KEYNOTE LECTURES

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Maintenance Programme of Shinkansen Structures

Programme d'entretien des ouvrages d'art du Shinkansen

Wartungsprogramm für die Shinkansen-Bausubstanz

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SUMMARY

It is a quart of a century since the Tokaido Shinkansen was opened for public service. The structures that have supported the operation of the bullet train system without any accident are showing signs of deterioration. This paper is intended to describe an outline of the Shinkansen structures and their maintenance inspection system, and to indicate fatigue damages of steel structures and retrofitting programme.

RÉSUMÉ

Vingt-cing années se sont écoulées depuis la mise en service du train à grande vitesse Tokaido Shinkansen. Les ouvrages d'art ont supporté les opérations du système de train à grande vitesse sans aucun accident mais montrent maintenant des signes de détérioration. L'article présente quelques données générales concernant les ouvrages d'art du Shinkansen et de leur système d'inspection et d'entretien, ainsi que certains dommages aux structures métalliques; un programme d'assainissement est esquissé.

ZUSAMMENFASSUNG

Ein Vierteljahrhundert ist vergangen, seit der Tokaido Shinkansen in den öffentlichen Dienst gestellt wurde. Die Bausubstanz, die den Betrieb der Hochgeschwindikeitszüge ohne Zwischenfall durchgehalten hat, weist Anzeichen von Schäden auf. Ziel dieses Beitrages ist es, eine Uebersicht über die Shinkansen-Bausubstanz und ein entsprechendes Wartungssystem zu geben und auf eventuelle Ermüdungserscheinungen der Stahltragwerke sowie ein Programm mit Instandstellungen hinzuweisen.

1. INTRODUCTION

The Tokaido Shinkansen, since its opening to the public in 1964, has played an important role as the main trunk line covering a distance of 515km between Tokyo and Osaka. With a maximum operating speed of 220km per hour, 230 trains run daily. The daily passenger turn-out reaches as high as 300 thousand.

This highly congested bullet train system has been supported without any accidents for 25 years by the Shinkansen structures. This is credited to the continuous research and the careful maintenance efforts.

Now that 25 years has passed since its construction, none of the structures have been fatally damaged yet. They, however, exhibit various signs of degeneration. In particular, the fatigue damage to steel structures attracts our close attention.

Currently, fatigue damage has developed only in the secondary members not incorporating any fatigue design. No fatal accidents have taken place yet because these fatigue cracks were discovered and well repaired soon.

A method for maintenance control of steel structures of all the Shinkansen structures will be described below.

2. OUTLINE OF SHINKANSEN STRUCTURES

The track structures of the Shinkansen were all designed in grade separation, and therefore include a number of varieties over the entire distance. Fig. 1 shows the ratio among various structures over the whole distance of the Tokaido Shinkansen, and Table 1, the number of bridges by type.

3. DESIGN AND FABRICATION OF STEEL BRIDGES FOR THE TOKAIDO SHINKANSEN

3.1 Construction of Tokaido Shinkansen

In face of the then severe transportation situation of the Tokaido Line connecting Tokyo and Osaka which is a conventional railway system, it was imperative to complete the construction of the Tokaido Shinkansen within as short a period as five years starting in 1959, if only to meet the requirements of the planned Tokyo Olympic in 1964.

Other considerations facing us included how to deal with the unknown effect on loading due to a service speed greater than 200km/h, the increase of tonnage and the increase of the number of trains in the future.

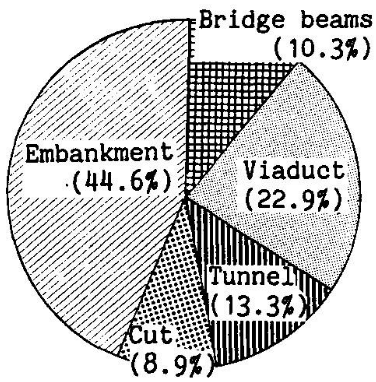


Fig. 1 Ratio of structures

Type	Structure	Number of spans
Steel	Deck Plate Girder (I section)	194
	Deck Plate Girder (Box)	139
	Through Plate Girder	155
	Composite Beam	258
	Through Truss	135
Concrete	Reinforce Concrete	3307
	Presstrest Concrete	389
	Total	4577

Table 1 Quantities by Bridge Type

Actually, however, we were allowed only about three and a half years in real terms as time was consumed for land acquisition and other additional procedures. Furthermore, the construction budget was limited. Thus we were forced to rely on standardization of as many designs as possible to attain maximum economy.

After studying these problems from various angles, the design concepts shown below were employed.

3.2 Design Code

Basic design items for steel bridges of the Tokaido Shinkansen which were established in 1959, as compared with those of the conventional railway system and the current bullet train system, are shown in Table 2.

Followings were principal points in design.

(1) The type of design live load has changed from that of the locomotive-hauled train load (KS load) to that of the electric car load (NP load) in which the entire axle load is a concentrated load of the same value of 160kN. (Fig. 2)

(2) Because the great number of loading repitition, conventional fatigue assessment which based on the 2×10^6 cycle fatigue strength became insufficient. However, there was no long life fatigue date available. As a result, a live load for fatigue design was 180kN which was increased by 20kN for the standard live load and fatigue was assessed at 2×10^6 cycles.

(3) The design fatigue life of the steel bridge structure is 70 years, as compared with 55 years for steel bridges in conventional railroad system.

(4) In order to secure riding quality during high-speed train operation, a deflection limit of 1/1800 was set to keep the acceleration below 0.2g (g : Acceleration of gravity).

3.3 Design Considerations of Fatigue

When a Shinkansen train crosses a bridge, the stress variation, as shown in Fig. 3, occurs. In this way, the shorter the span, the more stress repetitions occur. In a stringer of about 2.5m long, for example, separate one stress cycle would

	Conventional Railway System (1964)	Tokaido Shinkansen (1964)	Current Bullet Train System (1988)
Max speed	120km/h	210km/h	260km/h
Design live load	KS-18	NP-18	N-18, P-19
Allowable stress	130 MPa	130 MPa	150 MPa
Deflectional ratio	Girder bridge 1/800 Truss 1/1000	1/1800	Span 40m or less 1/1800 (*) * Reduced according to span for 40m or longer
Fatigue	$\sigma_{fa} = \frac{\sigma_{f0}}{1-2/3\kappa}$ $\kappa = \frac{\lvert\sigma\rvert\ min}{\lvert\sigma\rvert\ max}$ σ_{fa}: Allowable fatigue stress range σ_{f0}: Standard allowable fatigue stress range	Ditto Live load 20kN more than standard (160kN)	$\sigma_{fa} = \beta \cdot \gamma \cdot \sigma_{f0}$ β: Variation factor depending on application range γ: Variation factor depending on type of track, frequency and member

Table 2 Comparison of Design Standards for Bridge of Shinkansen and Existing Railways

