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New Approaches to Monitoring and Repair of Concrete Structures

Innovations dans la surveillance et la réparation des structures en béton

Neue Wege zur Ueberwachung und Instandsetzung von Betonbauwerken

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SUMMARY

Corrosion of reinforcement due to severe micro-environments and/or concrete quality has caused serious damages to concrete structures. To avoid damage and repeated damage after repair, new approaches to durability design, monitoring and repair of concrete structures, based on the mechanisms governing the deterioration processes, are presented. Like for load design, design for durability should be performance-based, taking into account the probabilistic nature of the processes and the material properties involved. Testing of the durability related quality parameters and monitoring the structures during use must be integral parts of the design and maintenance procedures for concrete structures.

RÉSUMÉ

La corrosion des armatures due à l'action d'environnements sévères et/ou à la pauvre qualité du béton sont à l'origine de détériorations graves des ouvrages en béton. Dans le but d'éviter ce type de dégradation et l'apparition de détériorations après remise en état, de nouvelles approches, en vue d'une bonne durabilité, sont présentées pour le projet, la surveillance et la réparation des ouvrages en béton. La définition de la durabilité peut, et doit, tout comme la définition de la charge, être exprimée en termes de performance et prendre en compte la nature probabiliste des processus et des propriétés des matériaux impliqués. Le contrôle des paramètres qualitatifs agissant sur la durabilité ainsi que le suivi des ouvrages en béton pendant leur exploitation, font partie intégrante des procédures d'étude et de surveillance de ces derniers.

ZUSAMMENFASSUNG

Korrosion an der Bewehrung infolge extremer Umwelteinwirkungen und/oder mangelhafter Betonqualität hat erhebliche Schäden an Betonbauwerken verursacht. Zur Vermeidung solcher Schäden und neuer Schäden nach Instandsetzungen werden neue Wege zur Dauerhaftigkeitsbemessung, Ueberwachung und Instandsetzung von Betonbauwerken vorgestellt. Dauerhaftigkeitsbemessung kann und muss, wie die Lastbemessung, auf Performance-Konzepten beruhen und auf probabilistischer Basis aufgebaut werden. Eine Ueberwachung der dauerhaftigkeitsrelevanten Qualitätsmerkmale und eine Ueberwachung der Bauwerke während der Nutzung sind integraler Bestandteil von Bemessungs- und Ueberwachungskonzepten.

1. INTRODUCTION

Corrosion of reinforcement caused by carbonation of concrete, chlorides and/or low quality of the concrete cover has caused serious damages to concrete structures in recent years. There are various reasons for durability failures, some of them being

- non-awareness of the problem by the people responsible for design, execution and use
- insufficient problem related education and knowledge of the same group of people
- poor codes of practice
- non-existing real design for durability
- insufficient durability related quality assurance and quality control procedures
- non-existing or insufficient maintenance during use.

All these aspects need to be covered by Design Concepts for Durability, both for the design of new structures and the design of repair of damaged structures.

One important basis for design and repair concepts and procedures must be a clear understanding of the mechanisms governing deterioration and the resistance of our materials and structures against deterioration. The understanding needs to start with the micro-environment, to continue with the interaction of the micro-environment with the structures, the transport and deterioration processes within the structure and to end with the consequences of repair measures. For one of the most important deterioration mechanisms, corrosion of the reinforcement, a new approach for durability design, monitoring and repair of concrete structures will be outlined in this paper.

2. CORROSION OF STEEL IN CONCRETE - MECHANISMS

2.1 Initiation Process

Carbonation

The mechanisms of carbonation of concrete and the consequences thereof are well understood nowadays. Due to carbonation the initially high pH-value of the concrete pore water - pH > 13 for OPC-concretes - falls below 9, thus causing depassivation of the reinforcement and the risk of corrosion. The main parameters influencing the rate of carbonation are

- permeability of concrete cover, mainly depending on w/c-ratio and curing
- alkaline buffer capacity of the matrix, mainly influenced by the type of binder
- moisture content of the concrete, depending on the micro-climatic conditons at the concrete surface.

Although no real durability design procedure exists, carbonation of concrete will normally not lead to corrosion problems as long as the requirements related to concrete composition, cover and curing given in the existing codes will be fulfilled.

Chlorides

Compared to carbonation problems in connection with chloride induced corrosion are by far more complex, complicated and unsolved:

- Transport and binding mechanisms of chlorides in the porous matrix, chloride treshold values as well as the corrosion process itself are far away to be fully understood in connection with the prevailing micro-environmental conditions.
- The severity of chloride environments varies within a wide range.

- Traditional protection measures (high quality of concrete cover) may be insufficient to ensure a sufficiently long service life (e.g. 50 years) in the case of severe attack (e.g. chloride containing splash water).
- Corrosion due to chlorides normally is much more severe compared to carbonation induced corrosion and may therefor affect the structural stability.
- A reliable repair of chloride induced corrosion is much more difficult and costly (if not impossible at a late corrosion stage in extreme cases) compared to carbonation induced carbonation.

Chlorides penetrate into the concrete within partly or completely water filled pores. They are partially bound chemically or physically within the cement matrix. Unlike carbonation, chloride penetration is not associated with a reaction front. On the contrary, a chloride profile with a chloride content decreasing from the concrete surface to the interior is usually found.

A detailed description of the major influencing factors on chloride penetration is given in [1]. They can be summarized as follows:

- Transport mechanisms: Two basic transport mechanisms are involved, chloride ion diffusion and chloride transport combined with water transport. Under practice conditions pure chloride ion diffusion is negligible. Practically all chlorides penetrating into the concrete are transported with water.
- Permeability: A decrease of the w/c-ratio from e.g. 0.6 to 0.4 increases the penetration resistance (expressed by the effective diffusion coefficient D_{eff}) by a factor of 2 to 4. Bad and good curing have influences in the same order of magnitude.
- Type of binder: By far more important is the type of binder. The penetration resistance ($1/D_{eff}$) is about a factor 10 higher for concretes made from slag- and fly-ash cements with high amounts fo blending agents compared to OPC concrete. By far the worst chloride penetration resistances show SRPC-concretes.

It is evident from current knowledge that the critical chloride content is not a fixed value but is dependent both on the quality of the concrete cover (type of cement, w/c-ratio, curing, thickness) and on environmental conditions. Fig. 1 presents this relationship in qualitative terms [2].

In various studies considerably varying results have been obtained with respect to the critical chloride content. Some of these contradicting test results can be explained by the influence of the test method and set up employed. However, as the interrelations are extremely complex, more precise conclusions as compared to Fig. 1 cannot be drawn at present. Critical chloride contents for existing structures can therefor be determined only for the specific situation of single structures on the basis of special investigations, taking all decisive influencing parameters and mechanisms (e.g. macrocell corrosion, see below) into account.

2.2 Propagation Process

Reinforcement corrosion visible with the naked eye is the outward manifestation of the effects of a number of small corrosion cells on the steel surface.

A corrosion cell is essentially a short-circuited battery consisting of a metallically and electrolytically connected anode and cathode. Unlike the process in a rechargeable battery, the corrosion process is not, however, reversible. The voltage of a corrosion cell is set up through differences in potential on the steel surface. The differences in potential needed for corrosion of the reinforcement may be caused by overlapping local differences in the chemical composition of the concrete, differing aeration conditions, inhomogeneities in the steel surface or uneven coating of the steel surface with corrosion products. In the case of concrete-covered steel, there are always differences in potential.

Cells where the differences in potential result from the combination of different metals are referred to as "contact cells", cells with differing concentrations of certain components in the electrolyte, in this case the pore solution of the concrete, are termed "concentration cells" and cells with differing levels of oxygen admission to the steel surface are known as "aeration cells".

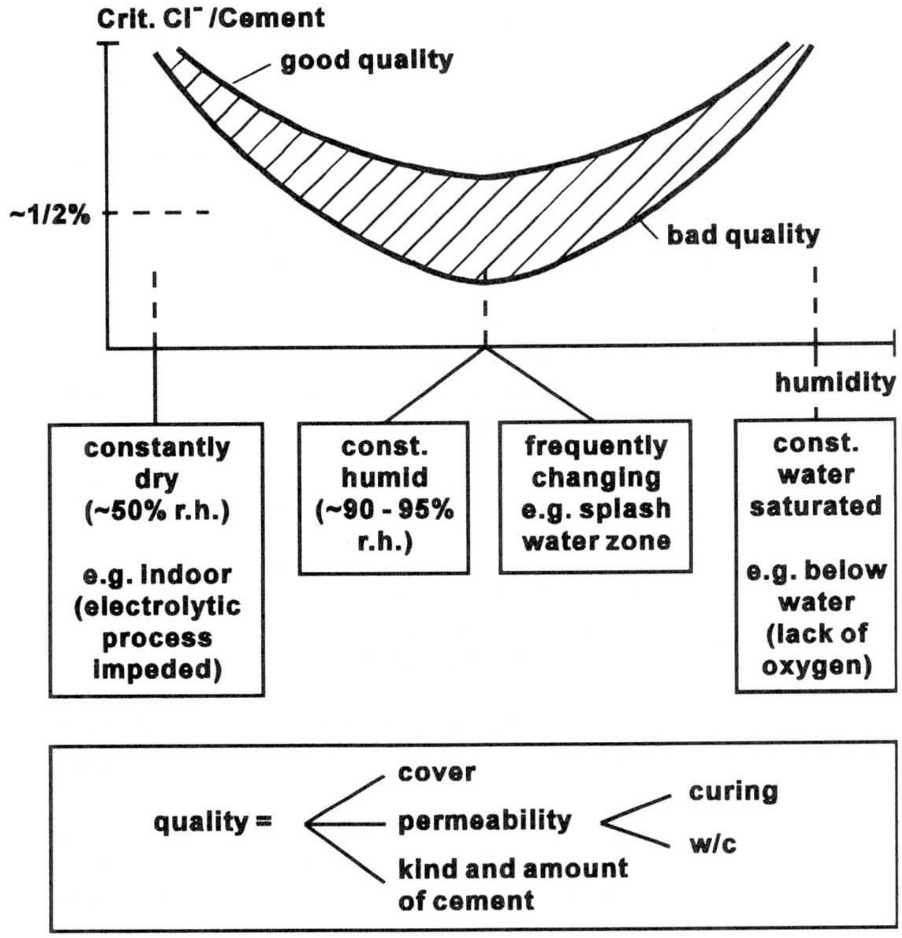


Fig. 1 Influence of concrete cover and environmental conditions on critical chloride content

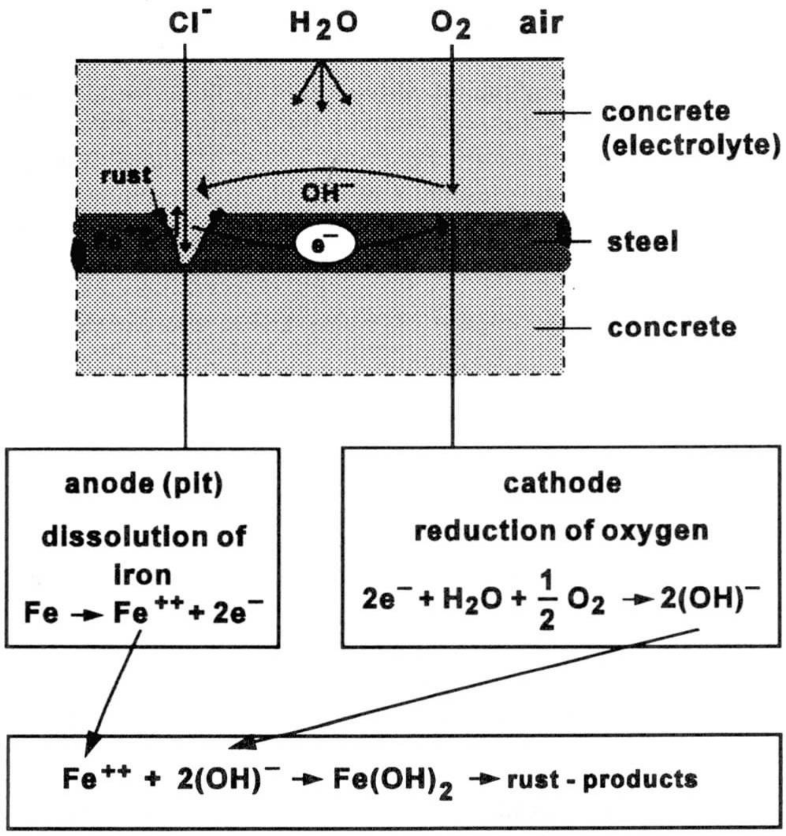


Fig. 2 Diagram showing chloride-induced corrosion of steel in concrete schematically

The corrosion process takes place in two sub-processes, as outlined in Fig. 2:

- At the anode, iron ions pass into solution, separating from the electrons. They are converted into rust products in further reactions.
- At the cathode, electrons, water and oxygen are converted into hydroxyl ions. The cathodic process doesn't cause any deterioration of the steel.

These hydroxyl ions transport the negative charge in the electrolyte through the electrical field created between the anode and the cathode, towards the direction of the anode. Near the anode, they react with the steel ions in solution. Depending on moisture and aeration conditions, this intermediate product may continue to react, producing the final corrosion products.

Individual processes are in fact much more complicated. The RILEM Report (60-CSC) [3] indicates the state of knowledge in this field.

In order for the corrosion process to take place, a number of preconditions for the anodic and cathodic process and for the electrolytic process must be satisfied simultaneously:

- As already noted, there must be differences in potential. The preconditions for sufficiently large differences in potential are, however, virtually alsways met, and in the case of chloride-induced corrosion these may be several 100 mV.
- Anodic and cathodic surface zones of the steel must be connected electrically and electrolytically, i.e. a flow of electrons and ions between them must be possible.
- The metallic connection necessary for an electron flow from the anode to the cathode is provided by the reinforcement system in the reinforced concrete. The electrolytic connection is represented by the concrete. This must, however, be sufficiently moist, since otherwise there can be virtually no migration of ions. In dry interior situations, for example, the electrolytic conductivity of the concrete is too low to permit corrosion of the reinforcement, even if the carbonation front reaches the reinforcement, leading to loss of alkalinic protection.
- Anodic solution of iron must be possible due to depassivation of the steel surface. The cathodic process can, however, take place even in zones with a passive steel surface.
- Sufficient oxygen must be available at the cathode. There must be continuous diffusion of oxygen from the surface of the concrete to the steel surface acting as the cathode. There is therefore practically no risk of corrosion to reinforced steel components which are permanently immersed in deep water.

If all conditions for corrosion are fulfilled simultaneously, the reinforcement will corrode. If only one of the conditions can be eliminated, corrosion can be prevented or brought to a stop. This knowledge is the basic key to fundamental repair principles (see chapter 4).

3. MONITORING AS A PART OF A RATIONAL DURABILITY DESIGN PROCEDURE

3.1 Design-For-Durability Approach

A performance based design is well established for load design, however no rational design approach for durability exists. We still apply simple deemed-to-satisfy rules (w/c, cover etc.) related to poor environmental classifications. A framework for a rational approach to design for durability is sketched in Fig. 3. It needs to start with the definition of the desired performance during the anticipated life time, e.g.

⇒ depassivation front (carbonation or chlorides) shall not reach the reinforcement, corrosion excluded.

Depending on the definition on performacne criteria, the aggressivity of the environment and the maintenance strategy (e.g. no maintenance possible) the basic defense strategy needs to be chosen.

In principle there are two basic defense or protection strategies

A Avoid the degradation reaction considered.
B Select optimal material composition and detailing to resist the degradation reaction considered.

