Coping with the Older Railroad Steel Bridges

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ABSTRACT

According to a recent estimate, major North American railroads possess roughly 50,000 steel bridges that make up approximately 880 miles of track and represent an investment of over 50 billion dollars. In addition to these, there are railroad steel bridges that belong to the short line railroads. About 40 percent of these bridges are reaching the age of 100 years or more. A majority of them were designed for a lighter loading than the one that exists today. Several have been upgraded while many others still need upgrading in response to increasing traffic and heavier axle loads.

This paper describes a simple way of evaluating both the strength and the fatigue life span of such bridges. Emphasis is placed on proper inspection, careful review of design, maintenance and the past performance of these bridges as these are the key factors in assessing their future useful service life.

The evaluation results in three categories: the bridges with sufficient capacity, needing no further action, the bridges with marginal capacity, requiring measures from reducing the existing state of stress to minor modifications of members, and the bridges with deficient capacity, necessitating planned programs of upgrading and replacement. Examples of different forms of upgrading are discussed, including reinforcing weaker components, adding new members, and changing the structural behavior of spans by introducing continuity, composite action, post tensioning and joint rigidity etc.

The paper concludes, that with a methodical approach to evaluating the capacity of the existing older railroad steel bridges, efficient and cost effective solutions may be found for their timely upgrading or replacement plans.
INTRODUCTION

In the year 1830, with the runs of “Tom Thumb” from Baltimore to Ellicott’s Mills and of “Best Friend of Charleston” on the South Carolina Canal and Railroad, the railroads took their foothold in the USA. In 1856, the inauguration of Illinois Central from Belleville to St. Louis followed.

The immense importance of the railroad was quickly realized in these years; where scattered communities and slow movements of people and goods relied on horse drawn carriages and boats. The railroads offered prospects of greater mobility, linking communities, growth in jobs, growth in trade and economy, and migration of settlers to the west. Consequently, railroad construction started with much enthusiasm. Between 1860 and 1920, railroads were rapidly expanding in all directions such that the track operated reached a peak of 254,037 miles in 1916. See Figure 1. The eagerness of laying down the rail was so great that all too often it was done with little preparation to the roadbed.

At the same time, a large number of railroad bridges were being built. The materials of their construction were wrought iron, steel, timber and stone and concrete masonry. Major streams were bridged with wrought iron or steel spans supported on masonry substructures. Valleys were bridged with timber trestles or steel viaducts, and small streams were bridged with timber or masonry bridges and culverts.

Most of the older bridges were designed for the heaviest locomotive that the railroad owner had at that time or was contemplating to acquire in the foreseeable future. The cars were comparatively light in weight. The traffic was also lighter than the volumes that trains are handling today. Many of the timber bridges have already been replaced with steel and concrete structures or culvert and fill. This trend is continues even today. A number of metal bridges have also been upgraded or replaced with more robust
structures as the demand of heavy axles grew. However, there are still many other steel bridges in service that are old and need attention and care for their continuing safe and reliable performance.

The maintenance and safety of the existing bridges is an important concern of all railroads. To assure adequate safety and determine the ongoing maintenance needs, a railroad bridge requires thorough regular inspections. These inspections should form the essential source of information for carrying out a comprehensive evaluation of its current capacity. The evaluation should identify any necessary work and indicate consequences of deferral of such work. The work should be well planned and performed in a timely, efficient, and cost effective manner in order to maintain good performance.

The computed bridge capacities might be divided into three categories: sufficient, marginal, and deficient capacity. Some pertinent features of the older railroad steel bridges and the ways of continuing to maintain their safe and reliable service are discussed with particular emphasis on bridges that possess marginal capacities.

BACKGROUND

1. Traffic

   a. Equipment:

   **Locomotives:** - The first locomotive to operate in the USA was the “Stourbridge Lion” in 1830. It was imported from England for the Delaware and Hudson Canal Company. It had two axles and weighed 7 tons. It could not be used for a regular service, as it was too heavy for the tracks of that time. So it was retired to stationary service after a trial run. Around the same time, Peter Cooper built a small vertical boiler locomotive called the “Tom Thumb” which also had two axles but weighed less than a ton. It was used on the Baltimore and Ohio Railroad and became the first American built locomotive to pull a
carriage of passengers. In 1830, the South Carolina Railroad was completed and in 1831,
it used the Miller’s two axle “Best Friend of Charleston” which weighed 4 tons and ran
for six months.

