete added with RHA was greatly reduced compared with the paste and concrete without RHA addition (4). As only 15-30% cement was substituted with RHA in the preparation of the paste and concrete it seems that this phenomenon can not be simply attributed to the reduction of the proportion of cement in the paste and concrete. In this study it has been observed that the SiO_2 in RHA can quickly react with the Ca^{2+} and OH^- ions in saturated $\text{Ca}(\text{OH})_2$ solution to form $\text{Ca}_{1.5}\text{SiO}_{3.5} \cdot x\text{H}_2\text{O}$ (PDF-33-0306, one kind of C-S-H gel, which can be also found in the paste with RHA addition). The XRD pattern of the product is shown in Fig. 14. This result indirectly shows: (1) the decrease of the amount of $\text{Ca}(\text{OH})_2$ in the hardened paste or concrete blended with RHA may be closely related to the quick reaction of RHA with the Ca^{2+} and OH^- ions released from the hydration of cement, (2) more C-S-H gel will be formed in the concrete blended with RHA, and (3) the hydration of the cement in RHA concrete will be improved as a portion of Ca^{2+} and OH^- ions in the hydrating cement are consumed by the reaction. The increment of the non-evaporable water in the hydrated paste blended with RHA gives evidence for this (Table 6).

DISCUSSION

Fig. 4 indicates that the 28-day compressive strength of the concrete with different water-cement ratios is all increased with the addition of RHA. The possible reasons for this are:

(1) Ground RHA is a porous material with its specific pore volume varying between 120 and 220 mm^3/g and specific surface (N_2 adsorption) being larger than 40 m^2/g . The pores in it are mainly distributed in the region of 2-5 nm with an average radius of 4 nm. With a so large number of mesopores in it RHA has a characteristics of high water absorption (6), therefore the practical water-cement ratio in RHA blended concrete is less than the used because a portion of free water has been adsorbed in the mesopores in RHA.

(2) The SiO_2 in RHA is very active, it can quickly react with the Ca^{2+} and OH^- ions in the liquid phase of hydrating cement. Therefore, compared with normal concrete, there will be more C-S-H gel and less portlandite in RHA blended concrete. On the other hand, the hydration of cement will be promoted by the added RHA. Both of them contribute to an increase in the strength of concrete.

(3) With the addition of RHA to concrete the average pore radius of concrete is decreased, especially the portion of the pores greater than 20 nm in radius is evidently decreased instead of the increase in the amount of small pores (Table 5 and Fig. 15), which shows the structure of concrete becomes compact.

With higher strength, lower content of portlandite, more C-S-H gel, and smaller pore size, the RHA blended concrete is surely superior to normal concrete in the resistance to acid attack, carbonation, and penetration of chloride-ion.

CONCLUSIONS

1. By using the two-step burning method proposed by the authors, RHA, with nitrogen-adsorption specific surface greater than $40 \text{ m}^2/\text{g}$ and good pozzolanicity, can be obtained either with a rotary kiln or with a batch furnace.
2. With the addition of the RHA burned in a batch furnace to concrete, the strength of concrete can be increased. The larger the w/c, the greater the increasing ratio of the strength. The resistance of concrete to the attack by 2% HCl solution, carbonation, and penetration (chloride-ion, water, and air) can be also improved by adding RHA to it.
3. The SiO_2 contained in the RHA is mainly in amorphous form and is very active, it can rapidly react with Ca^{2+} and OH^- ions to form C-S-H gel.
4. The effect of RHA on the increase in strength and the improvement on the resistance of concrete to acid attack, carbonation, and penetration is mainly attributed to: the practical water-cement ratio of concrete is decreased due to the addition of RHA; more C-S-H gel and less portlandite are formed in concrete; and the average pore size of concrete is reduced.

