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GUIDELINES FOR REPAIR AND REHABILITATION OF STEEL BRIDGES

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FOR REPAIR AND REHABILITATION
OF STEEL BRIDGES

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GUIDELINES FOR REPAIR AND REHABILITATION OF STEEL BRIDGES

1. INTRODUCTION

1.1. The Steel and Composite Structures Committee (B-5) of the Indian Roads Congress was reconstituted in 2006 with the following personnel:

Ghoshal, A.	*Convenor*
T.K. Bandyopadhyay, Dr.	*Co-Convenor*
Ghosh, U.K.	*Member-Secretary*

Members

B.P. Bagish, Dr.
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Ex-officio Members

President, IRC
DG(RD) MOSRT&H
Secretary General, IRC

1.2. At its first meeting held on 30th April, 2003, the former Steel Bridges Committee (B-7) felt that in the light of renewed interest in Steel and Composite Highway Bridges and Flyovers, there is a need to bring out separate documents for different types of superstruture and also for strengthening/rehabilitation of old Steel Bridges which are still in service. Since the IRC document "Guidelines on Techniques for Strengthening and Rehabilitation of Bridges" (IRC:SP:40) does not comprehensively cover steel bridges, the Committee felt that there is a need to bring out a sperate document entitled "Guidelines for Repair and Rehabilitation of Steel Bridges". While highlighting the special requirements for design and fabrication, it was decided that the Guidelines would generally be in line with relevant IRC Codes and Special Publications. Additional inputs from pertinent AASHTO Guide Specifications and Manuals, NCHRP Reports, RDSO Guidelines and text book were also considered in the preparation of the Guidelines.

1.3 The draft of the Guidelines was prepared by a Sub-committee comprising of the following members:

S/Sh. A. Ghoshal	Convenor
U.K. Ghosh	Member
Dr. T.K. Bandhyopadhyay	Member
K.N. Sreenivasa	Member
Dr. B.P. Bagish	Member
R.K. Goel	Member

The former B-7 Committee under the Convenorship of Shri P.B. Vijay in its meeting held on 12th December, 2005 had finalized the draft Guidelines inviting comments, if any. The draft was referred to the newly constituted Bridges Repair and Rehabilitation Committee (B-8) by the IRC for its comments, keeping in view IRC:SP:40. It was vetted by the B-8 Committee, with minor modifications in its meeting held on 11th March 2006. The newly constituted Steel and Composite Structures Committee (B-5) in its meeting held on 9th May, 2006 endorsed the modified draft and recommended for its placement before the Council through BS&S Committee.

The draft document was approved by the Bridges Specifications and Standards Committee in its meeting held on 19th October, 2006 and the Executive Committee authorized Secretary General, IRC to place the same before Council, The document was approved by the IRC Council in its 179th meeting held on 18th November, 2006 in Panchkula subject to incorporation of some suggestions.

The B-5 Committee considered the views of the Council in its meeting held on 9th March, 2007 and felt that the suggestions were already in place in the document and recommended that the document could be published.

2. SCOPE

The topics covered in the present document aim at restoring the bridges to their originally intended service level or to retrofit them upto capacity presently required. Inadequacies could be due to various reasons:

(a) Deterioration due to various causes e.g., corrosion, cracks, buckling etc.

(b) Inherent structural deficiency

(c) Deficiency due to introduction of new loading and/or design criteria

(d) Geometrical inadequacy due to changed traffic demand.

(e) Inadequate design (loading standard)

(f) Changes (increases) in magnitude of loading, e.g., earthquake.

Assessment of residual fatigue life and rating of existing steel bridges form part of maintenance activities and are not covered in the present publication. It also does not cover replacement of an entire bridge, nor a new construction.

For rating and posting of bridges reference is made to IRC : SP:37.

3. NATURE OF INADEQUACIES

3.1 DETERIORATION

Deterioration in steel bridges can be classified according to two broad causative factors, viz, natural deterioration and deterioration due to man-made situations. Examples of the former are those caused by atmospheric corrosion, earthquake, floods, fire etc. Deterioration caused by pollution, stress corrosion, fatigue, deficiencies in material characteristics, foundation settlement, accident, war, terrorist attack etc. come under man-made situations.

In most of these situations, the effect of distress depends on the type of the bridge, details adopted, quality of the structure, type of environment and above all, the level of routine maintenance work.

3.1.1 Atmospheric corrosion

Atmospheric corrosion in steel is essentially an electrochemical process of flow of electricity and consequent chemical changes. Two important points are to be noted in this connection:

- Corrosion occurs only under concurrent presence of water and oxygen. In the absence of either, corrosion cannot occur.

- All the corrosion occurs at the steel metal itself and not at the surface mill scales.

The immediate or direct effect of atmospheric corrosion is loss of area of either the steel member itself or the fasteners, causing increase in stress in the member or the fasteners. Indirectly, this makes the member as well as the fasteners vulnerable to stress corrosion and fatigue failure.

3.1.2. Stress Corrosion

Locations subjected to high tensile stress are prone to higher rate of corrosion. This phenomenon is commonly referred to as ‘stress corrosion’ As the cross sectional area of an already highly stressed member is reduced due to corrosion, the resultant increase in stress may initiate crack. This type of distress is found mostly in specific areas where high concentration of stress is developed, such as eye bars of pins in suspension and cable stayed bridges.

3.1.3 Brittle fracture

Brittle fracture is characterised by a low stress fracture of the material, which usually occurs suddenly with little or no plastic deformation and other warning signs.

There are three key factors which lead to brittle fracture. These are :

– **Metallurgical feature** : Depending primarily on chemical compositions which affect Carbon Equivalent and grain size and heat or mechanical treatments, some steels are less ductile than others and are therefore more susceptible to brittle fracture.

– **Temperature** : Brittleness of steel increases with lowering of ambient temperature below a point when ductility transition occurs. Geographical location of a bridge is thus very important for examining this aspect. The earlier steels were more susceptible to brittleness.

– **Service conditions** : Brittle fracture can occur at low service stresses as a result of certain distribution pattern of force fields, such as locations of stress concentrations due to abrupt change of sections, notches, holes, weld defects etc. Also due to complex stress situations in thicker plates, these plates carry higher risk of brittle fracture.

3.1.4 Lamellar tearing

Lamellar tearing is the separation of parent metal caused by 'Through thickness' strains induced by weld metal shrinkage. When the resultant stress is carried in the 'through thickness' direction, any lack of cohesive strength of the steel material in this direction causes the plate to be separated. Shape of non-metallic matter in the steel (manufacturing defects) as well as factors related to welding process (e.g. Preheat, weld restraint in the through thickness direction etc.) are contributory factors for lamellar tearing of steel.

3.1.5 Fatigue Cracking

In bridges, steel elements are subjected to moving loads, which cause fluctuations of stresses in steel elements. This fluctuation of stresses reduces the ultimate strength of steel member considerably as compared to gradually applied static load. Thus, a member may be able to withstand a single application of the design load, but may fail if the same load is repeated for a large number of times. This phenomenon of progressive localised permanent structural change due to fluctuating stresses, which may initiate cracks in the member, is termed 'fatigue'. This reduction in strength depends on two factors, viz., number of local repetitions (cycles) and range of stress due to these load repetitions . Fatigue cracks occur at the tension zone of the members. This tension zone varies from member to member or within the same member, depending upon application of moving loads. Also, this phenomenon is applicable to connections/ joints which are subjected to load repetitions.

In welded joints, the fatigue strength of steel tends to be reduced due to pronounced changes in the structure (hard grain formation) in and around Heat Affected Zone (HAZ) and properties (lowering of ductility) of the steel due to improper or no treatment of HAZ. As a result, welded bridges are more prone to fatigue cracks than riveted/High

Strength Friction Grip (HSFG) bolted ones. Also, the crack developed at the weld tends to progress and may affect both the connecting components and surrounding members/ elements/or connectors (due to increase in stresses) and consequently, the entire structure may be damaged.

In road bridges, the range of stresses is not high, because of lighter moving loads compared to dead loads and less vibration as compared to railway bridges. Thus fatigue related distresses directly due to range of stresses are not very common in road bridges. However, stress raisers in tension zone, such as sharp notches or corners, abrupt change in cross sections, can cause stress concentration. Also atmospheric corrosion in many cases, reduces cross sections of members, resulting in increase in stress levels, which for a particular loading cycle may initiate cracking leading to fracture.

3.1.6 Damages due to accidents, floods, earthquakes, etc.

Physical damages (buckling) of different bridge components due to accidents are quite common. Steel bridges spanning across roadways underneath and having inadequate headroom are very often damaged due to vehicular collision from underneath contact. In cases of through and semi-through type steel bridges, vehicles using the bridge may damage individual members while passing. Also there are many examples of vessels using waterways and colliding with bridge structures from underneath.

Bridge may be damaged by natural calamities such as floods, landslides, earthquakes or by explosions from war action, sabotage etc.

For steel bridges supported by non-ductile substructure elements, at these locations supertructure has the likelihood of getting damaged mostly in the form of buckling and/or connection fracture of diaphragm braces.

Steel sections can suffer slow erosion due to weathering action, viz., sand storm, wave action etc.