With this humble beginning, the railroads in the USA started expanding rapidly
in all directions. To meet their demands, larger, heavier, faster and more powerful steam
 locomotives were built. This trend continued till 1942, when the largest steam locomotive
called “Big Boy” 4-8-8-4 was built for the Union Pacific Railroad. The engine with its	
tender weighed 552 tons (67.5 kips/ driving axle), and where the track conditions
 permitted, it could operate at 60 mph.

Around 1923, the first diesel electric (DE) locomotive was built in the USA, it
 weighed 8.5 tons. Subsequently, the DE locomotives were built and used in switching,
passenger and in freight service. Because the DE locomotives were lighter, more reliable,
and offered maximum engine power over a wide range of speeds, they quickly started to
replace the steam locomotives. By 1960’s in the USA, no steam locomotives were left in
freight service. Many designs of the DE locomotives followed afterwards. These days,
 locomotives with more power, high tractive effort, and low maintenance are being
 produced. One such example is the SD90MAC that has 6000 HP, 6-axle, weighs 420,000
 lbs (70.0 kips/ axle) and can operate at a maximum speed of 75 mph.

**Freight Cars:** - The first freight car started in the USA in 1825 for hauling granite blocks
from the Granite Company of Massachusetts to Bunker Hill Monument. It had 2 axles
and weighed about 10 tons. By the turn of the last century, hopper cars, flat cars, box and
gondola cars, tank cars and refrigerator cars of different sizes and capacities were built
and used. The earlier cars were of wood but these were replaced by riveted steel and then
by all welded steel construction.
Around the 1950’s, the piggyback, bulkhead, bi and tri-level auto rack cars came into use. These were followed by 4, 5 and 7-pack cars. For handling large dimensional shipments multi-axle articulated cars were introduced.

The freight cars kept on increasing both in size and capacity. To keep the axle weights within the permitted limits, cars with more axles and to reduce the tare weight, cars with lighter material (such as aluminum) were built.

b. **Trains:**

Before World War II, the trains were generally shorter and relatively slower. The freight trains would consist of one or two steam locomotives and about ten to thirty cars. The cars were lighter compared to the locomotives. After WWII, the trains started to increase in length and weight. With the introduction of more powerful DE locomotives, train lengths increased dramatically. The 1950’s saw increase in unit train operation. This was followed by the intermodal, auto-rack and double stack traffic. The freight shipments have steadily increased ever since. It is not uncommon these days to see a freight train comprising of 150 or more cars and pulled by three or more locomotives. Sometimes all the locomotive units are at the head end of a train while other times they are distributed at the head end and in the middle of a train. The weight on the axle of the freight car of a train can be heavier than the weight of axle of the locomotive. Moreover, such trains are operating more frequently on some lines than ever before.

2. **Bridges**

   a. **Statistics:**

   According to one source, in 1888, there were 208,749 railroad bridges of all kinds in the United States of America with a total length of 3,213 miles. During the
1950’s, a trend of amalgamating railroads commenced and this resulted in shedding or abandoning of the non-profitable and heavy maintenance lines. This phenomenon created many regional and short line railroads. To day, there are only a few Class 1 railroads left. A survey conducted in the late 1990’s, indicated that there were about 98,000 railroad bridges of all kinds in the USA and Canada. They amounted to about 1,700 miles in length. Approximately, 51% were steel, 18% concrete and 31% timber bridges. Currently, the number of steel bridges is about 50,000. They comprise about 880 miles of track and represent an investment of over 50 billion dollars.

b. **Railroad Bridge Loading:**

Prior to 1895, most railroads designed their bridges for the heaviest locomotive in use or expected to be acquired in the foreseeable future. So, there was great variation in the weights and wheel spacing of the engines. Also it was a practice to obtain a minimum of eight bids for awarding a bridge contract. The design computations were done manually as no calculators were available then. Eight designs per contract meant laborious and costly computations. Because of the diversity in bridge design loadings there were often problems with moving the equipment of one railroad over to another railroad. This situation created a dire need for a uniform bridge design loading.

