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# Rating of steel bridges considering fatigue and corrosion

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**Abstract.** In the present work, the capacity ratings of steel truss bridges have been carried out incorporating dynamic effect of moving vehicles and its accumulating effect as fatigue. Further, corrosion in the steel members has been taken into account to examine the rating factor. Dynamic effect has been considered in the rating procedure making use of impact factors obtained from simulation studies as well as from codal guidelines. A steel truss bridge has been considered to illustrate the approach. Two levels of capacity ratings- the upper load level capacity rating (called operating rating) and the lower load level capacity rating (called inventory rating) were found out using Load and Resistance Factor Design (LRFD) method and a proposal has been made which incorporates fatigue in the rating formula. Random nature of corrosion on the steel member has been taken into account in the rating by considering reduced member strength. Partial safety factor for each truss member has been obtained from the fatigue reliability index considering random variables on the fatigue parameters, traffic growth rate and accumulated number of stress cycle using appropriate probability density function. The bridge has been modeled using Finite Element software. Regressions of rating factor versus vehicle gross weight have been obtained. Results show that rating factor decreases when the impact factor other than those in the codal provisions are considered. The consideration of fatigue and member corrosion gives a lower value of rating factor compared to those when both the effects are ignored. In addition to this, the study reveals that rating factor decreases when the vehicle gross weight is increased.

**Keywords:** steel truss bridge; operating rating; inventory rating; Load and Resistance Factor Design; fatigue; traffic growth rate

## 1. Introduction

Rating of existing bridges is a continuous activity of an agency to ensure the safety of the structures and users. The evaluation provides necessary information to repair, rehabilitate, close, or reconstruct the existing bridges. In many countries, since highway bridges are designed for their design vehicles which are specified in the code, most of the engineers tend to believe that the bridge will have adequate capacity to handle the actual present traffic. This belief is generally true if the bridge was constructed and maintained as shown in the design plan. However, changes in a few details during the construction phase, failure to attain the recommended concrete strength, unexpected settlements of the foundation after construction, and unforeseen damage to a member

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could influence the capacity of the bridge. In addition, old bridges might have been designed for a lighter vehicle than the ones presently in use, or by a different design code. Also, the live-load-carrying capacity of the bridges may have altered as a result of deterioration, damage to its members, aging, added dead loads, settlement of piers, or modification to the structural member. Sometimes it may be necessary to transport the heavy machineries from one location to another location. In that case the current live-load-carrying capacity of the bridge is to be judged in order to ensure safety.

Biezma and Schanack (2007) studied different reason of steel bridge collapse. They found that 35% was due to accidental overload and impact. Schelling and Fu (1984) compared different methods of capacity rating and presented regression curves for practical applications. Cai and Shahawy (2003) discussed few issues related to capacity evaluation of existing bridges from field test results. Imam and Righiniotis (2010) investigated behavior of riveted stringer to-cross-girder connections in a typical, short-span bridge. Pipinato and Modena (2010) performed fatigue reliability assessment analysis of steel bridge along with traffic estimation, taking into account various scenarios of traffic increase in order to assess the possible remaining fatigue life.

Fatigue damaging effect was included in rating method by Mohammadi and Polepeddi (2000). Some researchers thought that various uncertainties involved in random parameters in bridge rating might be taken care of by introducing reliability index in rating factor (Akgul and Fragopol 2004). Studies on the similar line had been conducted by Sasmal *et al.* (2006) using Fuzzy mathematics. In most of the studies relating to capacity rating of bridges, live load was considered without studying dynamic time history of response of bridge due to passing vehicles and the effect of fatigue. The corrosion in exposed steel members especially in saline environment may also lead to the reduction of rating factor. Capacity loss in steel girder of a bridge due to corrosion has been studied for static load by Kayesar and Nowak (1987, 1989). Studies available so far on steel bridge rating have not addressed the factors involving cumulative dynamic effect of live load and environment in the evaluation of rating factors. In the present study, dynamic effect of moving load has been considered from the time history of stresses generated by the passage of vehicle at uniform speed. SAP2000 finite element package has been employed to model existing steel truss bridge. Upper load level capacity rating (operating rating) and lower load level capacity rating (inventory rating) both have been obtained using IRC Class 70 R, IRC Class AA wheeled (IRC 6: 2000) and HS20-44 (AASHTO 2008) load. Strength reduction in the truss member due to corrosion attack has been incorporated using resistance factor. Fatigue reliability index has been obtained by considering different random variables on fatigue detail parameter, Miner's critical damage index and traffic growth rate. Partial safety factor, calculated from fatigue reliability index has been used in the rating formula. Rating factors with and without fatigue for different loading have been compared.