In industrial areas, chemical corrosion may occur due to presence of chemicals (chlorides, oxides of sulphur etc.) in the atmosphere.

All the above situations need to be appropriately examined for requisite redressal.

3.2 Other Inadequacies

3.2.1 Inherent Structural deficiency

A bridge structure may suffer inadequacy due to under design or faulty construction, needing strengthening.

3.2.2 New loading or design criteria

It is quite common that heavier loading standards and changed codal requirements based on improved knowledge are introduced from time to time, which may render the bridge structure inadequate, needing strengthening.

3.2.3 Geometrical inadequacy

It is sometimes necessary to introduce increased vehicle clearance requirements to meet new traffic demands, such as introduction of new types of vehicles, container services etc. This situation mostly affects through type bridges, necessitating modifications in structural arrangements.

4 REPAIR AND REHABILITATION PROCESS

The broad activities in this process are

1. Study of the bridge and its environment.
2. Locating damages/ defects/ deficiencies by inspection.
3. Calculation of stress level, residual

stress capacity and residual life

4. Evaluation of results after analysis of calculation.
5. Design for rehabilitation.
6. Preparation of drawings and specifications.
7. Fabrication.
8. Erection.

These activities are briefly discussed in the following paragraphs.

4.1 Study of the Bridge and its Environment

4.1.1 History of the bridge

This activity involves study of available records and drawings pertaining to the concerned bridge. If adequate records and drawings are not readily available, interviewing old employees of the organisation or persons residing in the vicinity of the site of the bridge may provide some valuable information. The data of construction and history of subsequent repair work or replacement of major members should be ascertained at this stage.

The date of construction provides the vital information about the age of the bridge. Knowledge of age has important bearings. Some of these are :

(a) This may give some idea about the material used in the construction and influence whether a particular type of repair work will be feasible or not. For example, material with high carbon or silicon content would preclude any repair work by welding.

(b) Age may give an idea about the loadings and stresses considered for the original design on the basis of the codes of practices prevalent at the time of construction.

(c) Age and historic data may give an idea about the number and magnitude of load cycles the bridge has been subjected to and thereby the proneness to fatigue failure can be assessed.

4.1.2 Environment

Review of the environment covers the effect of the environment on the existing bridge, as also the effect of the rehabilitation work on the environment.

Regarding the former, the following situations are relevant :

(a) Low clearance between the lowest point of the bridge and the highest flood level (HFL).

(b) Presence of water spray or moisture in the vicinity such as waterfall or marshy wet land.

(c) Presence of industrial units nearby, which emit corrosive fumes or discharge chemical effluents.

(d) Presence of salinity in the atmosphere.

The rehabilitation scheme should consider these environmental hazards and recommend appropriate protective measures.

As regards the effect of rehabilitation work on the existing environment, dumping of debris, release of chemicals, spills of waste materials should be avoided. These aspects should be considered at the planning stage and proper specification should be incorporated in the bridge rehabilitation document itself.

4.2 Locating damages/ defects/ deficiencies by inspection

4.2.1 General

For effective rehabilitation of a bridge, the first step is to locate the damages/ defects/ deficiencies incurred by its various components. For this purpose a Special Inspection is carried out. This inspection comprises of the following main activities :

(a) Measurement of the overall dimensions of the existing structure including sections and thickness of all main members.
(b) Detection of defects in or deterioration of different members.
(c) Examination of joints and fastenings
(d) Observation of the behaviour of members and cracks when subjected to vibratory loads.

4.2.2 Inspection personnel

Ideally the structural designer who is entrusted to develop the rehabilitation scheme should participate as a member of the inspection team. This would enable him to understand the condition of the structure, location and extent of the damaged area much better than going through the report pages prepared by someone else, and thus formulate a better rehabilitation strategy. It is important to note that certain components of a bridge are highly inaccessible. Therefore, reliable and efficient working hands are to be included in the inspection team to carry out the inspection of such inaccessible areas. When it is not possible for the designer to inspect the bridge, the inspection report assumes much higher importance, as the designer has to fully depend on this report for developing his strategy for rehabilitation.

The bridge inspector should, therefore, be conversant with the behaviour of the structure under actual loading conditions. He should be conversant with the design and construction features of the structure. Deterioration of material due to corrosion, weathering, fatigue etc., should be easily apparent to him. He should be able to identify the areas which are prone to deterioration. He should be able to properly interpret what is observed and report the same correctly in clear language and by simple sketches, if necessary.

It is often preferable to form an inspection team comprising of engineers and technicians, with experience and knowledge of diverse areas such as structural design, construction, maintenance, emergency repair etc. Assistance of specialist agencies may also be sought to help the inspection team for special structures such as movable bridges, suspension bridges, cable stayed bridges etc.

4.2.3 Areas to be inspected

While all the components of a bridge need inspection, there are certain areas, connections and splices which are susceptible to serious defects, and therefore need particular attention during inspection. Some of these are :

- Members with high design stresses or subjected to reversal of stresses
- Members and joints where water may get collected due to inappropriate drainage system
- Joints which are not easily accessible to painting
- Surfaces of the windward side of the bridge situated near the sea
- Notches in flange/ web at the ends of stringers (for fatigue cracks)
- Web members of through and semi-through truss members for damage due to collision from vehicular traffic
- Camber of trusses
- Members which are closed type like "box channels" etc., where the inner side cannot be painted
- Fatigue crack prone areas of welded girders such as end of cover-plates in flanges with multiple plates, butt-welded connections of flange plates of different thicknesses, welded splices, fillet welds across the direction of tensile stress, intersecting welds, welds at ends of transverse and longitudinal stiffeners and welds of attachments to web and tension flange or tension members, bracing and cross frame connections with stiffeners, etc.

4.2.4 Inspection tools

Some of the most useful inspection tools are : a 2m pocket tape, a 30m steel tape, chipping hammer, paint scraper, wire brush, plumb bob, vernier or jaw type callipers, small level, steel straight edge, feeler gauges, spanners, wrenches, rivet testing hammer, calibrated torque measuring wrench for HSFG bolts, magnifying glass of (10X or higher magnification), binocular, flash light, sensitive thermometer, mirror, piano wire and camera. Precision type levelling instrument and theodolite for checking camber/ deflection sway etc. may also be arranged if necessary.

4.2.5 Inspection Equipment

For inspection of structural elements situated above deck level, simple equipment such as ladders, portable platforms, planks etc. may be used for inspection. However, in bridges where underside structures are not easily accessible, temporary scaffolding system, special equipment such as bucket snoopers, custom made travelling gantries etc. are required to be used. Where access from roadway below is available, truck mounted hydraulically operated telescopic hoists fitted with bucket or platform may be usefully employed.

4.2.6 Visual Inspection

As a first step the bridge structure should be visually inspected. Most cracks are first detected during the visual inspection. Visual inspection is carried out by naked eye, or by using binoculars from a convenient location.

Usual and most reliable sign for detection of cracks during visual inspection is the oxide or rust stains that develop at the crack after the paint film has cracked. At this level of inspection only larger cracks will be detected.

In the next step, close visual inspection of critical locations and suspect details (that show no visual evidence of cracking through paint film) should be done using magnifying glass of 10X or higher magnification. Cracks detected during earlier inspections should also be inspected in detail for their extension. It may be necessary to remove the paint film for the inspection; however, this should be done carefully so that any fine crack remains intact for detection.

4.2.7 Non-destructive Testing (NDT) methods

For further detailed inspection, non-destructive testing (NDT) methods are generally employed. Some of the common methods are briefly described below :

(a) **Thickness measurement :**

The remaining thickness of a corroded member can be measured with the help of callipers, where access is available on both sides of the member. Where such access is not available, ultrasonic thickness gauge can be used. The equipment is very handy and can measure the thickness from any one surface to an accuracy of 0.1mm. The gauge usually gives a digital reading.

(b) **Crack testing**

Cracks in steel can be detected by using several non-destructive tests. Some of these are described below :

Dye penetration test :

This test is a simple and low cost non-destructive testing for detecting minute surface cracks.

First the surface area is to be cleaned to remove any dirt, rust or paint, to enable dye penetrant to enter into the crack. The dye penetrant is then applied over the surface by spraying or brushing. The dye seeps into any cracks or other defects open to the surface. After allowing a penetration time of about 20 minutes, the excess penetrant is cleaned by using a solvent. A developer (like a chalk powder) of contrasting colour with high absorbent quality is then applied by dusting. In case of any surface defect, the dye penetrant is drawn out from the crack by blotting action of the developer and appears as stain on the chalk surface. Sufficient time is to be allowed for the dye penetrant to blot on the developer. The surface is then examined by using a magnifying glass. The surface is to be finally cleaned after the test (Ref: IS:3658:1981).

Ultrasonic Test :

This method is suitable for detecting surface or sub-surface defects in steel. A high frequency sound beam is introduced into the area to be tested by means of an ultrasonic transducer. The sound beam travels through the steel, and as soon as the crack is met, it reflects back to the transducer. This produces a voltage impulse, which appears in the cathode ray tube (CRT). In this test access from only one side of the material is required. Since portable testing machines are available, this test can be conveniently carried out at a bridge site. However, this test requires specialised skill in interpreting the pulse-echo pattern appearing on the screen (Ref: IS-3664:1981 and IS-4260:1986).