Around 1895, several different design load configurations were offered as bridge standards. The design loading system presented originally by Theodore Cooper in 1880 was later adopted by AREA (now AREMA) as a standard for bridge design loading. It consisted of two Consolidation type steam locomotives with tenders weighing 106.5 tons followed by a series of cars weighing 3000 lbs per lineal ft. This was called Cooper’s E-30 loading because each driving axle weighed 30,000 lbs. The design loading was gradually increased as the railroads started to use heavier equipment. In 1906, it was Cooper E-50, and subsequently in 1935, it was increased to Cooper E-72. Figure 2 shows
Cooper’s design loading recommended by AREMA at different times. Some railroad bridges are now being designed for as high as Cooper’s E-100 loading.

Another important factor to note is that the older bridges were designed for steam locomotives that had hammer blow impact. The impact due to the diesel locomotives do not have hammer blow, so the resulting impact is lower, as shown in Figure 3.

The Cooper E- rating of a bridge span is a measure of its load carrying capacity. The Cooper E- rating of a car, a locomotive, or a train, is a measure of its effect in terms of forces generated in a given span.

c. Material, Design and Fabrication

**Cast Iron** – The strength of cast iron depends on the amount of carbon in it. Its strength in compression is considerably more than in tension. It is a brittle material and has poor weldability.

**Wrought Iron** – It has a fibrous microstructure. It is stronger along its fibers than across its fibers. It is soft, very ductile and malleable. Its toughness is lower than steel. It is weldable and resists corrosion far better than steel. Its ultimate strength varies between 45 to 55 ksi. AREMA recommends a minimum yield stress of 25 ksi for rating bridges built with wrought iron.

**Structural Steel** – Steel has a uniform microstructure. Its strength is the same in all directions. Three general types of steels were common, namely, soft with ultimate strength varying between 40 to 55 ksi, medium with 55 to 70 ksi, and high steel with 70 to 80 ksi. The brittleness increases and ductility and weldability decreases from softer to harder steels. Steel made by the open-hearth method was considered more reliable (less cracking) than steel manufactured by the Bessemer process. AREMA recommends a minimum yield stress of 30 ksi for rating bridges built with these steels.
**Alloy Steel** – Some bridges were built with nickel steel, manganese steel or silicon steel. The alloy steels had higher ultimate strength, better toughness and more corrosion resistance than ordinary steels. AREMA recommends a minimum yield stress of 50 ksi for rating bridges built with nickel steel and 45 ksi for silicon steel.

The bridges constructed between 1860 and 1910 were generally designed for approximately Cooper E-40 to E-50 loading plus hammer blow impact. The fabricators of the bridges often provided the design computations as well. The bridge contracts based on ‘fixed price’ generally produced economical structures where the members and connections had just sufficient material for the specified designs. On the other hand, the bridge contracts based on ‘price per pound of steel’ produced structures that often had more material in members and connections than the minimum required.

The fabrication of steel was exclusively carried out with rivets. The holes for rivets were punched or sub-punched and reamed. Drilling of holes was rare as it was slow and expensive. Some connections were made with pins and pin plates or eye bars, particularly where zero moment or less secondary stresses were desirable.

Bolting was not used for fabrication. Its use was limited to replacing of loose or missing rivets or making temporary assemblies. Around 1960’s, as the installation of the HS bolts with the “turn-of-the-nut” method was introduced; the fabrication of steel with bolting then became fairly common. Drilling of holes in steel also became easier and faster with the pneumatic and electric drills. Welding was rarely used other than for repairing the corroded members. It was not known at the time that poor welding practices would cause problems. Beginning in late 1960’s, with the development of better structural welding techniques, the fabrication of welded bridge components also became quite common.
BRIDGE CONDITION

1. Inspections

Bridges are undoubtedly safer because they are inspected. The accidents just do not happen they are often the cause of inadequate inspection practices. Inspections must be of high quality and should involve careful, thorough, and complete examination of all members of a bridge. They should depict accurately the information about defects, weaknesses and abnormalities.

Benefits of regular and proper inspections of older bridges cannot be overlooked. They not only help in planning the necessary work but also help in discovering and monitoring any problems, thereby reducing expensive maintenance, reducing operating hazards, preventing structural failures and averting emergencies. Therefore, no laxness in inspections should be permitted. Regular inspections have significant influence on the load capacity rating, the remaining fatigue life evaluation, on reliable and safe train operation, and on the development of meaningful short and long-term recommendations for the continuing health of bridges.