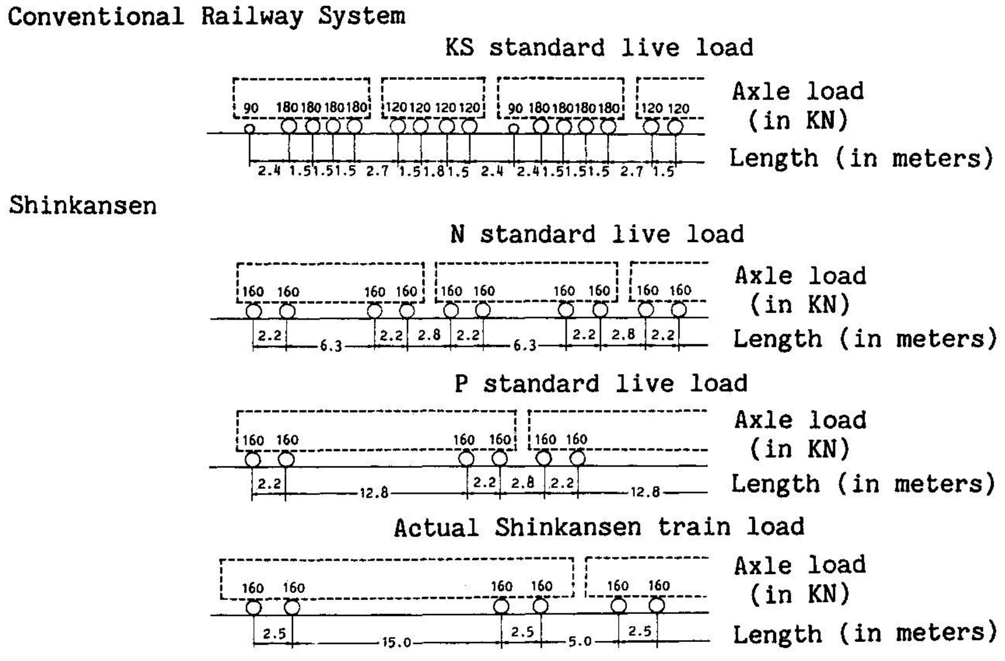


Fig. 2 Design live load

occur for each axle by calculation. In an actual bridge, however, the rail stiffness and the diffusion effect of the ties dampen them to about 50 million for members 6m or less in influence line length, 20 million for those 20m or less and about 3 million for those 25m or more in seventy years. In order to determine the fatigue strength used for design, on the other hand, the following points were studied. The fatigue test was conducted on various joints. As a result, considering that the butt welded joint has a fatigue limit of about 10 million stress cycles (110 MPa), its ratio to the fatigue strength at 2×10^6 (130 MPa) that had been used for design of steel bridges in conventional railroad systems, that is, 130/110 (=1.18) was introduced as a load factor, which was multiplied by the average axle load of the Shinkansen (151kN), thus reaching the design load of 180kN which mentioned above.

3.4 Considerations for Fabrication Processes

Fatigue strength depend on the quality of the welds. In order to satisfy the quality requirement of welded joints, the all welders engaged in bridge fabrication. The qualification was required of all welders. For the purpose of securing a sound welds, standardization was introduced and an acceptable level for weld defects for a nondestructive test was set up. In parallel to these actions, a quality control technique was used for part of the structures in order to assure an efficient physical inspection of the great mass of bridges fabricated.

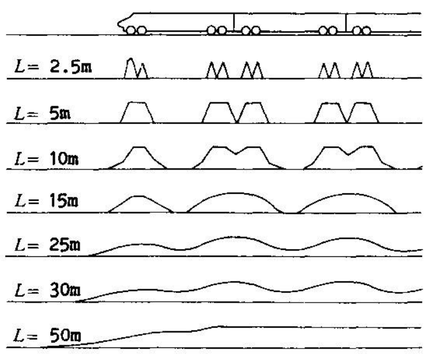


Fig. 3 Stringer stress waveform under train operation

4. MAINTENANCE INSPECTION OF STRUCTURES

4.1 Inspection System

All the structures are inspected once every other year. The inspection is made visually from inspection deck, called a "general inspection". According to the result of this general inspection, conditions of structure are evaluated and classified into four classes, and the structures which are especially problematic are subjected to a detailed inspection, called a "special inspection" by using precision instruments devises, and tools.

The general inspection is conducted at Maintenance of Way Depots located at regular intervals of about 50km along the line. Each depot is occupied with 6 to 8 staff members engaged in maintenance of the structures. A total of more than 5,000 personnel tackle the general inspection every year.

The special inspection is conducted in four structural inspection centers where civil engineers with highly professional knowledge are assigned. At each center, about 20 engineers are engaged in the inspection and subsequent repair and reinforcing work.

All the inspection results, especially those regarding fatigue damage to the steel bridge, are all reported to the headquarters, which in cooperation with the Railway Technical Research Institute, and the causes and retrofitting methods were studied.

4.2 Evaluation and Rating

The inspection results are arranged and reported based on degree of damage and the influence on train operation.

Criteria for general inspection are divided into four classes including A, B, C and S. The basic approach is shown in Table 3.

A structure which is considered to pose a problem as a result of the general inspection is covered under class A and is subjected to a special inspection. For the special inspection, the criteria are subdivided into three ranks. These subdivisions mainly depend on structural redundancy. The basic approach is shown in Table 4.

On the basis of these inspection results, a construction schedule for repair and retrofitting is formed.

	Basic Concept for Classification
A	Urgent need to take action.
B	Likely to degenerate to rank A in the future; requires action appropriately.
C	Minor damage.
S	Sound.

Table 3

4.3 Maintenance of Steel Bridges

With regard to the inspection, evaluation and rating in steel bridge maintenance program, various manuals and standards have been prepared. Routine maintenance and inspection duties are performed based on those. The nature of the manuals is briefly described below.

(1) Important inspection points are illustrated to eliminate inadvertent cases of overlooking.

(2) The manuals include many experience case for reference of evaluation.

(3) The critical fatigue crack size is indicated to prevent catastrophe failure.

	Basic Concept for Classification
AA	Needs immediate action.
A1	Action is urgent (within about one year).
A2	Action is urgent (within about two years).

Table 4

(4) In order to evaluate ultimate strength, remaining life and runability quantitatively, a procedure and a criterion are indicated.

(5) Many cases have been cited in the manuals to serve as practical applications.

5. FATIGUE DAMAGE AND RETROFITTING

5.1 General Description of Damage

The steel bridges in the Tokaido Shinkansen have suffered very sevire loading condition caused by high-speed train operation and highly repetitive frequencies. Nevertheless, a fault in the structure has never led to an accident.

In the process of their service, however, some types of fatigue damage have been observed as shown in Fig. 4. Of these types of damage, some fatigue cracks which are comparatively rare in the conventional railroad system, have come up. A fatigue crack has not yet been detected at important portion of main members which lead to catastrophic failure where fatigue was assessed in the design stage. Fatigue cracks are, however, often discovered at such secondary members as side walks, connections of attached facilities and diaphragms or secondary local portions of the main members.

