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Fatigue in High Strength Reinforcement Bars: An Overview

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Abstract
Fatigue is a process of progressive structural change in a material subjected to transient loads, stresses or strains. Fatigue strength is defined as the maximum transient stress range (*S*) that may be repeated without causing failure for a specified number of loading cycles (*N*). The stress range is defined as the algebraic difference between the maximum and the minimum stress in a stress cycle: $S = fmax - fmin$; that is: the transient stress. Most ferrous materials exhibit an 'endurance limit' or 'fatigue limit' below which failure does not occur for an unlimited number of cycles, *N*. High strength bars can optimize design and cost as a result of high strength concrete, shallower sections of concrete and shuttering; with smaller foundations to carry the smaller loads. It is noted that normal allowable design stress is not invalidated by fatigue considerations.
Keywords: Fatigue, endurance limit, stress range, ADTT, relative rib area, AASHTO, DL, $ADTT_{SL}$, ADT, tensile strength, welded wire fabric, yield strength, anchorage, offshore, salinity

1. Introduction
Fatigue was hardly ever considered in early bridge design. The allowable 125-140MPa design stresses were low and conservative. Seldom does one get to hear of bridge failure occasioned by fatigue. Over time, it has become necessary for bridges to carry heavier loads through wider spans. A partial solution came by way of availability of high yield strength steel reinforcement in the developed world. These types and grades of steel reinforcement have yield strength in excess of 550 MPa. Granted, using steel with this higher capacity could provide various benefits to the concrete construction industry by reducing member cross sections and reinforcement quantities, leading to savings in material, shipping, and placement costs. (Corley, W.G; et al (1982).

The lack of explicit provisions in ACI 318 is almost certainly indicative of a lack of observed fatigue-related problems in those structural concrete members and systems used in existing building applications. Many designers are apprehensive of using welded wire fabric; but in a study, these misgivings have been found baseless.(Wilast, A. et al (2007). Welded wire fabric affords faster works for tanks, foundations, chimneys, cooling towers etc. In a sense, fatigue in an ordinary structure is taken care of indirectly and by default: low design stress, proper quality concrete raw materials, adherence to construction standards with respect to ratio of mix for concrete; and bar type, size, grade, spacing (both reinforcement and stirrups); bar cover; proper curing; service environment (non wet or salty), high temperatures for long periods. Fatigue considerations are important in the design of bridges and offshore structures.

1.1 Structures Prone to Fatigue
The usual, everyday house is regarded as carrying a dead load. Small loads as when a 50gm weaver bird or 2kg marabou stork perches on the roof are trivialities. Human and furniture circulation, as well as winter snow deposits merit no concern. But when 500 people hold a disco in a fifth floor hall the event is noted. So also one should note the wind speeds on a structure and factor this in the design. Thus assuming a dead load on a structure is not truly realistic.

Fatigue is a process of progressive structural change in a material subjected to transient loads, stresses or strains. It is always indicated for bridges, crane beams and offshore structures.

- Bridges carry varying dynamic load, largely non-periodic. That bridges span water bodies means that some bridge details come in contact with water. Over many years this water undermines the bridge structure and, depending on the initial quality, may reach the steel reinforcement. Wet corrosion then sets in, reducing the cross section of the bar and raising the stresses. Sometimes salt may be used to de-ice a bridge: remnants of such salt may find its way to the deck steel reinforcement. Bridges spanning salty ocean waters must contend with both wetness and salinity.
- Crane beams are a case of varying dynamic load.
- Offshore structures are subject to random dynamic loading in the form of variegated wind and water waves; as well as being permanently wetted by salty water. Any cracks present will quickly result in corrosion fatigue.

