

Strengthening Concrete in Seismic Regions

Catherine W. French
Michael E. Kreger



AVA Documents
Document Sharing Hub

High-Strength Concrete (HSC) in Seismic Regions

Editors

Catherine W. French

Michael E. Kreger



international

SP-176

DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is June 1, 1999. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the September/October 1999 issue of either ACI Structural Journal or ACI Materials Journal, depending on the subject emphasis of the individual paper.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 1998
AMERICAN CONCRETE INSTITUTE
P.O. Box 9094
Farmington Hills, Michigan 48333-9094

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Jane Carroll

Library of Congress catalog card number: 98-85209

PREFACE

In May of 1993, approximately twenty researchers and five representatives from construction firms met in Kyoto, Japan, for the *First Multilateral Meeting on Structural Performance of High-Strength Concrete in Seismic Regions*. Four countries (United States, Japan, New Zealand and Australia) were represented at the meeting. The 3-day meeting was divided into eight sessions covering current research programs and applications of high-strength concrete in the respective countries. The objectives of the meeting were to exchange information and to develop a coordinated program for further information exchange, evaluation of information, and development of design guidelines for the use of high-strength concrete in seismic regions.

The follow-up meeting was held in November 1994 in Honolulu, Hawaii, and was attended by twenty-seven participants from the U.S., Japan, Canada, New Zealand, and Hong Kong. The *Second Multilateral Meeting on Structural Performance of High-Strength Concrete in Seismic Regions* consisted of thirteen sessions. Six of the sessions concentrated on the following behavioral topics: bond and anchorage, confinement, flexural members, axially-loaded members (columns and walls), beam-column joints, and shear and torsion. An additional session was devoted to presentation and discussion of design concepts and applications of high-strength concrete (HSC) in seismic regions. The remaining six sessions consisted of large and small working group sessions. During the small group sessions, participants were divided into groups of five to ten members to discuss the results of the previous sessions. Summaries of the small working group were then presented to the entire group for additional comment and conclusion during the large working group sessions.

This ACI Special Publication comprises selected papers which were the outcome of the *Second Multilateral Meeting on Structural Performance of High-Strength Concrete in Seismic Regions*. The working group discussion summaries are also included in this special publication. The editors are appreciative of the efforts of the authors and reviewers of these papers. The cooperation of the authors in the careful revision of their papers in accordance with reviewers' comments is greatly appreciated.

Particular appreciation is expressed for the efforts of the non-native English-speaking participants in developing their manuscripts.

Editors

Catherine W. French
Professor of Civil Engineering
University of Minnesota

Michael E. Kreger
Professor of Civil Engineering
University of Texas at Austin

CONTENTS

BOND AND DEVELOPMENT OF STEEL REINFORCEMENT IN HIGH-STRENGTH CONCRETE—AN OVERVIEW by S. L. McCabe.....	1
BOND AND ANCHORAGE OF REINFORCEMENT IN HIGH-STRENGTH CONCRETE by S. Fujii, H. Noguchi, and S. Morita.....	23
BOND AND ANCHORAGE OF REINFORCEMENT IN CONCRETE—NEW ZEALAND CODE PROVISIONS by D. Bull.....	45
CONFINEMENT OF HIGH-STRENGTH CONCRETE FOR SEISMIC PERFORMANCE by S. K. Ghosh.....	57
CONFINED CONCRETE WITH HIGH-STRENGTH MATERIALS by D. Kato, F. Watanabe, M. Nishiyama, and H. Sato.....	85
CONFINEMENT OF HIGH-STRENGTH CONCRETE by M. Saatcioglu, P. Paultre and S. K. Ghosh.....	105
EFFECT OF HIGH-STRENGTH CONCRETE (HSC) ON FLEXURAL MEMBERS by C. J. Fasching and C. W. French.....	137
RELIABILITY OF LOAD-DEFORMATION ESTIMATE FOR HIGH-STRENGTH REINFORCED CONCRETE BEAMS by S. Otani and S. Nagai.....	179
EFFECTS OF INCREASING CONCRETE STRENGTH ON THE DIMENSION OF BEAMS by H. J. Pam, H. Tanaka, and R. Park.....	197

PERFORMANCE OF HIGH-STRENGTH CONCRETE (HSC) COLUMNS CONFINED WITH RECTILINEAR REINFORCEMENT by A. Azizinamini and M. Saatcioglu.....	213
FLEXURAL STRENGTH AND DUCTILITY OF HIGH-STRENGTH CONCRETE COLUMNS by R. Park, H. Tanaka, and B. Li.....	237
BEHAVIOR AND DESIGN OF HIGH-STRENGTH RC WALLS by J. W. Wallace.....	259
TESTS AND ANALYSES OF HIGH-STRENGTH REINFORCED CONCRETE SHEAR WALLS IN JAPAN by T. Kabeyasawa and H. Hiraishi.....	281
EVALUATION OF U.S. SHEAR STRENGTH PROVISIONS FOR DESIGN OF BEAM-COLUMN CONNECTIONS CONSTRUCTED WITH HIGH-STRENGTH CONCRETE by E. I. Sagan and M. E. Kreger.....	311
SHEAR STRENGTH OF BEAM-COLUMN JOINTS WITH HIGH-STRENGTH MATERIALS by H. Noguchi, S. Fujii and M. Teraoka.....	329
HIGH-STRENGTH CONCRETE BEAM-COLUMN JOINTS OF MOMENT RESISTING FRAMES by R. Park, H. Tanaka and X. Z. Xin.....	357
SHEAR STRENGTH OF RC MEMBERS WITH HIGH-STRENGTH CONCRETE by F. Watanabe and T. Kabeyasawa.....	379
APPLICATION OF HIGH-STRENGTH CONCRETE IN SEISMIC REGIONS by L. A. Wyllie, Jr.....	397
NEW RC DESIGN GUIDELINES FOR HIGH-RISE REINFORCED CONCRETE BUILDINGS USING HIGH-STRENGTH MATERIALS by S. Otani, M. Teshigawara, M. Murakami, and T. Okada.....	405

DESIGN CONCEPTS AND APPLICATIONS OF HIGH-STRENGTH CONCRETE IN NEW ZEALAND by D. Bull.....	423
DESIGN APPLICATIONS OF HIGH-STRENGTH CONCRETE IN SEISMIC REGIONS by P. Mendis.....	437
BOND AND ANCHORAGE OF REINFORCEMENT IN HSC: DISCUSSION SUMMARY.....	445
CONFINEMENT: DISCUSSION SUMMARY.....	449
FLEXURAL MEMBERS (BEAMS): DISCUSSION SUMMARY.....	451
AXIALLY LOADED MEMBERS (COLUMNS): DISCUSSION SUMMARY.....	453
AXIALLY LOADED MEMBERS (WALLS): DISCUSSION SUMMARY.....	455
BEAM-COLUMN JOINTS: DISCUSSION SUMMARY.....	459
SHEAR AND TORSION: DISCUSSION SUMMARY.....	461
DESIGN CONCEPTS/APPLICATIONS: DISCUSSION SUMMARY.....	465

SP-176-1

Bond and Development of Steel Reinforcement in High-Strength Concrete—An Overview

by S.L. McCabe

Synopsis: Considerations regarding bond and development of reinforcement in high strength concrete (HSC) are presented from a North American perspective. The information contained in this paper is a compilation of information from various sources and represents a survey of the basis for North American approaches to bond of normal and high strength concrete under monotonic and cyclic loading. The paper was presented in part at the Second US-Japan-New Zealand-Canada Multilateral Meeting on the performance of HSC held in Honolulu November 29-December 1, 1994.

Keywords: Bonding; cyclic loading; high-strength concrete; reinforced concrete

Steven L. McCabe is Associate Professor of Civil Engineering at the University of Kansas. His research interests include earthquake engineering and structural dynamics as well as finite element methods and damage mechanics. He is an active member of ASCE, CEB and ACI among others. Currently, McCabe is the chair of ACI Committee 439, Steel Reinforcement, and is a member of the ACI Building Code subcommittee 318-B, Development and Reinforcement.

INTRODUCTION

The development of increased concrete compressive strengths has been a major factor in the recent success of reinforced concrete in the marketplace. The increase in strengths has played a major role in decreasing member sizes, and hence dead weight, while providing economic advantages for designers, contractors and owners.

As with any technological step forward, the effects of increasing strength on structural concretes continues to require study. The overall system performance is not just a question of increased compressive strength, but rather a complex interaction of the properties of these new concretes and the demands placed on them. These high strength concretes (HSC) can be roughly defined as those with strengths of 56 MPa (8000 psi) and above. At the present time, mix designs with strengths of 105 MPa and above are routinely available in the marketplace, while in the not too distant past, a 56 MPa concrete mix was considered on the edge of technology. The important thing to observe is that much of the available data and knowledge, as well as present building code provisions, are based on testing and experience gained with lower strength concretes. Therefore, these higher strength concretes represent essentially a new class of material and should be treated as such in design. Thus, it may not be a simple case of scaling the existing design provisions by the cylinder (or cube) strength as the compressive strength increases. There may be significant changes in the behavior itself requiring that the existing rules may need to be modified or replaced.

The bond and development of reinforcing steel in high strength concrete represents a case in point. The existing bond data has been largely based on tests and observation of behavior for concretes at or around 30 MPa in strength. As the cylinder strength increases, there are significant changes in the behavior of the concrete and its failure mechanisms. Higher strength concretes tend to exhibit smoother failure surfaces and more brittle behavior than lower "normal" strength concretes (NSC). The failure surfaces are smoother since the cement paste can be stronger than the aggregates and so failure surfaces propagate through the aggregates instead of around them as is the case in NSC. These changes are significant and make existing bond relationships, based in many cases on moderate concrete strength, less accurate than intended.

The questions become even more profound when one considers that high strength concrete is typically used on columns and other primary load carrying elements whose reliability must be assured for structural survival. Moreover, many newer applications involve the use of HSC in structural systems that will be subjected to significant cyclic loading demands from earthquake. The effects of load reversals on bond and development of reinforcement of normal concretes are reasonably well understood from a qualitative standpoint but design rules are still the subject of research. The effects of load reversals on HSC are even more of a

question since the response to loading is different, as are the failure mechanisms.

This paper will summarize what is known on the subject of bond and development of reinforcing steels placed in HSC. The view provided here is that of the author as a member of the North American research and design community, and will approach this problem from that perspective. What will follow is a summary of the load carrying mechanisms and the relationships and behaviors that have been established to date for HSC under monotonic and cyclic loading environments. The paper is organized so that information on behavior of NSC and HSC concretes under monotonic loading is presented, followed by specific information on cyclic loading effects of the type encountered in earthquake.

Behavior under Monotonic Loading

Background--There are three primary mechanisms in bond: adhesion of the bar to the concrete as well as resistance of the bar to movement through the surrounding concrete by friction and by interlock of the bar deformations in the concrete. Once some level of load has induced loss of adhesion and the frictional resistance has been overcome, the primary mechanism left is interlock. The concrete wedges against the reinforcing bar deformations and thus restrains the bar from moving relative to the surrounding concrete. The properties of the bar and its surface as well as the concrete environment all impact the effectiveness of this restraint.

Earlier versions of the ACI Building Code utilized a bond stress approach whereby the force developed in the bar divided by the surface area of the bar, a bond stress, was limited to a specific value. The assumption here was that the bond stress was *uniform* over the entire length of the development region.

$$u = A_b f_s / \pi d L_s = \pi d_b^2 f_s / 4 \pi d_b L_s = d_b f_s / 4 L_s$$

Given this simple starting point, the role of research has been to identify the parameters that control the bond effectiveness so as to determine design relationships. To that end there have been many studies conducted over the years to establish the behavior of reinforcing steels under monotonic loading. These studies have largely concerned normal to moderately high strength concretes. The design rules that have been developed are largely empirical in nature and are "skewed" towards the moderate end of the strength scale.

These earlier studies include those by Abrams (1) and a large study by Clark (15) that was a fundamental study of all bars produced in the United States. The result of this work was the elimination of deformation patterns that were not able to produce acceptable bond performance. Other important studies are those by Tepfers (45, 46), Orangun, Jirsa and Breen (40, 41), Ferguson and Breen (27), Ferguson and Thompson (29), Mathey and Watstein (38), Chamberlin (11, 12), and Chinn et al. (13).

Design Rules--The end result of these studies was the development of the design rules to predict bond performance. In particular, the Orangun, Jirsa and Breen study (40, 41) resulted in an important relationship to empirically depict the bond stress of bars under development:

$$u = [1.2 + 3C/d_b + 50 d_b / L_s + A_{tr} f_{yt} / 500 s d_b] \sqrt{f'_c} \quad [\text{Customary Units}]$$

$$u = [0.10 + 0.25C/d_b + 4.15 d_b / L_s + A_{tr} f_{yt} / 41.52 s d_b] \sqrt{f'_c} \quad [\text{SI Units}]$$

In this equation it can be seen that there are several discrete elements. The

4 McCabe

term $3C/d_b$ relates the effects of the confining effects of cover and the negative effects of close bar spacing on bond. The f'_c term is taken as the square root to indicate that the bond strength is a function of tensile strength and is not linear with cylinder strength. The bond strength is linear with bar diameter. The effects of confinement are contained in a separate term that is linear with confining steel area. There also is a term that is a constant and reflects the fact that a bar with no cover would still exhibit limited bond strength.

