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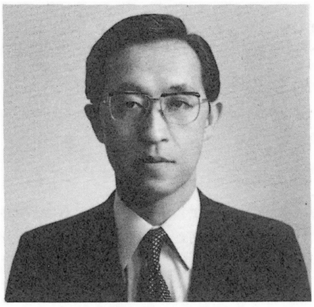
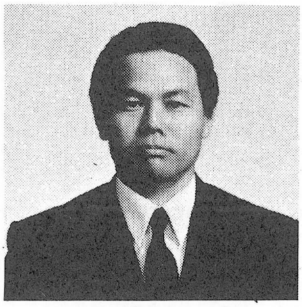
COMMENTS TO DRAFT OF MC-90 OF COMITE EURO-INTERNATIONAL DU BETON



Yoshio OZAKA

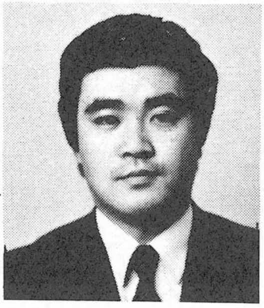
Hajime OKAMURA



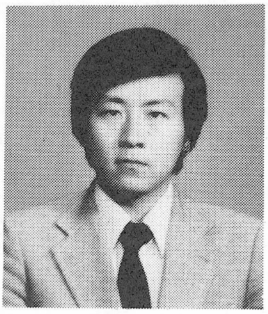
Tada-aki TANABE

Koji OTSUKA

Keitetsu ROKUGO



Junichiro NIWA



Koichi MAEKAWA

PREFACE

The civil engineering group of the Japanese members of various commisions, task groups. and general task groups of CEB wishes to present the following comments and new proposals regarding the draft of MC-90 now undergoing deliberation. Although these comments and proposals have been examined by the above mentioned Japanese civil engineering group, basically, the individual authors are responsible for the contents of the comments and proposals. These reports have been written giving essentials only for convenience in discussions of MC-90. For details, the individual papers published, or to be published, should be referred to.

Yoshio OZAKA is a professor of structural engineering at Tohoku University, Sendai, since '72. Recieved his Doctor of Engineering Degree in '72 from his almamater. Senior Engineer, Structures Design Office, Japanese National Railways since '55. MEMBERSHIPS ; Delegation of Japan to CEB, Commission "Seismic Design" of FIP, ACI, IABSE, JSCE, JCI, JPCEA. Interested in anti-seismic behavior and cracking property of concrete structure and structural safety problem.

Hajime OKAMURA is a professor of civil engineering at the University of Tokyo, Tokyo, Japan. His research interest is in durability design of reinforced concrete, divelopement of higher performance concrete and unmerical expression of reinforced concrete. He is a member of JSCE, JCI, IABSE and CEB, and a fellow of ACI.

Tada-aki Tanabe is a professor of Civil Engineering Department, Nagoya University. He is the chairman of JCI Committee on the Termal Stress of Massive Concrete Structures besides a member of various committees of both JCI and JSCE. His main research is directed to the dynamic failure mechanism of RC structures besides the thermal stress.

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Junichiro NIWA is an associate professor of civil engineering at Yamanashi University, Kofu, Japan. He received his Doctor of Engineering Degree from the University of Tokyo in 1983. His research interest is in strength, deformation and design of reinforced concrete mebers under shear and torsion. He is a member of JSCE, JCI and IABSE.

Koichi MAEKAWA is presently an associate professor of Civil Engineering at University of Tokyo, Tokyo, Japan. His research interest includes constitutive modeling for concrete and reinforced concrete, numerical simulation of RC structural response and workability of fresh concrete. He is a member of JSCE, JCI and AIJ.

Evalution of γ_M-Factor in Relation to Quality Assurance Level

Yoshio OZAKA (Tohoku University)
Tada-aki TANABE (Nagoya University)

1. GENERAL

The quality of Concrete structure reflects the construction process capability. It may be said that a-priori judgement on the level of quality assurance can be formed with a certain belief on a basis of the experimental knowledge at the stage of design calculation. Therefore γ_M-factor in structural design should be evaluated in relation to this a-priori judgement there-on.

2. SCORE OF TOTAL QUALITY ASSURANCE OF CONSTRUCTION

For making γ_M-factor reflect the level of total quality assurance, it is necessary to evaluate, at the stage of design calculation, the level of total quality assurance in some quantitative way. In order to do so, the evaluation is to be expressed in score number "T", which is called here as "Total Score of quality assurance".

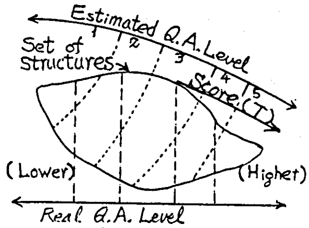


Fig 1 Normlization of Total Score T

Total Score T is to be expressed by integral number of 1 to 5 and be normalized so that 5 is corresponding to the most excellent state of quality assurance, 3 to the normal and 1 to the worst (see Fig 1). In order to evaluate the total quality assurance, it is necessary to express state of individual quality assurance measures. To Individual Score is given a weight (α i), which is corresponding to the contribution to the achievemnt of total quality assurance. Before having performed exhaustive investigations on the weight of individual scores, all individual weights are temporarily assumed being equal one another.