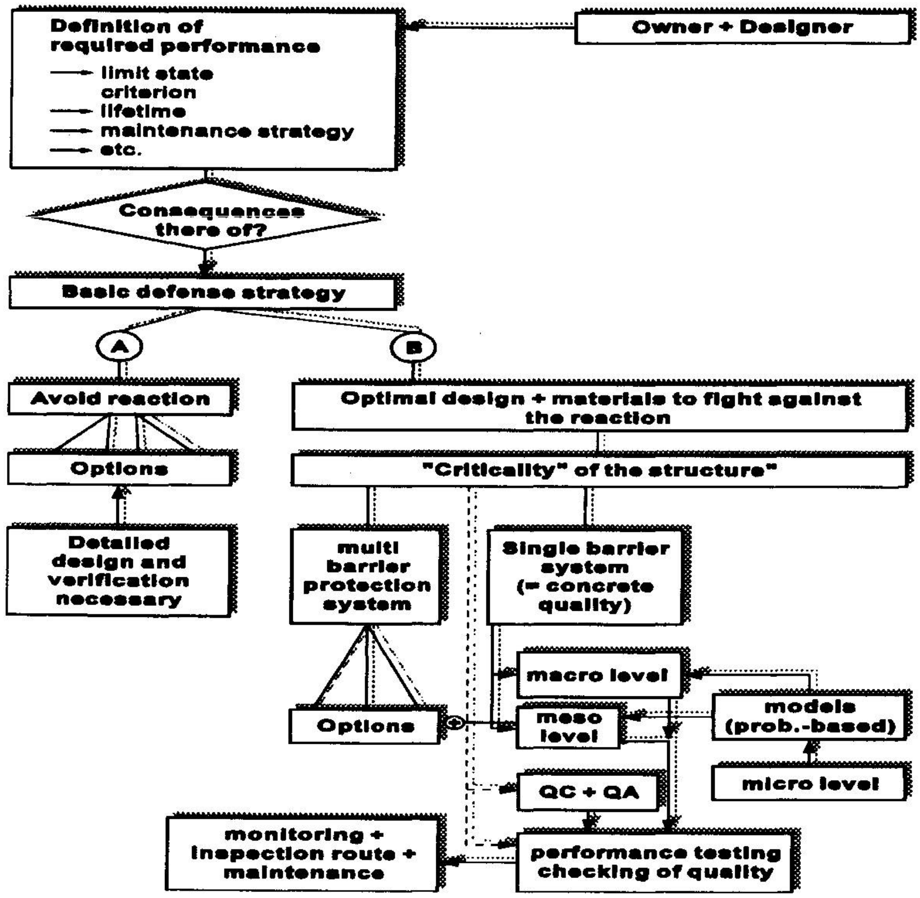


Fig. 3 Format for Durability Design

Strategy A can be subdivided into three possibilities:

A1 Change the environment, e.g. by tanking, membranes, coatings etc.
A2 Select non-reactive materials, e.g. stainless steel, coated steel.
A3 Inhibit the reaction, e.g. cathodic protection or cathodic prevention.

Strategy B is to fight against deterioration by optimal design and choice of materials. The choice of the protection strategy very much depends on the micro-environmental aggressivity. For structures or parts of a structure exposed to very aggressive environmental conditions or/and long target service lifes (100 years or more) multi barrier protection strategies are advisible. Such a strategy could be the provision of

- excellent concrete quality plus
- increased cover plus
- extra protection for the concrete or the reinforcement plus/or
- provisions for later extra protection should it become necessary.

In addition to these strategies monitoring of the structures e.g. by installing sensors into the most sensitive parts of the structure is indispensible (see chapter 3.2).

The design of concrete quality and concrete cover may be done on different levels. For the majority of structures the „macro-level" is sufficient. Macro-level means to apply prescriptive rules like we use than nowadays (w/c-ratio, cover thickness etc.), however based on probabilistic models describing the deterioration mechanisms encountered and related to better defined micro-environmental conditions.

Identifying and classification (quantifying) the aggressivity of the environment, modelling the transportation of aggressive substance into and within the concrete, and modelling the possible deterioration mechanisms, becomes the first challenge to overcome if service life designs shall become realistic. The definitions and classifications of environments in existing standards are absolutely insufficient for a real durability design. This will be examplified in a simplified way for a partly immersed sea structure (see Fig. 4). If we, for example, consider the risk for reinforcement corrosion, the sketched micro environments (1 to 5) and transport mechanisms cause extremely varying aggressivity along these structures. The difference in the risk of reinforcement corrosion between the low aggressive areas (under water and air zone) and the highly aggressive areas (splash water zone) may be in the range of one order of magnitude.

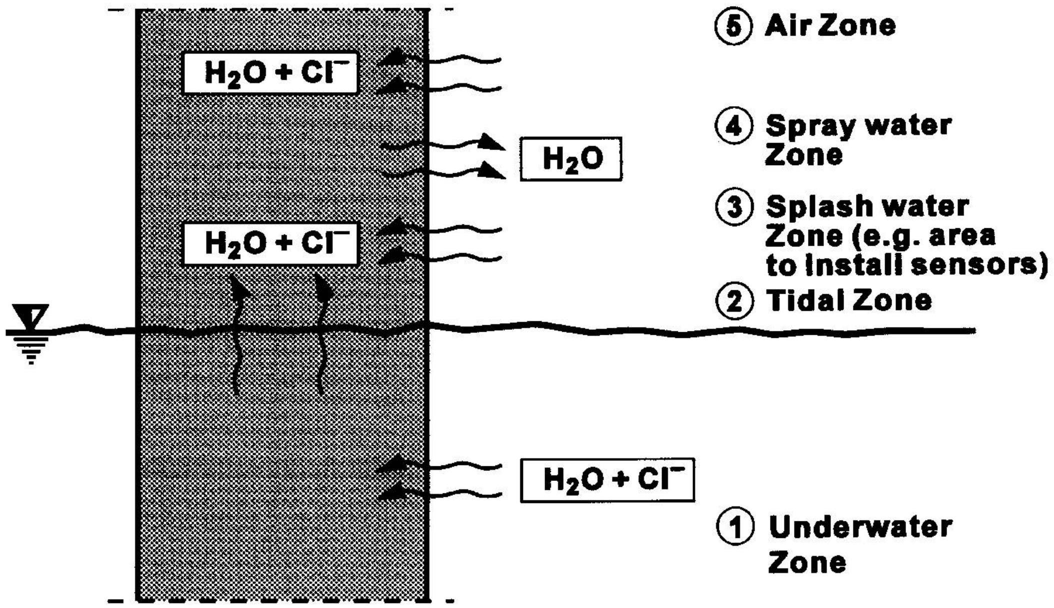


Fig. 4 Extremely varying micro-environmental conditions
Example: Marine Structure

For structures as sketeched in Fig. 4 the meso-level (see Fig. 3) approach is appropriate. The time dependant penetration of chlorides can be calculated using simplified penetration laws modelling the prevailing penetration mechanisms. Within such a model the effective diffusion coefficient could be the relevant material parameter, e.g. in a very simplified way

$$d_{Cl} = K \cdot \sqrt{D_{eff}} \cdot t^n$$

d_{Cl}: penetration depth of chlorides (critical chloride content)
K: factor describing the micro-environment
n: exponent describing the micro-environment
D_{eff}: effective diffusion coefficient

As the effective diffusion coefficient is the performance parameter of concrete governing the resistance of the structure it needs to be incorporated in the material characterisation, starting from the production of the binders to the execution of the structure. The overall system is sketched in Fig. 5. If we replace D_{eff} in Fig. 5 by strength we realise exactly the system we are using for load design since ever.

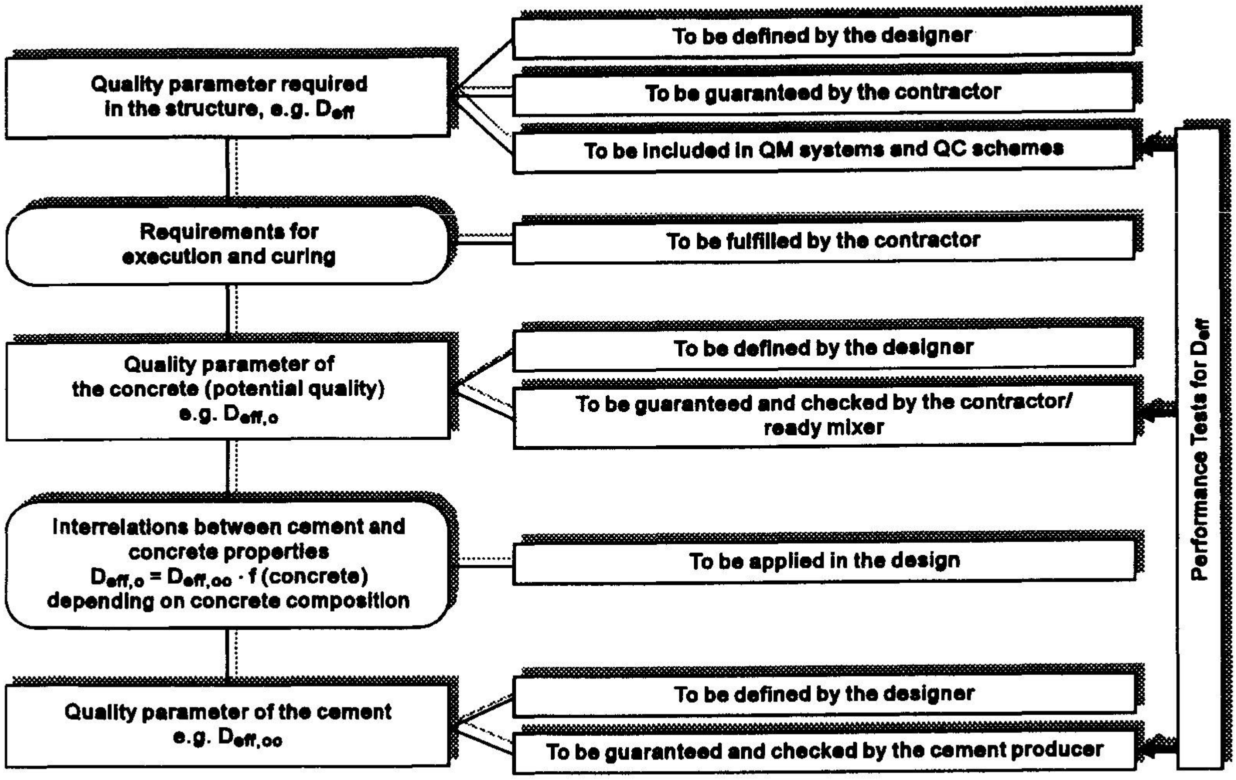


Fig. 5 Performance based durability design at the meso-level

Coming back to Fig. 3, the amount of quality control to check the real quality gained in the structure depends on the criticality of the structure, i.e. the consequences of durability failures and/or the possibilities to repair them.

3.2 Corrosion Monitoring Systems

3.2.1 Location of the Sensors

As mentioned above, monitoring of the structure, e.g. by installing sensors, is an integral part of an overall durability design approach, especically for structures exposed to very severe environments. Having analized the structure with respect to the severity of the micro-environments and sensitive areas (e.g. construction joints, areas of possible salt and water accumulations etc.) it is normally easy to define the most critical areas where sensors should be installed (see Fig. 4).

3.2.2 Corrosion Sensor

The development of the corrosion sensor was based on an extensive research program on the main factors influencing chloride induced macrocell corrosion of steel in concrete [4...8]. These investigations have been carried out using macrocell current measurements between anodically and cathodically acting steel surface areas. It was shown that the corrosion rate of the reinforcement can easily be monitored continuously by these electrical current measurements.

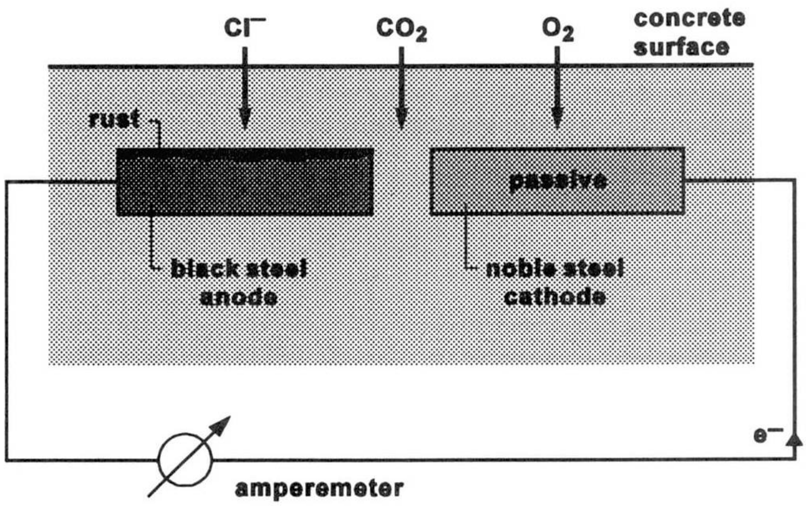


Fig. 6 Macrocell consisting of a black steel anode and noble metal cathode

The operation of a macrocell consisting of a piece of black steel (anode) and a noble metal (cathode) is shown in Fig. 6. In chloride free and non-carbonated concrete, both electrodes are protected against corrosion due to the alkalinity of the pore solution of the concrete (passive state). The electrical current between both electrodes is negligibly low under such conditions. If, however, a critical chloride content is reached, or if the pH-value of the concrete decreases due to carbonation, the steel surface of the anode is no longer protected against corrosion. Provided that the cathode material is corrosion resistant in chloride contaminated or carbonated concrete (e.g. stainless steel, platinum), and sufficient moisture and oxygen are available, oxygen reduction takes place at the surface of the cathode. The local separation of anodically and cathodically acting areas leads to an electron flow between the black steel and the cathode, which can easily be measured at the external cable connection.

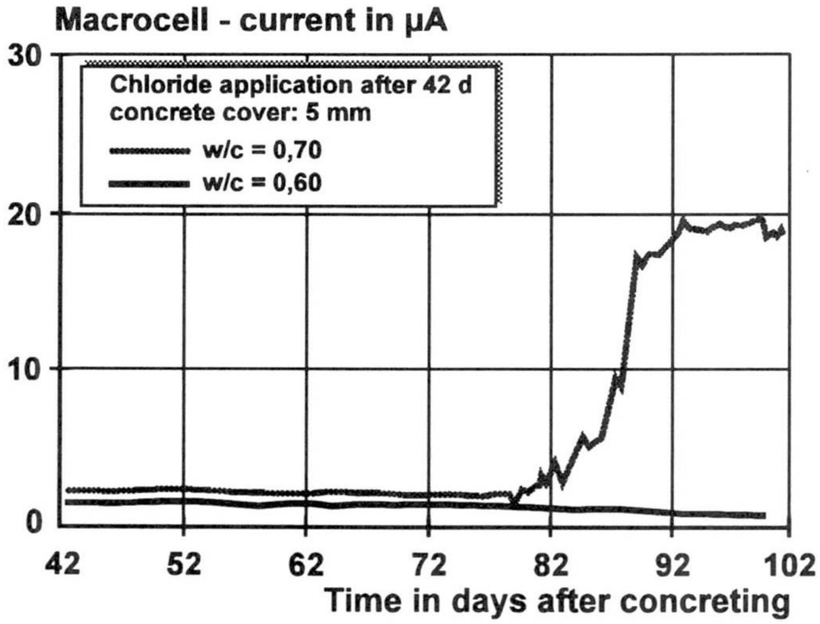


Fig. 7 Time-dependent behaviour of the electrical currents between anode and cathode

Fig. 7 shows the result of electrical current measurements (demonstration test in the laboratory) between a black steel anode and a stainless steel cathode in two concretes with different water-cement ratios. The macrocells were embedded with a concrete cover of only 5 mm (0.20 in.) to initiate corrosion by applying a chloride solution on the concrete surface within a short period of time. The results of the macrocell current measurements showed that the critical chloride content reached a depth of 5 mm (0.20 in.) at the specimen with w/c = 0.7 about 80 days after concrete placement. This caused a significant increase of the macrocell current while the specimen with a lower w/c and a higher resistance against chloride diffusion remained passive.

