Maintenance programme of Shinkansen structures

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Programme d'entretien des ouvrages d'art du Shinkansen

Wartungsprogramm für die Shinkansen-Bausubstanz

Katsutoshi ISOURA
Civil Engineer
Central Japan Railway Co.
Tokyo, Japan

Katsutoshi Isoura, Civil Engineer: Born in 1943. Graduated from Kyoto University. Since he joined the company in 1966, he has been engaged in railway construction and maintanance. At present, he is a General Manager of Tracks and Structures Department.

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.

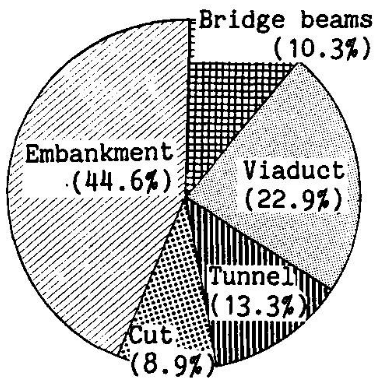


Fig. 1 Ratio of structures

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

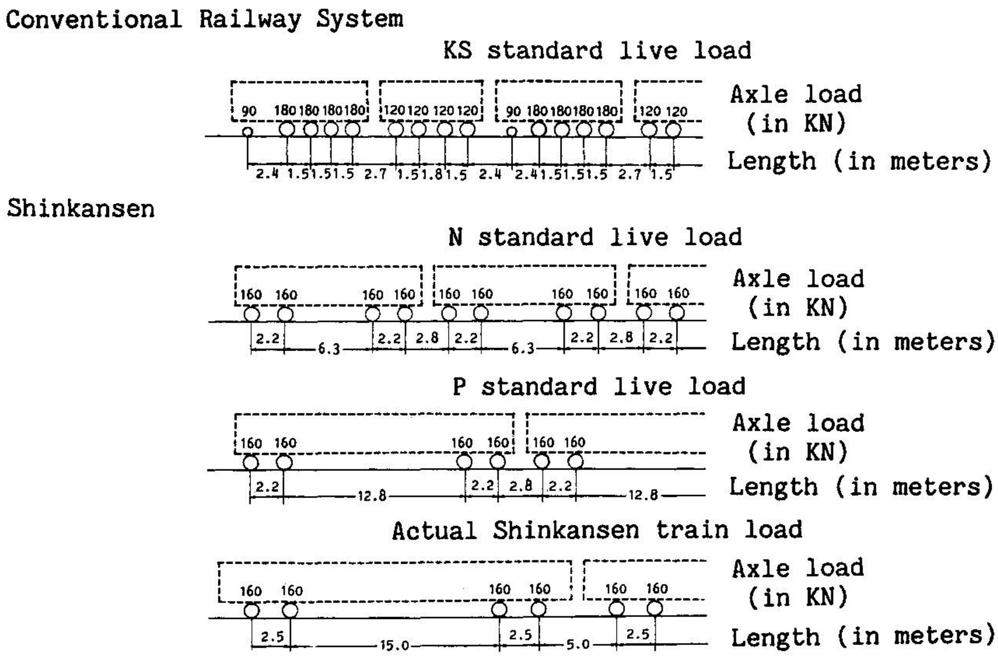


Fig. 2 Design live load

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.

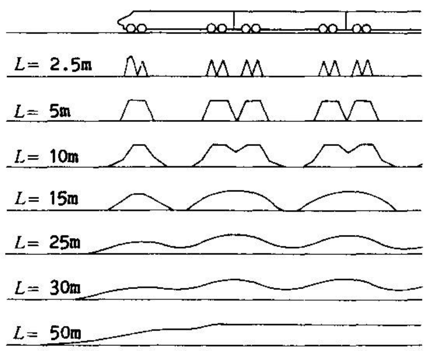


Fig. 3 Stringer stress waveform under train operation

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

	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

4.3 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.

(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.