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TABLE 1—CHEMICAL COMPOSITION (%) AND PHYSICAL PROPERTIES OF THE RHA USED IN THE EXPERIMENT

Loss on Ignition	2.93	CaO	0.38	TiO ₂	0.01	Total	99.85
SiO ₂	91.90	MgO	0.21	P ₂ O ₅	0.36	Spec. Gravity	2.20
Al ₂ O ₃	0.25	Na ₂ O	0.05	MnO	0.16	N ₂ Adsorption (m ² /g)	46.86
Fe ₂ O ₃	0.41	K ₂ O	2.78	C	0.41		

TABLE 2—PROPERTIES OF THE FINE AND COARSE AGGREGATE

Material	Specific Gravity	F. M.	Water Absorption (%)	Percentage of Solid Volume
River Sand	2.66	2.46	1.26	66.2
Crushed Stone	2.72	6.99	0.28	57.9

TABLE 3—PHYSICAL PROPERTIES AND CHEMICAL COMPOSITION* OF THE USED CEMENT

Specific Gravity	Blaine Surface (m ² /kg)	Normal Consistency	Setting Time (h : min)		Comp. / Flex. Strength** (MPa)		
			Initial	Final	3d	7d	28d
3.16	330	27.6 %	2 : 08	4 : 30	15.8/3.4	27.0/5.2	41.3/7.2

* SiO₂ = 21.29%, Al₂O₃ = 5.60%, Fe₂O₃ = 3.00%, CaO = 62.96%, MgO = 2.24%, SO₃ = 1.47%, K₂O = 0.31%, Na₂O = 0.30%

** 40 × 40 × 160 mm mortar specimens made at a water-cement ratio of 0.65 and sand-cement ratio of 2.0

TABLE 4—MIXTURE PROPORTIONS OF RHA BLENDED CONCRETE

w/c (%)	RHA Addition (%)	s/a (%)	Water (kg/m ³)	RHA (kg/m ³)	Cement (kg/m ³)	Sand (kg/m ³)	Stone (kg/m ³)
80	0	52	186.5	0.0	233.1	951.1	897.2
	10			23.3	209.8	946.4	892.8
	20			46.6	186.5	941.8	888.5
	30			69.9	163.2	937.4	884.3
	50			116.6	116.6	927.9	875.3
75	0	50	184.5	0.0	245.9	912.1	925.4
	10			24.6	221.3	907.1	920.2
	20			49.2	196.7	902.4	915.4
	30			73.8	172.2	897.9	910.8
	50			123.0	123.0	888.3	901.1
65	0	46	184.5	0.0	283.8	827.3	979.2
	10			28.4	255.4	821.9	958.8
	30			85.2	198.6	811.4	947.0
55	0	44	184.5	0.0	335.4	771.6	997.3
	10			33.5	302.0	767.2	989.9
	30			100.6	234.7	755.9	975.2

TABLE 5—PORE DISTRIBUTION OF THE TESTED CONCRETE

w/c (%)	RHA Addition (%)	Ave. Pore Radius (nm)		Total Pore Volume (mm ³ /g)	
		28d	91d	28d	91d
65	0	35.2	40.0	82.9	77.5
	10	28.2	23.1	91.6	86.9
	30	19.2	14.4	99.5	99.5
75	0	29.3	47.7	92.3	74.9
	10	17.9	24.9	81.8	94.0
	30	17.5	16.9	97.1	104.1
80	0	32.9	—	87.2	—
	10	41.2	—	95.4	—
	30	26.4	22.9	82.1	82.3

TABLE 6—NON-EVAPORABLE WATER (%) IN PASTES (W/C + 0.45)

RHA Addition (%)	1d	3d	7d	28d	91d
0	8.05	11.31	14.47	16.65	17.33
10	8.64	11.39	14.74	16.92	18.10

