Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 51 (1986)

Rubrik: Session D: Construction and inspection

SESSION D

Construction and Inspection

Leere Seite
Blank page
Page vide

Quality Assurance – The Japanese Way

Pratique japonaise de l'assurance de la qualité

Zur Art und Weise der Qualitätssicherung in Japan

Ken UMEDA
Senior Managing Director
Kajima Corporation
Tokyo, Japan

Ken Umeda, born in 1921, a member of Architectural Institute of Japan, received his engineering degree from Tokyo University in 1944. He has been with Kajima Corp. since 1947 and is currently responsible for the nuclear power related business as well as the international operations of the firm.

SUMMARY

This paper describes some characteristics of quality assurance activities in Japan and the outline of an example of the company-wide quality assurance activities of a general contractor. It deals with the background of quality assurance activities undertaken by Japanese construction contractors, and highlights some characteristics of quality control in Japan. It shows quality assurance practices among general contractors. Some tasks to be tackled by Japanese contractors in the future are mentioned.

RÉSUMÉ

La contribution décrit quelques caractéristiques des activités de l'assurance de la qualité au Japon, d'une façon générale et dans le cas d'une grande entreprise de construction. Il traite de l'origine des activités d'assurance de la qualité au Japon et souligne quelques caractéristiques du contrôle de la qualité dans ce pays. Il donne quelques exemples d'assurance de la qualité dans les entreprises générales de construction. Quelques futures tâches à résoudre par les entrepreneurs japonais sont mentionnées.

ZUSAMMENFASSUNG

Der Beitrag beschreibt einige Merkmale der in Japan praktizierten Qualitätssicherung und erläutert diese am Beispiel einer weltweit tätigen Bauunternehmung. Im ersten Teil werden die Grundlagen beschrieben, der zweite beleuchtet einige allgemeine Merkmale japanischer Qualitäts-Kontrolle, während der dritte sich mit der Übertragung dieser Grundlagen auf Bauunternehmungen befasst. Der letzte Abschnitt beschreibt in aller Kürze die von den Japanischen Unternehmungen in Zukunft zu lösenden Probleme.

1. INTRODUCTION

The quality control activities in Japan which originated in the steel and chemical industries have spread to assembly industries such as home appliances, automobiles and precision machinery. They are now further expanding into such industries based on orders from client as construction, and even to enterprises which are mainly engaged in sales and services.
In particular, worthy of special mention is the enthusiasm with which construction firms are now wrestling with quality control.
The reasons for such enthusiasm - so much so that quality control might be said to have become a boom - could be the following.

a. The fact became widely known that while many Japanese manufacturers acquired the ability to provide high-quality but cheap products through productivity improvement, the reason behind such success was the great efforts spent for quality control.
b. The oil crises which have occurred twice since 1974 exerted marked influence on Japanese enterprises.

Before the oil crises, the Japanese construction industry, which had a substantial market domestically, did not have to worry about quality control to the same extent as did the manufacturing industry. After the oil crises however, many such enterprises came to recognize, as a natural matter of course, that they had to do something to upgrade their own competitive power through rationalization of
management.
Given such circumstances, the writer would like to discuss some of the characteristics of quality control efforts now being made by Japanese construction firms, and in particular, the Quality Assurance activities which form the core of such efforts.

2. QUALITY ASSURANCE (QA) ACTIVITIES IN JAPAN

2.1 Meaning of the Word "Quality"

A notable characteristic of QA activities in Japanese enterprises is the interpretation of the word "quality". That is, they have pursued "quality which can satisfy the requirements of customers," and extended efforts for its realization.

It was W.A. Shewhart who analyzed the meaning of quality in an industrial field. He pointed out that quality has both an objective/physical side and a subjective side, and that the goodness of a thing is a function of its subjective perception.

Subsequently, however, researchers, including Shewhart himself, carried out studies on statistical quality control as a means of achieving high quality, but on the basis of a definition of quality which was mainly based on objective perception (i.e., the producer's definition).

There were other definitions which sought to change this current of thinking:

- "A product that is maximally useful and has a market" (W.E. Deming)
- "Satisfaction of customer's needs" (J.M. Juran)
- "Products which can satisfy the requirements of consumers" (Kaoru Ishikawa) (i.e., the user's definition)

The latter definitions are said to be effective in developing new markets for new products.

2.2 Company-Wide QA Activities

Quality assurance activities in Japan are not something carried out only by quality control departments. Nor are they limited in manufacturing departments. They are company-wide activities.

In order to achieve "quality which can satisfy the requirements of customers" as mentioned in 2.1 above, what is needed are company-wide activities: accurate grasping of customer needs (marketing department), conversion of such needs into technological specifications for production (planning and designing departments), production based on design (production department), and maintenance and after-sale services (service department).

In order to systematically carry out these activities, the roles of various department and the method of implementation are determined, and consolidated into a System of Quality Assurance.

2.3 Quality Control (QC) Circle Activities

QC circle activities are contributing toward quality improvement.

Quality improvement is not the sole responsibility of QC specialists. QC circles, which are made up of production line workers, also share that responsibility. This is based on the belief that workers who are actually working in the workshops should know best about troubles on the production line, and that such knowledge should be utilized for quality improvement.

Part of the reason also is that having workers participate in such efforts will motivate them to work and provide a place for education and training.
Today, QC circle activities are playing an important role in improving quality, but the key point in all of this activity is to respect the initiative of the participants. We should understand that they originated as members' study groups, and as they gained more in capability, they were able to achieve good results in making quality improvements. For if we expect quick results in quality improvement from the outset, there is a possibility that activities may fail.

3. TYPICAL EXAMPLE IN STRUCTURAL ENGINEERING AND CONSTRUCTION

The writer will elaborate, using a practical example, on how the aforementioned "quality assurance activities in Japan" can actually be applied in the field of structural engineering and construction.

3.1 Typical Steps of Quality Assurance in Structural Engineering and Construction

More often than not, Japanese major contractors are possessed of in-house integrated capacities regarding R & D, design and engineering, and construction. Accordingly, when a project is awarded to such a major contractor, Q.A. will be implemented under single responsibility.
In one typical major contractor of Japan, Quality Assurance activities for a design-and-build type contract are classified into steps as described below:

Step 1.

(1) Preparation of "Quality Requirement Chart", by which qualities of the subject to design-and-build are developed from general requirements the Client to specific and concrete objectives of quality to be assured.
(2) Determination of design concept so as to best satisfy such quality targets as and when developed.

Step 2.

Preparation of "Quality Table (1)", by which the co-relation between each item of quality assurance and each element of design criteria is evaluated.

Step 3.

(1) Preparation of "Table of Technology Required", which lists all elements of proven technology and data necessary for achieving quality targets and whether they are available within the company's in-house resources or not, is carefully checked.

(2) Undertaking of "Failure Mode and Effect Analysis (FMEA)", by which all foreseeable causes of failure which may affect the required function of the subject of construction are evaluated.

(3) Combining the results of above (1) and (2), it is then determined whether or not further R-and-D and/or field test must be conducted by the Company's Research Center and/or his Construction Technology Dept.

Step 4.

If the execution of R-and-D and/or field tests, is required then methods and techniques of the scientific QC, are fully utilized such as:

- regression analysis
- analysis of variance
- design of experiment
- etc.

Step 5.

(1) Feeding back results of Step 4. to "Quality Table (1)".

(2) Determination of every basic element of design criteria.

(3) Execution of design, including structural and electro-mechanical design, and preparation of all technical documents including specifications, drawings and others.

Step 6.

(1) Preparation of "Quality Table (2)" by the Company's construction organization, in which are specifically described such matters as:
  - at which stage of construction process,
  - what points of quality must be checked,
  - in referrence to what standards,
  - by what ways and means,
  - under whose responsibility,
  - how records must be kept,
  - what remedial action must be taken, should it become necessary, and under whose responsibility

(2) Preparation of "QC Process Chart" by the Site Organization basing on "Quality Table (2)".

(3) Execution of construction in strict accordance with "QC Process Chart".

3.2 Example: Development of Large-sized PC Cryogenic Tank

As a result of the rapid increase of LPG and LNG importation after the 1974 oil crisis, there emerged a need for safe and economical storage of LPG and LNG at cryogenic condition. In line with such trend, above mentioned contractor undertook the planning, designing and construction of a large-sized PC cryogenic tank. How the above-stated steps of QA activities were applied in this case will be presented using slides or OHP at the Tokyo Symposium.

4. FUTURE ISSUES

The foregoing characteristics of QA activities are a result of accumulation of many years' activities in manufacturing companies. The writer believes that

even in the construction industry, methods that take the industry's distinctive characteristics into account should be studied and developed in order to establish QA activities that are both systematic and scientific.

a. Establishment of a method of evaluating the quality of public works and buildings, such as evaluation by users.
   (The term "quality" includes quality of work, costs, delivery dates, and safety)
b. Allocation of roles and responsibilities related to quality assurance, such as among client, engineer, and contractor.
c. Penetrating understanding and consideration of human aspects, such as study of human error.

Acceptable Size of Weld Defect in Steel Buildings

Tolérances dans les défauts de soudure d'ossatures métalliques

Zulässiges Ausmass von Schweissfehlern bei Bauwerken aus Stahl

Ben KATO
Prof. of Struct. Eng.
University of Tokyo
Tokyo, Japan

Ben Kato, born 1929, received his degree of Dr. of Engineering from the University of Tokyo, where he has served as lecturer and associate professor and now occupies the chair of steel structures and welding engineering.

Koji MORITA
Assoc. Prof. of Struct. Eng.
Chiba University
Chiba, Japan

Koji Morita, born 1941, received his degree of Dr. of Engineering from the University of Tokyo. He has served as research assistant at the University of Tokyo and associate professor at Tokyo Denki University.

Hirao FURUZAWA
Assistant of Welding Eng.
University of Tokyo
Tokyo, Japan

Hirao Furuzawa, born 1927, received his degree of Dr. of Engineering from the University of Tokyo.

Hagai SHIMOMURA
Lecturer of Struct. Eng.
Gifu Technical College
Gifu, Japan

Hagai Shimomura, born 1955, received his degree of Dr. of Engineering from the University of Tokyo.

SUMMARY

The important factors which reduce the maximum strength of welded joints involving defects are determined by factorial experiments, and the regression equations for the maximum strength of cross and butt joints are derived as a function of these factors. Finally, a method for estimating the acceptable size of weld defect is proposed for both joints.

RÉSUMÉ

L'expérience laisse apparaitre les principaux facteurs influençant la résistance maximale des connections soudées présentant quelques défauts. Une formule donnant la résistance extrême des assemblages bout à bout et perpendiculaires est établie à l'aide de ces facteurs. Une méthode d'estimation des tolérances acceptables pour les défauts de soudure est proposée pour de tels assemblages.

ZUSAMMENFASSUNG

Der vorliegende Bericht beschreibt die wesentlichen Faktoren, die die Festigkeit von fehlerhaften Schweissnähten herabsetzen und stützt sich dabei auf systematisch angelegte Versuche und eine statistische Auswertung der Resultate. Auf dieser Grundlage wird eine Methode zur Abschätzung des zulässigen Ausmasses von Schweissfehlern bei Stumpf- und Stirnnähten vorgeschlagen.

1. INTRODUCTION

The main design force on steel buildings is quasi-static one such as seismic force and wind force. In order to estimate the maximum strength and the deformation capacity of joints involving weld defects subjected to quasi-static load, various factors such as the kind, location and dimensions of weld defect, the type and dimensions of the joints,the loading condition etc. should be considered.

In this paper, the influential factors are selected statistically by the factorial experiments (1) and then the regression equations estimating the maximum strength of cross and butt joints are obtained as a function of these influential parameters using the test results hitherto reported in Japan (2,3,4). And the acceptable size of weld defect is discussed for both joints.

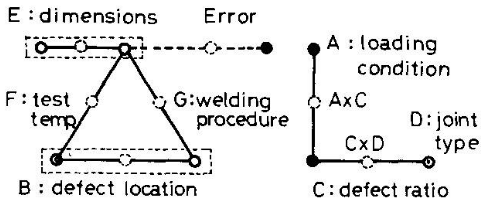
2. FACTORIAL EXPERIMENTS

2.1 Test Series I

2.1.1 Test Scheme

The experimental factors and the layout of the specimens into L 16 Orthogonal Array are shown in Fig. 1. The levels of the factors are as following;

- Loading Condition (A); The monotonic loading (Level 1) and the incremental cyclic reversal loading (Level 2) reflecting the severe earthquake condition, are adopted. The loading scheme of the latter is shown in Fig. 2.
- Defect Location (B); The defect kind is limited to lack-of-penetration defect and 4 levels shown in Fig. 1 are selected.
- Defect Ratio (C); The designed ratio of the nominal defective area to the gross sectional area is 3% (Level 1) and 6% (Level 2). The defect ratio is designed by varying the defect length, while the defect height is fixed to 20% of the thickness of the specimens.
- Joint Type (D); The cross joint (Level 1) and the butt joint (Level 2) are adopted as shown in Fig. 3.
- Dimensions of the Specimen (E); 4 levels are selected as shown in Fig. 1 considering the thickness of beam flange practically used.
- Test Temperature (F); Room temperature of about 293K (Level 1) and low temperature of 253K (Level 2) considering the cold region in Japan are adopted.
- Welding Procedure (G); CO_2 arc semi-automatic welding (Level 1) and manual arc welding (Level 2) which are usually used are adopted. Low alloy high strength steel which is designated as SM50 (specified minimum tensile strength by JIS is ${}_b\sigma_{ST} = 490\ N/mm^2$) is used for the base metal.



Levels of B and E (mm)

Level	1	2	3	4
B				
E (T)	12	19	25	32
E (W)	120	152	150	128

Fig. 1 Design of Experiment (Test Series I)

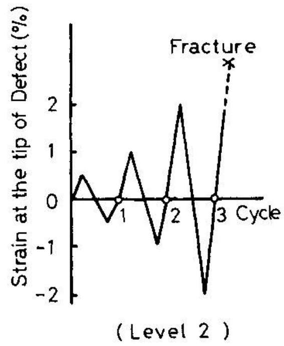


Fig. 2 Loading Condition

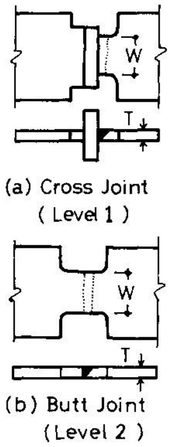


Fig. 3 Joint Type

2.1.2 Test Results

As the characteristic values for the maximum strength and deformation capacity of the joints, the following indexes are considered.

$$\frac{\sigma_B}{{}_b\sigma_B} = \frac{\text{nominal maximum stress of the specimen}}{\text{actual tensile strength of base metal at test temperature}}$$

$$\frac{{}_dEl_u}{{}_bEl} = \frac{\text{uniform percentage elongation at the maximum strength of the specimen}}{\text{percentage elongation of base metal at test temperature}}$$

As for the uniform percentage elongation of the specimens loaded by incremental cyclic reversal loading, the accumulated ones obtained by the method shown in Fig. 4 are adopted.

The results of the analysis of variance for the maximum strength index $\sigma_B/{}_b\sigma_B$ and the deformation capacity index ${}_dEl_u/{}_bEl$ are shown in Table 1 and the followings can be seen from this table.

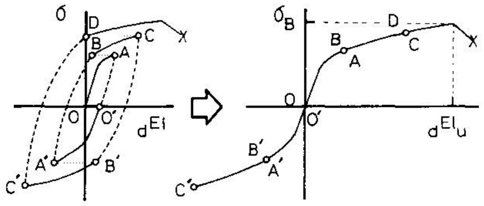


Fig. 4 Accumulating Method

- Maximum Strength
The contribution of the dimensions of the specimen is large and this factor is significant by 1% level. As seen in the result of point estimation shown in Fig. 5 a), the maximum strength shows a tendency to decrease as the dimensions of the specimen increases except for the case of Level 1 (T=12mm). This exception can be attributed to the fact that the specimens of Level 1 have buckled at compression side of loading. Although the defect location is significant by 5% level, a clear tendency cannot be seen. The maximum strength will be not influenced by the defect ratio of 3% to 6% range, though it can not be concluded definitely since the contribution ratio of the interaction is rather large.

	$\frac{\sigma_B}{{}_b\sigma_B}$	$\frac{{}_dEl_u}{{}_bEl}$
A	-	-
B	11.9*	5.3
C	5.6	-
D	4.8	22.7*
E	38.1**	23.4
F	-	-
G	-	-
A×C	10.0	-
C×D	21.0	12.1
Er.	8.6	36.5

* : Significant (5% Level)
** : Significant (1% Level)
Er.: Error Term

Table 1 Contribution Ratio (Test Series I)

The contribution of the loading condition, the test temperature and the welding procedure can be negligible.

- Deformation Capacity
The contribution of the dimensions of the specimen and the joint type is large, and the latter factor is significant by 5% level. The results of point estimation of these factors are shown in Fig. 5 b) and c). As for the dimensions of the specimen, the deformation capacity decreases largely at Level 4 (T=32mm), and the effect of the difference of the joint type is also large. But the contribution of other factors is almost negligible.

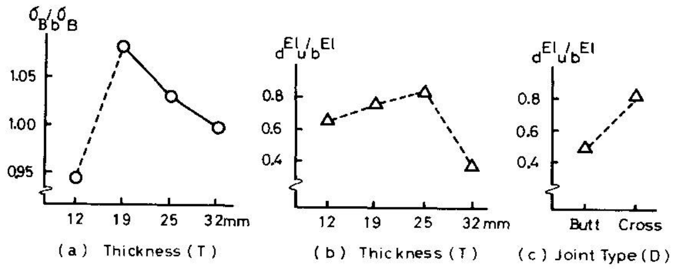


Fig. 5 Point Estimation for Influential Factors (Test Series I)

2.2 Test Series II

2.2.1 Test Scheme

The factors which are found to be necessary to research further from the results of the analysis of variance of Test Series I are as following;
- Defect Location (B); The defect kind is also limited to lack-of-penetration defect and 4 levels shown in Table 2 are selected laying emphasis on the cases of defects at the edge of plates.

- Defect Ratio (C); The levels in this series are 6% (Level 1) and 9% (Level 2). Reflecting on the results of Test Series I, the larger defect ratios are selected. The defect height is fixed to 5mm for T=19mm and 8mm for T=32mm.
- Joint Type (D); The cross joint (Level 1) and the butt joint (Level 2) are adopted again laying emphasis on the former.
- Dimensions of the Specimen(T,W); This factor is divided into two factors, i.e., the plate thickness(T) and the width(W) of the specimen. Two levels of these factors are T=19mm (Level 1), 32mm (Level 2) and W=100mm (Level 1), 300mm (level 2) respectively.
- Steel Grade (M); This factor is added in this series. The levels are SM50 (Level 1) and quenched and tempered high strength steel designated as SM58 whose $_b\sigma_{ST}$ is 568.4N/mm^2 (Level 2).

As for the factors whose contribution is small in Test Series I, the level is fixed to single, i.e., A=monotonic loading, F=room temperature, and, G= CO_2 arc semi-automatic welding. The layout of the specimens into L8 Orthogonal Array is shown in Table 2. This series consists of five subseries, but, the specimens are only 20 in number because another subseries can be constructed by changing half specimens of one subseries.

(a) Level of Factors

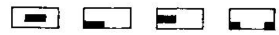
No.	B	C	D	M	T	W
1	1	1	1	1	1	1
2	4	2	1	2	1	1
3	1	1	1	2	1	2
4	4	2	1	1	1	2
5	1	2	1	1	2	1
6	4	1	1	2	2	1
7	1	2	1	2	2	2
8	4	1	1	1	2	2
9	4	2	2	1	1	1
10	1	1	2	1	1	2
11	4	1	2	1	2	1
12	1	2	2	1	2	2
13	2	1	1	1	1	1
14	2	1	1	2	1	2
15	2	2	1	1	2	1
16	2	2	1	2	2	2
17	3	2	1	2	1	1
18	3	2	1	1	1	2
19	3	1	1	2	2	1
20	3	1	1	1	2	2

(b) Layout of the Specimen

	Subseries (Sub.) (1)	(2)	(3)	(4)	(5)
Specimen No.	1	13	1	13	1
	2	2	17	17	9
	3	14	3	14	10
	4	4	18	18	4
	5	15	5	15	5
	6	6	19	19	11
	7	16	7	16	12
	8	8	20	20	8

(c) Levels of Factor B

1	2	3	4



Remarks

Sub.	(1) - (4)	(5)
Level	D = 1	M = 1

Table 2 Design of Experiment (Test Series II)

2.2.2 Test Results

The results of the analysis of variance for the maximum strength index $\sigma_B/_b\sigma_B$ and the deformation capacity index $_dEl_u/_bEl$ are shown in Table 3 and the followings can be observed from this table.

- Maximum Strength

The contribution of the defect location is large in Subseries (1) and (3), but, negligible in Subseries (2) and (4) in case of cross joints. As seen in the results of point estimation of these subseries shown in Fig. 6 a), the effect

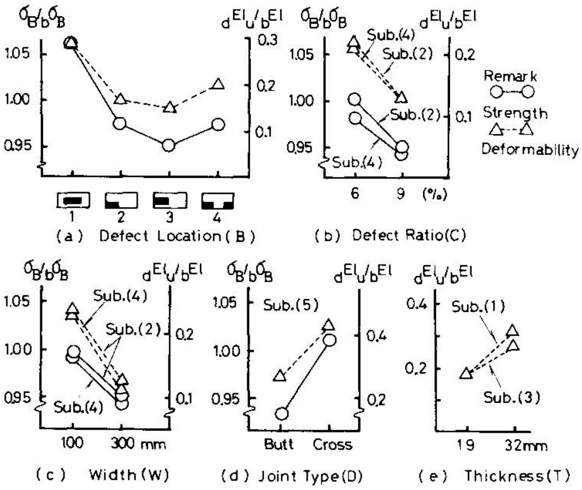


Fig. 6 Point Estimation for Influential Factors (Test Series II)

(a) Maximum Strength ($\sigma_B/_b\sigma_B$)

Sub.	(1)	(2)	(3)	(4)	Sub.	(5)
B	79.6**	-	92.1**	-	B	-
C	5.1	44.2*	-	21.5	C	-
M	-	-	-	-	D	32.2*
T	-	-	-	14.4	T	-
W	4.8	34.2*	-	21.5	W	46.4*
TxW	-	-	-	7.4	TxW	-
Er.	10.5	21.6	7.9	35.2	Er.	21.4

(b) Deformation Capacity ($_dEl_u/_bEl$)

Sub.	(1)	(2)	(3)	(4)	Sub.	(5)
B	21.4	-	45.9*	-	B	-
C	-	18.5	-	17.1	C	-
M	-	23.5	-	4.7	D	-
T	46.9*	-	14.7	-	T	16.4
W	-	27.0	12.0	44.9*	W	63.7*
TxW	-	-	-	2.3	TxW	-
Er.	31.7	31.0	27.4	31.0	Er.	19.9

Table 3 Contribution Ratio (Test Series II)

of the edge defect is larger than that of the center one, but the effect of the defect location in the direction of thickness can be negligible. In case of butt joints, the effect of the defect location is indicated to be negligible by the results of Subseries(5).

From the results of Subseries (2) and (4) which consist of the specimens of cross joints and their defect location is at edge, the followings can be observed. The contribution of the defect ratio and the width of the specimens is large, and the maximum strength of the joints decreases with the increase of the defect ratio and the width of the specimens as seen in Fig. 6 b) and c).

The contribution of the joint type is large in case of Subseries (5). As seen in the results of point estimation (Fig.6 d)), the deterioration of the maximum strength of cross joints is smaller than that of butt joints. As for the steel grade, the contribution seems to be negligible throughout all subseries.

- Deformation Capacity

In case of cross joints, the contribution of the defect location, the defect ratio and the width of the specimens is large, and their results of point estimation are shown in Fig. 6 a),b) and c). Although the contribution of the joint type is non significant, the deformation capacity of cross joints is larger than that of butt joints as seen in Fig. 6 d). From above results, the deformation capacity correlates with the maximum strength when the joints are fractured in ductile manner.

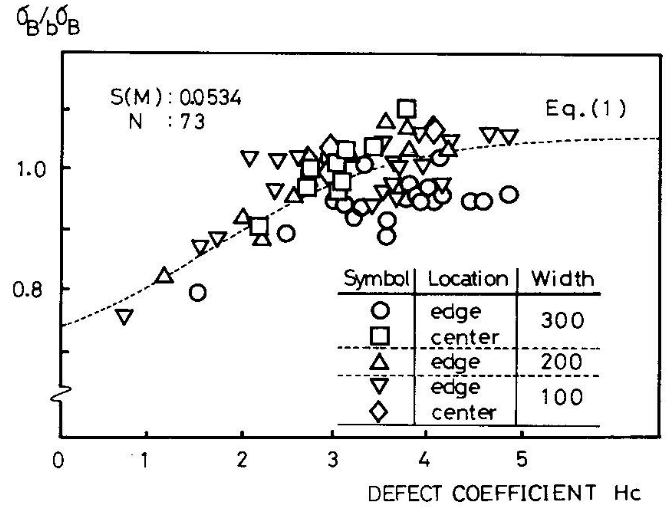


Fig. 7 $\sigma_B/{}_b\sigma_B$ versus H_c Relationship for Cross Joints

3. ESTIMATION OF THE ACCEPTABLE SIZE OF WELD DEFECT

3.1 Estimation of the Maximum Strength of the Joints Involving Weld Defect

3.1.1 The Maximum Strength of Cross Joints

- Regression Equation for the Maximum Strength of Cross Joints:

From the results of the experiments hitherto reported (2,3) and of the experiments in Chapter 2 whose welding procedure is CO_2 arc semi-automatic welding or manual arc welding, steel grade is SM50 or SM58, and, defect kind is lack-of-penetration, the regression equation for the maximum strength is obtained by the method of least squares as

$$M_c = \sigma_B/{}_b\sigma_B = (1.06\, e^{H_c} + 3.6)/(e^{H_c} + 5.4). \quad \ldots (1)$$

Where, $H_c = 1.3 \ln((W/l_s)^{0.7} \times (T/h_s)^{1.4})$ is the defect coefficient for cross joints, ls is the defect length measured from the fracture surface after testing, hs is the defect height measured. The ranges of the experiments are $0.1 \leqq l_s/W \leqq 0.76$ and $0.16 \leqq h_s/T \leqq 1.0$.

The relationship between the test results and the regression equation is shown in Fig. 7. As seen in this figure, the deterioration of the maximum strength of the joints whose width is W=300mm and defect location is edge is somewhat larger compared with that of other joints. This result coincides with the result of the factorial experiments.

- Other kind of defect and other welding procedure

a) Crack Defect

In the experiment(1), crack defect is induced in the weldment. The test results $\sigma_B/{}_b\sigma_B$ are shown in Fig. 8 by the marks **O** .

As seen in this figure, the deterioration of the maximum strength is larger compared with the case of lack-of-penetration defect. The crack defect of this experiment is hot crack induced naturally by welding the joint in high restraint as shown in Fig. 9 a). The procedure of making lack-of-penetration defect is as follows. After providing the special edge preparation which has a projection with specified size as seen in Fig. 9 b), groove welding is excuted.

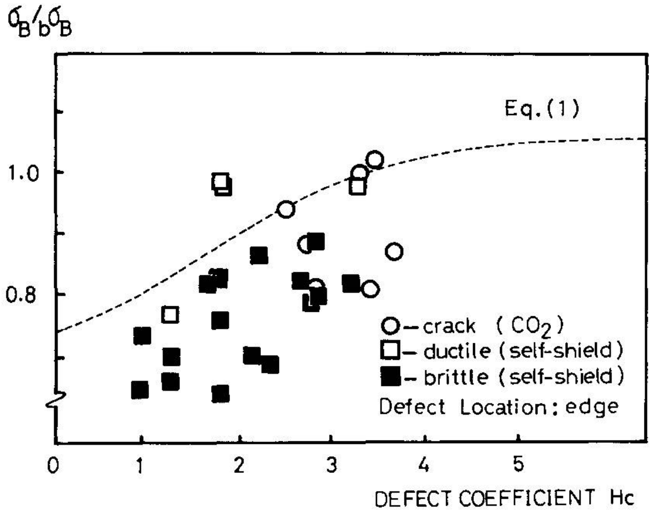


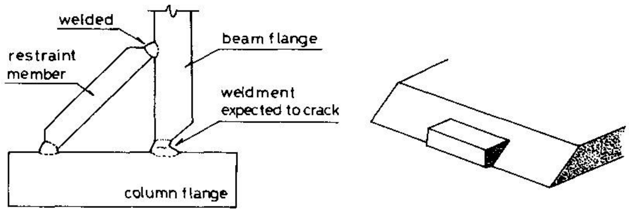
Fig. 8 Test Results of Cross Joints

b) Self-shielded Arc Semi-automatic Welding

The test results $\sigma_B/{}_b\sigma_B$ of the specimens of this welding procedure are shown in Fig. 8 by the marks □ and ■. The specimens marked with □ were fractured in ductile manner, while the specimens marked with ■ were fractured in brittle manner.

As seen in this figure, the deterioration of the maximum strength is remarkable compared with that of other welding procedures. This can be attributed to the poor toughness of weldment obtained by this welding procedure (3).

So, these cases should be regarded as out of scope.



(a) Crack Defect (b) Lack-of-Penetration Defect

Fig. 9 Method of Making Weld Defect

3.1.2 The Maximum Strength of Butt Joints

- Regression Equation for the Maximum Strength of Butt Joints: From the results of the experiments hitherto reported (4) whose welding procedure is CO_2 arc semi-automatic welding or manual arc welding, the regression equation for the maximum strength is obtained by the method of least squares as

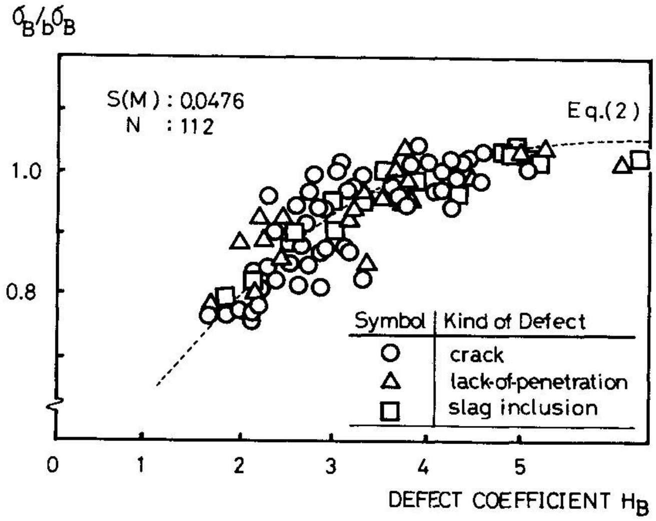


Fig.10 $\sigma_B/{}_b\sigma_B$ versus H_B Relationship for Butt Joints

$$M_B = \sigma_B / {}_b\sigma_B = (1.06\, e^{H_B} + 1.7)/(e^{H_B} + 4.6) . \quad \dots (2)$$

Where, $H_B = 1.5 \ln((W/l_s)^{1.0} \times (T/h_s)^{0.8})$ is the defect coefficient for butt joints. The ranges of the experiments are $0.1 \leqq l_s/W \leqq 0.6$ and $0.07 \leqq h_s/T \leqq 0.86$.

The relationship between the test results and the regression equation is shown in Fig. 10. As seen in this figure, the difference of the influence among the defect kinds can not be observed. This can be attributed to the method of making artificial defect (5). The method of making crack defect in these experiments differs from the method adopted in the specimens of cross joints. No difference can be seen between artificial crack and lack-of-penetration in the mechanical viewpoint.

3.2 Acceptable Size of Weld Defect

3.2.1 Acceptance Criteria of Weld Defect

The following criteria, i.e., Level A and Level B are considered in this paper.

Level A: $\sigma_B \geqq {}_b\sigma_y \quad \dots (3)$

Where, ${}_b\sigma_y$ is the actual yield stress of base metal. This step assures the condition that the welded joint is not to be fractured before the member connected yields in tension, and can be applicable to the joints of tension member such as the bracing members from the view point of seismic design.

Level B: $\sigma_B \geqq {}_b\sigma_{ST} \quad \dots (4)$

This step can be applicable to the joints with moment gradient such as the beam-to-column joints.

The acceptable size of the lack-of-penetration and slag inclusion defects is discussed in the followings when the welding procedure is limited to CO_2 arc semi-automatic welding or manual arc welding, and the steel grade is limited to SM50 whose ${}_b\sigma_{ST}$ is 490 N/mm².

3.2.2 Acceptable Size of Weld Defect for Level A Joints

Eq.(3) can be rewritten as $M = \sigma_B / {}_b\sigma_B \geqq {}_b\sigma_y / {}_b\sigma_B = Y$, where Y is the yield ratio of base metal. The mean value and standard deviation of Y for SM50 are $Y_m = 0.686$ and S(Y)=0.0455 from the statistical surveys (6). Based on the assumption that the values M and Y are the normal independent random variables, the next condition must be satisfied in order to satisfy Eq.(3) by 95% confidence limits.

$$M_m \geqq Y_m + 1.645\sqrt{(S(M))^2 + (S(Y))^2} \quad \dots (5)$$

	Level A	Level B
Cross Joint	$H_C \geqq 1.0$	$H_C \geqq 4.3$
Butt Joint	$H_B \geqq 2.0$	$H_B \geqq 4.4$

Table 4 Result of Analysis

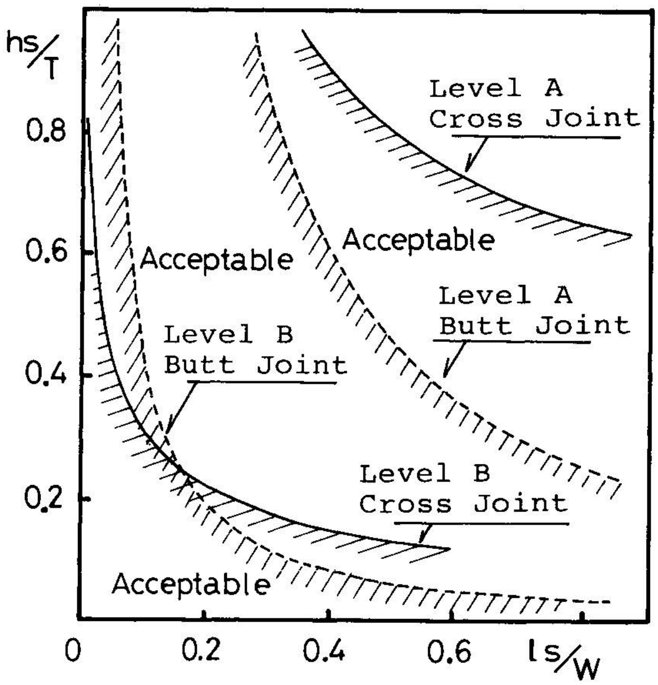


Fig.11 Acceptable Size of Weld Defect

Where, Mm=Mc of Eq.(1) for cross joints and Mm=M_B of Eq.(2) for butt joints, and S(M) is the standard deviation of M(see, Fig. 7, 10).

The results of the analysis are shown in Table 4. The acceptable limits thus obtained are shown in Fig. 11 in terms of l_s/W and h_s/T .

3.2.3 Acceptable Size of Weld Defect for Level B Joints

The next condition must be satisfied in order to satisfy Eq.(4) by 95% confidence limits.

$$M_m \geqq ({}_b\sigma_{ST}/{}_b\sigma_B)_m + 1.645\sqrt{(S(M))^2 + (S({}_b\sigma_{ST}/{}_b\sigma_B))^2} \quad \ldots (6)$$

Where, $({}_b\sigma_{ST}/{}_b\sigma_B)_m$ and $S({}_b\sigma_{ST}/{}_b\sigma_B)$ are the mean value and the standard deviation of the values $({}_b\sigma_{ST}/{}_b\sigma_B)$. $({}_b\sigma_{ST}/{}_b\sigma_B)_m=0.9317$ and $S({}_b\sigma_{ST}/{}_b\sigma_B)=0.02957$ are obtained from the statistical surveys on SM50 (16mm $\leqq$ T $\leqq$ 50mm, N=74).
The results of the analysis and the acceptable size of weld defect are also shown in Table 4 and Fig. 11.

4. CONCLUSIONS

The conclusions obtained in this paper are as following;
- Through the factorial experiments, it was found that the important factors which give influence to the strength of welded joints are the geometric shapes of the joints and the size of weld defect, and also it was found the effects of such factors as the welding procedure, the steel grade, the loading condition, and the test temperature are insignificant.
- As the regression equations estimating the maximum strength of the joints involving weld defect, Eq.(1) for cross joints and Eq.(2) for butt joints are obtained respectively on the basis of the test results hitherto reported.
- A method to evaluate the acceptable sizes of lack-of-penetration and slag inclusion defects are presented for the joints subject to tension (Level A) and for the beam-to-column joints (Level B), and it is illustrated in Fig. 11 for cross and butt joints with SM50 grade steel.

REFERENCES

1. KATO B., MORITA K., FURUZAWA H., The Influence of Weld Defects on the Quasi-Static Strength and Deformation Capacity of Welded Joints, Journal of Structural and Construction Engineering (Transactions of AIJ), April 1985.

2. FUJIMOTO M., IZUMI M. et.al., Deformation Capacity of Defective Joints (Part 1,2,3), Transactions of AIJ, February 1980, May 1981, June 1983.

3. SATO K., TOYODA M., OKAMOTO S., General Yielding Behavior of Welded Components in Steel Framed Structure (Part 1,2), JOURNAL OF THE JAPAN WELDING SOCIETY, January 1981, July 1981.

4. KATO B., FURUZAWA H., MORITA K., The Estimation of Weld Defects with the Ultrasonic Angle Beam Testing (II), Transactions of AIJ, June 1976.

5. KATO B., MORITA K., FURUZAWA H., Estimation of Weld Defects Through Ultrasonic Testing, WELDING JOURNAL, NOVEMBER, 1976.

6. AOKI H., MASUDA K., Statistical Investigation on Mechanical Properties of Structural Steel Based on Coupon Tests, Journal of Structural and Construction Engineering (Transactions of AIJ), December, 1985.

Quality Assurance of Welded Structures

Assurance de la qualité des constructions soudées

Qualitätssicherung bei geschweissten Konstruktionen

Janusz W. MURZEWSKI
Prof. Dr.
Politechnika Krakowska
Kraków, Poland

Janusz Murzewski, born 1928, received his civil engineering degree at Politechnika Krakowska, 1955. In 1959/60 he was fellow of the Applied Mathematics Division at Brown University, USA. 1963–70 – chairman of Department of Mathematics, since 1970 – head of Dep. of Metal Structures in Kraków. He is author of the book «Safety of Building Structures» and more than 200 papers.

SUMMARY

According to new concepts, the fraction of welds which should be subjected to radiographic and/or ultrasonic control depends on the nature of the loads and consequences of failure. Three classes of structures are defined and not every welding workshop is suitable for carrying out each class of weld. Classification of safety has been discussed by some international organizations. It may give differences in design strengths depending on consequences of failure and coefficients of variation of both loads and material properties.

RÉSUMÉ

Selon de nouveaux concepts, les soudures qui doivent être soumises au contrôle radiographique ou ultrasonique dépendent de la nature des charges et des conséquences d'une éventuelle rupture. On définit trois classes de structures; tous les postes de soudure ne seront pas autorisés pour chaque classe. La classification de la sécurité est discutée par des organisations internationales. Cela conduit à des résistances de calcul différentes en fonction des conséquences de rupture et des coefficients de variation des charges et propriétés des matériaux.

ZUSAMMENFASSUNG

Der Umfang der mittels Röntgenstrahlen oder Ultraschall zu prüfenden Schweissnähte sollte nach neueren Vorstellungen von der Belastungsart und den Konsequenzen eines allfälligen Versagens abhängig gemacht werden. Drei Bauwerksklassen werden definiert. Nicht jede Werkstatt darf für alle Klassen Schweissarbeiten ausführen. Die Klassifizierung wurde von verschiedenen internationalen Gremien diskutiert und es ist vorgesehen, die Rechenwerte der Festigkeiten von den Folgen eines Versagens, dem Variationskoeffizienten der Beanspruchung sowie den Baustoffeigenschaften abhängig zu machen.

1. INTRODUCTION

Structural failure are often originated by rupture of welded connections. Butt welds in tension condition happen to be the weak points, the most frequently. Mechanical properties of the weld material may be quite good in average, even better than the properties of original steel members but there is a little more probability of random defects in welds although approved welders may guarantee their quality. Formerly, design codes for welded structures recommended a reduction of stresses in welds under tension ~20%. The reduction used to be achieved by additional plates or other elements which could increase the cross-section at a joint. Such usage has been abandoned, because additional elements give stress concentration points and they do not make connections any safer. Nowadays, the whole cross-section of a member shall be rather overdimensioned if the welded joints are just at the point where the stress would reach the design strength. However, quality control of welds becomes more and more appreciated in the last decades thanks to developments in radiographic and ultrasonic techniques. Therefore standard specifications admit no strength reduction of welds provided that the modern testing methods are applied. Recently, a proposition is under discussion, to get rid of any stress analysis of butt welds but to introduce an intensive quality control by means of physical methods. However, it is an economic question, because costs of the entire weld testing may be excessive. That is why a classification of welded structures and quality control differentiation are suggested [1]. Draft specifications of the Polish Committee for Standardization PKNMiJ are given under discussion recently [2]. Similar recommendations have been already elaborated in other Central-European countries. The new rules presented in the next chapter are simplified a little in order to keep attention on the main topics of safety classification.

2. CLASSIFICATION OF WELDED STRUCTURES

An option will be given to designers in some cases of welded joints (e.g. groove welds subject to tension)

- either the design strength shall be reduced
- or a radiographic and/or ultrasonic control of joints shall be ordered.