2. Properties of the Bars
Chemical composition of reinforcement bars is specified in standards. The alloying elements are carbon, manganese, molybdenum, silicon, zinc, chromium, vanadium, copper, nickel, columbium, aluminium. The exact

choice of these elements, and the amounts, are left at the manufacturer's discretion. The resulting bar should have minimum ultimate tensile strength of 1035MPa. Hot rolling and cold twisting are viable alternatives. Maximum carbon contents are 0.25% for grade 250 hot-rolled steel and cold-worked steel and 0.4% for grade 460/425 hot-rolled steel. For the latter to be weldable it is necessary for the carbon equivalent to be less than 0.51%. Maximum sulphur content =0.050% and maximum phosphorus content=0.04% (ASTM 722-1998).

2.1 Types of Fatigue Test

2.1.1 Axial Tests

Axial tests are usually conducted on "as received" bars in conventional fatigue machines. The advantages are: (a) tests are cheap and can be run at relatively high frequencies, up to about 150 Hz, so that long endurances can be obtained quickly; and (b) applied stresses can be calculated unambiguously. The great disadvantage is that it is difficult to grip the bars without introducing high local stresses which cause the fracture to occur near the grips.

It is also difficult to avoid the introduction of secondary stresses caused by lack of straightness in the bars or poor alignment in the testing machine. A variety of methods have been used in attempts to grip the bars satisfactorily, all of which have involved introducing an interlay between the bar and grips so that load transfer is spread evenly over the bar surface. The interlays have included leather strips wrapped around the bars**,** casting the ends of the bars into a low melting point alloy**,** bonding the ends into epoxy resin, and shot blasting the ends and wrapping with thin aluminium sheet (Tilly,G.P. 1979)**.**

Care need be taken to ensure axial loading and that fractures occur in the central section of the gauge length. The loading is wholly tensile and the ratio (R) of minimum to maximum cyclic stress is 0.2

2.1.2 Bending tests on reinforced beams

The main advantage is that the test beam simulates service conditions such as the interactive effects at the steel-to-concrete interface.

``After being cast in steel forms and cured for 24 hours under polythene sheeting, the beams were stripped and allowed to cure at 25°C and 100 per cent relative humidity until loading commenced. Beam age at test varied from 15 to 40 days. Beams were simply supported over a span of 1800 mm and centrally loaded (Fig. 3). Three conditions of test were used, viz, in air, in natural sea water and in 3 per cent NaCl Solution, all with sinusoidal cyclic loading at a frequency of 6.7 Hz. Not each bar type was necessarily subjected to all three conditions. Throughout the period of test water was continuously aerated and circulated around individual beams up to their mid¬height (Fig. 3); when tests ran for longer periods than one week the water was replaced each week''.(Roper, H-1982)

3. Factors Affecting Fatigue

Fatigue strength is by no means the dominating requirement of reinforcement and the rib patterns are designed to give good pull-out strength and crack control. Material is supplied against a minimum characteristic strength i.e. 0.2% proof stress. The fatigue strength of the steel in reinforcing bars depends upon chemical composition, microstructure, inclusions, and other variables. However, it has been shown that the fatigue strength of reinforcing bars may be only one-half of the fatigue strength of coupons machined from samples of the bars. In addition, reinforcing bar specifications are based on physical characteristics (ACI -1980).

3.1 Bar diameter

It is generally recognised that fatigue strength decreases with increase in bar diameter.

For plain cylindrical specimens the effect is relatively small. The explanation usually given for size effects is that bigger sections have a statistically greater likelihood of containing large flaws. Another contributing factor may be that smaller diameter bars can be more effectively worked. Also, studies have shown that in the vicinity of a stress concentration the minimum size of flaw to permit crack growth reduces with increased thickness of specimen.