ACI Committee 408 (4) also has taken data from a variety of sources and has developed their own database of bond test data and produced an expression for bond and development of Grade 420 [60] reinforcement:

$$L_{db} = 5500 A_b / [\phi K^* \sqrt{f'_c}] \quad [\text{US Customary Units}]$$

where K^* is the smaller of

- (a) $0.5d_b + C_c + K_{tr}$ or
- (b) $0.5d_b + C_c + [\sum K_{tr}/n]$; but no larger than $3d_{db}$

From these equations the familiar design expression found in the ACI Building Code (2) through the 1992 edition can be developed:

$$L_d = 0.04 A_b f_y / \sqrt{f'_c} \quad [\text{US Customary Units}]$$

$$L_d = 0.02 A_b f_y / \sqrt{f'_c} \quad [\text{SI Units}]$$

Here it can be seen that these design expressions represent a portion of the Orangun equation that has been simplified for design use. Also it can be seen that the K factor of the 408 equation has been replaced by a constant. For comparison purposes, given a standard bar configuration of a No. 8 Grade 420 [60] bar in 28 MPa [4000 psi] concrete, this equation predicts about a 30 bar diameter development length.

The important concept to note here is that the equations are empirical in nature and so are based on "best fit" equations to the data. As can be seen in Fig. 1, the data employed by Orangun et al. exhibited scatter that is customary in studies of bond because of the many variables that can affect the bond capacity of these systems. Thus, the equations predicted from the data are not exact for any particular data point, or design case, but rather encompass the range of behavior observed. This equation, and the derivative expressions from it, represented a significant step forward in the study of bond in the United States. The work established a philosophy of viewing bond as being comprised of three separate mechanisms that is still in use today. Lastly, it is important to note in the present context that this study contained only 62 specimens and that the cylinder strengths of the data all were below 42 MPa.

Studies by Darwin et al. (21) confirm that the bond capacities can be described in terms of parameters that include concrete strength, bar size, side and edge cover, bar spacing and the degree of confining steel. As shown in Fig. 2, the differences in the bond behavior when cover or confinement steel is present can be significant. A study of test data with no confining steel revealed an equation of

$$L_{db} = 0.06 A_b f_y / \sqrt{f'_c} \quad [\text{US Customary Units}]$$

which provides longer anchorage lengths than the ACI 318 expression above, providing a 45 bar diameter development length for the "standard bar configura-

tion," a value comparable with those obtained under present ACI 318-95 rules for development length.

De Vries et al. (39) reviewed the existing data and the ACI 318 and 408 expressions to simplify the procedures for designers. A rewritten form of the 408 equation was obtained as

$$L_{db} = 0.12 A_b f_y / \sqrt{f'_c} \quad [\text{US Customary Units}]$$

$$L_{db} = 1.4 A_b f_y / \sqrt{f'_c} \quad [\text{SI Units}]$$

In another important study Sozen and Moehle (44) evaluated data from some 223 tests from around the world. Some data was at 70 MPa and above, although most was from NSC tests. Typical results are presented in Figs. 3a and 3b show the scatter of the data and the role of compressive strength and bar size in the bond strength. Another significant aspect of these studies by Moehle et al. was the reorientation of development length to calculations to be in terms of numbers of bar diameters. This change allowed designers to return to the form used in earlier versions of the ACI Building Code and to make the design process more efficient.

Moreover, the work by Moehle and Sozen, together with research by others during the 1980's and early 1990's, provided a basis for a discussion of bond and development and the general question of how to depict bond in an accurate and straightforward manner. This work was a basis for the revised bond equation found in the 1995 ACI 318 Building Code (3):

$$L_d/d_b = [3d_b f_y / 40] \frac{\alpha \beta \gamma \lambda}{[(c + K_{tr})/d_b]} \quad [\text{US Customary Units}]$$

where α is the top bar factor and is either 1.0 or 1.3; β is the epoxy coating factor; γ is the size factor and λ is the lightweight aggregate factor; c is the cover or spacing and K_{tr} is the sum of transverse steel acting to confine the bars being developed.

The resulting equation also has the characteristic square root function of concrete strength. Significantly, the development length equation now is linear with bar diameter providing a simpler format for application by designers in practice while giving longer anchorage lengths than predicted in earlier Building Code editions. For the "standard bar configuration", the new equation provides a 40 bar diameter prediction for development length.

Another important observation is that the ACI 318 limits the concrete strength to 10 000 psi [69 MPa] in calculating the development lengths. This limit is recognition that the data is limited in the HSC regime and that the behavior is different.

Factors Affecting Bond Capacity of NSC-- It has been shown over the years that there are a number of factors that affect bond capacity of normal strength concrete and steel. These factors, represented in the expressions discussed earlier include:

Concrete Material Properties. Concrete slump, consolidation, aggregate strength and size, and the compressive strength of the concrete all play a role in bond. As a general rule the compressive strength decreases the required bond length by approximately the square root of the compressive strength for NSC.

The square root function implies that the tensile strength of the concrete may control the bond strength by delaying or preventing splitting cracks in the cover of the member that would - if large enough - allow the bar to slip relative to the member. There is evidence that the fracture energy, G_f , defined as area under the stress-crack opening curve, may in fact be a better parameter to use to determine the effectiveness of the concrete in anchoring a bar and preventing slippage (4). The larger the fracture energy, the more ductile the concrete is and the more the concrete is able to redistribute load from one region of the bar to another as cracking occurs. Bond data where fracture energy of the concrete is specifically measured is not a normally reported parameter that makes evaluation of the historical data difficult to do.

Reinforcing Bar Characteristics. The reinforcing bar itself can affect bond. In particular the rib height, rib spacing and the face angle and the surface condition all can play a part in altering the bond capacity. The general nature of these effects has been known for quite some time. Studies by Darwin et al. (14, 22, 23) concerning development of a high relative rib area bar with increased bond performance have shown that the geometry of the rib in terms of height, profile, spacing and inclination all play a role in anchoring the bar. Increased deformation heights and reduced spacings act to increase the relative rib area, the ratio of deformation bearing area to shearing area, from current values of 0.07 to values above 0.10. The effects appear, however, only when the bar is confined either by two or more bar diameters of amounts of cover or by transverse steel or both. Tests conducted where low covers are present with no transverse steel result in splitting failures at loads similar to those obtained with standard bars. The effects of increased rib height on the bar can lead to increased peak load values indicating that the bond process is more efficient and higher loads can be achieved, thus decreasing the required amount of development length for design, as shown in Fig. 4. Without confinement the increased bond performance of these high relative rib area bars is less pronounced (22, 36).

The presence of epoxy or other surface coatings on the steel can have a profound effect on the bond capacity. Studies by Treece and Jirsa (48) and Johnston and Zia (35) resulted in the present set of adjustment factors found in the ACI Building Code where the development length is increased by a factor of as much as 1.5. The epoxy coating essentially makes the bar surface "slick" reducing friction and adhesion leaving only mechanical interlock to anchor the bar. Work by Cairns (10), Choi et al. (14), Darwin et al. (20), and Cleary and Ramirez (16) confirmed that the bond capacity of epoxy coated steel is reduced by as much as 50% of that of an uncoated bar of the same size for bars that have low levels of confinement.

Confinement. A significant enhancement of bond performance is produced by confinement. Whether by additional cover and wide bar spacing, or by additional confining steel, the confinement can play a large role in placing the concrete around the bar(s) in more of a three dimensional state of compression. This action increases the capacity over that found in a unconfined configuration (22, 23). As was shown in Figs. 2 and 4, the bond capacity of beam end test specimens is significantly increased with the presence of confinement. While the test result may be a higher load at failure, perhaps a more significant effect is that the failure is ductile and that the load can be maintained even after the peak value is attained, as shown in Fig. 5.

Studies of Bond Capacity of HSC

The bond capacity of steel embedded in HSC has been studied recently in a number of investigations. It is important to note that up until recently there have been relatively few data on HSC bond with most data at strength levels of

56 MPa and below. The studies that can be summarized as follows:

Hadje-Ghaffari et al. (30) conducted a limited study of bond with concrete strengths of 84 MPa and compared their results with beam end specimens with results from 48 MPa tests. The data showed virtually no increase in capacity with increasing concrete strength. These tests, however, were with unconfined specimens where splitting failures would be expected to occur and not be delayed by confinement. In a follow-up study, Darwin and Graham (22) studied the effects of bar and concrete parameters on bond strength and found that aggregate properties significantly affected bond performance.

Hwang, Leu and Hwang (34) studied the bond performance of specimens with 70 MPa concrete, thus placing their results in between the NSC and HSC regimes used in this presentation. Their paper compared results with Orangun, 318-89 and present 318-95 predictions and found that the equations generally did quite well in predicting anchorage strength with the ACI equations being more conservative in their predictions. In addition, the effects of confinement was more pronounced in HSC than in NSC.

Kimura and Jirsa (36) used a small "modified" pull out specimen and found that bond stress and splitting were indeed a function of the square root of the cylinder strength. The results show that the bond stresses, stiffness and first cracking strengths approximately follow the square root law with compressive strength.

French et al. (29) tested HSC using beam end specimens. The investigators found that the concrete compressive strength was a strong factor in strengths up to 70 MPa and that the effects were less pronounced as the concrete entered the high strength regime at strengths above 70 MPa. The data for No. 6 and 11 bars all exhibited a nonlinear curve that tended to "fall off" as the strength went above 70 MPa, as shown in Figs. 6 and 7.

The overall conclusion that can be reached is that while there is limited data available there is strong evidence that bond in HSC may not follow a square root law when the compressive strengths exceed a threshold value, perhaps 70 MPa. Moreover the results are dependent on the test parameters, perhaps significantly more so than with NSC.

One of the most definitive studies has been performed by Azizanamini et al. (7, 8) in which the bond capacity of HSC splice specimens with compressive strengths of 35-42 MPa for NSC and up to 100 MPa for comparison HSC specimens. The splice capacities were evaluated using the "Texas specimen" first used by Treece and Jirsa (48). The results indicate that the compression capacities are linear with cylinder strength while the tensile capacity follows approximately the square root law. Load is carried by the first few deformations of the reinforcing bar and that little load redistribution occurs. Crushing occurs at a higher mean stress in HSC than is normally found with concrete, thus the failure occurs rapidly. Load cannot be redistributed and carried by other deformations farther down the bar since the tensile capacity has not increased by the same amount and splitting occurs at a lower mean stress. Therefore, the bar attempts to carry the load through compression of the HSC around the deformations and once this is lost there is no alternative load path. Finally, the assumption that development length is linear with the number of deformations involved may hold for NSC but may be unconservative for HSC. There may be little additional capacity gained by simply adding more bar length alone. Failure of the concrete around the first highly loaded deformations may produce a failure that can progress quickly along the bar leading to a splitting failure of the cover. Thus, the requirement for a minimum amount of confining steel is more important with HSC than with NSC, particularly with small

covers, to prevent these nonductile splitting failures from occurring.

Observations on Monotonic Bond Performance of HSC – Based on the studies done to date, a number of observations on the bond and development of steel in HSC:

1. The failure surfaces exhibited by HSC are smoother than those found in NSC, typically including a failure plain through, rather than around, aggregates. This smooth surface is indicative of a lack of interlock between failure surfaces. Moreover, the failure behavior tends to be sudden and brittle, more so than with NSC.
2. When reinforcing steel is being developed, there appears to be less “load sharing and redistribution” than in NSC. The bar deformations at the bar ends tend to take more load in any connection, what is significant is that in HSC there is a limited amount of redistribution of load into the connection or development length. Rather the load tends to be concentrated at the ends making the standard assumption of a uniform “bond stress” over the development length less correct than in NSC. When the critical failure stress is reached, there is limited load redistributed and carried in the interior of the bar length and, thus, less redundancy in the system. It has been noted that increased splice lengths do not correlate with an increase in connection capacity because of this phenomenon. Thus the normal approach of providing more deformations, bar development or splice length, to engage the surrounding concrete to achieve more capacity may not be applicable with HSC.
3. The failure stress locally in the concrete at splitting appears to be not linear with compressive strength across the strength range and is most probably a square root function of concrete strength. However, the square root relationship does not appear to hold at higher strength levels, perhaps 70 MPa. The failure process involves the formation of microcracks around the bar as the concrete is dilated due to bar loading and slippage relative to the surrounding concrete. The critical level of failure stress that can be reached prior to splitting failure of the concrete is related to the fracture properties of the concrete and not solely to strength. The fracture energy does not increase with compressive strength as would be expected. Thus, failure caused by fracture may occur at levels similar to those for NSC. Once the failure is initiated, it tends to be more sudden than with NSC with less redistribution of the load from the points of failure into the other portions of the bar development length or connection.
4. Confinement is important in HSC to an even larger degree than with NSC. Because the behavior is nonductile, designers need to properly detail structures so that the adverse effects of HSC can be controlled. However, there is limited data in this area upon which to build design rules, thus the need for more study. It is clear that high strength stirrups, that is, those with higher yield strength than “normal” stirrups are not of significant assistance because the HSC does not permit the stirrups to yield. Thus, adjusting the yield strength does not increase the capacity of the system. The data suggest that the effect of stirrups may be more related to area and stiffness provided than to yield force as is conventionally assumed in NSC.
5. The top bar effect is reduced in HSC probably because of the limited amount of bleeding around bars and the lower water cement ratio to start with. For compressive strengths above 100 MPa, top bar factors approaching one have been reported (14).

Bond Capacity of HSC under Cyclic Loading – The bond of reinforcing steel in NSC and HSC is made more complex by cyclic loading, such as that

induced by earthquake. Here the loading direction changes and the area in front of the deformations is alternately loaded in compression and then in tension. Thus the concrete is subjected to load reversals that accelerate the damage process and can lead to bond failures at lower loads than found in monotonic loading. The proper detailing of reinforcing steel in a cyclic load environment is, thus, even more important than under unidirectional loading.