There are added detailed specifications of control requirements in construction [2]. Two indices are defined in order to decide about extent of the control

- ZA depends on nature of loads as well as stress level in comparison with the limit strength of a structural element under consideration (Table 1),
- ZB depends on economic damage in comparison with the average Annual Salary (AS) as well as probability of loss of human life in consequence of a structural failure (Table 2).

Nature of loads	Stress level ratio <50%	50-80	>80%
Static	0	0	1
Dynamic	0	1	2
Fatigue	1	2	3

Table 1 Index ZAcalibration

Material losses	Danger for life improb.	probable	very pr.
<1 AS	0	2	4
1 - 10	2	4	6
>10 AS	4	6	8

Table 2 Index ZBcalibration

The index ZAis the criterion for determination of the fraction of welds which should be subject to a radiographic and/or ultrasonic control as well as the permissible defectiveness of welds (Table 3).

The sum ZA+ZBdefines the class of a structural element. The maximum value defines the class of structure Z,

$$Z = \text{Max} \ (ZA+ZB), \tag{1}$$

Three classes are proposed (Table 4). The fabricaton shops shall be devided in three categories. Only the I-st category plants shall be licenced to construct welded structures of any class

ZA	Fraction of radiographic/ultrasonic tests	Permissible defectiveness class
3	min 50%	max 2
2	min 25%	max 3
1	min 10%	max 4

Table 3 Quality control requirements

Z	Safety class of the structure	Category of welding shop
>7	1	I
3-7	2	I and II
<3	3	I, II and III

Table 4 Safety classification

Implementation of the new system of quality assurance of welded structures is not easy. The problem of workshop categorization is very controversial. There are needs for a more systematic inspection and quality control in building steel structure. It should approach perhaps the quality control system which is achieved by ship classification organizations such as the Lloyd's Register or Det Norske Veritas.

3. PROBLEM OF SAFETY DIFFERENTIATION

Safety classes of welded structures (Table 4) are analoguous to ones recommended by the Joint-Committee on Structural Safety, JCSS, for any civil engineering structure. Consequences of failure, i.e.

- material damage in comparison with initial costs,
- loss of human life and limb,

were discussed as the criteria of classifiation [3]. Somwhat different criteria are given by the Nordic Committee on Building Regulations NKB [4] for Ultimate Limit States, ULS, (Table 5) and the German Institute for Standardization, DIN [5] for the ULS and SLS - Serviceability Limit States (Table 6)

Failure consequences	Failure type (ULS)		
	ductile (strain hardening)	ductile (no extra capacity)	brittle and instability
Less serious	3.1	3.7	4.2
Serious	3.7	4.2	4.7
Very serious	4.2	4.7	5.2

Table 5 Safety Index β according to NKB

Class	U L S			S L S		
	Risk to human life	Economic consequences	β	Economic consequences	Impediments in use	β
1	no	sligth	4.2	sligth	sligth	2.5
2	exists	considerable	4.7	considerable	considerable	3.0
3	great	great	5.2	great	severe	3.5

Table 6 Safety indices β according to DIN

The index β is a semiprobabilistic measure of safety defined for independent load and material properties as follows

$$\beta = (\bar{R} - \bar{S}) / \sqrt{\sigma_R^2 + \sigma_S^2} \quad , \tag{2}$$

where $\bar{R}$ – mean capacity of the structural system,
$\bar{S}$ – mean peak load during the service life,
σ_R – standard deviation of the random capacity R ,
σ_S – standard deviation of the random load S .

A random safety margin, Δ , is defined as follows,

$$\Delta = R - S \tag{3}$$

and the safety index β is the reciprocal of its coefficient of variation (c.o.v.)

$$\beta = 1/v_\Delta \quad , \tag{4}$$

where $v_\Delta = \sigma_\Delta / \bar{\Delta}$ – the normal c.o.v. according to the definition of probablity

If the Gauss probability distributions are assumed for random variables R and S the probability of failure P is a function of index β (Table 7)

β	5.2	4.7	4.2	3.0	2.5	2.0
P	1*E-7	1*E-6	1*E-5	1*E-3	5*E-3	1*E-2

Table7 Relation of probability of failure and safety index

The formula (1) has a meaning for the level-2 probability based design, so called. A more practical safety classification, suitable to level-1, i.e. Load and Resistance Factor Design (LFRD), has been recommended by the Soviet Building Design Standard GOSSTROY [6] (Table 8).

Structural class	Degree of importance	Examples of structures	Reduction coefficient for design strength
I	Very great national and/or social importance	Main structures of power and metallurgical plants Stacks of height > 200 m TV towers, theaters, kindergartens, hospitals, museums etc.	1.0
II	Important	Structures not included to classes I nor III	0.95
III	Limited importance	Stores of agricultural products, chemicals, coal etc. 1 storey residential buildings, electric line supports etc.	0.9

Table 8 Safety classes according to GOSSTROY

A similar but more detailed list of this type is given in the Polish draft of welded structures classification [2]. It is extended to other civil engineering structures. It may help, together with index ZA, to precise the quality control classification. However, the sense of safety clasifications according to Tables 5, 6 and 8 is to influence the structural analysis and to dimension the structural members adequately and not to differentiate the control requirements. This is the main difference between the two approaches Both have the same goal: to moderate the probability of failure according to its consequences. One point seems to be not right in the welded structure classificatin: the nature of loads should influence the value of design strength and/or workmanship rules and not the safety classes themselves.

4. CONCLUSIONS

Structural analysis and adequate dimensioning of structural elements for the loads acting upon and possible consequences of failure are usually good measures for quality assurance. But there are also other measures which enable to keep the probability of failure on a necessary low level. This is a more intrinsic control system (radiographic and/or ultrasonic tests in the case of welds) which can be adjusted to differentiated safety requirements. The requirements depend on material losses KM and risks to human life. The latter can be replaced by the value KA of life insurance policy. This gives one argument of safety classification

$$K = KM + KA \ . \tag{5}$$

The other one is the fluctuation of random variables (actions and material properties) and in the case of particular types of structures :

- the coeficient of variation of loads v_S and not necessarily the nature of loads (static-dynamic). The value v is important in partial safety factors calibration procedures (LFRD) as well as the level-2 probability considerations. Such conclusion is also derived from a solution of optimization problem of design values R^* , S^*. The solution, given in Ref. [7], is such that the hazard scale, so called, should keep its specific value $1/k$ for each structural class

$$h(R^*) \cdot R^* = h(S^*) \cdot S^* = 1/k, \tag{6}$$

where $h(R^*)$ - risk that the random capacity R downcrosses a design value R^* ,
$h(S^*)$ - risk that the random load S upcrosses a design value S^* ,
k - index of capitalized economic and social consequences of structural failure.

The hazard functions h(.) depend on types of probability distributions. E.g., there are equations

$$h(R^*) \cdot R^* = \sqrt[u_R]{1/\gamma_R} \ , \qquad h(S^*) \cdot S^* = \sqrt[u_S]{1/\gamma_S} \tag{7}$$

for the Extreme Value probability distributions (the Weibull and the Frechet, respectively)

γ_R , γ_S - the partial safety factors,
u_R , u_S - the dimensionless parameters of variations (proportional to the normal c.o.v.'s).

Thus the c.o.v's $/v_S$ or $u_S/$ of anticipated loads, acting on a structure should be taken under consideration in the quality assurance programmes.

REFERENCES

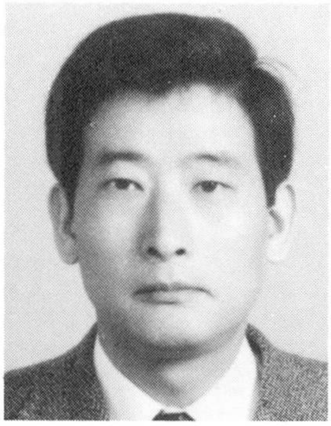
1. Szydlik W., Rational approach to quality question of welded joints (in Polish). Przeglad Spawalnictwa, 1981, nr 3
2. PKMNiJ, Welding. Welded structure classification (in Polish). Draft Polish Standard, 1984
3. Ligtenberg F.K., e.a, General Principles of Safety Differentiation, JCSS Document, March 1982
4. NKB, Recommendation for Loading- and Safety Regulations for Structural Design. Report No 36, Nov. 1978
5. DIN, General Principles on the Specifications of Safety Requirements for Structures. Beuth Verlag, Berlin-Köln 1981
6.GOSSTROY SSSR, Rules of evaluation of importance of building and engineering objects for structural design (in Russian), Moscow 1982
7. Murzewski, J., Safety of Building Structures, (in Polish) Arkady Warsaw 1970, (in German) Verlag Bauwesen, Berlin 1974

Estimation of Construction Accuracy of Steel Bridges

Evaluation du degré de précision dans l'exécution de ponts métalliques

Abschätzung der Ausführungs-Genauigkeit von Stahlbrücken

Katsuyuki TAKEMURA
Senior Res. Eng.
Kawasaki Heavy Ind., Ltd.
Hyogo, Japan



Katsuyuki Takemura, born 1944, received his degrees of B.E. in 1967, M.E. in 1969 from Kyoto University. He is now an engineer in Steel Structure & Industrial Equipment Division, and has been engaged mainly in research and development of various technical problems of bridge construction.

Fujikazu SAKAI
Section Manager
Kawasaki Heavy Ind., Ltd.
Tokyo, Japan

Fujikazu Sakai, born in 1941, received the degrees of B.E., M.E. and Dr. Eng. (1970) from Univ. of Tokyo. After he gained experience in bridge designing, finite element analysis of structures/fluids and seismic design of liquid storage tanks, he became engaged in research and development at manager level.

SUMMARY

A methodology to estimate construction accuracy is presented by treating initial imperfections as random variables. A stochastic finite element method and optimization technique are applied. A numerical example on a cable stayed bridge provides some discussion about the method to improve construction accuracy.

RÉSUMÉ

Une méthode d'évaluation du degré de précision d'une construction consiste à considérer les imperfections initiales comme des variables aléatoires. On applique alors la méthode stochastique des éléments finis et la technique d'optimalisation. Une méthode permettant d'améliorer le degré de précision de la construction est illustrée à l'aide de l'exemple numérique d'un pont à haubans.

ZUSAMMENFASSUNG

Die vorgeschlagene Methode zur Abschätzung von Ausführungs-Genauigkeit besteht darin, anfänglich vorhandene Unvollkommenheiten als Zufallsvariablen zu betrachten. Auf diese stochastischen Grössen werden sodann die Methode der Finiten Elemente und die Optimierungstechnik angewendet. Eine Schrägseilbrücke dient als Zahlenbeispiel und gestattet die Auseinandersetzung mit dem Problem der Festlegung der anzustrebenden Ausführungs-Genauigkeiten.

1. INTRODUCTION

Because of the presence of various kinds of initial imperfections introduced during the construction processes of bridges, the geometrical shape and stress distribution do not always conform to those intended by designers when completed. Such construction errors deviated from design values represent the important quality characteristics of the bridge. For the purpose of quality assurance, the constructor should make much effort to achieve good construction accuracy by taking effective quality control activities.

This paper presents a methodology to estimate stochastically the effects of initial imperfections upon the construction accuracy of bridges, and based on this, some discussions about the procedures to improve construction accuracy, especially the operation of cable length adjustment in the erection of cable stayed bridges.

The factors of initial imperfections which causes construction error are discussed first. The variation of dead load and stiffness parameters from design values, and also dimensional imperfections of each member are taken into account for initial imperfections. Secondary, analytical approach is described. The stochastic finite element method [1] is applied to analyze the effects of initial imperfections upon the construction accuracy, and the optimization technique [2] is used for the cable length adjustment. By a numerical example, some useful and important informations concerning the quality control of cable stayed bridges are presented.

2. INITIAL IMPERFECTIONS AND CONSTRUCTION ACCURACY

2.1 Initial Imperfections

Even though there will possibly be a variety of initial imperfections which may lead to the construction error, the representative ones of them are considered to be the following items as shown in Fig. 1

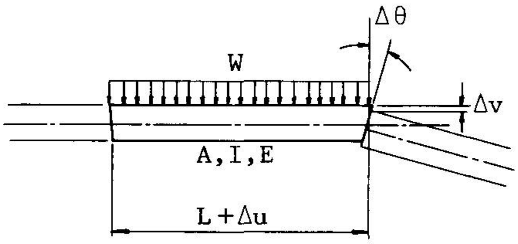


Fig.1 Initial Imperfections

The first one is uncertainty of self weight (W) and stiffness parameters such as sectional area (A), moment of inertia (I) and Young's modulus (E) of each member. They are decided deterministically in design, but actually random variables whose precise values are not known. The dimensional variation in thickness, width and height of each component may be the factors to the variation of the design constants. The variation of wire radius will be related to the cable stiffness together with the variation of Young's modulus. Furthermore, the modeling error will probably become an important factor. Dead load, which will be fluctuating spacially, is usually simplified to uniform load. Load tests on actual bridges frequently certify the contribution of secondary elements, for instance non-composite concrete deck slab, lateral bracing, etc., to the bridge rigidity even if they are assumed, as the design phylosophy, not to act to the global behavior.

Secondary, there will be some dimensional imperfections such as member length error (Δu), misalignment (Δv) and inclination ($\Delta\theta$) at the joint connecting to the adjacent member block. They are introduced during the fabrication and erection process. The dimensions of each member are usually inspected during the shop assembling, and their tolerances are specified in the code. The member length error of ±2 or ±3 mm is allowed depending on it's length for the highway bridge in Japan [3]. The joint inclination will be caused by warping of the member, mis-perpendicularity at the member end surface as well as the angular displacement due to welding or bolting of the joint during the member assembling process.

Due to the complexity of sources and the difficulty of measurement, the statistical data available to estimate the error in the imperfections described above will not be sufficient except that accumulated from the inspection results of member dimensions.

2.2 Construction Accuracy

According to the final inspection results on actually constructed bridges, some error in geometry are observed, and also some degree of deviation in stress distribution from the original design one will probably be induced.

In case of cable system, the cable tension is usually measured because of the importance to adjust to the design value. An example of the histogram of the cable tension error for a few cable stayed bridges [4, 5] constructed in Japan is shown in Fig.2.

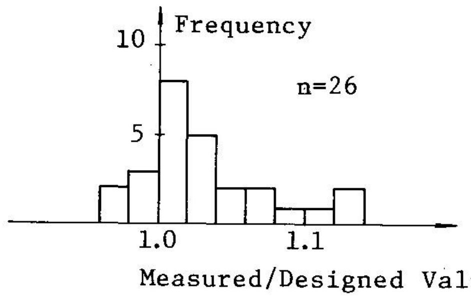


Fig.2 Histogram of Cable Tension Error

3. ANALYSIS TO ESTIMATE CONSTRUCTION ACCURCY

3.1 Construction Accuracy by Member Assembling

The bridge system will be considered to be the assembly of many members as shown in Fig.1. This member can be represented as the equivalent beam element whose stiffness matrix and end forces are functions of randomly varying initial imperfections.

If load vector, stiffness matrix and displacement of the system are expressed in terms of design values P_D, K_D and X_D, mean error terms P, K, X and the fluctuating component ΔP, ΔK and ΔK, respectively, linear elastic equations of actual and designed systems are represented by the following equations.

$$P_D + P + \Delta P = (K_D + K + \Delta K) \cdot (X_D + X + \Delta X) \qquad (1)$$

$$P_D = K_D \cdot X_D \qquad (2)$$

Subtracting Eq.(2) from Eq.(1), and then separating it into the mean error term and fluctuating component, the following expressions of geometrical error are obtained.

$$X = \bar{K}^{-1} (P - K \cdot X_D) \qquad (3)$$

$$X = \bar{K}^{-1} (\Delta P - \Delta K \cdot \bar{X}) \qquad (4)$$

in which $\bar{K} = K_D + K$ and $\bar{X} = X_D + X$ are mean stiffness and displacement, respectively, and the product $\Delta K \cdot \Delta X$ is neglected as is expected to be small. Eq.(3) can be solved by the ordinary deterministic approach.

The load vector of Eq.(4) expressed as

$$Q = \Delta P - \Delta K \cdot \bar{X} \qquad (5)$$

is caused by the variation of various initial imperfections r. The first order approximation of Eq.(5) can be obtained by taking partial derivative of Q with respect to each ramdom variable r_i. That is

$$Q = \frac{\partial Q}{\partial r} \cdot \Delta r \qquad (6)$$

$\partial Q/\partial r_i$ is a load vector due to unit imperfection of r_i, and in case of dimensional imperfections, it is represented as nodal forces equivalent to those enforcing the member to have unit deformation.

The convariance matrix C_{xx} of ΔX can be obtained from Eq.(4) and (6) as follows

$$C_{xx} = \Delta X \cdot \Delta X^T = H \cdot C_{rr} \cdot H^T \tag{7}$$

where

$$H = \bar{K}^{-1} \frac{\partial Q}{\partial r} \tag{8}$$

is the influence matrix due to r, suffix T means the matrix transpose.

Eq.(7) shows the relationship between the covariance C_{rr} of the initial imperfections r and that of responce (geometrical error) X caused by r. The diagonal elements of C_{xx} represent the variances and the off diagonal elements give information regarding the degree of correlations among responces of various parts of structure.

Stress errors of each member are also evaluated as the same form of Eq.(7). In this case, the influence matrix H can be calculated by using the corresponding displacement and stress matrix as well as the end forces of each element.

3.2 The Effects of Cable Length Adjustment

In the erection of cable systems such as cable stayed bridges and Nielsen type bridges, cable adjustment is usually taken to improve construction accuracy by varying the cable length using shim plates, screws, etc. It is usually impossible to adjust everthing perfectly, and therefore the items to be controlled for the purpose of adjustment, such as geometry of bridges and/or cable tension, will be decided.

Separating the construction errors in vector Y and Z which are the purpose of adjustment or not of this operation, respectively, then the construction accuracy Y_1 and Z_1 after adjustment are expressed as the following equations.

$$Y_1 = Y + A \cdot s \tag{9}$$

$$Z_1 = Z + B \cdot s \tag{10}$$

in which, A and B are influence matrix due to unit cable length adjustment. Construction accuracy Y and Z before adjustment are random variables whose stochastic nature is evaluated by means of the method described in 3.1.

The vector s representing the change in cable length are assumed that they are so determined [2] as to minimize the sum of squares of the components of Y_1. That is, the objective function is expressed as

$$\Omega = Y_1^T \cdot \rho \cdot Y_1 \tag{11}$$

in which ρ is the diagonal matrix whose diagonal elements represent the weight coefficients of construction accuracy to be improved. From the simultaneous equations of $\partial\Omega/\partial s_j = 0$ for each cable j, the vector s is obtained as follows.

$$s = -(A^T \cdot \rho \cdot A)^{-1} \cdot A^T \cdot \rho \cdot Y \tag{12}$$

By substituting Eq.(12), Eq.(9) and Eq.(10) are written as

$$Y_1 = Y - A \cdot (A^T \cdot \rho \cdot A)^{-1} \cdot A^T \cdot \rho \cdot Y = Y - A_1 Y \tag{13}$$

$$Z_1 = Z - B \cdot (A^T \cdot \rho \cdot A)^{-1} \cdot A^T \cdot \rho \cdot Y = Z - B_1 Y \tag{14}$$

Eq.(13) and Eq.(14) show that the construction accuracy after the operation of cable length adjustment can be expressed as the linear equations of that before adjustment. Therefore, their stochastic nature can be evaluated by using that of Y and Z.

It is obvious that the mean of Y_1 and Z_1 are obtained by using that of Y and Z in these equations. Covariance matrix of Y_1 and Z_1 are also expressed as the following equations.

$$\begin{cases} C_{Y_1Y_1} = C_{YY} + A_1 \cdot C_{YY} \cdot A_1^T - (A_1 + A_1^T) \cdot C_{YY} \\ C_{Z_1Z_1} = C_{ZZ} + B_1 \cdot C_{YY} \cdot B_1^T - B_1 \cdot C_{YZ} - B_1^T \cdot C_{YZ}^T \\ C_{Y_1Z_1} = C_{YZ} + A_1 \cdot C_{YY} \cdot B_1^T - A_1 \cdot C_{YZ} - B_1^T \cdot C_{YY} \end{cases} \tag{15}$$

From Eq.(15), the variance and correlation coefficient of various construction errors are obtained.

4. NUMERICAL EXAMPLE

4.1 Model Description

The cable stayed bridge as shown in Fig. 3 is considered as a numerical example. The design value, mean and standard deviation of the main parameters are indicated in Tab.1. Some dimensional imperfections as shown in Tab.2 are also assumed. The total number of initial imperfections in the bridge system is 134, and they are assumed to be statistically independent to each other, even if there will possibly be some degree of correlations between them. Under the above condition, the construction accuracy and that improved by the operation of cable adjustment are calculated by means of the method described in 3.

	Design Value	Mean	Standard Deviation
Dead Load	W_D = 18.0 t/m	$1.02W_D$	$0.03W_D$
Girder Stiffness	I_D = 1.8 m^4	$1.05I_D$	$0.05I_D$
Cable Stiffness	A_D = 0.06 m^2 ~ 0.03 m^2	$1.02A_D$	$0.01A_D$

Tab.1 Design Values and Their Imperfections

It should be pointed out that in the design of this type of bridge, the cable tensions are usually so determined as to reduce the bending moment at every sections of girder and tower. The bending moment diagram and cable tensions intended by designers of this bridge model are assumed as that shown in Fig.3. This design condition is related to the analysis as can be seen in Eq.(3) and Eq.(4).

		Standard Deviation
Cable Length		3 mm
Block Length		1.5 mm
Joint Indication	Girder	1/5,000 rad.
	Tower	1/10,000 rad.

Tab.2 Dimensional Imperfections

4.2 Estimated Construction Accuracy

The ranges of construction accuracy in the ($\mu \pm 2\sigma$) level are represented by the region between two solid lines in Fig.4, where μ and σ are mean and standard deviation of construction errors. The probability that the construction

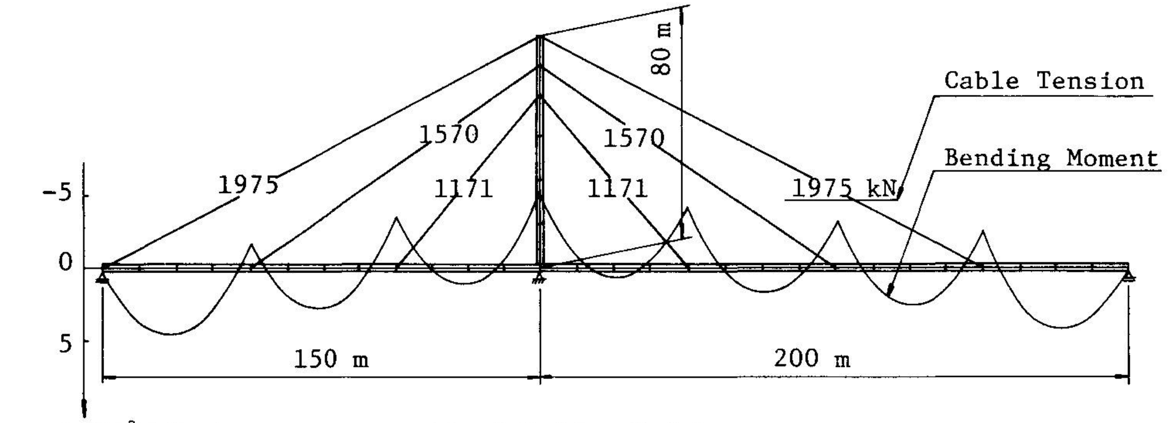


Fig.3 Bridge Model

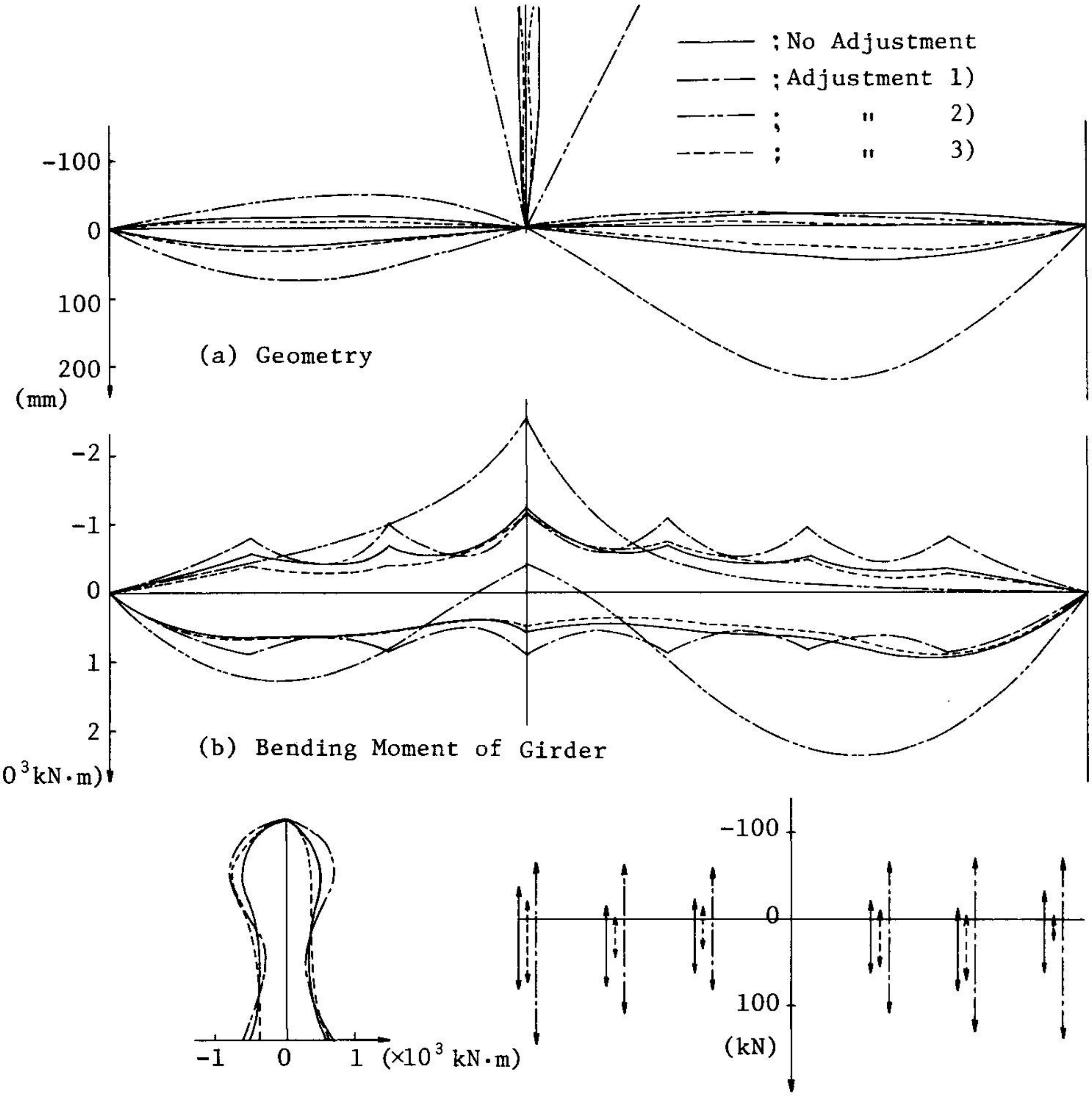


Fig.4 Estimated Construction Error (μ±2σ level)

accuracy falls into these ranges is 95%, as they are considered to be Gaussian distribution by the central limit theorem.

The mean of geometrical error is mostly caused by dead load discrepancy from design value. The effects of mean error in both dead load and girder stiffness superimpose to the bending moment of girder.

Fig.5 shows the standard deviation of geometrical and bending moment error of girder. The effects of sensitive imperfections affecting to the construction accuracy is also plotted together with the total effects of all imperfections. The geometrical error is dominately affected by the randomness of dead load. On the other hand, the bending moment of girder is notably sensitive to joint inclinations, particularly to that of neighbouring joints.

Correlations between each imperfections has essential meaning to the bridge quality. If the correlation coefficients equal to -0.5 are considered for the inclinations between adjacent joints, construction error of girder is drastically reduced as shown in Fig. 5. The fact of this suggests the importance to decrease effectively the construction error during each construction process by using the informations in preceeding processes.

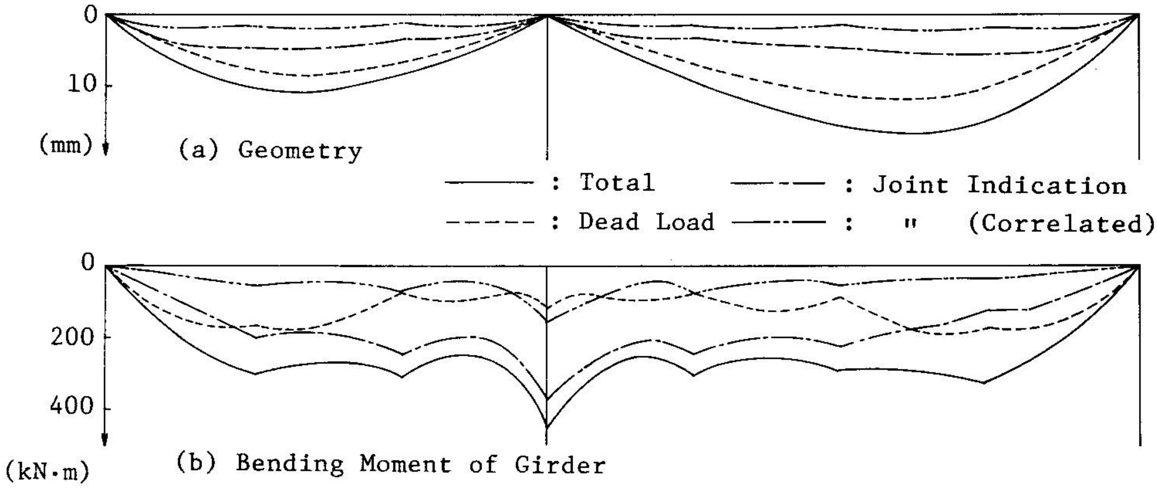


Fig.5 Standard Deviation of Construction Error

4.3 Improved Construction Accuracy by Cable Adjustment

The following three cases of cable adjustment are considered.

1) adjustment of geometry
2) adjustment of cable tension
3) adjustment of both geometry and cable tension

For the geometrical adjustment, both of deflection of girder and horizontal displacement of tower are considered to be improved. Coefficients ρ in Eq.(11) are set so that the weight of geometrical error 1 mm and the tension error 1 t becomes identical on the purpose of adjustment in case 3).

The ranges of construction accuracy after cable adjustments is shown in Fig.4. In case of geometric adjustment, both of bending moment and cable tension become deteriorated even if geometrical error will be negligible small. If cable tension is adjusted, it is obvious that the tension of all cables and the bending moment of tower conform perfectly with the designed ones. However, the error of geometry and bending moment of girder become considerably large. So as to obtain a well balanced bridge system, the simultaneous adjustment of geometry and cable tension as case c) should be made.

By calculating the improved effects of cable adjustment for each type of imperfections, it has become clear that the cable adjustment has different effects on the error due to various imperfections.

The construction error due to imperfections of cable itself (stiffness and cable length error) can perfectly be absorbed by any cases of adjustment. The same effects of adjustment are also obtained for the imperfections of block length except the error of horizontal length of girder and hight of tower.

The dominating imperfections affecting to the deteriorated geometry and overstressing of girder induced by cable adjustment (see Fig.4) is dead load imperfections. The adjustment of geometry is, therefore, appropriate for the dead load imperfections.

In case of joint inclinations and stiffness errors, the geometry will be spoiled if emphasis is put on stress improvement, and improvement of geometry leads to the contrary circumstances.

The standard deviation of stress caused by the error of bending moment is 30 to 50 kgf/cm^2, and improvement by cable adjustment cannot be expected so much. Coefficient of variation of cable tension error is about 2% before adjustment and improved to 1% or so by cable adjustment.

5. CONCLUDING REMARKS

The authors would like to conclude that the proposed method to estimate the construction accuracy of bridges is worth-while to notify quality items to be essentially controled, effective methods to improve the construction accuracy and the stress level that should be considered in design. By applying the method to a cable stayed bridge, the following knowledges are obtained.

1) The degree of correlations between imperfections has considerable influences on the bridge quality. Therefore, it is important not only to reduce the imperfections themselves, but to decrease the combined effects of them during the construction processes.

2) In case of cable adjustment, attention should be payed to improve both of geometrical error and cable tension error to lead the well-balanced system as a whole.

At the construction of complicated structure such as cable stayed bridges, it will become important to improve the bridge quality on the basis of high degree of cooperation between designers, fabricators and erectors who are concerned in each bridge project. It will be the author's pleasure that this paper contributes to such quality control activites.

REFERENCES

1. Handa K. and Anderson K., Application of Finite Element Methods in the Statistical Analysis of the Structures. Proc. of ICOSSAR '81, Trondheim, Norway, June 1981

2. Fujisawa N. and Tomo H., Computer-Aided Cable Adjustment of Stayed Bridges. Proc. of IABSE, Vancouver, Canada, November 1985

3. Japan Road Association, Standard Specifications for Highway Bridges. February 1980

4. Matsumura H. et al., Measurement and Adjustment of Cable Tension of Cable Stayed Bridges. Bridge and Foundation, Vol.13, No.8, 1979 (in Japanese)

5. Kurata O. et al., Structural Characteristics and Load Test of 2-span non-continuous Cable Stayed Bridge. Bridge and Foundation, Vol.15, No.1, 1981 (in Japanese)

Safety of Compressed Stiffened Panels with Initial Imperfections

Sécurité à la compression de plaques nervurées présentant des imperfections

Sicherheit gedrückter Rippenplatten mit Ausführungs-Ungenauigkeiten

N.N. STRELETSKY
Professor
TSNII Proektstalkonstruct.
Moscow, USSR

N.N. Streletsky was born in 1921. In 1947 he graduated from the Moscow Institute of Civil Engineering. He is specialist in metal bridges, steel and reinforced concrete structures and limit states theory. He is head of the department of engineering structures.

M.D. KORCHAK
Cand. Sc. (Eng.)
Moscow Inst. Steel & Alloys
Moscow, USSR

M.D. Korchak was born in 1941. In 1969 he graduated from the Kharkov Institute of Civil Engineering. He is specialist in the field of stability and safety of metal structures.

V.S. DANKOV
Cand. Sc. (Eng.)
TSNII Proektstalkonstruct.
Moscow, USSR

V.S. Dankov was born in 1944. In 1966 he graduated from the Moscow Automobile Road Institute. He is specialist in metal bridges and structures and senior research scientist of the engineering structures department.

SUMMARY
Some practical recommendations on the correction of tolerances for initial imperfections of compressed stiffened panels, based on the analysis of their behaviour with regard to interaction of buckling forms, are given. It is shown, that all tolerances should be made dependent on the stiffness relationship of the panel constituent elements.

RÉSUMÉ
L'article présente des recommandations pratiques pour la correction d'imperfections initiales dans des plaques nervurées en compression. Elles se basent sur l'étude du comportement des plaques sous l'effet du voilement. Il est montré que les tolérances devraient être exprimées par rapport aux rigidités des éléments isolés d'une plaque.

ZUSAMMENFASSUNG
Der Beitrag enthält Empfehlungen zur Festlegung geometrischer Ausführungs-Toleranzen für unter Längsdruck stehende Rippenplatten. Diese Empfehlungen stützen sich auf die rechnerische Ermittlung des Verhaltens solcher Bauteile unter Einbezug der Beulformen. Es wird gezeigt, dass die Toleranzen von den Steifigkeitsverhältnissen der die Rippenplatte bildenden Teile abhängig gemacht werden sollten.

Steel stiffened panels are used as lower orthotropic plates (bottom chords) of box-shaped continuous bridge superstructures. At the stage of erection and, in the vicinity of intermediate supports, at the stage of performance such steel stiffened panels work intensively in compression. Buckling of such panels is the reason for some bridges failures. That's why the need for the most economical solutions of the panels safety improvement is very actual.

Loadbearing capacity and safety of a compressed stiffened plate depends on accidental initial imperfections and, first of all, on geometrical imperfections of the plate as a whole and its elements. Thus, the improvement of the safety should be realized through scientifically confirmed application and reasonable keeping to the tolerances (geometrical imperfections limiting values) on steel stiffened plates fabrication.

Let's consider, as an example, the panel of the compressed stiffened plate with longitudinal stiffeners (Fig. 1), where L is the panel length, the transverse girders, restraining of the panel are assumed to be absolutely rigid, "a" is the distance between the longitudinal stiffeners. Compressive stresses in the plate are assumed to be uniformly distributed over its area.

The panel considered is characterized by the following three forms of buckling under compression: overall buckling of the plate with its vertical geometrical imperfections at displacement LQ_1, local buckling of the longitudinal stiffeners with lateral geometrical imperfections of the stiffener upper edge at displacement LQ_2; local buckling of the bridge elements between the longitudinal stiffeners.

The design often considers an ideal plate structure without initial imperfections, for which the critical stress of the overall buckling is equal to σ_E, the critical stress of local buckling of the longitudinal stiffener is equal to σ_S and the critical stress of the local buckling of the plate parts between the longitudinal stiffeners is equal to σ_L. For the analysis of various real plates the relationships $x_1 = \sigma_E / \sigma_S$ and $x_2 = \sigma_E / \sigma_S$ of critical stresses for an ideal plate are used. For an equistable ideal plate $x_1 = 1$ and $x_2 = 1$. A real plate with initial imperfections may exibit an abrupt failure at stresses considerably lower than critical stresses of an ideal plate. The results of investigations /1-5/ showed, that the interaction of various buckling forms for real plates was a complex process and for reduction of the panel sensitivity to imperfections there should be $x_1 < 1$ and $x_2 > 1$, i.e. for the critical stresses of the corresponding ideal plate the unequalities $\sigma_S > \sigma_E > \sigma_L$ are desirable.

It was shown /3, 5/, that the local buckling of the plate may be considered to be insignificant. That's why, we consider only the interaction of the vertical relative imperfections of the panel as a whole, ε_1, and the horizontal relative imperfections of the upper edge of the longitudinal stiffener, ε_2. The relative imperfections are found in relation to the length L.

The problem is to be solved with the theory of disasters /6/. The advantage of the most complete presentation of a real structure with a large number of degrees of freedom by a model with two degrees of freedom was used. The steel behaviour is assumed to be elastic.

Let's consider the panel element with a width "a" with initial geometrical imperfections ε_1 and ε_2 . Let's introduce the characteristics of stiffness: stiffness in an axial force K_1=EA; stiffness in bending K_2 = EJ; a structural parameter $\kappa = k_2/k_1 l^2$. Here A is a cross sectional area of the plate with a width "a"; J - is a moment of inertia of a longitudinal stiffener. $\lambda = \frac{P}{EA}$ is a nondimensional parameter of the loading, where P is a compression load of the plate part considered.

By modifying the system potential energy according to the displacement parameters and taking the independence ε_1 and ε_2 from each other, in contrary to / 6 /, we obtain the following equilibrium equations:

$$\lambda Q_1\left[\frac{1}{2}+\frac{1}{4}(Q_1^2+Q_2)\right]-\alpha\sqrt{2}\left[\frac{1}{32}Q_1(8-6Q_1+5Q_1^2+6Q_2^2)-\frac{1}{2}-\frac{1}{8}Q_2\right]-1=0 \qquad (1)$$

$$\lambda Q_2\left(\frac{1}{4}+\frac{1}{8}Q_1^2+\frac{1}{8}Q_2^2\right)-\kappa\left[Q_2-\varepsilon_2+\frac{1}{3}(Q_2-\varepsilon_2)^3\right]-$$
$$-0{,}885\,\alpha Q_2(8-4Q_1+2Q_2^2+3Q_1^2)+Q_2=0\,, \qquad (2)$$

where $\alpha=\left[(1+\varepsilon_1)^2+(1-\varepsilon_1^2-\varepsilon_1^2)\right]^{\frac{1}{2}}$

By substituting the values ε_1 , ε_2 and K into the equations (1) and (2), we obtain the systems of equations with two unknown values - a vertical relative displacement Q_1 and a horizontal relative displacement Q_2. Solving these systems of equations, we get the relationships of Q_1 and Q_2 to the load λ at definite values ε_1, ε_2 and K. The plots for these relationships are given in Fig. 2 and 3.

The curve 1 in Fig. 2 represents the ideal panel ($\varepsilon_1 = \varepsilon_2 = 0$) which buckles from the panel plane in the elastic stage. The curves 2, 4 and 6 in Fig. 2 represent the panels with imperfections of only one type, i. e. overall imperfections $\varepsilon_1 = 0{,}03$ and, correspondingly, the displacements $Q_2 = 0$. The curves in Fig. 3 represent the panels with imperfections of the longitudinal stiffeners $\varepsilon_2 = 0{,}03$ at $\varepsilon_1 = 0$ and, correspondingly, at displacements $Q_1 = 0$. The dotted parts of the curve 6 in Fig. 2 correspond to the unstable conditions of the structure.

The dot-and-dash lines 4, 5 and 7 in Fig. 2 correspond to the simultaneous presence of both types of initial imperfections and $\varepsilon_1 = \varepsilon_2 = 0{,}03$. In these panels, increasing of the load parameter λ leads to simultaneous development of relative displacements Q_1 and Q_2. The analysis showed, that in the cases investigated the maximum value of the panel loadbearing capacity was reached at $Q_2 \approx 0.2$. The plots are given for three values of the structural parameter K = 0,3; 0,5 and 1,0. The structural parameter K = 0,3 corresponds to the panels with very weak stiffeners; at the further decreasing of K, such a panel turns into a plate. The structural parameter K = 1 corresponds to the panel with a very strong stiffening set, at further increasing of this parameter such a panel turns into a rigid compressed bar.