The inspector should know what to look for, where to look for it, and should possess sufficient knowledge and judgment to realize the importance of what is discovered at the bridge. Any questionable condition discovered, should be further investigated as to why such condition exists and how it could be fixed or otherwise mitigated. As a general rule, the inspector should look for defects by following the path of the load through the structure. The closer the member is to the source of the loading, the more apt it will be to give trouble. Also the heavier the equipment, the higher the stresses in bridge structures. More frequent inspections may be necessary for bridges that are handling heavier traffic.

The inspector must be alert and observant that no conditions develop, which could evolve into hazard, and would affect the safety of the railroad operations and the public.
Particularly, the following areas should be given a closer look during inspections:

a) **Bridge Decks:**
   - Track for proper alignment and surface, especially near the ends of the bridge and its approaches.
   - Bridge approaches for full section of free draining material to adequately support the track.
   - Bridge deck ties for decay, checking and splitting, plate cutting, proper bearing, loose tie plates, and for condition of fasteners. Deck for sufficient layer of clean and free draining ballast.

b) **Steel Spans:**
   - For loss of material or cracking in main members, connections and secondary members. For members that are torn, bent, twisted or misaligned.
   - Connections and points on the structure where discontinuity or restraint is introduced. Areas that have abrupt change in cross-section.
   - Areas of excessive or unusual vibration, members that carry unequal and or excessive stress and loose connections. Condition of bracing.
   - Locations of stress concentration such as rivet or bolt holes, pin connections, nicks, cuts, notches, grinding marks, tack or plug welds, sharp copes, rough flame cuts, unfilled holes, and undercuts etc. Welded repairs to riveted or bolted members.
   - Rigid connections where out-of-plane bending is inducing distortion that could cause cracking such as stringer floor beam connections, transverse bracing and hanger connections, and small gaps at end connections. For cracks in tension members such as tension chord splices, diagonals and hangers.
   - Structural details which other bridges are known to have exhibited fatigue problems.

c) **Span Bearings:**
• Seized or malfunctioning bearings, rusted, bent, broken or dislocated components, dished bed-plates, sinking and pumping of bearings.

d) **Substructure:**

• Bents and viaduct towers for out of plumb, differential settlement and adequacy of sway and longitudinal bracing.

• Condition of the masonry substructure.

2. **Construction Details**

Before carrying out load capacity rating and the remaining fatigue life evaluation, all available plans; inspection reports and other pertinent documents regarding construction, repairs, alterations and strengthening of the bridge should be reviewed. This is to identify weak, damaged or fatigue prone components, to note all the repairs or alterations carried out, and to verify original construction details, operating restrictions, track eccentricity and super-elevation of the track etc. that have bearing on the overall assessment.

**TRAFFIC HISTORY**

The knowledge of the traffic history is very important for estimating the remaining fatigue life of bridge components. The traffic densities, the way- bills or the scale weights, the weight limits and other documents with similar information about the railroad line and the bridge are used to develop ‘load versus frequency histograms’ or ‘load spectrums’ for the bridge members. These load spectrums provide graphical views of the past, the present and the forecasted future traffic of the bridge.
CAPACITY ASSESSMENT

1. **Load Capacity Rating**

   The objectives of the rating analysis are to determine adequacy of the capacity of bridge components for the forces generated by the equipment operated on the bridge, to determine appropriate action when the capacity of the bridge is of a concern, and to ensure safety of public and operations by assigning the bridge a rating corresponding to the rating of its weakest member.

   (a) **Considerations:**

   It is important to have accurate information about the type of material used in the bridge being rated and its mechanical properties. When this is not available, it should be obtained by testing the test specimens. The steels used in the older bridges were generally of lower strength and the allowable stresses used in design were also more stringent. They did not have toughness controls that the steels of today have.

   The rating computations must take the latest physical condition of the bridge into consideration. These include the track and deck condition, track eccentricity, dead load additions, member section loss due to corrosion, wear and tear, cracking, fastener condition, looseness of joints, excessive deformation, condition of bearings, integrity of bridge components under traffic and the condition of bridge approaches etc. Any damage sustained by the bridge due to collision, fire, ground movement, stream flow, hurricane, earthquake, weather, overloading, and poor maintenance practices etc. must be considered in the capacity assessment.

   (b) **Rating Types:**

   [Content continues...]
There are two types of ratings used for the existing bridges, namely, normal rating and the maximum rating. For the normal rating, the load level considered is that can be carried for the expected life of bridge (i.e. regular traffic). Basic allowable (design) stresses are used for computing with reduced impact due to reduced speed below 60 mph.