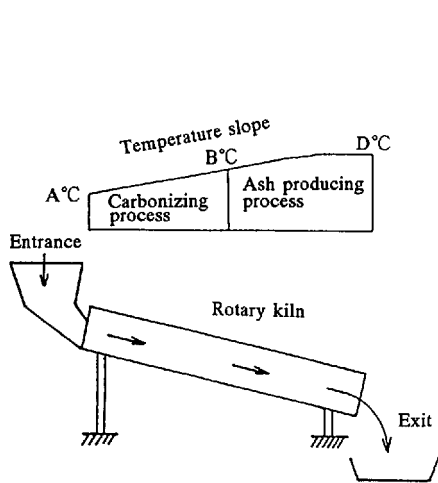


Fig. 1—(left) Continuous RHA production system with a rotary kiln

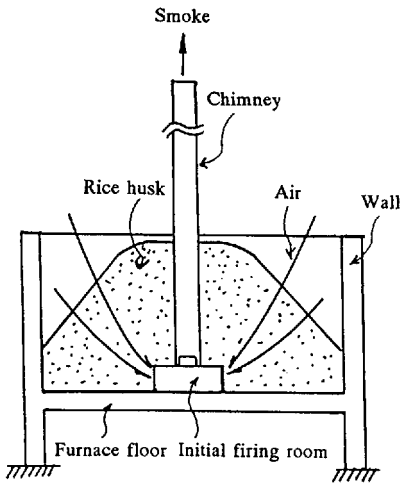


Fig. 2—(right) Schematic diagram of the batch furnace (batch system)

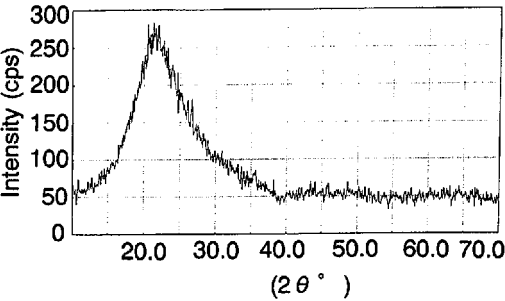


Fig. 3—XRD pattern of RHA

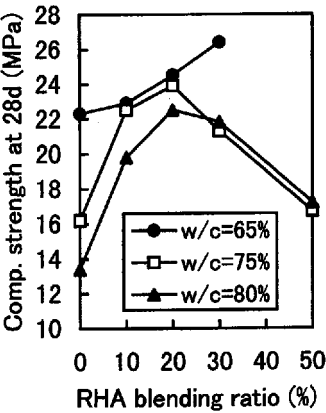


Fig. 4—Compressive strength of the concrete cured in water for 28 days vs. RHA blending ratio