They are in many cases caused by the out-of-plane vibration or dis-placement of members, settlement of the fulcrum due to failure of the bearing or differential deflection of the main girder affecting stress-concentrated parts such as copes. A retrofitting procedure and improvement detail for some typical damage will be described below.

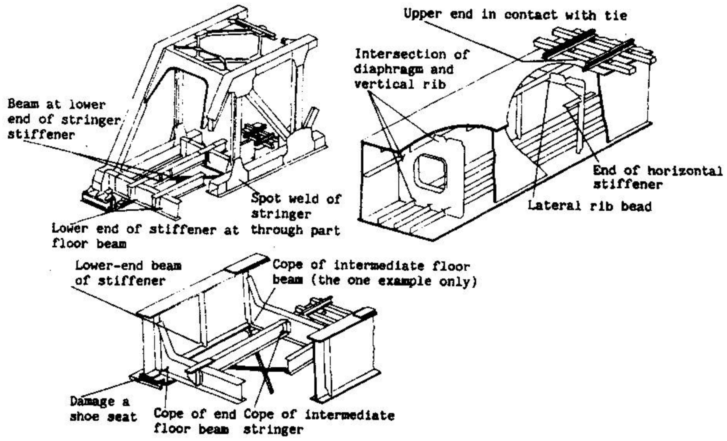


Fig. 4 Outline of damage

5.2 Coped Floor Beam of Through Type Girder Bridge

This damage is primarily caused by the settlement of bearing, therefore occurrences are significant in end floor beams. Causes and retrofitting methods were studied through various measurements on actual bridges, structural analysis and fatigue tests. The result described below were obtained.

(1) The crack was causes by the fact that the end of lower flange of the floor beam was coped to connect with main girder which induced stress concentration. The settlement of supporting point was also one of causes of this fatigue damage. (See Fig. 6)

(2) By applying an additional plate to the web plate to increase loading capacity, a sufficient reinforcing effect is obtained.

(3) Based on those experiences the details of the currently designed bridge have been improved as shown in Fig. 7.

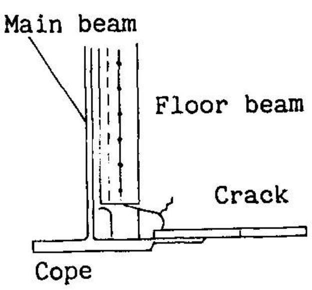


Fig. 5 Crack at floor beam cope

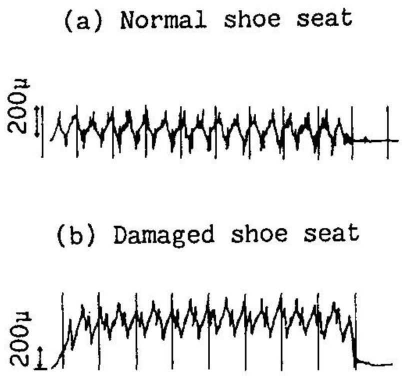


Fig. 6 Actually-measured strain waveform of floor beam

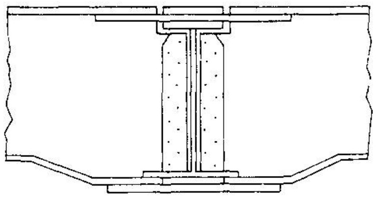


Fig. 7 Improved details of floor beam

5.3 Cope of Stringer

As shown in Fig. 8, this damage is a crack initiated at the stringer web plate of the through plate girder or the through truss bridge.

The stringer is a member which is directly subjected to the train load, and for the great effect of lateral force and impact by train. It is necessary to avoid a local stress concentration by improving the lateral rigidity or keeping the stress flow as continuous as possible. Some bridges employed at the time, however, had a lower flange of the stringer not connected to the floor beam, resulting in a vibration of the stringer. Furthermore, the stress concentration at the cope was combined to lead to cracking.

In order to get rid of this problem, it is necessary to increase the lateral rigidity of the stringer end as well to connect the lower flange with the floor beam web plate by extending the lower frange to the stringer end.

Fig. 10 shows a structural detail of connection of the stringer and floor beam used for subsequent designs.

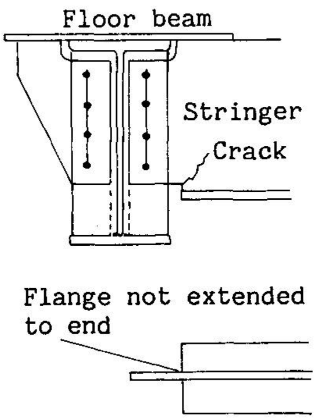


Fig. 8 Crack at stringer web plate

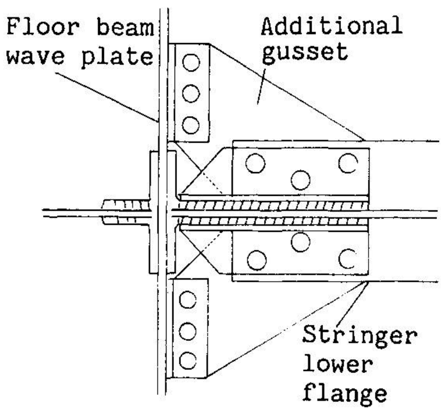


Fig. 9 Repair drawing of stringer

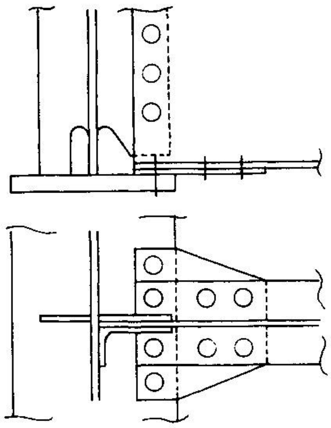


Fig. 10 Improved details of stringer end

5.4 Diaphragm in Box Section Deck Type Girder Bridge

In the box-section girder bridge, a diaphragms are provided at intervals of 5 to 6m in order to improve the torsional rigidity. As shown in Fig. 11, fatigue cracks were observed on the surface of diaphragm at the toe of fillet weld which connected longitudinal rib and diaphragm. The main causes of this phenomenon are high welding residual stress, stress concentration at weld toe and high structural constraint. A diaphragm with such a structural detail appears to develop an out-of-plane vibration with the passage of a train and develops considerable fatigue at the restrained weld.

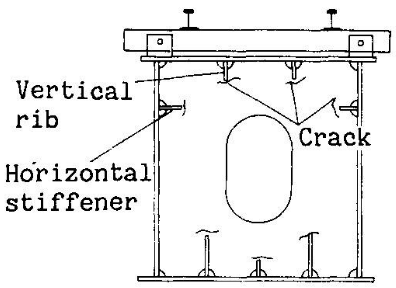


Fig. 11 Crack of box-section diaphragm

This damage is repaired by drilling a stop hole at the crack tip, or re-welding after gouging and TIG arc remelted were applied at the toe of fillet welds. Also, in a current design, an out-of-plane vibration-proofing is considered or a diaphragm with a ribs (Fig. 12) is introduced.