There is very limited information available on the performance of bond specimens under cyclic loadings. The best single source of information on the bond considerations of reinforced concrete systems for cyclic loadings is the ACI 408 State-of-the Art Report on Bond under Cyclic Loads (5). It is stated in this report that data on this subject is limited. However, seismic excitations are such that the observations applicable to monotonic situations generally hold under seismic demands as well. The key appears to be to hold the bond stresses to approximately 80 percent of ultimate bond stress or less. Limiting the bond stress helps control the demand in the bond areas. Once the area surrounding the bar is damaged by large stress demands, bond slippage will occur and the damage process will begin.

Another observation made in the ACI 408 report is that confinement is essential to keep the bond regions intact and to control damage and bar slippage. Once slippage occurs, stiffness of the member will be compromised. Thus, as in monotonic loading, the role of confinement appears to be the primary factor in determining how a member will behave under cyclic loading. The role of proper detailing, therefore, becomes a paramount consideration when one moves from NSC to HSC.

In the area of cyclic loading and its effects on bond, a significant consideration is bond slip as mentioned above. The larger the slippage of bond, the lower the stiffness of the member resulting in increased deformations. The prediction of the actual performance of bond during loading – the slip -- is difficult to measure experimentally and generally is reported as a loaded end slip or unloaded end slip (21). While this type of information is helpful, it is important to be able to predict how the slippage occurs as a function of demand on the bar. This information can help in defining better concepts for determining bond performance, as well as providing computational model for use in analysis. The area of bond models has not been a primary focus of North American research but rather has been employed by engineers in Europe and Japan as a means to study bond. Work by Elgehausen et al. (26) was fundamental in defining the bond model employed by the CEB Model Code (17). Recent work by Bijag (9) shows how useful this type of approach can be in understanding what the actual behavior is, and how to improve performance. Moreover, these types of models are useful in computer-based analysis of dynamic response to excitations to predict actual behavior more closely.

CONCLUSIONS

The bond of reinforcing bars is a complex phenomenon that has occupied researchers for most of this century. Experimental work has shown that the bond process can be greatly affected by many factors that only now are beginning to be understood. Most of this work, however, has concerned normal strength concrete. What research that has been done on HSC has shown that prediction of bond capacities should be approached with caution.

In particular, the challenge of designing HSC structures to resist seismic or other cyclic loading regimes is an area that needs significant work. Existing research indicates that longer development or splice lengths do not translate into increased safety and in fact can be dangerous. Instead designers need to carefully

detail members to confine the concrete around the development or splice areas to overcome the tendency of HSC to nonductile failures. Adequate levels of confinement are more important with HSC than with NSC and are necessary to permit load redistribution along a splice or developing bar as highly loaded regions of the bar-concrete interface fail and release their load. This behavior is significantly different than in NSC where redistribution is more likely to occur even with nominal levels of confinement.

In summary, the role of HSC in modern reinforced concrete structures will continue to grow. To support this increased usage, research is needed to answer the fundamental issues relating the behavior of this new material to design. One of the primary needs is to build on the work already completed in NSC and HSC to increase our knowledge of bond and development of steel reinforcement in HSC.

REFERENCES

1. Abrams, D. A. (1913), "Tests of Bond Between Concrete and Steel," *Bulletin No. 71*, University of Illinois Experiment Station, Urbana.
2. ACI Committee 318 (1992) *Building Code for Reinforced Concrete* (ACI 318-89) (Revised 1992) and *Commentary* - ACI 318R-89, Farmington Hills, MI, 353 pp.
3. ACI Committee 318 (1995), *Building Code Requirements for Structural Concrete* (ACI 318-95) and *Commentary* - ACI 318R-95, American Concrete Institute, Farmington Hills, MI, 369 pp.
4. ACI Committee 408 (1990), *Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension* (ACI 408.1R-90), American Concrete Institute, Farmington Hills, MI.
5. ACI Committee 408, (1992), *State of the Art Report on Bond under Cyclic Loads* (ACI 408.2R-92), American Concrete Institute, Farmington Hills, MI, 32 pp.
6. ASTM. "Standard Specification for Epoxy-Coated Reinforcing Steel Bars (ASTM A 775/A 775 M - 94d)," *1995 Annual Book of ASTM Standards*, V.4.02, American Society of Testing and Materials, Philadelphia, PA, pp. 406-411.
7. Azizinamini, A, M. Stark, J. J. Roller and S. K. Ghosh (1993). "Bond Performance of Reinforcing Bars Embedded in High-Strength Concrete," *ACI Structural Journal*, V. 95, No. 5, Sept.-Oct., pp. 554-561.
8. Azizinamini, A, M. Chisala, and S. K. Ghosh (1995), "Tension Development Length of Reinforcing Bars Embedded in High-Strength Concrete," *Engineering Structures*, V. 17, No. 7, pp. 512-522.
9. Bigaj, A.J., J. A. Den Uijl and J.C. Walraven (1996), "A Bond Model for Ribbed Bars in HSC and NSC - an Experimental Study," *Proceedings*, 4th International Symposium on Utilization of High-Strength/High Performance Concrete, Paris, 10 pp.
10. Cairns, J., (1992), "Design of Concrete Structures Using Fusion-Bonded Epoxy Coated Reinforcement," *Proceedings*, ICE, V. 94, Feb., pp. 93-102.
11. Chamberlin, S. J., (1956), "Spacing of Reinforcement in Beams," *ACI Journal*, *Proceedings*, V. 53, No. 1, July, pp. 113-134.

12. Chamberlin, S. J. (1958), "Spacing of Spliced Bars in Beams," *ACI Journal, Proceedings*, V. 54, No. 8, Feb., pp. 689-698.
13. Chinn, J., P. M. Ferguson, and J. N. Thompson (1955), "Lapped Splices in Reinforced Concrete Beams," *ACI Journal*, V. 52, No. 2, Oct., pp. 201-214.
14. Choi, O. C., H. Hadje-Ghaffari, D. Darwin and S. L. McCabe (1991), "Bond of Epoxy-Coated Reinforcement: Bar Parameters," *ACI Materials Journal*, V. 88, No. 2, March-April, pp. 207-217.
15. Clark, A. P (1946), "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars," *ACI Journal*, V. 18, No. 4, December.
16. Cleary, D. B. and J. A. Ramirez, (1989), "Bond of Epoxy Coated Reinforcing Steel in Concrete Bridge Decks," *ACI Materials Journal*, V. 88, No. 2, Mar-Apr., pp. 146-149.
17. Committee Euro-International du Beton (CEB) (1990), *CEB-FIP Model Code*, Lausanne, Switzerland.
18. Cusens, A. R. and Y. Yu, (1993), "Bond Strength and Flexural Behavior of RC Beams with Epoxy-Coated Reinforcing Bars," *The Structural Engineer*, V. 71, No. 7, pp. 117-124.
19. Darwin, D., S. L. McCabe, and O.C. Choi, (1990), "Evaluation of Bond Performance of Epoxy-Coated Reinforcement Using Nonlinear Finite Element Analysis," *Proceedings, First ASCE Materials Engineering Conference*, Denver, Aug, V. 1, pp. 135-144.
20. Darwin, D., S. L. McCabe, H. Hadje-Ghaffari and O.C. Choi, (1990), "Bond Strength of Epoxy-Coated Reinforcement in Concrete –an Update," *Proceedings, First ASCE Materials Engineering Conference*, Denver, Aug, V. 1, pp. 115-124.
21. Darwin, D., S. L. McCabe, E. K. Idun, and S. P. Schoenekase (1992), "Development Length Criteria: Bars Not Confined by Transverse Reinforcement," *ACI Structural Journal*, V. 89, No. 6, Nov.-Dec., pp. 709-720.
22. Darwin, D. and E. K. Graham (1993), "Effect of Deformation Height and Spacing on Bond Strength of Reinforcing Bars," *ACI Structural Journal*, V. 90, No. 6, Nov.-Dec., pp. 646-657.
23. Darwin, D., M. L. Tholen, E. K. Idun and J. Zuo (1996), "Splice Strength of High Relative Rib Area Reinforcing Bars," *ACI Structural Journal*, V. 93, No. 1, Jan.-Feb., pp. 95-107.
24. DeVries, R. A.; J. P Moehle, and W. Hester, (1991) "Lap Splice Strength of Plain and Epoxy-Coated Reinforcements: An Experimental Study Considering Concrete Strength, Casting Position, and Anti-Bleeding Additives," *Report No. UCB/SEMM-91/02*, Dept. of Civil Engineering, University of California Berkeley, 86 pp.
25. Eligehausen, R, (1979), "Bond in Tensile Lapped Splices of Ribbed Bars with Straight Anchorages," *Publication 301*, German Institute for Reinforced Concrete, Berlin, 118 pp. (in German).
26. Eligehausen, R, E. P. Popov, and V.W. Bertero, (1983) "Local Bond Stress-Slip Relationships of Deformed Bars under Generalized Excitations," *Report*

No. UCB/EERC-83/23, University of California Berkeley, 169 pp.

27. Ferguson, P. M. and J. E. Breen (1965), "Lapped Splices for High Strength Reinforcing Bars," *ACI Journal*, V. 62, No. 9, Sept., pp. 1063-1078.
28. Ferguson, P. M. and J. N. Thompson, (1965), "Development Length of High Strength Reinforcing Bars," *ACI Journal*, V. 62, No. 1, Jan., pp. 71-94.
29. French, C., R. Leon, and T. Grunddhoffer, (1992), "Bond Behavior of Uncoated and Epoxy-Coated Reinforcement in Concrete," *Structural Engineering Report No. 92-04*, University of Minnesota Center for Transportation Studies, May, 208 pp.
30. Hadje-Ghaffari, H., D. Darwin and S. L. McCabe (1991), "Effects of Epoxy Coating on Bond of Reinforcing Steel to Concrete," *SM Report No. 28*, University of Kansas, Lawrence, July, 288 pp.
31. Hamad, B. S. and J. O. Jirsa, (1990), "Strength of Epoxy-Coated Reinforcing Bar Splices Confined with Transverse Reinforcement," *ACI Structural Journal*, V. 90, No. 1, Jan.-Feb., pp. 77-88.
32. Harajli, M. H. (1994), "Embedded in Plain and Fiber Reinforced Concrete," *ACI Structural Journal*, V. 91, No. 5, Sep.-Oct., pp. 511-520.
33. Hester, C. J.; S. Salamizavaregh, D. Darwin, and S. L. McCabe (1993), "Bond of Epoxy-Coated Reinforcement: Splices," *ACI Structural Journal*, V. 90, No. 1, Jan.-Feb., pp. 89-102.
34. Hwang, S. J., Y. R. Leu and H. L. Hwang (1993), "Tensile Bond Strengths of Deformed Bars in High-Strength Concrete," *ACI Structural Journal*, V. 93, No. 1, Jan.-Feb. pp. 11-20.
35. Johnston, D. W. and P. Zia (1982), "Bond Characteristics of Epoxy Coated Reinforcing Bars," *Report No. FHWA-NC-82-002*, Center for Transportation Engineering Studies, Civil Engineering Department, North Carolina State University, Raleigh, 63 pp.
36. Kimura, H. and J. O. Jirsa (1992), "Effects of Bar Deformation and Concrete Strength on Bond of Reinforcing Steel to Concrete," University of Texas Ferguson Structural Engineering Laboratory, *Report PMFSEL 92-4*, September.
37. Maeda, M.; S. Otani, and H. Aoyama, (1991), "Bond Splitting Strength in Reinforced Concrete Members," *Transactions of the Japan Concrete Inst.*, V. 13, pp. 581-588.
38. Mathey, Robert and D. Watstein (1961), "Investigation of Bond in Beam and Pull-Out Specimens with High-Yield-Strength Deformed Bars," *ACI Journal*, V. 32, No. 9, Mar., pp. 1071-1090.
39. Mirza, S. Ali and J. G. MacGregor (1986), "Strength Variability of Bond of Reinforcing Bars in Concrete Beams," *Civil Engineering Report Series No. CE-86-1*, Lakehead University, Thunder Bay, Ontario, Jan., 35 pp.
40. Orangun, C. O., J. O. Jirsa, and J. E. Breen (1975), "The Strength of Anchored Bars: A Reevaluation of Test Data on Development Length and Splices," *Research Report No. 154-3F*, Center for Highway Research, Univ. of Texas at Austin, Jan., 78 pp.

41. Orangun, C. O., J. O. Jirsa, and J. E. Breen (1977), "Reevaluation of Test Data on Development Length and Splices," *ACI Journal*, V. 74, No. 3, Mar., pp. 114-122.
42. Rezanoff, T.; A. Akanni, and B. Sparling (1993), "Tensile Lap Splices under Static Loading: A Review of The Proposed ACI 318 Code Provisions," *ACI Structural Journal*, V. 90, No. 4, July-Aug., pp. 374-384.
43. Sakurada, T; N. Morohashi and R. Tanaka (1993), "Reinforcement on Bond Splitting Strength of Lap Splices," *Transactions of the Japan Concrete Inst.*, V. 15, pp. 573-580.
44. Sozen, M. A. and J. P. Moehle (1990), "Development Length and Lap-Splice Lengths for Deformed Reinforcing Bars in Concrete," *Report to the Portland Cement Association and Concrete Reinforcing Steel Institute*.
45. Tepfers, R., (1973), "A Theory of Bond Applied to Overlapped Tensile Reinforcement Splices for Deformed Bars," *Publ. 73.2*, Div. of Concrete Structures, Chalmers University, Göteborg
46. Tepfers, R., (1982), "Lapped Tensile Reinforcement Splices," *Journal of the Structural Division*, ASCE V. 108, No. ST1, Jan., pp. 283-301.
47. Thompson, M. A. J. O Jirsa, J. E. Breen, and D. F Meinheit. (1975), "The Behavior of Multiple Lap Splices in Wide Sections," *Research Report No. 154-1*, Center for Highway Research, The University of Texas at Austin, Feb., 75 pp.
48. Treece, R. A. and J. O. Jirsa (1989), "Bond Strength of Epoxy-Coated Reinforcing Bars," *ACI Materials Journal*, V. 86, No. 2, Mar.-Apr., pp. 167-174.
49. Zekany, A. J.; S. Neumann, and J. O. Jirsa (1981), "The Influence of Shear on Lapped Splices in Reinforced Concrete," *Research Report No. 242-2*, Center for Transportation Research, Bureau of Engineering Research, University of Texas at Austin, July, 88 pp.