## 2. Methodology

### *2.1 Rating principle*

In general, the resistance of a structural member ($R$) should be greater than the demand ($Q$) as follows

$$R \geq Q_d + Q_l + \sum_i Q_i \tag{1}$$

where, $Q_d$ and $Q_l$ are the effect of dead load and live load respectively where $Q_i$ is the effect of load *i*. Eq. (1) applies to design as well as evaluation. In the bridge evaluation process, maximum allowable live load needs to be determined. Maintenance engineers always question whether a fully loaded vehicle (rating vehicle) can be allowed on the bridge and, if not, what portion of the rating vehicle could be allowed on a bridge. The portion of the rating vehicle will be given by the ratio of available capacity for the live-load effect to the effect caused by the rating vehicle. This ratio is called the rating factor (*RF*) and is expressed as

$$RF = \frac{R - (Q_d + \sum_i Q_i)}{Q_l} \tag{2}$$

The basic equation for rating factor, Eq. (2), which has been used for a steel bridge is based on resistance of the members of the bridge and the demand for the rating vehicles. Codes in different countries have different live load standards and design rules to determine the resistance of the steel members against various combinations of loads. However the basic equations for the rating factor will be independent of live load standard adopted in any country. When the rating factor equals or exceeds unity, the bridge is capable of carrying the rating vehicle. On the other hand, when the rating factor is less than unity the bridge may be overstressed while carrying the rating vehicle. Thermal, wind, and hydraulic loads may be neglected in the evaluation process because the likelihood of occurrence of extreme values during the relatively short live-load loading is small. Thus, the effects of the dead and live loads are the only two loads considered in the evaluation process (*The Manual for Bridge Evaluation,* AASHTO 2008). However, the interaction of moving live load with bridge imposes a time varying force in the bridge that needs to be considered in a rating procedure. The dynamic effect of live load is incorporated in bridge design code by impact factor. For the present study, impact factor has been calculated from dynamic time histories for different vehicle speed. It is worth mentioning that impact factor found in codal provision does not specify any dependency on the vehicle speed. In the present study, a partial safety factor to account for the effect of fatigue damage occurring in the components has been incorporated. Strength reduction in the member due to corrosion also has been incorporated in the rating equation. In light of this concept, and in compliance with the LRFD approach (*LRFD Bridge Design Specification*, AASHTO 2007), the following rating equation has been proposed

Rating Factor using LRFD method is given by

$$RF = \frac{\phi R - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm \gamma_p P}{\gamma_{LL}(LL + IM)} \tag{3}$$

Rating Factor incorporating fatigue damage using partial safety factor ($\gamma_{fat}$), can be expressed as

$$RF = \frac{\phi R - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm \gamma_p P}{\gamma_{fat}\ \gamma_{LL}(LL + IM)} \tag{4}$$

in which *RF* is a rating factor; $\gamma_{DC}$, $\gamma_{DW}$, $\gamma_p$, $\gamma_{LL}$ are LRFD factor for structural components and attachments, wearing surfaces and utilities, permanent load other than dead load, Live load factor respectively; $\gamma_{fat}$ is the fatigue partial safety factor; *DC*, *DW*, *P*, *LL* are dead load effect due to structural components and attachments, wearing surfaces and utilities, permanent load other than self weight of the members, live load respectively; IM is a dynamic load allowance. *R*=Nominal member resistance; $\varphi$=Capacity reduction factor ($\varphi_c$. $\varphi_s$. $\varphi_n$); $\varphi_c$ = condition factor; $\varphi_s$= system factor; $\varphi_n$, a resistance factor, given in AASHTO LRFD bridge design specification (*LRFD Bridge Design Specification*, AASHTO 2007) as.

$$\phi_n = \frac{R_m}{R}\exp[-0.55\beta COV(R_m)] \tag{5}$$

where $R_m$, is mean resistance, $COV(R)$ coefficient of variation of $R$, β is the reliability index.