The probabilistic model of section resistance may be assumed of distribution. Distribution parameters may have some dispersion partly caused by many various chance factors and vary largely depending on the quality assurance level. Here we have probabilistic density function $\xi_0(R|m_R, Cv_R)$ or $\xi(R|m_R, p_R)$ of a section resistance (R), under the condition of values of parameters being given. Probabilistic density functions of these parameters m_R, Cv_R and p_R are expressed by $\rho(m_R)$, $\beta_0(Cv_R)$ and $\beta(p_R)$ respectively.

Fig 2 shows an example of histogram of concrete mean strength (m_c). Mean strength is assumed conforming to normal distribution. Curves of probabilistic density are classified in 5 categories in relation to quality assurance level, as shown in Fig 3.

Fig 5 shows the simulation curves of probabilitic density of coefficient of variation of concrete (Cv) corresponding to the state of process.

Fig 6 shows an example of distribution of fraction defective p in concrete lots obtained in construction works under ordinary control level. It may be admitted to assimilate distribution of fraction defective p to β - distribution, for convenience of calculation.

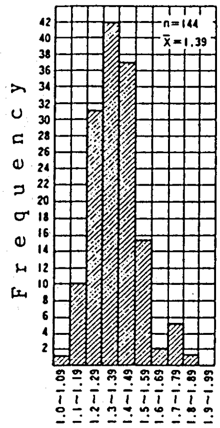


Fig 2 Observed Ratio of Mean Strength to Characteristic Strength of Concrete

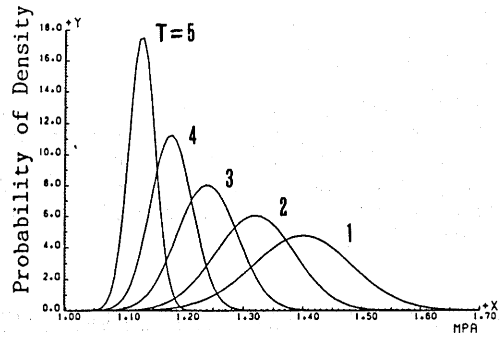


Fig 3 State of Process in Terms of Distribution of Ratio m_c/f_{ck}

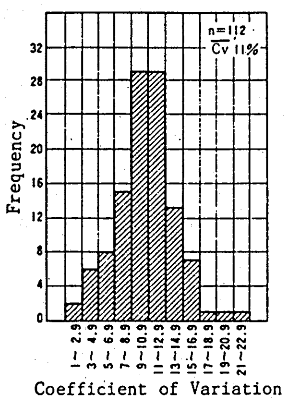


Fig 4 Obserbed Coefficients of Variation of Concrete Strength (Cv)

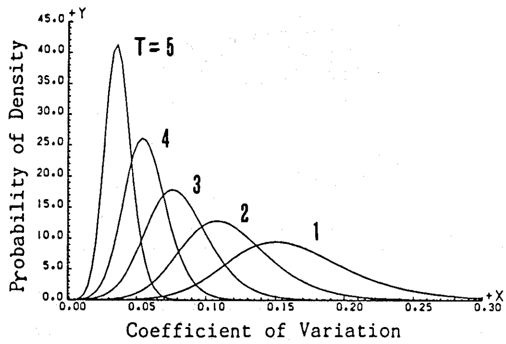


Fig 5 State of Process in Terms of Distribution of Cv (Field Concrete before Inspected)

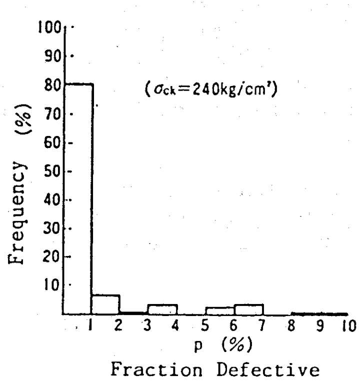


Fig 6 Fraction Defective p to Characteristic Strength of Concrete (f_{ck} = 24 MPa)

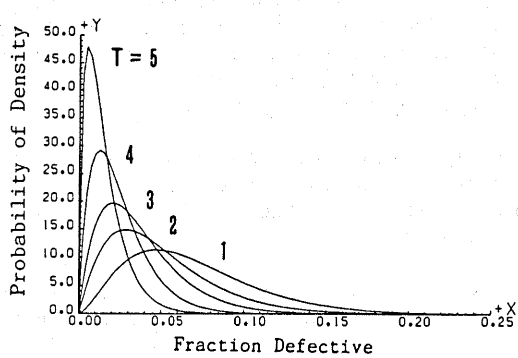


Fig 7 State of Process in Terms of Distribution of Fraction Defective of Concrete, Before Inspected

3. FAILURE PROBABILITY OF REINFORCED CONCRETE SECTION GOVERNED BY COMPRESSION

After this concrete lot has been accepted probability of the fraction being in the vicinity of p is $\beta(p)L(p)dp$, which can be the same as probability in a member section. Under the condition of mean value m_c and fraction defective p of concrete being given, the coefficient of variation Cv can be obtained.