Comparison of the curves 2, 3 and 6 with the curve 1 in Fig. 2 shows, that the weaker the reinforcement, the greater the degree of the panel loadbearing capacity reduction and deformation capacity improvement, when compared with an ideal panel, which is quite natural.

It is interesting to note, that quite a weak initial imperfection of the longitudinal stiffener (ε_2 = 0,03) at K = 0,5, i. e. at a common reinforcement, leads to the reduction of the ultimate load λ from 1,46 to 0,7 (the curve 8 with its maximum in the point A and the curve 2 in Fig. 3), however, at weak stiffeners (K = 0,3) this negative effect is even more pronounced (see curves 4 and 3).

The combined influence of the two imperfections ε_1 and ε_2 is also the more pronounced, the weaker are the stiffeners. It follows from the comparison of curves 4,5 and 7 with the respective curves 2, 3 and 6 in Fig. 2.

The results of our calculations, given in Fig. 2 and 3 are similar to the results of other investigations /1-6/, performed with different methods. The conformity of the results helps to distinguish a common characteristic: the loadbearing capacity of the stiffened plates is considerably influenced by the relationship of the panel constituent elements stiffness values and the panel as a whole and there exists such a range of such relationships (K>0,5), in which the negative influence of the inevitable in practice initial imperfections is very insignificant. This offers considerable scope for improvement of safety and efficiency of compressed stiffened panels through fabrication tolerances refinement, which keep under control the largest admissible geometrical imperfections /7, 8/.

Nowadays, the fabrication tolerances depend only on geometrical sizes of a separate element or a panel as a whole. It would be sound practice to establish the tolerances as a function of relations of the stiffness values of the structure constituent elements, which would be influenced by the structure designation and its performance conditions and may be characterized by the above mentioned parameters $x_1 = \sigma_E / \sigma_S$ and $x_2 = \sigma_E / \sigma_L$, i. e. by the relationships of critical stresses in a ideal plate. This permits to make fabrication tolerances in some cases not so severe.

Below, some recommendations on tolerances establishment on overall initial geometrical imperfections of the plate Δ_1, imperfections in a separate stiffener Δ_2 and imperfections in the plate part between the stiffeners Δ_3 are given.

The tolerances Δ_1 should depend on the parameter x_1. The plates with thin stiffeners ($x_1 > 1$) require maintaining of the tolerance Δ_1 = 1/750. At strong longitudinal ribs ($x_1 < 1$) the tolerance Δ_1 may be increased to 1/500. This preferential tolerance may be used for weaker stiffeners (x_1 = 1 - 1,5) on condition of introducing a more severe tolerance Δ_2 on bending of longitudinal stiffeners (which is difficult to realize in practice), or by using rigid in the lateral direction longitudinal stiffeners instead of flat ones.

As to the tolerances on the longitudinal stiffeners bending, Δ_2, two ways are possible: (1) a definite reduction of stiffeners rigidity within the limits of x_1 = 1,0-2,0 at maintaining strict tolerances Δ_2 of about 1/500; or (2) making stiffeners sufficiently rigid ($x_1 \simeq 1$) with a slight reduction of restrictions of

Δ_2 to 1/400.

It is possible to reduce the influence of local imperfections of the plate between the stiffeners by decreasing its thickness within the limits admitted by the fabrication and performance conditions. It is known, that in flexible plates the postcritical reserve of the strength may compensate for the imperfections influence. In case a thick plate (x_2 = 0,9 - 1,0) is required by the performance conditions, then the tolerance Δ_3 should be limited by the value about 1/200, because the initial geometrical imperfections in the rigid plates are not desirable.

In the other cases ($x_2 > 1$) the initial imperfections at the level Δ_3 = 1/100 may be considered to be insignificant.

The given recommendations on the tolerances Δ_2 and Δ_3 application are comparable to the actual values of initial geometrical imperfections in the stiffened plates. Thus, in Checkoslovakia and West Germany the value ε_2, obtained as a result of 243 measurements for 5 bridges, varies within the limits 1/953 - 1/243 at an average value ε_2 = 1/330, the value ε_3, obtained as a result of 1620 measurements, varies from 1/359 to 1/90 at an average value ε_2 = 1/109. As a rule the values ε_2 and ε_3 are higher than those admitted by the Specifications of many countries. So, the Specifications of such countries as Belgium, West Germany and Great Britain admit the value Δ_2 = 1/500, in the USSR this value is equal to 1/400, as to the value of the tolerances Δ_3, it is specified as 1/250 in Belgium and West Germany and 1/200 in Great Britain, the tolerance Δ_1 is 1/750 according to the USSR Specifications.

The above described approach to the problem of safety of compressed stiffened panels of box-shaped bridge superstructures and similar structures permits within the limits of not very severe tolerances to fabricate safe in operation structures of box section beams by means of the plate and its stiffeners rigidity control. Besides, an opportunity is obtained to estimate quite precisely the loadbearing capacity of the as-fabicated structures and those in performance, in which the real level of initial imperfections is above the admissible one.

REFERENCES

1. Ivergaard V. Imperfection sensitivity of a wide integrally stiffened panel under compression. Int. J. Solid and Structures, 9, 1973, p. 177.
2. Ellinas C.P., Croll T.G. The basis of a design approach for stiffened plates stability problems in Eng. Structures and Computers. London, 1978, pp. 401-421.
3. Маневич А.И. К теории связанной потери устойчивости подкрепленных тонкостенных конструкций. Прикладная математика и механика, 1982, вып.2.
4. Теребушко О.И., Адуевский А.В. Устойчивость и закритическая деформация подкрепленных панелей с начальными напряжениями. Строительная механика и расчет сооружений, 1985, № 2, с.39-42.

5. Sridharan S. Doubly symmetric interactive buckling of plate structures. Int. J. Solids and Structures, 1983, V. 19, No. 7, pp. 625-641.

6. Niwa Y., Watanabe E., Nakagawa N. Catastrophe and imperfection sensitivity of two-degree-of-freedom system. Proc. ASCE, 1981, No. 307, pp. 99-111.

7. СНиП Ш-I8-75. Металлические конструкции. М.,Стройиздат,I976.

8. Dowling P.T., Frieze P.A., Harding T.E. Imperfection sensitivity of steel plates under complex edge loading. Stability of steel structures. Liege, 1977, pp. 305-314.

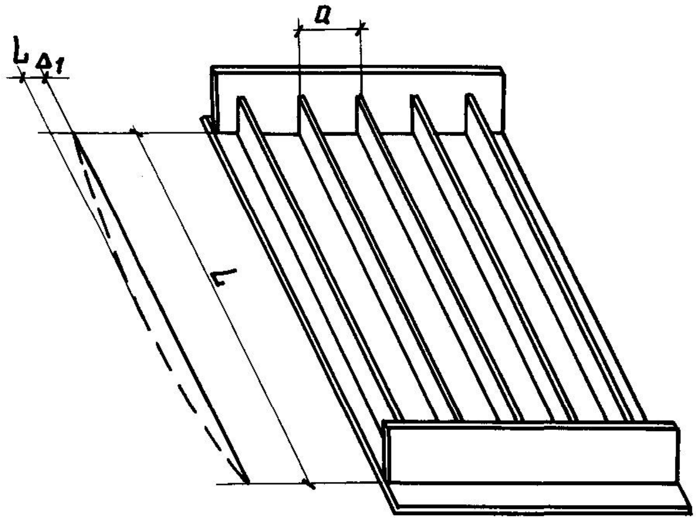
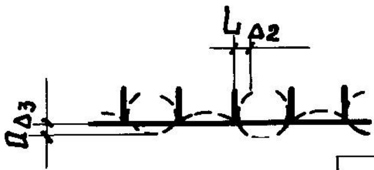


Fig.1



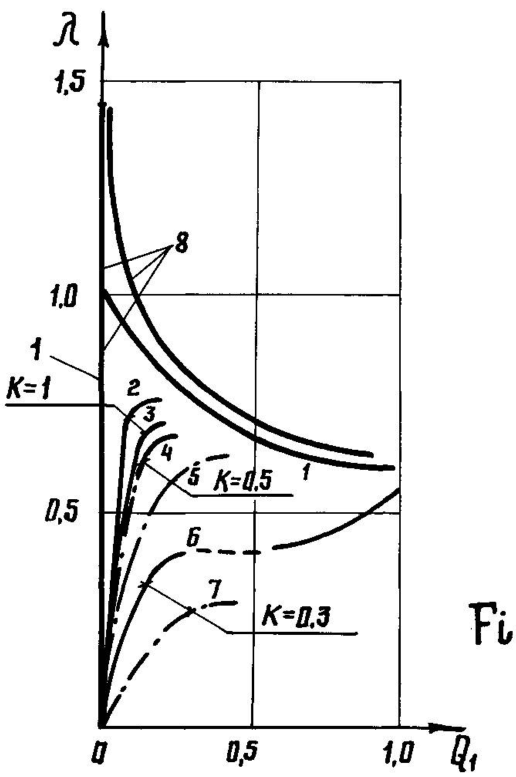


Fig.2

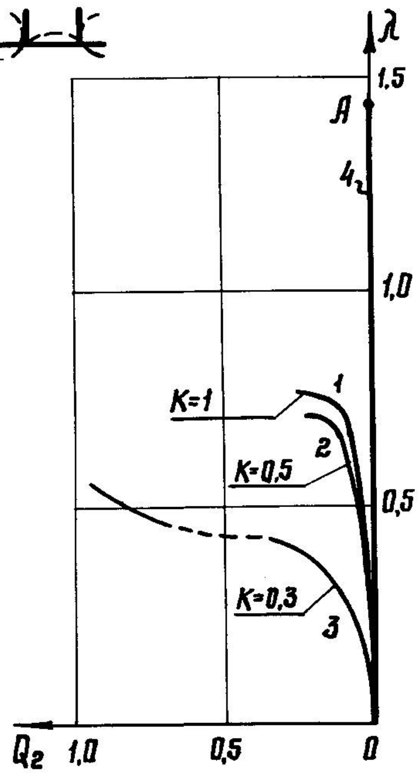


Fig.3

Quality Control of a Double Deck Railway Bridge in Steel

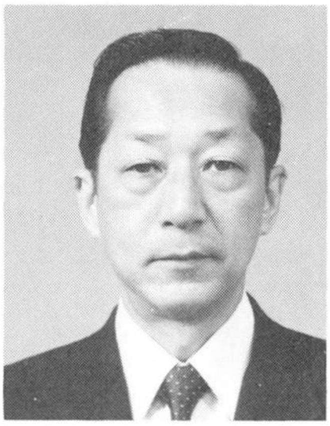
Contrôle de la qualité d'un pont de chemin de fer à double tablier

Qualitäts-Kontrollen beim Bau einer Doppeldeck-Eisenbahnbrücke aus Stahl

Takeshi SUGA
Civil Engineer
Japan Railway Constr. Public Corp.
Tokyo, Japan

Takeshi Suga, born in 1943, gained his civil engineering degree at the Univ. of Tokyo, Japan. He has designed many steel railway bridges for the construction of the Keiyo Line, JNR and is experienced in the design and management of steel railway bridges.

Shinichi MATSUDA
Civil Engineer
Mitsubishi Heavy Ind., Ltd.
Yokohama, Japan



Shinichi Matsuda, born in 1932, gained his civil engineering degree at the Univ. of Tokyo, Japan. He is well experienced in the production of steel structures such as bridges and gates.

SUMMARY

This was the first triple-main-truss railway bridge with a double steel deck built in Japan. During the fabrication of this complex structure, quality control was performed to assure the safety, giving priority to maintain both the finishing accuracy of each member and high quality in welding. The method and procedure of fabrication were devised to give the required quality. Excellent results were efficiently achieved.

RÉSUMÉ

Il s'agissait du premier pont de chemin de fer à trois fermes principales et à double tablier supérieur construit au Japon. Au cours de la construction d'une structure aussi complexe, un contrôle de la qualité a été effectué afin d'assurer la sécurité, l'objectif principal étant d'obtenir une grande précision de finition de chacune des membrures et des soudures de qualité supérieure. Les méthodes et procédés de construction ont été établis afin d'obtenir le niveau de qualité requis. Des résultats extrêmement satisfaisants ont été obtenus.

ZUSAMMENFASSUNG

Die als Beispiel dienende Eisenbahnbrücke aus Stahl ist die erste in Japan gebaute Doppeldeck-Brücke mit drei Fachwerk-Hauptträgern. Während des Baus dieser komplexen Konstruktion wurden Qualitäts-Kontrollen zur Gewährleistung der Sicherheit durchgeführt, wobei die Einhaltung der vorgeschriebenen Fertigungsgenauigkeit der einzelnen Teile und der Qualität der Schweissnähte Vorrang hatten. Zur Erreichung der geforderten Qualität wurden neue Methoden und Herstellungs-Verfahren eingesetzt, welche in effizienter Weise ausgezeichnete Ergebnisse erbrachten.

1. PREFACE

The Dai-ichi Yumenoshima Bridge on the Keiyo Line, Japanese National Railways (JNR), is a steel railway bridge spanning National Route 357 at Yumenoshima, Tokyo, Japan. It is a double-decked truss bridge having a double track line and an end part of a passenger platform on each deck (See Fig. 1). This railway bridge is the first one of its type built in Japan.

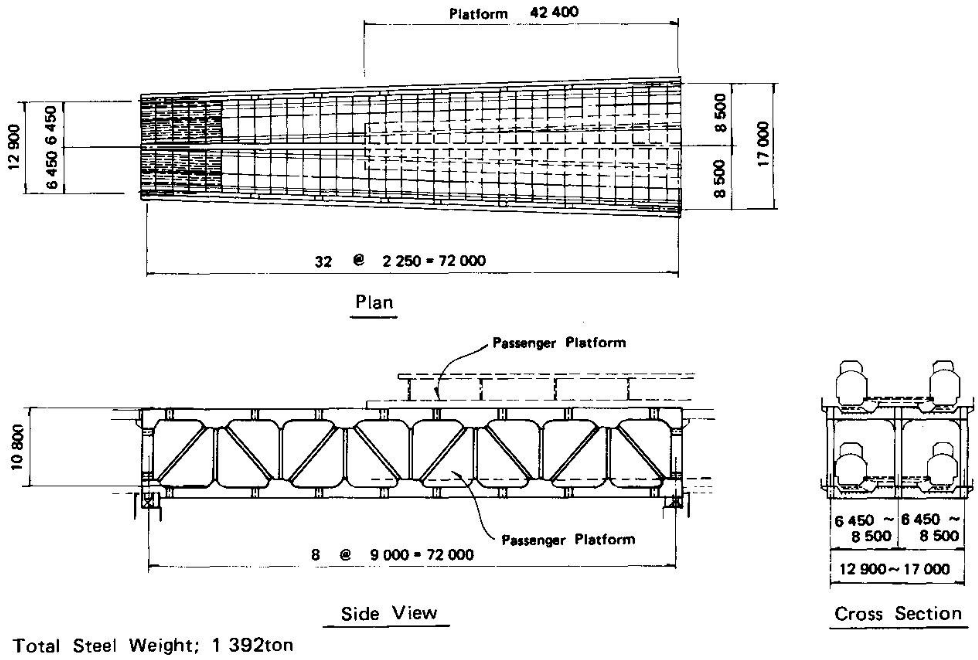


Fig. 1 General View

The major characteristics of the bridge are as follows:

- The triple main truss structure type was selected taking the various conditions on substructure works and the economic efficiency into account.
- The plan of the bridge has a trapezium shape with its width enlarged toward one end to provide an end part of a passenger platform between the tracks.
- For the prevention of noise, a steel deck using a ballasted floor system was adopted. And the floor was designed to have shallow depth.
- As the member forces of each structural member was expected to be significantly large because of the live loadings on the upper and lower double track lined decks, the web members were designed to be spliced on four planes at each panel point.
  Furthermore, a careful check for the fatigue strength was made for each structural member.

Generally, it is quite important to prevent fatigue failure in the case of steel railway bridges because of the number of stress cycles with large amplitudes due to the repetition of live loadings is more than those of any other steel structure.
Especially as this triple-main-trussed double decked and an unsymmertical bridge had such a complex structures, the utmost safety investigation including a three dimensional analysis of the framework and stress analysis of panel points by means of FEM was carried out at the design stage.
The following is a report of the quality control activities that were achieved, reflecting the intentions of the purchaser, Japan Railway Construction Public Corporation (JRCC), during fabrication.

2. PRODUCTION POLICY AND QUALITY CONTROL

In the production of the bridge, JRCC specified the testing, inspection and quality assurance in detail, based upon Japan Railway Standards (JRS) 05000-1, "Fabrication of Steel Bridges." Furthermore, JRCC had discussed the structural peculiarities of the bridge with the fabricator and established a special fabrication procedure before the commencement of production.
In the abovementioned procedure, careful attention was paid to following items:

2.1 Priority control

2.1.1. Pursuit of dimensional precision

The bridge was a highly complex structure consisting of a huge number of members, joints and connecting bolts and had to have high rigidity. To ensure the design strength and safety of the structure, it was essential that the members be assembled precisely and smoothly. For this reason, the rigorous dimensional precision of each member was pursued to the extreme.

2.1.2. Stringent quality control in welding

For the sake of safety assurance against fatigue failure due to the large number of repetitions of live loadings, special attention was paid to the welding and finishing method at the panel points (especially to fillets) and the welding continuity where stringers cross floor beams.

2.2 Quality control program

In addition to the inspections by JRCC that included mold loft inspection, material inspection, parts inspection, and shop assembly inspection, the fabricator carried out various autoromous quality control activities

2.2.1 Work procedure

A work procedure was established for each production step to assure the quality of products.

2.2.2 Quality control by shop workers

For each production step determined by JRCC, the production staff of the fabricator provided detailed check sheets which clarified and detailed the production procedure, designed dimensions and shapes and given standards, etc. The shop workers autonomously managed the quality control of each product.

2.2.3 Independent inspections

At each major production step, fabricator's quality control specialists executed various inspections such as material inspection, mold loft inspection, bead test, parts inspection, shop assembly inspection, and others.

2.2.4 Investigation of Q.C. team

The managers and production engineers of the fabricator had organized a quality control team and investigated the production shop regularly to verify the achievements of autonomous quality control activities and to give the appropriate instructions.

2.3 Policy in establishment of production procedure

The following procedures were established to efficiently satisfy JRCC's quality requirements without producing any delay in the given production schedule.

2.3.1 Dimensional precision

To minimize the adjustment or modification at the stage of trial assembly of the bridge, stringent dimensional precision of each member was pursued to be extreme for each production step.
For example, Table 1 shows the cutting and boring procedures.
Furthermore, the most suitable fabrication sequence and special jigs were carefully devised.

Table 1 Cutting and Boring of the Pieces

		Cutting			Boring		
					PRE-STAGE		POST-STAGE
		Numerical Control (NC)	Flame Planer (F/P)	Automatic Gas	NC	Girder Type	NC or Compressed Air type
Main Chord	Web PL.	O				O	
	Flange PL.		O	O		O	
	Brakets	O				O	
	Diaphragm	O					
	Splice PL.		O	O	O		O (NC)
Floor system	Deck PL.		O	O		O	
	Stringers		O	O		O	
	Floor Beam	O					O(Air)
	Splice PL.		O	O			

2.3.2 Quality control in welding

Every possible potential problem was carefully forecast and solved prior to the actual welding work. In particular a mock up trial was carried out to determine the fabrication and welding sequence of the steel deck.

3. FABRICATION OF MAIN TRUSS AND STEEL DECK

3.1 Fabrication of main truss

Fig. 2 shows a typical block of chord members. To ensure the smooth distribution of stress at the panel points, web members were designed to be spliced on four planes. The following fabrication procedures were executed in order to satisfy the target tolerance (± 1.5 mm) for both each block length and the diagonal length of the gross section of each member.

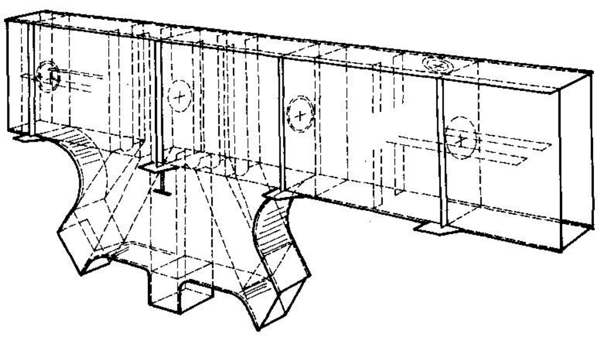


Fig. 2 Block of Upper Chord

- A NC machine was employed to mark and cut the web plates and the gusset plates of the panel points. Prior to cutting, the cutting lines, water-lines and boring positions were checked and confirmed.

- To match the hole positions, pairs of gusset plates were precisely layered and bored to the same size.
- After welding the web plates and gusset plates, the pair of welded members were again layered to reconfirm the dimensions of the hole and check the welding distortion.
- The bolt holes at one end of each block were bored at the stage of piece cutting. The ones at the other end of each block were bored after the abovementioned dimensional reconfirmation.
- To ensure the precision of the sectional dimensions of block, each diaphragm was machined on all four sides to precise dimensions. Furthermore, similar machine - finished temporary diaphragms were set with bolts at each end of the block when the pieces were assembled (See Fig. 3).
- The fillets of panel points were well welded and finished (See Fig. 4).
- Each end face of blocks was machine finished to ensure the designed dimensional precision.
- After machine finishing, the actual dimensions and the location of the bolt holes were carefully measured. A high speed NC boring machine programed with such measured data was employed to bore the splice plates.

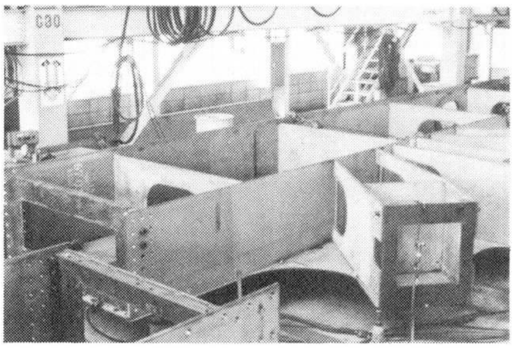


Fig. 3 Shop Assembly of Main Chord

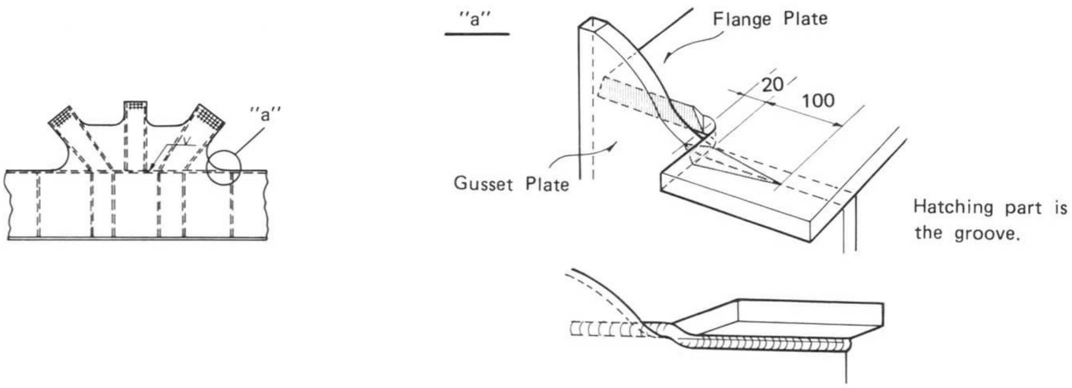


Fig. 4 Details of the Fillet for Welding

3.2 Fabrication of steel deck

Fig. 5 shows a typical block of the steel deck. The block has joints on four sides and the directions of the bend line on both sides are not in parallel. The following delineates the fabrication procedure of the steel deck block.

- After cutting of the deck plates a NC styler was laid on the plate to confirm the designed shape and the bending and boring positions. Then, bolt holes except those to be located at the bends, were bored

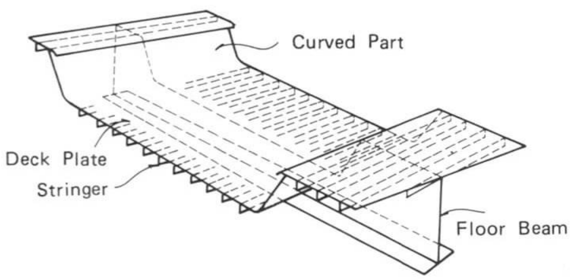


Fig. 5 Block of Steel Deck

before the cold bending of the plate. The bolt holes on the bends were bored during trial assembly of the bridge, considering the tight fitness with splice plates.
- A NC machine was employed for the cutting of the web plates of the floor beams to ensure precision of shape, dimension and root gap of the many slits where stringers crossed floor beams (See Fig. 6).
- Three to six contiguous steel decks were progressively set on the surface plate and assembled to eliminate the deformation of steel deck and ensure the precise facing to the contiguous blocks (See Fig. 7).
- The continuity of welds between the stringers and the deck plate at the crossing point of stringers and floor beams was well ensured as shown in Fig. 8.

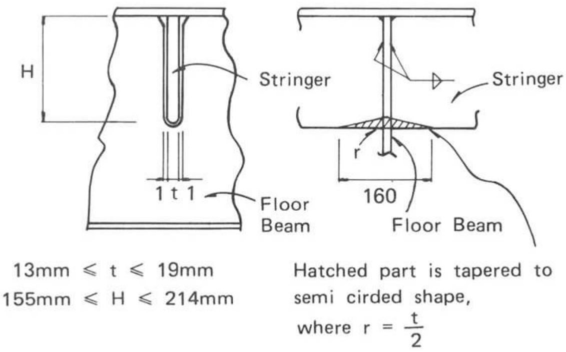


Fig. 6 Details of Slit Where Stringer and Floor Beam Meet

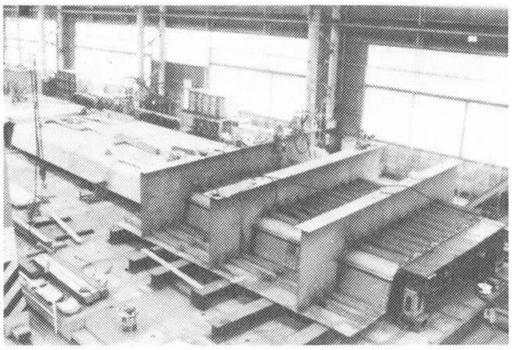


Fig. 7 Assembling of the Blocks

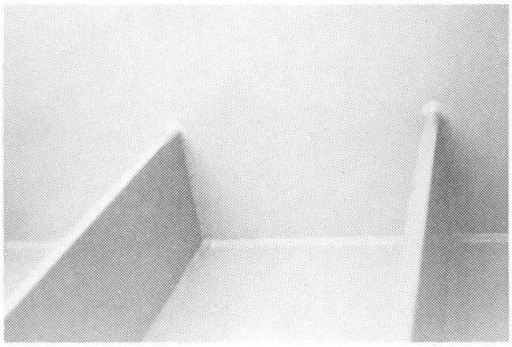


Fig. 8 Weld at the Slit

- To ensure the straightness of the longitudinal axis of each chord, bolt holes of the joints of the floor beams and the main truss were marked and bored after welding was completed. The target tolerance for the distance between both end holes of the floor beam was ± 1.5 mm.

4. POSTSCRIPT

Owing to the appropriate instructions of JRCC and the utmost quality control efforts of the fabricator, the rigorous precision of frame works shown on Table 2, and a high reliability of welding was achieved.
The erection of the Dai-ichi Yumenoshima Bridge was performed in due course and completed in 1986.

Table 2 Comparison Table of Tolerance Values (JRS) and Measured Values

Item	Tolerance (JRS)	Measured Value	
		Range	Mean Value
Span Length	$\pm(5 + 0.15L) = \pm 15$mm	+1 ∿ +2mm	1.6mm
Straightness of the Chord	(3 + 0.1L) = 10mm	-5 ∿ +4mm	-0.4mm
Height of Main Truss	$\pm(4 + 0.5H) = \pm 9.4$mm	-1 ∿ +4mm	1.47mm
Width between Main Trusses	$\pm(3 + 0.5B) = \pm 6$ ∿ 7mm	-5 ∿ +5mm	0.78mm
Camber	(+) 3 + 0.15L (-) 3 + 0.05L = -6 ∿ 12mm	-3 ∿ +7mm	2.35mm

L: Span Length 72m, H: Height of Truss 10.8m
B: Width between Main Trusses 6.45 ∿ 8.5m

5. ACKNOWLEDGEMENT

The authors wish to thank Professor J. Tajima, Faculty of Engineering, the Univ. of Saitama, Japan, for his assistance with the stress analysis of the panel points at the design stage of the bridge.

REFERENCE

1. IKEUCHI K., KOJIMA S., SHIMIZU S.
3-Shukou Daburu Dekki Warren Gata
Torasu Ko Testudokyo no Sekkei Seko
(Designing and Fabricating of 3-Main-Warren-Truss Railway Bridge)
Bridge and Foundation Engineering, March 1986.

Leere Seite
Blank page
Page vide

Quality Control at Erection Site of Iwakurojima Bridge

Contrôle de qualité sur le chantier du pont Iwakurojima

Qualitätskontrolle beim Bau der Iwakurojima-Brücke

Masamitsu OHASHI

Born 1928, received his Dr. civil engineering degree at Kyoto University, Japan. He was engaged in research at Laboratory of Civil Engineering, Ministry of Construction and in construction of big bridge projects like Kanmon Highway and Kojima–Sakaide route. Now, he is Councilor in H. S. B. A.

Minuro MATSUZAKI

Born 1936, received his civil engineering degree at Hokkaido University, Japan. He was engaged in the Construction of Ohnaruto, and Innoshima Bridges and Kojima–Sakaide route. Now he is chief of Second Engineering Department in H. S. B. A.

Chikara MIYASHITA

Born 1943, received his M. E. civil engineering degree at Nagoya University, Japan. He was engaged in Construction of Tozaki Bridge and Kojima–Sakaide route. Now, he is vice chief of the Kojima construction office in H. S. B. A.

Masahiko YASUDA

Born 1945, received his civil engineering degree at Kyoto University, Japan. He was engaged in Construction of Ohnaruto and Iwakurojima Bridges. Now, he is a staff member of 1st construction Bureau H. S. B. A.

SUMMARY

Iwakurojima bridge is a road and railway combined truss type cable-stayed bridge with a field-welded composite steel deck. This report describes the problems associated with field execution of this type of bridge and countermeasures, as well as general quality control.

RÉSUMÉ

Le pont Iwakurojima est un pont en treillis haubané avec tablier métallique composé soudé sur place. *Il sert au trafic routier et ferroviaire. Le rapport traite des problèmes concernant l'exécution* de ce type de pont ainsi que le contrôle de qualité en général.

ZUSAMMENFASSUNG

Die Iwakurojima-Brücke ist eine Fachwerk-Schrägseil-Konstruktion für kombinierten Strassen- und Eisenbahn-Verkehr. Die Stahl-Fahrbahn wurde an Ort geschweisst. Der vorliegende Beitrag beschreibt zunächst die entsprechenden Ausführungsprobleme und die getroffenen Gegenmassnahmen für die Arbeiten an der Fahrbahn und dann die übrigen Massnahmen der Qualitäts-Kontrolle.

1. OUTLINE OF IWAKUROJIMA BRIDGE AND CHARACTERISTICS OF ITS COMPOSITE STEEL DECK

Iwakurojima bridge, currently under construction on the Kojima-Sakaide segment of the Honshu-Shikoku bridge project, is Japan's first combined truss type cable-stayed bridge. At 790m in overall length and 420m in central span length, it is among the largest in the world.
Fig. 1 shows a general view of Iwakurojima bridge. It employs a double-deck truss: the upper deck will carry 4 traffic lanes, while the lower will accomodate an ordinary double-track railway and a planned double-track Shinkansen. Two 11-cable fan-shaped arrangements are employed on each side, anchored at the upper chord member panel point. The steel deck is to be combined with the upper chord member for the following reasons:

- Horizontal component of longitudinal cable tensile force is concentrated on the upper chord side.
- Combined bridges require minimal deformation and stress amplitude, which is achieved by increasing cable regidity. Therefore, upper chord members must have a cross section capable of receiving cable pre-stress.
- Upper chord member thickness and quenched and tempered steel use are reduced, eliminating the need for upper lateral members.

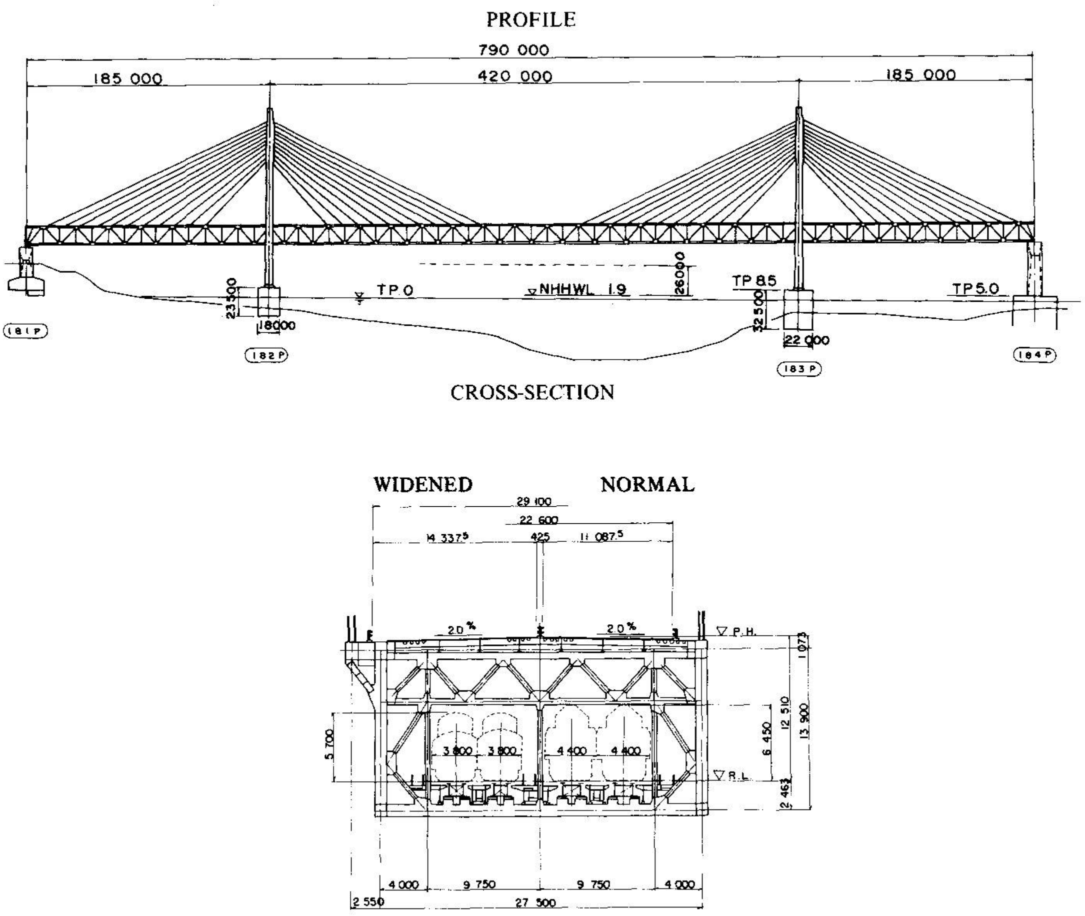


Fig. 1 General View

Steel deck is field welded to minimize damage to pavement structure and steel joint weight.

The following are problems associated with the composite steel deck:

- Field welding requires that the effects of contraction on main truss be considered during design and execution.
- Connection with main and floor trusses requires high member dimensional accuracy.

In view of the above, steel deck is longitudinally divided into three erection blocks, as shown in Fig. 2, each of which are temporarily assembled with all other members in the factory. Holes for interconnection bolts, final processing to plane dimensions and field weld groove accuracy are confirmed.

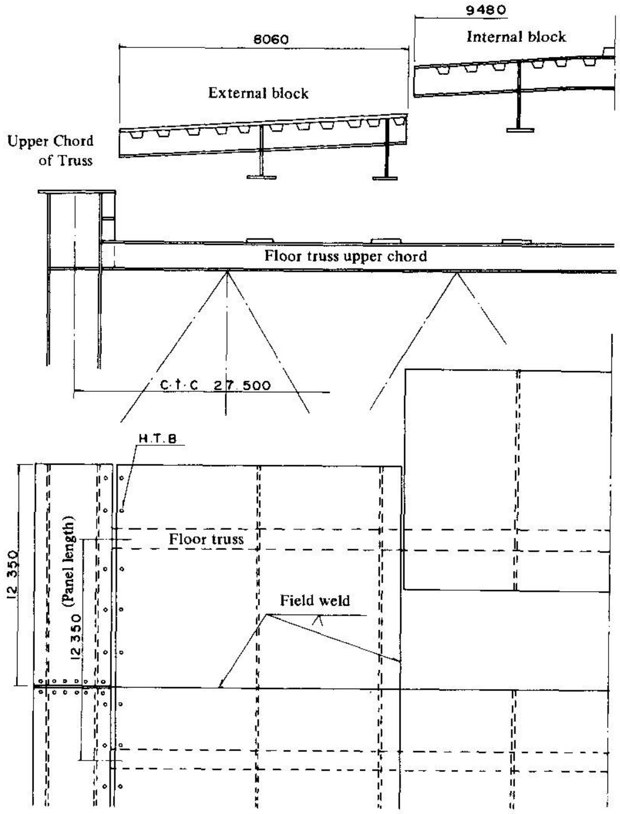


Fig. 2 Composite Steel Deck Construction

2. ERECTION METHOD OUTLINE

The Iwakurojima bridge erection sequence is roughly depicted in Fig. 3. For girder erection, an en bloc erection method is actively employed to minimize the work period and improve accuracy. To resist both dead and live loads, steel deck must be combined with main truss, necessitating simultaneous erection of block and cantilever with main truss at the factory and on the site, respectively. Fig. 4 shows the classification of girder erection methods and steel deck-upper chord member connection methods.

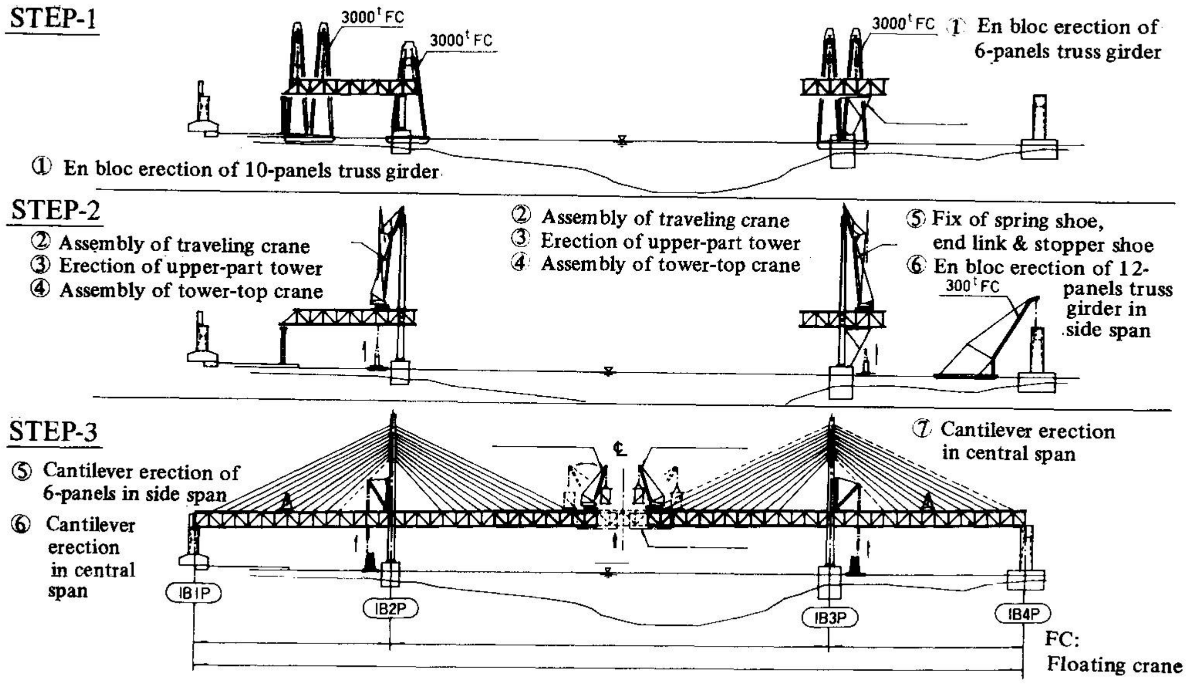
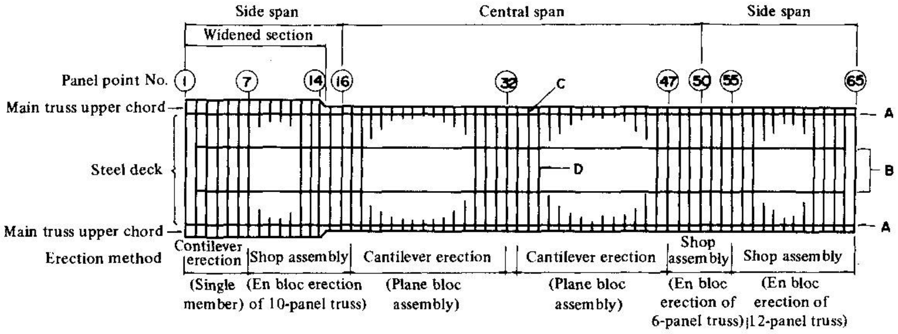


Fig. 3 Illustration of Erection Sequence



Joint \ Panel Point No.			①~⑭	⑭~⑯	⑯~㊼	㊼~53	53~54	54~65	Remarks
Longitudinal direction	A	Between upper chord and steel deck	W	W	B	W	B	W	W: Welding B: H.T.B.
	B	Between steel decks	W	W	W	W	W	W	
Transversal direction	C	Upper chord	W	B	B	B	B	B	
	D	Steel deck	W	W	W	W	W	W	

Fig. 4 Classification of Steel Deck Connecting Methods

At the beginning of 1986, center span cantilever and cables are under construction. Completion of girder erection is planned for the autumn of 1986.