### 2.2 Effect of corrosion

Corrosion is one of the main causes of deterioration of steel bridges. The primary cause of corrosion is reaction between moisture and de-icing salt with steel members. The source of water and salt is either from deck leakage, from the accumulation of road spray or condensation of moisture from atmosphere. The rate and severity of corrosion depends on several environmental conditions such as temperature, relative humidity, time of exposure, salts, pollutants and conditions of applied coatings. Komp (1987) has evaluated rate of corrosion in different environmental conditions, such as rural, urban and marine. The rate of corrosion is found to be considerably increased in marine environment due to presence of salt. Two most common types of corrosion, viz, general corrosion and pitting corrosion are largely encountered in steel bridges. In the first type of corrosion, deterioration is distributed over large surface area while in the later, the deterioration is localized.

Corrosion related problems are considered to be the most important factors leading to age-related structural capacity loss of steel structures. A steel member subject to general corrosion has a random distribution of thickness over its area. The likelihood of these variations in thickness to form plastic hinges that may influence the buckling and ultimate strength of corroded member cannot be ruled out without further analysis. Khedmati *et al.* (2011) proposed a random thickness reduction model for ship and offshore structure. However, the main objective of this paper is to incorporate corrosion effect on a steel member in the rating of steel bridge. Reduction in the member strength due to corrosion attack has been taken into account using LRFD reduction factor which is given in Eq. (5). Reduced thickness ($t_p$) due to random corrosion occurring in both sides of the member is given by

$$t_p = t - d_w + r_1 + r_2 \tag{6}$$

in which $t$ is an original thickness of a member in millimeter; $r_1$ and $r_2$ are the random numbers, corresponding to the random thickness change of the members, produced by normally distributed function with zero mean and specified standard deviation as suggested by Ohyagi (1987). The same author has given an equation for uniform thickness reduction ($d_w$) as

$$d_w = 0.34\, n_y \tag{7}$$

where $n_y$ is the number of years of exposure and $d_w$ is the uniform reduction in thickness in millimeters after $n_y$ years of exposure.

### 2.3 Fatigue reliability assessment

The objective of reliability analysis in a structural member or system is to estimate its probability of failure. Its complement i.e., the probability that there would not be a failure is known as the reliability. Recognizing the role of resistance and load uncertainties in such calculations, it is convenient to construct a limit state function that differentiates between failed and safe states and can be mathematically expressed in terms of all of the known random variables. With well-established numerical techniques, it is then possible to estimate the probability of failure or reliability of the structural component or system under consideration.

### 2.3.1 Probability density function (PDF)

Fatigue calculations for steel bridges are generally developed based on loads arising from passing vehicles, especially single trucks. Due to the randomness of the actual traffic flow, vehicle-induced loads generate variable-amplitude stress ranges in bridge details. Most of the useful material properties for fatigue analysis have been derived from fatigue tests under constant-amplitude stress cycles. However, in case of bridges subjected to moving vehicle and excited by roughness of deck the nature of dynamic loading imposed in bridge is random. Several methods have been proposed to model variable-amplitude fatigue loadings for fatigue analyses. The most practical of these methods is based on statistical modeling that can help to derive an equivalent stress range by using the statistical distribution of the full loading spectrum. Such an equivalent stress range can then be used to characterize the variable-amplitude fatigue loadings as is commonly done for bridge structures. The ASCE Committee on Fatigue and Fracture Reliability (1982a-d) discussed possible use of the Rayleigh, Weibull, Beta, Polynomial, and lognormal distributions for fatigue analysis. Among this distribution, lognormal distribution has been used in the present study and its probability density function (PDF) is given as

$$f_{S_R}(S_R) = \frac{1}{\sqrt{2\pi}\zeta_{S_R}.S_R}\exp\left[-\frac{1}{2}\left(\frac{\ln S_R - \lambda_{S_R}}{\zeta_{S_R}}\right)\right] \tag{8}$$

where $\lambda_{SR}$ and $\zeta_{SR}$ are distribution parameters that can be estimated from the mean ($\mu_{SR}$) and the coefficient of variation ( $\delta_{SR}$) of the stress range data as follows

$$\lambda_{S_R} = \ln(\mu_{S_R}) - 0.5\zeta_{S_R}^2 \tag{9}$$

$$\zeta_{S_R} = \sqrt{\ln(1+\delta_{S_R}^2)} \tag{10}$$

A closed-form expression for the $m^{th}$ moment of $S_R$ with the lognormal distribution (ASCE Committee on Fatigue and Fracture Reliability 1982c) can be given as

$$E[{S_R}^m] = \mu_{S_R}^m (1+\delta_{S_R}^2)^{\frac{m(m-1)}{2}} \tag{11}$$

The equivalent stress range, $S_{RE}$, derived from the variable-amplitude stress range spectrum with the assumed lognormal distribution can be obtained as,