To monitor the corrosion risk for the reinforcement depending on its distance from the concrete surface, several anodes can be placed in the actual concrete structure at defined cover depths. The cathodes should be positioned at locations that are near the anodes and that are not water saturated because oxygen is needed at the cathodically acting metal surface.

Fig. 8 shows exemplarily a possible arrangement of anodes and a locally separated cathode.

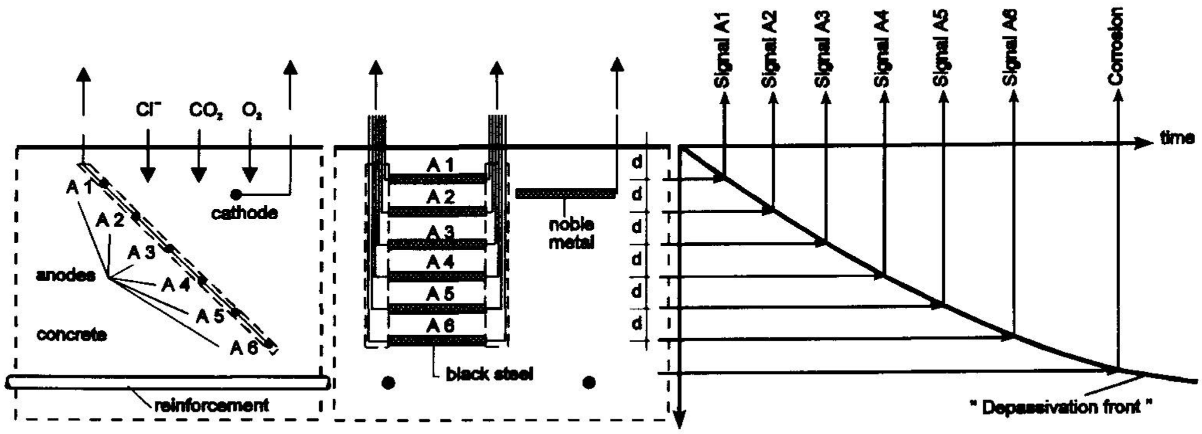


Fig. 8 Arrangement of the anodes and cathodes to monitor the corrosion risk for the reinforcement

Normally all the electrodes are disconnected from each other to prevent electrochemical interactions between them. Only during the electrical current measurements the anodes are coupled with the cathode one after the other.

As long as the critical chloride content and the carbonation front have not reached the surface of the first anode A_1, all the electrical currents are negligibly small. As soon as the steel surface of the first anode A_1 is depassivated due to the action of chlorides or carbonation the electrical current between A_1 and the cathode increases significantly whereas the currents of the other electrodes remain zero.

In the course of time the other anodes will also be depassivated one after the other. By measuring the electrical currents continuously or in regular intervals the relationship between the depth of the critical chloride content or carbonation and time can be determined. Having this information, the time to corrosion of the reinforcement can be estimated by extrapolation using appropriate calculation models.

A typical layout and arrangement of a corrosion sensor consisting of 6 single anodes is shown in Fig. 9. Each of the 6 black steel anodes is positioned 50 mm (2 in) from the next one to prevent interactions between the anodes.

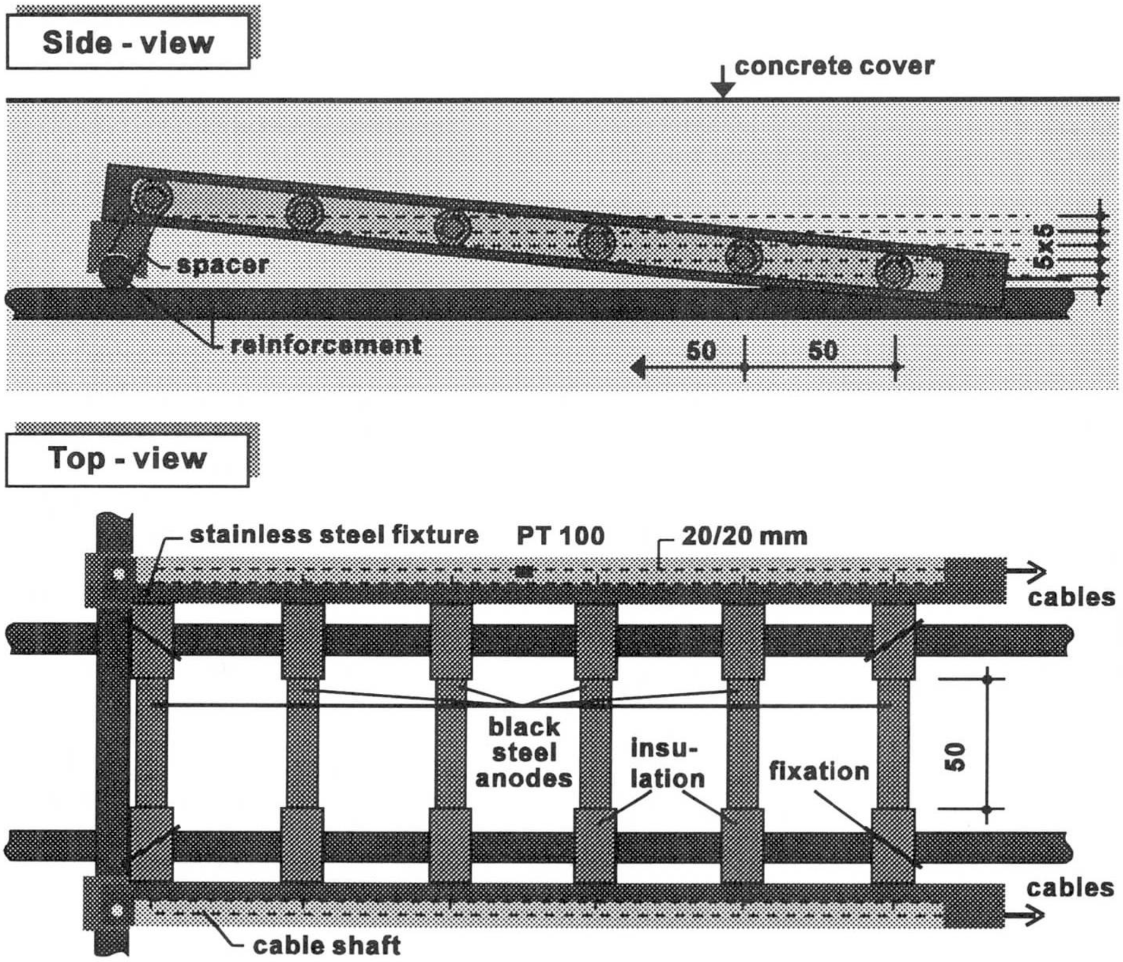


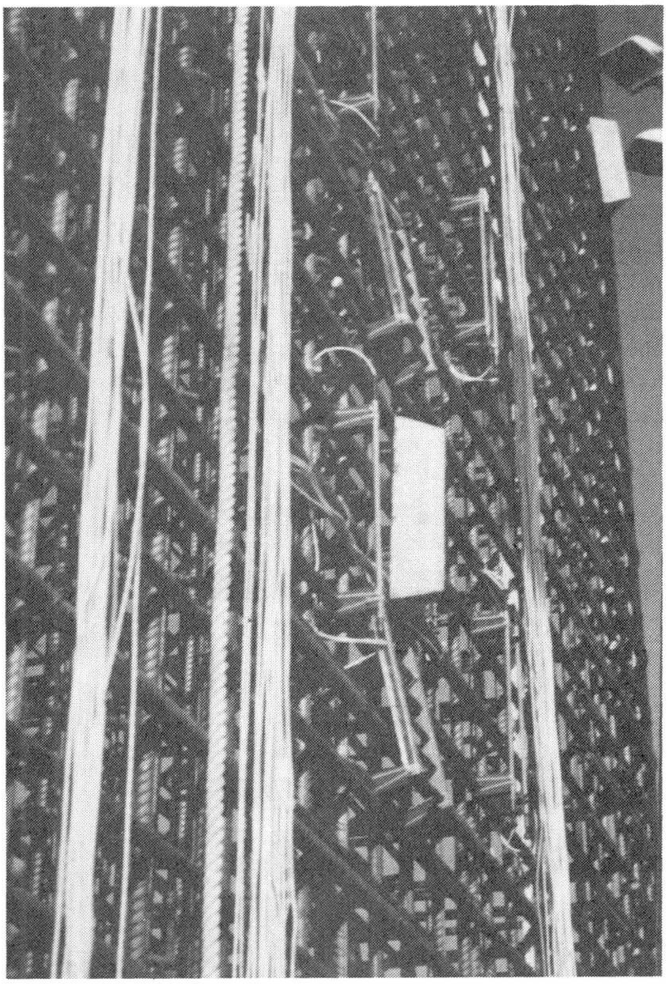
Fig. 9 Design of a set of 6 anodes

Fig. 9 also shows that the cables are lead through a stainless steel fixture to the measuring device. The cable shaft is filled with epoxy resin that acts as mechanical protection for the wires and a PT 100 temperature sensor that can be additionally installed. Two wires are connected with each single anode to get a redundant system. This allows the cables and cable connections to be checked after installation by resistance measurements.

The fixture is separated from the anodes by an insulation. This geometrical design of the sensor ensures that the concrete cover above every single anode is not affected by parts of the sensor, and that the penetration of chlorides into the concrete and the carbonation process are also not influenced.

The layout of the sensor system allows, besides the current readings other measurements improving the information on the overall corrosion risk within the monitored structure:

- The measurement of the potential between the anodes or the reinforcement and the noble cathode gives a further information on the onset of corrosion.
- The simultaneous measurement of the temperature by means of an incorporated temperature sensor allows a more detailed interpretation of the current readings.
- The anodes can be used as measuring electrodes for AC resistance measurements at different distances from the concrete cover. This type of readings for example can be used to monitor the efficiency of coatings, preventing water ingress into the concrete.



Altogether about 500 sets of sensors have been installed so far into reinforced and prestressed concrete structures

- 8 sets of sensors for Bridge Schießbergstraße near Cologne,
- 204 sets of sensors for the tunnel of the Great Belt Link,
- 180 sets of sensors for the pier shafts and prestressed girders of the Western Bridges of the Great Belt Link,
- 42 sensors for the anchor blocks of the Eastern Bridge of the Great Belt Link,
- 6 sensors for the Nötsch Bridge in Austria,
- 20 sensors for a bridge in Egypt.

All installations have been done successfully without complications. No depassivation of sensors has happened so far.

Fig. 10 Installed sensors (6 sets of anodes and 1 cathode - Pier shaft of the Western Bridge, Great Belt Link

3.2.3 Multi-Ring-Electrodes

Apart from other measures, one possible means of repairing damage at the concrete surface due to reinforcement corrosion is to apply a coating to the concrete surface to reduce the water content of the concrete. If the coating has a sufficient high penetration resistance against water and if no water enters the concrete from other sources, the water content of the concrete will remain low after the application of the coating, or the concrete will dry out slowly, provided that water can evaporate through the coating or through the opposite concrete surface.

In this way the corrosion rate of the reinforcement can be decreased significantly because the electrolytic conductivity of the concrete is reduced considerably. If the water content of the concrete is below a limit value, the corrosion rate of the reinforcement will be negligibly low. This limit value for the water content is not a fixed value but depending on different parameters, e.g. the concrete composition, chloride content and carbonation depth.

A multi-ring-electrode method for determining the water distribution within the concrete cover (between the reinforcement and the steel surface) has been developed at the Institute for Building Materials Research in Aachen, Germany. The electrode is used to determine AC resistance between nine noble metal rings, allowing the water content to be estimated at eight different distances from the concrete surface (see Fig. 11).

To estimate the effect of different types of coatings, time-dependent changes in resistance following wetting and drying of coated and uncoated surfaces were monitored. Fundamental laboratory investigations on the influence of concrete compositions, carbonation and chloride application have been carried out.

In addition to the above mentioned investigations multi-ring-electrodes have been successfully installed into existing structures. For this purpose the multi-ring-electrode will be fitted in a drilled hole and subsequently coupled to the old concrete. In laboratory tests coupling of the multi-ring-electrodes to hardened concrete has been performed on existing test specimens, allowing to compare directly the results of sensors installed into new structures with the results of sensors installed into existing structures. The tests show that the installation into existing structures provides reliable data on the moisture distribution within the concrete cover.

More informations about test results with this equipment can be found in [9].

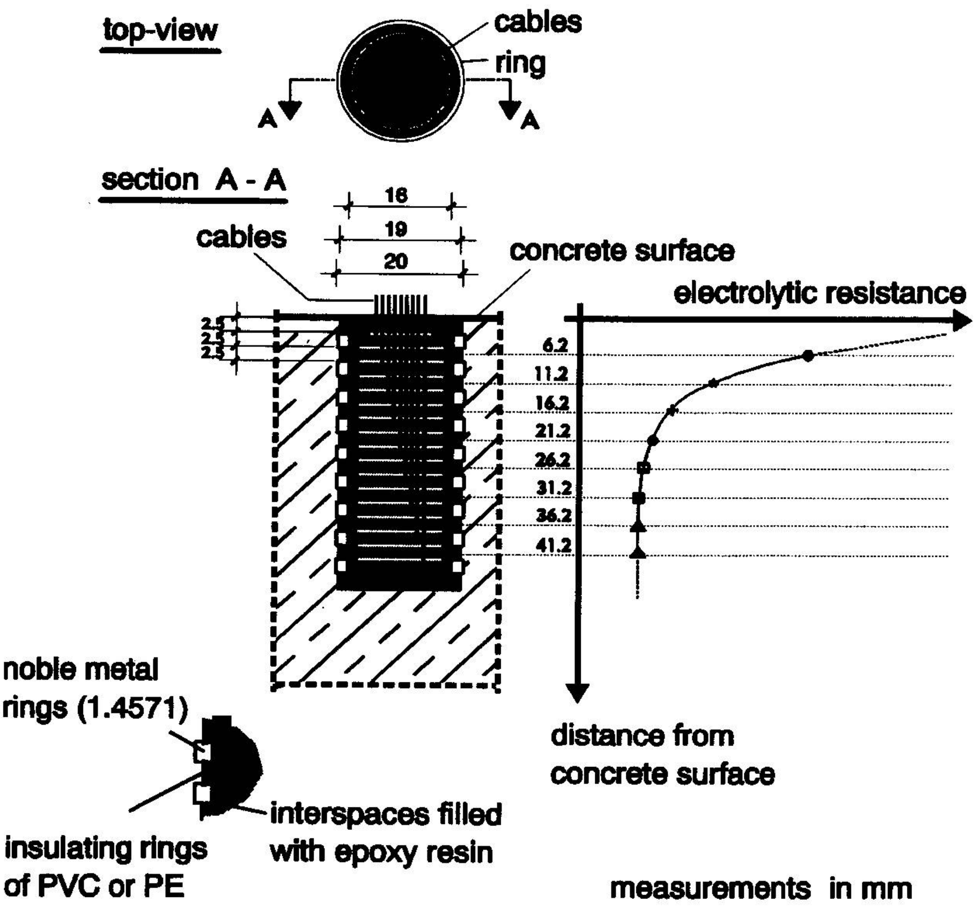


Fig. 11 Schematic representation of the multi-ring-electrode and a qualitative diagram of measured values

4 REPAIR STRATEGIES TO REPAIR CONCRETE STRUCTURES DAMAGED BY STEEL CORROSION - THE NEW RILEM APPROACH

The RILEM Technical Committee 124-SRC has prepared a RILEM Technical Recommendation on „Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion", published in [10].