As designers know that probability of mean strength in the vicinity m_c could be assumed $\rho(m_c)dm_c$, and also concrete with fraction defective probability $\beta(p)dp$ would be accepted by probability L(p), failure probability of the section (P_f) can be discribed as follows ;

$$P_f = \int_0^{\infty} \rho(m_c) \int_0^{1} \beta(p)L(p) \int_0^{\infty} \xi(R|m_R, p_R) \int_R^{\infty} \eta(S|m_S, \sigma_S)\,dS\,dR\,dp\,dm_c$$

$$= \int_0^{\infty} \rho(m_c) \int_0^{1} \beta_0(Cv)\ L_0(Cv) \int_0^{\infty} \xi_0(R|m_R, Cv_R) \int_R^{\infty} \eta(S|m_S, \sigma_S)\,dS\,dR\,dCv_R\,dm_c$$

where ;

- m_c : Mean strength of concrete lot in individual structure.
- $\rho(\cdot)$: Probabilistic density of mean strength of concrete lot in individual structure.
- R : Quantity corresponding to resisting force of a section.
- $\xi(\cdot)$, $\xi_0(\cdot)$: Probabilistic density function of resisting force of a section in terms of fraction defective and coefficient of variation respectively. Generally normal distribution may be used.
- Cv_R : Coefficient of variation of resisting force R.
- m_R : Mean of resisting force R.
- S : Quantity corresponding to acting force.
- $\eta(\cdot)$: Probabilistic density function of acting force.
- m_S : Mean of acting force.
- : Standard deviation of acting force.
- p_R : Fraction defective of resisting force, which is equal to that of concrete strength(p).
- $\beta_0(\cdot)$: Probabilistic density function of coefficient of variation.
- L(p), L_0(Cv) : OC-curve of sampling inspection in terms of fraction defective and coefficient of variation respectively.

Under the condition that failure probability of a section is preserved in the vicinity of 10^{-5} , characteristic safety factor must vary widely depending on the variation size of parameters of concrete strength. Table 1 shows simulation results obtained under the condition of coefficient of variation of action being 0.4.

Table 1 Characteristic Safety Factors ($P_f = 10^{-5}$)

NATIONAL CODE	Distribution Type of Acting Force	Total Score T: 5	4	3	2	1
FRANCE	Normal	1.49	1.49	1.56	1.81	2.49
	Log Normal	2.35	2.24	2.24	2.27	2.58
	Extreme-1	2.22	2.17	2.14	2.20	2.57
UNITED KINGDOM	Normal	1.50	1.53	1.71	2.99	1.56*
	Log Normal	2.37	2.36	2.35	3.35	1.66*
	Extreme-1	2.26	2.25	2.25	3.34	1.67*
JIS(a)	Normal	1.50	1.53	1.76	4.92	1.68*
	Log Normal	2.37	2.36	2.35	3.95	1.75*
	Extreme-1	2.23	2.21	2.28	3.95	1.72*

* : value corresponding to $P_f = 10^{-3}$

4. CONCLUDING REMARKS

1) It may be desirable to compose probabilistic models of distribution parameters, for estimating the values to be realized in construction work, at the stage of design calculation.

2) Simulation desity curves of distribution parameters used in this paper may be thought acceptable for the purpose of this study.

3) Simulation was carried out under some assumptions and idealizations. But the result suggests necessity of increasing γ_M-factor according to variation size of distribution parameters.

4) In the case of reduced inspection being used for acceptance, γ_M-factor should be increased corresponding to estimated level of quality assurance of construction ; By Japanese inspection, 10 to 20 percent increase of γ_M-factor may be necessary in the case of Total Score 3.

5) In the case of estimated quality assurance level being of Total Score 2 to 1, it is only possible to guarantee failure probability of 10^{-3} , if reduced inspection is used for acceptance as in Japan.

6) When inspection method is so tightened as in France, normal value of γ_M-factor can be used in the case of quality assurance level being of Total Score down to 2.

5. REFERENCES

1) CEB-CIB-FIP-RILEM. Recommended Principles for the Control of Quality and the Judgement of Acceptability of Concrete. BULLETIN D'INFORMATION No.110, 1975, May.

2) Quality Assurance for Building, Synthesis Report. BULLETIN D'INFORMATION No.184, 1988, May.

3) Yoshio OZAKA, Some Basic Investigations of Verification Method for Quality of Field Concrete. PROCEEDINGS OF JAPAN SOCIETY OF CIVIL ENGINEERS No.158, 1968, October.

4) Yoshio OZAKA, Fundamental Research on Quality Control of Cement Concrete. CONCRETE LIBRARY No.28, Japan Society of Civil Engineers, 1970, December.