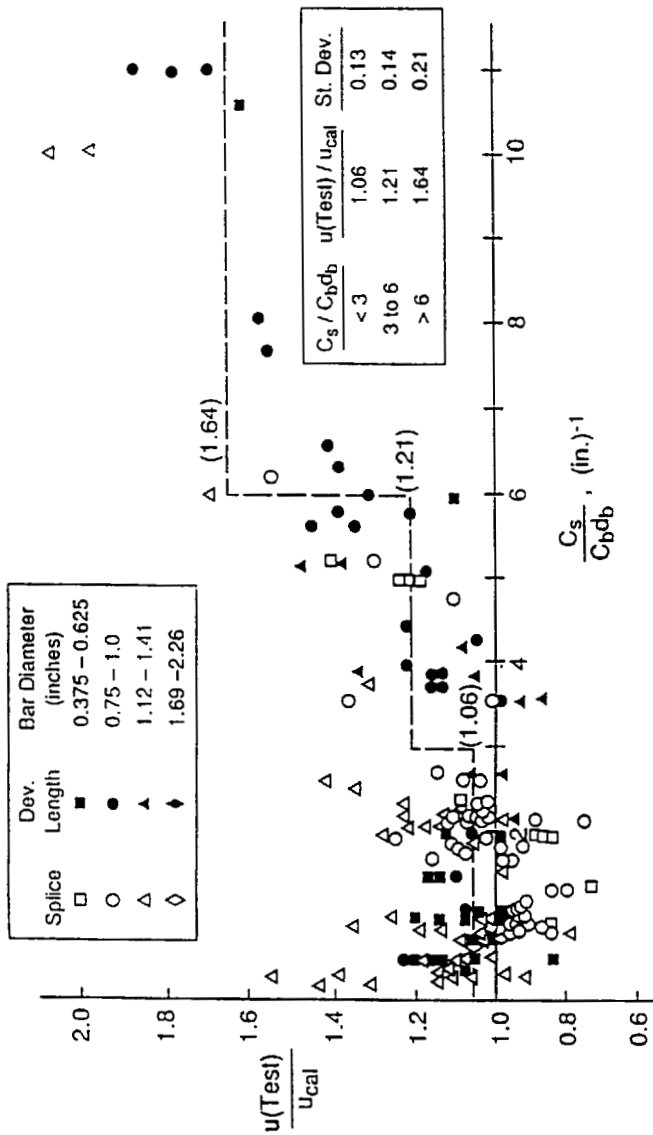


Fig 1—Ratio of bond stresses-test to calibration $[u(\text{test})/u_{\text{cal}}]$ versus $C_s/(C_b d_b)$
(from references 40 and 41)

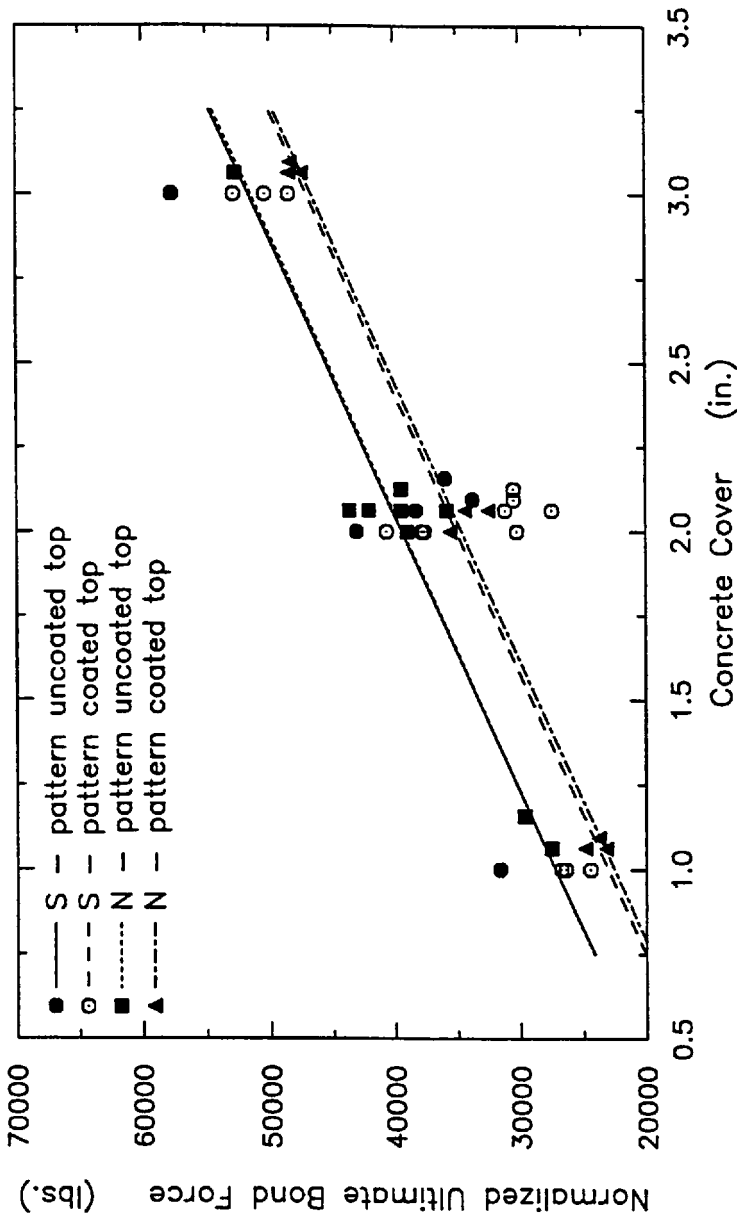


Fig 2—Normalized ultimate bond force versus concrete cover for No. 8 bars (from Reference 30)

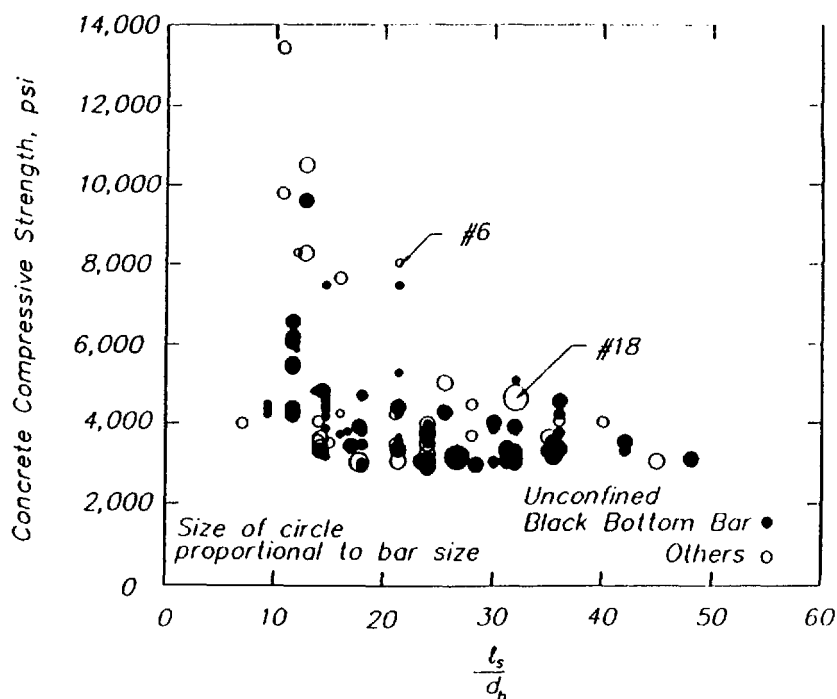
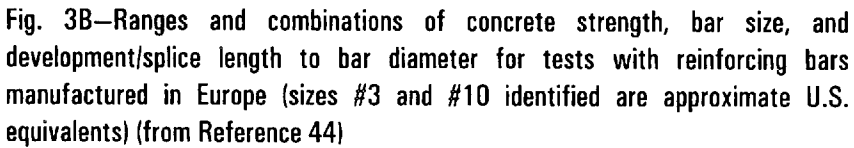


Fig 3A—Ranges and combinations of concrete strength, bar size, and development/splice length to bar diameter for tests with reinforcing bars manufactured in the U.S.A. (from Reference 44)



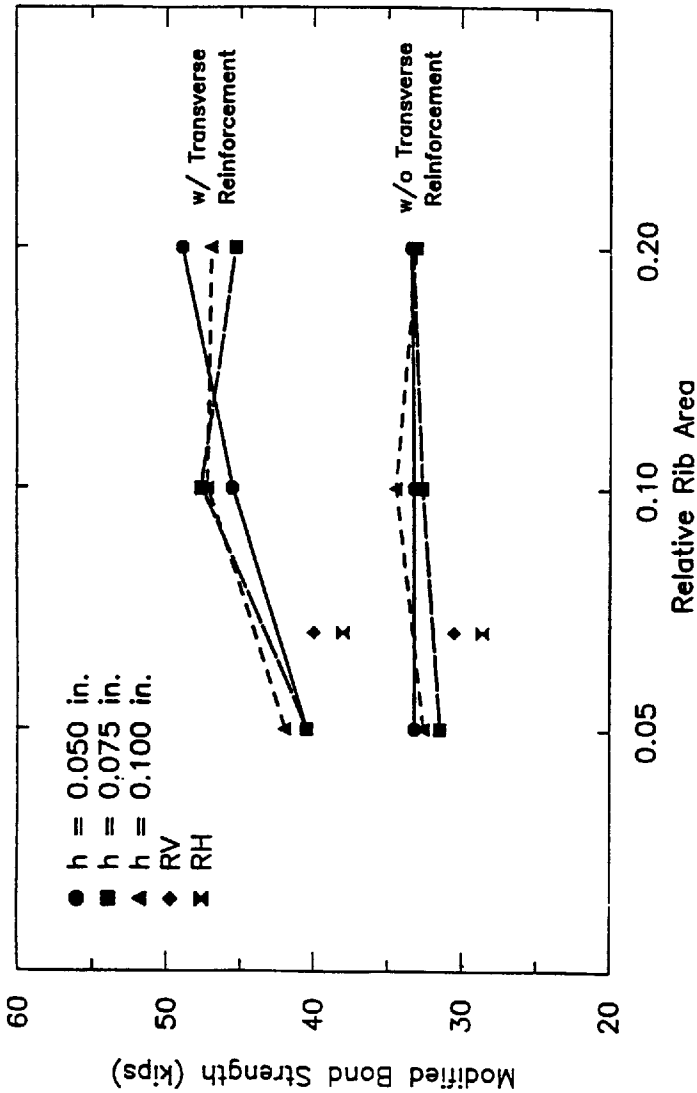


Fig. 4—Modified bond strength versus relative rib area comparing the effects of confinement provided by transverse reinforcement. Beam-end test specimens modified to include hooked auxillary reinforcement (cover=2 in., bonded length= 12 in.) (from Reference 22)

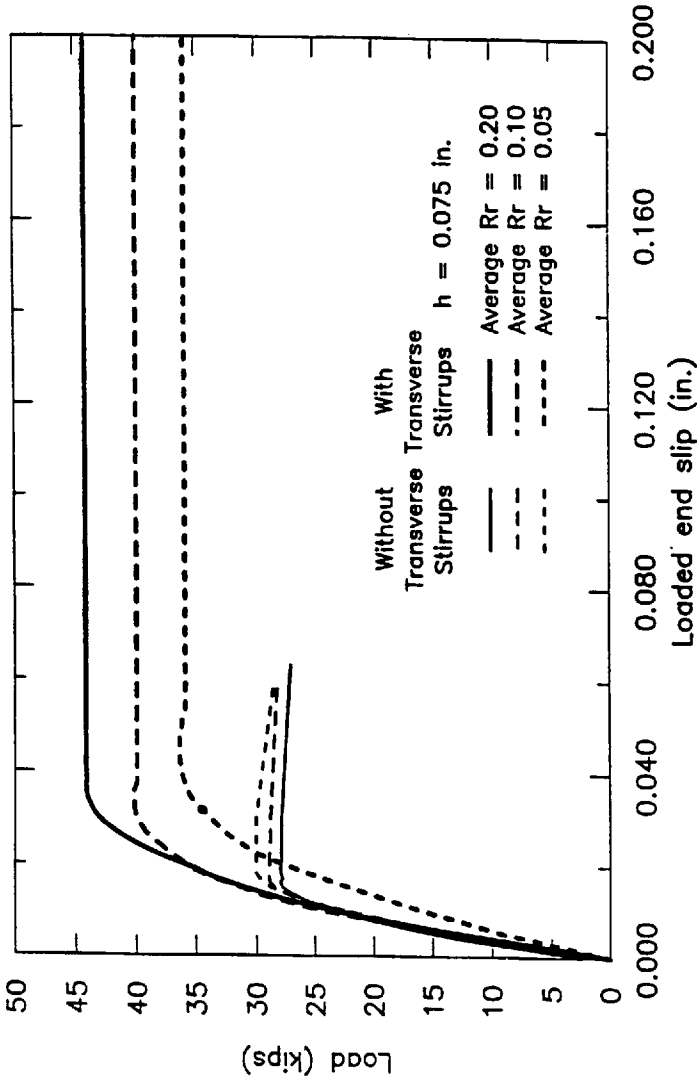


Fig. 5—Average load-loaded end slip curves for test bars with rib height: $H = 0.075$ in. and relative rib areas = 0.20, 0.10, and 0.05 (cover = 2 in., lead length = $1/2$ in., bonded length = 12 in.) (from Reference 22)