Radiographic examination :

In this method both surface and sub-surface defects can be detected. X-rays or Gamma rays are passed through the member to be tested, which create an image on a photosensitive film. The defects are shown on the film as dark lines of shaded areas. In this method, permanent record of every test is available. Permanent record can be made available through radiographic examination as compared to ultrasonic testing method. The former can be considered more reliable. However, for radiographic examination access from both sides of the test area is required - with radiation source placed on one side and the film placed on the other side. It requires specialised skill in performing this test (Ref: IS-1182:1983). Also, strict safety regulations are required to be observed during this examination. For protection against radioactivity refer IS:2598-1966.

Holography

Holography is a variant laser technique used to obtain 3D images and is used as a tool for non-destructive testing of materials. The detection of defects can be observed at the micro level. This is an effective tool for testing in highly localised zone and is quite effective in detection of the location of flaw.

Magnetic Particle Test :

This test is suitable for detection of surface or sub-surface cracks. In this method magnetic field is first set up in the member to be inspected by means of an electric power source or permanent magnet. Fine dry iron particles are then dusted on the test area. Alternatively, liquid detection medium carrying magnetic iron powder can also be used. Crack causes discontinuities in magnetic field which results in a pattern of collection of iron particles along the crack and the outline of the crack becomes clearly visible. A highly trained inspector is required to perform this test successfully. The method is only effective in a limited range of situations and is normally not popular for use in field conditions (Ref: IS:3703:1980 and IS:5334:1981).

4.2.8 Testing for physical and chemical properties

Sometimes, it is considered necessary to carry out tests to establish the physical and chemical properties of certain members of a bridge. For example in cases of requirement of welding repairs, the selection of electrode would depend upon the chemical composition of the steel. For this purpose a specimen (coupon) is taken from the steel structure itself. However, these specimens should not be taken indiscriminately from the main members. These members should first be checked by the designer vis-à-vis strength and stability. In case a coupon is obtained from a main member, appropriate bolted repair fulfilling the equivalent sectional requirement at a particular section detail should be designed and introduced.

4.2.9 Field load test and instrumentation

Sometimes, the static behaviour of a bridge is examined experimentally by applying actual or simulated design loadings and observing the effects on the critical members by instrumentation. Prior to loading the structure, strain gauges are fixed at critical locations. The structure is then loaded by placing trucks and/or train of wheel loads of known weights satisfying the requirements of relevant IRC code at various points of the bridge and the strain is recorded. Based on the strains at different locations the actual stresses in the members are calculated and compared with the theoretical allowable design stresses.

Testing may also be carried out by passing test vehicles over a bridge at incremental speeds to ascertain the overall behaviour of the structure under normal traffic load. Vibrations, opening of cracks in damaged members, behaviour of bearings are some of the features that may be observed during such tests. Measurements can be taken by various types of instruments including electronic and laser techniques etc.

4.2.10 Safety aspects

During inspection, top priority must be given to safety. It is necessary to draw out a comprehensive safety programme well in advance of the proposed inspection activity. This programme should cover the safety and welfare of the persons at work as well as members of the public against risks to accident, health and safety arising out of the activities at work. The programme should include standard bridge inspection safety procedures of the concerned bridge authorities, as well as additional safety requirements such as traffic control procedures to conform to local regulations. Safety vests, helmets, work boots should be used by every member of the inspection team. Where climbing is required, suitable safety belts should be used. Special precautions should be taken for night time work. A first aid box should accompany the inspection team.

4.2.11 Photography

Clear and sharp photographs are very useful documents to support bridge inspection report. Thus modern cameras fitted with wide angle and telescopic lenses are very useful during inspection. It is advisable to include in the photograph a clearly marked scale or an easily recognisable item for easy comprehension of the scale of the detail.

4.3 Calculation of stress level and residual stress capacity

Based on the results of the inspection, calculation of stress levels for all critical members of the bridge structure should be carried out in respect of both dead and live loads. The dead loads should include the estimated load for additional materials for repair and strengthening. Calculation of stress levels would enable the designer to assess the residual capacities of individual members and joints that are available for live loads and other incidental loads and compare these with the actual load effects on these members and joints. This would help to identify the members and joints which are deficient and need strengthening.

To assess the capacity of joints by calculations is more difficult since the inspectable parts of the joint are restricted and also the design of joints mostly give rise to stress concentrations and plastic (unknown) redistribution of load between the various connectors (i.e. rivets, bolts, welds). However, obvious deficiencies can be studied for their effect on the capacity of the joint and some measures to overcome defects can be considered, if it can be reliably executed.

4.4 Design for Rehabilitation

This activity is carried out broadly in two stages, viz., concept stage and design stage.

4.4.1 Concept stage

During this stage the various options for solution are considered in detail. A few relevant points need to be considered in this context :

- It is necessary to have a clear understanding of the cause of the problem in order to arrive at a workable solution.

- Practical aspect of implementation needs to be considered. Also, a solution using standard equipment is preferable to one, which needs special equipment.

- Rehabilitation work is generally labour intensive. Thus, solutions, which can be implemented easily by using minimum specialist labour even at the cost of higher material input, are almost always the most economic solution.

- In order to reduce the down time to the minimum, partially fabricated units (to be assembled and fixed at site) should be considered in preference to fabrication in situ.

- All above shall be in conformity to the provisions in IRC:24-2001

4.4.2 Design Stage

Having identified a few viable schemes these are subjected to rigorous analysis and design work in order to finalise the strategy for rehabilitation. A few relevant aspects are discussed hereunder :

(i) **Dead load stresses**

Members of an erected bridge are already subjected to dead load effects. Therefore prior to undertaking rehabilitation work, the structure should be relieved of the dead load. If this is not done, the existing members will continue to carry the dead load and will already be stressed to the extent of dead load effect. Consequently, the capacity of the new material will remain underutilised, as this cannot reach the permissible stress level without overstressing the existing members. In case it is not practicable to relieve the dead load, the new material should be considered to carry the live loads only.

There are a few methods for relieving the dead load stresses of an existing bridge. The most common method is to jack up the girder at a few locations and provide temporary support underneath.

Provision of temporary or permanent external prestress is also a very efficient method of relieving effect of D.L.. For cases with large heights and also over perennial rivers the external prestressing for rehabilitation of bridges has definite advantages.

(ii) **Redundancy and fracture critical member**

A redundant structure has, within itself, a multiple load carrying mechanism, so that, if one mechanism fails or weakens, the load will be carried by another mechanism. A non-redundant structure, on the other hand, does not have multiple load carrying mechanism and consequently failure of a single element (fracture critical member) may cause collapse of the structure.

The failure of any joint will have

similar effect. The joints are more difficult for inspections and repair. Hence, providing alternative paths for load transfer is an important consideration in evolving rehabilitation strategy.

(iii) **Fatigue effect**

Fatigue effect should be considered while developing the rehabilitation details. Some of the details which need particular attention include stress raisers in tension zone such as notches, sharp corners, sudden changes in cross sections which cause stress concentration etc.

While developing welded details for rehabilitation, the following recommendations would help in reducing fatigue related cracks:

- Butt welds are preferable to fillet welds
- Fillet welds across the direction of stress should be avoided
- Intersections of longitudinal and transverse welds should be avoided

(iv) **Connections**

New fasteners should be compatible with the existing fastening system . As far as possible welding should be avoided on existing riveted/bolted connections. If used, welding should be designed to transfer entire load. However, before selecting welding as an option, weldability of the parent material must be ascertained first.

Defective rivets are best replaced by turned and fitted bolts, as the load transfer behaviour of these bolts are nearly identical to that of rivets. If, However, High Strength Friction Grip (HSFG) bolts are used the efficacy of the existing rivets should be checked.

(v) **Eccentricity**

While adding new materials for strengthening an existing damaged member, care should be taken to ensure that the centre of gravity of the strengthened section coincides, as far as possible, with the centre of gravity of the original section, in order to avoid secondary stresses due to eccentricity. In case it is not possible to achieve this requirement, the effect of eccentricity should be considered in the design.

4.5 Drawings and Specifications

Drawings and specifications for rehabilitation scheme prepared by the engineer should be clear and unambiguous. All details as also sequence of proposed operation at site should be clearly indicated in the drawings and specifications. The working drawings prepared by the contractor should be based on the engineer's conceptual design drawings, but must follow the actual measurements at site. The drawings should clearly indicate the scope of work for inclusion of new element and/or deletion of existing elements satisfying the design requirements. Also, drawings should clearly specify and demarcate the existing and new elements of the structure.

4.6 Workmanship

Fabrication and erection of every part of the work should be done most accurately, so that the parts fit properly together on erection. Flame cutting and/ or dismantling of existing members should be done carefully so as not to damage the adjacent steel work. While dismantling existing members, adequate temporary supports should be provided to ensure stability of the entire structure. Care should be taken to eliminate any differential settlement between temporary supports.

4.7 Implementation at site

Implementation of a rehabilitation scheme for an existing bridge is mostly a time bound project. Therefore, prior detailed planning and proper monitoring during implementation are imperative for successful completion of such a project.

In a rehabilitation work, certain problems not envisaged earlier, may arise at site and the supervisory team at site is often called upon to solve such problems on the spot. The team at site should therefore be well equipped to meet such contingencies.