For the maximum rating, the load level considered is that can be supported at infrequent intervals (i.e. occasional traffic). Therefore, higher than basic allowable stresses are used with applicable speed restrictions where necessary.

(c) Bridge Member Rating Procedure:

The loads considered for the purpose of rating are dead and live loads, impact, wind forces, centrifugal forces, lateral forces and longitudinal forces. Recently, the formula for longitudinal forces has been revised. The allowable stresses are increased by 25% when wind forces are considered as long as these stresses do not exceed the allowable stresses for the maximum rating.

The bridge structures were analyzed for static and moving loads in the past as simple plane frames. However, now, with the availability of computer software (SAP, STAAD, ETABS etc.) more rigorous analyses such as space frame with different load, joint and support configurations can be performed. The software computes the member forces (axial, shear, bending, torsion), displacements and influence lines. These can also perform dynamic modeling and other more sophisticated analyses.

Once the member forces and their geometric properties are known, the member stresses and live load rating (in bending, shear, or axially as the case may be), $MR_{LL}$, is determined in terms of Cooper’s equivalent from the following relationship.

$$MR_{LL} = \frac{(M_{LLC} - D_{LF}) \times 80}{E80_F} \text{ .................................... (1)}$$
Where; $M_{LLC}$ is the computed live load capacity of the member $= M_{TC} / (1 + I)$, $M_{TC}$ is the total capacity of member for normal rating or for maximum rating as the case may be, $D_LF$ is the stress in the member due to the dead load, $I$ is the impact factor, and $E80_F$ is the stress in the member due to Cooper E-80 loading on the bridge. Its value is obtained either from analysis or from already computed tables.

(d) **Equipment Rating Procedure:**

The equipment could consist of one or more locomotives, cars or trains that are operated on structure of a given span. The number of axles, the axle spacing, the coupled length and the axle loads of the equipment are shown on their individual equipment diagrams. The effect of equipment loading is also ascertained in terms of bending moment, shear or axial stresses as generated in members of a bridge. The rating of equipment is the maximum effect that it produces on a bridge member and it is also expressed in terms of Cooper equivalent.

When the equipment rating exceeds the bridge member rating, appropriate measures are required to safeguard the bridge.

2. **Fatigue Evaluation**

The purpose of fatigue evaluation is to minimize the probability of failure as a result of fatigue crack initiation and growth. This affects the maximum service life, which the structure is designed for. There are two approaches to deal with bridge fatigue; to ensure the structure is fail safe, or to limit the usable life to one that is shown to be safe for certain period of use. The later approach is considered here, as it is manageable and more cost effective.
In a ductile fracture, the material stretches before breaking. So the failure takes place slowly and often there is time to react and prevent a disaster. Contrary to this in a brittle fracture, the material breaks suddenly and there is no advance warning, and therefore the failure could have catastrophic results. Such failures are better controlled now by specifying material with greater toughness.

(a) Considerations:

The main factors that govern fatigue strength of a member are the number of stress cycles, $N$, the magnitude of stress range, $S_R$, and the type and location of detail. Numerous laboratory tests were conducted at constant-amplitude on different details to failure. These tests have shown the following relationship.

$$N \cdot (S_R)^b = c$$

Where: $b$ is the slope of a straight line plotted on a log-log scale, taken as –3, and $c$ is the intercept of the line. For different degrees of stress concentrations, bridge members have been classified into eight different detail categories. These are referred to as A, B, B’ C, D, E, E’ and F type details. These are shown as $S$-$N$ curves in Figure 5. They represent approximately the lower 95% confidence limit for the test data in each category.

The trains are now heavier, longer and more frequent than they were in the past. Therefore produce higher stress ranges and more loading cycles. Consequently, these bridges appear to be significantly affected now by the cumulative fatigue damage. For riveted and bolted structures, the fatigue categories of the common types of details are as given in the AREMA Manual for Railway Engineering.
The fatigue evaluation procedures are intended to realistically reflect the actual conditions that occur in the structure under consideration. This evaluation involves the following general steps.