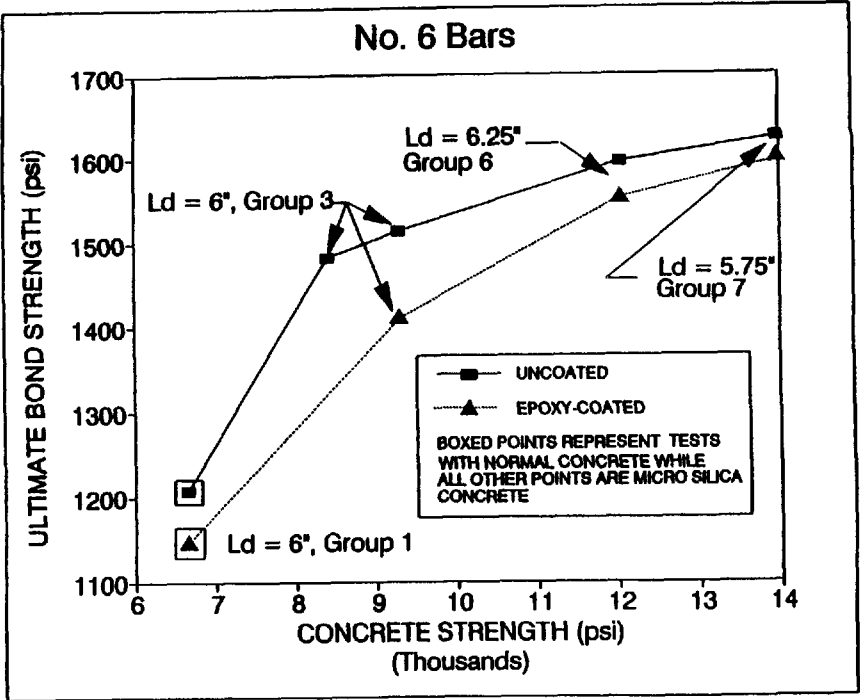


Fig. 6—No. 6 bars, bond strength versus concrete strength (from Reference 29)

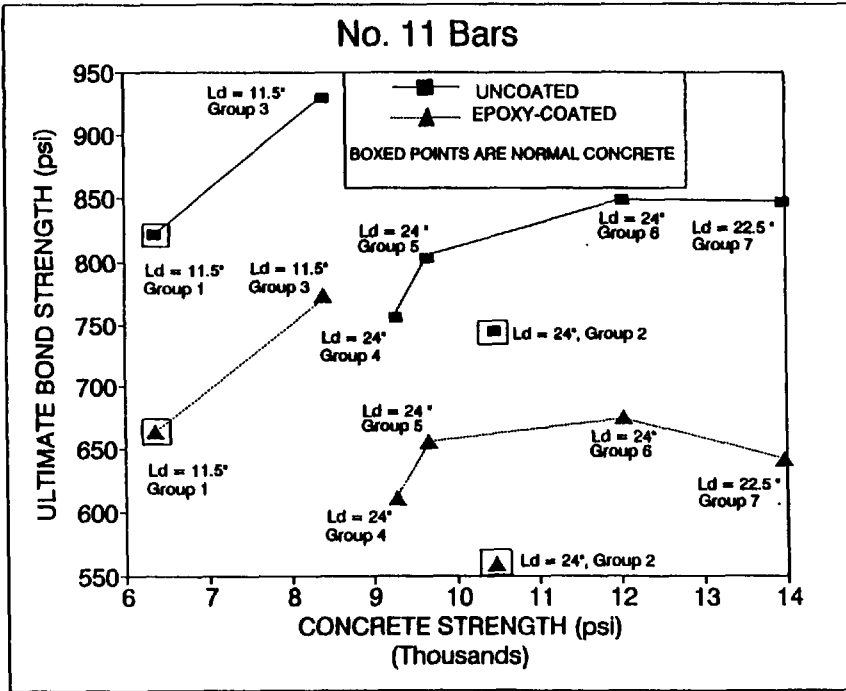


Fig. 7—No. 11 bars, bond strength versus concrete strength (from Reference 29)

SP 176-2

Bond and Anchorage of Reinforcement in High-Strength Concrete

by S. Fujii, H. Noguchi and S. Morita

Synopsis: In this paper, research on bond and anchorage of reinforcement in high strength concrete were reviewed. They were classified to three groups: research on bond capacity with splitting of surrounding concrete, bond deterioration of bars passing through beam-column joints and anchorage capacity of hooked bars in beam-column joints. A characteristic property of low tensile strength relative to the high compressive strength results in a small increase of bond and anchorage capacity if the failure mode is governed by concrete splitting. Transverse reinforcement is more important for high strength concrete. The effect of concrete strength is more for the bond which failed in concrete crushing or direct shearing at the interface, such as the bar passing through the joint. High compressive strength and high rigidity of stress-strain curve makes the local bond-slip curve stiffer. Low sedimentation and low bleeding effects make the top bar effect small. By analyzing available research, bond and anchorage capacities were evaluated quantitatively for practical design use.

Keywords: Anchorage (structural); bonding; high-strength concrete; joints (junctions); reinforced concrete; shear properties; steels

Shigeru Fujii is an Associate Professor of Architectural Engineering at Kyoto University. He received a Doctor of Engineering from Kyoto University in 1992. His research interests include bond and anchorage, shear resistance mechanism in reinforced concrete. He is a member of AIJ and JCI.

Hiroshi Noguchi is a Professor of Architectural Engineering at Chiba University. His research interests include aseismic design of reinforced concrete structures and applications of FEM on shear and bond problems in reinforced concrete structures and members. He is a member of technical committees of AIJ and JCI. He is also a member of ACI, IABSE and CEB.

Shiro Morita is a Professor of Architectural Engineering at Kyoto University. He received AIJ Award in 1970 and JCI Award in 1980 for his papers on bond and cracking in reinforced concrete. His research interests include the field of concrete and reinforced concrete engineering. He is a member of technical committees of AIJ and JCI. He is also a member of ACI, IABSE and CEB.

INTRODUCTION

In order to develop super highrise reinforced concrete buildings in seismic zones, the Ministry of Construction managed a five-year research project called 'New RC Project' from 1988 to 1993 in Japan. Through these research activities, many findings on bond and anchorage characteristic behaviors of high strength reinforcement (ranged from 300 to 1200MPa in yield strength) in high strength concrete (ranged from 40 to 120MPa) had been rapidly accumulated. In this paper, recent research work on 1) splitting bond failure of longitudinal bars or lap splice bars, 2) bond deterioration of beam bars passing through interior beam-column joints and 3) hooked bar anchorage in exterior beam-column joints are outlined and reviewed. Based on this research work, bond and anchorage design equations which are applicable to high strength concrete up to 120MPa are proposed, and future research needs are identified.

SPLITTING BOND CAPACITY IN RC MEMBERS

Research Work

Experimental work on splitting bond strength in beams with high strength concrete were conducted by several researchers in Japan (1,2,3). In these tests, partial beam specimens (often called stub beam or cantilever beam tests) or simple beam specimens were used. Regarding the splitting bond strength of lapped splices with high strength concrete, research works conducted by Azizinamini et al. (4) and Olsen (5) are available.

Effect of Concrete Strength

Figure 1 shows the effect of concrete strength on splitting bond strength (1), in which all variables were constant except concrete strength. The splitting bond strength increases almost proportionally to

the square root of concrete strength f_c . A similar result was also obtained in the other research (3). In the strength equation proposed by Kaku et al.(1), the bond strength was evaluated to be proportional to $f_c^{0.6}$ derived from the regression analysis using available test data.

On the other hand, test results on lapped splices (4,5) showed that the bond strength normalized by the square root of f_c becomes smaller for high strength concrete in comparison with ordinary strength concrete (see Figure 4). Azizinamini stated that the non-uniform bond stress distribution along bars in the case of high strength concrete contributes to this feature. Olsen stated that the bond strength is governed by the fracture energy of the concrete. It should be noted that these splice test specimens had no transverse reinforcement.

Effect of Casting Position

In ordinary strength concrete up to 30MPa, the strength ratio of top cast bar to bottom cast bar was given as 0.82 in average (6). Figure 2 shows a variation of the strength ratio with an increase of concrete strength, which was analyzed by Maeda et al.(7). Strength reduction for top-cast bars is reduced as the concrete strength increases. This ratio was also evaluated by Kaku et al.(1) using the test data without stirrups, where the ratio was 0.82 at 30MPa in f_c and reached 1.0 at 100MPa. Low sedimentation and low bleeding effects due to reduced water-cement ratio make the top bar effect small.

Effect of Bond Length

Figure 3 shows the effect of the bond length on the bond strength with high strength concrete of 100MPa (1). Average bond strength decreases as the bond length increases. This effect reduced for the specimens with transverse reinforcement. Figure 4 shows the lapped splice test result conducted by Olsen (5), in which the test specimens had no transverse reinforcement. The same effect of splice length can be seen, but this effect is more significant for high strength concrete specimens than ordinary strength concrete specimens. For high strength concrete, especially in the case of no transverse reinforcement, a local bond stress distribution at the ultimate stage is highly non-uniform along bars due to the brittle behavior in local bond-slip relationships, which is a cause of the larger effect of splice length on average bond strength.

Effect of Transverse Reinforcement

Research results showed that the placement of transverse reinforcement was effective to improve the bond strength and ductility (1, 2, 3). Considering the brittle failure mode and a relatively low tensile strength in the case of high strength concrete, the confinement by

transverse reinforcement becomes more important than in the case of ordinary strength concrete.

In previous strength equations, the parameter such as $A_{tr}f_{yt}/s d_b$ (8) or $A_{st}/s N d_b$ (6) was introduced for evaluating the strength increment due to transverse reinforcement. (A_{tr} and A_{st}/N are the area of transverse reinforcement per one longitudinal bar, s is the average spacing of the transverse reinforcement, d_b is longitudinal bar diameter and f_{yt} is the yield strength of transverse reinforcement.) However, recently conducted tests showed that these parameters were not necessarily sufficient. First, an increase in the number of stirrup legs by adding supplementary ties to support intermediate bars is effective to improve bond strength. This effect should be taken into account. Secondly, the effectiveness of the yield strength of transverse reinforcement should be examined. Maeda et al.(7). formulated the relationship between the bond strength increase and the actual confining stress of transverse reinforcement. Larger confining stress improves the bond strength. However, higher yield strength of stirrups does not mean higher stress development in actual members at bond failure. The test result conducted by Kaku et al.(9) showed that the bond strength of the specimen with high strength stirrups (1350 MPa) was as same as the strength of the specimen with ordinary strength stirrups (350 MPa). It is noted that the strength equation which includes the yield strength of transverse steel over-estimates the bond strength when high strength stirrups were used.

Development Length Equation

Based on the test results, the bond strength equations were proposed by several researchers in Japan (1, 2 and 7). Each equation has an improvement in agreement with test results for high strength concrete and various transverse reinforcement arrangements. Reviewing these equations, the following strength equation can be derived for practical design use (10) which was based on Kaku's proposal (1).

For top cast bars, the average bond strength τ is given by summation of the strength without transverse reinforcement τ_{co} and the strength increase due to transverse reinforcement τ_{st} :

$$\tau = \tau_{co} + \tau_{st} \quad (\text{in MPa}) \quad (1)$$

$$\tau_{co} = \{ 0.1456 + (0.218 + 4.55 \frac{d_b}{l}) b_i \} \cdot 0.313 \sqrt{f_c} \quad (\text{in MPa}) \quad (2)$$

$$\tau_{st} = (1.6 - 12 \frac{d_b}{l}) \cdot 35.5 k_1 \frac{A_{st}}{s N d_b} \cdot 0.313 \sqrt{f_c} \quad (\text{in MPa}) \quad (3)$$

where, b_i : coefficient on concrete cover or bar spacing in a section
(see Fig.5)

$$\begin{aligned}
 b_i &= \min. (b_{si}, b_{ci}, b_{vi}) \\
 b_{si} &= (b - Nd_b)/Nd_b \\
 b_{ci} &= 3C_c/d_b \\
 b_{vi} &= 3C_v/d_b
 \end{aligned} \tag{4}$$

C_c : minimum cover for a bar at corner in a section (cm)

C_v : minimum cover for an isolated bar (cm)

b : beam width (cm)

N : number of longitudinal bars in a layer

k_1 : coefficient considering stirrup legs effect

$$k_1 = 0.4 + 0.6 n/N \quad (\text{when } b_i = b_{si}) \tag{5}$$

$$k_1 = 1.0 \quad (\text{with } N=2 \text{ in Eq.(3) when } b_i = b_{ci})$$

$$k_1 = 0 \quad (\text{when } b_i = b_{vi})$$

n : number of legs of transverse reinforcement in a spacing s

s : spacing of a set of transverse reinforcement (cm)

l : bond length (cm)

d_b : diameter of longitudinal bars (cm)

A_{st} : total area of a set of transverse reinforcement in a spacing s (cm²)

(When $b_i = b_{ci}$, A_{st} is the area of a particular stirrup supporting the corner bar.)