The repair strategies and principles presented in this RILEM Recommendation are related to the design of corrosion protection of ordinary reinforcement.

The Recommendation describes the basic possibilities, strategies and methods for improving or restoring the corrosion protection of ordinary reinforcement. It is written for design engineers and consultants and describes basic strategies and requirements. As it is intended to give a basis for decision making, it does not contain detailed requirements for the materials to be used or guidelines for the execution of the repair work.

The strategies described apply to the majority of normal repair situations.

The format of this RILEM Technical Recommendation is sketched in Fig. 12. Strategy Level 1 deals with various possibilities of interventions, one of them being an intervention into the corrosion process. This strategy is treated in detail within Strategy Level 2 (chapter 4 of [10]). Based on the aim to avoid further corrosion after repair, different Principles of Repair are defined. Based on these principles possible Methods of Corrosion Protection are elaborated including the basic requirements needing to be fulfilled to ensure a successful repair. The presented options for repair are based on the existing knowledge at the time of drafting the document and are related to the possible deterioration processes prior to and after repair works.

The principles of repair are based on the electrochemical processes causing corrosion (see chapter 2.2).

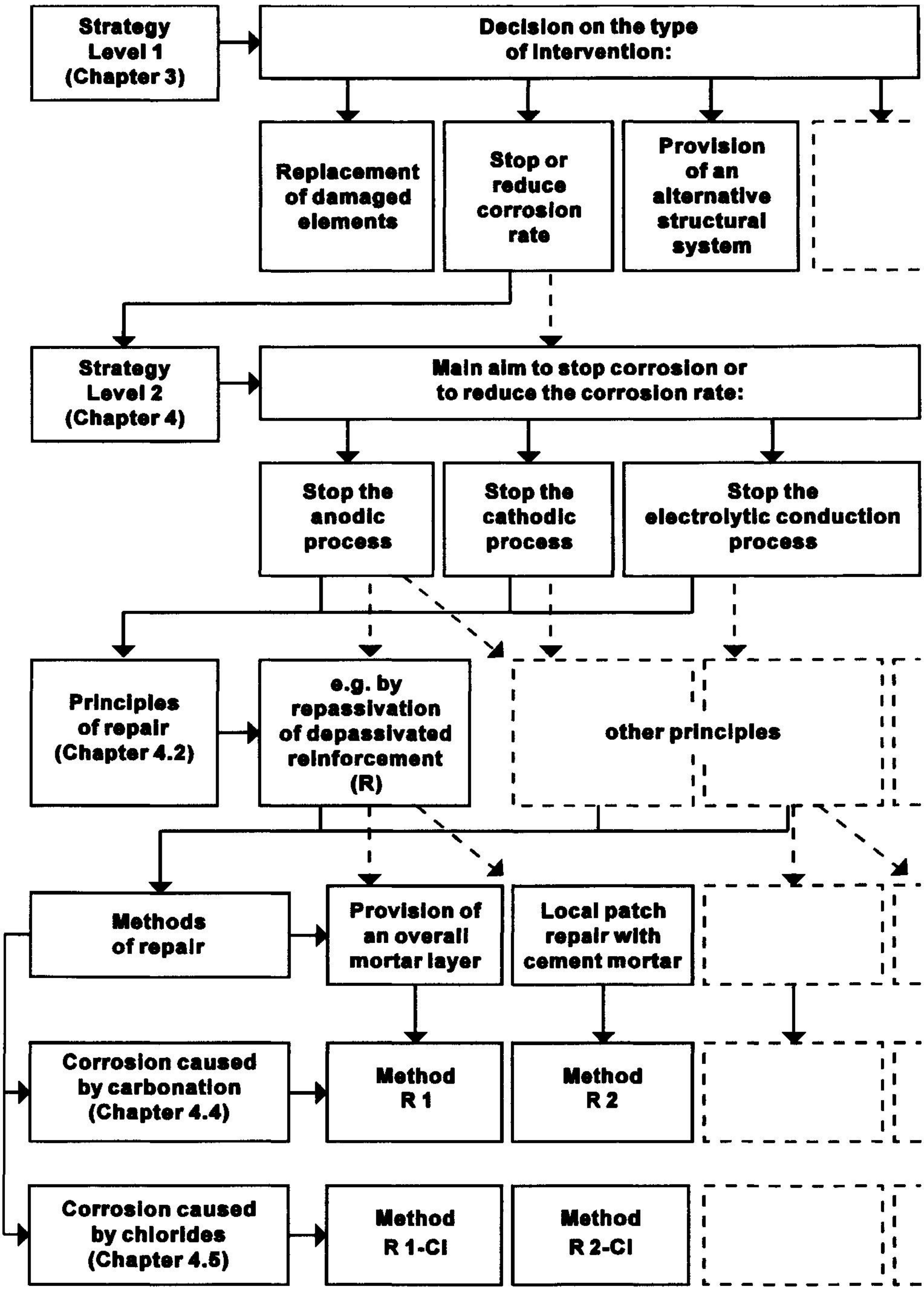


Fig. 12 Format of the Technical Recommendation

The basic aim for repair should be

- to stop the anodic process,
- to stop the cathodic process, or
- to stop the electrolytic conduction process.

In order to achieve one of these effects, different repair principles, presented schematically in Fig. 13, are possible. The repair principles to restore corrosion protection need to be designed on the basis of the electrochemical corrosion processes at the steel surface and the chemical and physical processes within the surrounding concrete.

Fig. 13 presents basic principles and examples of techniques based on these principles. There are no generally proven repair procedures to stop the cathodic process, e.g., by the total block of oxygen access to the surface of the reinforcement. Although theoretically possible, the concept of stopping the cathodic process therefore is not included in Fig. 13.

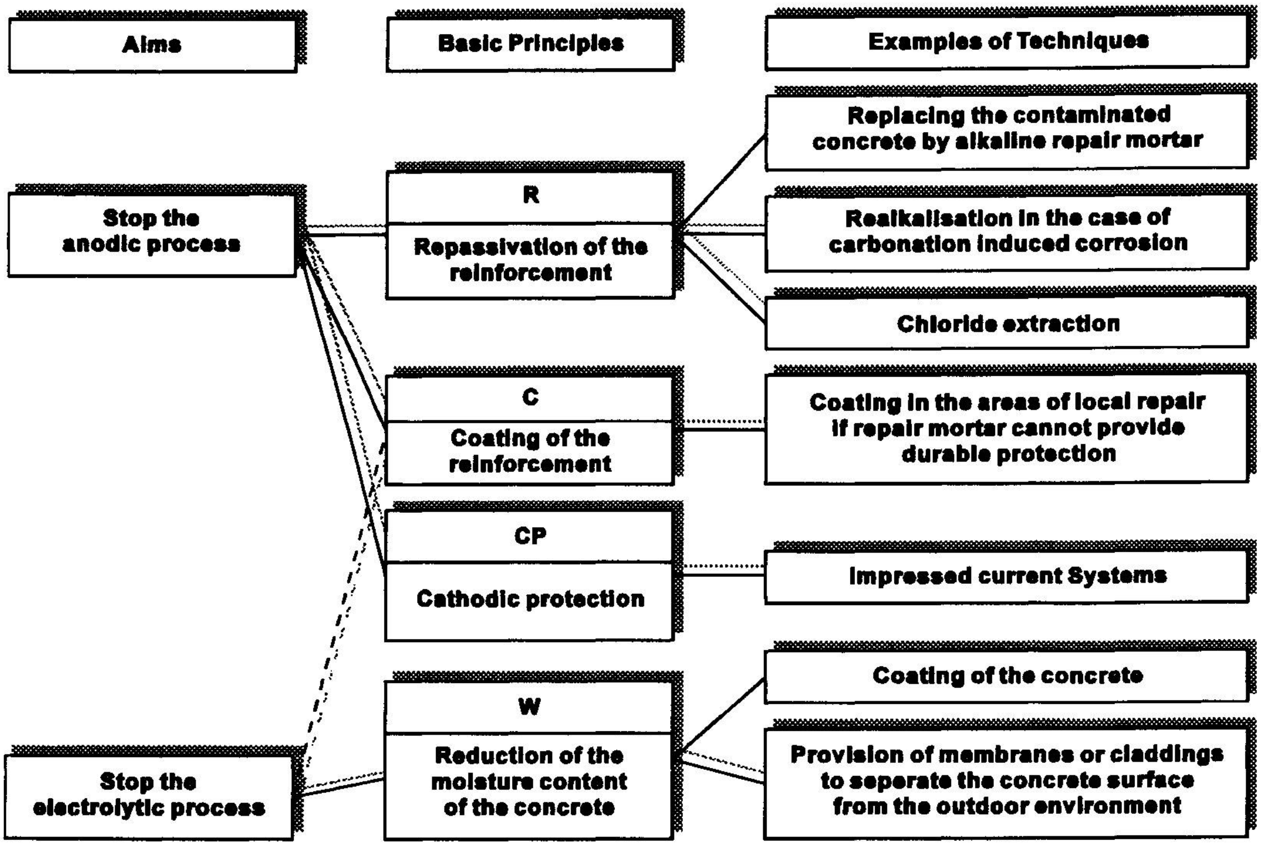
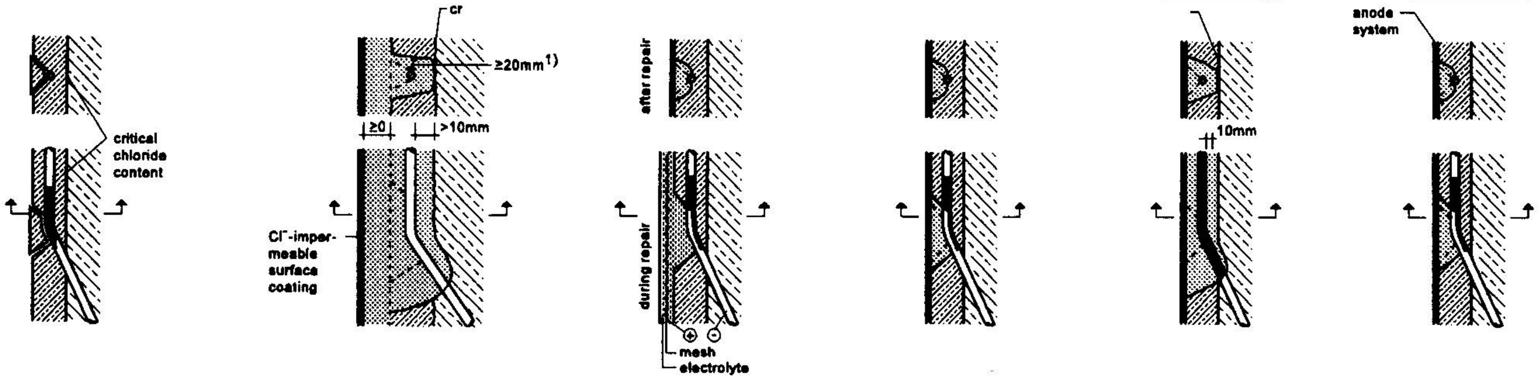


Fig. 13 Principles of repair to stop corrosion

For the different basic principles sketched in Fig. 13 different methods for repair must be elaborated, depending on the cause of depassivation (i.e., carbonation or chlorides).

As an example the basic methods for the repair of chloride induced corrosion, as presented in [10] are shown in Fig. 14.

Type of damage		R1 Repassivation Alkalinic repair mortar/concrete	R2 Repassivation Electrochemical Cl^--removal (see section 4.7.3)	W Limitation of moisture content surface coating	C Coating of the reinforcement	CP Cathodic Protection



on

		R1	R2	W	C	CP
concrete removal			X	X		X
		X			X	
surface treatment of steel	SA 2 1/2 + coating	coating not allowed	coating not allowed	not necessary	X	coating not allowed
	removal of rust	X including rust in pits	X including rust in pits	X including rust in pits		X
repair mortar/ concrete	alkaline, with buffer	X	X			conductive X
	no alkalinity, required			X	X	
surface treatment of concrete	not required					X
	required (Cl^--impermeable)	X	X	X	X	

Fig. 14 Basic methods for the repair of chloride induced corrosion

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Maintaining and Extending the Lifespan of Steel Bridges in Japan

Maintenance et réhabilitation des ponts métalliques au Japon

Wartungsarbeiten und lebensverlängernde Massnahmen an Stahlbrücken in Japan

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SUMMARY

A review is made of fatigue problems related to maintaining and extending the lifespan of steel bridges in Japan, mainly on the Tokaido Shinkansen bullet train system opened in 1963, Tomei Expressway opened in 1969 and the Honshu-Shikoku bridge project which is under construction.

RÉSUMÉ

Le rapport traite des problèmes de fatigue concernant la maintenance et la durée de vie des ponts métalliques au Japon, en particulier sur la ligne ferroviaire à grande vitesse Shinkansen Tokaido inaugurée en 1963, sur l'autoroute Tomei ouverte au trafic en 1969 et pour le projet des ponts de Honshu-Shikoku en cours de construction.

ZUSAMMENFASSUNG

Diese Arbeit ist ein Bericht über Ermüdungserscheinungen in Bezug auf Wartungsarbeiten und lebensverlängernde Massnahmen an Stahlbrücken in Japan. Es werden dabei Schwerpunkte gelegt, nämlich auf das Tokaido Shinkansen Eisenbahnsystem (1963 eröffnet), die Tomei Autobahn (1969 eröffnet) und die Honshu-Shikoku Brücke, die zur Zeit im Bau ist.

1. INTRODUCTION

A review is made here centered on the problems attributable especially to fatigue, in regard to maintenance of steel bridges in Japan mainly on the Tokaido Shinkansen bullet train system, Tomei Expressway, and Honshu-Shikoku bridge project. The Tokaido Shinkansen and Tomei Expressway were constructed as the forerunners of high-speed railways and automobile expressways of Japan and can be considered as infrastructures which had brought about the high-level economic growth of Japan. It may be said that the problems entailed by steel bridges in Japan are more or less covered by taking up these two. The Honshu-Shikoku bridge project, which make up a large-scale bridge project with construction now in progress, include the world longest suspension bridge, Akashikaikyo Bridge, and cable-stayed bridge, Tatara Bridge, in the world. The newest bridge technology of Japan is concentrated in this project.