(a) **Variable Amplitude Stress Range Spectrum** – For the individual members (usually weaker and fatigue prone) these are calculated from the actual loadings. When the loading test results are not available, a review of the traffic history and the stress analysis is performed to estimate the spectrum. Figure 4 shows a typical stress spectrum of a girder span. The number of stress cycles in a member depends on factors such as the number of vehicles in a train, the number of axles in each vehicle and their axle spacing and the loaded length of that particular member. For a long member, such as truss chords there may be only one cycle per train whereas for a short member, such as floor beam, stringer, or hanger, there may a cycle per a pair of trucks or even per truck. The number of load cycles produced in a member can also be estimated by comparison of the test results of a structure of similar type for which the test data is available.

(b) **Effective Constant Amplitude Stress** - The variable-amplitude stress spectrum of the member is then related to an equivalent or constant-amplitude stress (also termed as effective stress range) by some cumulative damage approach such as Miner’s Law used with rain flow counting or other cycle counting techniques. The AREMA recommends the following root mean cube (RMC) relationship.

\[ S_{re} = \alpha \left( \sum \gamma_i S_{ri}^3 \right)^{1/3} \]  

\( (3) \)
Where: \( S_{re} \) is the effective stress range for total number of variable stress cycles, \( N_v \).

\( \gamma_i \) is the ratio of the number of occurrences of \( S_{ri} \) to the total number of variable stress cycles \( N_v \). \( S_{ri} \) is the stress range of cyclic stress corresponding to the number of occurrences \( n_i \). The stress range is the algebraic sum of stresses of the opposite signs, and, \( \alpha \) is the ratio of \( S_R \) measured to \( S_R \) calculated which may be less than one.

(c) **Fatigue Strength Curves** - The resulting applied stress range parameter, \( S_{re} \), is compared to the fatigue strength \((S-N)\) curve that corresponds to the detail being considered to obtain the remaining fatigue life (in terms of the number of stress range cycles).

If the effective stress range, \( S_{re} \), for the member spectrum, is below the constant-amplitude limit (i.e. the dotted horizontal lines on the \( S-N \) curves) or when all the stress cycles are entirely in compression, the fatigue life of the member is assumed to be infinite and no further fatigue check is necessary. However, if it is not, then the remaining life is finite and is determined from the corresponding fatigue strength \((S-N)\) curve.

When the calculated remaining life is found to be inadequate, appropriate measures are required to rectify the situation.

3. **Capacity Classification**

The “capacity” of the weakest member of a bridge implied here includes the load carrying capacity and the acceptable remaining fatigue life in years. For sake of convenience, the capacity of a bridge can be classified into one of the following:
a. **Sufficient:** The calculated load capacity rating and the evaluated remaining fatigue life both are adequate. The current traffic on the bridge is handled without restrictions. Maintenance performed on the bridge is of preventive nature.

b. **Marginal:** The calculated load capacity rating and/or the evaluated remaining fatigue life are marginal (say within 10% of the required). Current traffic is handled with or without restrictions. The bridge requires frequent surveillance and maintenance. This situation would continue in future unless appropriate measures are taken to improve the condition.

c. **Deficient:** The calculated load capacity rating is inadequate and/or the evaluated remaining fatigue life is either too short or has been exhausted. Traffic is being handled risking some measure of safety. The bridge requires continuous maintenance with service disruptions and or public inconvenience. The bridge needs component strengthening or renewal, or an outright replacement.

PERFORMING RETROFIT

1. **Bridges with Marginal Capacity** – There are several possible measures that can be explored for improving the existing condition (i.e. load rating and fatigue life). Some of these are given below.

   (i) **Carry out rigorous evaluation:**

   Recalculate the capacity using rigorous analysis that gives results closer to the actual behavior of the structure. Verify the properties of bridge material by testing coupons taken from the bridge and use them for computations. Conduct field load test to determine the actual stresses in members that are instrumented. Use this data to refine the
computed stresses in other members. Past experiences have indicated that the measured stresses are often lower than the computed stresses and thus can yield higher load capacity and better remaining fatigue life.

(ii) **Improving live load distribution:**

Improve line and surface of track to prevent plate cutting, wide gauging and misalignment. Use larger size rail and larger size tie plates on bridge decks. Provide adequate anchorage of CWR on bridge and its approaches. Provide proper super-elevation on bridges with curved track. Where possible reduce track eccentricity on the bridge. The ballast decks should have a layer of sufficient thickness of free draining ballast under and around the ties. Provide full ballast section at bridge approaches. Enlarge bedplates of bearings. Add/ reinforce the existing bracings of spans, bents and towers.