For the bars other than top bars, the following coefficient k_2 shall be multiplied;

$$\begin{aligned}
 k_2 &= 1.22 & (f_c \leq 30 \text{ MPa}) \\
 &1.32 - 0.0033 f_c & (30 < f_c \leq 100 \text{ MPa}) \\
 &1.0 & (100 \text{ MPa} < f_c)
 \end{aligned} \tag{6}$$

The required development length l for the bar stress σ_s is given by the following equation:

$$l = \frac{0.8 \frac{\sigma_s}{\sqrt{f_c}} - (4.55 b_i - 426 \frac{k_1 A_{st}}{s N d_b})}{0.1456 + 0.218 b_i + 56.8 \frac{k_1 A_{st}}{s N d_b}} \cdot d_b \tag{7}$$

BOND CAPACITY OF BARS PASSING THROUGH INTERIOR BEAM-COLUMN JOINTS

Research Work

There is some research available to examine the bond capacity of beam bars passing through beam-column joints with high strength concrete. Morita et al.(11) conducted the pull-push type bond tests simulating actual seismic loading condition of interior beam-column joints with high strength concrete and high strength steel. The cyclic

loading tests of beam-column sub-assemblages focusing on the bond deterioration in high strength concrete were carried out by Kashiwazaki et al.(12), Kitayama et al.(13) and Hayashi et al.(14), where the relationship between beam bars' bond ability and the performance of the sub-assemblage was investigated.

Results of Pull-Push Type Bond Tests

Figure 6(a) shows the pull-push type bond specimen developed by Morita et al.(11). The test specimen is simulating a half portion of a beam-column sub assemblage (a shaded area in Fig. 6(b)). Pull-out force was applied at the end of the test bar and push-in force was applied simultaneously at the opposite end surface of the concrete stub. This test method successfully reproduces the pull-push condition of beam bars in an interior beam-column joint under earthquake type loading. Test variables were concrete strength varied up to 120MPa, yield stress of test bars up to 700MPa, bar diameter of 19, 25 and 35mm, column depth (400 and 600mm) and unit width per a beam bar (100 and 150mm) and column axial stress level (0, 1/6 and 1/3 f_c).

The envelopes of local bond stress-local slip curves in the joint core region are shown in Fig. 7, grouping them into three to show the influence of each main test variable. The increase in concrete strength or column axial stress level gave higher local bond strength and higher rigidity of bond-slip curves. Also, the local bond strength increased as the ratio of the specimen width to bar diameter increased. Based on the test results, the local bond strength τ_u in joint core region was formulated as follows;

$$\tau_u = 2.2 \cdot \left(0.86 + 0.84 \frac{N}{B \cdot D \cdot f_c} \right) \cdot \frac{B_o}{d_b} \cdot \left(\frac{f_c}{35} \right)^{0.66} \quad (\text{in MPa}) \quad (8)$$

where, N: column axial load (positive in compression)
 B: column width (cm)
 B_o: unit column width per one beam bar (cm)
 D: column depth (cm)
 d_b: beam bar diameter (cm)

As shown in the above equation, the effect of concrete strength was evaluated in proportional to $f_c^{2/3}$ for high strength concrete. This equation gives a maximum local bond stress τ_u in a joint core region. However, considering that the maximum local bond stress was retained in large slip stages and that the most of bond resistance is developed at this region after beam yielding, this formula can be used as an index of the overall bond capacity of beam bars in joints. In these tests, failure modes (the grade of the bond deterioration) of each specimen was examined against the ratio τ_u/τ_f , where τ_f means the average bond stress

required for beam bars to develop the tensile yield stress σ_y at the front face and compressive yield stress at the far face of an interior column, which is given by the following equation:

$$\tau_f = \frac{\sigma_y \cdot d_b}{2 \cdot D} \quad (9)$$

Figure 8 shows the relationship between the obtained failure modes and the bond index τ_u/τ_f in comparison with the applicability of the current requirement of AIJ (Architectural Institute of Japan). AIJ Design Guideline (1990) recommends that the bond index D/D_{cal} should be larger than 1.0 for weak-beam and strong-column frames. This requirement is based on the consideration of energy dissipating capacities of story shear-story drift curves of a sub-assembly. D_{cal} is defined as follows;

$$D_{cal} = 0.32 \cdot \frac{\sigma_y \cdot d_b}{\sqrt[3]{f_c}} \quad (f_c \text{ and } \sigma_y \text{ in MPa}) \quad (10)$$

Failure modes observed in the tests were classified into four groups which were described in Fig. 8. The bond index τ_u/τ_f provides a good indicator in comparison with D/D_{cal} . When the bond index τ_u/τ_f was larger than 1.0, the bond failure was avoided within three cycles of reversed loading in post-yield range. In this research, the criteria of three cycles of reversed loading correspond to the beam bar tensile stress of $1.10-1.15\sigma_y$ or strain of 3-4% at the column face.

Results of Beam-Column Sub-assembly Tests

The relationship between input joint shear levels and the bond deterioration of beam bars passing through a joint was investigated by Kashiwazaki et al.(12) using beam-column sub-assembly specimens. The diameter and the number of beam bars were the experimental variables. The influence of joint shear stress levels on bond stress at beam yielding was not observed. Although every specimen showed the poor energy dissipating behavior under load reversals in post-yield range due to beam bar bond deterioration, the difference was not so clear among specimens with a variation of the index τ_u/τ_f . Needs for a re-examination of parameter B_o/d_b in Eq.(8) was suggested.

Kitayama et al.(13) performed beam-column joint tests with a variation of beam bar diameter with high strength concrete. In Fig. 9, local bond stresses along a beam bar in a joint core region are plotted against story drift angle. Bond deterioration was observed in Specimen I3 ($f_c=41.4\text{MPa}$) and I5 ($f_c=85.4\text{ MPa}$). The increase of local bond

strength was not directly proportional to the concrete strength. Based on the test results, the following criteria was developed to prevent the bond deterioration before beam bars yielding;

$$\begin{aligned} U_b/f_c &< 0.2 \quad \text{for top reinforcement,} \\ U_b/f_c &< 0.25 \quad \text{for bottom reinforcement} \end{aligned} \quad (11)$$

where, U_b is defined as follows;

$$U_b = \frac{(1+\gamma) \cdot \sigma_y \cdot d_b}{4 \cdot D} \quad (12)$$

γ is a ratio of compressive to tensile beam bar areas.

The effect of bi-directional loading on bond behavior was also examined in this research. It was reported that the little bond deterioration occurred even under bi-directional loading when the beam bar strain slightly exceeded the yield strain.

Hayashi et al.(14) examined the beam bar bond performance for several combinations of bar diameter and yield strength keeping the joint shear force constant. Test results showed that the flexural compressive failure of beams was predominant when the bond index D/D_{cal} was 0.74, and that significant bond deterioration occurred for the specimens with the bond index D/D_{cal} of 0.57. In the case of high strength steel, an early bond deterioration was observed in comparison with ordinary strength steel. The differences of energy dissipating capacity in hysteresis curves were small among bond-deteriorated specimens with different bond index values. The criteria of design requirement for bond ability, which the current AIJ Design Guideline based on, should be re-examined for the structures using high strength concrete and high strength steel.

A proposal of Bond Requirement in Beam-Column Joints

Based on the above discussion, modifying the effect of the parameter B_o/d_b in Eq.(8) conservatively, a following requirement can be developed;

$$\frac{d_b}{D} \leq 1.34 \cdot \left(1 + \frac{N}{A_g f_c}\right) \cdot \frac{f_c^{2/3}}{f_y} \quad (13)$$

where, N : column axial compressive load

D : column depth

A_g : gross area of column section,

f_y : specified yield strength of beam bars (MPa)

(actual yield strength σ_y was estimated as $1.1f_y$ in this equation)

When the steel area of top and bottom beam bars is different, the following factor k_1 can be multiplied to the above equation for the beam bars with more steel area;

$$k_1 = \frac{2}{1 + \frac{A_{sb}}{A_{st}}} \quad (14)$$

where, A_{st} : beam bar area of more amount side (usually top bars)

A_{sb} : beam bar area of less amount side (usually bottom bars)

CAPACITY OF 90° HOOKED BAR ANCHORAGES

Failure Modes of 90° Hooked Bar Anchorages

Figure 10 shows the anchorage failure modes of 90° hooked bars in beam-column joints: a) splitting failure of column side cover, b) local crushing failure inside hook and c) raking-out of front concrete of a column, which was classified by Joh et al.(17). When the side cover is small or when the transverse confinement is not sufficient, splitting of side cover governs the anchorage capacity. When the bar is anchored in massive concrete, local crushing of the concrete inside the bent portion occurs and the bar is pulled out. When a large number of bars are anchored with a narrow spacing in relatively short anchorage length, the concrete inside the bent is raked out as a whole. It is a kind of shear failure.

Anchorage Failure due to Splitting of Column Side Cover

Figure 11 shows a test specimen and loading conditions in the research conducted by Kawakami et al.(15) and Yamada et al.(16). In these experiments, details of bent portions, projected development length, transverse reinforcement, bar diameter and side cover were test variables adding to the concrete strength variation (40, 80 and 120MPa). All specimens failed in anchorage with splitting of column side cover. Figure 12 shows an example of load-slip curves for specimens with different concrete strength levels. As the concrete strength increased, the maximum load and the slip value at the peak load were increased. The effect of concrete strength on anchorage strength was examined in Fig. 13 using all available test data by Fujii et al.(18). The anchorage strength and concrete strength of each test specimen are given as the strength ratios relative to the test specimen of 40MPa concrete. For the specimens with high strength concrete, the effect of concrete strength can be described as a linear function of $f_c^{1/3}$ rather than square root of f_c . It should be noted that the previous strength equations based on the square root of f_c tends to over-estimate the anchorage strength for high strength concrete.

A Proposal for Design Equations for 90° Hooked Bar Anchorages

The anchorage failure due to local crushing of concrete inside the bend can be avoided by regulating the radius of the bend for combinations of the strength levels of concrete and steel. For the anchorage failure with column side cover splitting, simplifying the strength equation proposed by Fujii et al.(18), the maximum tensile stress developed in 90° hooked bar anchorage f_u can be formulated as follows;

$$f_u = 200 \cdot k_c \cdot k_j \cdot k_d \cdot k_s \cdot f_c^{0.4} \quad (\text{MPa}) \quad (15)$$

where, the factors k_c , k_j , k_d and k_s are given as follows;

$$k_c = 0.43 + \frac{0.1 \cdot C}{d_b} \quad (\leq 1.0) \quad (16)$$

$$k_j = 0.8 + 0.5 \frac{l_{dh}}{j} \quad (1.0 \leq k_j \leq 1.4) \quad (17)$$

$$k_d = 0.038 \cdot \frac{l_{dh}}{d_b} + 0.54 \quad (\leq 1.0) \quad (18)$$

$$k_s = 0.7 + \frac{0.46 \cdot d_s^2}{d_b^2} \quad (\leq 1.0) \quad (19)$$

where, C: column net side cover to the outermost anchored bar (cm)

d_b : anchored bar diameter (cm)

l_{dh} : projected development length (cm)

j: internal lever arm of beam section at column face (cm)

d_s : diameter of transverse reinforcement positioned at the hook portion (cm)

The configuration of the hook should satisfy the requirement of standard hook in the ACI Building Code. In the case of double layered bar arrangement, anchorage capacity should be calculated for the condition of inner layered bars and the reduction factor of 0.75 should be multiplied. This strength reduction of inner layered bars was observed in tests conducted by Murakami et al.(22)

CONCLUSIONS AND FUTURE RESEARCH NEEDS

Reviewing the recent research work, bond and anchorage abilities in high strength concrete were examined. A characteristic of relatively low tensile strength of high strength concrete results in a small increase of bond and anchorage capacity when the failure mode is governed by concrete splitting. The effect of concrete strength is more for the bond which failed in concrete crushing or direct shearing at the interface, such as the bar passing through the joint. High compressive strength and high rigidity of stress-strain curve makes the local bond-slip curve stiffer. Low sedimentation and low bleeding effects make the top bar effect small.

The effect of other parameters, such as concrete cover, development length, transverse reinforcement etc. was similar to previous research findings for ordinary strength concrete (19, 20). Important features can be summarized as follows (18);

- 1) The larger the column side cover, the higher the anchorage strength. This effect was not observed when the ratio C_o/d_b (C_o is the column side cover to the center of the bar) is larger than six. (Fig. 14(a))
- 2) A narrow spacing of neighboring anchored bars decreases the anchorage strength. (Fig. 14(b))
- 3) As the projected development length l_{dh} increases, the anchorage strength increases. However, the anchorage strength does not increase proportionally to l_{dh}/d_b when the ratio l_{dh}/j is kept constant. (where j is the internal lever arm in the beam critical section. (see Fig. 11) This effect becomes small when l_{dh} is larger than $16d_b$. (Fig. 14(c))
- 4) Transverse reinforcement improves anchorage strength. (Fig. 14(d))

Anchorage Failure due to Raking-Out of Concrete inside The Bent Portion

In the AIJ Design Guideline, it is required that the joint shear stress should be evaluated for the effective shear resisting area using projected development length l_{dh} not the whole column depth for exterior beam-column joints. However, in the case of short projected development length, such as a half of the column whole depth, the ultimate strength is smaller than the estimation in AIJ. The evaluation method for the anchorage failure due to side cover splitting cannot be applied to this failure mode. The effectiveness of longer l_{dh} on the joint capacities is widely recognized, but there exists a need for shorter l_{dh} to avoid the congestion of bars in corner columns or to simplify the precast concrete construction system.