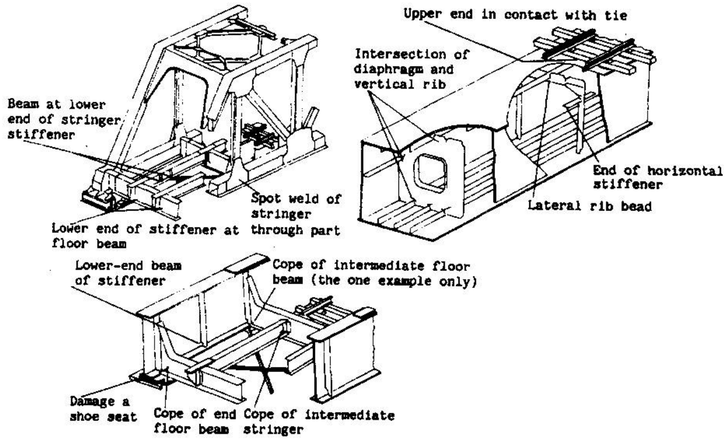


Fig. 4 Outline of damage

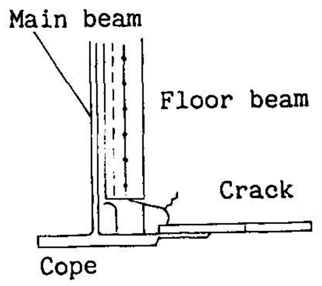
5.2 Coped Floor Beam of Through Type Girder Bridge

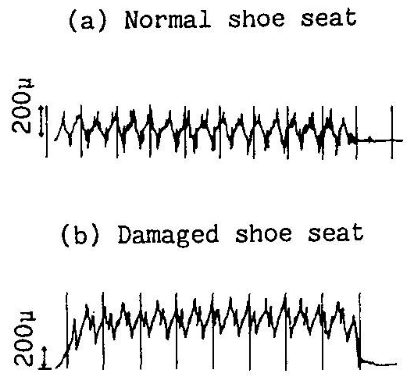
This damage is primarily caused by the settlement of bearing, therefore occurrences are significant in end floor beams. Causes and retrofitting methods were studied through various measurements on actual bridges, structural analysis and fatigue tests. The result described below were obtained.

(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.





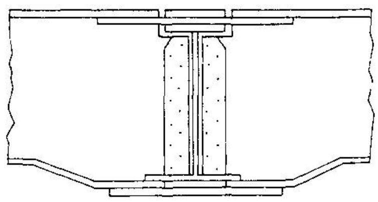


Fig. 5 Crack at floor beam cope

Fig. 6 Actually-measured strain waveform of floor beam

Fig. 7 Improved details of floor beam

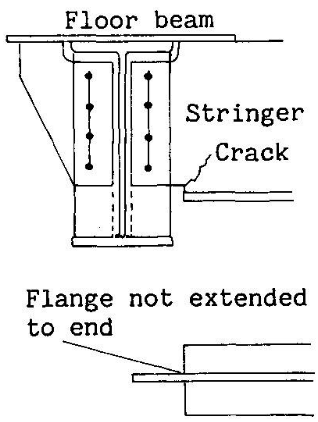
5.3 Cope of Stringer

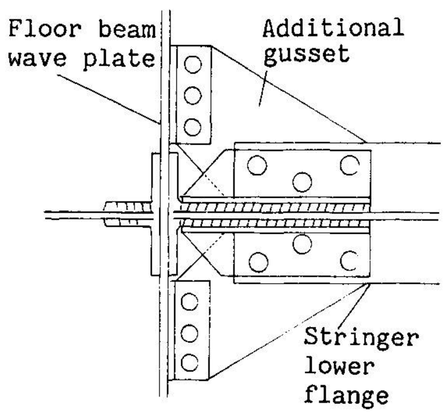
As shown in Fig. 8, this damage is a crack initiated at the stringer web plate of the through plate girder or the through truss bridge.

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.





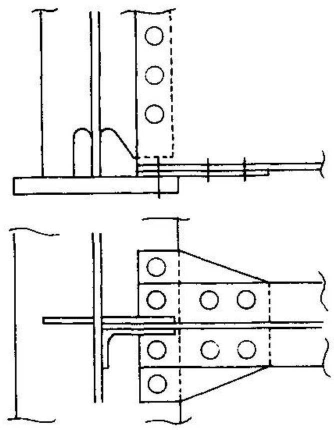


Fig. 8 Crack at stringer web plate

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.

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.