3. DESIGN CONSIDERATIONS RELATING TO MAIN GIRDER ERECTION ACCURACY

3.1 Effect of Contraction Caused by Field Welding on Main Truss Camber

As a rule, the welding sequence shown in Fig. 5 is used in bridge construction to prevent transverse weld from affecting the main truss camber. At the following welding points, however, effects on the main truss camber is unavoidable. To counteract this, appropriate measures are taken in camber fabrication:

- Field welding points on upper chord member upper flange at widened side span
- Closing point in central span
- Closing point in en bloc erection section

3.2 Fabrication and Erection Errors

In designing cross-sections of tower, girder and cable, additional stress is included to compensate for the following fabrication and erection errors:

- Main truss assembly error
- Erection error at closing point
- Tower tilt from vertical
- Cable fabrication and erection error

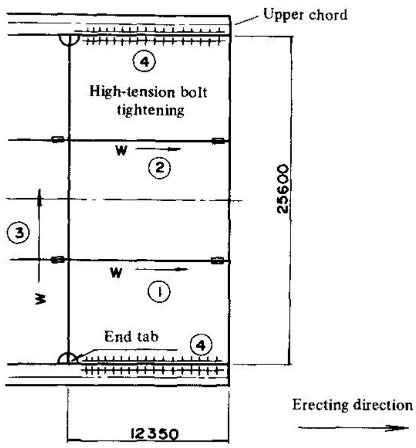


Fig. 5 Steel Deck Connecting Sequence

4. REQUIRED QUALITY AT ERECTION SITE

Erection members fabricated at the factory must meet both quality and accuracy specifications in the Fabrication Standards for Steel Bridges etc. in the

Honshu-Shikoku Bridge Project and other related standards. This section describes these required field erection quality.

4.1 Field Weld Quality

Field welds on steel deck are checked via visual inspection; moreover, X-ray and penetration inspections are carried out for welds on deck plates and trough ribs, respectively, in accordance with the Honshu-Shikoku Bridge Project Standards for Field Welding of Steel Deck.

4.2 Main Girder and Railway Stringer Erection Accuracy

- As concerns main girder field erection accuracy, vertical tolerance for designed camber is determined as follows:

  $$\delta a \leqq \pm\{25 + 0.25(L - 5)\}\text{mm}$$

  where,

  L: span length (m)

  Tolerance for each span is thus determined to be the following: Central span; ±118mm max., Side span; ±59mm max.

- Railway stringer installation accuracy

  In this bridge, railway track is fastened directly to steel stringer upper flange to reduce dead load. The small adjustment allowance for track installation, $-2 \sim +8$, necessitates an installation accuracy control value, as shown in Fig. 6.
  Target camber in this figure, the smooth longitudinal curvature of railway stringer upper flange, is determined according to completed figure of main truss camber after closing.

1	Relative difference between target camber and actual installation height of railway stringer upper flange	10mm max.
2	Relative difference in deviation from target value between adjoining panel points of railway stringer upper flange	6mm max. (Maximum positive error among four points) − (Maximum negative error among four points) ≦ 6mm
3	Relative difference in deviation from target value between 4 points at same panel point of railway stringer upper flange	3mm max. Difference between errors at four points ≦ 3mm

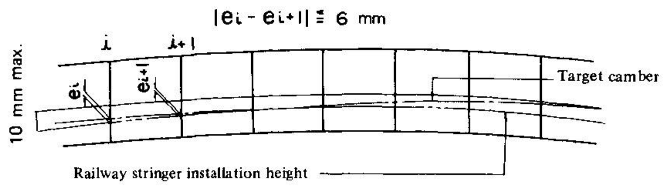


Fig. 6 Railway Stringer Installation Accuracy

5. QUALITY CONTROL AT ERECTION

5.1 Erection Work and Quality Control

The erection method used in bridge construction is outlined in 2. Field quality control for cantilever and cable erection is roughly classified into the following categories:

- Quality control of joints as to welding, bolt connection and painting, etc.
- Control of completed girder figure: camber, linearity, etc.
- Control of cable tension and truss member stress at erection

Fig. 7 shows the overall flow chart for girder and cable erection and quality control which includes the items above.

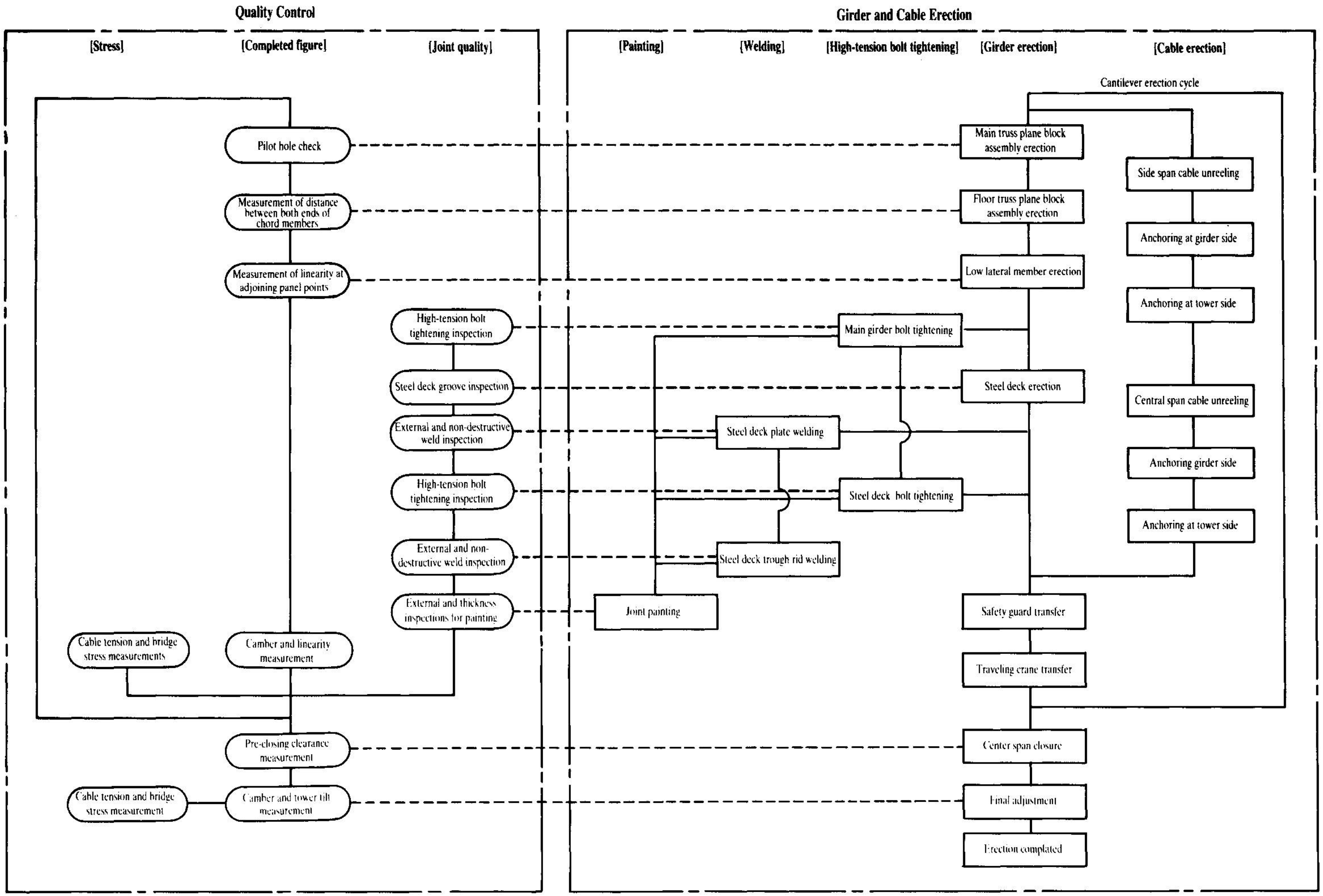


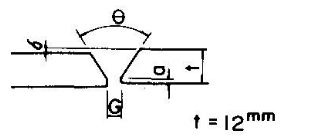
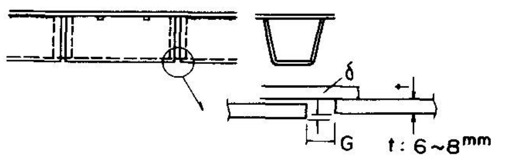
Fig. 7 Overall Flow Chart

5.2 Quality Control for Field Welds

Field weld quality control is roughly classified as follows:

- Welding procedure test under conditions similar to the filed prior to actual erection
- Control of welding conditions and joint geometry
- Quality inspection as specified in 4.

Table 1 Welding Method and Tolerance in Joint Geometry

Weld Point	Welding Method	(mm) G	(mm) δ	(°) θ	(mm) o	Remarks
Deck plate butt weld	Submerged arc welding	3^{+7}_{-3}	0 ± 2	50 ± 5	1 ± 1	
	CO_2-gas arc automatic welding	7^{+3}_{-4}	0 ± 2	50 ± 5	1 ± 1	
Trough rib butt weld	Shielded metal arc welding	10^{+5}_{-1}	0 ± 2			

5.3 Erection Accuracy Control

5.3.1 Erection accuracy control in the field

The following two methods may be used for erection accuracy control of central span girder cantilever and cables:

1. Duplication of factory fabrication accuracy is emphasized; no cable shim adjustment may be made during erection (bolt hole control method); and

2. Optimum cable shim is determined at each erection phase based on the results of erection calculations using configuration, stress, temperature measured in site and erection conditions (measurement control method).

Method 1 has been applied, as a rule, in the construction of Iwakurojima Bridge. Configuration and stress are measured to confirm accuracy during erection and after completion.

5.3.2 Erection accuracy control by adjusting main truss and steel deck bolt holes

In main truss erection, upper and lower chord member bolt holes, into which drift pins are driven during shop assembly, are used as erection pilot holes (into which drift pins are initially driven during erection) to reproduce figures in shop assembly.
In Steel deck welding, both deck plate and upper chord are connected and fixed by driving drift pins into bolt holes drilled during 3-dimensional shop assembly.
After welding, drift pins and temporary connection bolts are removed to release residual stress and thus minimize the effect of steel deck welding contraction on main truss figure.

5.3.3 Configuration and stress measurement

Table 2 shows the measured configuration and stress for the Iwakurojima bridge.

Table 2 Configuration and Stress Measurement

Items to Be Measured	Measurement Point	Measurement Method
Girder figure	Main truss lower chord member, surface	Measure water head using manometer.
Grider linearity	Upper chord member, tip	Measure shift from observation foundation using transit.
Cable tension	Near lower anchoring points	Use tension meter.
Tower tilt from vertical	Tower top	Measure diagonal distance from tower base of 2P and 3P using geodimeter.
Girder stress	Steel deck and lower/upper chord members, panels Nos. 16, 32, 49, 56	Use strain gauge.
Support reaction	Tower link, end link	

5.3.4 Railway stringer erection accuracy

Railway stringer erection accuracy is to be adjusted by machining adjuster plate at stringer supporting point.

6. RESULTS AND DISCUSSION

- The composite structure and erection methods used for this bridge minimize the effect of contraction upon cantilever erection part resulting from field welding of steel deck. Thus even should a composite steel deck be used, erection accuracy equal to that of an ordinary non-composite steel deck or bolt-connected steel deck can be obtained.
- Erection difficulties associated with contraction resulting from field welding of steel deck include the following: difficulty in inserting floor truss due to longitudinal welding; and wider than usual weld root opening at joint with next panel steel deck due to transverse welding. The former may be offset using erection jigs, while the wider root opening in the latter case, as it remains within tolerance, will not be considered a problem in post-welding inspection.
- Final erection accuracy, stress measurement, cable shim adjustment and railway stringer erection accuracy shall be discussed in a subsequent paper.

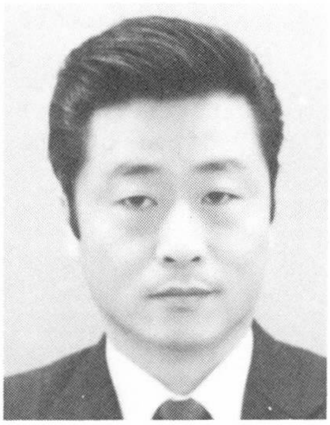
Quality Control in the Erection of Cable-Stayed Bridges

Contrôle de la qualité au cours de la construction des ponts haubannés

Qualitäts-Kontrolle beim Bau von Schrägseilbrücken

Takao YAMAMOTO
Civil Engineer
Sumitomo Heavy Ind., Ltd.
Yokosuka, Japan

Toshio KITAHARA
Civil Engineer
Sumitomo Heavy Ind., Ltd.
Yokosuka, Japan



Takao Yamamoto, born in 1947, received his M.E. degree from Kyoto Univ., 1974 and Eng. degree from Stanford Univ., 1978. He has been involved in bridge engineering works from research to erection at Sumitomo Heavy Ind., Ltd. and is a qualified professional engineer.

Toshio Kitahara, born in 1942, received his M.E. degree from Nihon Univ., 1968. He was engaged in the study of wind engineering at the Univ. of Tokyo in 1970 & 1971 and is a senior engineer for technical problems in bridge engineering at Sumitomo Heavy Ind., Ltd.

SUMMARY

The quality control aspects in the erection of cable-stayed bridges are generally discussed with the presentation in terms of error analysis. This approach is definitely effective if combined with the use of computer controlled measuring instruments. In this paper, however, it is shown that error simulation itself may present rather decisive data, taking the example of the Adhamiyah Bridge constructed in Baghdad, in 1984.

RÉSUMÉ

Le contrôle de qualité lors de la construction des ponts haubannés est généralement basé sur l'analyse d'erreurs. Cette approche par simulation est très efficace, plus spécialement lorsqu'elle est combinée avec l'utilisation d'instruments de mesure commandés par ordinateur. Cette étude, basée sur le pont d'Adhamiyah construit en 1984 à Baghdad, montre que l'approche par simulation peut aboutir à des résultats concluants.

ZUSAMMENFASSUNG

Die Gesichtspunkte der Qualitäts-Kontrolle beim Bau von Schrägseilbrücken werden umfassend auf der Grundlage von Methoden der Fehler-Analyse dargestellt. Kombiniert mit computergesteuerten Messinstrumenten ergibt sich so ein sehr wirksamer Weg der Qualitäts-Kontrolle am Bau. Am Beispiel der 1984 in Bagdad erstellten Adhamiyah-Brücke wird jedoch gezeigt, dass die Simulation von Fehlern und das Verfolgen der Auswirkungen allein schon wesentliche Hinweise für eine zielgerichtete Qualitäts-Kontrolle geben können.

1. INTRODUCTION

Cable-stayed bridges are generally required to take rather consistent erection control because of their highly statically indeterminate structural system.
In other words, as for ordinary type of bridges fabrication errors arising from cutting, welding and assembling are considered to be the main factors for the overall quality control and certain guarantee is given to their completion quality as far as appropriate closure procedure is applied.
As for suspension bridges, hunger tensions are assumed to be uniform over the span just like a distribution of dead load. And the fabrication and erection errors of main cables to be regarded as dominant factors for overall erection quality can be adjusted by hunger length. Therefore, it may be concluded that the fabrication quality can be more or less reflected to the completion stage.
On the other hand, in the case of cable-stayed bridges more unknowns are introduced due to the fact that the cables to be connected between the girder and the tower directly are not installed at the time of shop assembly. Also, theoretical evaluation on the overall non-linear behavior, the effective width etc., becomes more complicated due to the installation of cables.
In this report, error analysis approach to predict influential region due to the errors occurring at each stage is attempted referring to some notable technical features. And practical examples are shown regarding the construction of the first cable-stayed bridge in the Middle East.

2. BASIC APPROACH IN QUALITY CONTROL

2.1 Clarification of Error Factors

At each construction stage different type of errors will occur at different time by different amounts. Therefore, it is almost impossible to clarify all the error factors so precisely. However, conceivable error factors can be summarized in such a characteristic diagram as shown in Fig.1. Among those errors, some outstanding errors may be indicated in consideration of structural characteristics, erection conditions and other empirical factors. In cases, simulation approach can be an effective method to represent their influences quantitatively.

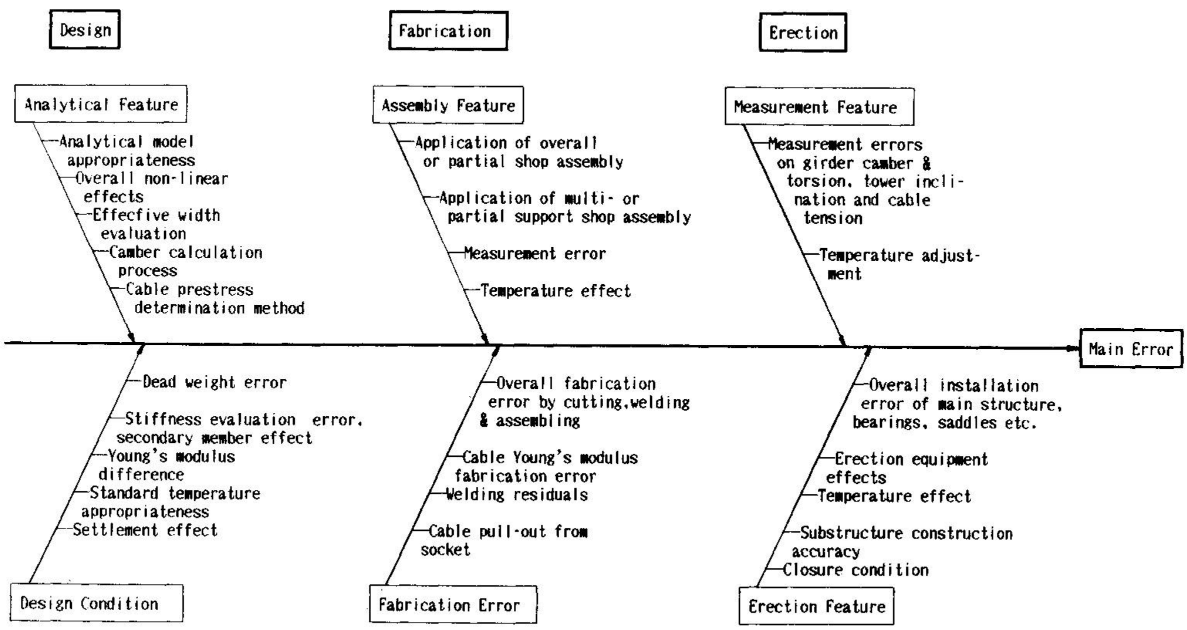


Fig.1 Characteristic diagram for main error factors

2.2 Basic Control Idea on Error Analysis

Those outstanding factors selected in that diagram are stochastically applied in the error analysis on the basis of propagation of errors as described below.

$$E_i = \sqrt{\sum_{j=1}^{m} (\Delta F_{ij})^2}$$

where E_i : total influenced value at point i of the bridge

ΔF_{ij} : influenced value at point i by the error factor j

m : number of error factors

On the other hand, the main objective of quality control is to restrain the deviation of cable tension and profile from calculated values within certain allowable range. And in case of exceeding such range, adjustments are carried out ordinarily to cable length using shim plates or such similar devices either for overall profile or for cable tension.
Hence, it is conceivable to establish some certain bases for erection quality control using the error analysis approach. From this point of view the possible flow chart for that is presented in Fig.2.

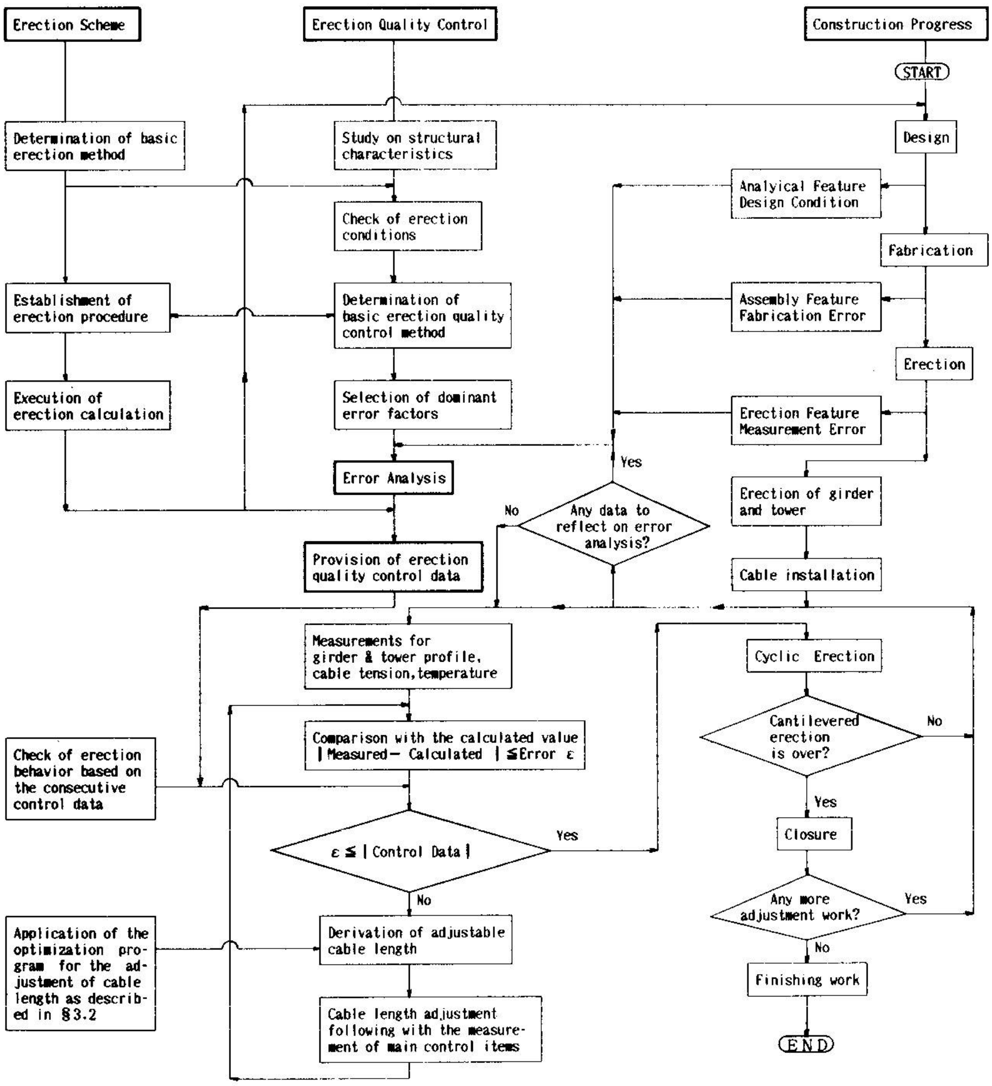


Fig.2 Flow chart for erection quality control of cable-stayed bridges

3. TECHNICAL FEATURES IN APPLICATION

3.1 Non-linear Behavior of Cables

For the non-linear effect of cables the well-known Ernst's equation has been practically applied. But the theoretical discrepancy in that equation was also indicated in its strict sense of application [1]. On the other hand, with the development of analytical approach such non-linear behavior was analyzed on the catenary configuration basis and combined into overall structural analysis [2, 3]. Owing to this, from erection to completion stage consistent erection calculation is to be proceeded once unstressed length is defined. Using this program which is also useful for the error analysis, comparison was made as shown in Fig.3 and Table 1 with the calculation results by the Ernst's equation for the erection of the Adhamiyah Bridge.

From these data it is pointed out that the difference between two approaches attains to 10~20% at the time of cable installation and even at the closure of steel box girder.

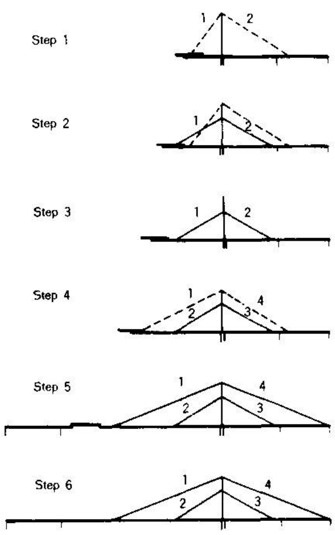


Fig.3 Erection Sequence

Erection Steps	Cable No.	f/ℓ	Cable Section Area (m^2)	Average Cable Tension (KN)	σ (KN/m^2)	Eq×10 (Ernst)	Eeq/Ec
1	1	1/318	0.01467	5935	404564	1.598	0.999
	2	1/217		3855	262662	1.558	0.974
2	1	1/142	0.0967	10290	106468	1.276	0.799
	2	1/139		10546	109058	1.296	0.810
3	1	1/275	0.0967	20052	207403	1.547	0.967
	2	1/270		20503	212033	1.550	0.969
4	1	1/302	0.01467	5690	388103	1.579	0.987
	2	1/292	0.0967	19914	205971	1.546	0.966
	3	1/272	0.0967	20532	212298	1.551	0.969
	4	1/321	0.01467	6092	415395	1.589	0.994
5	1	1/160	0.0394	9094	230839	1.455	0.909
	2	1/216	0.0967	15755	162964	1.494	0.934
	3	1/215	0.0967	16275	168330	1.504	0.940
	4	1/157	0.0394	9153	232261	1.459	0.912
6	1	1/123	0.0394	7004	177649	1.313	0.821
	2	1/215	0.0967	15677	162130	1.492	0.933
	3	1/209	0.0967	15873	164092	1.477	0.934
	4	1/123	0.0394	7220	183182	1.337	0.836

Table 1 Comparison of cable non-linearity

3.2 Profile and Cable Tension Adjustment

In the progress of erection work it is inevitable to provide some adjustments to overall profile or cable tension. Under the condition that the main adjustment is for overall profile, the following equations are applied.

Overall Profile ; $\{\Delta L\} = [A]^{-1}\{\Delta X \pm \alpha\}$

Cable Tension ; $\{\Delta L\} = [B]^{-1}\{\Delta T + \kappa T \pm \beta\}$

Where $\{\Delta L\}$, $\{\Delta X\}$: cable length to be adjusted and profile error
$\{T\}$, $\{\Delta T\}$: calculated cable tension and its error
$[A]$, $[B]$: influence matrices upon profile and cable tension by unit length adjustment
α, β, κ : allowable error residuals and weight coefficient
$\{\ \}$, $[\ ]$: column vector and matrix

In the above equations both $\{\Delta L\}$ do not coincide each other generally, which attributes to the minimization problem of $\sum_{i=1}^{n} (\Delta L_i)^2$ (n: cable numbers).

3.3 Design Considerations

With the development of digital computer, new analytical method for cable-stayed bridges have been developed and have become more accurate as indicated in § 3.1. In the application stage, however, there arises a wide range of design problems as tabulated in Table 2. These items are to be respectively studied according to its structural characteristics, erection features etc., referring to § 2.

Notable Items	Practical Applications	Remarks
1.Evaluation on non-linear behavior of cables	Ernst's modified Young's modulus	10 ~ 20 % of errors are introduced at erection stages.
	Non-linear analysis based on the flexibility approach	Effective for erection stages and long spans bridges.
2.Derivation of cable prestress	1.Derived as additional initial tension by equalizing the bending moment distribution of girder at the completed stage. 2.Derived from balancing of cable horizontal forces at the completed stage 3.Derived from the zero bending moment condition at the closure. 4.Derived from the overall stress check for the whole structure from erection to completion.	The concept of cable prestress is based on the linear analysis and is not applicable for the said non-linear analysis where consistent calculation is possible from erection to completion as cable unstressed length is defined for the completion stage.
3.Evaluation on effective width	Overall width is effective.	Ordinarily, overall width is applied for camber calculation and effective width for stress check.
	Derived from the equivalent span length based on the continuous girder assumption where zero bending moment points are regarded as supports.	At each erection stage and loading case, detailed check calculation is required.
	In the above case cable anchorage points are regarded as supports.	ditto
	Other application such as by shear lag analysis.	Usually applied for bending moments, and for axial forces overall width is applied. Above check is also required.
4.Calculation procedure for fabrication camber	1.By eliminating each load in the opposite way to the erection sequence. 2.By superposition according to the erection sequence. Results are confirmed to be within allowable range by repetitive calculation from derived camber to final profile.	For composite girder, the influence of weight and stiffness of formwork is to be checked.
5.Closure procedure	No closure deformation and stress had better be realized.	In the case of remaining, such influences are to be checked including camber calculation.

Table 2 Design considerations

4. QUALITY CONTROL IN THE ADHAMIYAH BRIDGE CONSTRUCTION

4.1 Structural Aspects

The Adhamiyah Bridge now renamed as the 14th Ramadan Bridge is taken as a practical example. This 4 span continuous composite box girder bridge as shown in Fig.4, has many innovative structural features such as tensile connection system at tower block joints and at brackets cantilevering out by 10.5m from the spine box, prestressed link bearing with rotational capability, diagonal bracing system between the outer girder and the spine box, etc. Especially, the longitudinally hinged connection system at the tower base was considered to have a significant influence on the erection quality control. Also, 750mm jacking down at one pier between the stages of concrete placement affected the fabrication camber calculation along with the evaluation on composite effect of the girder during the erection stages.

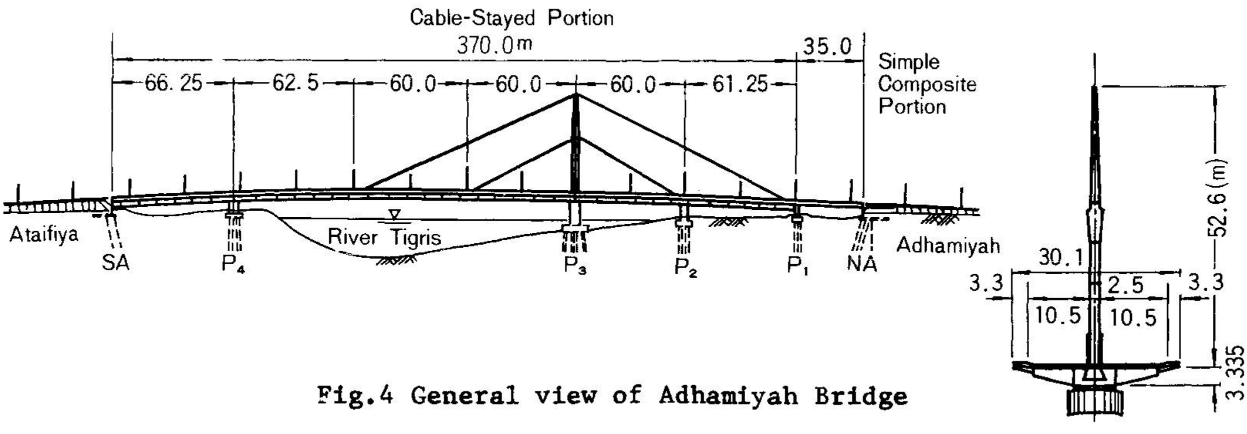


Fig.4 General view of Adhamiyah Bridge

4.2 Erection Features

In the erection sequence as partially shown in Fig.3, following erection methods were applied. Between P1 and P3 the push-out method using the erection girder and between P4 and SA the staging method were adopted respectively. Between P3 and P4 the maximum 162.5m cantilevered erection method was applied as shown in Photo.1. In any method it is noted that the trial assembly was carried out at site with three consecutive boxes to check their fabrication camber. Then, subsequently each one box was separated and advanced to erection front for final setting. Thus, the cantilevering was carried out cyclically using temporary and permanent cables.

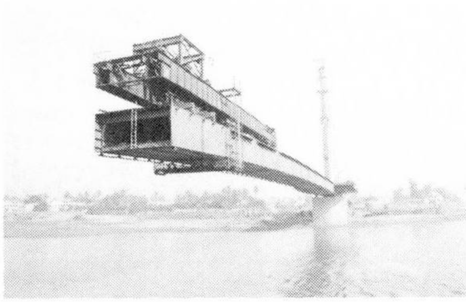


Photo.1 Cantilevered erection

4.3 Outline of Applied Erection Control

Considering the structural characteristics and the erection features as mentioned above, it is pointed out that the profile of the tower designed as an axial member is to be checked to restrain the excessive bending moment due to the unbalanced cable horizontal forces especially at the time of cable installation. Then, referring to this point the erection calculation was executed in details with determining the cable unstressed length to be balanced horizontally.
One example is given for the installation of inner main cables. The erection views before and after the transfer of temporary cable tension to the inner main cables are given in Photos.2 and 3 respectively. In Fig.5 the installation results are presented in terms of cable tension and pulling and slackening lengths. Throughout the erection, the detailed erection calculation was provided in similar way for all other stages. Additionally, as for the measurement, typical equipments such as theodolites, levels and jacks with electrically controlled pump were applied. Also, in order to avoid temperature effect the erection of the spine box was carried out early in the morning. For cables, installation was made according to their marked length and not to tension. Measurements were surely executed before dawn.

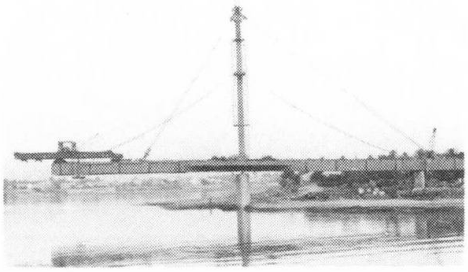


Photo.2 View before the transfer of cable tension

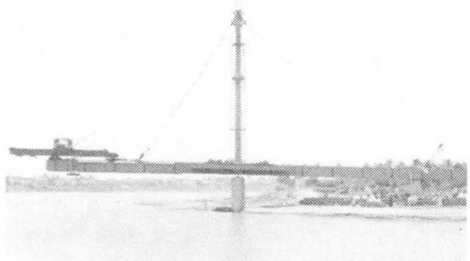


Photo.3 View after the transfer

Thus, on the basis of the above erection procedure and erection calculation the error analysis was applied for important erection steps.

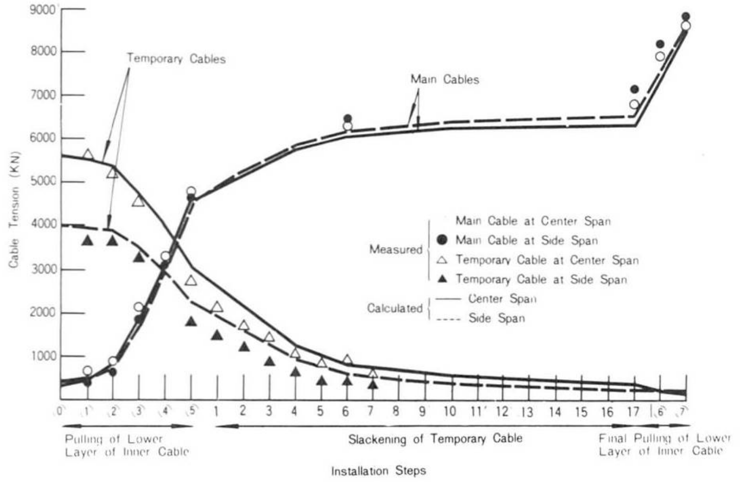


Fig.5 Results for the cable tension transfer

4.4 Practical Example of Error Analysis

Some design and erection features regarding the quality control of this bridge have been presented in the discussion so far. Other notable features were as follows.

(1) 7 cases of fabrication camber calculation were carried out for composite effect of the girder, overall non-linear effects, effective width evaluation, influence of formworks of concrete, erection sequence, etc. including 750mm jacking down at P4. Then, one reasonable solution was adopted from the various cases of results.

(2) The distance between cable anchorages in the girder and the tower was measured at the shop assembly. These results were reflected on the final cable unstressed length.

(3) The standard Young's modulus for locked coil rope was applied for the erection calculation.
The difference from the fabricated one was confirmed to be within $\pm$ 2 % and was reflected upon the error analysis.

(4) The pulling-out of strand from socket was confirmed by the tensile test under 50 % of the breaking load prior to the fabrication. Then, 5mm pulling-out was adopted and reflected upon the final cable unstressed length.

Taking those quality features into account, the error analysis was proceeded according to § 2.1 and 2.2. Some examples are presented in Figs. 6 ~ 9. In these cases cable unstressed length was regarded as the representative error factor.
Namely, those errors with 1/10000 of fabrication accuracy, $\pm$10mm of installation and measurement errors, 5mm of pull-out of strand and others were totaled as $\pm$30mm for the outer main cable and $\pm$20mm for the inner main cables.The results by the error analysis and by the erection calculation and the measurement are given in Figs.6 and 7 respectively for the stage where the installation of the outer main cable was completed. Similarly, in Figs.8 and 9 the results are shown for the stage where the erection of the spine box girder

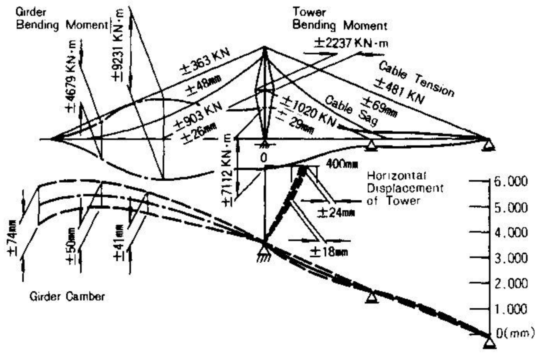


Fig.6 Influence of cable unstressed length error at the outer cable installation

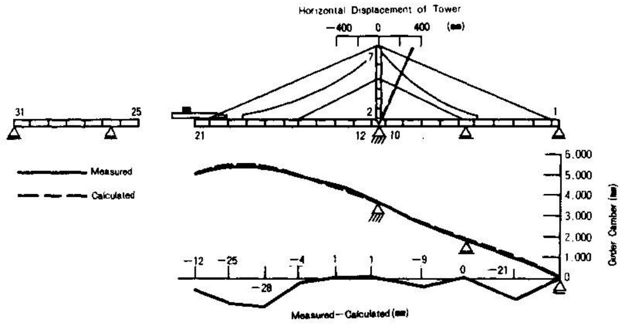


Fig.7 Error distribution between calculated and measured for the above stage

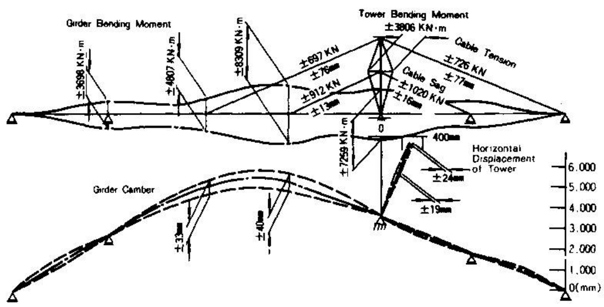


Fig.8 Influence of cable unstressed length error at the completion of spine box

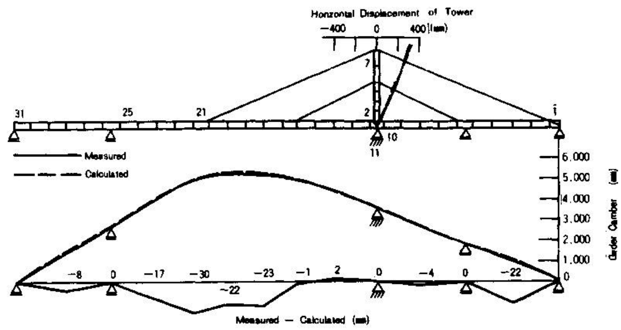


Fig.9 Error distribution between calculated and measured for the above stage

was finished.
From those results, it is known that the distribution of influential values due to the error analysis is similar to those differences between calculated and measured values. In this sense it can be stated that the error analysis itself may give certain criterion for the erection quality control.
Additionally, some other influences were checked as shown in Table 3 for the completion stage of spine box. Also, the final profile after the placement of concrete is shown in Photo.4 where the maximum difference for the girder deflection between calculated and measured values was 66mm at the outer main cable anchorage of the center span.
Considering the fact that any shim plate was never used for the adjustment throughout the erection, it is clear that the final results present very high erection accuracy.

Error		Girder					Tower		Cables			
		Def. (mm)		Bending Moment (KN·m)					Cable Tension (KN)			
Factor	Amount	Outer Cable	Inner Cable	P_4	Outer Cable	P_3	Top Disp. (mm)	B.M. (Max.) (KN·m)	Center Outer	Center Inner	Side Inner	Side Outer
Basic Value	—	147	9	−70592	26664	−22847	280	103	8495	17736	18070	8692
Girder Weight	+5%	−23	16	−5513	3561	−3522	−5	14	167	677	677	177
Girder Stiffness	+10%	−10	1	−481	795	−137	2	20	−88	39	29	−88
Cable Young's Modulus	−2%	7	6	863	1001	−912	−1	9	−98	−39	−49	−88
Temperature Overall	+25℃	0	0	−20	29	69	0	0	−10	10	10	−10
Cable / Girder	15℃ / 0℃	3	1	1972	491	29	−1	−2	20	39	39	20
Cable only	+25℃	75	72	9702	10693	11134	−18	109	−932	−569	−677	−893

Table 3 Influence of other errors

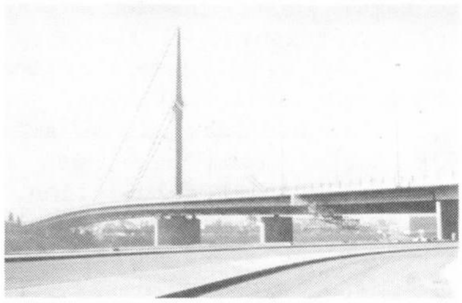


Photo.4 Completion view

5. CONCLUDING REMARKS

The outline of error analysis approach was presented for the erection control of cable-stayed bridges, taking the example of the Adhamiyah Bridge construction. In today's advanced technology, more precise measuring instruments capable of real time treatment will surely be applied in combination with the computer system at site. Considering the recent tendency of increase of multi-cable type stayed bridges, such system will be definitely utilized and contribute to the simplification of quality control works or even total erection works.
On the other hand, as described so far, it is also true that the error analysis based on simulation approach will present decisive data for the erection progress possibility.
When combined with one of those advanced systems by computer, this desk approach will provide more effective way of erection quality control just in the same way as the simulation approach has been a powerful method to the construction of suspension bridges.