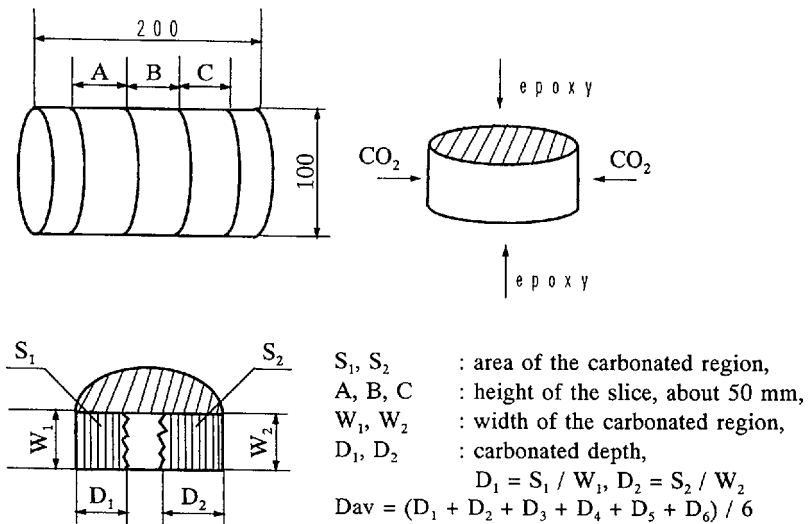


Fig. 5—The method of determining the carbonation depth of RHA blended concrete

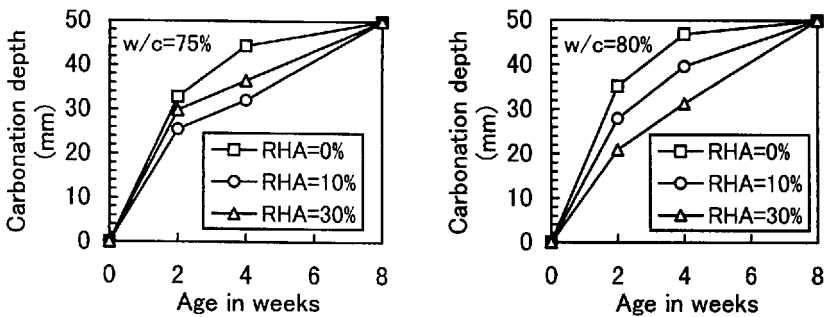
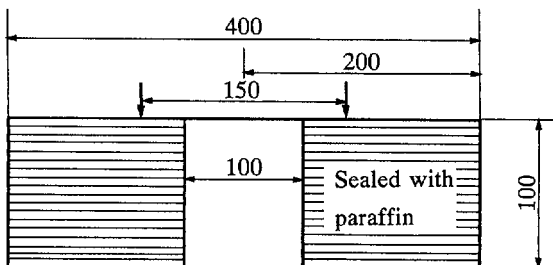
Fig. 6A—(left) The effect of RHA addition on the carbonation of the concrete at a w/c of 75% Fig. 6B—(right) The effect of RHA addition on the carbonation of the concrete at a w/c of 80%

Fig. 7A—The method of sealing the concrete specimen of 100 x 100 x 400 mm with paraffin wax

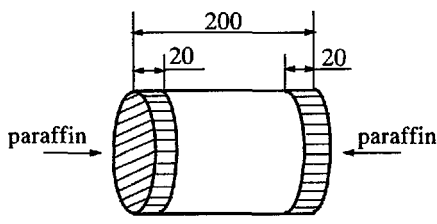


Fig. 7B— The method of sealing the concrete specimen of 100 x 200 mm with paraffin wax

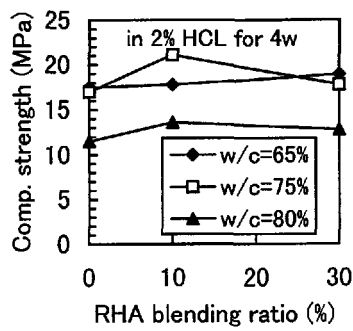


Fig. 8A—(left) The compressive strength of the concrete cured in 2% Hcl solution for 4 weeks vs. RHA blending ratio

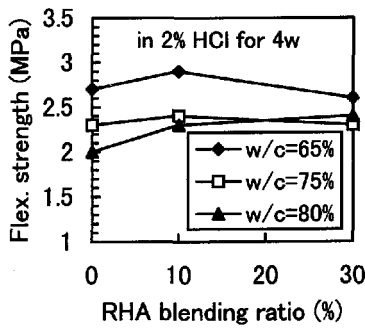


Fig. 8B— (right) The flexural strength of the concrete cured in 2% Hcl solution for 4 weeks vs. RHA blending ratio

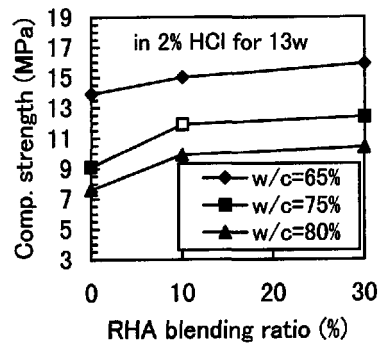


Fig. 8C—(left) The compressive strength of the concrete cured in 2% Hcl solution for 13 weeks vs. RHA blending ratio

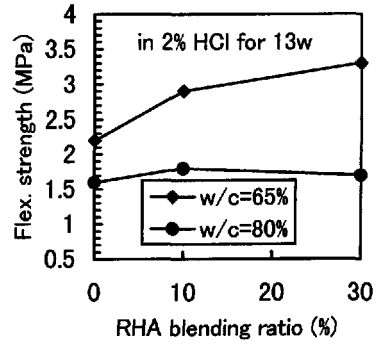


Fig. 8D— (right) The flexural strength of the concrete cured in 2% Hcl solution for 13 weeks vs. RHA blending ratio