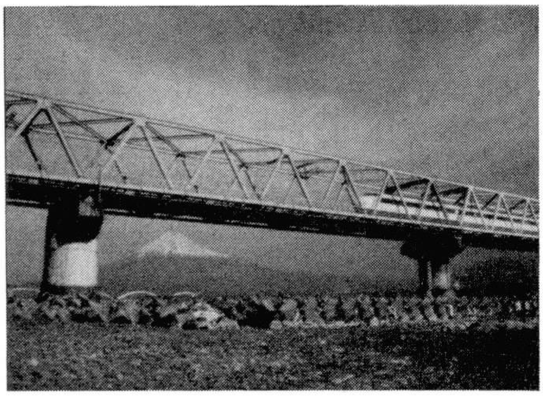
2. BRIDGES OF TOKAIDO SHINKANSEN

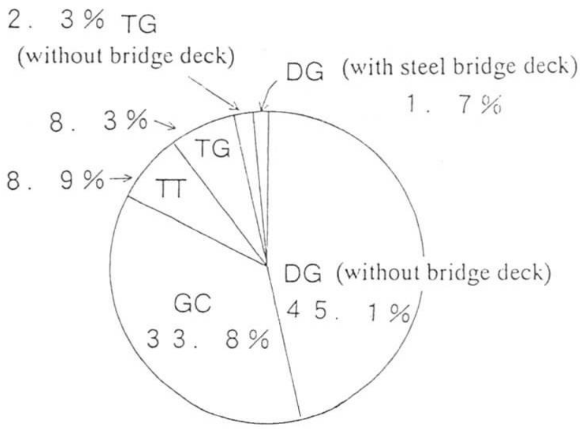
2.1 Description of bridges

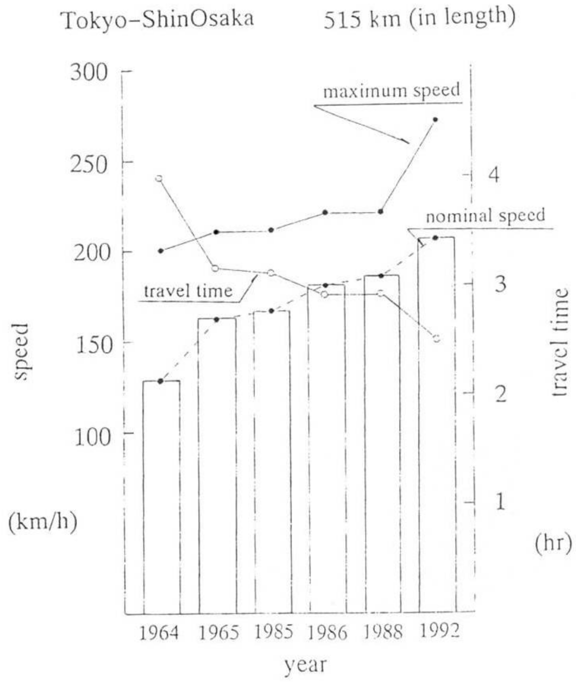
The outline of steel bridges in Tokaido Shinkansen are given in Table 1. Fig.1 shows the change of train speeds, travel time and the number of trains in one day from the opening to the present state. These have all changed in the direction of severity for structures. Only, the weight of train changes for the lighter.

Table 1. Outline of steel bridges in Tokaido Shinkansen

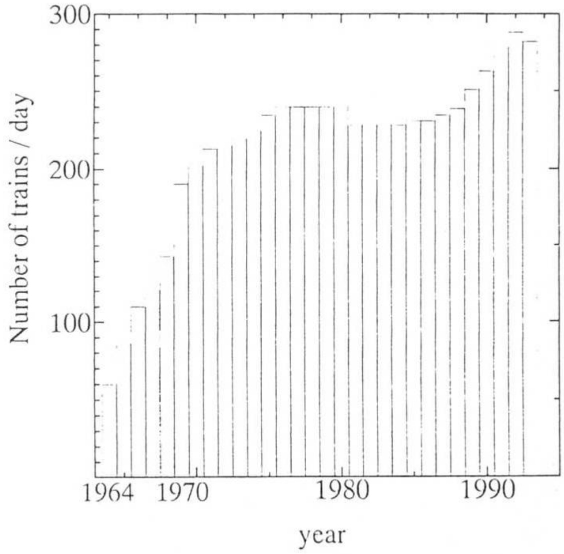
Type of Steel Bridge	Ratio	Number of girders	Total Length
Deck Plate Girder (without bridge deck)	45.1%	682	12.17
Composite Girder	33.8%	510	9.12
Through Truss	8.9%	135	2.40
Through Plate Girder (with steel bridge deck)	8.3%	125	2.24
Through Plate Girder (without bridge deck)	2.3%	34	0.62
Deck Plate Girder (with steel bridge deck)	1.7%	25	0.46
Total	100%	1511	27Km





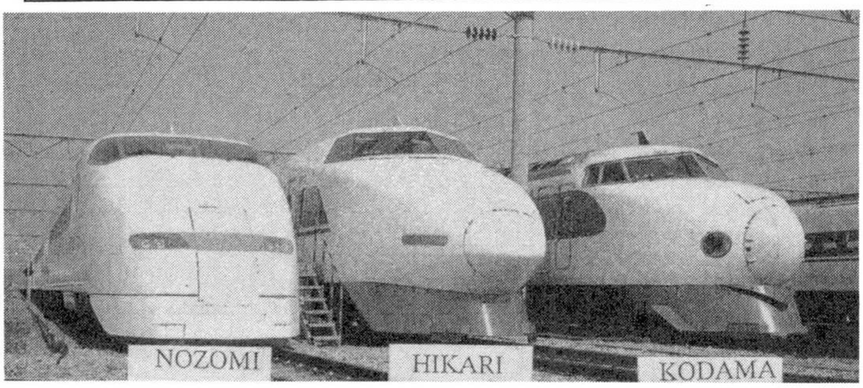


(a) speed and travel time



(b) number of trains

Name	Car Series	Weight(t/train(16cars))	Capacity	Max Speed(km/h)
KODAMA	0	970	1391	220
HIKARI	100	925 ▲45	1321	220
NOZOMI	300	710 ▲215	1323	270



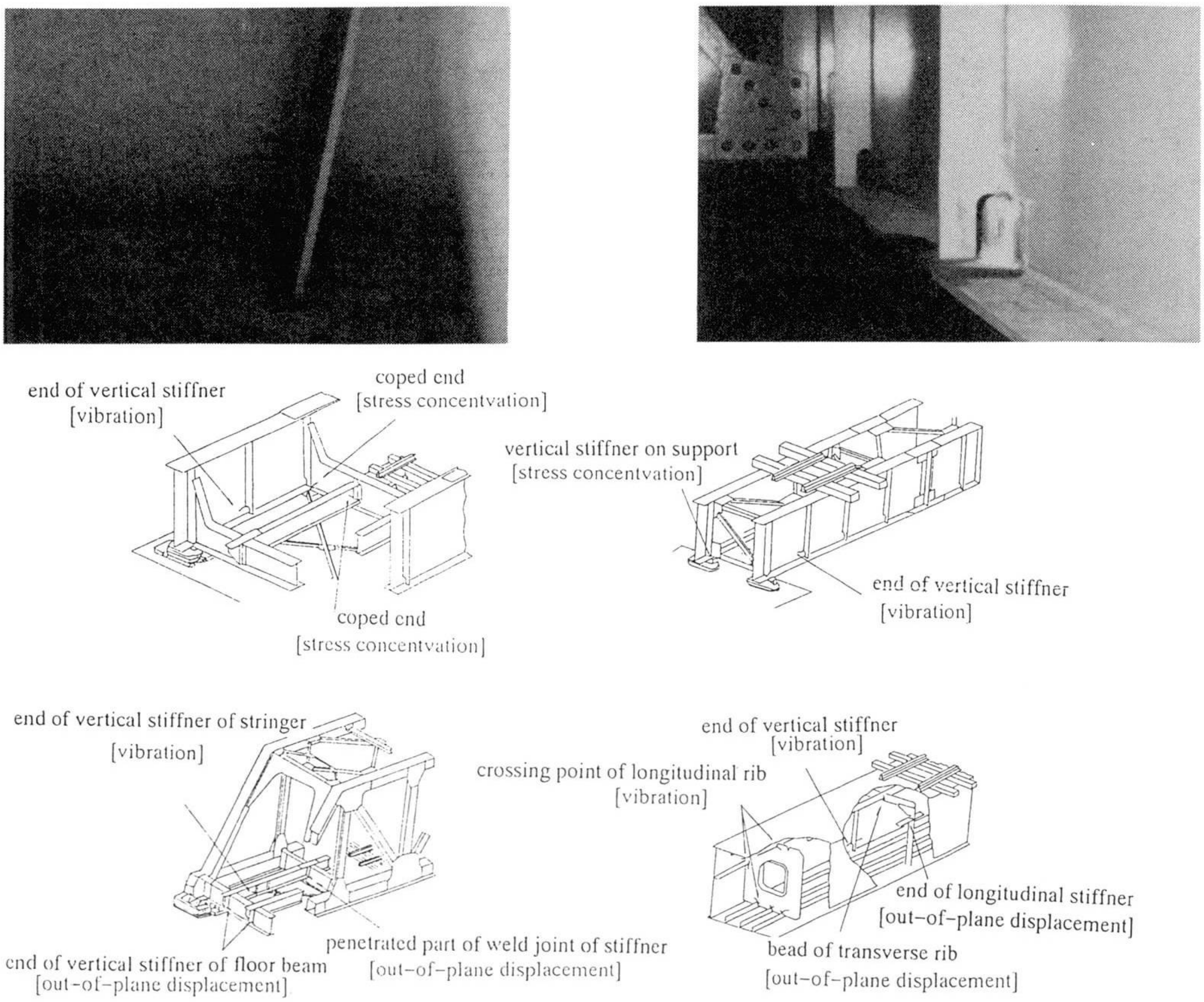
(c) train cars

Fig.1 Changes of service conditions

Fatigue design of steel bridges had done based on design specifications established in 1960. These specifications were strongly influenced by DIN, but especially with regard to welded joints, the results of fatigue tests which were carried out in Japan were taken up in the establishment. The fatigue design standards had set up allowable fatigue stresses based on fatigue strengths at two million cycles which were considered as fatigue limits, but considerations such as to take design fatigue train load as 18 tons against design train load of 16 tons with the aim of reflecting difference in influence line length on number of design stress repetitions are given.

2.2 History of fatigue cracking

Occurrences of fatigue damage began to be reported concerning bridges of the Tokaido Shinkansen from around 10 years after having been put into service. An outline is given in Fig. 2, with cases of damage being those resulting from stress concentrations due to structural details of members, those due to out of plane displacements occurring between perpendicularly crossing members such as main girders and cross beams or cross beams and stringers, and those due to vibration induced distortion by high-speed operation of trains peculiar to Shinkansen[1].



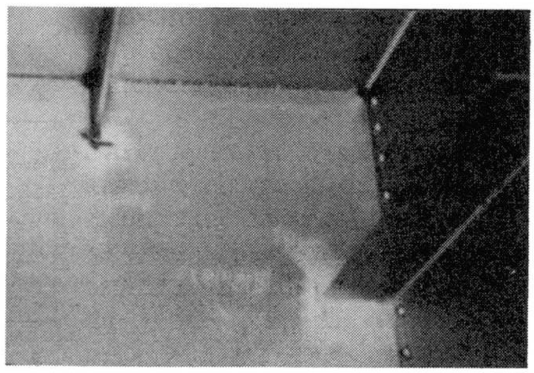


Fig.2 Fatigue damage in steel bridges of Tokaido Shinkansen

2.3 Maintenance inspection

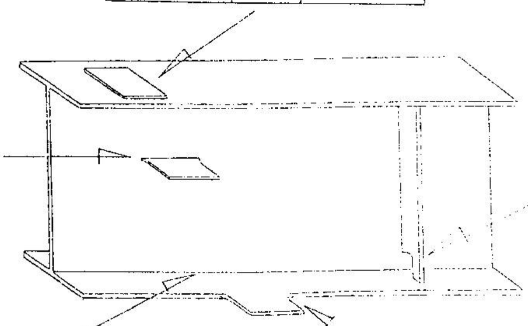
Inspections of Shinkansen consist of periodic regular inspections have been arrived out every two years and individual special investigations have been formed when any damage is found during a periodic regular inspection. Inspections of structures are made by engineers of Central Japan Railway Co. which manages and operates this line. These engineers have thorough knowledge about fatigue and corrosion which occur in bridge structures, and possess the capability of carrying out stress measurements, nondestructive test, etc. They also periodically undergo training in skills concerning these technologies.
Selection of contents of special individual special investigations and evaluations are made by specialists consisting of engineers of the Railway Technical Research Institute and university professors, including the author. The necessary retrofitting measures are discussed by the above mentioned group, upon which recommendations are made to Central Japan Railway Co..

2.4 Damage prevention works

When what had been assumed at the time of designing and the subsequent condition of use are considered, certain parts in bridge structures are approaching the ends of their design service lives. Therefore, a number of preventive and protective set-ups against fatigue damage have been established. What have great differences in allowable fatigue strengths between the time when Shinkansen was designed and the current design standards are longitudinal weld between flanges and webs, and ends of gusset plates welded to webs (Fig.3). They are the points in damage prevention work.

Joint	Sleeper pad	
Code	Class	2×10^6 Fatigue Strength
Old code '60	$\bar{D}$	120MPa
Current code '92	F	65MPa

Joint	Out-planegusset(r=20mm)	
Code	Class	2×10^6 Fatigue Strength
Old code '60	D	100MPa
Current code '92	F	65MPa



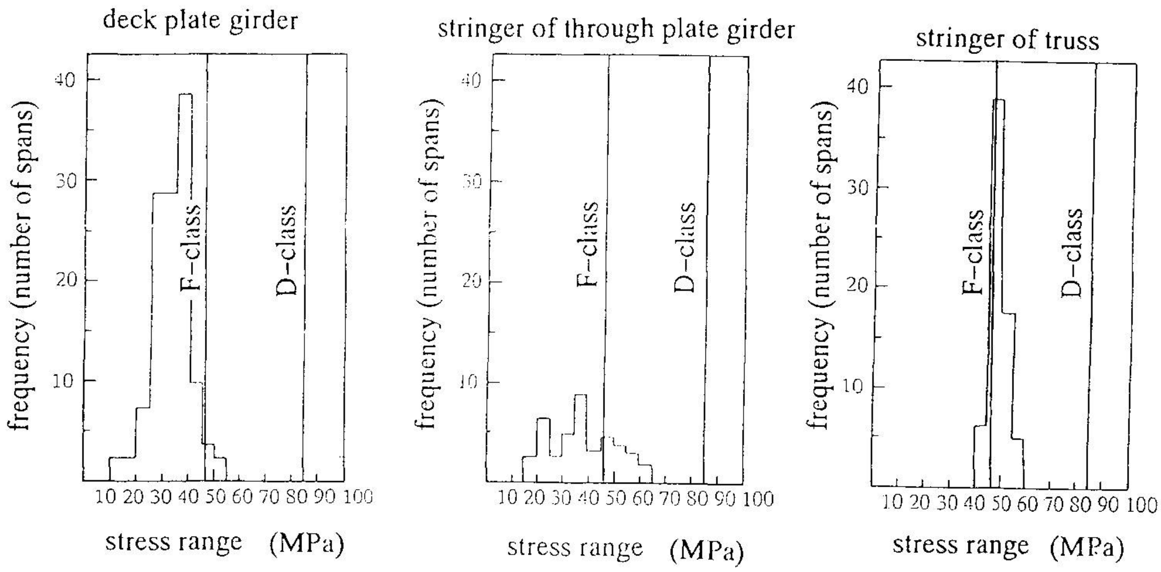
Joint	Cruciform weld	
Code	Class	2×10^6 Fatigue Strength
Old code '60	D	100MPa
Current code '92	E	80MPa

Joint	Longitudinal weld	
Code	Class	2×10^6 Fatigue Strength
Old code '60	B	150MPa
Current code '92	C	125MPa