5. REMEDIAL SOLUTIONS FOR COMMON INADEQUACIES

This section briefly deals with suggested remedial solutions for inadequacies that commonly occur in existing steel bridges. The items covered are :

(a) Repair of deficient members

- Corrosion
- Cracks
- Buckling and bending

(b) Upgrading the structure for increased loading:

- Stepwise Strengthening of deficient components
- (A useful and economic strategy to follow is strengthen the weakest member to raise the overall capacity to that of 2nd Weakest member's and proceed in this manner to 3rd, 4th. etc. weakest members)
- Introduction of supplementary members
- Reduction of dead load
- Modification of structural system

(c) Modification in the structure for increase of clearance dimensions.

5.1 Repair of Deficient Members

5.1.1 Corrosion

Corrosion is the most common cause of damage in existing steel bridges. The solution for rehabilitation of a corrosion-damaged member depends primarily on the degree of corrosion and its extent over the surface area. Some typical solutions for damage due to corrosion are described below:

(a) When top flange of a beam or girder is damaged due to corrosion, a steel plate of adequate size is provided over the top flange and secured to top flange by bolts, using the holes of the existing rivets after reaming as necessary. Use of high strength turned and fitted bolts are generally preferred over other types of bolts, since these bolts can be made to fit tightly (slip free) to match the existing rivet holes and thus their behaviour pattern becomes somewhat identical to that of the other adjoining existing rivets. This ensures satisfactory transmission of forces to the additional material.

(b) Web plates of a beam or girder damaged due to corrosion can be repaired by fixing corrosion plates of adequate size, preferably on both sides of the web and securing these by high strength turned and fitted bolts. The details of the arrangement will depend on the location, extent and degree of the damage.

(c) Corroded secondary members, such as lateral bracings, can be repaired by fixing corrosion plates on the bracing angles at the location of damage by means of bolts. However, when a bracing member is badly corroded at a number of locations, it is preferable to replace the member rather than repair it.

(Some of these measures are illustrated in Fig. 1 to 4 of the Annexure.)

5.1.2. Cracks

Cracks occurring in isolated locations can be repaired by drilling a hole of 13.5 to 23.5mm diameter at about 20mm beyond the tip of the crack, along its assumed line of further progress, to arrest the crack propagation. This will generally be a temporary repair and should be followed by fixing splice plates or splice angles with adequate number of bolts on either side of the crack. This is a very common solution for isolated cases of cracks. For multiple cracks in a single member it may be desirable to replace the cracked member by an identical member. Alternatively only the portion of the member, which has been damaged, may be replaced and an adequate splice connection between the new part and the existing part may be provided.

In welded girders, typical cracks may occur in the web near the lower end of the welded stiffener connection to the web. These cracks can be repaired by drilling holes beyond the tips of the crack and then gouging out the cracked portion and depositing weld metal in its place, followed by removing the excess metal by grinding. However, for undertaking field welding, the chemical composition of steel should be ascertained and appropriate electrode should be chosen in consultation with experts. A suitable, bolted splice on the other face of the web would provide added strength.

(Some of these measure are illustrated in Fig. 5 to 7 of the Annexure)

5.1.3 Buckling and Bending

Local Buckling and bending of members due to vehicle collision or accident may be rectified either by mechanical means or by application of heat. However, the latter method is not popular amongst technicians.

For mechanical straightening, the recommended process is to slowly apply heat in the damaged area and then straighten it by mechanical means avoiding impact load. The member should then be allowed to cool without application of any external aid. Straightening of the member without application of heat (i.e. in ambient temperature) is generally not recommended, as the heavy external loads required for this process might adversely affect the properties of steel.

5.2 Upgrading the Structure for Increased Loading

In general, the procedures available for upgrading the structure for increased loading are:

5.2.1 Strengthening of Deficient Components

Capacities of rolled beams and plate girders can be increased by adding cover plates to the top and bottom flanges near the midspan. Lengths of the cover plates and their cut-off points are to be determined by design check. In case of rolled beams and welded plate girders the cover plates can be welded. However, in order to avoid fatigue related cracks, it is preferable to extend the cover plates to the ends, instead of terminating these at the theoretical cut-off point. For riverted plate girders, cover plates are to be fixed by bolts in a similar manner described in clause 5.1.1. above

For truss bridges, capacities of deficient main members such as top and bottom chords, diagonals and verticals can be enhanced by providing additional steel areas to these members by bolting plates to the webs or flanges of the members.

5.2.2 Introduction of Supplementary Members

Capacity of top chords and other compression members of a truss bridge can be increased by reducing the effective lengths of these members. This can be achieved by subdividing the panels by introducing new members.

5.2.3 Reduction of Dead Load

Live load capacity of a bridge can be

increased if the dead load of the structure is reduced. One common example of this solution is replacement of the existing RC deck slab by orthotropic steel deck system.

5.2.4 Modification of Structural System

Capacity of an existing bridge can be increased by incorporating changes in the details, so as to modify its structural system. For example, simply supported spans of longitudinal stringer beams in the deck system can be converted to continuous beams by suitably modifying the end connection details, thereby improving their load carrying capacity. Another method of augmenting the capacity of an existing bridge is by providing additional supports from below the underside of the girder at one or more points. Similarly, shifting of support points from the ends of the bridge to the next inward panel point may increase the capacity of the girder. The new configuration would be a reduced span with cantilevered panels at the ends. Another innovative method for increasing the load carrying capacity of an existing bridge is by introducing counterbalancing forces in the structural system by means of external post-tensioning tendons, which work much the same way as in a post-tensioned concrete beam. This procedure induces new stresses in the structure and reduces the effects of the existing dead or live loads on the structure. Thus the live load capacity of the bridge is increased.

5.3 Modification for Increase of Clearance Dimensions

Existing through type bridges may need modifications to accommodate such requirements. End portal systems and sway bracings are the most common members that are affected. These components would be required to be relocated to clear the new dimensions. In case space is not available within the structure, it may be necessary to place the portal bracings and sway bracings above the bridge structure and fix these on to stools placed over the top chords at node points to clear the new clearance diagram. It is necessary to carry out design checks to ensure that the adopted system is adequate to transmit the lateral forces from the top chords to the bearings.

5.4 Some sketches showing a few typical connections covering modifications/ rehabilitation work elaborated above are given in the Annexure.

6. BEARINGS AND BED BLOCKS

COMMON PROBLEMS AND REMEDIAL SOLUTIONS

Bearings are primarily required to transmit loads to the foundations and to allow movements of the supporting superstructure. Although relatively small components in bridge structures, these are of significant importance for the proper functioning of any bridge. In many occasions distress in the superstructure as well as in substructures have been found to be due to improper functioning of the bearings. In this section, some of the common problems associated with bearings and their remedial solutions have been discussed.

6.1 Corrosion

As discussed earlier, corrosion and rusting in steel are mostly caused by water. dust and debris, which often collect at the location of bearings, have the tendency to absorb and retain moisture and thereby cause corrosion. Therefore, it is imperative to ensure that debris are not allowed to accumulate at the location of bearings.

Heavy corrosion on the contact surfaces increases the coefficient of friction significantly, thereby impeding the movement of sliding plates or rollers, and rendering the bearings ineffective. Often smaller components such as tooth bars or pins get corroded, resulting in restriction in the movement of the bearing.

When bearings show effects of severe corrosion, these may need temporary removal and thorough check up. These should be rehabilitated by replacing damaged components, if necessary, and then re-erected after painting and greasing. Where components do not show any major loss of section due to corrosion, they may need only cleaning in situ and greasing.

6.2 Misalignment

Misalignment of bridge bearings may restrict the movements of the superstructure and induce additional forces in the bridge structure. Misalignment is caused by excessive vibration of the superstructure due to moving loads, severe earthquake, error in setting the bearings defective fabrication, or non linear and non-ductile substructure movement due to severe earthquakes or settlement of foundation due to overload.

A misaligned bearing can be rehabilitated by first jacking up the bridge superstructure to relieve the load on the bearing, introducing temporary props to support the structure, and then re-setting the bearing components with correct alignment, giving due consideration to the temperature effect for the inclination of the rollers. Normally jacking points are pre-located in a steel bridge. However, in case these are not available in a particular bridge, it will be necessary to develop suitable jacking points, considering the stability of the bridge and adequacy of the concerned member to be jacked.

6.3 Tilting of Bearings

Tilting of bearings may be due to the movement of either the substructure or of the superstructure or both. In either case, remedial measures for such movements should be implemented first, prior to re-setting of the tilted bearing. Otherwise, the problem may recur after some time. Re-setting of tilted bearings can be done in the similar manner as in case of misaligned bearings. Also, the movements can take place due to occurance of severe earthquake.

6.4 Damage in Bed Blocks

The concrete bed blocks are subjected to significant vertical and horizontal forces. Consequently, these locations in many old bridges have been found to deteriorate. This may be due to lack of proper and uniform contact between the underside of the bedplate and the top of the bed block or due to severe earthquake. Repeated impact loads from the vehicles due to poor roadway surface above the misaligned deck joints may aggravate the situation. Malfunctioning of bearings due to reasons discussed earlier may also add to the distress.