Joh et al.(17, 21) investigated the effect of several parameters on the anchorage capacities failed in raking-out of concrete with the variation of concrete strength up to 80MPa. Figures 15(a) and (b) show the bar arrangement of the standard specimen and the observed typical crack pattern. Some of the research findings are as follows;

- 1) The contribution of concrete strength to the anchorage strength can be described by a linear function of square root of f_c . (Fig. 16(a))
- 2) The anchorage strength increases linearly with an increase of transverse reinforcement ratio. (Fig. 16(b))
- 3) Taking into account of these findings, the strength equation for 'raking-out type' anchorage failure was derived by Joh et al.(Fig.16(c))

34 Fujii, Noguchi and Morita

The roll of transverse reinforcement becomes important as the concrete strength increases.

In order to establish more rational bond and anchorage design method from low to high strength concrete consistently, further research efforts are required especially on the following issues:

- 1) Difference of bond failure mechanism among lap splice failures, anchorage failure of cut-off bars and splitting bond failure of continuous longitudinal bars in members,
- 2) Bond behavior of double layered bar arrangements, bundled bars, and compression bars,
- 3) Interactions of shear failure and bond and anchorage failure in beams, columns and in beam-column joints,
- 4) Additional deformation and energy dissipating behavior of members due to bond and anchorage degradation under reversed cyclic loadings

REFERENCES

- 1) Kaku, T., Yamada, M., Iizuka, S. and Zhang, J. : A proposal of Bond Strength Equation for R.C. Members Including High Strength Concrete Level, Proceedings of the International Conference on Bond in Concrete, Riga, Latvia, Sept. 1992
- 2) Ichinose, T. and Yokoo, S. : Bond Splitting Strength of RC Members using High Strength Concrete, Proceedings of the Japan Concrete Institute, Vol.14, No.2, 1992 (in Japanese)
- 3) Takeuchi M., Fujisawa M., Kaminosono T., Murata T. and Murakami H. : Bond Behaviors of R/C Members with High Strength Reinforcing Bars and High Strength Concrete, Summary Papers of AIJ Annual Meeting, 1989, pp.353-356 (in Japanese)
- 4) Azizinamini A., Stark M., Roller J.J. and Ghosh S. K. : Bond Performance of Reinforcing Bars Embedded in High-Strength Concrete, ACI Structural Journal, Vol.90 No.5, Sept.-Oct. 1993, pp.554-561
- 5) Olsen N. H. :Strength of Lapped Splices in High Strength Concrete, ACI SP121-10, High-Strength Concrete, Second International Symposium Edited by Hester W.T., 1990, pp.179-193
- 6) Morita, S. and Fujii, S. : Bond Capacity of Deformed Bars due to Splitting of Surrounding Concrete, Proceedings of the International Conference on Bond in Concrete, Paisley, Scotland, June 1982
- 7) Maeda, M., Otani, S. and Aoyama, H. : Proposal on Splitting Bond Strength in R.C. Members, Proceedings of Structural Engineering Symposium, AIJ, No.38-B, 1992 (in Japanese)
- 8) Orangun, C. O., Jirsa, J. O. and Breen, J. E. : A Reevaluation of Test Data on Development Length and Splices, Journal of ACI Proceedings, Vol.74, Mar. 1977
- 9) Kaku T., Hattori S., Matsuno K. and Matsui H. : Influence of Yield Stress of Transverse Reinforcement on the Bond Strength of Reinforced Concrete Members, Proceedings of the Japan Concrete Institute, Vol.16, No.2, 1994 (in Japanese)

- 10) Miyanaga H., Kato M. and Fujii S.: Predicability of the Proposed Bond Splitting Strength Equation for Reinforced Concrete Members before Flexural Yielding, Summary Papers of AIJ Annual Meeting, 1994, pp.649-650 (in Japanese)
- 11) Morita S., Fujii S., Murakami H. and Yamada T.: Bond decay of beam bars in joint region under load reversals, Proceedings of the 10th World Conference of Earthquake Engineering, 1992, pp.3157-3162
- 12) Kashiwazaki T., Kitayama K. and Noguchi H.: Experimental Studies on the Bond Performances of Beam Main Bars of R/C Interior Beam-Column Joints with Ultra High Strength Materials, Proceedings of Japan Concrete Institute, Vol.14, No.2, 1992, pp.397-400 (in Japanese)
- 13) Kitayama K., Lee S., Otani S., and Aoyama H.: Behavior of high strength R/C beam-column joints, Proceedings of the 10th World Conference of Earthquake Engineering, 1992, pp.3156-3156
- 14) Hayashi K., Teraoka M., Mollick A.A. and Kano Y.: Bond Properties of Main Reinforcing Bars and Restoring Force Characteristics in RC Interior Beam-Column Subassemblages Using High Strength Materials, Proceedings of Japan Concrete Institute, Vol.15, No.2, 1993, pp.583-588 (in Japanese)
- 15) Kawakami, S., Murakami, M. and Kubota, T.: Factors Which Affect on Anchorage in Exterior RC Joint with High Strength Concrete, Proceedings of the Japan Concrete Institute, Vol.12-2, 1990, (in Japanese)
- 16) Yamada, T., Kawakami, S., Fujii, S. and Morita, S.: Characteristic Behaviors of Bent Bar Anchorage in Exterior RC Joints with High Strength Concrete, Proceedings of the Japan Concrete Institute, Vol.12-2, 1990, (in Japanese)
- 17) Joh O., Goto Y. and Shibata T.: Anchorage Failure Modes and Performance of 90-Degree Hooked Beam Bars Anchored to R/C Beam-Column Joints, Proceedings of Japan Concrete Institute, Vol.15, No.2, 1993, pp.159-164 (in Japanese)
- 18) Fujii S., Morita S., Kawakami S. and Yamada T.: Re-evaluation of test data on 90 degree bent bar anchorage, Journal of Structural and Construction Engineering (Transaction of AIJ), No.429, Nov. 1991, pp.65-75
- 19) Pinc, R., L., Watkins, M., D. and Jirsa, J., O.: Strength of Hooked Bar Anchorage in Beam-Column Joint, CESRL Report, No.77-3, Univ. of Texas, Nov. 1977
- 20) Fujii, S., Kondo, G., Morita, S. and Goto, S.: A Study on Bent Bar Anchorage in Exterior Beam-Column Joint (Part 1, 2), Summary Papers of AIJ Annual Meeting, 1983, pp.1821-1824 (in Japanese)
- 21) Tera, M. et al.: Raking-out failure of 90-degree hooked bars in R/C beam-column joints (Part 1, Part 2), Summary Papers of AIJ Annual Meeting, 1994, pp.545-548 (in Japanese)
- 22) Murakami, M. and Kubota, T.: The Capacity of a Double Layer 90-deg. Hooked Bar Anchorage in External Beam-column Joints, Proceedings of Pacific Conference on Earthquake Engineering, New Zealand, Nov. 1991,

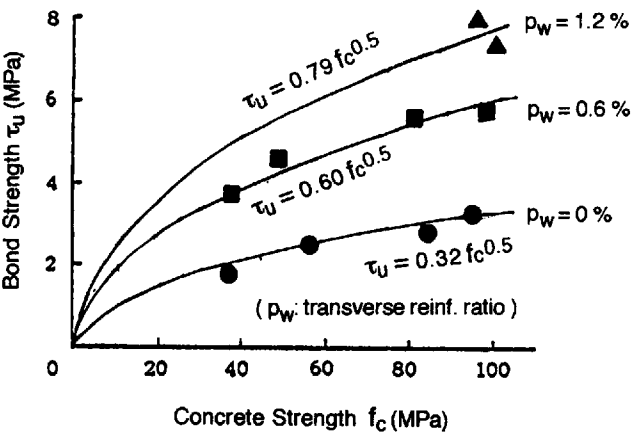


Fig. 1—Effect of concrete strength on splitting bond strength (1)

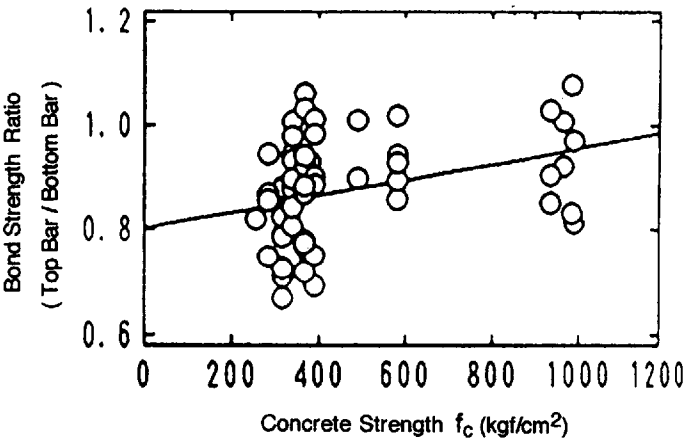


Fig.2—Concrete strength versus top bar effect (7)

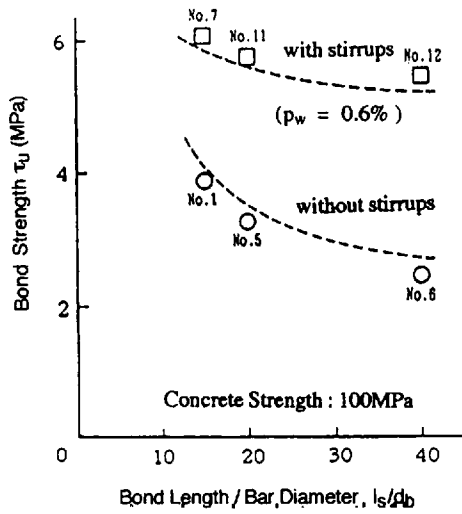


Fig. 3—Effect of development length on bond strength (1)

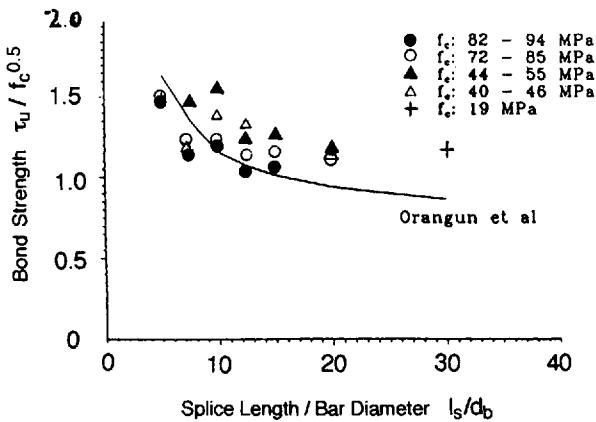


Fig. 4—Effect of splice length on lap splice strength (5)

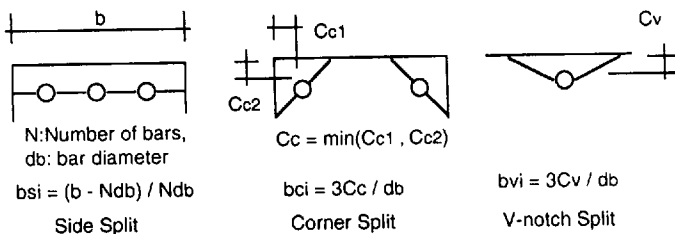
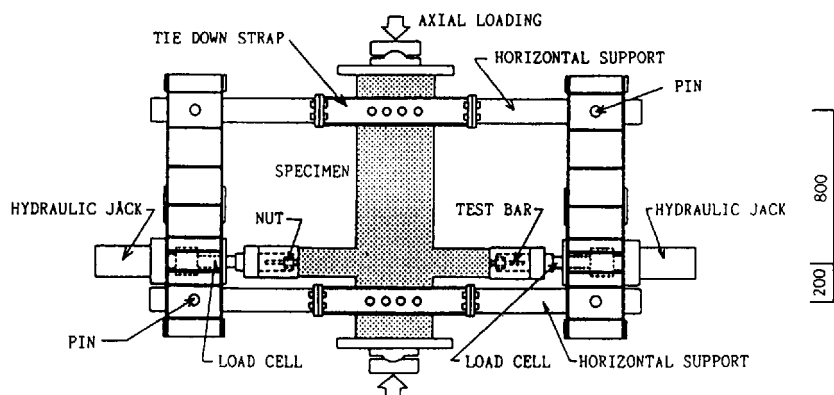
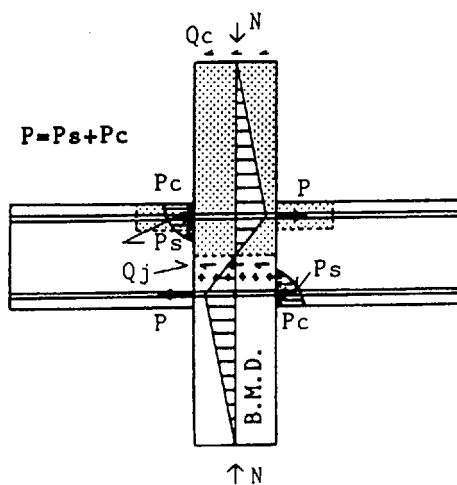


Fig. 5—Splitting failure patterns and definition of coefficient "bi"