Joint	In-plane gusset(r=20mm)	
Code	Class	2×10^6 Fatigue Strength
Old code '60	C	126MPa
Current code '92	F	65MPa

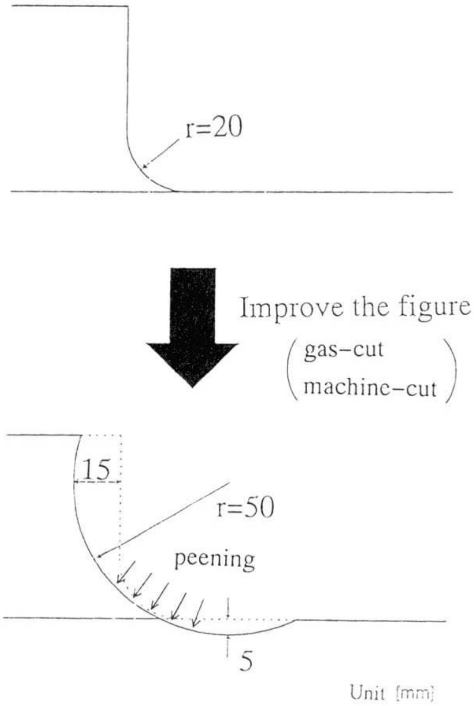
Fig.3 Changes of allowable fatigue stress from '60 code to '92 code

One of damage prevention works is measurement of actual stresses. This is done regarding plate girders and the stringers of truss girders. To elaborate, since stresses in bridge members actually occurring are normally low compared with those in design calculations, it is thought to consider maintenance based on stresses actually occurring. Stress measurements for this purpose consist of grasping the bending stress on average occurring in the bottom flange and the location of the neutral axis. The reason for actual stresses being low compared with calculated stress in case of railway bridges, lies in the load distribution due to rails and secondary members. The results of stress measurements are illustrated in Fig. 4, in which it is clearly shown that the possibility of fatigue damage in a fairly large number of members is high on the basis of design calculated stress, however, there is hardly any cause for concern when actual stress is observed. Regarding longitudinal welds joining flanges and web (categorized into C class), it was learned that there was ample allowance from measurements of actual stresses, but the degrees of allowance at the ends of gussets attached to flanges (categorized into F class) were slightly small. Hence, remedial measures regarding fatigue strength are presently under study for this gusset detail. Methods being considered are to increase the radius of the fillet at the end of the gusset and, further, to add residual compressive stress by peening, and the effectiveness are being examined through fatigue tests (Fig.5).



Fatigue category	longitudinal joint	: C
	vertical stiffener end	: E
	flange gusset detail	: F

Fig.4 Variation of measured maximum stress range and fatigue assessments



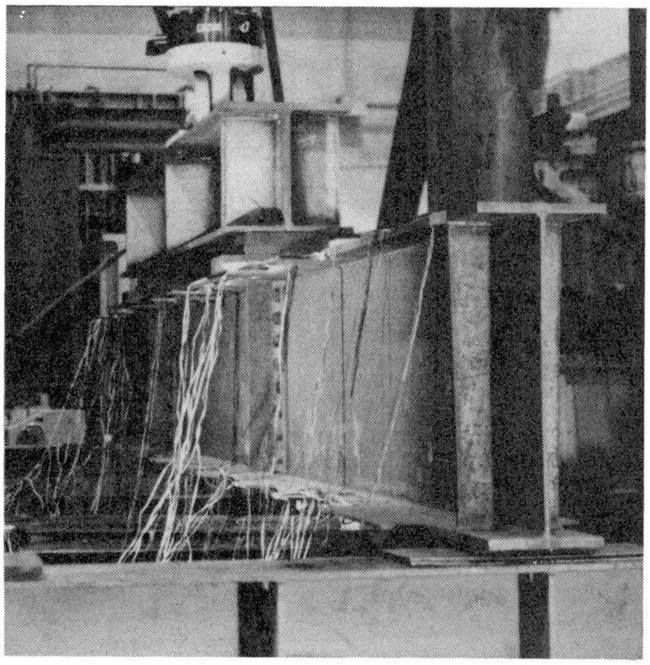


Fig.5 Improving work of flange gusset detail and fatigue test

3. TOMEI EXPRESSWAY

3.1 Description and history of bridges

The numbers highway bridges constructed in Japan by year are shown in Fig.6. It can be seen that construction of infrastructure facilities in Japan has been concentrated in a certain period. As a consequence, an enormous number of bridges will become antiquated at once, and measures to cope with such a situation will be of grave importance.
Tomei Expressway is the first expressway in Japan, which was constructed in 1965-1968. There are 112 plate-girder bridges (composite and non-composite) on the part of the Tomei Expressway under the Tokyo First Operation Bureau of the Japan Highway Public Corporation which operates about 85% of Tomei Expressway. Of these, approximately 58 percent are quadruple-main-girder bridges, 37 percent are triple-main-girder bridges, and the remainder small numbers of quintuple-main-girder and sextuple-main-girder bridges. The thicknesses concrete deck slabs are 180 to 200 mm for triple-main- girder bridges while 170 to 190 mm for quadruple-main-girder bridges. Besides plate girder bridges, there are some truss girder bridges and arch type bridges on Tomei Expressway. In all of the bridges damage began to occur in the reinforced concrete deck slabs four to five years after being put into service, and stringers for supporting deck slabs are being added starting in order from bridges in the poorest condition. The transitions in the states of service of the Tomei Expressway are shown in Fig.7. Traffic volume has increased sevenfold in 30 years. A feature is the high proportion of large vehicles such as trucks and trailers. Vehicle scales are installed in main traffic lanes at Nihondaira on the Tomei Expressway. According to records kept since opening of service, the maximum weights of automobiles passing have been constant with the times (Fig.8).

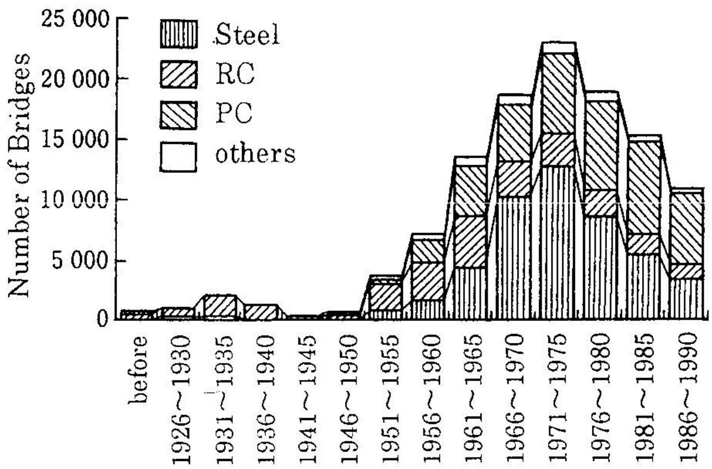


Fig.6 History of bridge construction

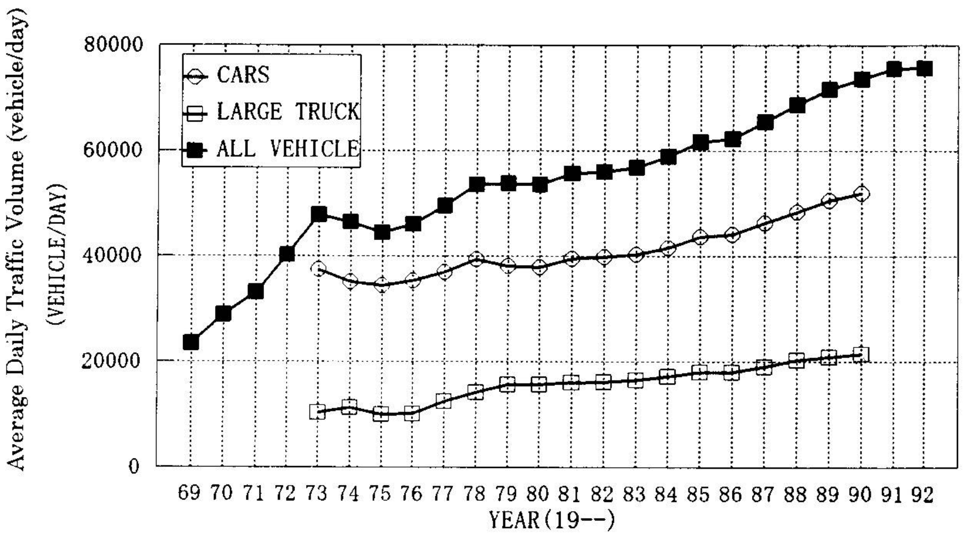


Fig.7 Change of daily traffic volume

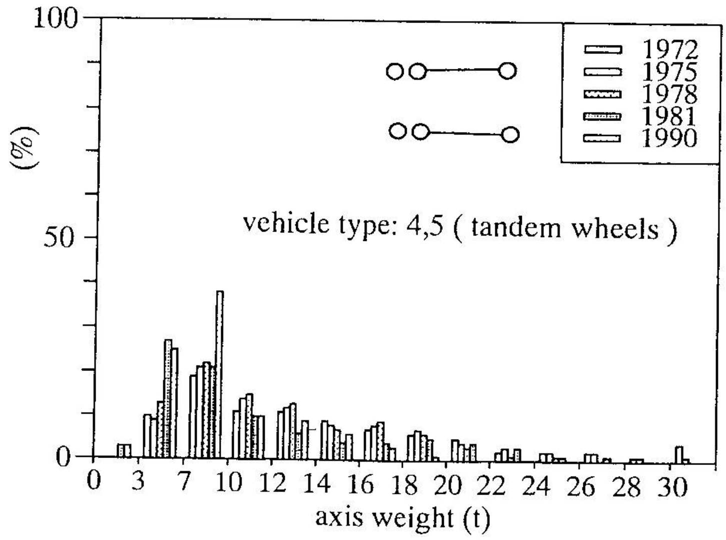
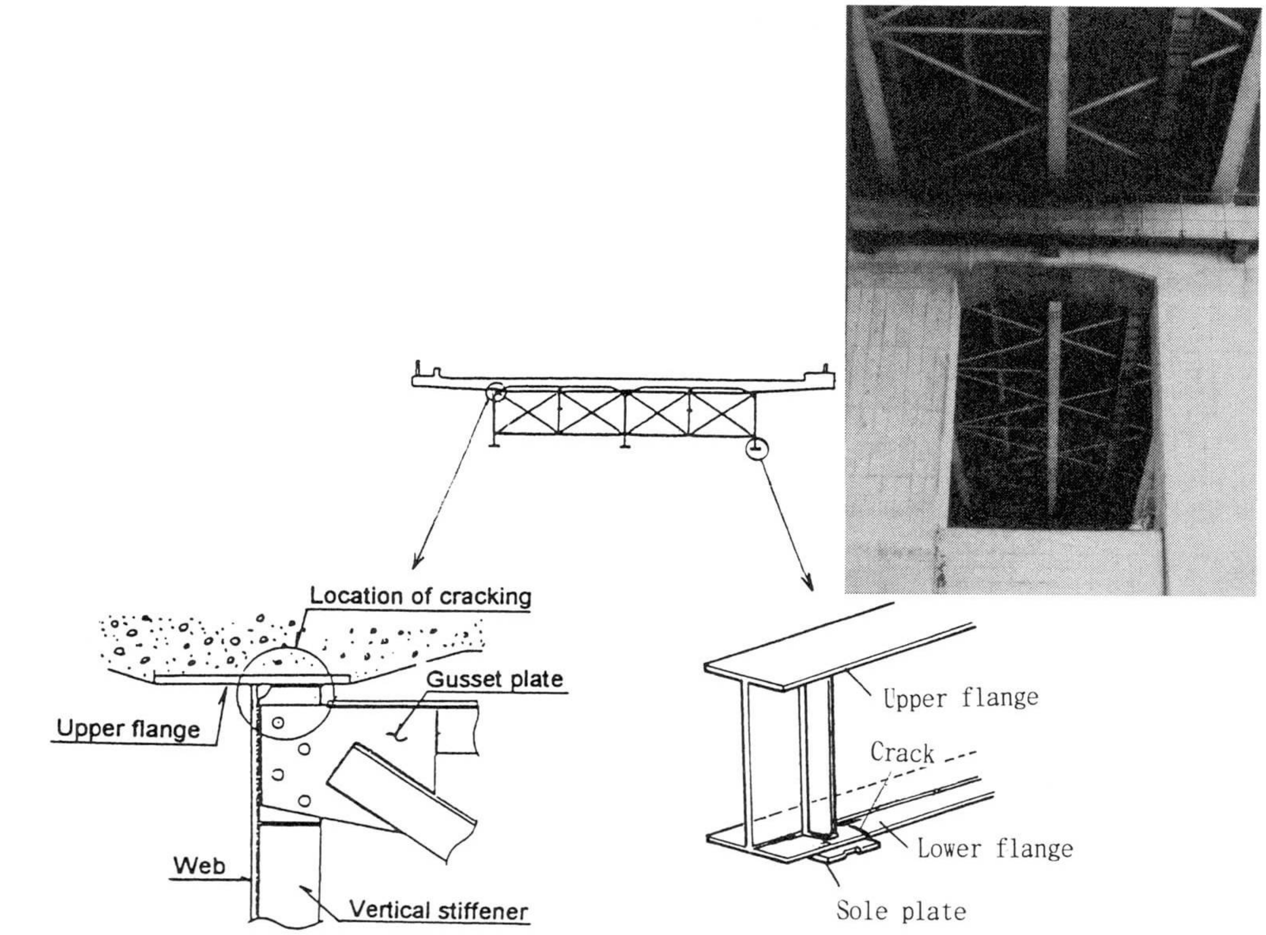


Fig.8 Measured axis weight

3.2 History of cracking and retrofitting works

The types of fatigue damage in steel plate girder bridges of the Tomei Expressway are shown in Fig. 9 [2]. Various types of fatigue damage have been observed. The percentage of occurrence is highest for fatigue cracks at the ends of vertical stiffeners with sway bracing attached. and detailed studies have been made on this fatigue. It has been found to be due to force acting so that displacement will occur orthogonally to the web from differences in deflections between girders and deformations of deck slabs.



sway bracing attached detail

sole plate on bearing

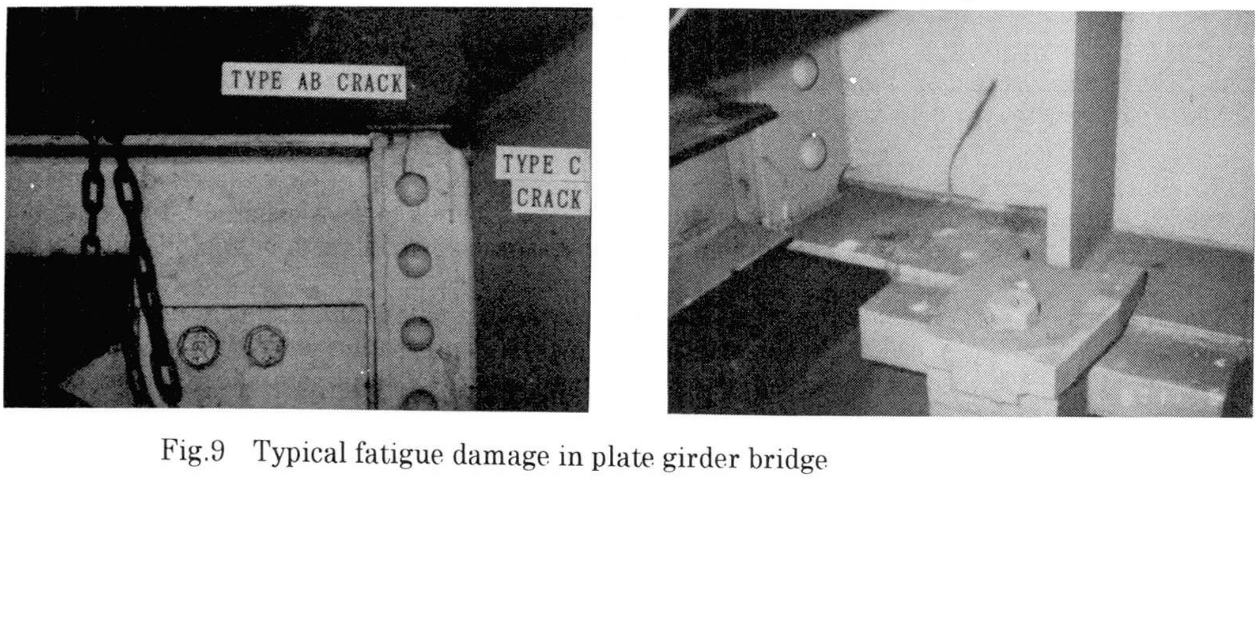


Fig.9 Typical fatigue damage in plate girder bridge

The measures taken against these fatigue cracks are differed according to the levels of cracking (Fig.10). For cracks with lengths at the surface not more than 20 mm, the basic measure applied is to add one pass of welding on top and then remelting with TIG dressing (Fig.11). For fatigue damage longer than that, damaged parts are all removed by gouging, and after rewelding, the surface is simultaneously treated with TIG dressing. The procedure of gouging, rewelding, and TIG dressing is predetermined in details. As a result of inspection approximately 5 years after such a measure had been taken, most parts were sound, but cracks were found again, although few in number, at repaired parts where fatigue cracks had been large.
At bridges with prominent damage, the load applied in the lateral direction by existing sway bracing is to be alleviated by adding sway bracing in improved joint details (Fig.12). The joint details between the sway bracing and main girder at this time was proven by fatigue tests to have been improved. The stresses at fatigue crack occurrence points were reduced approximately 20 percent by providing this measure, while further, stresses were reduced approximately 30 percent by detaching the horizontal members of existing sway bracing.