Before undertaking repair work, the cause of the distress in bed blocks should be investigated. Damaged bed blocks should be repaired, or if necessary even replaced, using stronger concrete mix. For this purpose, the superstructure has to be first jacked up to relieve the loads, and supported on temporary props. The bearings should be reinstalled only after the concrete has hardened.

6.5 Replacement

When a bearing suffers major defects such as severe corrosion in rollers, cracks in main components etc. it is advisable to replace such a bearing in its entirely. Also when a bridge is situated in a remote place, it may be easier and cheaper to replace a bearing rather than to repair it. Furthermore, in non-standard bridges (such as bridges with excessive skew), bearings are subjected to multidirectional rotations and existing traditional bearings are not designed for such conditions. In such cases the existing bearing would need replacement by modern elastomeric or pot or spherical bearings, to accommodate this type of movement.

The following points need to be considered in the event of replacement of bearings:

- The height of the new bearings has to fit the existing available height.
- The new bearing must satisfy the required functional demands viz. horizontal and/or rotational movements as necessary.
- In case existing bearings are replaced by new bearings with elastomer materials, the horizontal force on the substructure may change. Therefore, the adequacy of the substructure should be checked in such a case.

7. POST REHABILITATION MAINTENANCE

As in the case of a newly constructed steel bridge, a rehabilitated bridge also needs to be protected from the hazards of deterioration due to natural as well as manmade situations, so that the investment made for rehabilitation is not wasted prematurely. Introduction of a well-planned and monitored inspection and maintenance regime is therefore necessary for all rehabilitated bridge structures. Such a system commonly termed 'Bridge Management System (BMS)' would ensure periodic inspection and recording of the current state of the structure and would keep the authorities informed, on a continual basis, about the condition of the bridge structure for taking timely remedial action.

Requirements for routine inspection and maintenance of bridges have been covered in other published literature (e.g.IRC:24-2001, IRC:SP:18 and IRC:SP:35). These are, therefore, not repeated here. This section is intended primarily to provide guidelines for preventative maintenance repainting needed for rehabilitated old steel bridges.

7.1 Frequency of Inspection

The frequency and level of inspection/ monitoring of rehabilitated bridge girders should be as laid down in IRC:24-2001 with the following modifications.

1) Two detailed inspections of the bridge should be carried out : one six months after the bridge is put to traffic use and the next after another six months.
2) Thereafter periodic routine inspection should be carried out every three years.
3) In special cases the competent authority should lay down the frequency and level of inspection.

7.2 Maintenance Repainting

7.2.1 Common Problems

It has been noticed from experience that quite often, areas, which are easily accessible, are painted regularly, whereas areas, which are not so easily accessible, do not receive proper attention. Thus areas, which have easy access, are generally not rusted over the years. On the other hand the inaccessible parts are often corroded. This aspect needs to be carefully considered during maintenance repainting.

It is necessary to remove all dirt, oil and rust from the surface of the member prior to application of the fresh coating. In case of heavy rusting, due attention should be given to ensure that the loss of sectional area is not beyond the permissible limit, in which case the member may have to be strengthened by adding corrosion plate prior to painting.

Another aspect needing careful consideration is excessive number of coatings on a member. Although thick coatings may appear to provide more protection to the steel surface, it may, in fact, be counterproductive, leading to cracking and flaking of the coat. This condition may need removal of the entire coating in the affected location.

The initial painting system adopted for a bridge structure and the quality of maintenance painting done over the subsequent period, have considerable influence over the efficiency of the maintenance repainting work being done currently. If the original painting system was inadequate for the service condition, or the workmanship was not upto the desired level, efficient repainting work becomes rather difficult, similarly, inadequate maintenance over the years may need extensive cleaning and often repair work entailing large scale patch painting prior to final coats of painting.

7.2.2 Selection of Protective System

If the performance of the existing paint is satisfactory, the same paint is normally applied over the existing one. In case, however, the existing painting system is not satisfactory, it may be necessary to go in for a new one. In such a situation the following aspects need careful consideration:

(a) Compatibility

The new protective system must be compatible with the existing system. Otherwise the new coat may not adhere to the existing one for a long period. Also, the existing paint may need special abrasive treatment to make the surface uneven to hold the new maintenance coat.

(b) Environment

For selection of a coating system, the following aspects need particular consideration:

- Environmental conditions such as polluted or non polluted, saline or non saline
- Local conditions such as intermittent splash of water, presence of harmful salts, fungi or bacteria etc.
- Likelihood of impact or abrasion from water borne debris

(c) Availability of the System

The new coating system and the facilities for its application are easily available to avoid delay in overall progress.

(d) Access of the Bridge

Bridges situated in remote areas where access for maintenance is both difficult and expensive, a more durable coating system may be preferable, even if the initial cost is more.

(e) Ease of application

This aspect is particularly important where there is shortage of skilled workmen. In such cases, systems not requiring specialist operators (for say, blast cleaning) would be preferable.

(f) Economy

For financial evaluation of the protective system, the initial cost as well as the future maintenance costs should be considered for evaluating the total cost. For this purpose total Life Cycle Costs (LCC) of some prima facie suitable systems should be calculated and compared. It has often been found that, in an aggressive environment or where the bridge is situated in a remote and inaccessible location, a special corrosion resistant painting system, with longer durability property, but with higher cost range may prove to be more economic over a period of time if analyzed with life cycle cost method.

(g) Other factors

Some of the other factors which need consideration are:

- Appearance (after final coat) vis-à-vis the surroundings.
- Time required to apply the coating system
- Past performance

7.2.3 Surface Preparation Prior to Repainting

As in the case of a new bridge structure, surface preparation is very important to make the new coating system effective. Unless the surface is properly cleaned and made free from rust or other chemicals, corrosion under the new coating is likely to start again.

8. REFERENCES

In preparation of this publication, the following Indian and International Standard and References were considered. At the time of publication, the editions indicated were valid. All Standards are subject to revision and the parties to agreements based on these guidelines are encouraged to investigate the possibility of applying the most recent additions of standards etc.

8.1 Codes, Manuals an Guidelines

S.No.	DOCUMENT Number. /PUBLICATION	TITLE OF THE DOCUMENT
1	IRC:24-2001	Standard Specifications and Code of Practice for Road Bridges Section V, Steel Road Bridges (Second Revision)
2	IRC:SP:18-1978	Manual for Highway Bridge Maintenance Inspection
3	IRC:SP:35-1990	Guidelines for Inspection and Maintenance of Bridges
4	IRC:SP:37-1999	Guidelines for Evaluation of Load Carrying Capacity of Bridges
5	IRC:SP:40-1993	Guidelines on Techniques for Strengthening and Rehabuilitain of Bridges
6	IS:1182:1983	Recommended Practice for Radiographic Examination of Fusion Welded butt Joints in steel plates (Second Revision)
7	IS:2598:1966	Safety Code for Industrial Radiographic Practice
8	IS:3658:1999	Code of Practice for liquid penetrant flow detection
9	IS:3664:1981	Code of Practice for Ultrasonic Pulse echo Contact and Immersion Methods
10	IS:3703:1980	Code of Practice for Magnetic Particle Flow detection
11	IS:4260-1986	Recommended Practice for Ultrasonic Testing of butt welds in ferritic steel
12	IS:5334:2003	Magnetic particle Flaw Detection of Welds-Code of Practice
13	AASHTO 1974	Manual for Maintenance/Inspection of Bridges
14	AASHTO 1983	Manual for Maintenance/Inspection of Bridges
15	AASHTO Guide 1989	Specification for Fatigue Design of Steel Bridges
16	AASHTO Guide 1989	Specification for Strength Evaluation of Existing Steel and Concrete Bridges
17	AASHTO Guide 1990	Specification for Evaluation of Existing Steel Bridges
18	HMSO London 1983	Bridge Inspection Guide
19	RDSO Indian Railways 1990	Guidelines for Inspection and Maintenance of Welded Bridge Girders

8.2 Reports and Books

S.No.	DOCUMENT Number. /PUBLICATION	TITLE OF THE DOCUMENT
1	NCHRP Report No. 206	Detection and Repair of Fatigue Damages in Welded Bridges 1979
2	NCHRP Report No. 271 1984	Guidelines fo Evaluation and Repair of Damages Steel Members
3	Oxford and IBH Publishing Co. Pvt. Ltd. New Delhi 2000	Repair and Rehabilitation of Steel Bridges by Utpal K.Ghosh
4	Wiley IEEE 1992	Bridge Inspection and Maintenance bi Parsons Brinkerhoff
5	Thomas Telford, London 2001	Steel Bridge Strengthening by L K Reid, DM Milne and RE Craig

9. ACKNOWLEDGEMENT

The above figures are reproduced by permission from the book entitled "Repair and Rehabilitation of Steel Bridges" by Utpal K.Ghosh (Oxford & IBH Publishing Co. (P) Ltd., New Delhi).