3.2 Bar Geometry (Deformations)

Deformations on reinforcing bars provide the means of obtaining good bond between the steel and the concrete. However, these same deformations produce stress concentrations at their base, or at points where a deformation intersects another deformation or a longitudinal rib. These points of stress concentrations are where the fatigue fractures are observed to initiate. Any evaluation of the influence of the shape of the deformations on fatigue properties of the bar must recognize that the rolling technique and the cutting of the rolls necessarily requires specific (ACI -1980) limitations and variations in the pattern. This applies to the height of the deformations, the slopes on the walls of the deformations, and also to the fillets at the base of the deformations. An analytical study has shown that stress concentration of an external notch on an axially

loaded bar may be appreciable. This study indicated that the width, height, angle of rise, and base radius of a protruding deformation affect the magnitude of the stress concentration. It would appear that many reinforcing bar lugs may have stress concentration factors of 1.5 to 2.0. Tests on bars having a base radius varying from about

0.1 to 10 times the height of the deformation have been reported. These tests indicate that when the base radius is increased from 0.1 to about 1 to 2 times the height of the deformation, fatigue strength is increased appreciably (ACI -1980).

For bars having longitudinal ribs, it has been found that when tested in concrete beams their disposition affects the fatigue strength. With the ribs in a vertical plane, the fatigue strength can be as much as 40% lower than when placed in a horizontal plane.

Manufacturers' identification markings, which appear as raised features, are very potent stress concentrations and cause premature fractures. In cases where these have been filed off the fatigue life was increased by about 100%. (Tilly,G.P. 1979).

3.3 Grade

The fatigue life (or fatigue strength) is not clearly a function of the grade of the bar (yield or tensile strength) (ACI-1980).

3.4 Stress Range

For experimental convenience, axial tests are usually conducted with pulsating tension cycles having a tensile minimum stress to ensure that components of the loading system are kept in alignment and do not become slack at minimum load. In a structure the stress ratio (minimum/maximum cyclic stress) is a function of the ratio of dead to live load and in bridges it is commonly in the range - 0.2 to 0.4. Most experiments are at stress ratios of 0 to 0.2. The effect of increasing the mean stress is to reduce the allowable stress range for a given number of cycles to failure. The introduction of a compressive phase in the cycling produces a disproportionate effect and endurances are longer for a given stress range. (Tilly,G.P. 1979), (ACI-1980).

3.5 Microstructure

This is a function of chemical composition as well as the process of bar manufacture: hot-rolling, cold twisting or Quench Self Tempered. We can have different phases; namely martensite, pearlite and ferrite; varied grain sizes and relative amounts and orientation. Inclusions are discontinuities in the structure; initiating cracks because of notch effects. Crack growth is slowed down in passing from pearlite to ferrite phase. (Marina Rocha Pinto P.N-2014)

However, reinforcing bar specifications are based on physical characteristics. Consequently, the variables related to the steel composition are of limited concern to practicing structural engineers. The variables related to the physical characteristics and use of the reinforcing bars are of greater concern. (ACI-1980)

3.6 Corrosion

Damage due to corrosion can be a serious problem in highway bridges particularly for concrete decks subjected to application of salt for de-icing. There have been numerous cases where salt has reached the reinforcement, resulting in corrosion of the steel and spalling of the concrete. Such damage has been reported for a large number of concrete bridge decks the most serious being in the United States. The consequences of corrosion fatigue in offshore structures are even more ramified.(Tilly, G.P 1979).

3.7 Bending

The effect of bends on fatigue strength of bars have been investigated. Fatigue tests were carried out on both straight and bent #8 deformed bars embedded in concrete beams. The bends were through an angle of 45^{o} around a pin of 6 in. (15.2 cm) diameter. The fatigue strength of the bent bars was a little more than 50 percent below the fatigue strength of the straight bars. (ACI -1980) .

3.8 Welding

In an investigation21 using Grade 40 and Grade 60 reinforcement with the same deformation pattern, it was found that the fatigue strength of bars with stirrups attached by tack welding was about one-third less than bars with stirrups attached by wire ties. For both grades of steel, the fatigue strength of the bars with tack welding was about 20 ksi (138MPa) at 5 million cycles. All of the fatigue cracks were initiated at the weld locations. (ACI -1980)

4. Design for Fatigue

The AASHTO (2007) limit for fatigue-induced stress in mild steel reinforcement is based on the outcome of NCHRP Project 4-7 as reported by Helgason et al. (1976). The maximum permitted stress range (f_f) in straight reinforcement resulting from the fatigue load combination is given as:

$$f_f \leq 145 - 0.33 f_{min} + 55(r/h)\text{MPa} \quad (1)$$

where f_{min} = algebraic minimum stress level (compression is negative); and r/h = ratio of base radius to height of rolled-on transverse deformations; 0.3 may be used in the absence of actual values.