REFERENCES

1. M.Ito and Y.Maeda, "Discussion of Commentary on the Tentative Recommendations for Cable-Stayed Bridge Structures", Journal of the Structural Division, ASCE, Vol.104 No.ST2, Feb,1978
2. A.H.Peyrot and A.M.Goulois, "Non-linear Analysis of Flexible Transmission Lines", Journal of the Structural Division, ASCE, Vol.104 No.ST5, May, 1978
3. T.Yamamoto and T.Kitahara,"Non-linear Analysis of Cable-Stayed Bridges under the Sag Change of Cables",Annual Convention of JSCE, 1979(in Japanese)

Safety Control for Large-Scale Underwater Blasting

Contrôle de la sécurité pour les travaux sous-marins réalisés à l'aide d'explosifs

Sicherheits-Überwachung bei grossen Unterwasser-Sprengungen

Hideo SUGITA

Born in 1931, received his Bachelor of Engineering degree at the University of Tokyo. He directed the underwater blasting work as chief of the Sakaide Construction Office of the Honshu-Shikoku Bridge Authority. Currently holds position as managing director for the Bridge and Offshore Engineering Association.

Michio YAMASHITA

Born in 1940, received his Bachelor of Engineering degree at Osaka Institute of Technology. He was in charge of the underwater blasting work as the works manager at the Sakaide Construction Office of the Honshu-Shikoku Bridge Authority. Currently holds position as a section chief of the Third Design Division.

Eiji KATAYAMA

Born in 1940, received his Bachelor of Engineering degree at Shibaura Institute of Technology. He was in charge of the underwater blasting work as the works manager at the Sakaide Construction Office of the Honshu-Shikoku Bridge Authority. Currently holds position as the chief of the Sakaide Construction Office.

Iwao OHTSUKA

Born in 1941, received his Bachelor of Engineering degree at the University of Kyoto. Engaged in the underwater blasting work as the chief of the Engineering Section at the Sakaide Construction Office. Currently holds position as a section chief of the Research Division.

SUMMARY

This paper deals with the safety control for large-scale underwater blasting, based on the findings of the Honshu-Shikoku Bridge Authority from the preliminary surveys, studies, technology developments, of the measurements taken while blastings were in progress, and from surveys and inspections of the field plants.

RÉSUMÉ

L'article traite du contrôle de la sécurité pour les travaux sous-marins de grande envergure réalisés à l'aide d'explosifs. Il se base sur les constatations faites par les Authorités des Ponts de Honshu-Shikoku à partir d'études préliminaires, études et développements technologiques ainsi qu'à partir des mesures effectuées en cours de réalisation et l'étude des installations sur le terrain.

ZUSAMMENFASSUNG

Der Beitrag befasst sich mit der Sicherheits-Überwachung von grossen Unterwasser-Sprengungen beim Bau der Brücken des Honshu-Shikoku-Projekts. Er stützt sich auf Voruntersuchungen, Studien, technische Entwicklungen, Messungen während Sprengungen sowie auf weitere Untersuchungen und Inspektionen vor Ort, die von der Bauherrschaft durchgeführt wurden.

1. OUTLINE

Of the 19 foundations built underwater to support the Kojima-Sakaide route of the Honshu-Shikoku connecting Bridge, blasting was carried out for 8 foundations, at water depths from 0 to 35 meters with tidal current speeds of 2 to 5 knots per hour, using a drill mounted on a self-elevating platform (SEP). Table 1 shows details of the drillings, chargings and blastings carried out.

Features of the North-South Bisan-Seto Bridge are that it is an extremely long, dual suspension bridge designed to straddle the Bisan-Seto traffic route. The structures of its undersea foundations are gigantic. Construction of foundations 6P and 7A could have affected the overall construction period and was hence very important. Other than strict observance to the established construction period, effort had to be made to make sure the following locational and other restrictions did not affect the success of the underwater blastings.

1. Distances of 250 to 600 meters offshore of the Bannosu district, water depths of 14 to 35 meters, and tidal speeds of 2 to 3 knots per hour.
2. Underwater blastings could not be allowed to affect the Asia Kyoseki Sakaide Refinery located adjacent to the Bannosu north revetment.
3. Commercial operation of the refinery was started in 1972. Its production capacity at the time of blasting for foundation 7A in 1979 was 80,000 bbl/day. A production increase up to 100,000 bbl/day was planned at the time of blasting for foundation 6P performed between 1980 and 1981.

Name of bridge		North-South Bisan-Seto Bridge					Iwakurojima Bridge		Hitsuishi-jima Bridge
Site		3P	4A	5P	6P	7A	3P	4P	2P
Depth (m)		-10	-10	-32	-50	-50	-24	-14	-28
Dimension (m)		23x57	57x62	27x59	38x59	75x59	22x46	36x32	24x46
Depth of water (m)		0~5	0~5	22~25	33~35	14~22	3~20	2~10	9~25
Current (knot)		5	5	5	3	2	5	4	4
Depth of drilling(m)		-10	-10	-31.5	-49.5	-49.5	-24	-14	-27.5
Total holes		663	1189	504	768	1632	572	832	432
Drilling interval(m)		2x2	2x2	2x2.2	2x2.25	1.8x2.25	2x2	2x2	2x2.5
Firing		DF	DF	US	EM	EF	DF	DF	EM
Inst. or delay		I	I	I	I	D	I	I	I
Charge (kg)	Max.	2240	3000	2016	1080	1440	1356	1279	480
	Min.	672	720	480	360	960	550	800	240
Total charge (t)		23.5	41.5	12.1	16.3	36.0	17.3	15.0	10.2
Time		15	19	8	21	32	15	13	33
Term		1979.2~ 1980.5	1979.1~ 1980.9	1979.1~ 1979.6	1980.11~ 1981.5	1979.1~ 1979.11	1981.11~ 1982.4	1982.5~ 1982.10	1983.2~ 1983.9

DF: Detonation fuse method
US: Ultrasonic wireless method
EM: Electromagnetic wireless method
EF: Electric firing method
I: Instantaneous firing method
D: Delayed firing method

Table 1 Performance results of underwater blasting

2. TECHNOLOGICAL METHODS FOR SAFETY CONTROL

2.1 Basic policy

At the time of implementing the underwater blasting program in 1971, the authority gave full recognition that it was a matter of cooperation between academic and industrial knowledge, so it commissioned the following organs to 20 preliminary surveys, studies and technological developments so that the large-scale underwater blastings for the foundations of the North-South Bisan-Seto Bridge could be carried out with safety and certainty: The foundation work was the critical area for the entire 9-year construction period.

The Research Institute for Safety Engineering (RISE):
Undertook surveys, studies, and the basic planning of the underwater blastings and issued instructions on measuring techniques.

Japan Industrial Explosives Association:
Mainly concerned with confirmation of explosives performance and instructions on explosives handling techniques.

2.2 Safety control system during work progress

Throughout the long period during which underwater blastings were carried out, surveys were conducted in the vicinity of the centers of blasting, measurements of the underwater pressures and ground vibrations were taken, and surveys and checks on the refinery installations were conducted both before and after each blasting. The data thus obtained was analyzed as speedily as possible. Each blasting followed a discussion meeting participated in by experts from the Research Institute for Safety Engineering, responsible persons from Asia-Kyodo Oil Co., Ltd. and engineers from the contractors and the authority.

The aim of these meetings was to check the results of measurements taken at each blasting so that the extent of the influence of the blasting on the neighboring environment could be discussed and influence decisions to be made for the subsequent blasting program and its implementation. The employment of such a discussion system made it possible to predict the safety of underwater blastings with a certain amount of certainty. The system, therefore, could be considered a technological method effecting the overall safety control of the field work.

3. SURVEYS, STUDIES AND TECHNOLOGICAL DEVELOPMENTS CARRIED OUT BY THE AUTHORITY

3.1 Outline of the 1971-1974 period

This period was slanted mainly toward developing explosives and confirming their performance, as well as conducting a water-tank experiment and in-situ experiment to estimate the influence of underwater pressure and ground vibrations. The first experiment conducted in the Omishima area was intended to confirm the performance of the developed explosives and accessories and for surveying the feasibility of employing the drilling-blasting method using an SEP. Subsequently, computer-aided programs were developed to predict the blast effect from underwater blasting and its influence on the vicinity, which followed in-situ experiments to improve prediction accuracy.

Figure 1 shows the simulation and measured values obtained at a measuring point about 600 meters from the underwater blasting center (about 1 ton) carried out for foundation 6P. The simulation value 2 cm/sec is somewhat higher than one measured value 1.35 cm/sec, but biased on the side of safety as an estimation value used to control the work.

3.2 Outline of the 1975-1979 period

This period was slanted toward selecting, prior to commencing the actual work, the optimum method of underwater blasting and for studying the safety of the

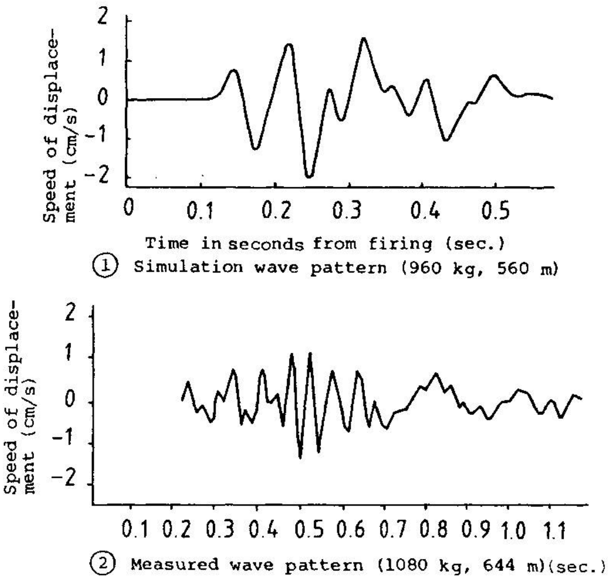


Fig. 1 An example of ground vibrating wave pattern

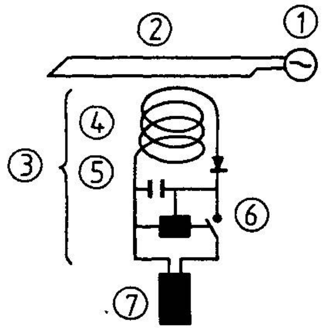


Fig. 2 Schematic diagram of the blaster; ① Oscillator, ② Exciting loop, ③ Blaster, ④ Pick up coil, ⑤ Firing condenser, ⑥ Electric switch, ⑦ Cap.

refinery installations. The offshore test work carried out in 1975 was concerned with the detonation fuse method and delayed firing method to establish drilling, charging and blasting methods and their influence on the adjoining refinery. Subsequently, surveys on the response of the instruments were conducted in the compound of the refinery to study how to secure the refinery installations during the progress of constructing permanent foundations 6P and 7A.

In order to decide the optimum method of firing for foundation 6P, an onshore experiment was conducted to confirm the electromotive force of the electromagnetic firing element. Figure 2 shows a schematic diagram of the electromagnetic firing method. Table 2 shows the contents of surveys and studies concerning the underwater blasting.

Years	Survey item	Development item	In-site experiment and item
1971 ~ 1974	① Performance of explosives and accessories. ② Feasibility of blasting the free side alone. ③ Influence of underwater pressures and ground vibrations on vicinity. ④ Damage to aquatic life and how to reduce it. ⑤ Effect of bedrock blasting. ⑥ Influence on refinery installations.	⑫ Dynamite GX-1, booster, cartridge and other accessories. ⑬ Computer-aided simulation.	Omishima experiments: ①, ②, ③ and ⑫. Water-tank experiments: ④ Bannosu onshore experiments I: ⑤, ⑥ and ⑬

Cont'd

Years	Survey item	Development item	In-site experiment and item
1975 ~ 1979	⑦ Detonation fuse method and delayed firing method. ⑧ Safety of refinery installations. ⑨ Electromotive force of electromagnetic firing element.	⑭ Overburden blasting method. ⑮ Explosives handling manual. ⑯ Electromagnetic firing method.	Offshore test work: ⑤, ⑥, ⑦, ⑭ and ⑮ Experiment at refinery compound: ⑧ Bannosu onshore experiment II: ⑨ and ⑯
1979 ~ 1981	⑩ Measuring of blasting influence. ⑪ Surveys and inspections of refinery installations.	⑰ Safety control system.	Actual work: ⑩ ⑪ and ⑰

Table 2 Surveys, studies and technological developments concerning underwater blasting

4. SAFETY CONTROL FOR LARGE-SCALE UNDERWATER BLASTINGS

4.1 Safety control system

In order for the underwater blasting work for foundations 6P and 7A to be completed safety and securely and in the minimum possible period, it was necessary to establish as large an area as possible for each blasting to make the areas to be blasted uniform, i.e. it was necessary to plan for the largest possible blasting. As the test work in 1975 made clear, the sensitive response of the refinery to the blasting vibrations, from the programmed blastings for actual works (foundations) carried out, first with small amounts of explosives, while confirming their safety, before making the scale of blasting larger.

The objective of discussion meetings was to make an overall analysis of the various data resulting from each blasting so that the safety of the refinery installations could be secured, and that, in addition to confirming the safety of the refinery installations, the conclusions on the amount of explosive for subsequent blasting, the method of measurement, and the time for blasting could be fed back to the works immediately. Table 3 shows the work performance organization by which the underwater blasting work for foundations 6P and 7A could be carried out in safety.

4.2 Measuring of blasting influence

To analyze the extent of the influence of a blasting on the refinery installations, it was necessary to measure the ground vibrations exactly in places where the structures are located. To enable the measuring work and data consolidation to be done efficiently, 6 points were established for measuring ground vibrations, 3 points for structural response and another 3 points for underwater pressures, with consideration given to the distances from the centers of blasting and the types of structures to be secured. These measuring points were established for 7A following the commencement of the blasting work. For the blasting for 6P, the displacements of the compressors and turbines caused by blasting vibrations were added. Figure 3 shows the locations of the measuring points within the refinery installation.

Engineer in charge	Authority and RISE	Contractor	Contractor and Asia Kyodo Oil Co.	Authority, RISE, Contractor and Asia-Kyodo
Contents of work	Survey and blasting programing	Under-water blasting work	Measuring of blasting influence Inspection of refinery installations	Discussion meeting Analysis of measured data Study of extent of influence blasting Review of next blasting plan Amount of charge for next blasting to be decided

Table 3 Underwater blasting work performance organization for 6P and 7A

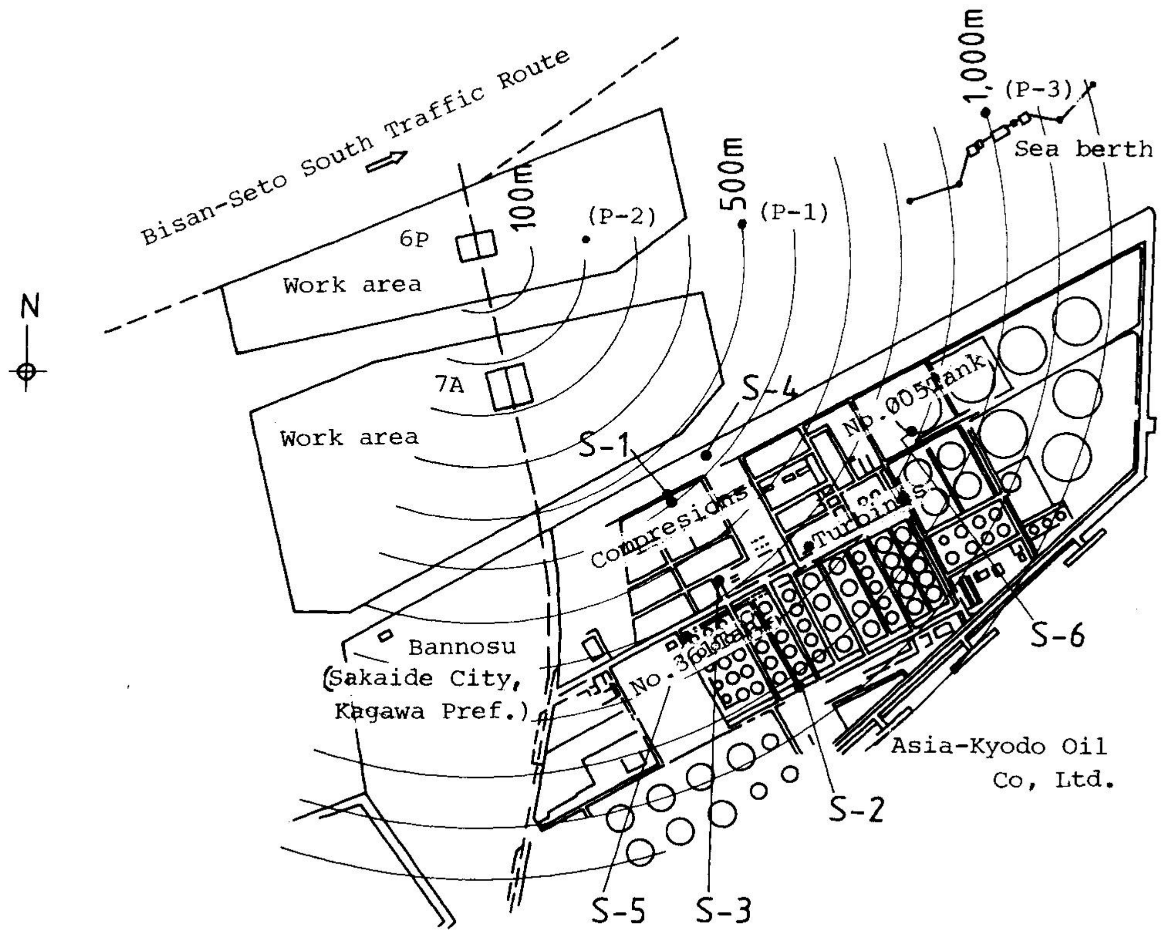


Fig. 3 Locations of measuring points

4.3 Results of refinery installation surveys and inspection

Past survey findings indicate that the aseismatic resistance of the refinery installations are problematic in that the instrumentation system consisting of level gages, flowmeters, pressure gages and other instruments malfunctions sometimes, partly due to the locations of the instruments in place and their supporting methods. For this reason, with the cooperation of the oil company, instruments were installed by means of an improved method, and safety inspections using a check sheet were conducted before and after each subsequent blasting. Table 4 shows the total number of the instruments suspected to have been affected by the blastings for 6P and 7A, and the maximum ground vibration values measured at points S_1 and S_5. Those effected by blasting are the instruments indicating some fluctuations in their commands and records, and they include those which showed secondary response to erroneous signals transmitted from the instruments affected.

Judging from the findings of the influence from the blastings and from the findings of surveys and inspections of the refinery installations, none of the instruments themselves was damaged, and therefore the blastings had virtually no effect on the operation of the refinery.

Site	Instrument affected by blasting	Maximum ground vibration value (Vertical component)	
		S_1 (distance)	S_5 (distance)
6P	74 units	1.9 cm/sec (650 m)	0.9 cm/sec (860 m)
7A	90 units	2.6 cm/sec (365 m)	1.0 cm/sec (580 m)

Table 4 Survey and inspection results of refinery installations

5. CONCLUSION

Underwater blastings for foundations 6P and 7A were able to be successfully completed 53 times without accident or disaster. However, the inspection of the refinery installations conducted after the 15th blasting for 6P showed that one of the heating pipes inside the reheating furnace for the kerosene desulfurizing equipment was disconnected from the guide support. The amount of explosives was reviewed at a discussion meeting held after the blasting to produce more intensified observation measures, and others, to be reflected in subsequent blastings.

The safety control system implemented this way before and after each blasting was perfect, and made it possible to discover influence on the refinery installations at the early stage, leading to the successful employment of effective measures and to a possible shortening of the long-term construction period.

The technological methods of handling such a safety control system was usefully employed in subsequent blastings for the Honshu-Shikoku Bridge Project, leading to the completion of all the blastings without accident as shown in Table 1.

Sicherheitsanforderungen und Qualitätssicherung im Kernkraftwerksbau

Safety Requirements and Quality Assurance for Nuclear Power Plants

Exigences de sécurité et assurance de la qualité dans la construction de centrales nucléaires

Wolfgang ERDMANN
Diplom-Ingenieur
Ver. Elektrizitätswerke Westfalen
Dortmund, Bundesrep. Deutschland

W. Erdmann, Jahrgang 1936, Studium an der Technischen Universität Berlin. Nach Tätigkeit bei den Berliner Wasserwerken und Elektromark Hagen seit 11 Jahren verantwortlich für den Hoch- und Ingenieurbau bei VEW Dortmund, einem der führenden Energie-Versorgungsunternehmen der Bundesrepublik Deutschland.

ZUSAMMENFASSUNG

Anhand der Befestigungtechnik mittels Dübeln werden die besonderen Anforderungen an die Qualitäts-Kontrolle im Kerntechnischen Ingenieurbau erläutert und deren Umsetzung in die Praxis dargestellt. Auf wirtschaftliche Konsequenzen wird hingewiesen.

SUMMARY

In connection with the fastening technique (dowel securing) the special demands on quality assurance of nuclear power constructions will be explained and the application into practice will be described. Economic aspects are also discussed.

RÉSUMÉ

Les exigences particulières du contrôle de la qualité dans le génie nucléaire sont expliquées et illustrées à l'aide de la technique de fixations et d'assemblages. Les applications dans la pratique et leurs conséquences économiques sont expliquées.

Durch die Forderung, Gebäude und Anlagen auch gegen die, wenn auch nur mit sehr geringer Wahrscheinlichkeit zu erwartenden Belastungen

- Flugzeugabsturz,
- Erdbeben,
- Druckwelle,
- Einwirkungen von innen,

auszulegen und die dabei unterstellte Auswirkung des Versagens eines Bauteils in Form von Austreten radioaktiver Stoffe in die Umwelt auf einen Umkreis der Anlage, der weit über den örtlichen Bereich des eigentlichen Schadens hinausgeht, sind die Anforderungen an die Sicherheit und Qualitätssicherung beim Bau von kerntechnischen Anlagen weitaus höher als im normalen Baugeschehen.

Folge davon ist, daß Bauteile, die in der konventionellen Baupraxis ohne Zulassung verwendet werden dürfen, aber auch bauaufsichtlich zugelassene Bauteile bei Verwendung im Kernkraftwerksbau einer besonderen Zustimmung und einer meist daraus resultierenden besonderen Qualitätskontrolle unterliegen. Beispielhaft dafür ist die Befestigungstechnik. Normalerweise interessiert dieser Bereich den Bauingenieur nur bei der örtlichen Eintragung großer Lasten in statischer Hinsicht. Aus sicherheitstechnischen Überlegungen wird im Kerntechnischen Ingenieurbau für die Anbringung von Komponenten, Rohrleitungen, Kabelpritschen und anderen Bauteilen grundsätzlich eine Befestigung über Ankerplatten verlangt. Diese Ankerplatten sind mit Kopfbolzen im Bauteil verankert. Daraus folgt, daß deren Lage, Anzahl, Abmessungen und Tragkraft nicht nur vor Beginn der Bauarbeiten, sondern bereits bei Aufstellung und Prüfung der statischen Berechnung festliegen müssen. Bei diesem Vorgehen kann es - bei einer Errichtungszeit von rd. 70 Monaten - nicht ausbleiben, daß nachträgliche Änderungen bezüglich Lage und Tragkraft sowie Ergänzungen notwendig werden. Diese sind dann nur über Dübelbefestigungen möglich und zulässig. Hier verlangt die Behörde auch für Dübel, für die eine allgemeine bauaufsichtliche Zulassung vorliegt, wegen der besonderen Lastfälle im Kernkraftwerksbau eine Zulassung im Einzelfall.
Eine solche Zulassung hat in der Bundesrepublik Deutschland folgende Bedeutung:

Grundsätzlich dürfen im Bauwesen in der Bundesrepublik nur bauaufsichtlich zugelassene oder mit einem besonderen Prüfzeichen versehene Bauteile verwendet werden. Liegt eine solche Zulassung oder ein Prüfzeichen für das vorgesehene Bauteil nicht vor oder umfaßt die Zulassung nicht die zu erwartenden Belastungszustände, so kann die Bauaufsichtsbehörde, die für die Genehmigung des Bauvorhabens verantwortlich ist, die Verwendung solcher Bauteile ablehnen. Der Antragsteller einer Baugenehmigung kann dann die genehmigende Behörde bitten, eine "Zustimmung im Einzelfall" zu beantragen. Diese Behörde, die sogenannte "Untere Bauaufsichtsbehörde", beantragt dann bei ihrer zuständigen (oberen) Landesbehörde eine Zulassung des entsprechenden Bauteiles für das geplante Bauvorhaben.

Häufig werden dem Antrag seitens des Antragstellers bereits Gutachten von Material-Prüfungsanstalten, Hochschulinstituten o. ä. beigefügt. Die Oberste Bauaufsichtsbehörde beauftragt dann im allgemeinen das "Institut für Bautechnik" in Berlin mit der Begutachtung aller eingereichten Unterlagen. Bei positiver Stellungnahme des Instituts kann dann die Obere Bauaufsichtsbehörde der Unteren Bauaufsichtsbehörde die beantragte Zulassung im Einzelfall aussprechen. Die Untere Bauaufsichtsbehörde teilt diese Zustimmung dem Antragsteller mit. Alle eingereichten Unterlagen sowie die Stellungnahme des Instituts für Bautechnik werden Bestandteil der Baugenehmigung für die Verwendung der Bauteile. Wie der Name schon sagt, handelt es sich um eine "Zustimmung im Einzelfall", d. h. Zustimmung zu einem bestimmten Produkt (nicht System) für die An-

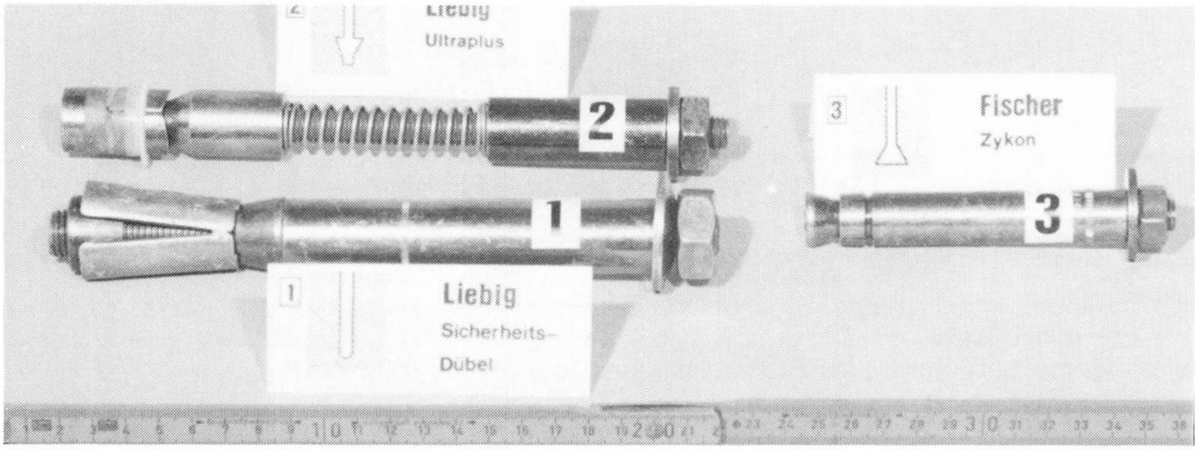
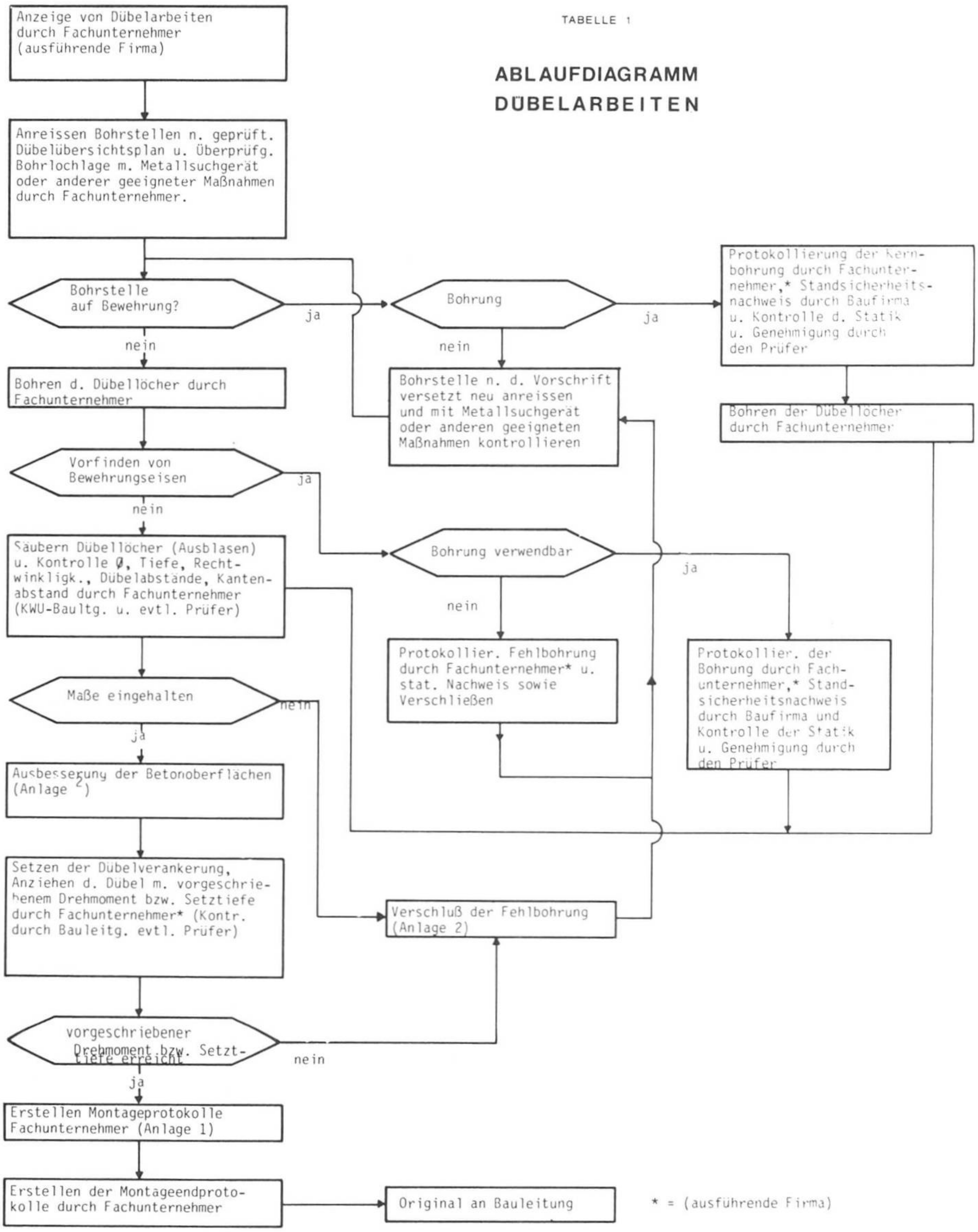


Fig. 1: Schwerlast-Dübel

TABELLE 1

ABLAUFDIAGRAMM DÜBELARBEITEN



wendung bei einem einzelnen, bestimmten Bauvorhaben. Die Zustimmung im Einzelfall bedeutet also nicht eine allgemeine bauaufsichtliche Zulassung.

Ein solcher Antrag auf Zulassung im Einzelfall mußte nun auch für die Verwendung von Dübeln beim Bau von Kernkraftwerken beantragt werden. Dazu wurde vom Ersteller des Kernkraftwerkes eine umfangreiche "zusätzliche Gütevorschrift" erstellt. Darin werden u. a. festgelegt

- der Anwendungsbereich für verschiedene Dübelarten in Abhängigkeit von

  der sicherheitstechnischen Bedeutung der Anschlußkonstruktion,
  der Last pro Dübel,
  der sicherheitstechnischen Bedeutung des betreffenden Bauwerks,
- allgemeine Planungs- und Ausführungsgrundsätze,
- die Qualitätssicherung bei ausführender Firma (Eigenüberwachung) sowie
- Fremdüberwachung und Dokumentation.

Aus der tabellarischen Übersicht ist die Komplexität der Aussagen und Anforderungen deutlich zu erkennen. (Tabelle 1)

Diese zusätzliche Gütevorschrift wurde gemeinsam mit den Prüfberichten verschiedener Hochschul-Institute über die Verwendung der zum Einsatz geplanten Dübel der Unteren Bauaufsichtsbehörde eingereicht.

Die Untere Bauaufsichtsbehörde hat diese Unterlagen an die Obere Bauaufsichtsbehörde weitergeleitet. Diese hat dann das Institut für Bautechnik in Berlin mit einer Stellungnahme beauftragt. In der Stellungnahme des Instituts für Bautechnik wurden neben allgemeinen Aussagen, z. B. über die Art der zulässigen Belastung ("vorwiegend ruhend"), die Zahl der maximal zulässigen Lastwechsel während der Lebensdauer des Gebäudes, Mindestbewehrung, Aussagen über die Herstellung der Bohrungen, die Behandlung von Fehlbohrungen, die Einschaltung des bautechnischen Prüfers und die Aufgaben des Fachbauleiters des Bauherrn festgelegt. Zusammen mit den schon zitierten Gütevorschriften des Erstellers des Kernkraftwerkes, den beigefügten Gutachten der Hochschulinstitute, der Stellungnahme des Instituts für Bautechnik und den Auflagen der unteren Bauaufsichtsbehörde in der Baugenehmigung für "........Dübelarbeiten........ sowie Zulassung im Einzelfall für die Verwendung von Dübeln zur Verankerung von Bau- und Anlageteilen......." ergab sich für die Verwendung von Dübeln im Kernkraftwerksbau hinsichtlich der Sicherheit und der Qualitätssicherung folgendes Bild:

1. Die mit Dübeln zu verankernden Lasten müssen vorwiegend ruhend sein, wobei Belastungen aus Erdbeben als vorwiegend ruhend aufgefaßt werden dürfen.
2. Bei schwellender Belastung dürfen während der Lebensdauer des Bauwerks höchstens 10^4 Lastwechsel auftreten.
3. Die Dübel müssen mit der vorgeschriebenen Verankerungstiefe in die Druckzone eines Bauteils einbinden.
4. Vor Beginn der Bohrarbeiten ist die Lage der Bewehrung festzustellen, um Fehlbohrungen oder Beschädigungen in der Bewehrung zu vermeiden.
5. Fehlbohrungen müssen mit hochfestem Material wieder geschlossen werden.
6. Alle Dübelarbeiten dürfen nur von Firmen mit besonderer Qualifikation ausgeführt werden.
7. Der Bauherr hat einen geeigneten Fachbauleiter zur Kontrolle der Dübelarbeiten zu stellen.

8. Alle Dübelarbeiten sind zu dokumentieren, wobei neben Art des Dübels, Lage und Zeitpunkt der Arbeiten auch festzuhalten sind:
   - Nachweis der Betonfestigkeitsklasse,
   - Prüfung auf winkelrechte Bohrung,
   - Reinigung der Bohrlöcher,
   - Bohrlochabmessungen,
   - aufgebrachtes Drehmoment,
   - Lage von Fehlbohrungen.

   Diese Dokumentationen sind 5 Jahre aufzubewahren.

9. Es dürfen nur Dübelfabrikate verwendet werden, für die eine Zulassung im Einzelfall vorliegt. Diese Zulassung bezieht sich ausdrücklich auf ein Fabrikat, nicht auf einen Dübeltyp.

   Anmerkung:

   Daß diese Vorschrift ein Monopol des Herstellerwerks sowohl hinsichtlich der Preise als auch der Liefermöglichkeiten nach sich zieht, ist leicht einzusehen, da ein Wechsel auf ein anderes Konkurrenz-Fabrikat nicht mehr möglich ist.

10. Der Typ des verwendbaren Dübels richtet sich
    - nach sicherheitstechnischer Bedeutung des Bauwerkes (sicherheitstechnisch relevant oder nicht relevant),
    - nach sicherheitstechnischer Relevanz des zu befestigenden Bauteils,
    - nach vorgesehener Lastaufnahme

    $p < 0{,}2$ kN

    $p < 0{,}2 < 1{,}5$ kN

    $p > 1{,}5$ kN

Für Verankerungen sicherheitstechnisch relevanter Bauteile sind zusätzlich vorgeschrieben

- der Nachweis, daß für den Lastfall Betriebslasten außerhalb des Betons keine Biegebeanspruchung auf den Gewindebolzen im Dübel wirkt,
- die Einschaltung des bautechnischen Prüfers,
- Aufbringung des vorgeschriebenen Drehmoments auf die Schraube des Dübels frühestens nach 2 - 3 Stunden,
- Anzeige des Beginns der Dübelmontage bei der Bauaufsichtsbehörde,
- Einhaltung bestimmter Achs- Randabstände der Dübel untereinander, auch bei Dübelgruppen,
- Nachweis der Begrenzung der Rißbreiten in Höhe der oberflächennahen Bewehrung bei Sicherheitserdbeben und Flugzeugabsturz auf max. 2 mm.

Für Einspanndübel läßt die Behörde gegenüber des bei Angabe der Normaltragkraft unterstellten Sicherheitsfaktors von 1,7 eine Verringerung für den Lastfall Betriebslast mit Auslegungserdbeben auf 1,4 sowie Kombinationen mit Sicherheitserdbeben, Flugzeugabsturz und vergleichbaren inneren Störfällen auf 1,0 zu.

Nach Festlegung des Dübeltyps für die Befestigung bestimmter Anlageteile an bestimmten Stellen und gegebenenfalls erforderlichen Nachweisen entsprechend den vorgenannten Punkten ergibt sich entsprechend den Auflagen für die Tätigkeit auf der Baustelle der im Ablaufdiagramm dargestellte Vorgang. (Tabelle 2)

Die strenge Eignungsprüfung der zur Anwendung vorgesehenen Dübel, die genauen Vorschriften über die Art der Anordnung und die geforderten Nachweise hinsichtlich Tragfähigkeit, Beanspruchung des Bauteils und Rißbreite sowie die gezeigten Vorschriften über die Durchführung der Arbeiten, deren Überwachung und Dokumentation können als ein Musterbeispiel sicherheitstechnischer Auslegung, d. h. der Qualitätssicherung bei der Ausführung angesehen werden. Entscheidend ist jedoch, daß das aufgezeigte und im Ablaufdiagramm zusammenfassend dargestellte Verfahren in dieser Form auch in der Praxis zur Anwendung kam. Das Fließschema zeigt auch die lückenlose Qualitätskontrolle aller Vorgänge auf der Baustelle im Zusammenhang mit Dübelarbeiten in Form von Eigen- und Fremdüberwachung. Technisch und verfahrenstechnisch erfüllt also dieses Verfahren höchste Anforderungen an Sicherheit und Qualitätssicherung im Bauwesen.

Für die Praxis sind solche Betrachtungen jedoch unvollkommen, wenn nicht auch die wirtschaftlichen Aspekte einer Prüfung unterzogen werden. Wie auf jedem anderen Gebiet läßt sich die Sicherheit beliebig hoch, oder anders ausgedrückt, der Grad der verbleibenden Unsicherheit beliebig verringern, sofern man wirtschaftliche Betrachtungen außer acht läßt.
Auch bei nur oberflächlicher Betrachtung der hier vorgestellten Verfahren zur Auswahl der Dübel und deren Einsatz auf der Baustelle wird deutlich, welcher wirtschaftliche Aufwand mit dem Setzen eines einzigen Dübels verbunden ist. Dabei wird der Materialwert oder Einkaufspreis des Dübels selbst sowie die Kosten für den reinen Vorgang des Bohrens und Einsetzen des Dübels kostenmäßig völlig uninteressant gegenüber dem Aufwand für Prüfung, Zulassung, Kontrolle und Dokumentation. Aufgrund von Nachkalkulationen errechnete Mittelwerte für diesen Aufwand bei einem Dübel haben Beträge zwischen 400,-- und 1.000,-- DM pro Stück ergeben.

Für ein Bauvorhaben wie ein Kernkraftwerk, in dem rd. 80.000 Ankerplatten angeordnet werden, bedeutet das bei einer - angenommenen - Fehlerquote von nur 5 % rd. 4.000 nachträgliche Ankerplatten.
Würden diese mit jeweils nur 4 Dübeln befestigt, sind dies 16.000 Dübel mit einem Aufwand von i. M. etwa 700,-- DM/Stück. Der Betrag für die nachträgliche Arbeit erreicht schon dann die Größenordnung von 11 Mio DM. Damit wird deutlich, daß der Preis für den Dübel selbst überhaupt keine Rolle spielt, d. h. daß es wahrscheinlich wesentlich weniger aufwendig, jedoch einfacher praktikabel, und damit wirtschaftlicher ist, ausschließlich die vom Stück-Preis her zwar teuersten, jedoch auch tragfähigsten Dübel einzusetzen, um das Risiko der Anordnung falscher, d. h. zu schwacher oder nicht zulässiger Dübel, durch Verwechselung oder nachträgliche Laständerungen zu minimieren.

Damit würde jedoch andererseits die bereits angesprochene Monopol-Stellung eines Dübelherstellers noch erhöht. Abhilfe wäre nur dadurch denkbar, daß verschiedene Hersteller eine allgemeine bauaufsichtliche Zulassung - zur Verwendung im Kernkraftwerksbau - für ihr Produkt erreichen und damit wieder eine Konkurrenz-Situation entsteht.