LIST OF FIGURES

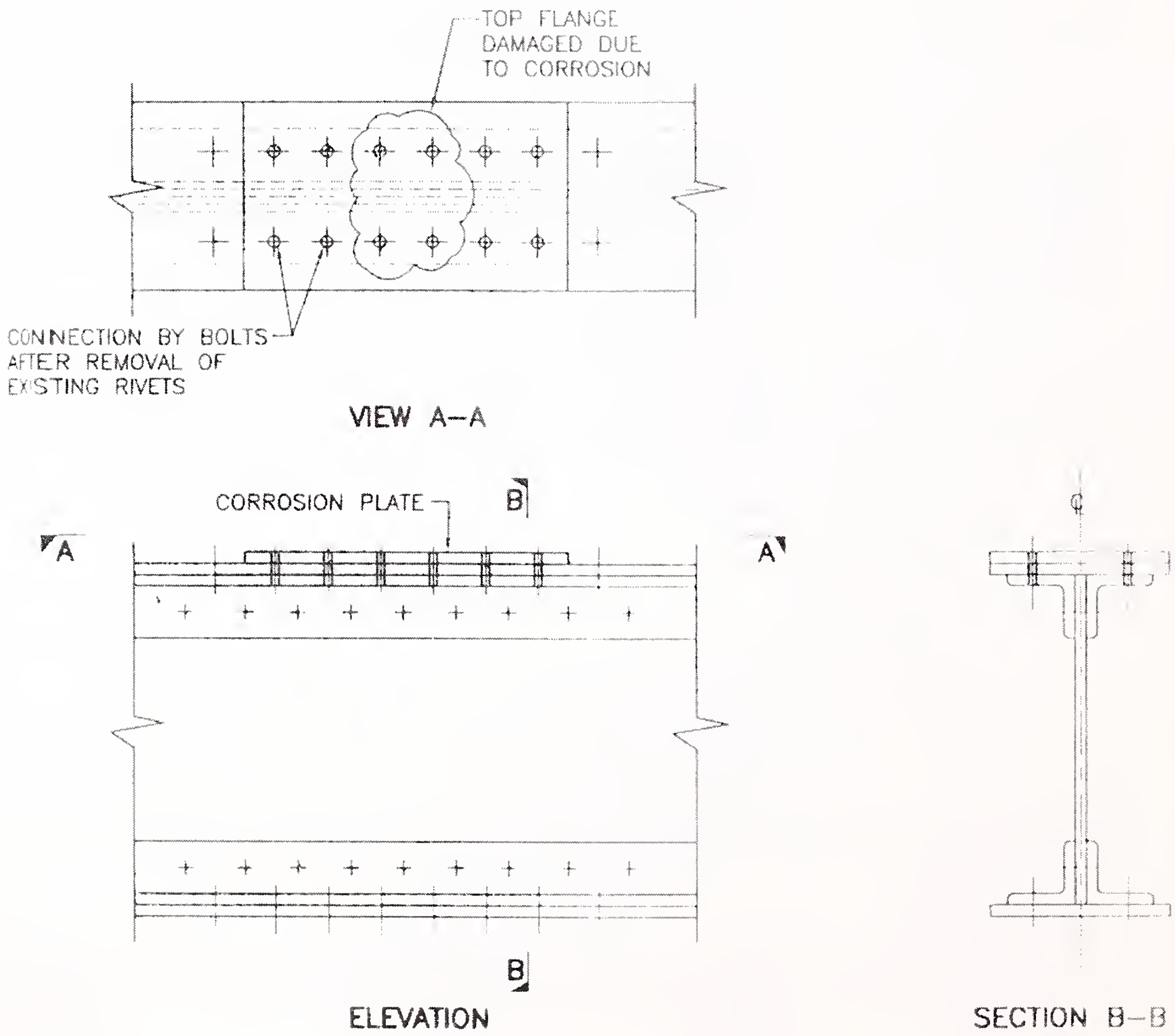


Fig. 1: Rehabilitation of corrosion damaged top flange plate of a riveted girder

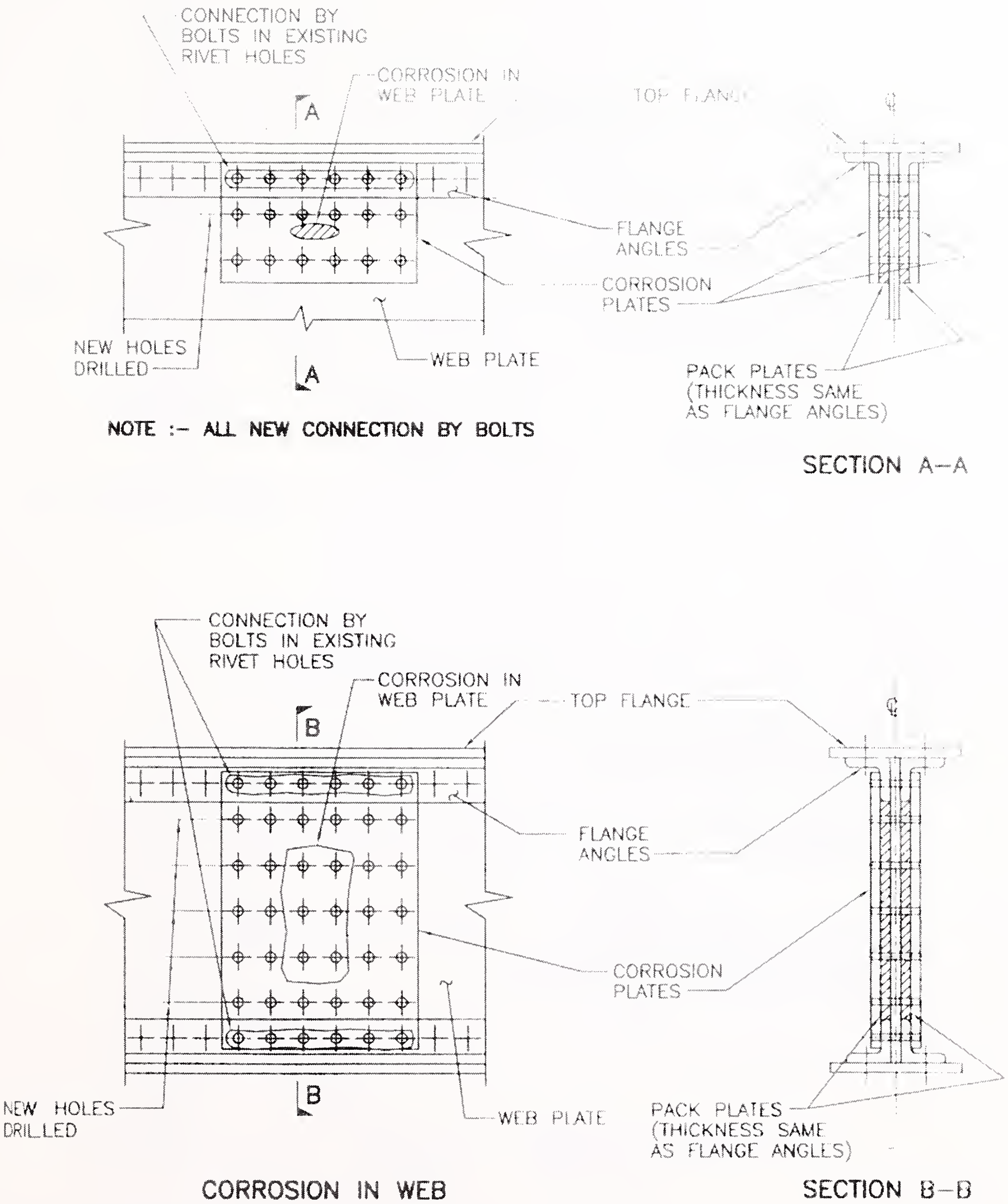
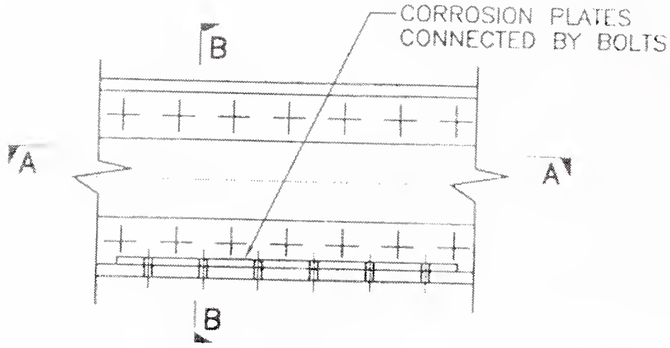
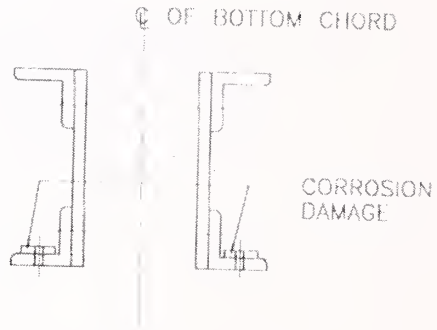


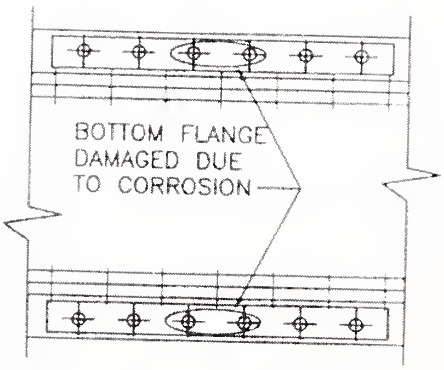
Fig. 2: Rehabilitation of corrosion damaged web plate of riveted girders