Evalution of Crack-Width for The Purpose of Design of Concrete Structure

Yoshio OZAKA (Tohoku University)

Even though there might be no precise evidence that corrosion of reinforcement in concrete progresses more in relation to larger width of crack in actual concrete structure, some experimental results suggest that reinforcement would be apt to rust in the vicinty of large cracks in some severe environmental condition.

Fig 1 shows the distribution of crack-spacings observed on wet concrete prismatic specimen, which has square configuration of 6 6 cm and a transverse-lag deformed bar of 16 mm diameter arranged along the centroidal axis of the specimen. Crack-spacings were observed at the state of being loaded through the bar in tension up to 200 MPa. Fig 2 shows the crack-spacing distributions observed on actual reinforced concrete bridges(A) of sections shown in Fig 3. Crack-spacings were measured as distances between adjacent cracks, on the surface of concrete, along longitudinal reinforcements. Fig 4 shows the examples of distributions of crack-spacings which are observed on unloaded prismatic specimens exposed outdoors. The configurations and dimensions of these specimens are corresponding to those of specimens used in loading-in-tension tests of which results are shown in Fig 1.

The distributions of crack-spacing of wet concrete specimen obtained by laboratory tests have rather weak skewness and approximately conform to normality. The ratio of the maximum spacing to the minimum is about 4, rather larger than 2. which is the value estimated from homogeniously elastic model (Fig.1). On the other hand, the distribution in actual bridges have intensive skewness to smaller spacing range and do not conforms to normal but log-normal distribution. The histograms have the form very similar to those obtained by unloaded specimens under the influence of drying.

According to a classical theory of cracking, crack-widths are deeply correlated to spacings, and this feature of cracking has been verified by many laboratory tests using prismatic concrete specimens.

But in actual bridges. relations between crack-widths and spacings are quite different from those of prismatic concrete specimens. In scatter diagram of widths and -spacings observed in bridge A, systematic trend of correlation of the two does not appear to exist. Crack-spacings were grouped in class of 5 cm interval. and distribution of widths was investigated in every class of the spacings, in which characteristic width was defined as the value with 5% upper fractile of crack-width distribution. Characteristic widths were found to have approximate linear relation to spacings in the range smaller than some limit, say 35 cm in Bridge A, which is probably depending on geometrical and mechanical conditions of bars and effective zones of concrete.

Equation(1) is proposed for evalution of crack-width in concrete structure and Table 1 shows results of calculation.

$$\begin{aligned} Wmax &= Ka(5C + 0.5(C_{\phi}\ \ \phi) + \phi/10\ \rho_r)(\sigma s/Es + \varepsilon_{\varphi c}) + Kc \\ Ka &= 0.6 \text{ for deformed bar} \\ Kc &= 0.008 \sim 0.01 \\ &= 80 \sim 90 \times 10^{-6} \end{aligned} \qquad (1)$$

REFERENCES ;

1) Yoshio OZAKA. Koji OTSUKA, "Cracking Properties of Axially-Loaded RC

Tensile Specimens under Influence of Drying" Transactions of the Japan Concrete Institute Vol.7, 1985.

2) Yoshio OZAKA, Koji OTSUKA, Yoshinobu MATSUMOTO, "Tension Cracking Behavior of Reinforced Concrete Beam Bridges" Proceeding of Japan Society of Civil Engineers, No.390/Vol.8, 1988, February.

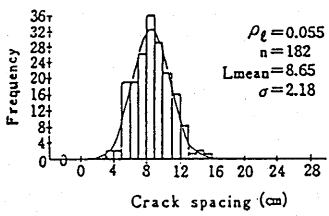
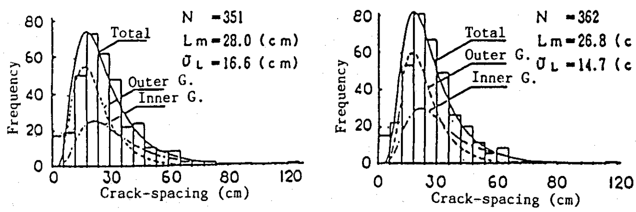


Fig. 1 Crack Spacings of Axially-Loaded Tensile Specimen



Two Years After Completion, Before placed in Service

Six Years in Service

Fig. 2 Bottom Surface of Girder (Bridge A)

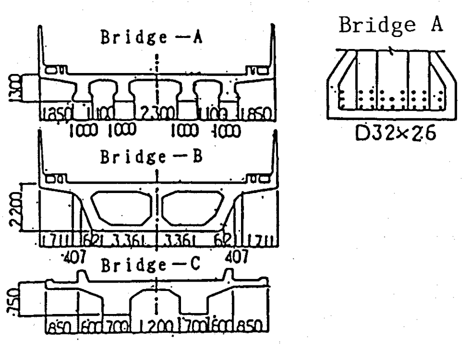


Fig. 3 Sections of Bridges

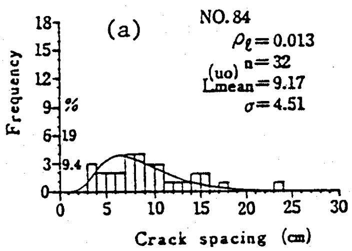


Fig. 4 Distributions of Transverse Cracks-spacings (outdoor exposure)