Recent revisions to AASHTO LRFD simply incorporate the default r⁄h ratio as follows:
$f_f \leq 165 - 0.33 f_{min}$ MPa (2)
Other studies sought to relate endurance limits (f_r) to yield strength (f_y), (Soltani, A. et al.-2012). They established that stress range is the critical parameter affecting fatigue. The endurance limit, (f_{min}) was seen to be independent and was pegged at 165MPa (24ksi). This led to the modified expression:
$f_f \leq 165 - 138(f_{min}|f_y)$MPa (3)
The AASHTO fatigue limits are derived largely from tests having N=2000000. However, as code-prescribed limits, they must be understood to be appropriate for the life of the structure. AASHTO (2007) provides some guidance as to (a) the definition of a fatigue cycle, and (b) the expected number of cycles over the life of a structure. For instance, on the basis of AASHTO-recommended values, a deck slab on an urban interstate may undergo approximately 1 million fatigue cycles per year as follows for [Eq.(4)]:
$N = 365 \times n \times ADTT_{SL} = 930750$ cycles (4)
where $ADTT_{SL} = 0.85(ADTT) = 0.85(0.15ADT)$ with ADT = 20,000 and n = 1.
$ADTT_{SL}$ is the single-lane, average daily truck traffic, ADT is the average daily traffic and n is the number of cycles associated with a single vehicle passage. The supporting girders [assuming the bridge span exceeds 12.2 m] will undergo twice this number of cycles (n = 2). Clearly, not all bridges see these many cycles.
A lightly traveled two-lane, rural highway bridge may undergo only about 100,000 cycles per year as follows:
$N = 365 \times n \times ADTT_{SL} = 109500$ cycles (5)
where $ADTT_{SL} = 1.0(ADTT) = 1.0(0.15ADT)$ with ADT = 1,000 and n = 2.

A simply supported beam having length L was considered. Nominal moments are determined at the midspan using the following loads:
DL = dead load (self weight). This value is determined for a range of values of DL/LL_{lane}.
LL_{lane} = specified lane load = 0.64 kip/ft (AASHTO LRFD)
LL_{truck} = greatest effect of design tandem and design truck . For truck on simple span, the minimum 32-kip axle spacing of 14 ft is used.
$LL_{fatigue}$ = effect of single design truck having 32-kip axle spacing of 30 ft.
It is recognized that the maximum moment does not occur exactly at the midspan; however, the error in making this assumption is quite small and becomes proportionally smaller as the span length increases.
From these moments, the STRENGTH and FATIGUE design moments are determined as follows:
$STRENGTH = 1.25\ DL + 1.75\ LL_{lane} + (1.75 \times 1.33)\ LL_{truck}$ (6)
$FATIGUE = (0.75 \times 1.15)\ LL_{fatigue}$ (7)
Where the 1.33 and 1.15 factors are for impact loading (IM).
In order to normalize for distribution, multiple lanes, etc., it is assumed that the STRENGTH design is optimized; therefore, the stress in the primary reinforcing steel under STRENGTH conditions is $\varphi fy = 0.9 fy$ regardless of bridge geometry. If this is the case, the reinforcing stress associated with the FATIGUE load is as follows:
$f_f = 0.9\ f_y$ x(FATIGUE /STRENGTH) (8)
Similarly, the minimum sustained load will result in a reinforcing stress of:
$f_{min} = 0.9\ f_y \times (DL/\ STRENGTH)$ (9)
The stress in the reinforcing steel under FATIGUE conditions is then normalized by the allowable stress [according to AASHTO Equation 5.5.3.2 (Equation 3 above)] to determine the ratio of transient (FATIGUE) stress to the calculated fatigue stress limit. The results from this approach were plotted for simple spans L = 10 to 160 ft and DL/LL_{lane} = 0.5, 1, 2, and 4. In this plot, the vertical axis reports the ratio $f_f / [24 - 20(fmin/fy)]$. Based on this approach, it is not expected that the fatigue limits will affect design using fy = 60 ksi over the range considered since the ratio of stress range/fatigue limit is less than unity for all cases. Only when $fy \geq$ 100ksi is this ratio exceeded, and then only for spans under 20ft. (NCHRP Report 679 (2011), Soltani, A. et al. (2012).)