ELEVATION

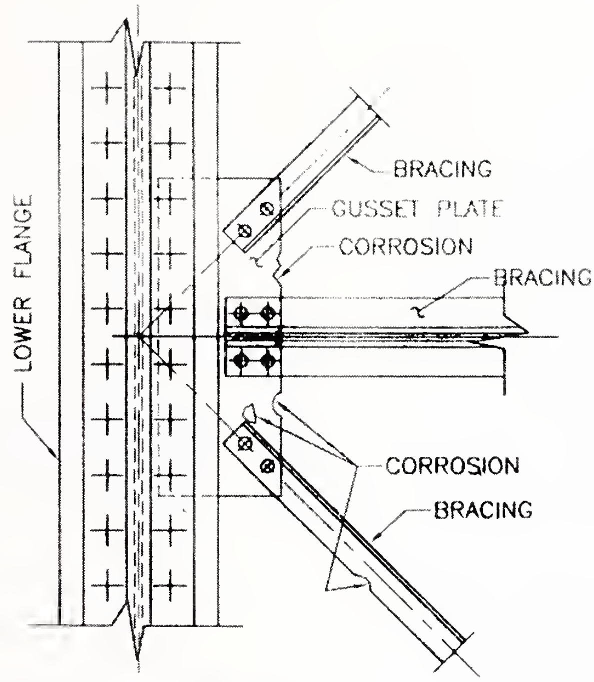


SECTION B–B

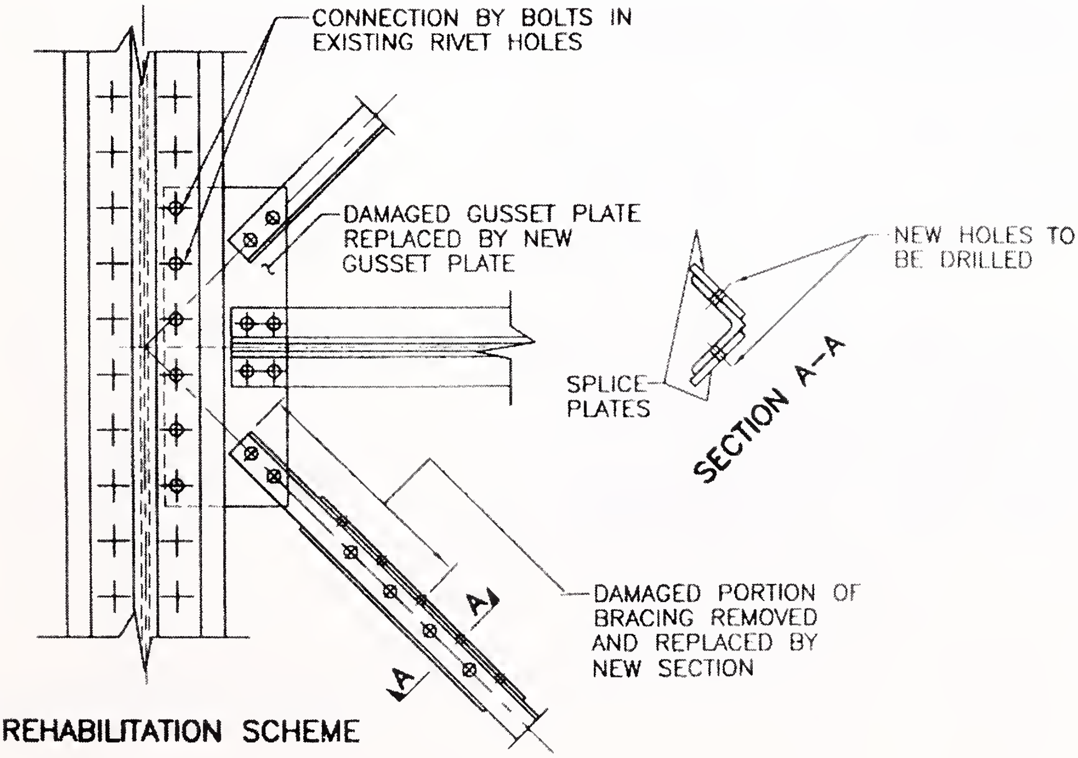


PLAN A–A

Fig. 3: Rehabilitation of corrosion damaged bottom chord of a truss bridge



(a) EXISTING DETAIL



(a) REHABILITATION SCHEME

Fig. 4: Rehabilitation of corrosion damaged lateral bracings

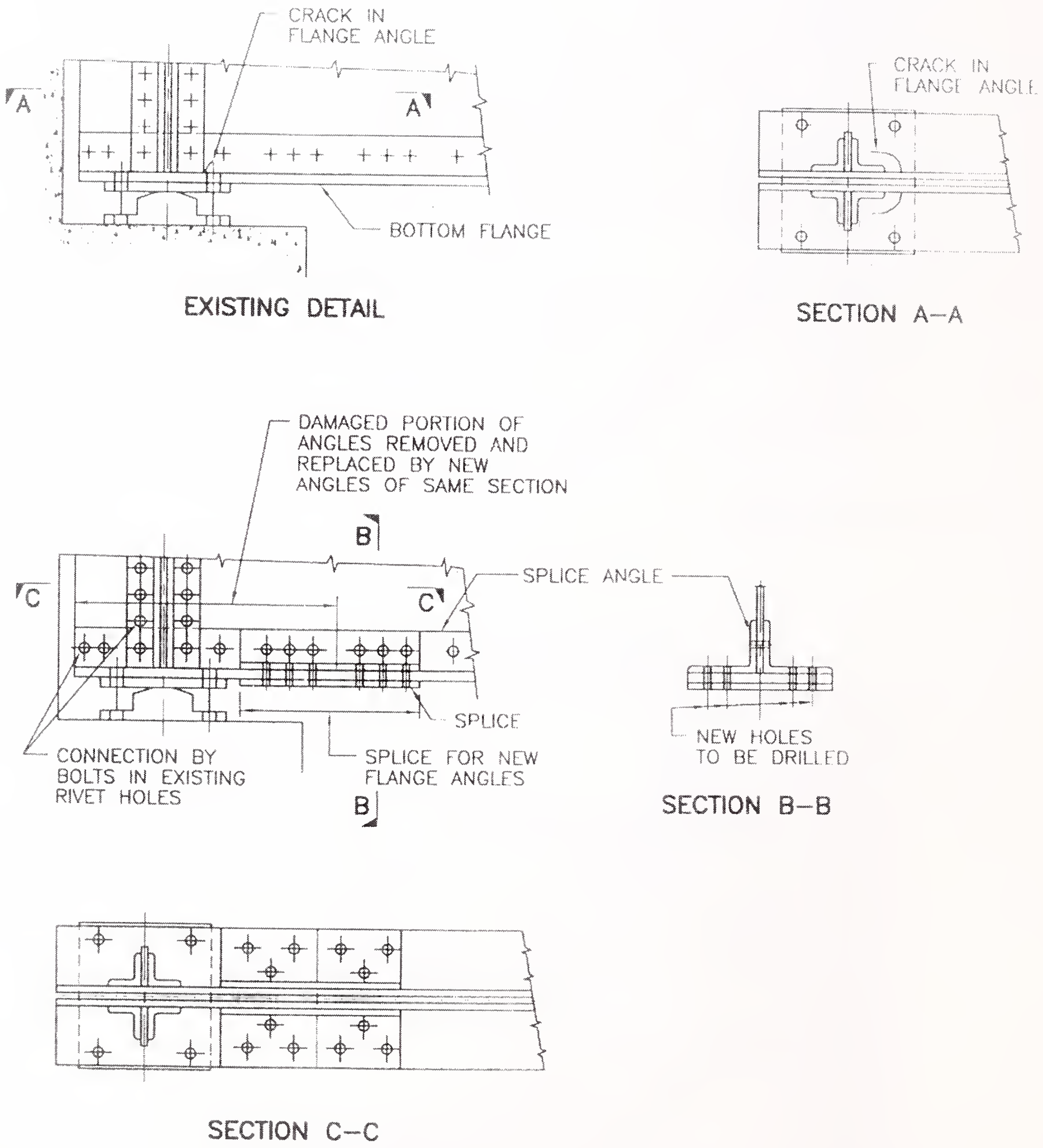


Fig. 5: Retrofit for crack near support in bottom flange angles of a riveted girder