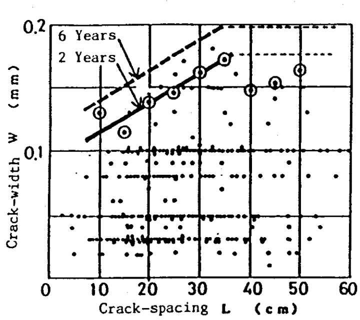


Fig. 5 Scatter Diagram of Crack-Spacing and -Width

Table 1 Max Crack-Width (Wmax)

Bridge		crack-width with-fractile of (mm)				crack-width calculated by			
		1%	3%	5%	7%	KAKUTA	CEB-FIP	JSCE	Proposed
A	bottom	0.25	0.20	0.18	0.16	0.30	0.10	0.20	0.22
	side	0.26	0.20	0.18	0.16				
B	bottom	0.22	0.18	0.16	0.15	0.28	0.05	0.25	0.27
	side	0.34	0.26	0.23	0.21				
C	bottom	0.13	0.10	0.08	0.07	—	0.08	0.15	0.14
	side	0.09	0.07	0.07	0.06				

Some Notes on Quality Assurance

Tada-aki TANABE (Nagoya University)
Yoshio OZAKA (Tohoku University)

The concept of quality assurance may be defined as such that the quality assurance is to assure the quality of structure for the predetermined period. According to this concept, the quality assurance is related to all the procedure starting from the definition of the structural objectives, determination of quality of structures which inevitably includes the quality assurance period, the structural design to accomplish the objective and the execution of construction works and so on.

The other way of defining quality assurance is as such that is similar to the definition in CEB Code Chapter 12. In 12.1, it said "In order that the properties of the completed structure be consistent with the requirements, the specifications and the assumptions made during the planning and the design, adequate quality measures shall be taken.

In this, no concept of quality assurance period is included. However, it is surely more reasonable to include the quality assurance period in the structural design of concrete structures and the future code is preferably oriented to that direction.

However, the question if the setting of assurance period for the reinforced concrete structure are practical and possible, might be raised.

To this question, the research report (1) by Japan Railway Company Higashi Nippon may be suggestive.

During the past 10 years, the numerous railway bridges and piers that are owned by JR Higashi Nippon have been reconstructed.

Simple statistical calculation showed that the expected life of those bridges and piers is about 127.5 years and the life of structures of 95% reliability is about 55 years.

The report suggests that with more data similar to this, it is possible and practical to set an assurance period for reinforced concrete structures.

It may be said that the quality assurance is now to guarantee the period in which structures will function safely and properly except for some anticipated and preacounted maintenance problems.

To achieve the purpose, the elements affecting the function of a structure through out its life should necessarily be analyzed including the initial designing stages. Very simply the reliability may be expressed as

$$R(t)=1-Fd-(1-Fd)Fc+A+M(t)$$

where

R(t) : reliability of a structure, the function of t
Fd : the probability of failure due to the misdesign
(1-Fd) : the probability that the design is properly performed so that the life period of a structure be assured
Fc+a+m: the probability of failure of a structure due to misconstruction, the excessive loading action, excessive environmental attack (chemical and physical) and mismaintenance. The function is also dependent on time, t.

However, it is easily seen that the procedure to define those function explicitly to such an extent that it may be used to practical design works is rather difficult at this moment.

With due regards to these, JSCE subcommittee recently drafted the code of durability design for reinforced concrete structures to assure the designed life of structures with out referring explicitly to above mentioned reliability calculation.

The reliability that the structure will constructed soundly and that the constructed structure will perform adequately during its life time, Rc(t), is expressed as

$$Rc(t)=1-Fc+A+M(t)$$

The assurance measures of Rc(t) just corresponds to the contents which is written in CEB chap 12.

In other words, the drafted contents of CEB quality assurance is a part of the assurance which is in accordance with the wider concept of the quality assurance.

The engineers did not have so far the clear conceptual image of assurance period for the structure they are engaged though they might implicitly maneuvered to make as good and as durabable structures as possible.

However, to achieve the construction of the rational and reliable structure, the clear setting of assurance period should be made and purposefully pursued by engineers and contractors and related professionals.

References

(1) H.Ichihara and M.Shimakura, "Estimation of Life Expectancy and Maintenance Cost of JR Line Concrete Bridges in Tokyo Area," Proceedings of JCI Symposium on Prediction and Design for Service Life of Concrete Structures, JCI-C13, Apr, 1988, pp.15-22.