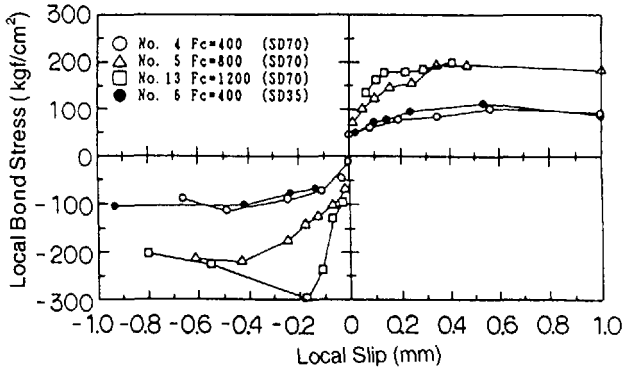


(a) Test Specimen and Loading Apparatus

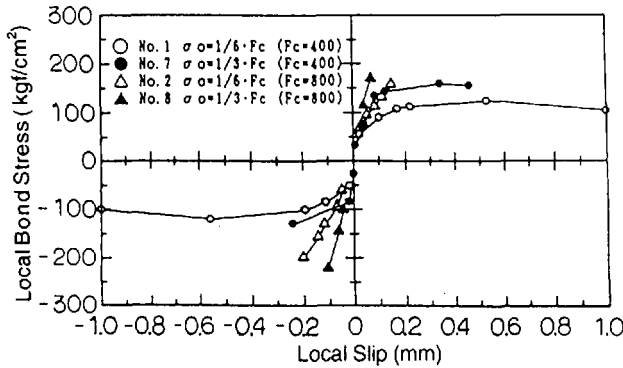


(b) Simulated Portion

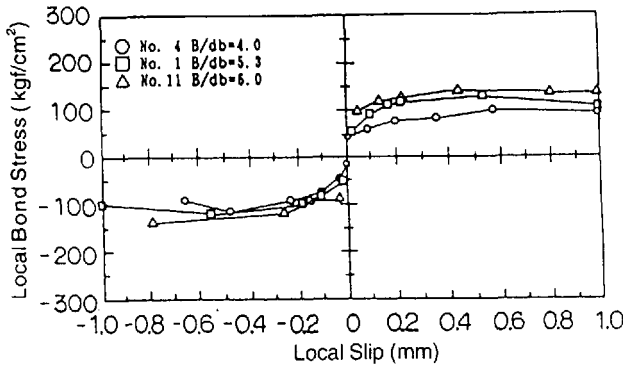
Fig. 6—Pull-push type bond tests simulating beam-column joint (11)



(a) Effect of Concrete Strength

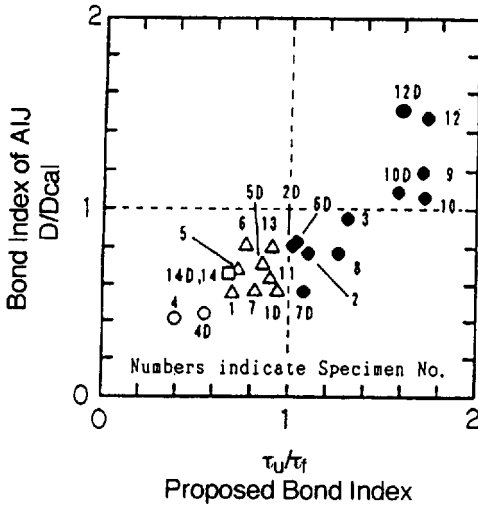


(b) Effect of Column Axial Stress Level



(c) Effect of Unit Column Width per Bar Diameter Ratio

Fig. 7—Envelopes of local bond-slip curves in joint core region (11)



Failure Mode

- : Bond failure before bar yielding
- △ : Bond failure within three cycles of reversed loading in post-yield range
- : No bond failure within above-mentioned loading cycles
- : Discontinue before bond failure due to shortage of loading capacity

Fig. 8—Grading of failure mode by bond index (11)

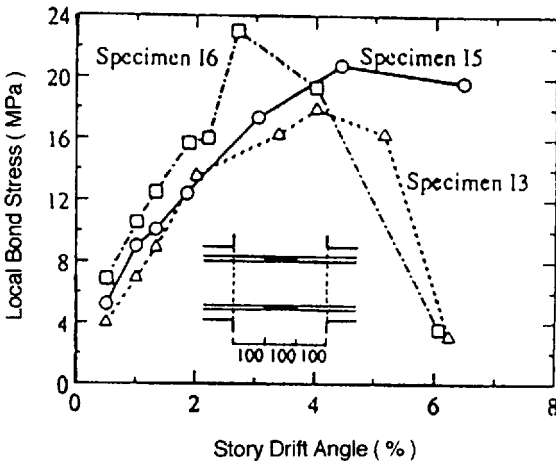


Fig. 9—Bond stress versus story drift relationships (13)

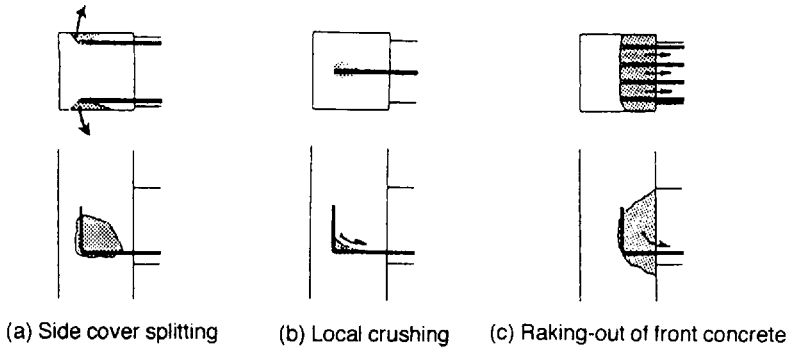


Fig. 10—Anchorage failure modes of 90°. Hooks (17)

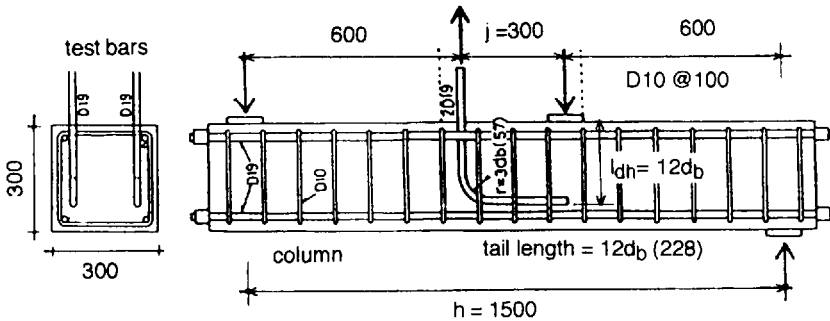


Fig. 11—Bent bar anchorage test specimen (16)

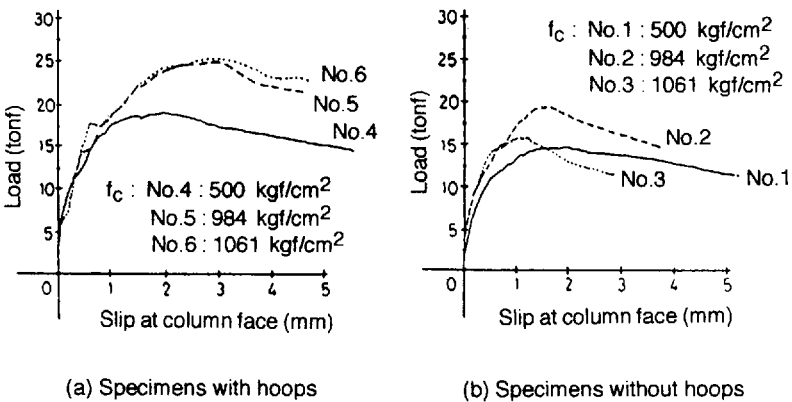


Fig. 12—Load-slip curves of bent-bar anchorages (16)

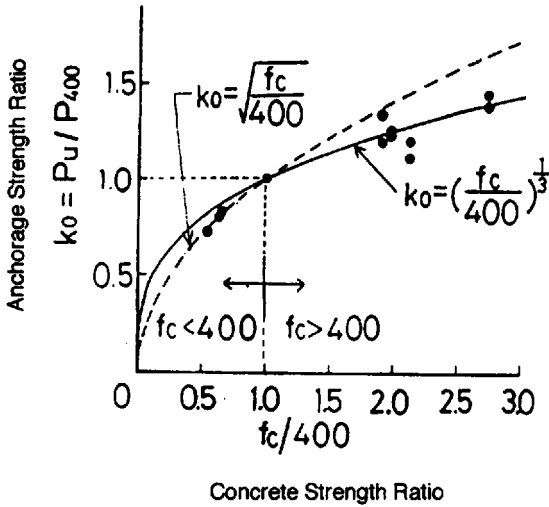
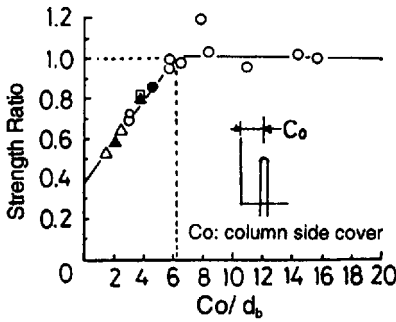
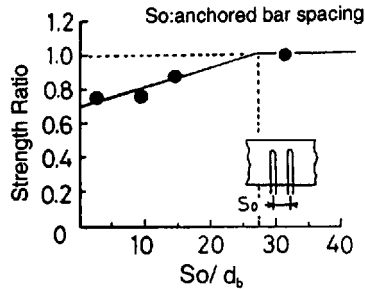


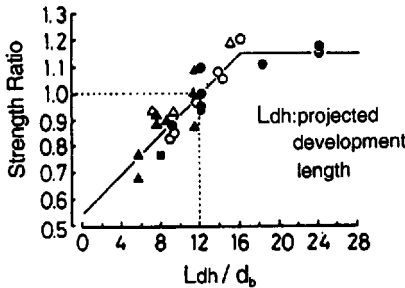
Fig. 13—Effect of concrete strength on anchorage strength (18)



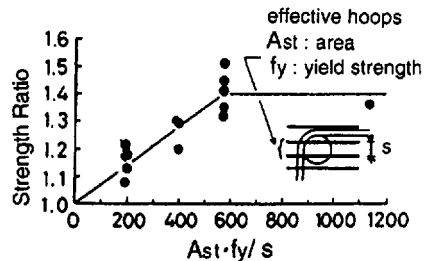
(a) Effect of Column Side Cover



(b) Effect of Beam Bar Spacing



(c) Effect of Projected Development Length



(d) Effect of Transverse Reinforcement

Fig. 14—Effect of several parameters of anchorage strength

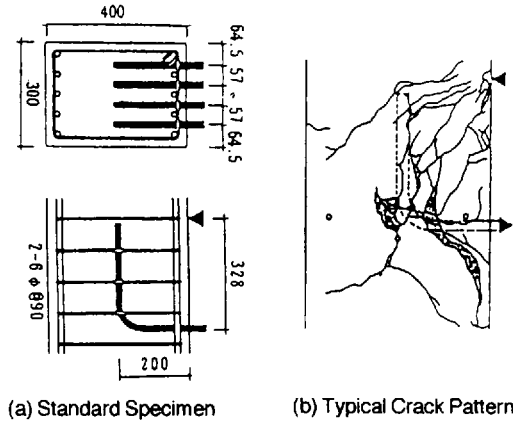
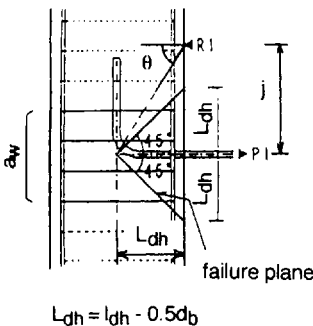
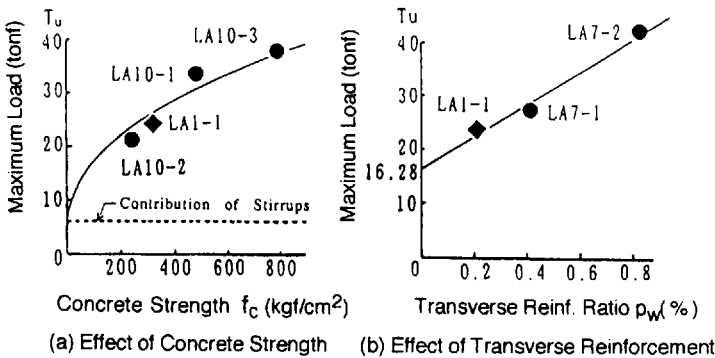


Fig. 15—Specimen and crack pattern for “ranking-out”-type anchorage failure (17)



Maximum anchorage force T_U

$$T_U = T_C + T_W$$

$$T_C = L_{dh} \cdot b_e \cdot \sqrt{f_c} \cdot (1 + 6.32 \frac{\alpha_0}{f_c}) \cdot \frac{1}{\sin \theta}$$

$$T_W = 0.7 \cdot a_w \cdot \sigma_{wy}$$

$$\sin \theta = \frac{j}{\sqrt{L_{dh}^2 + j^2}} \quad (\text{in kgf/cm}^2)$$

$$b_e = B_c - n d_b$$

(B_c : column width, n : number of beam bars)

α_0 : column axial stress,

a_w : total area of hoops crossing failure planes,

σ_{wy} : yield strength of hoops

(c) Strength Equation for 'Raking-out Type' Anchorage Failure

Fig. 16—Test results and strength equation for “ranking-out”-type anchorage failure (17)

SP 176-3

Bond and Anchorage of Reinforcement in Concrete—New Zealand Code Provisions

by D. Bull

Synopsis: This paper presents an overview of the provisions for anchorage and development of reinforcement in concrete for New Zealand concrete design code : NZS 3101:1995 (1). These provisions take into account the nature of high strength concrete (compressive strength $f'_c > 55$ MPa (8000 psi)) and the expected performance under seismic loading. The criteria for development lengths for straight reinforcement (with specific surface deformations) and those for bars terminated with hooks are largely based on recent studies of Sozen and Moehle and ACI 318: 1989. Simple, conservative equations are presented along with less conservative equations of more complexity.

Keywords: Anchorage (structural); bonding; compressive strength; high-strength concrete; reinforced concrete; seismic loading