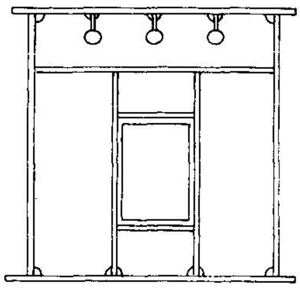


Fig. 12 Improved diaphragm

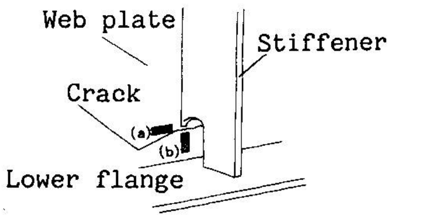
5.5 Vertical Stiffener in Web Plate

5.5.1 Outline of Damage

Fatigue cracks, as shown in Fig. 13, initiated at the lower end of the vertical stiffener attached to the web plate of the stringer of truss girders or the web of box-section deck girder bridge.

This crack originated primarily at the toe or root of the fillet weld around the lower end of the vertical stiffener and graw horizontally into the base metal of the web plate. Such a crack may sometimes propagate along the weld toe, followed by progress in the direction horizontal to the base metal. Generally, the trend as shown in Fig. 14, is observed for the box-section girder and the truss stringer.

This type of fatigue damage is increasing more and more and has many points where it is likely to develop. As a result, how effective a retrofitting measure can be taken is a great problem therefore, the matters described below were studied.



Note: (a) and (b) show positions of strain gage at the time of stress measurement

Fig. 13 Crack of web plate at lower end of stiffener

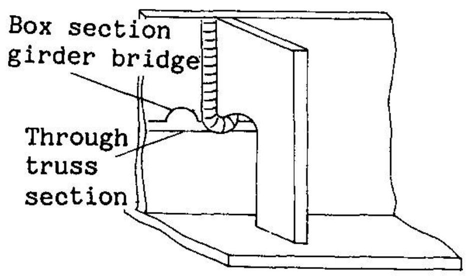


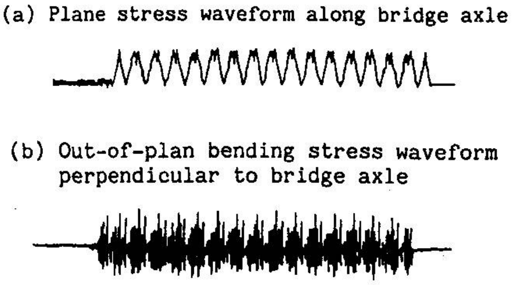
Fig. 14 Progress of cracking

5.5.2 Retrofitting Procedure

Investigation by measurement of actual bridges and analysis revealed that the main cause of this damage was the stress at the toe of the fillet weld was induced by the out-of-plane vibration of the web plate at the lower end of the stiffener.

Assessment of fatigue on this detail in the design stage is performed on the assumption that membrane stress in the longitudinal direction in the web plate is exerted on the weld around the vertical stiffener as a cruciform joint. Actually, however, the out-of-plane bending is induced actually as shown in Fig. 15.

In order to improve fatigue performance of this transverse fillet welded joint, several methods are available. Among them, we have employed the tungsten inert arc gas (TIG) are remelting process which is considered most economical for its effect. This welding process is a technique which is used to finish weld toes to smooth shapes by remelting the toes with non-consumable tungsten electrodes. Fig. 16 shows a fatigue test result with the effect of the TIG arc remelting process, indicating a considerably high effect.



Note: For the position of strain gage at the time of stress measurement, see Fig. 14.

Fig. 15 Plane vibration and out-of-plane vibration

Fig. 17 shows an outline of the retrofitting process with TIG arc remelting. About ten thousand joints in actual bridges were finished following by this method.

As explained above, some failures have been discovered on the Tokaido Shinkansen and have been repaired before they reached the serious phase.

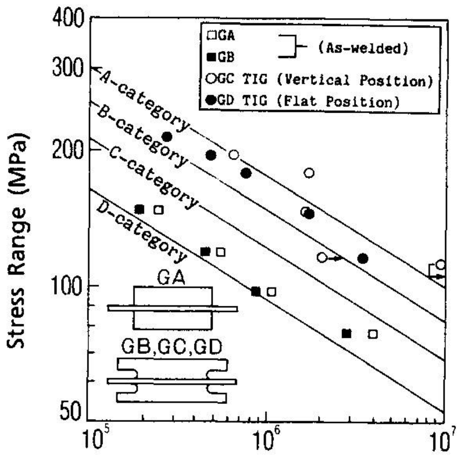


Fig. 16 Results of fatigue test

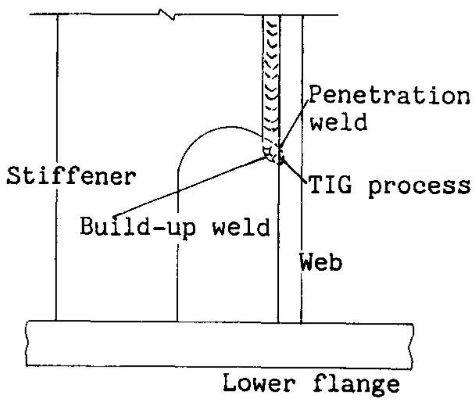


Fig. 17 TIG welding process

5.6 Prediction of Fatigue Damage

Because, most structural details in steel bridges were standardized, it is likely that similar failures occur at certain points when a certain type of fatigue failure occurs. A more rational maintenance program is an important subject to cope with this serious problem.

One preventive method is to grasp the degree of fatigue damage of the bridges based on stress measurements in actual bridges. The rain flow counting method is used to obtain stress range histgram from stress records and the equivalent stress range is calculated by applying teh Miners law. The values indicated in Table 5 are applied for the evaluation of fatigue strength. This table has been prepared for the measured stress based evaluation of fatigue assessment.

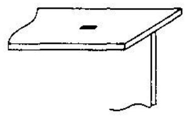
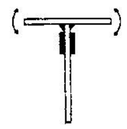
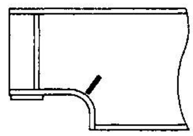
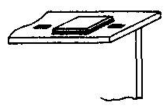
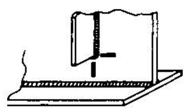
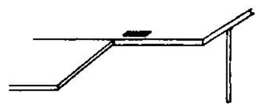
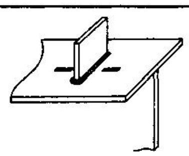
Type of structual joint	Standard allowable fatigue stress at 2×10^6 cycles (MPa)	Slop	Ellustration example and locating of strain gages
Base metal and weld metal in members connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	125	3	
Base metal adjacent to full or parallel penetration groove welds or by continuous fillet welds perpendicular to the direction of applied stress which induced by out of plane bending.	80	3	
Basemetal adjacent to full penetration groove welds at the reduced section end of girder.	65	3	
Base metal adjacent to fillet welded tie-base-plates when weld are ground to provide smooth profile	80	3	
Base metal adjacent to fillet welds at the end of vertical stiffener in the longitudinal direction of girder (for membrane stress in longitudinal direction).	80	3	
Base metal adjacent to fillet welds at the end of vertical stiffener in the perpendicular direction of girder (for bending stress which induced by out of plane displacement of frange).	65	3	
Base metal at details attached by groove welds subject to longitudinal loading, when provided with trasition radius equal to or greater than 20mm and weld end ground smooth.	80	3	
Base metal at details attached by fillet welds with detail length L, in direction of stress less than 40mm.	65	3	

Table. 5 Standard Allowable fatigue Stress Range at 2×10^6 Cycles for Measured Equivalent Stress Range

6. CONCLUDING REMARKS

Appropriate preventive program, including appropriate retrofitting works, have been taken from the early stage of fatigue damage in steel structures in the Tokaido Shinkansen. As a consequence, we have been maintaining the tracks in good condition without any serious accidents. After 25 years since the beginning of service, however, there is a great demand for increasing train speed up to 270km/h or a greater transportation capacity. This will probably add more and more to the burden of the structures.