To the Commission VI: DETAILING-Task Group VI/1: ANCHORAGE ZONES

Koji OTSUKA

1. Introduction

In the present draft of MC90, sub-chapter 6.10 is "VERIFICATION OF ANCHORAGES AND LAPS" and chapter 9 is "DETAILING". Clauses 6.10.1 to 6.10.8 are a revised version of the corresponding parts of chapter 17 in the CEB/FIP Model Code 78. Clauses 6.10.9 and 6.10.10 dealing with anchorages and tensile forces in the anchorage zone of prestressing tendon are newly proposed parts. Sub-chapter 9.1 is "General rules for detailing" and an amended version of the corresponding parts of chapter 17 in the CEB/FIP Model Code 78. Sub-chapter 9.2 is "Detailing of structural members" and for the clause 9.2.1 to 9.2.4, Commission VI has agreed to keep practically the text of MC78. However, some parts have already been or should be revised.

For the chapter 6.10 the fundamental philosophy to calculate the design anchorage length is the same as in the JSCE standard specification for design and construction of concrete structures though MC90 is more precise and complicated. Therefore it seems that this chapter is acceptable for Japan except some details of requirements which are different from those in the JSCE standard specification. The differences come from the different treatment for the bars of large diameter. In MC78 there were, and still in MC90 there are, some requirements to limit the use of large bars (diameter $\phi > 32$ mm). Concerning it, in JSCE standard specification, there are no limitation for using them up to 51 mm.

It may be correct to be conservative to use bars of large diameter. However, it is very important to use them to construct huge concrete structures economically.

At the TG VI/1 meeting of CEB in DUBROVNIK on August 1988, I explained about actual use of the large diameter bars in Japan and their treatment in Japanese specification. I also said that the special limitation for the use of large diameter bars in MC90 was not necessary at least up to 51 mm. After discussions, it was decided that I should propose a revision of that requirement at next task group meeting in Budapest on March 1989. This report will be used at the next task group meeting.

2. Clauses to be revised

The clauses which are thought to be revised are as follows.

(1) Clause 6.10.1: ULTIMATE BOND STRESS.

When bars of large diameter(diameter: $\phi > 32$ mm) are used. the design anchorage lengths must be increased according to the bar diameter.

(2) Clause 9.1.4: ADDITIONAL RULES FOR HIGH-BOND BARS OF LARGE DIAMETER.

For high-bond bars of diameter $\phi > 32$ mm, the rules below supplement those given in clause 9.1.1 and 9.1.2.

(3) Clause 9.1.4.1.1: Minimum depth of the elements

Bars of diameter $\phi > 32$ mm can be used only in elements of depth at least equal to 15 .

3. Propose of the revision

(1) For the clause 6.10.1: DESIGN BOND STRENGTH

The formula(6.10.1) to calculate the design bond stress fbd in the present draft is :

$$f_{bd} = \eta_1 \eta_2 \eta_3 \frac{f_{ckt\,0.05}}{\gamma_c} \qquad [6.10.1]$$

where

$f_{ctk0.05}$ values are given in table 1, clause 2.3.3.1

$\gamma_c = 1.5$

η_1 consider the type of reinforcement.

η_2 consider the position of the bar during concreting.

" Proposed parts "

η_3 consider the bar diameter :

$$\eta_3 = 1.0 \qquad \text{for } \phi \leq 50 \text{ mm}$$

$$\eta_3 = \frac{150-\phi}{100} \qquad \text{for } \phi > 50 \text{ mm.}$$

(2) For clause 9.1.4: ADDITIONAL RULES FOR HIGH-BOND BARS OF LARGE DIAMETER

For high-bond bars of diameter $\phi > 50$ mm, the rules below supplement those given in Clause 9.1.1 and 9.1.2.

(3) For clause 9.1.4.1.1 Minimum depth of the elements

Bars of diameter $\phi > 50$ mm can be used only in elements of depth at least equal to 15ϕ.

1988 Sept., Dubrovnik

Discussion about Tensile Stress-Crack Opening Diagram in Chapter 2 of CEB-FIP Model Code 1990

Keitetsu ROKUGO (Gifu University)

1. PHYSICAL MEANING OF THE AREA UNDER THE DIAGRAM

The area under the tensile stress-crack opening (strain softening) diagram represents the fracture energy G_F. For the bilinear diagram such as shown in Fig. 1:

$$(f_{ct} \cdot w_1 + s_1 \cdot w_c)/2 = G_F \qquad (1)$$

Therefore, the crack opening at break point w_1 can be given by the following simple equation instead of equation 2.4.12.

$$w_1 = (2G_F - s_1 \cdot w_c)/f_{ct} \qquad (2)$$

where, $s_1=0.15f_{ct}$ in Fig. 2.4.4 and values for w_c are given in the table in Chapter 2. Or the table for the complete crack opening w_c can be replaced with the following equation:

$$w_c = (2G_F - f_{ct} \cdot w_1)/s_1 \qquad (3)$$

2. PREFERABLE SHAPE OF THE STRESS-CRACK OPENING DIAGRAM

A parameter study has shown that several bilinear diagrams can simulate the real behavior equally well [1]. When the break point stress s_1 of the bilinear diagram was taken to be at 1/4 of the tensile strength f_{ct}, in order to obtain realistic simulation results, the crack opening at the break point w_1 and at the complete opening w_c were almost $0.75G_F/f_{ct}$ and $5.0G_F/f_{ct}$, respectively, irrespective of concrete composition, loading rate, and ligament length of specimens [1]. Based on these results, we have proposed a new bilinear model as shown in Fig. 1, where $s_1=f_{ct}/4$, $w_1=0.75G_F/f_{ct}$, and $w_c=5.0G_F/f_{ct}$ [2]. Since w_1 and w_c of the proposed model are the function of G_F, the model is well adaptable for the future change in the value of G_F.