As of now, there is no universally agreed upon system of fatigue study or fatigue design for rebars. Results can show much scatter and variation. Tests that crack near the grips are invalid. Also, how to deal with the impact effect of a dynamic load? What about fatigue in other members e.g. anchor hooks (eye bolt), plates and girders? Making bridge expansion joints watertight is useful: salinated water arising from de-icing is made to run off to drainage rather than seep into the deck.

5. Discussion

There has been much research into fatigue of reinforcement. This has been intensified in recent years by the introduction of higher strength materials, the development of advanced applications such as offshore structures and the adoption of new design codes. In addition it is becoming recognised that features such as corrosion, type of bar, form of manufacture, etc. can cause the fatigue lives to be substantially lower than are normally given in reference data.

There is no evidence of the fatigue limit which is usually considered to develop at about 2 x10^6 cycles.

Five fractures occurred at beyond 10^7 cycles, the longest being at 97 x 10^6 cycles. Tests which survived 10^8 cycles were stopped unbroken but were treated as broken in analysis of the data. Tests which failed at the grips or were stopped short of 10^8 cycles for other reasons have been excluded. (Tilly,G.P & Moss, D.S-1982)

Presently accepted values for the fatigue or endurance limit for reinforcing steel are applicable to higher-strength bars and likely are conservative. Fatigue considerations will rarely affect the design of typical reinforced-concrete members having reinforcing steel with $f_y \leq 690$ MPa (100 ksi).

6. Conclusion

The design Provision for fatigue in the current AASHTO specifications was initially adopted in 1974. In this Provision, the limiting stress range in reinforcing bars depends on the minimum stress level and the ratio of base radius to height of the transverse lugs. Stress range, bar diameter and surface geometry especially the lug base radius to height ratio (r/h) have significant effects on the fatigue strength of the bars. Relative rib area has no effect on fatigue performance.

Designing for fatigue using the generally accepted endurance limit of 165MPa is sufficient and conservative for most applications, except for high strength rebars with $fy \geq 100ksi$. For bridges and offshore structures, the fatigue effects of corrosion fatigue are acute. Some designers try stainless steel reinforcement. Another method is to design offshore structures largely loaded in compression; with no-tension members. Unable to avoid wetness and salinity, a sensible option is the use of proper quality concrete raw materials, proper casting of the concrete and adequate bar cover.

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Figure 1: The MTS 810 fatigue testing machine with Test Star control software (Wilast, A. et al 2007).

Bar Size	No. tests	f_y (ksi)	f_{min}	f_r (ksi)	reference
#8	157	40	$0.10f_y$	38	Pfister, J.F. and Hognestad, E. 1964
		60		34	
		75		39	
		40	$0.30f_y$	33	
		60		31	
		75		32	
		40	$0.10f_y$	32	
		75		35	
		40	$0.30f_y$	28	
		75		30	
#5	19	40	$0.25f_y$	27	Lash, SD. 1969
		60		31	
		75		38	
#8	72	40	$0.10f_y$	30	MacGregor, J.G., et al. 1971
		60		30	
		75		30	
		40	$0.40f_y$	25	
		60		25	
		75		25	
#5	not reported	40	$0.10f_y$	32	
		60		33	
		75		36	
#10	not reported	40	$0.10f_y$	31	
		60		29	
		75		30	
		49	$0.17f_y$	28	Wascheidt, H. 1965
		53	$0.16f_y$	28	
		64	$0.13f_y$	28	
		88	$0.10f_y$	31	
		50		30	Kokubu, M. and Okamura, H. 1965
		57		30	
		70		31.5	
#3, #4 & #5	n.r.	120	0	45[1]	DeJong 2005
#5	1	120	$0.10f_y$	32	present study