(iii) **Reducing live load impact:**


(iv) **Structural modifications to reduce state of stress:**

Prevent loss of material due to corrosion with protective coatings. Repair corroded or damaged members. Relieve members and connections of any residual or locked-in stresses that may be present due to fabrication process, mechanical damage or end restraint using heat treatment, grinding, mechanical or ultrasonic peening etc. Reinforce, replace weaker members or add new members. Reduce unnecessary dead weight on bridges. Improve existing fatigue category of members by replacing rivets with properly tightened HS bolts in reamed holes, grinding of rough cuts and copes or stress relieving
etc. Carry out other retrofits such as repairing cracks and heat straightening of deformed members.

(v) **Operational Measures:**

**Bridges:** Apply weight and/or speed restrictions as appropriate. Prevent braking or accelerating on bridges. Avoid speeds of unit trains that would cause resonance in bridge spans. Allow only single train at a time on a weak bridge that was built for double track. Use a balanced speed on super-elevated bridges. Operate empties or lighter traffic on weaker route if such alternative is available. Replace rail break castings with straight rail on moveable bridge during non-navigational periods.

**Train Make-up:** Reduce lading on cars where possible. Consider cars that have light tare weight. Use longer cars (wider axle spacing and truck centers) as opposed to shorter cars. See Figure 6. Use cars with say 6-axles compared to cars with 4-axles. See Figure 7. Have heavier cars in train preceded and followed by lighter cars. See Figure 8. Ensure cars are properly loaded with sides and ends balanced.

**Enforcement Measures:** Install overload detectors on busy routes to ensure that carloads are not exceeding the permitted limits. Install impact detectors at appropriate locations to detect out-of-round and flat wheels. Monitor train speed by radar to ensure that train speeds are not exceeding the posted limits.

2. **Bridges with Deficient Capacity**

The measures listed above for bridges with marginal capacity also apply to bridges with deficient capacity. In addition, the following measures may be considered for increasing the live load capacity of bridges.
Reduce effective length of span by adding supports or by altering support constraints or by introducing support continuities. Where feasible develop composite action, such as introducing shear studs and cast-in-place deck. Alter the structural behavior of span from purely flexural to truss or arch type behavior. Convert hinged connections to rigid connections. Increase transverse and longitudinal stiffness by adding traction bracing and sway bracing. Where possible consider applying external pre-stress to main members. Institute more frequent inspections to assure adequate safety at all times while preparing for strengthening or replacement of the bridge.

a) **Strengthening:** While carrying out strengthening of a bridge, any members that have poor details or are weak should also be fixed even though such members do not affect the load rating or the fatigue life. These may include lateral or traction bracing, diaphragms, stiffeners, eye bars, bearings or corroded members.

b) **Replacement:** When strengthening is not cost effective, the bridge should be replaced. The replacement should be designed for the current and future demands in mind as well as the use of improved materials, better quality fabrication and erection techniques should be employed.

3. **Performing Work**

   For obtaining the maximum benefits, the work of retrofit, strengthening and replacement of a bridge should be well planned and carried out in a manner that would ensure safety, timeliness, efficiency, cost effectiveness and without or minimal disruption to railroad operation and inconvenience to public.
CONCLUSIONS

The older steel bridges were designed for different traffic than they are subjected to these days. The materials, the fabrication, and the erection techniques employed in their construction were also different in many ways than the ones in use today. However, they have provided railroads with safe and reliable service that is now reaching 100 years or more. This is because they have been maintained. With better understanding and care that would involve thorough regular inspections, comprehensive capacity evaluations and performing the work identified, in a timely, efficient and cost effective manner, many of them can still provide many more years of safe and reliable service.
Figure 1. – Track Miles of Railroad Versus Year – US Railroads

Cooper Design Loading
(1895 – 1995)

Figure 2. Cooper’s Design Loading (AREMA)
Figure 3. – Impact – With Hammer Blow and Without Hammer Blow

Figure 4. Stress Spectrum of a Typical Girder.

Figure 5. – Stress Range versus Number of Cycles – Log Log Scale
Figure 6. – Influence of Short versus Long Cars

Figure 7. – Influence of Number of Axles on Car

Figure 8. – Influence of Preceded and Followed by Empty Cars