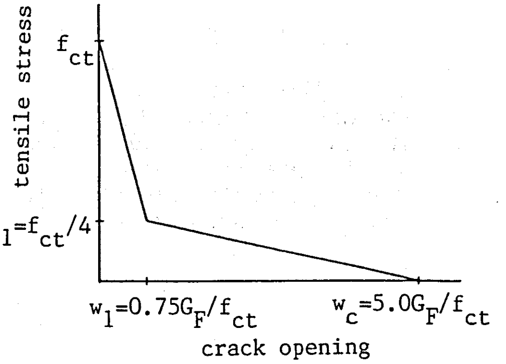


Fig. 1 Proposed bilinear model.

3. REFERENCES

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[2] Rokugo, K., Iwasa, M., Suzuki, T., and Koyanagi, W.: Testing methods to determine tensile strain softening curve and fracture energy of concrete, Proc. of Int. Workshop on Fracture Toughness and Fracture Energy, Sendai, Oct. 1988.

Abstract of the Recommendation for the Durability Design of JSCE

Junichiro NIWA (Yamanashi University)

1. INTRODUCTION

The Subcommittee on the durability design of concrete structures (Chairman, Prof. Hajime OKAMURA) was formed in 1988 by the Concrete Committee of the Japan Society of Civil Engineers (JSCE). The Subcommittee intended to make the Recommendation of the Durability Design and has published the Recommendation in this February. The Recommendation will be translated into English edition soon. As you can see the detail of contents of the Recommendation in the English edition, I would like to introduce the abstract and characteristics of the Recommendation briefly in this short contribution.

2. EXAMINATION OF THE DURABILITY

The proposed design method has to be carried out before the construction of the intended structure. Therefore, the prediction for the residual life or the estimation for the durability of existing structures is not a subject of this design method. The proposed design method is characterized by the quantitative provisions. The examination of the durability shall be made by confirming that T_p is not less than S_p for all of the parts of the structure.

$$T_p \geqq S_p \qquad (1)$$

T_p is the durability index which shall be determined in accordance with the details of material, construction and design of the intended structure and S_p is the environmental index which is defined according to the environmental condition and the desired period without maintenance.

3. ENVIRONMENTAL INDEX

The environmental index, S_p is defined as Eq.(2).

$$S_p = S_o + \Sigma(\Delta S_p) \qquad (2)$$

S_o is the fundamental value of the environmental index under the standard environmental condition and ΔS_p is the incremental value of the environmental index under the severe environmental condition, such as cold places, severe weather actions or marine environment and so on. When the desired period without maintenance is determined to be 50 years, S_o is defined as 100.

4. DURABILITY INDEX

The durability index, T_P is calculated by Eq.(3).

$$T_P = 50 + \Sigma T_P(I,J) \tag{3}$$

The fundamental value of the durability index is determined to be 50. $T_P(I,J)$ is the durability point which estimates quantitatively the influence of factors relating to the durability of the structure.

5. DURABILITY POINT

$T_P(I,J)$ are divided into eight main items. Each items are as follows.

I=1	materials for concrete
I=2	properties of fresh concrete and reinforcement
I=3	design crack width
I=4	shape of member, detail of reinforcement and drawings
I=5	placing of concrete
I=6	placing of reinforcement, formwork and falsework
I=7	prestressing
I=8	protection of concrete surface

Further, each items are divided into sub-items ($J \leq 6$). For example, the item on materials for concrete (I=1) is divided into the following sub-items.

J=1	kind of cement
J=2	content of absorbed water of aggregate
J=3	grading of aggregate
J=4	admixture

The points of each sub-items are defined quantitatively and the durability point, $T_P(I,J)$ can be obtained as the sum of the points of each sub-items.

6. CONCLUSION

The method of the proposed durability design is very similar to the examination of safety of a structure. For example, in the examination of the ultimate limit state for failure of a cross section, the design member force and the design capacity of a member cross section correspond to the environmental index and the durability index, respectively. As the result of the examination, if the requirement for durability (Eq.(1)) can not be satisfied, it is necessary to change the details of material, construction or design of the intended structure.

Discussion on the Bond Modeling for CEB Model Code
at the 26th Plenary Session in Dovrovnik (1988)

Koichi MAEKAWA (University of Tokyo)
Hajime OKAMURA (University of Tokyo)

1. Introduction

This document is the record of discussion and contribution by the authors as Japanese participants regarding the clause of bond model for computing moment-curvature of sections and member deflections. The major discussing point was that the bond stress-slip relationship is not a general constitutive law, but CEB Model Code 1990 should be based on the bond stress-slip-strain relations as a generalized behavior model.