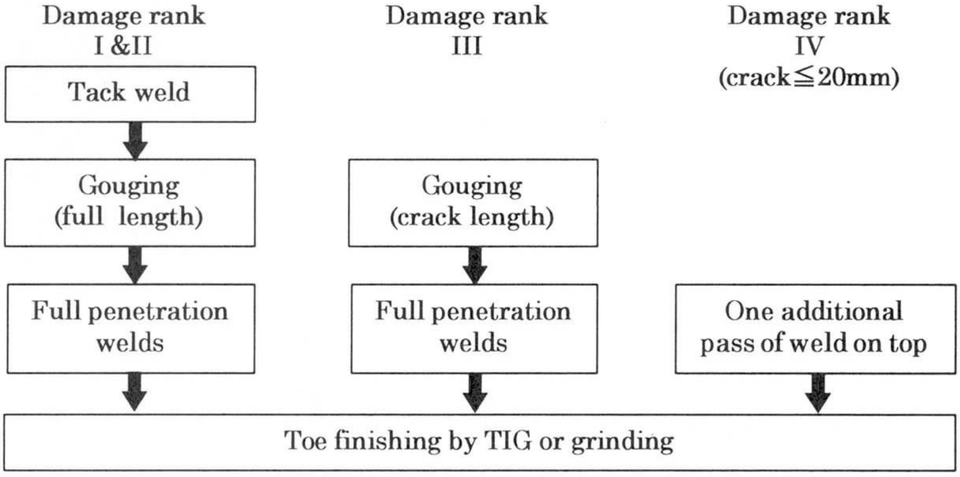
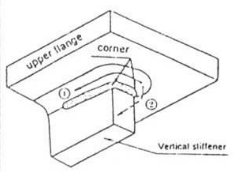


Fig.10 Retrofit methods depend on damage ranks

TIG-dressing conditions

position aimed at by TIG arc	the toe of weld bead
electrod	thoriated tungsten 3.2mm in diameter
current	240 A
flow rate of argon	10 ℓ/min
TIG arc movement	10~20 cm/min 

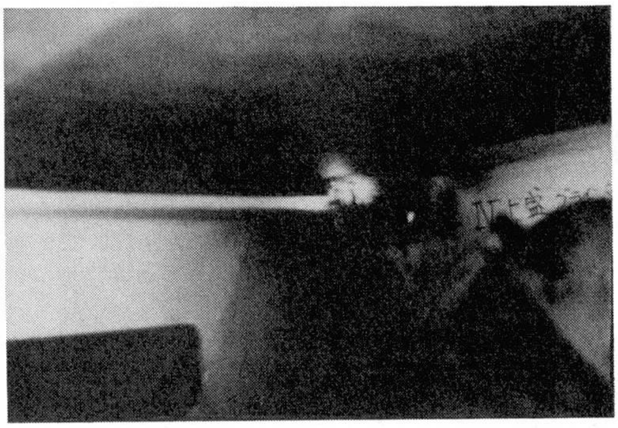


Fig.11 Finishing by applying TIG dressing

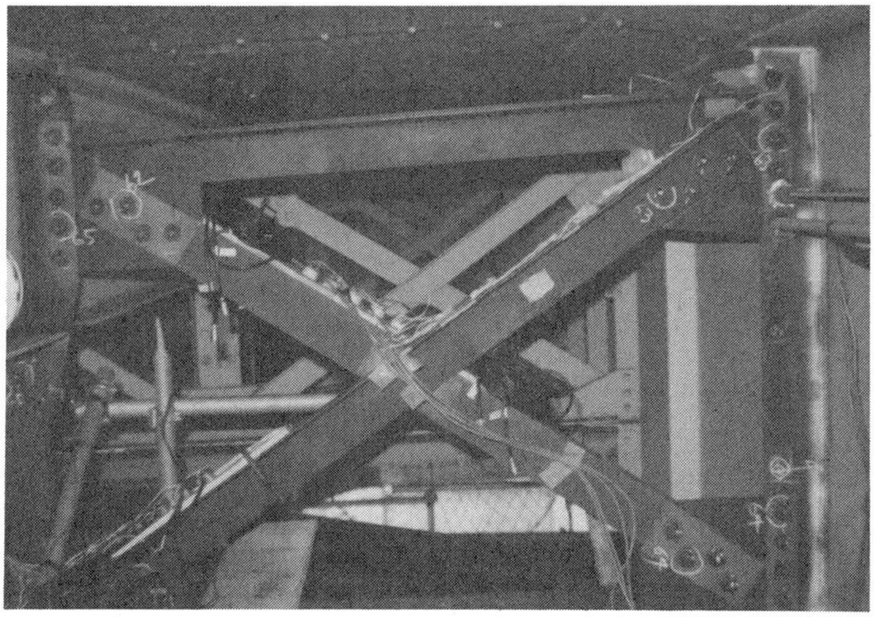


Fig.12 Adding work of sway bracing

What have become conspicuous recently are cracks around sole plates of bearing points of main girders (Fig.13).Fatigue cracks have occurred in fillet welds for attaching sole plates to main girders and fillet welds between main girders and webs The feature of this damage is that it occurs in a girder which has impairment of the rotating function of its support. The bearing point is the point where loading concentration is the severest in a bridge, while structurally , since this location is at a distance from the neutral axis, the stress is different from that according to the beam theory, and the deterioration of the support function causes such a condition to come about. Regarding this damage, the sole plate is exchanged for a larger one, attaching it with high-strength bolts, while further, structural improvements such as to add vertical stiffeners are provided (Fig.14).

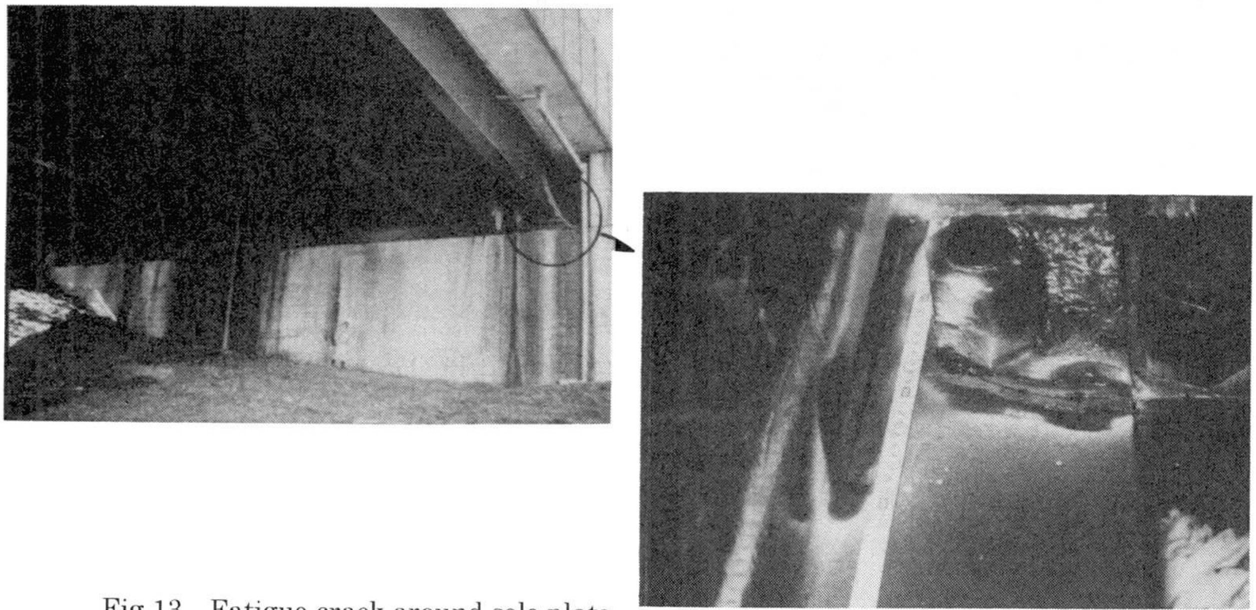


Fig.13 Fatigue crack around sole plate

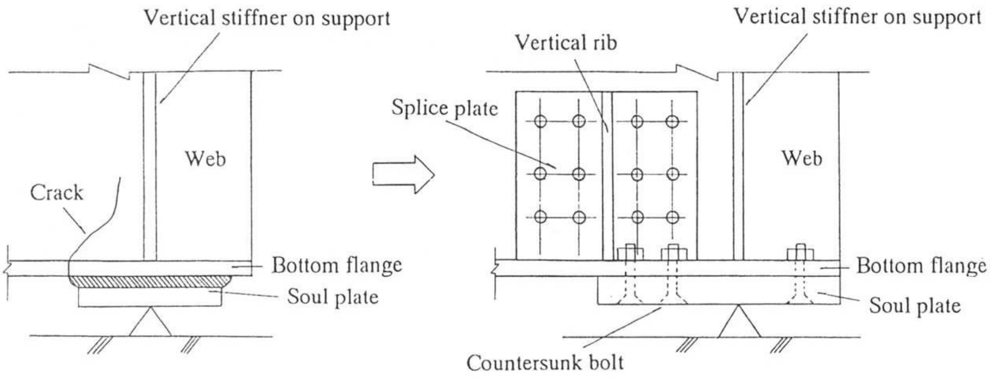


Fig.14 Retrofit detail for cracks at sole plate

3.3 Damage prevention works

Damage to reinforced concrete deck slabs is a general type of damage in highway bridges constructed in Japan around that time. On the Tomei Expressway stringers were added between main girders as a measure against such damage to reinforced concrete slabs. Recently, work to increase thicknesses of deck slabs is also being performed(Fig.15). It has been ascertained that there works also has effect in alleviating stresses at the sway bracing attachment portion.

Work to add an extra lane is going on at the Tomei Expressway to increase its capacity. This work has been taken as an opportunity to improve fatigue strengths of existing parts by changing structural details. Damaged reinforced concrete deck slabs have been changed for thicker reinforced concrete deck slabs or composite deck slabs or steel bridge decks (Fig.16).

The Design live load for throughway bridges in Japan was changed on Nov., 25 1993, The maximum weight of design truck 25 ton varied with the length of truck and the distribution of axes in the new rules, as against the weight of design truck was 20 ton in old rules. The studies on the influence of this change on the existing structures has been proceeding by many institutions and committees.

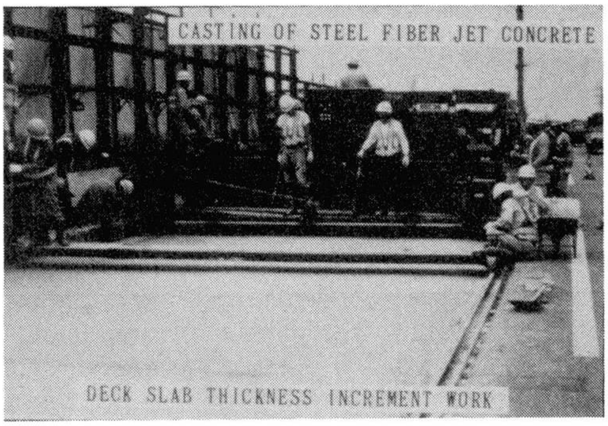


Fig.15 RC deck slab thickness increment work

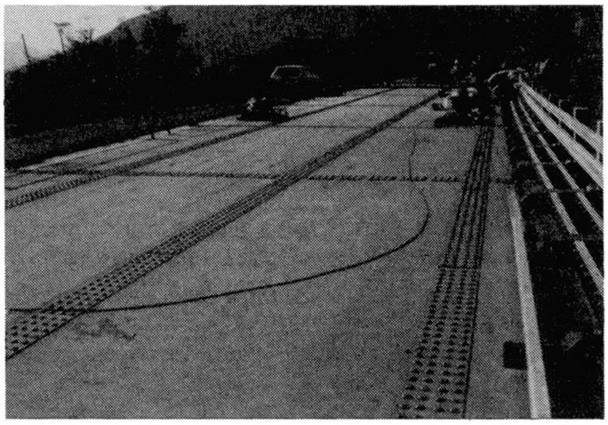


Fig.16 Replacement works of damaged RC decks to prefabricated steel bridge deck

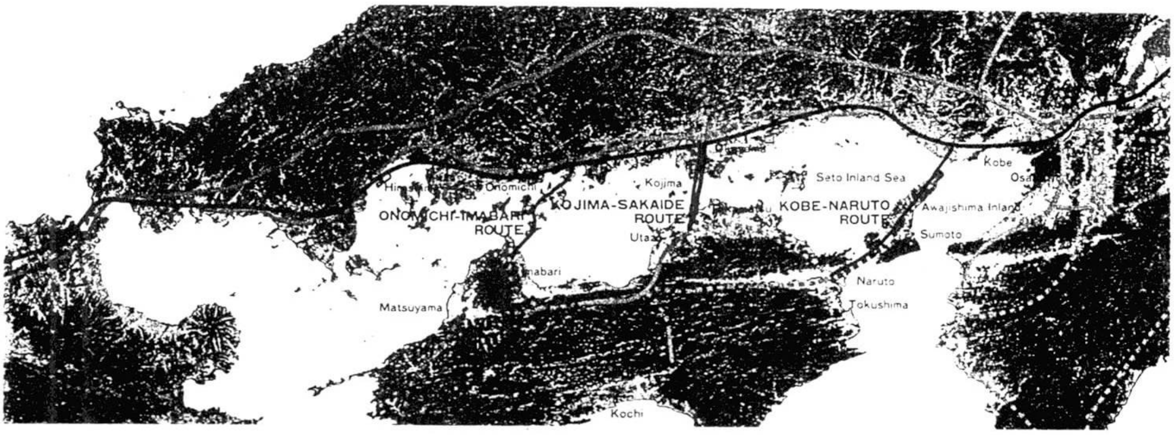
4. HONSHU-SHIKOKU BRIDGES

4.1 Fatigue design and quality control

Of the three routes of the Honshu-Shikoku Bridge project (Fig.17), the central route consists of combined highway and railway bridges with fatigue design made a priority item along with wind-resistant design from the beginning. Since high-strength steels of 800-MPa class were used in large quantities, large-scale fatigue tests were started from 1970(Fig.18), and these tests have made great contributions to the fatigue design in Japan of today. Numerous tests of full-size or large-scale models (Fig. 19) were carried out for investigation and ascertainment of plate thickness effect on fatigue strengths of various welds and fatigue behaviors due to secondary stresses attributable to structural details. Test specimens about 15 mm in plate thickness and 100 mm in plate width had been mainly used in past fatigue tests, but specimens with plate thickness of 45 to 75 mm were used in this case.