Wirtschaftlich wäre das aber für den Hersteller nur dann interessant, wenn er mit einer entsprechenden Nachfrage rechnen kann. Die ist jedoch - zumindest gegenwärtig bei der Situation der Kernenergie in der Bundesrepublik - nicht zu erwarten.

Übersicht der Planungs- und Ausführungsbedingungen für Dübelverbindungen (Quelle: KWU/D)

Tabelle 2

Planungs- und Anwendungskriterien	Last pro Dübel: D					
	sicherheitstechnisch <u>nicht</u> relevante Bauwerke			sicherheitstechnisch relevante Bauwerke		
	$D \leq 0{,}2$ kW	$0{,}2 < D \leq 1{,}5$ kW	$D > 1{,}5$ kW	$D \leq 0{,}2$ kW	$0{,}2 < D \leq 1{,}5$ kW	$D > 1{,}5$ kW
1	2	3	4	5	6	7
Zulässige Dübel	Decken: Metalldübel mit Zugzonen-zulassung Wände: wie in Decken und handelsübliche Kunststoffdübel Größe 4 bis 8	Metalldübel mit Zugzonenzulassung und Verbundanker z.B. - Liebig-Si-Dü M 8, M 10 - Fischer Zykon M 6 + M 10 (2) - Fischer Automatik (2) FA M 10+M 16, FAC M 10+M 12 - Hilti HSL M 8, M 10 (2) - Spitfix Anker M 10, M 12 (2) - Upat-Si-C M 8, M 10 (2) - Verbundanker (Fischer, (3) Hilti, Liebig, Upat)	Hilti HSL M 12 Liebig-Si-Dü M10-M20 Fischer Zykon M 12 Verbundanker (Fischer, Hilti, Liebig, Upat)	Fischer Zykon M 6 - M 10	Fischer Zykon M 10 (4) Fischer Zykon M 12	Liebig-Einspannanker Ultra-plus M 12 Fischer Zykon M 12
Zulassung für Dübel	Metalldübel: ja Kunststoffdübel: nein	ja	ja	ja	M 10: ja M 12: liegt noch nicht vor	Liebig-Dü: ja Fischer-Dü: liegt noch nicht vor
Zustimmung im Einzelfall erforderlich (wird pro KKW von OBB erteilt)	nein	Liebig M 10: ja sonst: nein	ja Verbundanker: nein	nein	ja	ja
stat. Nachweis erforderlich	nein	nein (1)	ja	nein	nein (1)	ja
Planungsunterlagen an Prüfing. f. Baustatik	nein	nein	ja	nein	nein	ja
KWU-Dokumentation	nein	ja	ja	nein	ja	ja
Dokumentation an Bauüberwachung	nein	nein	ja	nein	nein	ja
KWU-Überwachung der Dübelarbeiten	ja (Stichproben)	ja (Stichproben)	ja	ja (Stichproben)	ja (Stichproben)	ja
Überwachung durch Bauüberwachung (LGA-La)	nein	nein	ja	nein	nein	ja
Nachweis über Dübelausfall	nein	nein	nein	nein	nein	nein

(1) Bei tragenden Konstruktionen (2.1.1.2 a) ist ein statischer Nachweis zu führen. Der Nachweis kann entfallen, wenn durch Verlegevorschriften, die typisierte Nachweise enthalten, die Tragfähigkeit sichergestellt ist.

(2) Zulässige Dübeltragfähigkeit nur zu 75 % ausnutzen (zul $D = 0{,}75 \cdot 1{,}5 = 1{,}12$ kW) bei höherer Ausnutzung ist statischer Nachweis und Planungsunterlagen an bautechnische Prüfer erforderlich.

(3) <u>NUR</u> begrenzt im Einzelfall für nachgewiesene Betondruckzone verwendbar.

(4) Zulässige Dübeltragfähigkeit nur bis $D = 1{,}0$ kW/Dübel gemäß Zustimmung im Einzelfall durch OBB-Bayern vom 11.10.1984.

Leere Seite
Blank page
Page vide

Monitoring of a Historical Wall Painting

Surveillance d'une peinture murale historique

Überwachung eines historischen Freskos

Antonio MIGLIACCI
Prof. of Civil Engineering
Technical University
Milan, Italy

G. MIRABELLA ROBERTI
Engineer, doctoral student
Technical University
Milan, Italy

Carlo MONTI
Prof. of Surveying
Technical University
Milan, Italy

Luigi MUSSIO
Prof. of Applied Geodesy
Technical University
Milan, Italy

SUMMARY
A continuous check has been made on the masonry wall on which Leonardo painted the «Last Supper», recording measurements of temperatures, displacements, forces, etc. over about two years. Some of these quantities were chosen to perform a statistical analysis of relevant physical relationships; in particular the time dependence of the displacements has been carefully studied. The present work illustrates the statistical methods and the relationships found and gives an assessment on the safety of the painting on the basis of the examined data.

RÉSUMÉ
Un contrôle systématique a été fait sur la paroi en maçonnerie sur laquelle Leonardo a peint «L'Ultima Cena» à Milan. Des mesures de température, de déplacement, de force, etc. ont été enregistrées pendent deux ans. Certaines de ces valeurs ont été choisies pour effectuer une analyse statistique des relations physiques existant entre elles; l'évolution des déplacements dans le temps a été étudiée soigneusement. Le texte explique les méthodes statistiques employées et les relations physiques trouvées, et formule des jugements sur la sûreté de l'œuvre d'art à partir des données examinées.

ZUSAMMENFASSUNG
Während zweier Jahre wurden am Mauerwerk, welches Leonardo da Vinci's berühmtes Bild «Das letzte Abendmahl» trägt, kontinuierlich Temperatur-, Verschiebungs- und Kräftemessungen durchgeführt. Einige dieser Grössen sind einer statistischen Untersuchung unterzogen worden, um physikalische Beziehungen zwischen ihnen aufzeigen zu können. Insbesondere wurde die Zeitabhängigkeit der Verschiebung aufmerksam beobachtet. Der vorliegende Beitrag erklärt die angewandten statistischen Methoden und die Beziehungen zwischen den verschiedenen physikalischen Grössen und äussert sich auf dieser Basis zur Sicherheit des Kunstwerks.

1. THE PROBLEM

The structural safety of existing structures may be a problem of great interest, even if it does not menace human lifes directly, when the survival of the building is to be assured for his special historical or artistical renown.

These buildings are somehow a really big patrimony of the whole society, and its safeguard is a duty that a civil society will consider of great importance.

In 1979 some alrming crack opening on the wall on which Leonardo da Vinci painted the celebrated "Last Supper" caused the intervention of the "Ministero dei Beni Culturali" by means of "Soprintendenza ai Beni Architettonici" of Milan.

Besides the different works made,some of with concerning the immediate safeguard of the monument, an automatic data acquisition and recording system was designed and set up, in order to collect a continuous checking of 76 instruments measuring temperatures, relative and absolute displacements, forces and settlements in some suitable points of the structure.

The collected data were represented in plots which showd, though in an empirical way, some presumable relationships existing among the observed quantities: in particular a very clear dependence of the displacements and of the forces on the temperature was noted, and an alrming time dependence was also suspected.

Some of these quantities were chosen to represent in the best way the phenomenon and to perform further analysis in order to more accurately describe the functional relationships among them.

2. THE METHOD AND THE STRATEGY

2.1 Basic hypothesis

The quanties selected are:
- 3 series of temperatures (namely T03, T07, T09),
- 4 series of "absolute" displacements of the wall and of the retaining steel frame installed on the backside of the wall (namely D4T, D4M, D7T, D7M);
- 2 series of forces interacting between the wall and the steel frame (namely F05 and f08).

First of all, some hypotheses was made, by engineering judgement, on the relationships existing among the measurements of displacements (d), temperatures (T), forces (f), and time (t), namely:

$$d = d\ (T,t),$$
$$f = f\ (d).$$

The method employed to find the "best" relationships is based on a multiple linear regression by means of least squares technique; the best estimate y of a quantity y on the base of the quantities x_i $(i = 1,...,r)$ is found in the form:

(*) See: Migliacci, A. et al., "Leonardo's Ultima Cena masonry wall", IABSE symposium Venezia 1983.

$$\hat{y} = a_o + \Sigma_i a_i x_i$$

If we know the value of y and $x_1,\dots,x_r$ in n points, in each of them we define a residual v as:

$$\hat{v}_{yi} = y_{o,i} - \hat{y}_i$$

and impose the condition: $\Sigma_j \hat{v}_j^2$ = minimum, to be realised among the possible choices of the coefficients a_i.
The autocorrelation function of the residuals $\hat{v}_j$ (ordered on a time axis) was then analysed in order to separate the signal (s) from the noise (n) of the process giving a new estimate of y by means of the last square collocation technique:

$$\hat{v}_y = \hat{s}_y + \hat{n}_y$$

$$\hat{\hat{y}} = \hat{y} + \hat{s}_y$$

$$y_o = \hat{\hat{y}} + n_y$$

Similarly a study of the crosscorrelation functions of the residuals of different quantities was performed to control the relationships not explained by the chosen model.

2.2 Modelling the observed quantities

The measured temperatures have shown a typical periodical time dependence, related to seasonal variations: thus they were modelled by means of a Fourier-type series based on harmonics of the foundamental period of one year:

$$T_o = c_T + \Sigma_i (a_{Ti} \sin (i\pi t/365) + b_i \cos(i\pi t/365)) + \hat{v}_T = \hat{T} + \hat{v}_T$$

The second step was to explain the displacements by a linear regression on the explained temperatures $\hat{T}_i$ and the time t:

$$d_o = c_d + \Sigma_i a_{d,i} + b_d t + \hat{v}_d = \hat{d} + \hat{v}_d$$

A particular study was performed to check the possible fitting of a less dangerous time dependence of the kind: $y = c - a e^{-bt}$, which supposes that the phenomenon is tending asymptotically to a constant.
These are the steps:
displacements on temperatures only:

$$d_o = c^*_d + \Sigma_i a^*_{d,i} \hat{T}_i + \hat{v}^*_d$$

time on temperatures:

$$t_o = c_t + \Sigma_i a_{Ti} \hat{T}_i + \hat{v}_t = \tilde{\bar{t}} + (t_o - \bar{t})$$

and so the comparison was made between:

$$\hat{v}^*_d = a^{**}\ \hat{v}_t + v_{\hat{v}_d}$$

and

$$\hat{v}^*_d = c^{**} - a^{**} \exp(-\hat{v}_t/365) + v_{\hat{v}_d}$$

the very little difference in the results between the two extimates of the displacements makes the choise between the two models impossible. So we can state that the phenomenon is possibly exhausting very slowly and a period of time of two years is not enough to predict his real behaviour.

The third step was then to explain the forces by the displacements (measured in correspondence of the force transducers)

$$f = c_f + \Sigma_i\ a_{fi}\ \hat{d}_i + \hat{v}_f = \hat{f} + \hat{v}_f$$

All the residuals v can then be broken in a signal s and a noise n, giving a new estimate of y.

2.3 The least squares collocation method.

The collocation method requires appropriate models to interpolate the empirical autocovariance function of the signal, obtained from the residuals of the linear regression. This interpolated function is then used (in addition to the model found by the linear or non-linear regression) to predict the value of the studied quantities at times where on observations are avaitable.

An hypothesis was made: the residuals can be seen as realizations of a broadly stationary random process with mean zero and autocovariance function of the kind

$$C\ (t_1,t_2) = C(|t_1 - t_2|).$$

Since x (t_i) are the n observations at different times $t_1,\ldots,t_i,\ldots,t_n$ the estimate of the empirical covariance at the time interval $\tau_k = |t_1 - t_2|$ is calculated from:

$$\gamma(\tau_k) = 1/n\ \Sigma_i (x(t_i) - \bar{x})\ 1/n_i\ \Sigma_j (x(t_j) - \bar{x})$$

$$\text{where } \tau_{k-1} < |t_i - t_j| \leq \tau_k$$

The "best fit" of the autocovariance function of the signal is then chosed among some available models, namely

E	$\gamma(\tau) = a \exp(-b\tau)$	N	$\gamma(\tau) = a \exp(-b\tau^2)$
EP	$\gamma(\tau) = a \exp(-b\tau)\ (1-c\tau^2)$	NP	$\gamma(\tau) = a \exp(-b\tau^2)\ (1-c\tau^2)$
EC	$\gamma(\tau) = a \exp(-b\tau) \cos(c\tau)$	NC	$\gamma(\tau) = a \exp(-b\tau^2) \cos(c\tau)$
ES	$\gamma(\tau) = a \exp(-b\tau) \sin(c\tau)/(c\tau)$	NS	$\gamma(\tau) = a \exp(-b\tau^2) \sin(c\tau)/(c\tau)$

Finally, the noise variance is found as $\sigma_n^2 = \sigma^2 - \sigma_s^2 = \sigma^2 - a$

The filtering of the process splits the residuals v in two parts, the signal s and the noise n:

$$\hat{s} = C_{ss} C_{vv}^{-1} v$$

$$\hat{n} = v - \hat{s}$$

where $C_{vv} = C_{ss} + \sigma_n^2 I$

and C_{ss} is the matrix of the autocovariance of the signal, I is the unitary matrix of the same dimension as C_{ss}.

The variance of the error of the estimated signal ($e = s - \hat{s}$) is given by:

$$\sigma_c^2 = \sigma_s^2 - C_{ss} C_{vv}^{-1} C_{ss}$$

The same relationships are employed for the prediction of the signal s in points (or at times) where no observations are available:

$$\hat{S}_p = C_{sps} C_{vv}^{-1} V$$

$$\sigma_{ep}^2 = \sigma_{sp}^2 - C_{sps} C_{vv}^{-1} C_{ssp}$$

where $e_p = s_p - \hat{s}_p$.

The interest of the prediction is particulary in the study of the future behaviour of the time series.

3. THE RESULTS.

Influence of the temperature on the front side of the wall and on the back side was examined, as well as influence of the temperature on the top of the vaults covering the "Cenacolo" (which is nearly the external temperature).
Displacements resulted to be clearly dependent on the difference between external temperature and the temperature on the front side of the wall; this is phisically well explained by the variation in the valut thrust due to the differential thermical dilatation.
The quite strong time dependence of the displacements measured on the top of the wall displays a non-stationnariety of the phenomenon, and justifies the alarm about wall safety and the works made for its safeguard.
According to phisical evidence, forces interacting between the wall and the steel frame results strongly dependent from the difference of the displacements of the wall (DiM) and of the frame (DiT) in corresponding positions, as is shown by the coefficients of the linear regression.

The goodness of fit is very satisfactory: the mean square error (m.s.e.) of the noise is about 6% of the general m.s.e. of the observated data for the forces, and between 7.15 and 16.12% for the dispacements.

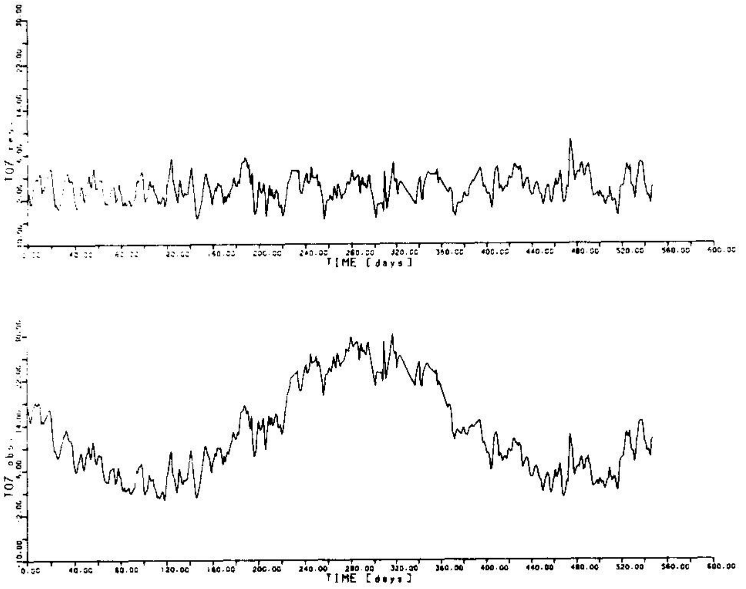
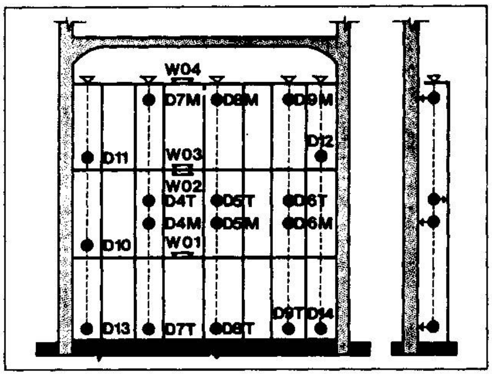


Fig. 1 Plots of temperatures T07 observed and residuals versus time.



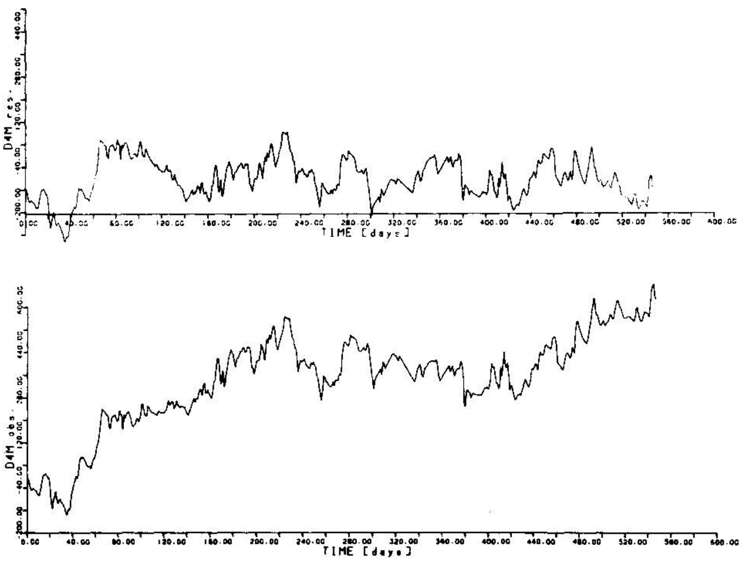
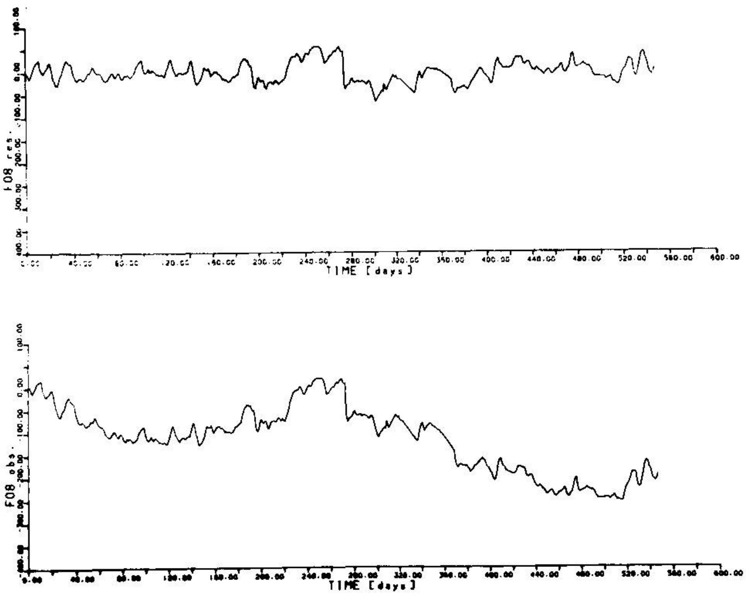


Fig. 2 Plots of displacements D4M observed and residuals versus time.



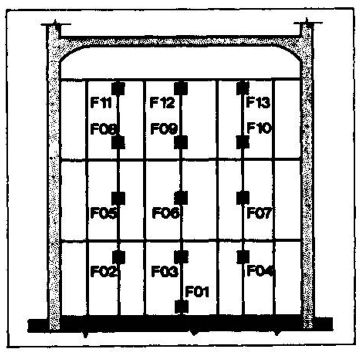


Fig. 3 Plots of forces F08 observed and residuals vs. time.

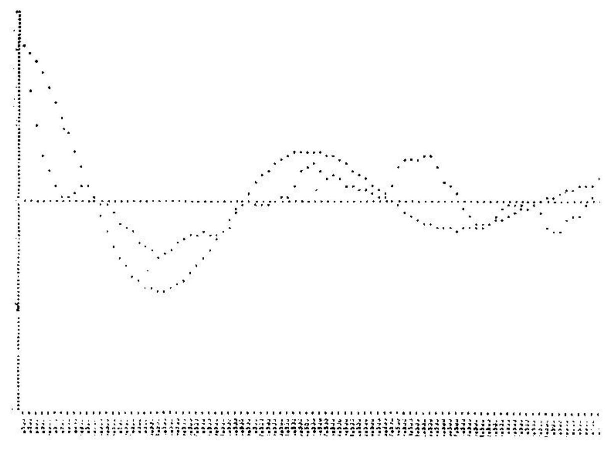


Fig. 4 Autocovariance of TO7 residuals

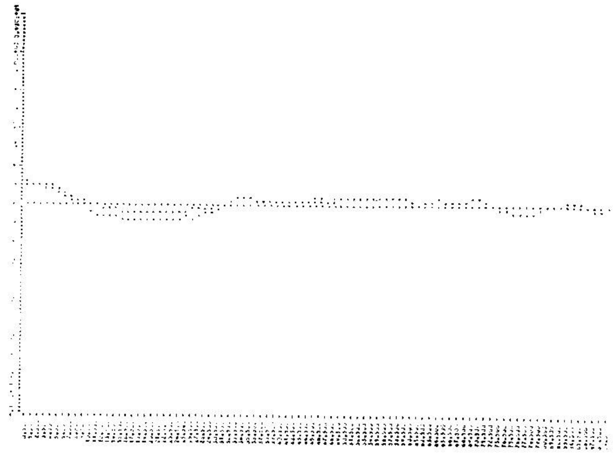


Fig. 5 Crosscovariance of TO7 and D4M residuals.

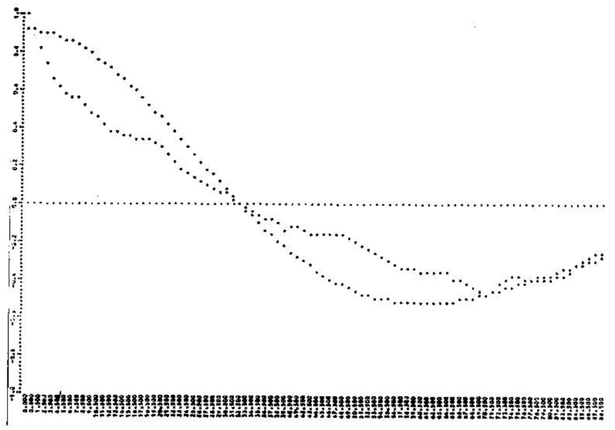


Fig. 6 Autocovariance of D4M residuals

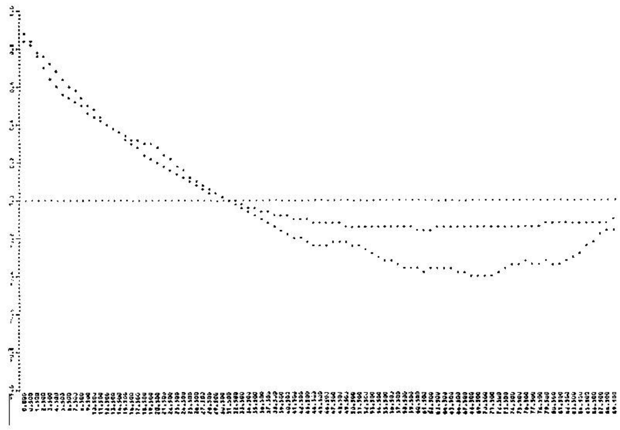


Fig. 7 Crosscovariance of D4M and D4T residuals.

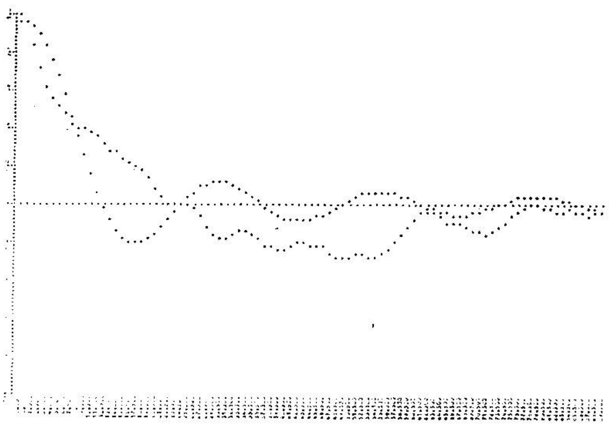


Fig. 8 Autocovariance of FO8 residuals

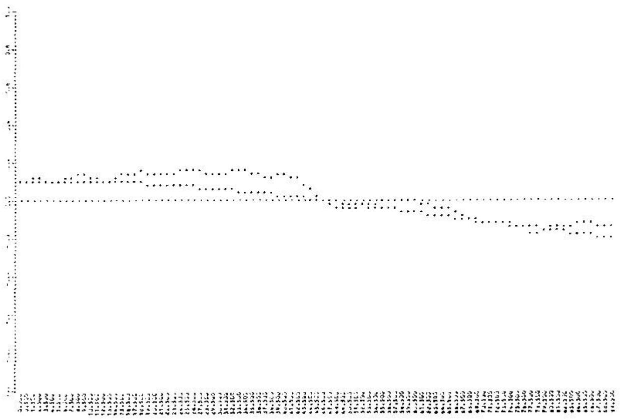


Fig. 9 Crosscovariance of FO8 and D4M residuals.

TAB. I - TEMPERATURES

	T03	T07	T09
x_m	17.37	12.02	18.77
σ_g	4.856	7.692	3.596
σ_e	4.688	7.338	3.381
σ_r	1.266	2.308	1.225
regression coefficients:			
k.t.	19.24	14.97	20.18
sin $2\pi t$	-6.788	-10.81	-4.912
cos $2\pi t$	2.808	3.394	1.877
sin $4\pi t$	-1.206	-0.738	-1.256
cos $4\pi t$	-1.099	-0.354	-1.081
sin $6\pi t$	0.262		0.331
cos $6\pi t$	-0.234		-0.407
sin $8\pi t$	-0.734	-0.640	-0.644
cos $8\pi t$	0.285	-0.241	0.173
autocovariance function:			
kind	N.S.	E.C.	E.C.
$\tau(\gamma=0)$	19.0	8.0	12.4
$\tau(\gamma=.5)$	5.3	2.0	4.4
σ_s	1.233	2.077	1.176
σ_n	0.219	0.968	0.293

TAB. II - DISPLACEMENTS

	D4T	D4M	D7T	D7M
x_m	117.7	339.2	396.6	458.9
σ_g	108.4	175.2	197.7	229.6
σ_e	87.5	158.6	182.6	214.6
σ_r	63.9	74.4	75.8	81.6
regression coefficients:				
k.t.	-271.3	-361.2	66.5	248.8
t	--	.7714	1.009	1.138
T03	-247.5	-258.1	-194.7	-156.3
T07	70.39	67.27	53.53	35.42
T09	205.5	221.8	148.9	116.7
autocovariance function:				
kind	N.P.	E.P.	N.P.	N.P.
$\tau(\gamma=0)$	33.5	31.6	29.0	25.6
$\tau(\gamma=.5)$	20.4	11.5	17.5	14.9
σ_s	63.52	73.98	75.33	81.18
σ_n	17.47	16.25	15.15	16.42

TAB. III - FORCES

	F05	F08
x_m	-51.05	-116.6
σ_g	34.77	76.83
σ_e	33.43	73.27
σ_r	9.54	21.92
regression coefficients:		
k.t.	-6.766	-10.12
D4T	0.237	0.373
D4M	-0.260	-0.447
D7T	0.340	0.814
D7M	-0.259	-0.701
autocovariance function:		
kind	N.C.	N.S.
$\tau(\gamma=0)$	20.83	12.3
$\tau(\gamma=.5)$	8.8	7.17
σ_s	9.502	21.83
σ_n	2.091	4.77

TAB. IV - CROSSCORRELATION FUNCTIONS : ρ_0, ρ_1, τ $(\gamma \to 0)$.

	T03	T07	T09	D4M	D4T	D7M	D7T	F05	F08
T03	1.0	0.642	.928	-.084	.056	-.307	-.168		
	.969	.615	.898	-.081	.055	-.296	-.163		
	110.	88.5	120.	61.5	68.5	78.5	100.		
T07		1.00	.681	-.031	.053	-.218	-.090		
		.821	.631	-.033	.037	-.205	-.090		
		100.	110.	110.	110.	47.5	140.		
T09			1.00						
			.942						
			120.						
D4M				1.00	.890			-.115	-.111
				.952	.832			-.111	-.107
				160.	220.			540.	220.
D4T					1.00			.105	.104
					.912			.110	.109
					180.			50.	210.
D7M						1.00	.963	-.418	-.459
						.936	.925	-.399	-.440
						180.	100.	540.	160.
D7T							1.00	-.241	-.291
							.938	-.231	-.281
							150.	120.	160.
F05								1.00	.956
								.951	.911
								47.5	78.5
F08									1.00
									.952
									120.

REFERENCES

1. DAVIES O.L., Statistical methods in research and production , Oliver and Boyed, Londra, 1961.

2. DRAPER N.R., SMITH H., Applied regression analysis, Wiley and Sons, New York, 1961.

3. HEISKANEN W.A., MORITZ M., Physical geodesy, W.H. Freeman and Company, San Francisco, 1967.

4. KENDAL M.G., STUART A. , The advanced theory of statistics, vol. 1, 2 e 3, Griffin, Londra, 1968-'73.

5. MIDDLEBROKS E.J. , Statistical calculation, How to solve statistical problems, Ann Arbor Science, Ann Arbor (Michigan), 1976.

6. MORITZ M., Advanced physical geodesy, Herbert Wichmann Verlag, Karlsruhe, 1980.

7. MORITZ M., Covariance function in least squares collocation, Reports of the Department of Geodetic Science, n. 240 Ohio State University, Columbus, giugno 1976.

8. MORITZ M., Least squares collocation, DGK Reihe A, Heft n. 75, 1973.

9. MORITZ M., Statistical foundations of collocation , Bolletino di Geodesia e Scienze Affini, anno 39, n. 2, aprile-maggio-giugno 1980.

10. MORITZ M., The geometry of least squares , Pubblication of the Finnish Geodetic Institute, n. 89, Helsinki, 1979.

11. MIKHAIL E.M., con contributi di Ackermann F.. Observations and least squares, Thomas J. Crowell Company Inc., New York, 1976.

12. PAPOULIS A., Probability, random variables, and stochastic processes, Mc Graw-Hill Book Company, New York, 1965.

13. TEWARSON R.P., Sparse matrices, Academic Press, New York, 1973.

14. WATSON G.N., Theory of Bessel functions, Mc Millan Company, New York, 1948.

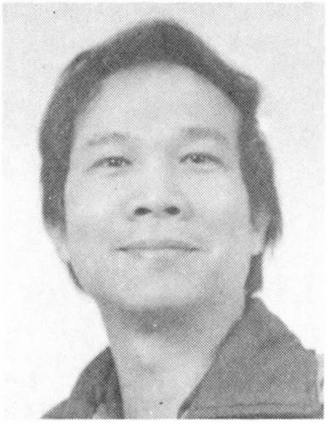
Leere Seite
Blank page
Page vide

Assessment of the Criticality of Bridge Components

Estimation de l'importance des éléments de ponts

Beurteilung der Wichtigkeit von Brücken-Komponenten

Fabian HADIPRIONO
Assist. Prof.
Ohio State University
Columbus, OH, USA



Fabian Hadipriono, born 1947 earned his doctorate degree at the University of California, Berkeley. He is active in research dealing with failure, reliability, and safety analysis. He is now teaching at the Ohio State University.

SUMMARY

Subjective assessments are frequently used during bridge inspection. This paper introduces a method developed to improve the inspection procedures. The concept of fuzzy sets is used here to determine the condition and the urgency for preventive care of bridge components. Case studies are presented.

RÉSUMÉ

Une appréciation subjective est souvent à la base du contrôle des ponts. L'article présente une méthode destinée à améliorer les procédures d'inspection. Le concept de «fuzzy sets» est empoyé afin de déterminer les conditions et l'urgence d'un entretien préventif des éléments de ponts. Des exemples sont présentés.

ZUSAMMENFASSUNG

Bei der Inspektion von Brücken werden oft subjektive Massstäbe angewendet. Der Beitrag befasst sich mit einer Methode zur Verbesserung der Inspektions-Vorgänge. Mit Hilfe der «Fuzzy Set»-Theorie werden der Zustand von Brücken-Komponenten und die Dringlichkeit von Sanierungsmassnahmen beurteilt. Beispiele erläutern das Verfahren.

1. INTRODUCTION

Experience has shown that subjective notions are frequently used by an inspector when monitoring a bridge. Very often the inspector has to identify and deal with variables which are uncertain. Many of them can only be estimated subjectively and qualitatively through the engineer's experience, knowledge, and judgment.

This paper suggests a method for quantifying the subjective assessments of the condition and urgency for preventive care of bridge components based on visual inspections through the use of fuzzy set concept. It is expected that this procedure could be used for quality control purposes. Fuzzy set operations in the following sections are limited to only those pertinent to the study in this paper. The basics of fuzzy set theory can be found in References 1 and 5. For clarity, numerical examples and case studies are presented.

2. FUZZY SET OPERATIONS

Qualitative evaluation of certain variables may determine the criticality of bridge components. Three variables are considered in this study: 1. Condition (CON) determines the state of a component, 2. Importance (IMP) describes the importance of a component with regards to its use or structural integrity, and 3. Prevention urgency (PRE) indicates the urgency of the preventive care for the component. These variables have certain linguistic values which are expressed by the following fuzzy set:

$$[xi/f(xi)];\ i=1,2,..,5 \qquad \text{(1)}$$

where "/" is a delimeter. xi and f(xi) represent the element and the membership function of the fuzzy set, respectively. The element indicates the level of the variable which, in this study, ranges from 1 to 5. The membership value shows the degree of membership of the corresponding element in the fuzzy set and is a real number in the interval [0,1].

2.1 Composite Fuzzy Relation

A fuzzy relation is an operation used to relate different fuzzy sets. Let A and B be two fuzzy sets such that $A \in \phi(X)$ and $B \in \phi(Y)$, where X and Y are the nonempty sets and where $\phi(X)$ and $\phi(Y)$ denote the classes of all fuzzy sets of X and Y, respectively. The membership function of the fuzzy relation, R from A to B, or R=AxB, is expressed by:

$$f_R(xi,yj) = f_{AxB}(xi,yj) = \underset{\forall xi \in X,\ \forall yj \in Y}{\Lambda} [f_A(xi), f_B(yj)] \qquad \text{(2)}$$

which can also be represented by a matrix, where the membership value of each element contained in R is obtained from the minimum (Λ) of the membership values $f_A(xi)$ and $f_B(yj)$.

To obtain the intersection of fuzzy relations, the max-min Composite Fuzzy Relation (CFR) is used. Suppose X, Y and Z are three nonempty sets, and A, B and C are their fuzzy sets, respectively, such that $A \in \phi(X)$, $B \in \phi(Y)$, and $C \in \phi(Z)$. Suppose

R1=AxB and R2=BxC. Then the CFR of R1 and R2 is defined by T=R1oR2, where $T \in \phi(X \times Z)$. Its membership function is expressed by

$$f_T(xi,zk) = f_{R1oR2}(xi,zk) = \bigvee_{\forall x \in X,\ \forall y \in Y,\ \forall z \in Z} [f_{R1}(xi,yj) \wedge f_{R2}(yj,zk)] \quad \ldots\ldots\ (3)$$

whose operation is similar to matrix multiplication except that multiplication is replaced by minimum ($\wedge$) and addition by maximum ($\vee$).

2.1.1 Numerical Example

If, for illustration purposes, there were a specification that relates the condition and urgency measure of a bridge component. Let's say, the urgency for preventive care of a "very good" (VG) component is "very unnecessary" (VUN). Now suppose that we are interested in the urgency measure of "fairly good" (FG) condition. VG, VUN, and FG are linguistic values which, in this example, are defined by using Eq. 1 as follows:

$$\begin{aligned} VG = VUN &= [1/0.0,\ 2/0.1,\ 3/0.5,\ 4/0.9,\ 5/1.0] \\ FG &= [1/0.3,\ 2/0.7,\ 3/1.0,\ 4/0.7,\ 5/0.3] \quad \ldots\ldots\ (4) \end{aligned}$$

Through the use of Eqs. 2 and 3, the relation R=VUNxVG, where $R \in \phi(PRE \times CON)$, can be found as shown in the first matrix of Eq. 5. Using Eq. 3, the fuzzy composition of this matrix and FG results in the membership values of the urgency measure PRE1:

$$T = RoFG = \overset{R=VUNxVG}{\begin{bmatrix} 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\ 0.0 & 0.1 & 0.1 & 0.1 & 0.1 \\ 0.0 & 0.1 & 0.5 & 0.5 & 0.5 \\ 0.0 & 0.1 & 0.5 & 0.9 & 0.9 \\ 0.0 & 0.1 & 0.5 & 0.9 & 1.0 \end{bmatrix}} \circ \overset{FG}{\begin{bmatrix} 0.3 \\ 0.7 \\ 1.0 \\ 0.7 \\ 0.3 \end{bmatrix}} = \overset{PRE1}{\begin{bmatrix} 0.0 \\ 0.1 \\ 0.5 \\ 0.7 \\ 0.7 \end{bmatrix}} \quad \ldots\ (5)$$

The corresponding value of the urgency measure is obtained by transposing PRE1:

$$PRE1 = [1/0.0,\ 2/0.1,\ 3/0.5,\ 4/0.7,\ 5/0.7] \quad \ldots\ldots\ (6)$$

which may be interpreted as close to "unnecessary."

2.2 Inverse Composite Fuzzy Relation

The inverse composite fuzzy relation (ICFR) operation was developed by Sanchez [4] to obtain the greatest membership values in an unknown relation R2 in T=R1oR2, if T and R1 are known. Sanchez defined the ICFR of R2 as $R1^T @ T$ where $R1^T$ is the transpose of R1. The membership function of R2 is expressed by

$$f_{R2}(xi,zk) = f_{R1^T @ T}(xi,zk) = \bigwedge_{\forall x \in X,\ \forall y \in Y,\ \forall z \in Z} [f_{R1^T}(xi,yj)\ \alpha\ f_T(yj,zk)] \quad \ldots\ldots\ (7)$$

The operation is the same as matrix multiplication except that multiplication is replaced by α operation. This requires that each membership value, r1, in matrix R1, be compared with the membership value, t, in T such that $r1\ \alpha\ t = 1$ if $r1 \leq t$ and $r1\ \alpha\ t = t$ if $r1 > t$. The addition is replaced by taking the minimum of the results of α operation.

2.2.1 Numerical Example

If $R \in \phi(PRExCON)$ and $PRE1 \in \phi(PRE)$ are again expressed by the first and third matrices in Eq. 6, using Eqs. 7, the membership values of the condition $CON1 \in \phi(CON)$ yields:

$$CON1 = \overset{R \in \phi(CONxPRE)}{\begin{bmatrix} 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\ 0.0 & 0.1 & 0.1 & 0.1 & 0.1 \\ 0.0 & 0.1 & 0.5 & 0.5 & 0.5 \\ 0.0 & 0.1 & 0.5 & 0.9 & 0.9 \\ 0.0 & 0.1 & 0.5 & 0.9 & 1.0 \end{bmatrix}} @ \overset{PRE1}{\begin{bmatrix} 0.0 \\ 0.1 \\ 0.5 \\ 0.7 \\ 0.7 \end{bmatrix}} = \overset{CON1}{\begin{bmatrix} 1.0 \\ 1.0 \\ 1.0 \\ 0.7 \\ 0.7 \end{bmatrix}} \quad \ldots\ldots (8)$$

The result of Eq. 8 shows the following condition value:

$$CON1 = [1/1.0,\ 2/1.0,\ 3/1.0,\ 4/0.7,\ 5/0.7] \quad \ldots\ldots (9)$$

which is different from the original condition value in Eq. 4. This difference is expected since the ICFR produces the greatest membership values for CON1 (Note that the fuzzy set for FG in Eq. 4 is the subset of CON1 in Eq. 9.).

2.3 Polynomial Fuzzy Sets

The CFRs described in the foregoing section were of monomial forms. However, if the fuzzy relation R1 in the CFR have multiple values, the fuzzy set composition takes the following polynomial form:

$$T = \bigvee_{i=1}^{n} (R1^{(i)} \circ R2) \quad \ldots\ldots (10)$$

T in Eq. 10 will become a constraint. Given this constraint, the problem to solve for unknown R2, if $R1^{(i)}$ is known, becomes one of how to simplify this polynomial form. Ohsato and Sekiguchi [3] transformed this form into i monomial forms through decomposition procedures such that $T=(R1^{(1)} \circ R2^{(1)})$, $T=(R1^{(2)} \circ R2^{(2)})$, ..., $T=(R1^{(n)} \circ R2^{(n)})$.

Hence, each monomial form is now solved using the previously described ICFR procedure to obtain the unknown $R2^{(i)}$. The solution of R2, that incorporates all $R2^{(i)}$ is

$$R2 = \bigwedge_{i=1}^{n} R2^{(i)} \quad \ldots\ldots (11)$$

where Λ is the conjunction (or intersection, in probability theory) of all $R2^{(i)}$.