Taking advantage of the standard design structures of the steel bridges, we selected some representative bridge of each type for monitoring fatigue crackings. Causes of fatigue damages, retrofitting and preventive measures have been studying in these monitoring bridges. This preventive program is necessary not only for steel structures, but also for concrete and soil structures.

In view of this, Central Japan Railway Company has starded A Structure Committee made up of members from outside who are versed in railway structures. While making efforts to secure the soundness of future structures, we are studying methods of replacement at the same time. We are being required to form a specific program to solve problems and while executing the plan, to establish a stable transit system.

REFERENCES

1. Taizo Nozawa, Yukio Yamada : "Present Situation and Problems of the Shinkansen Bridges", Railway Civil Engineering 19-3, March 1977

2. Kenji Sakamoto, Makoto Abe : "Fatigue Damage of Steel Railway Bridges and Repair", RTRI Report 2-11, November 1988

3. Kenji Sakamoto, Makoto Abe, Chitoshi Miki : "Fatigue Damage of Steel Railway Bridges and Repair", JCOSSAR '87 Thesises, Safety and Reliability of Structures

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Durability Aspects in Maintenance, Repairs and Rehabilitation

Aspects de la durabilité dans la maintenance, la réparation et l'assainissement

Dauerhaftigkeitsaspekte bei der Unterhaltung, Instandstellung und Sanierung

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SUMMARY

Durability of construction materials is defined, and problems of the quantification thereof are outlined. Design life of structures is considered in terms of repair requirements. Investigative procedures are considered and this leads on to the consideration of surveys of structures and application of some repair methods.

RÉSUMÉ

La durabilité des matériaux de construction est définie et leurs valeurs quantitatives sont commentées. La durée de vie des constructions est exprimée en fonction des besoins en réparations. Des méthodes d'analyse et de surveillance sont présentées, ainsi que des méthodes de réparation.

ZUSAMMENFASSUNG

Die Dauerhaftigkeit der Baustoffe wird definiert und die Probleme bei deren Quantifizierung umrissen. Die Lebensdauer von Bauwerken wird von den erforderlichen Instandstellungsarbeiten abhängig gemacht. Anschliessend werden Untersuchungs- und Instandstellungsmethoden besprochen.

1. INTRODUCTION AND DEFINITION

"Durability is an essential attribute of a building material". While this statement would be agreed upon by most persons concerned in the construction, maintenance, repair and rehabilitation of buildings , the exact meaning of the term durability could lead much discussion. The Shorter Oxford English Dictionary defines durability as "the quality of being able to withstand change , decay or wear". In applying such a definition to a property of concrete, steel or any other building material, it must be appreciated that none of these materials will continue to fulfill its role indefinitely, and, the same material may last a long period in one environment while deteriorating rapidly in another. Consider, for example, a high C3A-cement concrete in dry air, and in contact with sulphate charged ground water, or an unpainted steel structure exposed to dry desert enviroment and in a marine climate.

Rather than consider the dictionary definition, one may seek a definition of durability, and its related concept, serviceabilty, in technical literature related, for example to concrete; in this case we find the following :

Durability - the safe performance of a structure or a portion of a structure for the designed life expectancy. (Note - because the forces of nature , coupled with some man created exposure, may cause progressive deteriorations, these recommendations do not preclude the need for normal maintenance), (from ASTM Recommended Practice for Increasing Durability of Building Constructions Against Water-induced Damage, E 241 -77).

Durability - the capability of maintaining the serviceability of a product, component, assembly or construction over a specified time, (from ASTM Recommended Practice E 632).

Serviceabilty - the capabilty of a building product, component, assembly or construction to perform the function(s) for which it is designed and constructed, (from ASTM Recommended Practice E 632).

In recent structures codes throughout the world significant changes to the durabilitity provisions for materials are occurring. A trend has developed wherein durability is defined with respect to a series of relevant serviceability states as well as designated design requirements in order to resist any specific environmental effects which might apply, such as temperature extremes and aggressive atmospheres. Several points arise from the definitions of durability and serviceability. The most important is that time plays an important role in the property of durability; durability is to be considered at the design stage of a project; and, "normal maintenance" may or may not be allowed for in considering durability.

As yet no-one has satisfactorily quantified the durability of concrete in service, although some attempts at such evaluation have been based on statistical concepts [1]. Durability analyses, statistically considered, have to include both the possibility of a certain event taking place, and the possible consequences thereof [2]. This leads to complicated risk-analysis procedures which are only really worthwhile if the damage mechanisms can be described reasonable well. This stage has not been reached even for thoroughly studied phenomena such as sulphate attack on concrete, where a change in the cation species without changes in the sulphate concentration, may entirely upset any prediction of the rate of attack [3]. It follows that, as yet, the necessary data for undertaking risk-analyses relating to durability of building materials such as concrete is not yet available to engineers under all anticipated exposure conditions.

2. DESIGN LIFE OF A STRUCTURE

A definition of the design life of a structure is "the minimum period for which the structure can be expected to perform its designated function, without significant loss of utility, and not requiring too much maintenance ". Words such as "expected", "designated", and "significant" require definition in turn. Somerville [4] points out that the concept of design life is not new, and quotes codes of practice dating back to 1950 in which figures, for both building components and buildings as a whole, were given. Since then, nominal design lives have often been prescribed. Somerville presents a relationship between the performance in durability terms, and time. This is reproduced as Fig.1. The objective of Somerville's presentation was to set a framework for future research and development work on durability. The diagram shows several important points which according to him reflect the apparent state of the art. These are:

The variation in inherent durability built in at the construction stage (which is not quantified).

The spread in performance, represented by the hatched area, (we need to know more about this in quantitative terms - about the spread itself, and the factors which contribute to it).

The need to define minimum performance requirements - on both axes.

The need for a lifetime performance plan lying somewhere between Curve 1 (normally much too expensive) and curve 3 (both dangerous in safety terms and expensive in remedial terms); some variant of Curve 2 is probably the answer.