$$S_{RE} = \mu_{S_R}(1+\delta_{S_R}^2)^{(m-1)/2} \tag{12}$$

### 2.3.2 Limit state function

The limit state function employed in the AASHTO fatigue reliability approach is defined as follows

$$g(X) = N_c - N \tag{13}$$

where $N_c$ is the critical number of stress cycles it takes for the specified detail to achieve fatigue failure, and $N$ is the total accumulated number of stress cycles applied on the detail. By definition, $g$(X) > 0 implies that the detail has not failed due to fatigue. The failure is assumed to occur when $g$(X) ≤ 0. It can be seen that the limit state function, given by Eq. (13), is directly related to the number of stress cycles.
The total number of cycles to failure is related to stress range amplitude. This is given by popular *S-N* relation as

$$N_f = AS_R^{-m} \tag{14}$$

where, $A$ and $m$ are material constant; $N_f$ and $S_R$ are number of cycle to failure and stress range amplitudes respectively. Miner (1945) proposed a linear damage accumulation theory to account for effects of fatigue on structural components or details subjected to variable-amplitude loading. Miner's damage accumulation index, $D$, is defined as follows

$$D = \sum_{i=1}^{k} \frac{n_i}{N_{f,i}} \tag{15}$$

where $n_i$ *is* the actual number of cycles associated with a specific stress range level, $S_{R,i}$, and $N_{f,i}$ is the number of cycles until failure under a constant-amplitude stress range level, $S_{R,i}$. Combining the *S-N* relation Eq. (14) and Miner's damage accumulation rule Eq. (15) for fatigue details under variable-amplitude stress ranges, the following expression can be obtained.

$$D = \frac{N}{A} S_{RE}^{m} \tag{16}$$

where, $D$ is Miner's damage accumulation index, $N$ is the total number of accumulated stress cycles, $A$ is a fatigue detail constant parameter in the AASHTO *S-N* relation; $m$ is the fatigue exponent in the S-N curve, and $S_{RE}$ is the Miner's equivalent stress range defined from the stress spectrum. According to Miner's rule, fatigue failure occurs when the damage accumulation index, $D$, reaches a critical value, Δ, which means

$$D \geq \Delta \tag{17}$$

where Δ (Miner's critical damage accumulation index) has a value approximately equal to 1.0 for metallic materials. Combining Eq. (16) and Eq. (17), the critical number of stress cycles, $N_c$, needed for fatigue failure under variable-amplitude loading with equivalent stress range, $S_{RE}$, can be expressed as

$$N_c = \frac{A.\Delta}{S_{RE}^{m}} \tag{18}$$

Hence, the limit state function for the AASHTO fatigue reliability approach can be rewritten as

$$g(X) = \left( \frac{A.\Delta}{S_{RE}^{m}} \right) - N \tag{19}$$

The accumulated number of stress cycles, $N$, is related to the traffic volume, particularly truck traffic volume, passing over the bridge and can be transformed into the number of years in service, $Y$. However, the transformation from $N$ to $Y$ should, in general, consider uncertainty in the traffic.

Fatigue reliability analysis can be implemented using the limit state function Eq. (19) when the related variables are completely described. A description of all of these related variables is presented next.

#### *2.3.3 Variables in the limit state form*

##### *2.3.3.1 Fatigue detail parameter (A)*

In logarithmic form Eq. (14) can be expressed as

$$\log_{10} N_f = -m.\log_{10} S_R + \log_{10} A \tag{20}$$

Fisher *et al*. (1970) studied fatigue test data from 374 steel beams and concluded that $log_{10} N_f$ can be assumed to follow a normal distribution. In the present study, $\log_{10} A$ is assumed to follow a normal distribution and $A$ as lognormal distribution.