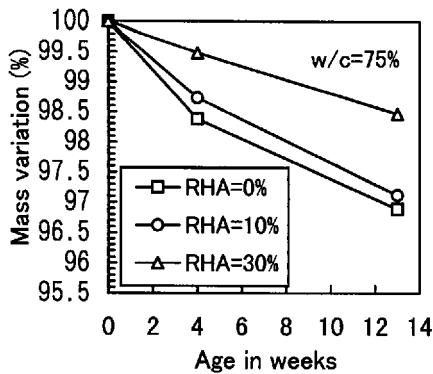
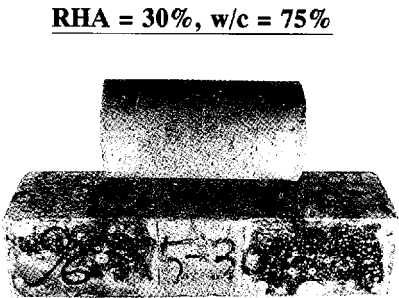
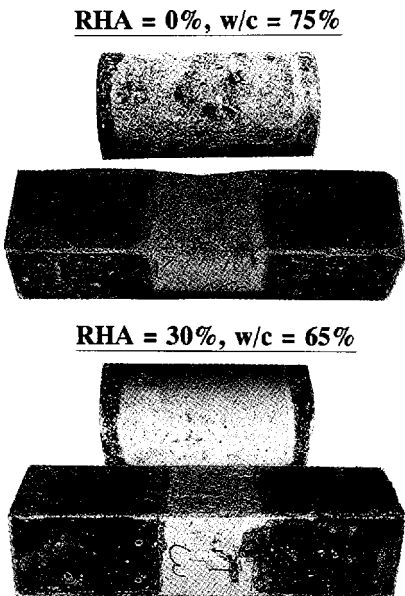


Fig. 9—Mass variation of the concrete cured in 2% Hcl solution with curing time



RHA = 0%, w/c = 75%

RHA = 30%, w/c = 75%

Fig. 10—Photos of the concrete specimens with or without RHA addition after cured in 2% Hcl solution for 4 weeks

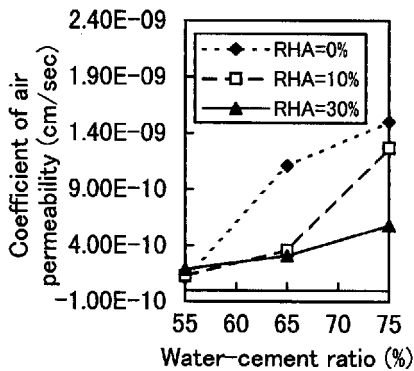


Fig. 11—Coefficient of air permeability of RHA blended concrete vs. water-cement ratio

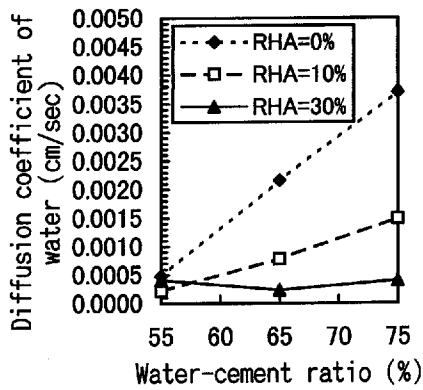


Fig. 12—Diffusion coefficient of water of Rha blended concrete vs. Water-cement ratio

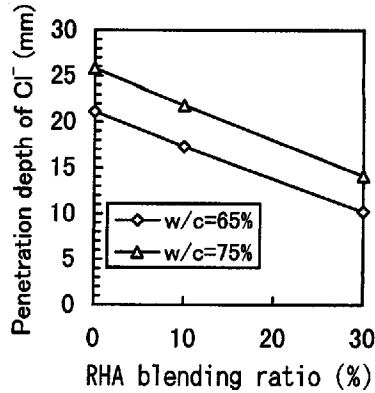


Fig. 13—The effect of RHA addition on the penetration of chloride-ion in concrete

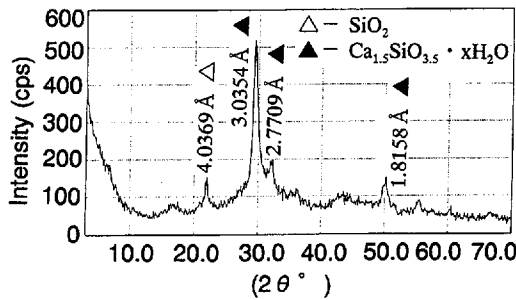


Fig. 14—XRD pattern of the product obtained by the reaction of RHA with saturated Ca(OH)₂ solution