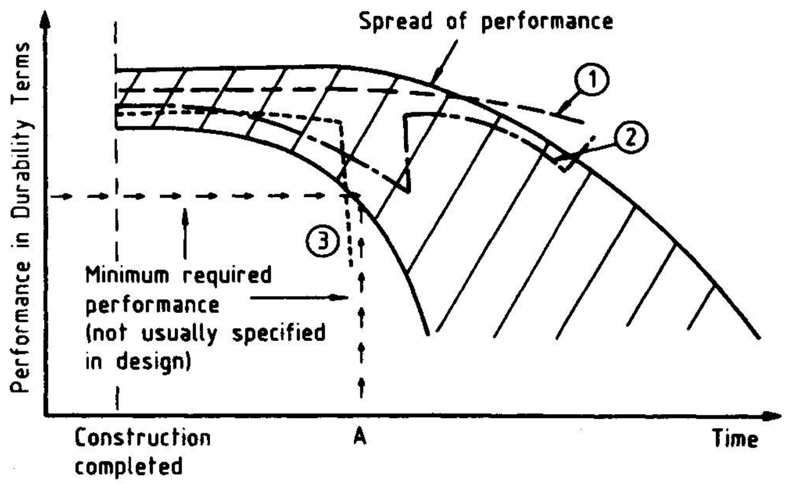

FIG.1 LOSS OF DURABILITY WITH TIME

Somerville notes that filling in the detail on the diagram requires much work in the future, although he suggests that current understanding would permit a start to be made. Great impetus would be given to this work by simply defining A, (the nominal design life). What is important here is the information required and applied to facilitate maintenance, repair and rehabilitation of a structure such that curve 2 can be folllowed. Not only is it important that the repair necessisary to extend the structure's useful life can be executed, but that it can be economically justified.

3. INVESTIGATIVE PROCEDURES

No maintenance, repair or rehablitation scheme can be successful in cost and long-term efficacy unless, prior to the implementation thereof, the effects of age and exposure conditions to which the structure has already been subject are properly assessed. The problems of such assessment are unique to each material or combination of materials used in the structure, and depend on the characteristics and performance requirements of the structure. Unless several essentially similar structures in comparable environmental exposure conditions are to be assessed, no common outline of investigative procedures can be decided upon. For this reason the investigation team must adopt certain of a host of techniques as a means to best assess any problems of durability, which may influence the structural engineer's particular proposed repair scheme. Many of these techniques are not necessarly familiar to the practising structural engineer who is ultimately responsible for decisions, as most are borrowed from disciplines such as applied physics, analytical chemistry, electro-chemistry, mineralogy, metallography and even geophysics. Often the results tend to be descriptive of the conditions rather than in the form of a set of all embracing specific numbers, and even then, many of the units are not those common to the the structural engineer. A comprehensive paper by Clifton [5] describes eighteen non-destructive evaluation methods, that can be used in assessing the condition of concrete and masonary materials and components in structures being rehabilitated or preserved. At least eight of these are covered by ASTM Standards.

When repairs are proposed for a specific type of problem on a particular section of a structure, then, provided that sufficient experience has been accumulated, a well defined assessment scheme can be outlined and followed. The best example of such a problem is the reinforcement corrosion, due to the use of de-icing salts, of concrete bridge decks in North America and elsewhere. In this case a scheme of required procedures has been defined by Manning and Bye [6]. Such a scheme must relate the aquired information back to the repair or maintenance procedure, and in the example chosen, the mapping of defects, information on levels of salt ingress, and other condition data allow cost and management planning to be made, provided the repair and maintenance procedures are themselves well defined, which in this example they are [7,8]. Under certain circumstances these procedures may, with caution, be directly applied to other similar structures such as decks of wharfs influenced by saline water.

The greatest problems arise when aspects of durability are divorced from serviceability during maintenance or repair considerations. The only time that this should be done is when durability is influencing nothing other than the visual apperance of a structure. Under all other circumstances durability of the materials needs to be linked directly to serviceabilty and structural performance criteria. For this reason investigations of structures are best performed as durability and serviceability surveys rater than simply assessments of the durability condition of the materials per se. Two other important objectives of such surveys are to define the actual cause of the durability problem, and to make available to the repair team a mapping of problem areas. From conclusions reached on causes, some assessments can generally be made as to the likely progress of deterioration under continued use of the structure, and its consequences. Also steps can be considered to decrease the attack rates or if possible eliminate the conditions under which the deterioration is progressing.

The use of maps of non-durable sections of structures is obvious. Such mappings can be achieved by many different techniques, but it has been found, in the experience of the writer, that provided the problem presents even subtle visual evidence of its occurrence, the simplest and most informative method is to combine in situ visual inspection with photo-interpretation, using either colour prints or video-tape. Such photo-interpretation methods are based on the same techniques as those used for study of aerial photographs and satellite imagery. Information from either photo-mosaics or video-tape can be rapidly accumulated in computer files, which can in turn be used to present the data in almost any required form. Under particular circumstances sterioscopic techniques may be of special help, but readily available image-analysis programmes for monocular images may be employed for most purposes.

When using such photo-interpretation techniques in particular, but also for general purposes of durability description, it has been found necessary to consider indivdual structural elements, or groups of similar element types, which make up the structure rather than the structure as a whole. This has not often been appreciated or emphasised during surveying of structures. Inspections of concrete structures in situ on large scale have been primarily stimulated by the requirements of maintenance programmes. Guidelines produced by the Transportation Research Board [9], on the assessment of bridge deterioration used a purely qualitative approach. Various defect conditions were assigned an "urgency index" relating to the necessity for repairs according to the observer. In other work [10,11,12] assessment of defiencies is made on the basis of qualitative engineering judgements. Condition surveys of Continuously Reinforced Concrete Pavements (CRCP's) proved impetus to place assessment on a more quantitative basis. Several published works [13,14] indicate the importance of a quantitive approach to the assessment of durability. Carrier and Cady [15], assessed the defiencies of CRCP's on a quantitative basis and then analysed the data using computer methods. Although quantitative in nature, their methods have the deficiency that they were only applicable to slab elements of similar design, and only a limited number of durability defects were considered. A method of universal application to full scale structures relating to a large number of individual concrete members for a larger number of observed deficiencies has been proposed in two publications by Roper et al [15,16]. In these the guidelines for evaluation of the durability of concrete structures apply to all types of concrete structures, be they in bridges, buildings or any other category. A complete durability assessment includes:

(i) Reviewing the structure considering design details, construction reports and maintenance data, all of which provide background information on the structure,

(ii) Classifying the structure in terms of first its use, and then its structural elements,

(iii) Classifying the deterioration phenomena that may occur on these structural elements,

(iv) Carrying out in situ durability assessments or measurements on individual structural elements, and

(v) Combining information and data collected on similar elements, thereby gaining an indication of the durability condition of the structure.

4. RESULTS OF SURVEYS OF CONCRETE STRUCTURES IN AUSTRALIA

In a recent Australia-wide survey by questionnaire on repair problems and procedures, 621 reported cases related to structural problems, whereas 484 were related to diminution of functional efficiency and surface aesthetics of the structures. Also reported were 590 structures which showed effect of concern to the owner and public, but not to the engineer. These figures do not support the conclusion that the necessity for repair of structures in Australia principally results from any non-durability of the concrete. Structural problems are still of great importance in the cost of repair.