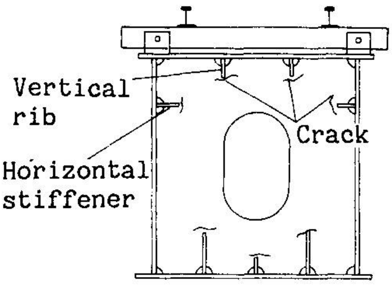


Fig. 11 Crack of box-section diaphragm

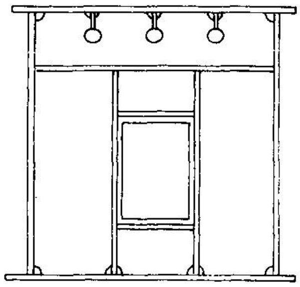


Fig. 12 Improved diaphragm

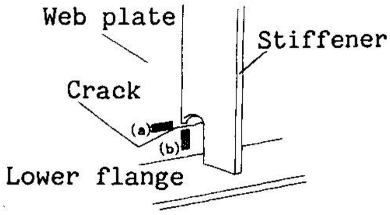
5.5 Vertical Stiffener in Web Plate

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

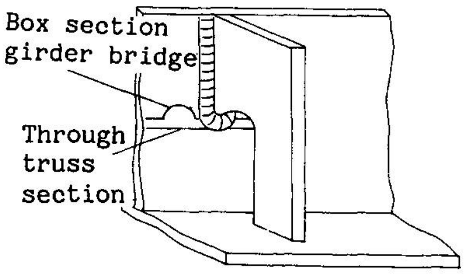
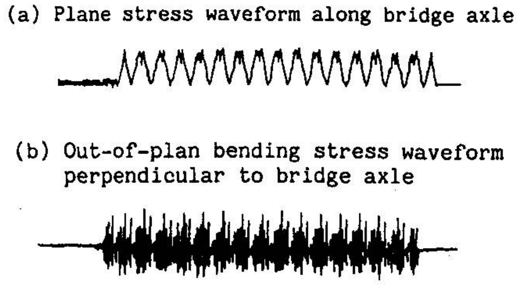


Fig. 14 Progress of cracking

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.



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

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.

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.

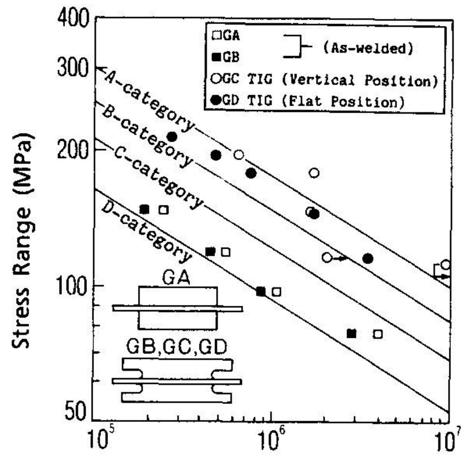


Fig. 16 Results of fatigue test

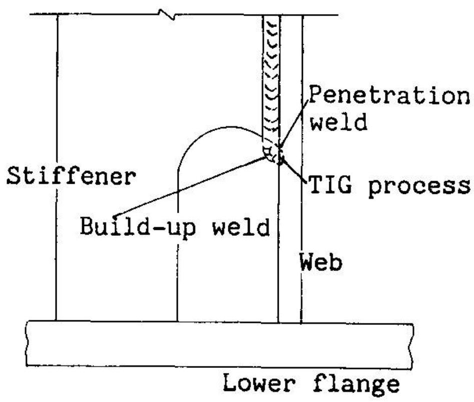


Fig. 17 TIG welding process

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.

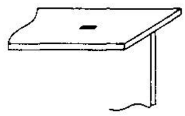
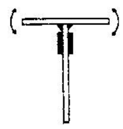
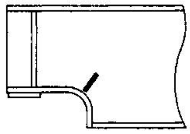
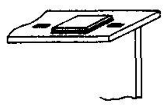
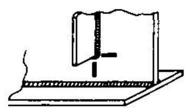
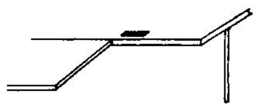
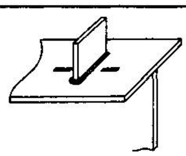
Type of structual joint	Standard allowable fatigue stress at 2×10^6 cycles (MPa)	Slop	Ellustration example and locating of strain gages
Base metal and weld metal in members connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied 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.

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