2. Background

The bond stress-slip relation in appearance is not exactly unique but highly affected by the location along a bar, boundary condition of embedded steel and a bar stiffness. We can see different bond stress-slip relationships in each location along a bar as shown in Fig.1(a), where a pull-out test with short embedment was carried out under the boundary condition where no strain of steel and finite slip is produced at the free end of a bar. The different boundary condition with no slip and finite straining in steel was generated by a axial tension test, and gives us different bond stress-slip relations as shown in Fig.1(b). In this case also, bond stress-slip relation is not uniquely obtained.

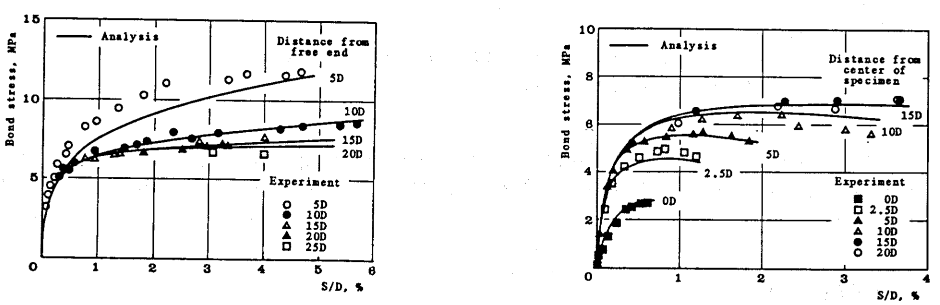

Fig.1 Local bond-slip relation, (a) short embedment, (b) axial tension
(Shima-Chou-Okamura, 1987)

On the other hand, the bond stress-slip relation becomes unique regardless of locations when a long embedment length, whose boundary condition at a free end is no slip and no strain in steel, is adopted. If the bond-slip relation would be a unique and general behavior model, this relation must be independent of the elasticity of a bar. But, it is reported that the obtained bond-slip relation in using an alminium bar with lower elastic stiffness is quite different from that obtained based on normal steel bars.

3. Proposal

We are against the bond-slip model and propose the "bond stress-slip-strain (of a bar) relationship" as a generalized behavior model. In case of the reinforcement embedded in massive concrete without splitting failure mode along a bar, the following equation is proposed by Shima and Okamura.

$$\tau = fc' \; f(s) \; g(\varepsilon) \qquad \text{(Mpa)}$$

$$f(s) = 0.73 \; (\ln(1 + 5s))^3, \quad g(\varepsilon) = 1/(1 + \varepsilon \times 10^5)$$

$$s = 1000 \; S/D$$

where, ε:strain, S:slip, D:bar diameter, fc':cylinder compressive strength.

The function "g" indicates the deterioration of bond performance due to bar straining which is considered indirectly to represent the fracturing of concrete around a bar at each location. The function f represents the maximum performance of bond under local slip with no local deterioration in concrete. By solving well-established bond equilibrium and compatibility equations with the bond-slip-strain model mentioned above, macroscopic behaviors are successfully computed as shown in Fig.1.

These equations are applicable to the bond performance after yielding of a bar. The bond stress-slip relations in post-yield range become very complex in appearance (See Fig.2), but the bond-slip-strain concept gives us fairly good prediction without any modification of the proposed equation as shown in Fig.2.

Since the relationship between slip and strain becomes unique provided that the infinitely long embedment of a bar is assumed, we have resultant unique bond stress-slip relation for the special case in which the parameter of strain does not appear explicitly as,

$$\tau = fc' \; f(s) \; g(\varepsilon(s)) = \tau(s)$$

If the long embedment length is assumed in design, the above equation which derives from the general governing model is proposed.

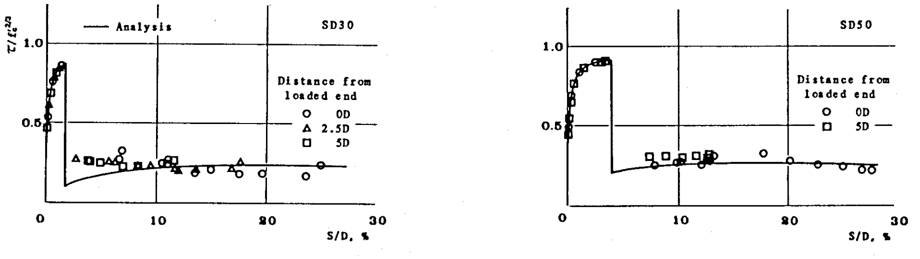

Fig.2 Local bond-slip relation after yielding of a bar
(Shima-Chou-Okamura, 1987)

4. Remarks

The bond-slip relation without bar strain as a parameter might be useful from a view point of the practical design, but its limited applicability should be recognized. The authors emphasize the importance of bar strain in the bond model as a "general behavior model".

Main Reference

Shima,H., Chou,L.L. and Okamura,H. : "Micro and Macro Models for Bond in Reinforced Concrete", Journal of the Faculty of Engineering, the University of Tokyo (B), Vol.XXXIX, No.2. (1987)