The use of Fly-ash in concrete has recently been the subject of considerable criticism [17,18]. In an extensive durability survey of NSW structures and Sydney buildings its role in the durability problems of actual in-service structures has been considered.Two dams, two pavements, a water channel, a retaining wall and a series of slabs on grade were studied, using apart from conventional Civil Engineering Techniques, those of electron microscopy and chemical analysis. A general conclusion was that PFA concrete fulfilled the requirements of the design engineer, and that durability problems were not accentuated by its use. Although carbonation rates were shown to be greater for PFA concretes than for comparable OPC concretes, overall durability was similar for structures build with the two types of concrete.

A total of 136 buildings were examined to assess their long-term durability. Neither PFA nor OPC concrete can boast a proud record of long-term durability, as between 80 and 90 percent of the building examined showed some type of durability defect. No evidence could be found to suggest that a sub-group of buildings, all of which were constructed using PFA concretes, are in any better or worse condition than buildings chosen at random which may have been constructed using OPC or PFA concretes. There is some evidence to suggest that, if problems occur in PFA concrete structures, their appearance is observable at somewhat earlier ages than is the case for all non-durable buildings.

The conclusions that in the "old days", buildings were better built and hence more durable, and that the introduction of modern mix designs or techniques has caused the non-durability, cannot be substantiated or refuted using the available data. The demolition of less durable members of older age-strata sets may influence such data significantly. As a general rule, lower percentages of non-durable buildings are found in older age groups, if stained buildings are not considered non-durable. For the total set of buildings it ws found that all age groups are similarly affected by corrosion of reinforcement (Roper, 1985).

5. REPAIR EXAMPLES INVOLVING STRENGTHENING

Janney [20] discusses repair techniques for columns or piers, which he states may need strengthening for one or more of the following reasons:-

(i) Concrete deterioration or low strength original concrete.
(ii) Corrosion of reinforcement or inadequate amount included in design or placed during construction.
(iii) Load increases over those originally provided for in original design due to unanticipated change in use.

He notes that columns and piers derive their load carrying capacity from the interaction of concrete and reinforcing. Repair or strengthening methods must, therefore, asure that added concrete and/or reinforcement act with the existing materials. He considers a column which has suffered corrosion of the vertical and tie reinforcement. "Assume this corrosion came about becuse the original concrete strength was low and the concrete cover was inadequate to protect the steel from a very humid atmospheric exposure. First, the strength of the core concrete must be determined by coring or by a combination of coring and pulse velocity measurements. Next, determine the loss of steel area that has resulted from the corrosion. In order to make this determintion, it is necessary to remove the corrosion products from the reinforcement. This cleaning is required before new concrete is placed around the steel anyway. After the strength of the concrete, the amount of steel and the yield point have been established, it is possible to determine the amount of added reinforcement and concrete needed to bring the column up to required strength. The fact that the original concrete and the reinforcement is stresed under dead load and the new concrete and steel will not be, must be taken into consideration in the design of repair. Steel required to replace that lost from corrosion of reinforcement in addition to the original amount, if required, is tied in place. All added vertical reinforcement should be surrounded by ties spaced and sized to meet applicable code requirements,ignoring ties that remain in the original column after removal of deteriorated concrete. If the elimination of the cause of corrosion, moisture or oxidizing agent cannot be assured, epoxy coated reinforcement may be used to minimize corrosion and extend the life of the structure being repaired.

If the column is long or the amount of replacement concrete represents a considerable percentage of that in the original column or pier, shear ties are recommended. The spacing and size of these drilled-in dowels to assist in holding the original and repair concrete together should be an engineering determintion. Replacement concrete may either be cast and vibrated in place or pneumatically placed. As stated previously, the use of pneumtic concrete should be carefully considered and applied by persons skilled and experienced in the use of this material."

Probably the most striking features of his recommended repair procedures are the extent to which original concrete is removed, the extensive replacement of reinforcement and the care taking in including ties, stirrups and extended lap lengths in the repair designs. From personal observation it would appear that in Australia there is a degree of reluctance to remove concrete to the extent recommended. It is probable that this reluctance is related to extra costs of propping and reforming, but unless such cost problems associated with sound repair methods are faced, repeated repairs will be necessary.

6. REPAIRS ASSOCIATED WITH STEEL CORROSION OF BUILDING FACADES

If the reader hopes to be informed that there is a single, proven, acceptably priced method for the repair and rehabilitaion of building structures suffering from problems of reinforcement corrosion then he should prepare himself for a disappointment, as this paper offers no universal panacea to troubled owners, engineers and architects. The question which must be answered is, "Why is there no set procedure?" There are two answers. The first is that the rusting of reinforcement is cused by many different factors despite the fact that carbonation or the presence of chlorides are often held to be the only reasons. Secondly, the substrate concrete to which the repair must adhere varies significantly in innate characteristics and to different ambient conditions.

6.1 The Influence of Factors Causing Distress

Experience suggests that a host of causes of corrosion are responsible for the observed damage to building facades. There is a significant danger that if the cause of the problem is not fully understood then a repair method will be used which will not improve durability. Consider the case of non-load bearing, pre-cast concrete mullions. These have, in at least one case, been fixed by dowels which have been pressure grouted with expansive mortar formed of a mixture of portland cement, sand, iron fillings and calcium chloride. The expansion of this type of grout never tends to cease, and cracking of the concrete along reinforcing bars results. If now the repair only concerns itself with the reinforcing bars there is no chance that a lasting repair will result. Similarly, if a panel set has been cast such that the reinforcement lies at the interface of veneer and backing concrete, each of which have different properties, local repairs have little chance of ensuring longevity of the panel set. If panels have been acid-dipped, to provide special surface finishes, then this too, may influence decisions with respect to repair or restoration procedures.

6.2 The Influence of Concrete Properties

The hygrothermal properties of concrete are such that they form an ill-matched set when compared with those of most patch materials. The shrinkage of even a shrinkage-compensated mortar patch is different to the dimensional change properties of a substrate concrete. Concrete may shrink on heating due to drying while an epoxy patch expands. Because of these types of problems, detachment of patches is always a possibility. A greater risk however is if depassivating ions such as chlorides remain at bar level and continue the expansive reactions.

Apart from these factors it has been shown by work at The University of Sydney, that patches of almost any type are much more successful on high strength concrete than on concretes of lower grade. This finding is considered to be extremely important as it explains why some inorganic and organic chemical patch materials may work very well when used on, say, a high strength prestressed concrete tank or oil production platform in the North Sea, but may not show satisfactory endurance performance on Sydney building facades. The reason for this difference is that, for lower grade concretes, diffusion continues to occur through the pore or crack system of the concrete at a relatively high rate when compared with movements through high grade concrete. Furthermore, there is often already a build-up of depassivating ions in the concrete of high permeability. These continue to cause corrosion adjacent to the repair, and they are supplemented by continued diffusion.