[1] One million cycle fatigue life

Table 1: Fatigue stress ranges (f_r) corresponding to a fatigue life of 2 million cycles (NCHRP Report 679-2011)

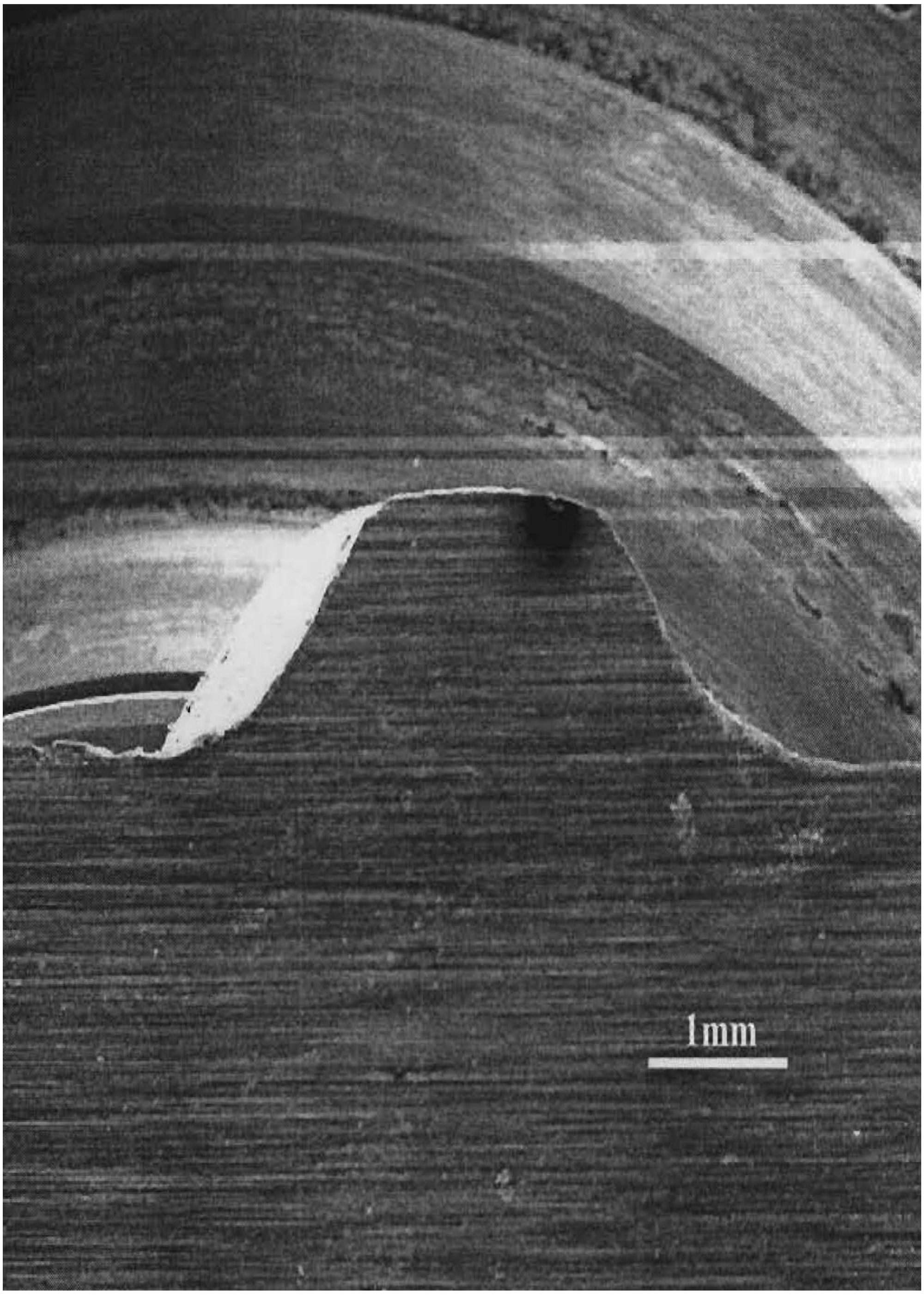

Figure 2: Scanning Electron Microscope Image of a Lug (Jun Fei-1999)

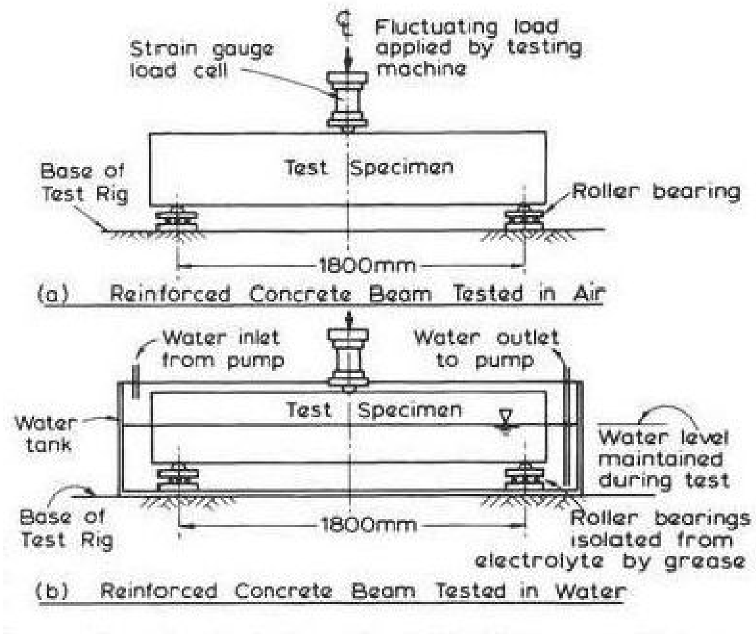