3. CASE STUDIES

This study involved the evaluation of a bridge deck which consists of seven components as listed in column 2 of Table 1. As mentioned earlier three factors, IMP, CON, and PRE, (also known as linguistic variables) are considered for the analysis. The fuzzy set model developed by the author is used for the variables. The model, shown in Figure 1, encompasses seven linguistic values whose relations between the membership function and fuzzy set element are represented in Table 2.

Fuzzy relations between the variables are presented in Table 3. R1 defines the relation between IMP and PRE which indicates that the more important a component the more susceptible it is towards the urgency for the preventive care. Therefore, important components are given negative values and are related to the negative quality of the urgency care. R2, relating CON to PRE, shows that the better the condition of a component, the less urgent (unnecessary) it is for its preventive care. These relations were used for the following case studies.

3.1 Case A

In this case, an inspector monitored a bridge deck and assigned values for the variables IMP and CON for each deck component. He/she used the linguistic values in Table 2 for the assessments, which were then entered in columns 3 and 5 of Table 1. Suppose that the summary of his/her assessments on the bridge deck condition as a whole was required and a decision about the urgency care for the bridge deck has to be made.

First, we should find the fuzzy relations R1 and R2 for each bridge deck components. For example, for the bridge deck floor, the value for IMP is VI (Table 1, column 3), which is related to VUR in Table 3; therefore, R1=VIxVUR. Suppose that the inspector rated the condition of this floor as poor, or P (Table 1, column 5); then, from Table 3, R2=URxP. A similar procedure was applied to obtain R1 and R2 of the other bridge deck components. The total effect on the bridge deck was determined by taking the disjunction (or union, in probability theory) of membership values of all R1's and R2's:

$$\text{R1Tot} = \begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.75 & 0.00 \\ 0.92 & 0.92 & 0.83 & 0.75 & 0.00 \\ 0.83 & 0.83 & 0.83 & 0.75 & 0.00 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \end{bmatrix} \quad \text{R2Tot} = \begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.83 & 0.00 \\ 0.92 & 0.92 & 0.83 & 0.83 & 0.75 \\ 0.83 & 0.83 & 0.83 & 0.83 & 0.83 \\ 0.75 & 0.75 & 0.83 & 0.92 & 0.92 \\ 0.00 & 0.75 & 0.83 & 0.92 & 1.00 \end{bmatrix} \quad \text{.............} (12)$$

where R1Tot ϵ ϕ(IMPxPRE) and R2Tot ϵ ϕ(PRExCON). Subsequently, the composition R1Tot o R2Tot = Ra, where Ra ϵ ϕ(IMPxCON), can be found as follows:

$$\begin{array}{rcc} & & \text{IMP(a)} \\ \text{Ra} = & \begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.83 & 0.83 \\ 0.92 & 0.92 & 0.83 & 0.83 & 0.83 \\ 0.83 & 0.83 & 0.83 & 0.83 & 0.83 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0.75 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \end{bmatrix} & \begin{matrix} 1.00 \\ 0.92 \\ 0.83 \\ 0.75 \\ 0.00 \end{matrix} \\ \text{CON(a)} & \begin{matrix} 1.00 & 0.92 & 0.83 & 0.83 & 0.83 \end{matrix} & \end{array} \quad \text{..........} (13)$$

Then, projection on variable space CON, by taking the maximum membership value in each column of matrix Ra, yields the membership value of the bridge deck total condition: "close to fairly poor." Projection on variable space IMP yields the total importance of the bridge deck which indicates "fairly important." These values are illustrated in Figure 2.

3.2 Case B

This case is concerned with the assessment of two inspectors whose consensus in determining the preventive care of a bridge deck is needed in addition to a certain existing maintenance policy. Suppose that the ratings of the inspector in Case A are used here as well as the ratings of another inspector. Their assessments on the variables IMP and CON of the bridge deck components are listed in Table 1. A similar procedure used in Case A is performed here to obtain the matrix Rb ϵ ϕ(IMPxCON) for the assessment of the second inspector. The result of the the total matrix Rb is shown in Eq. 14.

$$
\begin{array}{ccc}
 & & \text{IMP(b)} \\
\text{Rb} = & \begin{bmatrix} 1.00 & 0.75 & 0.50 & 0.75 & 0.75 \\ 0.92 & 0.75 & 0.50 & 0.75 & 0.75 \\ 0.83 & 0.75 & 0.50 & 0.75 & 0.75 \\ 0.75 & 0.75 & 0.50 & 0.75 & 0.75 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \end{bmatrix} & \begin{array}{c} 1.00 \\ 0.92 \\ 0.83 \\ 0.75 \\ 0.00 \end{array} \\
\text{CON(b)} & \begin{array}{ccccc} 1.00 & 0.75 & 0.50 & 0.75 & 0.75 \end{array} &
\end{array}
\qquad \text{.........} \quad (14)
$$

The projection on spaces IMP and CON summarizes the bridge deck condition and is shown in Figure 2. Comparison between the two cases shows a more conservative assessments for CON from the second inspector but a similar result for IMP.

Let us suppose that the importance of the bridge deck as a whole was determined based on a certain policy (e.g., from the Department of Transportation) and is considered as "fairly important," or FI. Based on Table 3, the relation, T, between IMP and PRE can be obtained and becomes a constraint to reach the consensus for the urgency care. Now the relations between the IMP, CON, and PRE are shown in Figure 3 and can be expressed as T = (Ra o X) V (Rb o X), or through decomposition process:

$$Ra^T @\, T = Xa \text{ and } Rb^T @\, T = Xb \quad \text{..............................} \quad (15)$$

where Ra,Rb ϵ ϕ(IMPxCON); Ra^T,Rb^T ϵ ϕ(CONxIMP); T ϵ ϕ(IMPxPRE); and Xa,Xb ϵ ϕ(CONxPRE).

Using Eq. 15, Xa and Xb are found as shown below:

$$
\underset{Ra^T \,\epsilon\, \phi(\text{CONxIMP})}{\begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.75 & 0 \\ 0.92 & 0.92 & 0.83 & 0.75 & 0 \\ 0.83 & 0.83 & 0.83 & 0.75 & 0 \\ 0.83 & 0.83 & 0.83 & 0.75 & 0 \\ 0.83 & 0.83 & 0.83 & 0.75 & 0 \end{bmatrix}}
@
\underset{T \,\epsilon\, \phi(\text{IMPxPRE})}{\begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.75 & 0 \\ 0.92 & 0.92 & 0.83 & 0.75 & 0 \\ 0.83 & 0.83 & 0.75 & 0.75 & 0 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0 \\ 0 & 0 & 0 & 0 & 0 \end{bmatrix}}
=
\underset{Xa \,\epsilon\, \phi(\text{CONxPRE})}{\begin{bmatrix} 1.00 & 0.92 & 0.75 & 0.75 & 0 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0 \end{bmatrix}}
$$

$$
\underset{Rb^T \,\epsilon\, \phi(\text{CONxIMP})}{\begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.75 & 0 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0 \\ 0.50 & 0.50 & 0.50 & 0.50 & 0 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0 \end{bmatrix}}
@
\underset{T \,\epsilon\, \phi(\text{IMPxPRE})}{\begin{bmatrix} 1.00 & 0.92 & 0.83 & 0.75 & 0 \\ 0.92 & 0.92 & 0.83 & 0.75 & 0 \\ 0.83 & 0.83 & 0.75 & 0.75 & 0 \\ 0.75 & 0.75 & 0.75 & 0.75 & 0 \\ 0 & 0 & 0 & 0 & 0 \end{bmatrix}}
=
\underset{Xb \,\epsilon\, \phi(\text{CONxPRE})}{\begin{bmatrix} 1.00 & 0.92 & 0.75 & 0.75 & 0 \\ 1.00 & 1.00 & 1.00 & 1.00 & 0 \\ 1.00 & 1.00 & 1.00 & 1.00 & 0 \\ 1.00 & 1.00 & 1.00 & 1.00 & 0 \\ 1.00 & 1.00 & 1.00 & 1.00 & 0 \end{bmatrix}}
\quad \text{..................} \quad (16)
$$

Through the use of Eq. 11 the conjunction of Xa and Xb yields:

$$
X \,\epsilon\, \phi(\text{CONxPRE}) = \begin{bmatrix} 1.00 & 0.92 & 0.75 & 0.75 & 0.00 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0.00 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0.00 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0.00 \\ 1.00 & 1.00 & 0.75 & 0.75 & 0.00 \end{bmatrix} \quad \text{............} \quad (17)
$$

Finally, projection on space PRE leads to the value of the prevention urgency as shown below:

$$X \in \phi(PRE) = [1/1,\ 2/1,\ 3/0.75,\ 4/0.75,\ 5/0] \ \ldots\ldots\ldots\ldots \quad (18)$$

which can be represented graphically in Figure 2. The consensus of the assessors' rating, including the constraint, yields a measure of "close to fairly urgent" for the bridge deck repair.

4. CONCLUSIONS

In this paper, the procedures for assessing bridge condition and the urgency for its preventive care quantify the subjective judgments provided by the bridge inspectors. This quantification process can only be performed through the use of fuzzy set concept. The fuzzy set manipulations can be performed with the help of computer programs to solve complex polynomial problems. The procedures described in this study allow a great deal of flexibility in determining the basic information such as that provided in Tables 2 and 3. This information can be updated or modified accordingly, depending upon the users need. The procedure also incorporates graphical representations for the values of the variables studied.

5. REFERENCES

1. HADIPRIONO F., Assessment of Falsework Performance Using Fuzzy Set Concepts. Structural Safety, 3, 1985, pp 47-57

2. HADIPRIONO F. and TOH H. S., Approximate Reasoning Models for Consequences on Structural Components due to Failure Events, Journal of Civil Engineering for Practicing and Design Engineers, 5, Febrauary, 1986.

3. OHSATO, A. and SEKIGUCHI, T., Method of Solving the Polynomial Form of Composite Fuzzy Relations and Its Application to a Group Decision Problem. Approximate Reasoning in Decision Analysis, North Holland Publishing Company, 1982, pp 33-44.

4. SANCHEZ, E., Resolution of Composite Fuzzy Relation Equations, Information and Control, Vol 30, 1976, pp 38-48.

5. ZADEH, L.A., Fuzzy Sets, Information and Control, Vol. 8, 1965, pp 338-353.

No. (1)	Component (2)	IMP (3)[a,b]	IMP (4)[b]	CON (5)[a,b]	CON (6)[b]
1	Floor	VI(-3)	I(-2)	P(-2)	VP(-3)
2	Wearing Surface	I(-2)	VI(-3)	G(+2)	P(-2)
3	Curbs and Walkways	FI(-1)	FI(-1)	VG(+3)	VG(+3)
4	Median	FI(-1)	FI(-1)	FP(-1)	P(-2)
5	Railing	I(-2)	VI(-3)	VP(-3)	P(-2)
6	Drainage	I(-2)	I(-2)	FG(+1)	G(+2)
7	Expansion Joints	I(-2)	FI(-1)	P(-2)	P(-2)

[a]Rating of inspector for Case A; [b]Rating of inspector for Case B.

Table 1 Bridge deck components and ratings for IMP and CON

Linguitic values (1)	Notation (Rating) (2)	f(x) (3)
Very-Good/Unimportant/Unnecessary	VG/VUI/VUN (+3)	$(x-1)/12;\ 1\le x\le 4$ $(3x-11)/4;\ 4<x\le 5$
Good/Unimportant/Unnecessary	G/UI/UN (+2)	$(x-1)/4$
Fairly-Good/Unimportant/Unnecessary	FG/FUI/FUN (+1)	$(3x-3)/4;\ 1\le x\le 2$ $(x+7)/12;\ 2<x\le 5$
Undecided (Fair:Btw FG-FP etc.)	UND (0)	1
Fairly-Poor/Important/Urgent	FP/FI/FUR (-1)	$(13-x)/12;\ 1\le x\le 4$ $(15-3x)/4;\ 4<x\le 5$
Poor/Important/Urgent	P/I/UR (-2)	$(5-x)/4$
Very-Poor/Important/Urgent	VP/VI/VUR (-3)	$(7-3x)/4;\ 1\le x\le 2$ $(5-x)/12;\ 2<x\le 5$

Table 2 Fuzzy set model H

IMP (1)	PRE (2)	CON (3)	PRE (4)
VI(-3)	VUR(-3)	VG(+3)	VUN(+3)
I(-2)	UR(-2)	G(+2)	UN(+2)
FI(-1)	FUR(-1)	FG(+1)	FUN(+1)
UND(0)	UND(0)	UND(0)	UND(0)
FUI(+1)	FUN(+1)	FP(-1)	FUR(-1)
UI(+2)	UN(+2)	P(-2)	UR(-2)
VUI(+3)	VUN(+3)	VP(-3)	VUR(-3)

Table 3 Relations between IMP, CON, and PRE

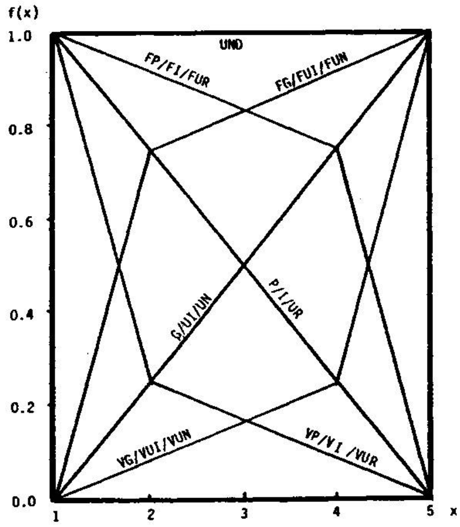


Fig. 1 Fuzzy set model H

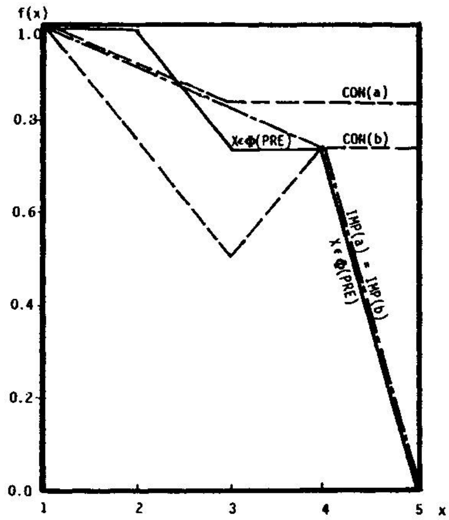


Fig. 2 Results from Cases A and B

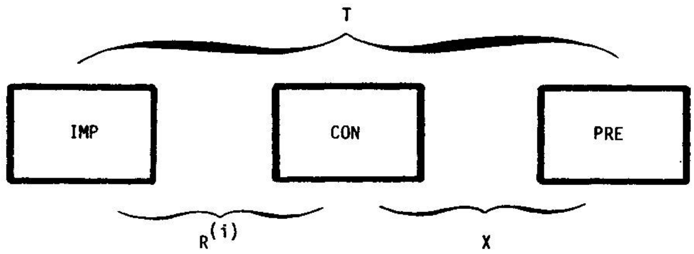


Fig. 3 Polynomial relations between IMP, CON, and PRE

Quality Concrete Production for Ganga Bridge, Patna, India

Réalisation de béton de qualité pour le pont sur le Gange à Patna, Inde

Herstellung von Qualitäts-Beton für die Ganges-Brücke in Patna, Indien

S.A. REDDI
Chief Engineer-Projects
Gammon India Limited
Bombay, India

S.A. Reddi, born in 1933, is a Fellow of Institution of Engineers (India). During the last 30 years, he was involved in the construction of more than 50 major prestressed concrete bridges. He is actively involved in the preparation of Standards for the Indian Standards Institution, Indian Roads Congress, etc. He has been a member of FIP-Working Group on Hot Weather Concrete.

SUMMARY

The various measures adopted for cost effective production of concrete and for ensuring quality during the construction of a 5,6 km long prestressed concrete bridge across the River Ganges at Patna, India are described. The quality control measures are devised in the context of the use of minimum equipment and labour intensive production practices in India. The results of statistical analysis of about 100000 concrete cube specimens are presented.

RÉSUMÉ

L'article décrit les nombreuses mesures prises pour la production avantageuse de béton et pour l'assurance d'une qualité correspondante pendant la construction d'un pont en béton précontraint, de 5,6 km de longueur, sur le Gange à Patna, Inde. Les mesures d'assurance de la qualité sont adaptées aux conditions de production en Inde, utilisant le minimum d'équipements et le maximum de travailleurs. Les résultats d'une étude statistique d'environ 100000 éprouvettes cubiques en béton sont présentés.

ZUSAMMENFASSUNG

Die vielfältigen Massnahmen zur kostengünstigen Beton-Herstellung für die 5,6 km lange Spannbeton-Brücke über den Ganges in Patna, Indien sowie für die zugehörige Qualitätssicherung werden beschrieben. Die Massnahmen zur Qualitätssicherung sind auf die investitionsarmen und arbeitsintensiven indischen Produktionsverhältnisse zugeschnitten. Die Ergebnisse aus der statistischen Prüfung von etwa 100000 Betonwürfeln werden mitgeteilt.

1. INTRODUCTION

A 5.575 km long bridge has been constructed across the River Ganges near Patna in Northern India. The foundations consist of 12m dia. concrete caissons sunk to a depth of about 55m below low water level. The cellular piers are in reinforced concrete. The superstructure consists of 121m single cell box girder spans, constructed using precast segments for the major portion of the bridge while the transition spans between land and water were cast insitu. (Fig. 1) About 300,000 cum. concrete of grades varying from 16 MPa to 45 MPa was used for the construction. The first stage of construction including the foundations and the substructure upto high flood level for four lanes of traffic and the balance substructure and the deck for two lanes of traffic was completed and the bridge opened to traffic in 1982. (Fig. 2). The second stage of the bridge construction was taken as a separate contract during the latter part of 1983 and is scheduled for completion in 1986.

2. SPECIFICATIONS FOR CONCRETE

The work is carried out as per Indian Roads Congress Specifications IRC:18 and IRC:21 for prestressed and reinforced concrete respectively. The Indian Standard Code of Practice for plain and reinforced concrete (IS:456) is also used. The concrete is accepted if the average strength of the group of cubes cast for each day is not less than the specified characteristic strength. 20% of the cubes cast for each day may have a value less than the specified strength provided the lowest value is not less than 85% of the specified strength. In terms of IS:456, the acceptance criteria is based on statistical analysis using standard deviation. Considering the large quantity of concrete involved, it was decided to monitor the quality using statistical concepts and at the same time satisfying the IRC Codes regarding standard of acceptance.

3. GRADES OF CONCRETE

The following grades of concrete were used :-

Caissons	16 and 20 MPa	A/c: 6.8	W/c: 0.55
Piers	25 MPa	6.3	0.50
	35 MPa	5.6	0.42
Deck	45 MPa	4.0	0.34

4. MATERIALS FOR CONCRETE

4.1 Cement

Indian cement conforming to IS:269 is used. In view of the considerable variation in the quality of cement produced by different factories (22-50 MPa @ 7 days), the source is carefully chosen after preliminary trials and the supplies are ensured from two factories only.

4.2 Aggregates

Coarse aggregates for reinforced concrete consist of natural river gravel obtained from a distance of about 300 km. Crushed broken stone was used for prestressed concrete, in terms of the contract requirements, even though tests with natural gravel indicated equal acceptability. Fine aggregates consisted of natural river sand. The use of natural gravel of 40mm maximum size

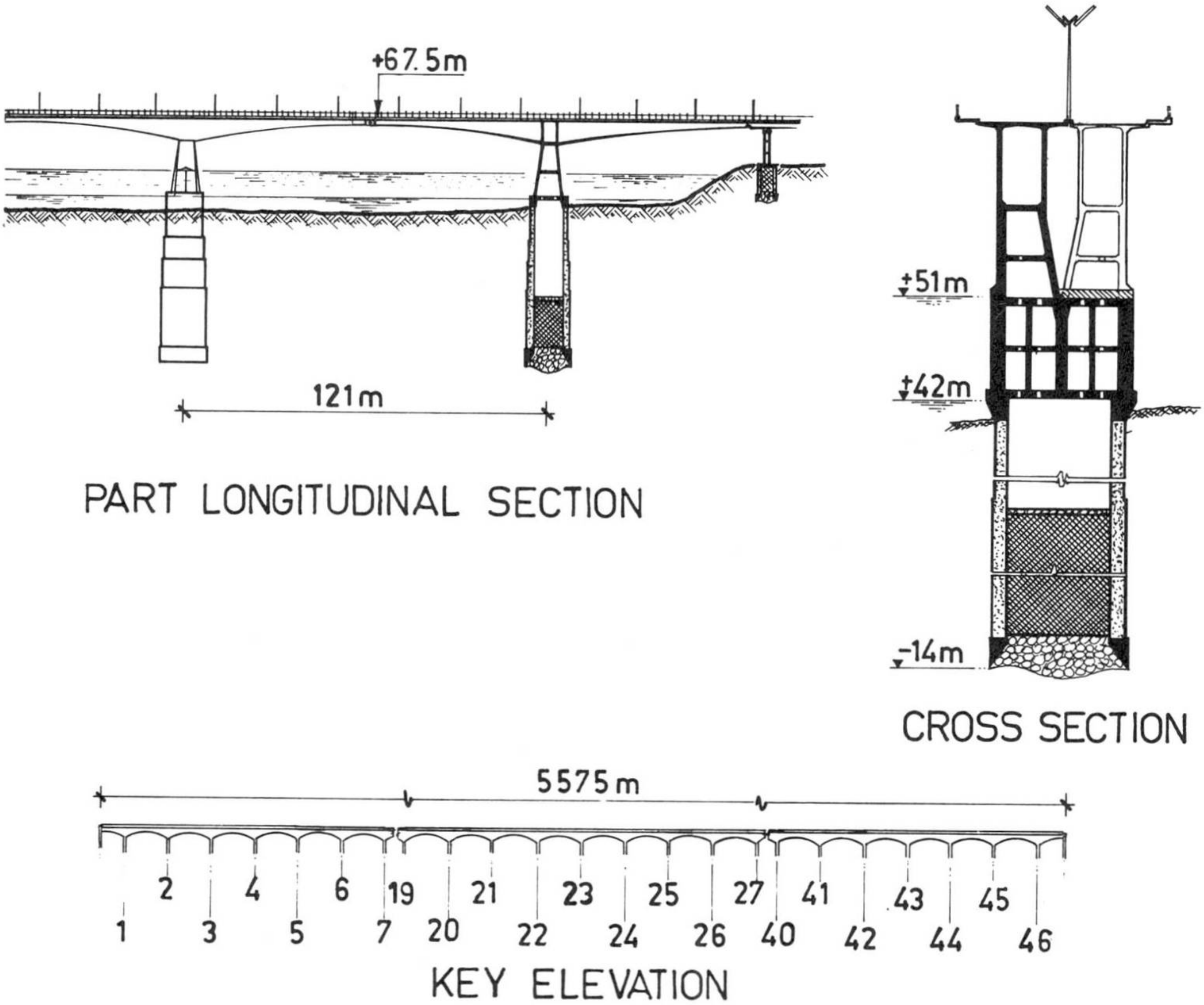


Fig. 1 Ganga Bridge at Patna - Details

Fig. 2 A View of the Completed Bridge (First Two Lanes)

particularly for Grade 35 concrete has resulted in substantial saving in cement (about 15%), partly due to the use of 40mm size and partly due to the use of rounded natural gravel. In order to facilitate use of 40mm maximum size in the reinforced concrete piers, the detailing of the reinforcement was suitably modified.

4.2.1 Grading of aggregates

The Indian Standard Code (IS:383) specifies the permissible range of gradings both for coarse and fine aggregates. While the individual gradings for coarse/

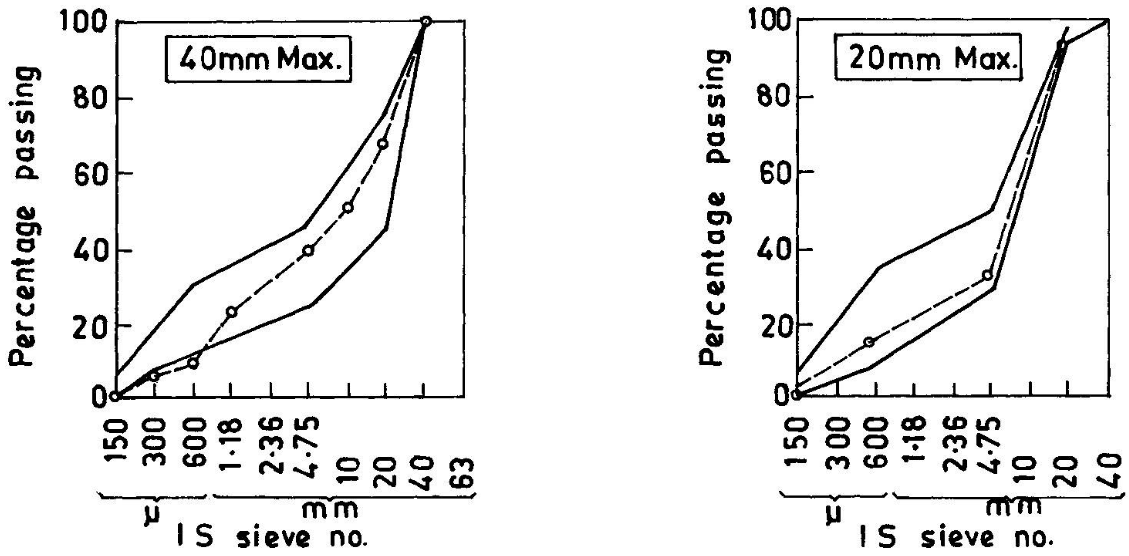


Fig. 3 Typical Combined Aggregate Grading

fine aggregates procured for the work were controlled only within a very broad spectrum, the grading of the combined aggregates was closely controlled. Sieve analysis of coarse and fine aggregates were conducted once a day and the nominal proportion adjusted to ensure the combined grading. (Fig. 3). The river gravel has a reasonably good natural grading and as such the material was used without further processing. The crushed stone of size 5mm to 20mm was obtained in one grade without separating the fractions below 10mm as is customary elsewhere. If the sieve analysis indicates deficiency or excess of fraction below 10mm, the grading is corrected by adding either 10mm or 20mm single size crushed stone, small quantity of which was kept at site as a standby. The sand was coarse (fineness modulus 2.5 to 3) and used without any processing.

4.3 Water

The river water obtained from the bridge site was used.

5. CONCRETE PRODUCTION

In view of the socio-economic compulsions prevailing in the country, minimum of equipment was used. Ordinary tilting drum type mixers (200 ltrs. capacity) were used for mixing concrete. The weight of aggregates was converted into volume to be measured using calibrated boxes. The contents of the boxes were checked for the weights daily prior to commencement of concrete and the height of the box adjusted if required. Water was fed into the mixer manually using 5 litre cans duly calibrated. The cement is received from the factory in 50 kg. jute bags and as such some loss in transit is expected. The cement bags are reweighed at site on a platform scale prior to use and any deficiency made good. The transportation of concrete is by manual labour. One tonne capacity hoists were used for vertical transportation for greater heights. Both internal and shutter vibrators were used for compaction.

6. MIX DESIGN

The target mean strength was arrived at based on the specification requirements and the standard of control expected to be realised by the construction agency. Very low workability was aimed at, in view of manual handling of the concrete. Apart from the 28 days strength requirements, high early strength (about 30 MPa at 72 hours) was also aimed at in order to ensure early prestressing. The strength-water cement ratio relationship was arrived at for a range of strengths based on trials at the field laboratory using the same cement and aggregates intended for the work. The choice of A/c ratio and W/c ratio are based on field laboratory tests.

6.1 Target mean strength

i) In terms of IS Code (IS:456), a margin of 1.64 times the standard deviation is added to the characteristic strength.

ii) In terms of IRC Codes, the lowest value should not be less than 85% of the specified strength. In practical terms, the margin required is three times the standard deviation to be added to 85% of the specified strength.

iii) The IRC Codes also specify the target mean strength as 1.33 times the characteristic strength for grades upto 35 MPa and a slightly lower value for higher grades.

The target mean strengths for grades 35 and 45 based on the alternatives are given below : (Values in MPa)

Characteristic strength	Standard deviation	Target Mean Strength(Alternatives) (i) ISI	(ii) IRC	(iii) IRC
35	4	42	42	47
45	5	53	53	58

The assumed standard deviations are based on previous experience of the construction agency on similar type of work. The construction agency chose to follow Alternative (ii), resulting in substantial economies while satisfying contractual requirements.

6.1.1 Cement consumption

The cement consumption for grade 20 was governed by minimum requirements for durability. For grade 45, high early strength requirement at three days governed the mix design. Typical cement consumption for different grades are -

20 MPa	:	310 kg per m^3	35 MPa	:	340 kg per m^3
25 MPa	:	320 kg per m^3	45 MPa	:	450 kg per m^3

6.1.2 90 days tests

The cube strength at 90 days was also monitored regularly with a view to eliminate any testing errors and also to utilise the 90 days strength for checking the technical requirements, in case of an odd 28 days low strength. None of the members of the bridge are expected to be subjected to full load before 90 days. However during actual execution there was no need to tap this reserve.

6.1.3 Standard deviations

Cumulative standard deviations were being worked out regularly for each grade of concrete as well as for each component of the structure. The cumulative standard deviations for all the grades of concrete were within the limits assumed in the preliminary mix design. However, the standard deviations for the components fluctuated substantially. For 45 MPa concrete, the standard deviation of components varied from 3 to 6 and the corresponding values for 35 MPa concrete ranged between 2 and 5 MPa. Even where the individual component standard deviations exceeded the values assumed in the mix design, all the individual cube strength results satisfied the contractual requirements, presumably because of the higher mean strengths.

7. QUALITY CONTROL

Two levels of quality control were operated -

a) The contractor's internal control

A separate quality control cell and a field concrete testing laboratory with a 300 tonne cube testing machine, with test sieves and cement testing facilities were established by the contractor at the project site. Here all the preliminary trials and major portion of works tests are carried out.

b) The owner's control

After selection of the mix by the contractor, the trials are repeated in the owner's laboratory before mix was approved. Spot checks on works tests are also carried out in the owner's laboratory, besides witnessing all the tests in the contractor's laboratory.

7.1 Tests on aggregate

Sieve analysis tests were initially conducted at the quarries in order to choose the right type of aggregates. Subsequently, random checks were made at the quarry in order to avoid infructuous expenditure in transporting substandard material over a distance of 300 km. The routine monitoring of grading of fine aggregate at the source was confined to checking out material passing through 600 micron sieve, as this factor was considered critical. On testing, the aggregate crushing value was obtained at 18% and the aggregate value at 22%. The corresponding upper limits permitted by the courts were 30% for concrete in wearing surfaces and 45% for all other cases. Tests for the grading and moisture content of aggregates were conducted daily.

7.2 Tests on cement

Tests for compressive strength of cement at 3 days and 7 days were conducted at the field laboratory at frequent intervals, and on receipt of each consignment from the factory, in addition to the manufacturer's test certificates, to determine the suitability of cement and also to monitor the deterioration during storage, particularly during the rainy season.

7.3 Workability of concrete

Slump and compaction factor tests were conducted daily. While the values of slump range from 0 to 10mm, the compaction factor is in the range of 0.7 to 0.8.

7.4 Works tests

The works test samples were taken daily and tested. The pressure gauges were calibrated by an independent testing laboratory once in a year. Master gauges were maintained at the site laboratory for more frequent checks, usually at monthly intervals.

8. ANALYSIS OF TEST RESULTS

Nearly 100,000 cube specimens have been tested for preliminary trials, strength at transfer of prestress, 7, 28 and 90 days strengths removal of formwork, etc. A number of tests on accelerated-cured concrete specimens as per IS:9013 were conducted. Tests were also conducted to obtain correlation between cube strengths and rebound values of Schmidt hammer which was used on two occasions to confirm the quality of the concrete where the cube test results were marginally lower.

9. HOT WEATHER CONCRETING

During summer months, with day temperatures reading 43°C, the concreting operations were restricted to either early morning or late evening. Survey of

Period		Nov. - Jan.		Feb. - Mar.		Apr. - Jun.	
Climatic conditions	Relative Humidity %	52-71		27-62		24-71	
		Max.	Min.	Max.	Min.	Max.	Min.
	Temp. $^{\circ}$C	32	7	38	9	43	19
Concrete Grade (MPa)		45	35	45	35	45	35
28 days average strength (MPa)		57	49	56	49	59	51
7 days average strength (MPa)		43	38	45	39	48	44
% of 28 days strength		76	79	79	80	81	85
90 days average strength (MPa)		61	53	69	54	64	57
% of 28 days strength		107	110	122	110	108	111

Table 1: Analysis of strength of concrete placed under varying climatic conditions.

available literature indicated possible reduction of 28 day strength under hot weather conditions. However both the trial mix results and the works test results (Table 1) do not indicate any reduction in 28 day strengths for concrete carried out during hot weather conditions, upto an ambient temperature of 43°C in the present case.

10. AGE-STRENGTH RELATIONSHIP

An analysis of a large number of test samples indicates the 7 days strength at 75 to 80% of the 28 days strength, particularly for higher grades of concrete, and the works test results at 7 days were evaluated accordingly. The 90 days strength have been found to be about 110% of the 28 days strength.

In terms of the ISI/IRC Codes, the 7 day and 90 day strengths are indicated as 66% and 110% respectively, of the 28 day strength. However, in view of the actual test ratios indicated above, the works cube test results at 7 days were evaluated, to ensure that these are not less than 80% of the characteristic strength.

11. MODIFICATIONS TO CONCRETE MIX

Based on the cumulative standard deviation for various grades of concrete, minor modifications in the mix proportions were effected from time to time, to ensure the design assumptions regarding target mean strengths. The maximum variation in A/c ratio was 0.2 and the W/c ratio for 45 MPa concrete fluctuated between 0.34 and 0.36. Though the grading of combined aggregate varied substantially from time to time within the upper and lower bound limits specified by the IS Code, such variations were not found to affect the strength of concrete at all. However, the workability was affected to some extent.

It may be noted that the mix proportions have been arrived at, based on the field laboratory trials and as such do not exactly fit into the pattern envisaged by various published methods. The percentage of fine aggregates used in the context of coarseness of sand is substantially lower than those recommended by the ACI or the British (Transport & Road Research Lab) method of mix design.

12. CONCLUSION

A level of technology appropriate to the socio-economic conditions prevailing in a developing country has been successfully evolved and realised in practice. The conception, design, engineering and construction of the bridge has been carried out indigenously out of internal resources, even though the contract was secured based on global tenders. Innovative methods of quality control as well as concrete construction which does not exactly fit into the traditionally accepted practices, have been successfully tried and implemented. A large amount of data collected from the various tests conducted in the field laboratory has helped in further rationalising the quality control methods for subsequent projects in the country.

CREDITS

Owners	:	Government of Bihar State, represented by its Public Works Department.
Designers & Contractors	:	Gammon India Limited, Bombay, India.

Quality Management for Arctic Offshore Concrete Structures

Gestion de la qualité des structures en béton armé, en mer arctique

Qualitäts-Management bei Offshore-Tragwerken aus Beton

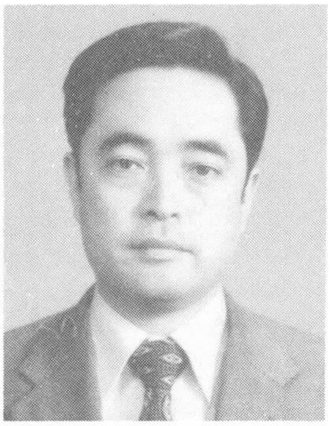
Mizuhito IGURO
Gen. Manager
Simizu Constr. Co., Ltd.
Tokyo, Japan

Tomoo SUZUKI
Assist. Manager
Nippon Kokan K.K.
Tokyo, Japan

Motokazu NIWA
Gen. Mgr., Technol. Div.
Penta-Ocean Constr. Co.
Tokyo, Japan

Mizuhito Iguro, born 1933, received his civil eng. degree at Waseda Univ., Tokyo. 8 years of site experience. Involved in the design of LNG in-ground tanks and marine structures, and in the detailed design and QC for the CIDS concrete structure.

Tomoo Suzuki, born 1950, received his M.S. degree at Kyoto Univ. He has since been involved in a number of designs and supervised projects dealing with bridges, offshore structures, and plant construction.



Motokazu Niwa, born 1939, received his civil engineering degree at the Univ. of Tokyo. He was involved in the construction of Super CIDS as a deputy manager in charge of the quality assurance of the concrete structure.

SUMMARY

This paper presents a systematic and practical approach to the internal quality management activities for constructions using high strength lightweight concrete with high freeze-thaw durability as well as water-tight caracteristics. The approach was applied to the construction of a steel-concrete hybrid mobile drilling structure, now in operation in the Beaufort Sea.

RÉSUMÉ

L'article présente une approche systématique et pratique des activités internes de gestion de la qualité pour des constructions en béton léger étanche, à résistance élevée et présentant une grande durabilité vis-à-vis du gel et du dégel. Cette méthode a été appliquée lors de la construction d'une plateforme de forage mobile, en construction hybride acier-béton. Cette plateforme est actuellement opérative en mer de Beaufort.
recherches pétrolières.

ZUSAMMENFASSUNG

Der vorliegende Beitrag stellt einen systematischen und auf die Praxis zugeschnittenen Ansatz eines firmen-internen Qualitäts-Managements für Leichtbeton-Bauwerke mit hohen Anforderungen an Festigkeit, Frostbeständigkeit und Wasserdichtigkeit vor. Der Ansatz wurde beim Bau einer mobilen, hybriden, aus den Baustoffen Stahl und Stahlbeton bestehenden Bohrinsel angewandt, welche heute unter arktischen Verhältnissen im Einsatz ist.

1. INTRODUCTION

The systematical and practical approach for the site work entails the definition of the quality for the required functional performances and structural integrities, and the developments of quality assurance program and of quality control manual accordingly. The system includes a quality control organization characterized by an internal quality control and inspection team, which stands independent of a construction execution team.

Quality items of the primary importance are cited in this paper and the practical approach to suffice these requirements along with the necessary countermeasures are described.

Particular interests lie in the control of the initial moisture content of the artificial lightweight aggregates to achieve the required durability as well as the control of thermal crackings during the construction, and the control of the dimensions and the weight of the final structure. Topics of further interests include quality control activities related to the use of silica fume as a concrete mineral admixture, the prestressing work, and fabrication of precast concrete elements. However, these topics are not covered in this paper.

2.SUPER CIDS AND ITS REQUIREMENTS

Super CIDS, the world's first Arctic mobile drilling unit, is composed of three modular units, namely the top Deck Storage Barges mounted with drilling facilities and living accomodation, the middle Concrete Basic Brick of 44 ft in height (BB44) constructed primarily of high strength lightweight concrete, and the bottom Steel Mud Base which rests directly on the seabed foundation. Super CIDS configuration and its dimensions are given in Fig.1(Ref.(1)).

The concrete basic brick consists of the bottom slab, external wall, shear walls, internal wall, silos, and the top slab. The interior is characterized by a honeycomb structure. Precast segment construction was employed to the silos. The rest was constructed by cast-in-place concrete to which prestressings were applied. Normal weight high strength concrete(NWC) was used for the internal wall and shear walls, while the rest was constructed using four different mixes of high strength lightweight concrete(LWC).

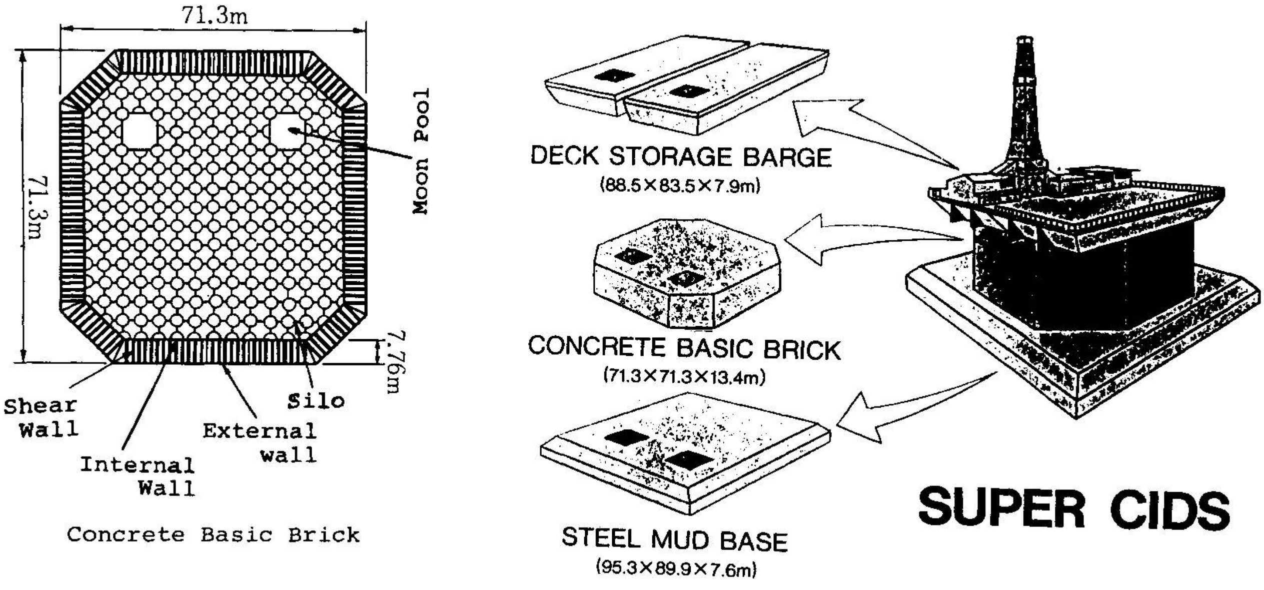


Fig.1 Super CIDS configuration

The concrete basic brick placed at the splash zone is designed to resist the most unlikely loads exerted on the structure. High strength characteristics as well as adequate durability and water-tightness are of the importance for the concrete basic brick to resist the severe temperature conditions as low as -50°c and highly intense ice pressure accompanied by the repeated freeze-thaw actions exerted on its members.