6.3 Repair Procedures

The Concrete Society Report on Repair of Concrete Damaged by Reinforcement Corrosion [21] was prepared by a working party established by the Concrete Society Materials Steering Group with the assistance of FERFA (Federation of Resin Formulators and Applicators) and the Association of Gunite Contractors. In their foreword they state that "As the repair of concrete can be a difficult task involving many operaions and using specialized materials, frequently in unfavourable circumstances, the working party has concluded that the most useful report it could produce would be one aimed primarily at specifiers who are dealing with repairs for the first time. It is, however, hoped that others, more experienced in the field, will find it of use as providing an overview of the subject."

The report deals with methods that are currently in common use for the repair of concrete damaged by reinforcement corrosion, and is particularly helpful in providing guidance on contract conditions, specification and measurement. It is less helpful when dealing with local patching, as it is stated: "When local patching rather than overall repair of concrete suffering from chloride-induced corrosion is carried out, further damage may occur in areas close to the repair. The extent and rate of such damage cannot at present be predicted. However, it has been found, in many circumstances, that is is more economical to carry out local patching and accept the need for further work later rather than to do much more extensive work when damage first appears. On theoretical grounds, it seems likely that in these cases the use of resin systems either to create a barrier bonding layer between the old concrete and the repair, or to provide a barrier on the steel surface, will result in less electrochemical interaction between the repair and adjacent original concrete.It could be helpful to record carefully the materials and methods used, so that, when more repairs are carried out later, performance can be judged and compatibility with new materials ensured."

7. APPLICATION OF CATHODIC PROTECTION TO CONCRETE MARINE STRUCTURES AND BRIDGE DECKS.

While there are still substantial technical questions to be answered before cathodic protection of reinforced concrete can be accomplished on a routine basis, it is the only protection method presently available which can be guaranteed to stop reinforcing steel corrosion after it has commenced. It is considered probable that within ten years many of the concrete structures in agressive environments will be cathodiclly protected either at the time of construction or as a restoration procedure.

8. CONCLUSION

Some aspects of the influence of durability on maintenance, repair and restoration of concrete structures have been considered. Lest it be believed from this paper that only concrete is a problem material with respect to durability, the reader is referred to a paper by Manning [22], discussing accelerated corrosion in weathering steel bridges in Canada, a problem also noted in Britian and Germany. Durability problems associated with roofing membranes, timber, plastics and bituminous materials are all noted by Wright and Frohnsdorf [23].

REFERENCES

1. Lever K.R. and Gray C.W., A Durability Study of Concrete Using Monte Carlo Simulation, Proceedings of the Fifth International Symposium on the Chemistry of Cement, Vol. III, Tokyo, 1968.

2. Sentler L., Statistical Evaluation of the Durability of Concrete, 9th FIP Congress, Stockholm, Colloquium C: Predictions of Service Life of Concrete Society Technical Report No. 26, 1982.

3. Chatterji S.K., Mechanisms of Sulphate Expansion of Hardened Cement Pastes, Proceedings of the Fifth International Symposium on the Chemistry of Cement, Vol. III, Tokyo, 1968.

4. Sommerville G., The Design Life of Concrete Structures, Structural Engineer, Vol 64A, No.2, Feb 1986.

5. Clifton J. R., Nondestructive Evaluation in Rehabilitation and Preservation of Concrete and Masonary Materials, ACI Infrastructure Seminar - Rehabilitation of Concrete Structures, 1985.

6. Manning D. G. and Bye D. H., Bridge Deck Rehabilitation Manual - Part 1. : Condition Surveys, Ontario Ministry of Transportation and Communications, Canada, 1983.

7. ACI Committee 345, Routine Maintenance of Bridges, Report No. ACI 345.1R-83, 1983.

8. ACI Committee 546, Guide for Repair of Concrete Bridge Superstructures, Report No. ACI 546.1R-80, 1980.

9. Transportation Research Board, Assessment of Deficiencies and Preservation of Bridge Sub-Structures Below the Waterline, National Cooperative Highway Research Programme Report 251, National Research Council, Washington, D.C., Oct.1984.11.

10. Rasheeduzzafar D.F.H. and Al-Gahtani A.S., Deterioration of Concrete Structures in the Middle East, American Concrete Institute Journal, Vol. 81, No. 1, Jan..-Feb.1984.

11. Fookes P.G., Pollock D.J. and Kay E. A. Middle East Concrete (2), Rates of Deterioration, Concrete, Vol.15, No.9, Sept.1981.

12. Fookes P.G., Comberbach C.D. and Cann J., Field Investigation of Concrete Structures in South-West England, Parts I and II, Concrete, Vol. 17, No. 3, March 1983, and Vol. 17, No. 4, April 1983.

13. McCullough, B.F. and Treybig, H.J., Condition Survey of Continuously Reinforced Concrete Pavements in North Central United States, 54th Annual Meeting, Transportation Research Board National Research Council, Transportation Research Record 572, Pavement Design, Performance and Rehabilitation, Washington, D.C., 1976.

14. Carrier R.E. and Cady P.D. Factors Affecting the Durability of Concrete Bridge Decks, ACI SP-47, Detroit, 1975.

15. Roper, H., Baweja, D. and Kirkby, G.A., Towards a Quantitative Measure of the Durability of Concrete Structural Members, Proceedings, International Conference on In Situ/Non Destructive Testing of Concrete, American Concrete Institute and Canada Centre for Mineral and Energy Technology, SP 82-32, 1984.

16. Roper, H., Baweja, D. and Kirkby, G.A., Durability - a Quantitative Approach, Concrete 85 - The Performance of Concrete and Masonry Structures, Institution of Engineers, Australia, Brisbane, 1985.

17. Ho, D.W.S. and Lewis, R.K., The Effects of Fly Ash and Water Reducing Agents on the Durability of Concrete, Commonwealth Scientific and Industrial Research Organisation, Division of Building Research, Melbourne, Australia, Research Report, 1981.

18. Ho, D.W.S. and Lewis, R.K., Carbonation of Concrente Incorporating Fly Ash or a Chemical Admixture, First International Conference on the Use of Fly Ash, Silica Fume, Slag and Other Mineral By-Products in Concrete, Montebello, Canada, 1983.

19. Roper, H., The Future of Concrete - Based on Jumping to Conclusions or Learning from Experience?, 12th Biennial Conference, Concrete Institute of Australia, Melbourne, 1985.

20. Janney, J.R., Maintenance, Repair and Demolition of Structures, Chapter 25, Handbook of Structural Concrete, Kong, Evans, Cohen and Roll Eds., Pitman Publishers, 1983.

21. The Concrete Society, Repair of Concrete Damaged by Reinforcement Corrosion, Concrete Society Technical Report No. 26, 1984.

22. Manning D.G. Accelerated Corrosion in Weathering Steel Bridges, Canadian Structural Engineering Conference, 1984.

23. Wright J.R. and Frohnsdorff G., Durability of Building Materials. Materiaux et Constructions Vol. 18 No. 105, 1985.