Figure 3: Test layout of fatigue specimens: In-beam fatigue test (Roper, H. -1982)

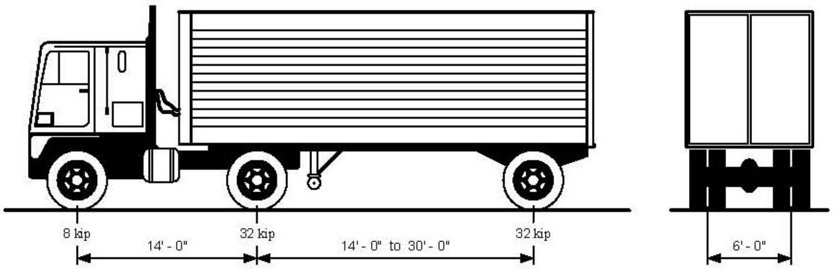

Figure 4: Design Truck (AASHTO-LRFD-2007)

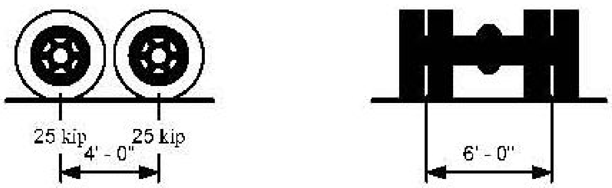

Figure 5: Design Tandem (AASHTO-LRFD-2007)

Figure 6: Schoharie Creek Bridge; collapsed 1987 (George A Christian,P.E; NYSDOT-2010)