Furthermore, Super CIDS is a mobile drilling unit which is capable of submerging and refloating using its ballasting-deballasting system. Such exploratory drilling structure requires its mobility even with a greater payload to assess an all-year-round operation in the ice infested waters, which in turn implies the benefits attained by the utilization of lightweight concrete and a strict weight control.

3. QUALITY MANAGEMENT AND ITS ORGANIZATION

3.1 Specifications

Quality requirements on the finished structure were stated implicitly in the contract document as ;
- -strength and serviciability
- -durability
- -weight/draft control
- -water-tightness
- -appearance and grade controls

Furthermore, the specifications for concrete material of primary importance are summarized in Table 1.

Fresh unit weight	LWC : 1.84 t/c.m
56-day design strength	LWC : 457 kg/s.cm
Concrete Cl' content	max.0.06%cement wt.
Air content	7 ± 2%
Air-void spacing factor	max. 250 μm
Air-void surface area	min. 24 s.mm/c.mm
Freeze-thaw durability index (ASTM. C666 A)	min. 80% after 300 cycles

Table 1 Concrete specifications

3.2 Quality management

In order to achieve the specified quality requirements for the intended purpose of the structure, these implicit statements of the quality requirements were redefined explicitly in a working format in accordance with the proposed materials and the construction procedures throughout the procurement, construction, and delivery phases. Extensive in-house research work and field mock-up tests were carried out to assess a set of criteria for the quality control activities. These preparatory work encompases all the phases of the construction activities of primary importance.

Site quality management system was developed to carry out the following tasks;
- development of Quality Control Manual
- quality control activities for fresh and hardened states of concrete
- inspections of rebar and tendon placements, formwork assemblage, and the construction joints
- quality control activities for prestressing operations
- quality control activities for precast fabrications and assemblage
- inspections and water-tightness tests of the finished structure

3.2.1 Quality Control Manual

Quality Control Manual consists of three distinct parts; Quality Assurance Program, Quality Control Chart, and Manufacturer's Standards.

Quality Assurance Program simply defines the quality requirements in an explicit manner, which forms a basis for developing the working format of Quality Control Chart. All the items of quality to be controlled during the construction are clarified in their relations to the intended performance of the structure. Furthermore, the intended properities and possible deviations are cited along with the factors which rank the importance of their consequences. A portion of Quality Assurance Program is shown in Table 2.

Requirement Specified by the Owner	Intended Properties	Quqlity Evolution System	Items for Quality Control
Draft of the structure conforms to the specified value.	Maximum fresh unit weight; 155 pcf for NWC 115 pcf for LWC Torelace of member thickness ; Wall thickness 3/8 in. Slab thickness 1/4 in.	Weight of concrete conforms to the specified value. -Unit weight of concrete conforms to the specified value. -Concrete volume	Test results of unit weight Dimensions of finished members

Table 2 Quality Assurance Program (extracted)

An extensive Quality Control Chart, a working format for the site quality control activities, was developed in accordance with Quality Assurance Program which explicitly defines the quality requirements. Criteria for inspections and countermeasures for the critical construction items were established after extensive laboratory tests and field tests. Table 3 shows a portion of the chart extracted for formworks.

Work		Items of Quality Control	Survey and Inspection				Countermeasure		STANDARD /SPEC	Record of Inspection
Category	Event		Criteria	When	Method	Frequency	Prompt Action	Recurrence Prevention		
Form work	Placement of forms	Deviation from base lines	Tolerance ±1/4 in.	Upon erection of first lift	Visual observation, scale measurement if necessary	8 points	Make necessary correction	Review of work standard	ACI 347-78	Check sheet
		Differential between adjacent units	Tolerance ±1/4 in.	After form assembly	-ditto-	8 points per each lift	-ditto-	-ditto-	-ditto-	-ditto-
		Form assembled form	Tolerance ±1/4 in. per 10 ft	After form assembly	Plumb and scale measurement	-ditto-	-ditto-	-ditto-	ACI 347-78 3.3.1.1	Data sheet

Table 3 Quality Control Chart (extracted)

3.2.2 Laboratory and field tests

The following laboratory and field tests are a part of the tests carried out to confirm the applicability of the proposed materials and procedures for the construction. The results were reflected in the quality management activities.

- developments of high strength lightweight and normal weight concrete mixes with high freeze-thaw durability
- field pumpability, bucket-tremie placeability, and compactability tests for the plant mixed concrete
- air-void distribution and volume change upon compaction
- field mock-up test of external wall for the evaluations of concrete constructibility, air-void system, heat of hydration, and thermal crack control measures
- adequacy of adhesives and sealing materials for precast silo connections and prestressing anchor pockets

3.3 Internal quality control organization and its responsibilities

Verifications by the authorities and certifications for the owner are illustrated in Fig.2, in which the detailed description of Quality Control Organization is shown. Quality control activities at the concrete plant and the precast fabrication plant were under the direct supervisions of the contractior's Quality Control Team.

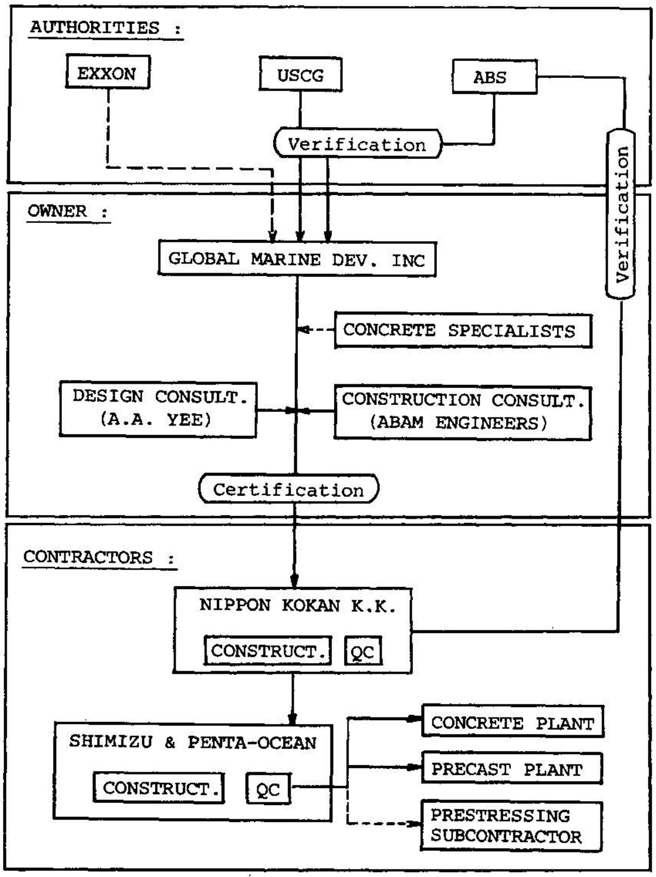


Fig.2 Quality Control Organization

The internal Quality Control Team was organized totally independent of the construction team. The team performed the following tasks;

- development of Quality Control Manual
- evaluation and acceptance of the owner's Inspection Manual in which the items and frequencies of the inspections conducted by the owner are described.
- in-house inspection work. All items subjected to inspections were covered comprehensively in accordance with and to comply with Quality Control Chart. The results of inspections were reported immediately to the construction team in charge of the specific work category. Prompt countermeasures were taken at this time and the procedures to avoid the recurrences were set forth.

4. QUALITY CONTROL FOR ATTAINMENT OF FREEZE-THAW DURABILITY

Use of lightweight concrete for the purpose of achieving the minimal draft poses a challenge to the construction technology in regards to the water absorbing nature of expanded-shale aggregate, which in turn causes the deterioration of concrete upon repeated freeze-thaw attacks. A solution to this problem as well as its quality control to assure the sufficient durability are the highlights of the construction of the concrete basic brick.

Among the various factors that contribute to the freeze-thaw durability of concrete, the amount of the free-water in the paste as well as in the aggregate is the most significant. Quality assurance activities were initiated with an extensive laboratory tests for the selections of concrete mixes and the concreting procedures to produce minimum free-water. The criteria for quality control were developed accordingly. The second step to the assurance activity was a large scale concrete placement test for the confirmation of the mix with site practices. The final step was to implement the planned quality control activities at the site.

4.1 Laboratory Tests

More than 200 batches of concrete were tested in order to finalize five different concrete mixes with optimal proportions. The result of the test is best represented by Fig.3 which clearly indicates the critical aggregate moisture content. The target value for the as-mixed aggregate quality was set at the maximum moisture content of 4%.

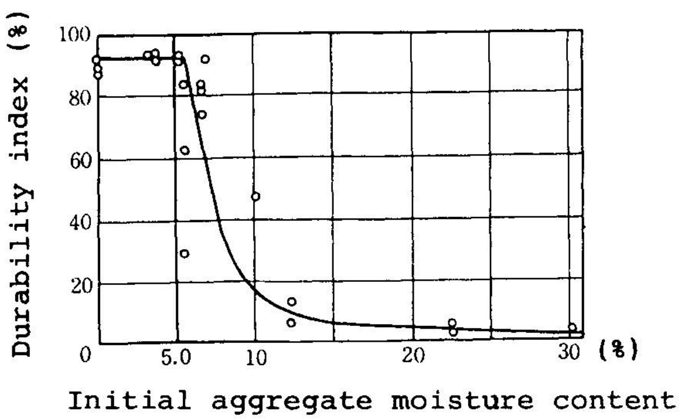


Fig.3 Freeze-thaw durability

4.2 Control of the aggregate moisture content

A proposed construction procedure for achieving the least moisture content of the as-mixed aggregate was to employ an absolute-dry aggregate upon batching. Moisture absorption was allowed during wet-mixing. The amount of mixing water was adjusted to compensate for this free water absorption.

Special cares and controls were taken during production, transportation, and stockpiling of the aggregate. Concrete was placed using a bucket-tremie system to minimize the pressure induced during the placement. The aggregate moisture content was monitored upon the delivery at the site using the mil-certificates and once a day during the concreting phase using an accelerated oven-drying procedure. This procedure, which had been calibrated to the maximum deviation of 0.2% from the standard procedure that takes 12 hours, employs an accelerated 40 minutes oven-drying procedure.

4.3 Site control work

A staff of the internal quality control team was dispatched to the concrete plant in order to enforce the quality control measures over the raw materials, especially the amount of admixtures, and to control the production rate to cope with the site work. Upon receiving concrete at the site, the quality controls were extended over the plasticization of base concrete, concrete placement and compaction procedures. Details on the concrete quality controls are given in Ref.(2).

5. CONTROL OF THERMAL CRACKING

Concrete that suffices all the quality requirements as depicted previously is inherently rich in its cement content. It is a well-known fact that such rich mix could pose a serious problem of thermal cracking due to heat of cement hydration and subsequent cooling effect during the construction. For this reason, thermal crack control measures were incorporated in the quality management.

In order to assess the most cost efficient procedure, a comprehensive analytical tool for the predictions of crack occurrence as well as the estimations of crack width and spacing had been developed. Detail description of the tool had been presented at the ACI 1984 fall convention. The information is available from the authors. (Ref.(3))

An imporvement of the proposed mix, scheduling of concreting lifts and blocks, and the selection of curing procedure were finalized in accordance with the result of the analysis for the given member configurations and reinforcing details. The applicability of the procedure was confirmed through the field large-scale test. During the construction, quality control measures were extended to the control of fresh concrete temperature and monitoring of concrete temperature rise as placed in the forms. The result of the monitoring was fed back to the construction to take the countermeasures with respect to the curing procedure. The duration of the specified curing method was also chosen to be the item for quality control. The construction was carried out commencing in September 1983 and completed in March 1984. As a result, the bottom slab and the main deck suffered no thermal cracks. Thermal cracks observed in the exterior ice wall was 0.04 mm in their maximum width. This maximum crack width corresponds to the value predicted using the proposed thermal crack evaluation system during the planning phase of the project. The result suffices with sufficient margin the specified level of the quality.

6. WEIGHT CONTROL

The total weight, and hence the draft of the final structure can be estimated from the designed quantity or from the results of the inspections on the final structure. However, either method could result in a significant error. Hence the total weight of the final structure was estimated on the basis of the delivered volumes of the materials which had been corrected for the returned or rejected volume and the losses during the construction. The volume was multiplied by the recorded unit weights of the materials, which resulted in the weight of the final structure.

The volume loss due to the concrete remained in the agitator truck was determined from the results of the measured weights of sample trucks. Furthermore, the volumetric change in the placed concrete due to compaction was measured on the sample concrete of unit volume. Three percent reduction in the concrete volume was observed. The figure 4 shows the flow for the concrete weight control. The histograms for the unit weight of fresh concrete are shown in Fig.5. The figure shows a stable result indicated by the coefficient of variation of approximately 1%. Furthermore, the actual draft upon float-out of the concrete basic brick deviated from the predicted value by a favorable margin of 1%.

7. CONCLUSIONS

Upon final inspection and hydrostatic tests, the concrete basic brick was completed, which was certified by the U.S.Coast Guard and ABS as well as the owner's representatives. The structural integrity and the seaworthiness of SUPER CIDS has been proven and acknowledged through a satisfactory performance in a severe arctic climate.

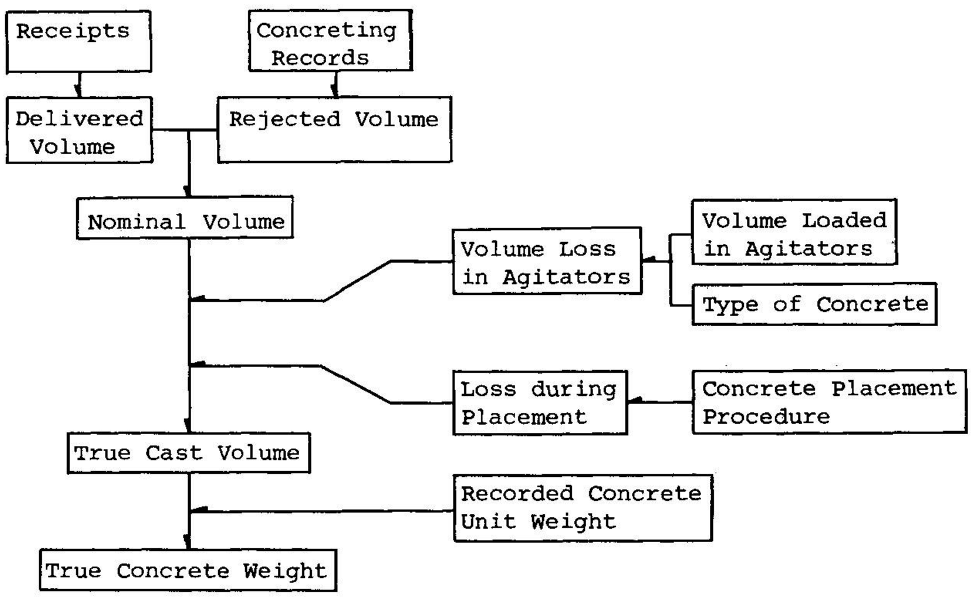


Fig.4 Concrete weight monitoring procedure

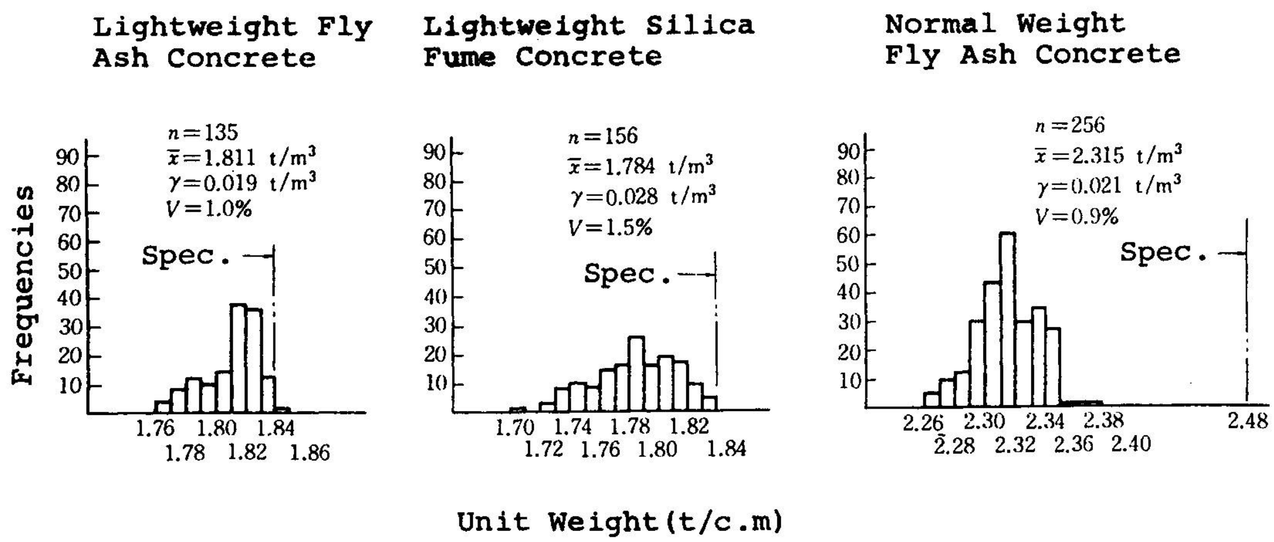


Fig.5 Concrete unit weight histograms

8. REFERENCES

(1) ALFRED A. YEE, FRED R. MASUDA, CHANG NAI KIM, AND DEAN A. DOI "CONCRETE MODULE FOR THE GLOBAL MARINE ISLAND DRILLING SYSTEM" Proceeding of the FIP/CPCI Symposia, 1984, Vol.2, P.23-30

(2) YOSHIRO ONO, TOMOO SUZUKI, MOTOKAZU NIWA, and MIZUHITO IGURO "CONSTRUCTION OF MOBILE ARCTIC DRILLING ISLAND - CONCERNING CONCRETE STRUCTURE" Proceeding of Japan Society of Civil Engineers, 1985, No.354/v-2

(3) JAMES F. MCNARY, TOYOHIKO NAKAJIMA, MIZUHITO IGURO, SADAMU ONO, and KAZUMASA INOUE "CONTROL OF THERMAL CRACKING IN THE CONSTRUCTION OF OFFSHORE STRUCTURES" American Institute of Concrete, 1984 Fall Convention

Mastering Global Quality by an Interactive Concept

Maîtrise de la qualité globale à l'aide d'un concept interactif

Umfassende Qualitäts-Sicherung durch ein interaktives Konzept

Luc TAERWE
Dr. Ir.
University of Ghent
Ghent, Belgium

Luc Taerwe, born 1952, graduated as Civil Engineer at the University of Ghent where he also obtained his degree of Engineer in Structural Mechanics and his Doctor's degree. As university assistant he is involved in research on non-linear analysis of concrete structures, cryogenic concrete, partial prestressing, quality assurance and stochastic modelling.

SUMMARY

Traditionally, the construction process is divided into different stages, each characterized by a typical quality assurance strategy that is related to the particular activities that take place. In this contribution the need for an interactive concept is emphasized as this is the only way to assure the global quality of a structure. To achieve this, it is necessary to investigate the relationships between the different construction stages, to indicate possible feed-backs and to identify information flows.

RÉSUMÉ

Traditionnellement, le processus de construction est divisé en différents stades, chacun d'eux étant caractérisé par sa propre stratégie d'assurance de la qualité et cela en fonction des activités particulières qui ont lieu. Dans cette contribution, l'accent est mis sur la nécessité d'une conception interactive, car c'est la seule façon d'assurer la qualité globale d'une construction. Pour y arriver il est nécessaire de chercher les relations entre les différents stades de construction, d'identifier et d'actualiser les flux d'informations.

ZUSAMMENFASSUNG

Der Bauprozess lässt sich in verschiedene Abschnitte gliedern, für die, entsprechend den unterschiedlichen Aktivitäten, jeweils ganz spezifische Strategien der Qualitätssicherung charakteristisch sind. Im folgenden Beitrag wird die Notwendigkeit eines interaktiven Konzepts hervorgehoben, welches allein die umfassende Qualität eines Bauwerkes sichern kann. Es ist deshalb notwendig, die Beziehung zwischen den verschiedenen Abschnitten des Bauprozesses zu untersuchen, die Möglichkeiten für Rückkopplungen anzugeben und die Informationsflüsse zu identifizieren.

1. THE CONSTRUCTION SCENARIO

As we are particularly interested in all types of interactions influencing the conception stage of a structure, it is necessary to establish a general *construction scenario*. The model given in fig.1 is certainly not complete, but it is, for our purpose, a sufficiently sophisticated reflection of reality. In stead of the "closed loop" pentagon that is used in [1], a linear model is deemed to be more suitable. Mainly two exterior sources of interaction affect the construction process.

From one side, the actions of nature constitute a system of random events that occur during the whole construction process. Although sometimes the extreme magnitudes of these events can be controlled or predicted, so that disastrous effects can be limited or reduced, we generally have to accept their occurrence. We use historical information and load surveys to elaborate appropriate safety formats.

The second class of exterior factors that influence the construction process is related to the social and natural environment. All human beings that will be affected by the existence of the structure, in particular the owner and the future users, express certain desires and expectations with respect to safety, serviceability and durability. These general and particular boundary conditions are treated in a formal way by building codes and professional experience, which are in fact the synthesis of information coming from innumerable realisations of similar projects.

Some of the most important types of feed-back are indicated in fig.1. Identification of all of them is not necessary and certainly not possible as an important number is hidden in customs and accepted procedures.

2. SOME BASIC CONCEPTS

In the title, some basic concepts are introduced that deserve some general comments before they are illustrated in detail in the following section.

2.1. The global quality

It is obvious that a global view on the quality level of the whole construction process should be available at the design stage. The following indications allow to set up a global appreciation.

The *global quality level* could roughly be obtained as an integral of contributions resulting from all stages or sub-stages of the construction process. However, in that way a mean value could result that conceals low and high quality levels, although this is what sometimes remains as the final impression of the owner.

To arrive at a more balanced appreciation, an extension of reliability formats could turn out to be useful. We define a quality function $q\ (\underline{X})$ similar to a limit state function and associate a quality index to each construction stage. To obtain the global quality index,the system has to be carefully modelled and serial and parallel connections have to be identified. This approach needs a judicious analysis of the construction process since certain dependencies exist between different stages. One aspect of dependency is that a deficiency of quality at one stage sometimes can be remedied during one of the successive stages. On the other hand however it can be amplified later on.

Although the practical application of this approach can be questioned, its main aim is to indicate that in practice, quality assurance is often too much differentiated. By this we mean that decisions are made separately for the different activities in the construction process and that the effect of a decision on

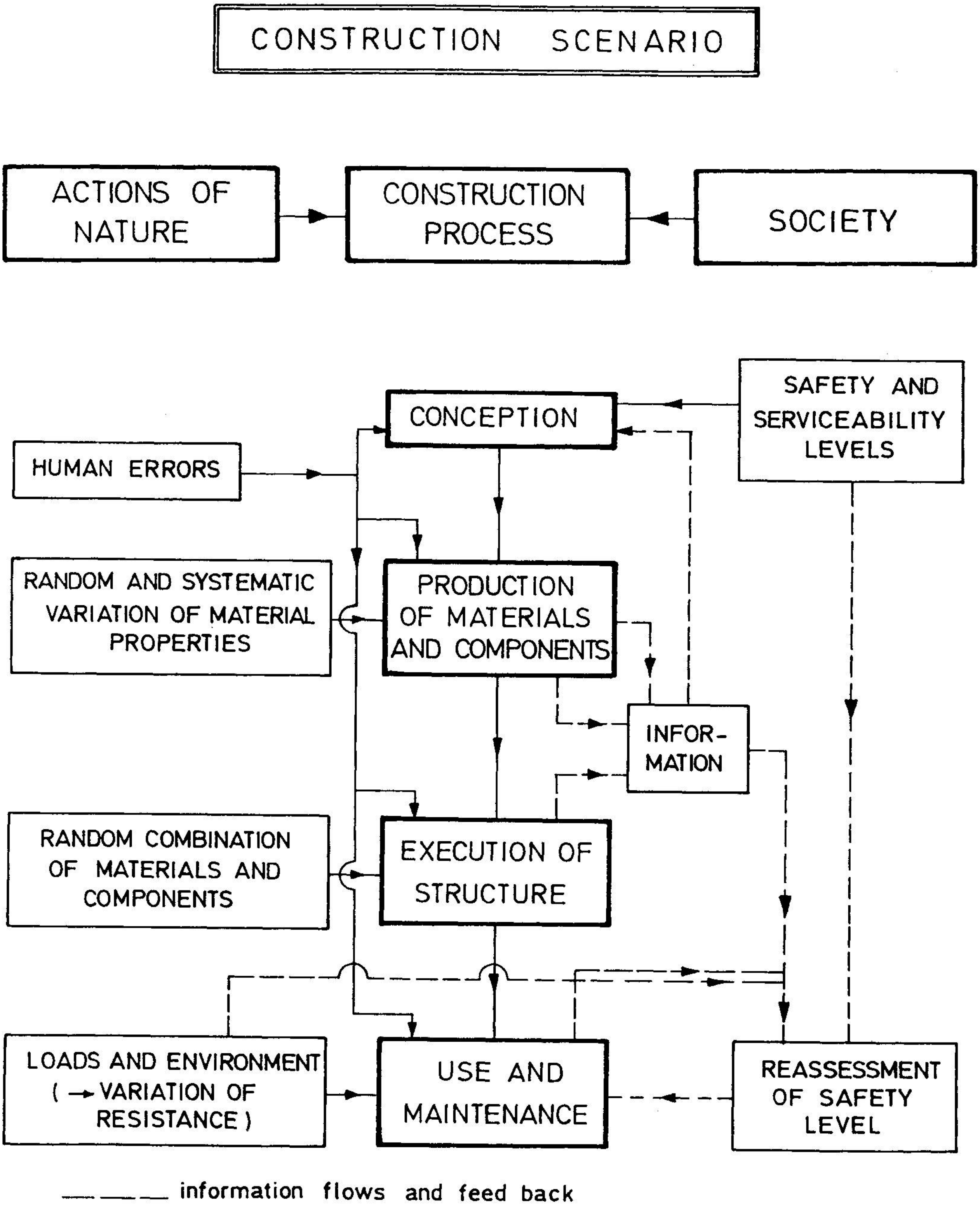


Fig.1 A simplified version of the construction scenario

the quality level of other activities is often ignored. This can lead to serious complications that are easily avoidable if appropriate planning at the design stage occurs.

These interactive aspects indicate the need for a global view, not only on the construction process but also on the quality level because it is precisely the global level that will determine the long term structural behaviour.

2.2. Conception of a structure

It is necessary to introduce the notion "conception of a structure" within the framework of this paper. As a first attempt, we could simply state that the conception of a structure includes planning and design. This definition is too limited however. The *conception of a structure* is the transformation or materialisation of the original idea into a real construction plan and execution guidelines. The conception of a structure gradually grows as more aspects are considered and more information becomes available. The final version has not only a static part that remains fixed during the execution stage, but has also a flexible part that is execution and time dependent. The interactive character is equally important in both parts and mainly depends on the number of feed - backs that exist and on the level at which they intervene. In this way the "conception" is not just a single stage in the construction process, but is integrated in and based on the whole construction scenario.

2.3. Quality mastering

An interactive conception of the construction process, automatically results in *"quality mastering"*, a notion that could be seen as a mature state of "quality assurance". Indeed, by assuring that a sufficient quality level will be obtained, we only look at the achievement of a predifined performance level, e.g. related to an execution stage or to supplied components or materials. With respect to the previously defined concepts "global quality" and "conception of structure", it is clear that we have to introduce quality strategies on a global or local scale, by making use of our knowledge concerning possible interactions, all types of information flows and data on production processes and execution schemes. Quality mastering reflects the insight we have in the activity we are dealing with and our ability to control the parameters that govern it. It follows that quality mastering also offers advantages in case of deficiencies, because it can easily offer alternative strategies.

One could also state that quality mastering leads to the most optimal solution of a construction process taking into occount in a flexible way the exterior constraints and boundary conditions that appear in the construction scenario.

3. PRACTICAL REALISATION OF AN INTERACTIVE CONCEPTION

In the following, different examples are discussed and some concepts are introduced to illustrate which events interact with the conception stage, in a direct or indirect way, and how these events influence the global quality of a structure.

3.1. Safety and serviceability criteria

a) The codified safety formats we use, are the reflection of the confrontation of existing structures with the actions of nature listed in the first column of fig.1. In fact they allow to forecast structural behaviour on the basis of engineering extrapolations of different types of information and experience, taking into account the requirements imposed by the social environment. The distinction between safety requirements (here related to the ultimate limit states) and serviceability requirements is not always clear. Moreover, a large degree of graduation and differentation exists.

b) Consider the case of concrete structures. Generally, crack limitation is considered as a serviceability limit state. In fact cracking influences corrosion and deterioration and hence affects durability, a property that is related to the way in which all limit states are affected by time. Mathematically it can be expressed by the evolution in time of structural reliability during the accepted reference period. In this way it is possible that certain serviceability limit states are less critical initially, but become decisive as the age of the structure increases. All things considered, cracking should not be interpreted as a serviceability limit state except perhaps for large crack widths that are not in agreement with structural aesthetics and that could alarm the public mind. Moreover, it is recognized nowadays that crack widths less than 0.4 mm generally are not dangerous with respect to durability [2]. Hence, graduation of maximum crack widths less than 0.4 mm as mentioned in [3], is only a conventional way of differentiating serviceability requirements without real physical background.

c) A remarkable example of the relativity of safety and serviceability is the Leaning Tower of Pisa (fig.2). Obviously, the probability of collapse is higher than all existing codes would allow. However, its serviceability as a "campanile" is not impaired since its impressive bell is still chiming perfectly. The presence of a local soil weakness underneath its foundation even increased its utility since it became a major touristic attraction. This exceptional example is not intended as an incentive to leave judgement of actual structures to our descendants but it illustrates that our value system can change in unexpected ways.

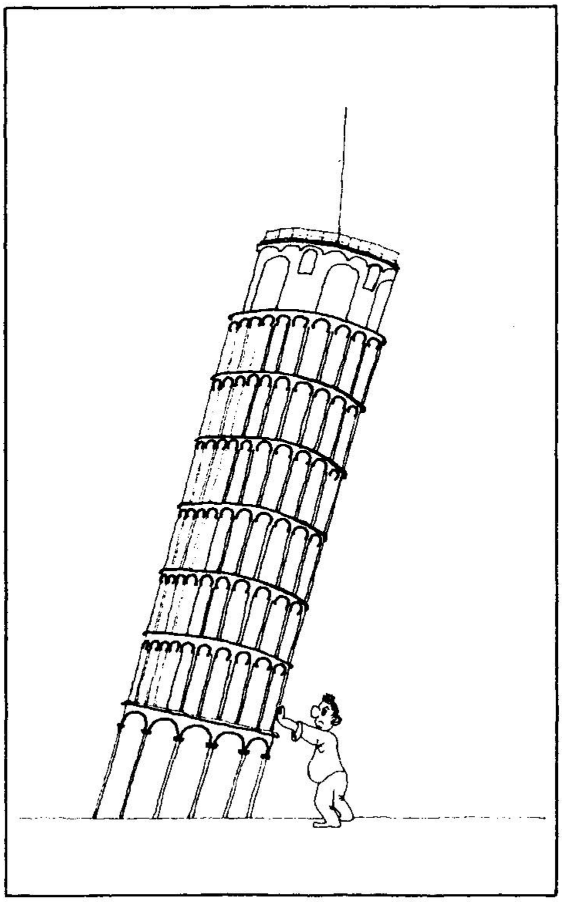


Fig.2

The relativity of reliability

d) Sometimes, new types of hazards arise that require the introduction of new safety regulations and that can even give rise to the use of new material combinations. In Belgium, regulations and research concerning fire resistance were thoroughly extended and updated as a consequence of a calamitous fire in a large shopping centre, causing the death of more than 200 people, some 20 years ago. Provisions for fire safety resulted in a wider application of composite steel - concrete columns since, by using concrete as a protective cover for steel columns, one can also benefit from its contribution to the column's bearing capacity.

e) In order to assure a sufficiently high global quality, it is clear that ambiguous and complicated design rules should be avoided as this may give rise to doubtful or illegitimate interpretations by the designer, generally in the sense that the most economical solution is retained.Often the consequences this solution has with respect to safety will not be investigated. The Belgian Standard for the design of concrete structures allows the application of limit state design as well as the allowable stress method. However, certain engineers use one method for a first limit state (e.g. bending) and the other method for a second limit state (e.g. shear), if this turns out to be more favourable. This approach reflects the poor "safety attitude" of certain designers.

3.2 Conceptual saturation and versatility of structures

a) Another interaction with society is related to the fact that our structures must be accepted by it and correspond with its needs. The appreciation that follows from succesful designs will result in a better motivation of designers and in the practical benefit of receiving more commissions.

To achieve this, it is often useful to introduce new concepts in order to avoid *"conceptual saturation"*. However, this should not result in an inordinate striving for novelties or for experiments with bizarre architectural compositions.

b) When the first tall buildings were erected, everyone was excited by the technical novelty of it. Later on it turned out that these structures, which had a very high environmental impact, became typical for a cool, impersonal grey style of building that often receives the label "steel, concrete and glass style". This caused the need for more harmonic structures (buildings, bridges, highways, ...) on a human scale and incorporated into the natural and historical environment. On the other hand this can lead to an exaggerated attention to detail whereas the global harmony is missing. This latter aspect is illustrated by the reflections of one of the members of the jury of the "Brickwork award for architecture" that is organized yearly in Belgium [4]. Considering the various contributions he writes : "Often there is no pronounced *Gestalt*, no immediately recognizable basic form, but rather a fashionable aftermath of juxtaposed spatial, physical, functional or technological elements" (fig.3). Again the "global quality" concept appears.

c) It is important that designers be fully aware of the mentioned conceptual saturation mentioned above, because the periodic renewal of the needs of the client and his evaluation pattern, determine in great measure the future of the building industry.

To cope with the future demands of the users, it is necessary to offer flexible and *versatile structures* not only with respect to serviceability requirements but also with respect to safety. To illustrate this latter aspect, we mention the growing interest in external prestressing of concrete bridges. This technique offers the advantage of easy removal of cables and allows to strengthen older bridges so that a heavier traffic load can be accomodated. By taking some necessary, yet simple precautions, the uncertain future demands can easily be coped with eventually .

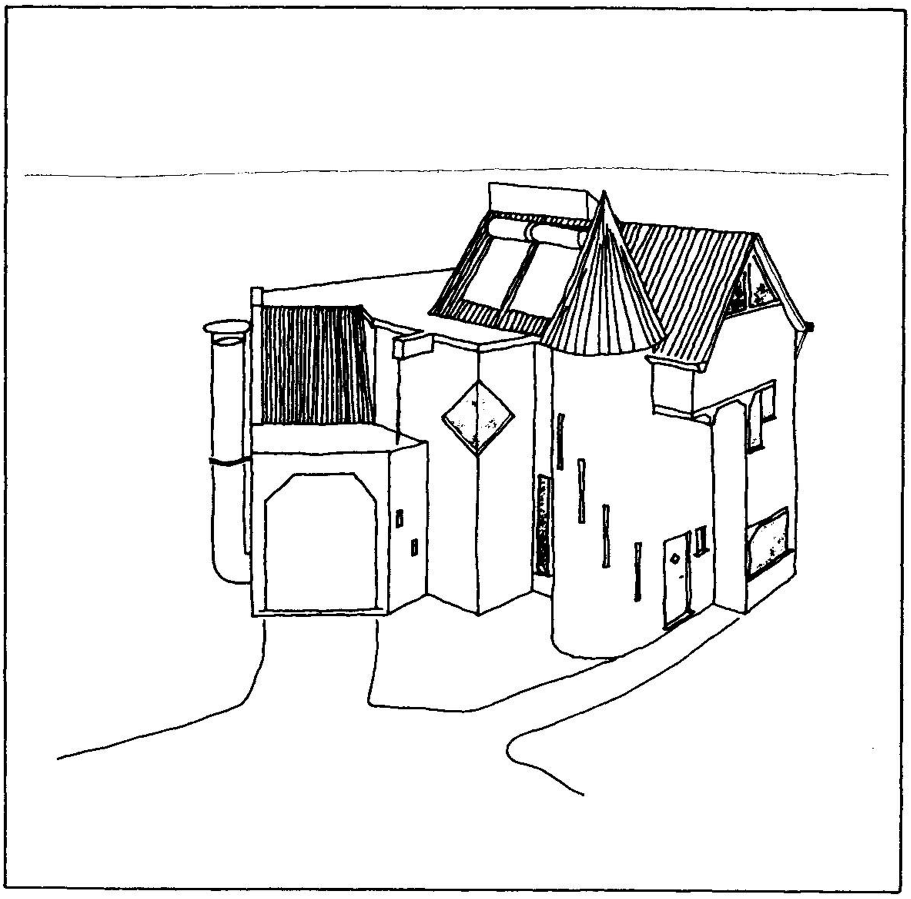


Fig.3 Architectural experiments versus global harmony (or quality) ?

Obviously we cannot proceed like the electronics industry and almost continuously create new needs by offering new or more performing products. This is related to the fact that the planned life-time of these goods is much lower than we are used to in building and in civil engineering, and that the total cost of structures is a large multiple of the cost of the products of the electronics industry (large mainframes excepted).

3.3 Interaction and communication between the different partners

a) Every partner involved in the construction process, projects the structure in his own realisation space. When the different resulting sub-spaces do not coincide, e.g. because the individual solutions are different, a compromise has to be reached in order to avoid lasting disagreements between the different parties. To avoid this situation, sufficient communication and exchange of information between the different partners is necessary already in the early stages.

b) This not only holds during the conception and execution stages but equally during the life-time of the structure. This latter aspect is strikingly illustrated by the example of a bridge failure discussed in [6]. The structure suddenly collapsed after 8 years of service. It turned out that failure occured because of dispersion of responsabilities between different agencies and departments of the Ministry of Public Works and because of a lack of communication between them.

c) At the conception stage, each party involved already thinks at the execution level and establishes its own optimal production strategy almost subconsciously. This optimal strategy results in a certain class of materials and a certain type of structure being applied in the major part of realisations just because they represent the most economical solution. As a consequence, a certain degree of standardization is established since alternative solutions or exceptional or novel structures generally appear more expensive. However, this is not necessarily so since in the neighbourhood of the minimum, the utility varies slowly. This general trend partly explains the traditional character of the building industry that is not very open to artistic inputs and new experiments. Let us quote [6] : *"L'art sème le désordre dans la vie..."*. Although we should not make the conception subordinate to execution, a beneficial interaction is desirable since,especially in difficult periods, the building industry really needs a sufficient number of technical innovations to remain competitive. One of the most innovative periods in the field of concrete structures was undoubtedly related to the development of prestressed concrete.

In the case of new materials, the higher costs are understandably caused by development, prooftesting and initially higher partial safety factors.

d) Sometimes the optimal solution follows from needs arising during the construction stage which have consequences with respect to design. Examples are the incremental launching method for bridge construction and the use of precast concrete slabs in buildings to reduce formwork costs. Both types of construction influence the design procedures (long term feed-back).

3.4 Human errors

In the following we point out how human errors reflect a kind of interaction with the past, but also with the future.

When considering human errors as an interaction with the past, we refer to experience that causes decreasing awareness and to routine that makes the inspection frequency no longer random or systematic but subjective and correlated with the production output.

When considering human errors as an interaction with the future, we consider the problems engineers were faced with by the introduction of new types of structures and the increase in dimensions of different common types of structures. Experience largely guided our capacity for extrapolation, but new phenomena appeared such as buckling, aeroelastic instability, temperature effects etc. These errors occur in the design stage but also erroneous actions in the operation stage are more likely to occur as structural complexity increases. Application of CAD and CAM offers the possibility to spend less time on routine activities and to focus attention on the particularities of complex systems. A possible disadvantage is that computer programs are generally developed for classical types of structures and that in this way innovation could be hampered.

4. CONCLUSIONS

- On the basis of a general construction scenario, it is investigated what types of interaction influence the conception stage of a structure. It is explained that the notion "conception of a structure" is broader than planning and design.
- It is emphasized that a global view on quality is necessary as it is found that in practice quality assurance is often too much differentiated.
- "Quality mastering" is introduced as a mature state of quality assurance.
- It is shown that a clear distinction between safety and serviceability

requirements is not always easy.

- An interactive conception is necessary in order to prevent conceptual saturation and to obtain versatile structures.
- Interaction between the different partners is discussed and it is pointed out that a certain degree of standardization almost automatically results.
- Finally, some aspects of human errors influencing the conception stage are summarized.

5. ACKNOWLEDGMENT

The author is grateful to his colleague Johan Verbeke for drawing figures 2 and 3, and to Prof.D. Vandepitte for suggesting some small improvements in the manuscript.

6. REFERENCES

1. CEB, Quality Control and Quality Assurance for Concrete Structures, Bulletin d'Information No. 157, 1983.
2. SCHIESSL P., Admissible crack width in reinforced concrete structures, Preliminary Report, Vol.2, IABSE-FIP-CEB-RILEM-IASS Colloquium, Liège, June 1975.
3. CEB-FIP, Model Code for Concrete Structures, Volume II, 3rd Edition, 1978.
4. Baksteenprijs 1985 - Verslag van de jury (visie van J.Baele), Bouwen met baksteen, Nr.40, 1985.
5. VANDEPITTE D., Collapse of the Bridge at Pulle, Proceedings IABSE Workshop, Rigi 1983, IABSE Reports, Volume 47.
6. PORTOGHESI P., Au-delà de l'architecture moderne, L'Equerre, Paris,1981.

Leere Seite
Blank page
Page vide