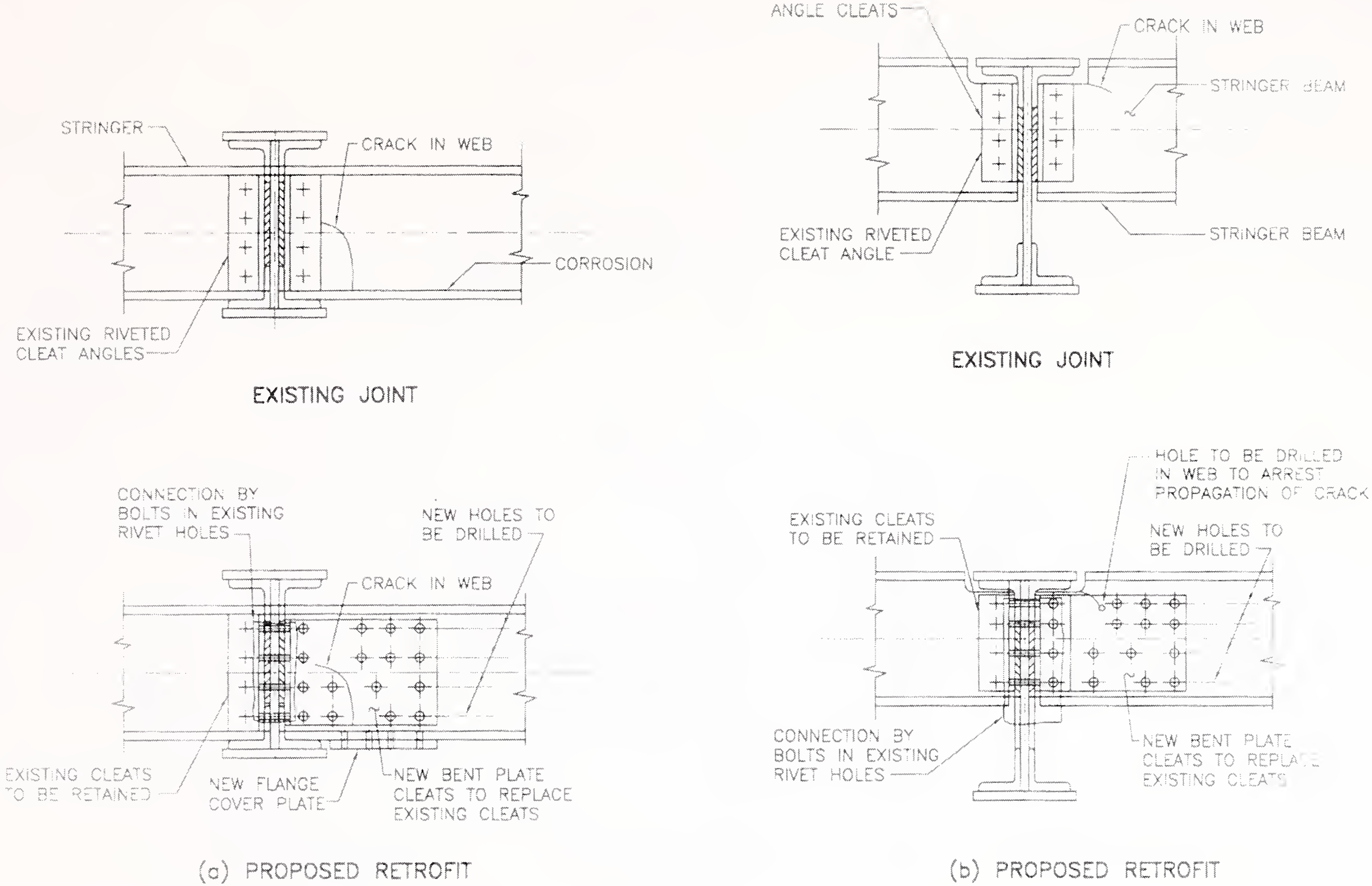


Fig. 6. Retrofit for crack at end of stringer beam

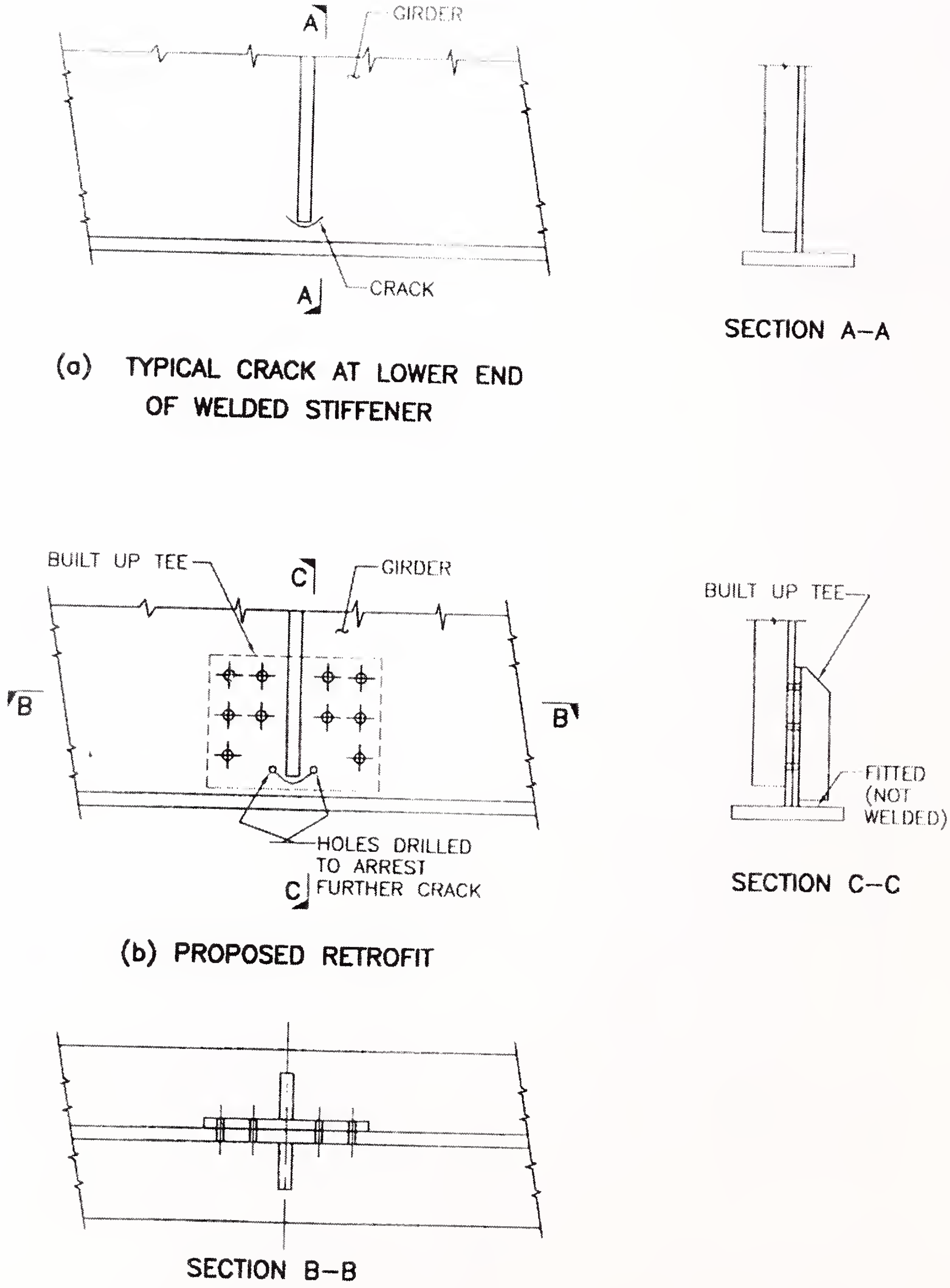


Fig. 7: Rehabilitation of crack in web of welded girder

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NOTIFICATION NO. 62 DATED 18TH JUNE, 2010

Sub: Addendum to IRC:SP:74-2007 "Guidelines for Repair and Rehabilitation of Steel Brides"

IRC:SP:74-2007 "Guidelines for Repair and Rehabilitation of Steel Bridges" was published in October, 2007. The Indian Roads Congress has decided to further amend the above document. Accordingly, the Addendum No. 1 is hereby notified.

This Addendum No.1 shall be effective from 1 July 2010.

R.P. Indoria

(R.P. Indoria)
Secretary General

ADDENDUM NO. 1 TO IRC SP 74:2007 "GUIDELINES FOR REPAIR AND REHABILITATION OF STEEL BRIDGES"

Clause No.	For	Read
Page 7 Clause 4.2.7 (b)	Non-destructive Testing (NDT) methods Crack testing	New Test **Acoustic Emission Technique** Acoustic Emission (AE) technique is one of the latest non-destructive testing (NDT) methods, which can be gainfully used for assessment of the condition of steel bridges. The technique is already in use for monitoring cracks in steel bridges in Western countries. Also, the technique is being used widely for monitoring corrosion and leak detection in aircraft and oil industries as well as in the atomic research centres and rocket industry in India. Acoustic Emission (AE) relates to elastic waves generated by a sudden redistribution of stress in a material. These waves propagate to the surface and are recorded by sensors. AE can result from the initiation and growth of cracks, slip and dislocation movements etc. Initiation and propagation of fatigue cracks can thus trigger AE. Detection and conversion of the elastic waves (related to AE) to electrical signals is the basis of AE testing. Analysis of these signals yield valuable information regarding the origin and importance of discontinuity in a material. This test can be carried out on line, requiring only limited time. This helps in identifying the affected region of crack including inaccessible areas. AE technique can only gauge the damage qualitatively. In order to get quantitative results (size, depth and overall acceptability), other NDT methods, such as ultrasonic testing, radiographic testing etc. are necessary. Another practical drawback of AE technique arises from loud extraneous noise in service environments.

(The official amendments to this document would be published by the IRC's in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the Date specified therin)