Various new and useful things have been learned from these fatigue tests, typical of which may be cited fatigue of longitudinal-bead welds of box section members (Fig.20). Such joints play almost no role in transmitting load, and simply are for assembly of members, but if defects such as blowholes exist at root portions, fatigue strength will be extremely reduced. The relationship between defect size and fatigue strength has been ascertained from the results of experiments on many large-sized models and from fracture mechanics models (Fig.21), based on which design allowable fatigue stress and corresponding allowable defect size have been determined (Fig.22).

Fabrication methods have been improved so that all truss members will have defects not more than the allowable limits. Joint portions are inspected using a newly developed automatic ultrasonic flaw detection system, and defects exceeding allowable values are repaired. These nondestructive inspection results have been organized as a data base, and play an important role in the maintenance plan.



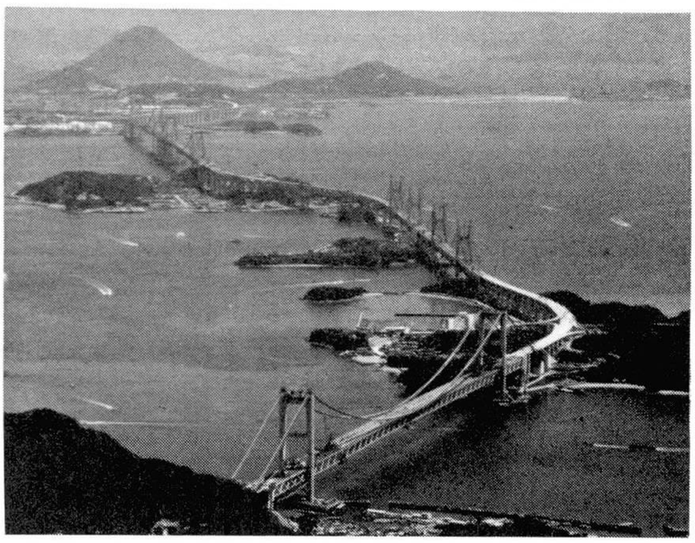


Fig.17 Honshu-Shikoku Bridge Project and general view of Seto-Ohashi (Kojima-Sakaide route)

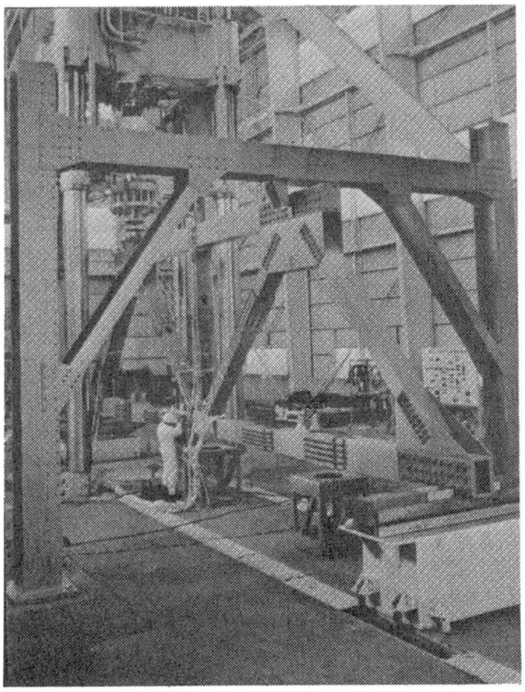
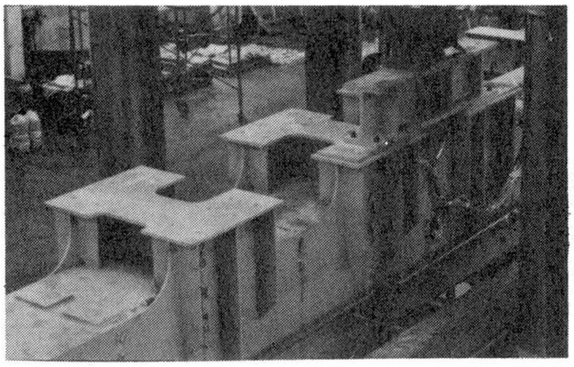
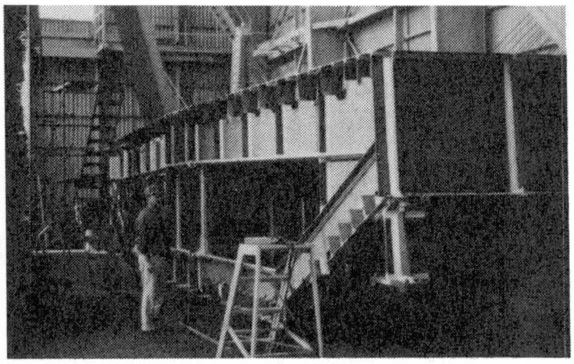


Fig.18 Large scale fatigue test
(truss panel joint)

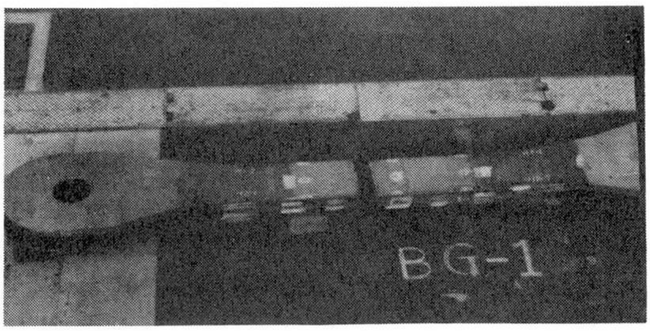


(a) floor beam of stiffening truss

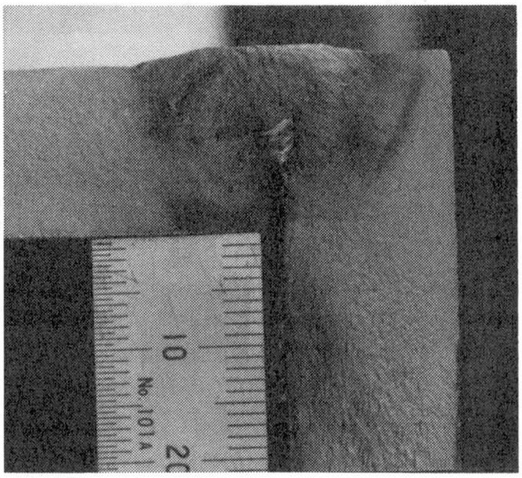


(b) box section stiffening girder

Fig.19 Examples of fatigue test

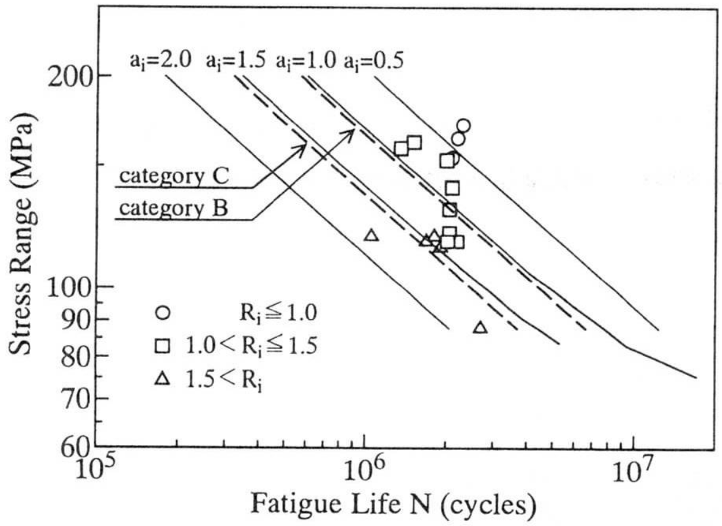


specimen



crack initiated from blowhole

Fig.20 Box section members



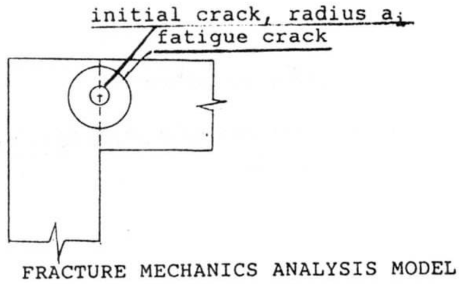
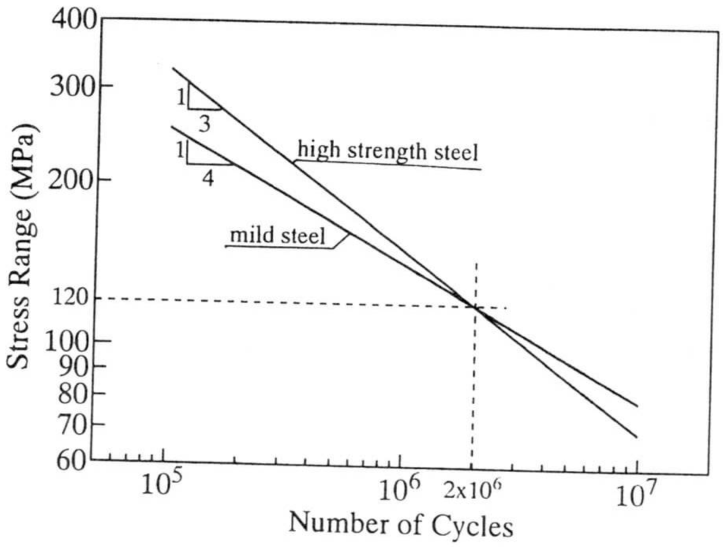


Fig.21 Results of fatigue test and fracture mechanics analysis



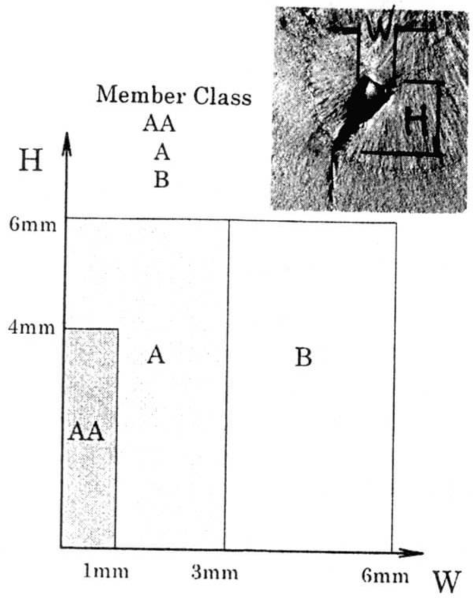
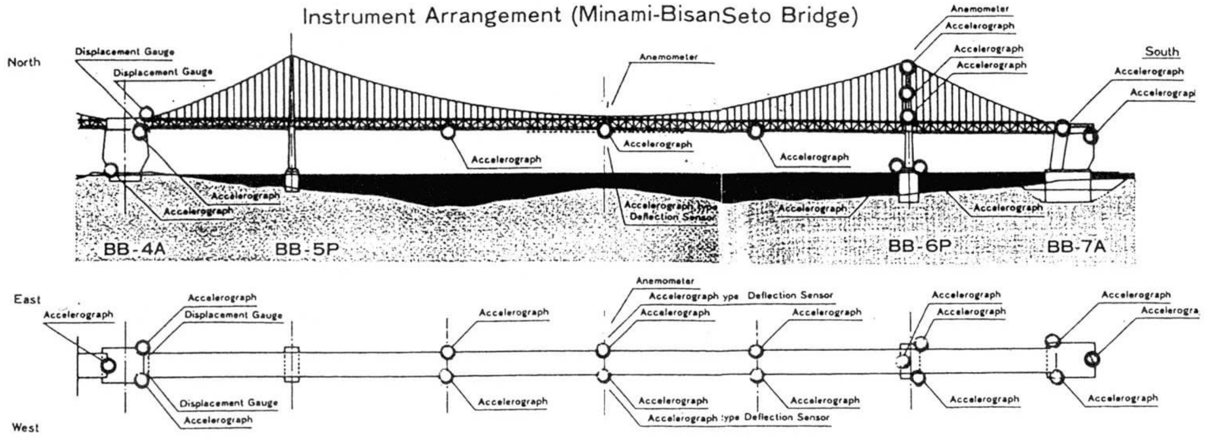
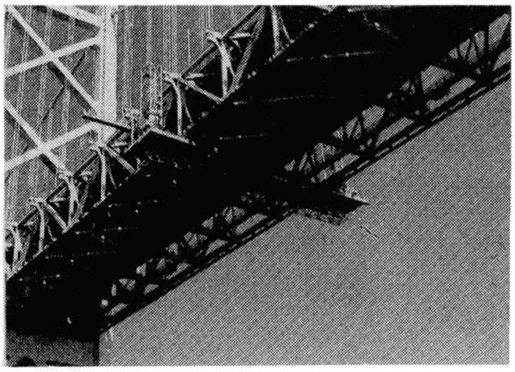


Fig.22 Allowable stress for corner joint and permissible size of blowhole in corner joint

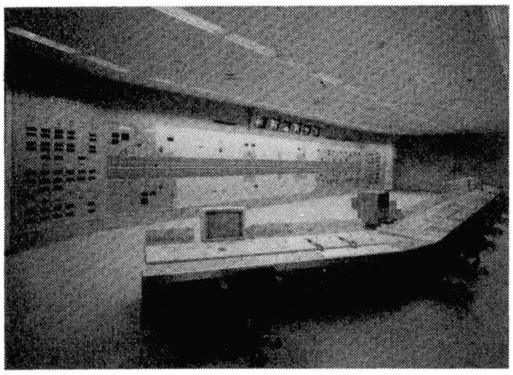
4.2 Maintenance

Various new technologies have been applied in maintenance also. Honshu-Shikoku bridges are equipped with inspection cars making it possible for on-hand inspections to be made of all members. And as shown in Fig. 23, various types of sensors such as accelerometers and displacement meters are installed and the behaviors of bridges are monitored at all times at a central control room. Besides these data being used for operations, the records comprise a base for formulating maintenance plans.





inspection car



control room

Fig.23 Instruments for monitoring of bridge behavior

5. CONCLUSIONS

The service environment of a bridge, such as loading, often differs greatly from what had been assumed at the time of designing. To evaluate the degrees of soundness of bridges from both the aspects of function and load bearing power taking such situations into account to set up maintenance plans including proper retrofitting and replacement is a matter of extreme importance.

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