##### *2.3.3.2 Miner's critical damage accumulation index (Δ)*

Miner's critical damage accumulation index in terms of resistance has been assumed as Lognormal with mean value of 1.0 and coefficient of variation (COV) of 0.3 for metallic materials (Wirsching 1984)

##### *2.3.3.3 Accumulated number of stress cycles (N) and number of years in service (Y)*

The passage of single trucks on steel bridges is the primary source of cyclic loading that can generate stress cycles and can cause fatigue damage. Truck passages and number of years in service are related to the accumulated number of stress cycles (Moses *et al* 1987) as follows

$$N(Y) = 365\ (ADTT)_{SL}\ C_s\ Y \tag{21}$$

where $N(Y)$ is the number of years in service, $(ADTT)_{SL}$ is the single-lane average daily truck traffic on the bridge, $C_s$ is the stress cycles per truck passage for the bridge span where the detail of interest is located and $Y$ is the number of years in service for the bridge. Both $(ADTT)_{SL}$ and $C_s$ can be taken as random variables in the fatigue reliability analysis. Moses *et al.* (1987) suggested that *ADTT* and $C_s$ can be treated as lognormally distributed random variables with coefficients of variation of 0.1 and 0.05 for *ADTT* and $C_s$, respectively. Mean values for *ADTT* and $C_s$ are estimated by using AASHTO LRFD (*LRFD Bridge Design Specification*, AASHTO 2007) specification. Since traffic growth is continuous phenomenon, a growing *ADTT* model can be formulated by considering allowance for the growth of truck traffic with time and a relationship between the accumulated number of stress cycles ($N$) and the number of years in service ($Y$) can be expressed as follows

$$ADTT(Y) = (ADTT)_0\ (1+r)^Y \tag{22}$$

$$N(Y) = 365 C_s \int_0^Y ADTT(y)\,dy = 365\ C_s\ ADTT_0 \left[\frac{(1+r)^Y - 1}{\ln(1+r)}\right] \tag{23}$$

where $(\mathrm{ADTT})_0$ is the average daily truck traffic in the first year in service and $r$ is the annual truck traffic growth rate. Both $(ADTT)_0$ is taken as log-normally distributed random variables with 72 mean values and 0.3 coefficient of variation while $r$ can be taken as normally distributed random variables with 5% mean value and 0.3 as coefficient of variation in the fatigue reliability analysis (Moses *et al.* 1987).

### 2.3.4 Evaluation of the fatigue reliability index (β)

Reliability of structural components is related to the probability of not violating a particular limit state. Based on the limit state given in Eq. (13), the failure probability of a structural member is defined as

$$P_f = P\,(g(X) < 0) \tag{24}$$

The reliability index, $\beta$ that is related to the probability of failure can be defined as

$$\beta = \Phi^{-1}(1 - P_f) \tag{25}$$

where $\Phi^{-1}(\,)$ denotes the inverse standard normal cumulative distribution function (CDF).
Based on the function $g(X)$ given in Eq. (20), the fatigue reliability index, $\beta$, can be derived (Kwon and Frangopol 2010), assuming that all random variables (i.e., $A$, $\Delta$, $C_s$ $S_{RE}$ and $ADTT$) are lognormal as follows

$$\beta = \frac{(\lambda_A + \lambda_\Delta - \lambda_{Cs} - \lambda_{ADTT}) - m\ln(S_{RE}) - \ln(365) - \ln(Y)}{\sqrt{\zeta_A^2 + \zeta_\Delta^2 + \zeta_{Cs}^2 + \zeta_{ADTT}^2}} \tag{26}$$

where

$$\zeta_y = \sqrt{\ln(1 + \delta_y^2)} \tag{27}$$

$$\lambda_y = \ln(\mu_y - \frac{\zeta_y^2}{2}) \tag{28}$$

in which $\zeta_y$ and $\lambda_y$ denotes the mean value and standard deviation of $ln\ y$ (i.e., $y = A$, $\Delta$, $C_s$ $S_{RE}$ and $ADTT$), respectively .

## 2.4 Partial safety factor

Adoption of a format makes the designer to determine partial safety factors to resistance, $\gamma_S$, and partial safety factor to load, $\gamma_R$ , for desired reliability level. Smith and Hirt (1987) proposed a safety format for calibrating constructional steel works. For safety, one can express

$$\frac{S_R}{\gamma_R} \geq \gamma_S S_{RE} \tag{29}$$

The fatigue strength $S_R$ is defined by the $S$-$N$ curve corresponding to the detail/joint which is evaluated. The equivalent stress range $S_{RE}$ which is defined earlier has been obtained from the resulting stress time histories due to the application of design load spectra and applying the rainflow counting method of cycle counting.
Total uncertainty in load is represented by $\delta_{SRE}$. In the lognormal safety format all variables are assumed to be log normally distributed. The partial safety factor for both resistance $\gamma_S$ and load $\gamma_R$ has been related to fatigue reliability index (β) as (Ranganathan 1999)

$$\log(\gamma_S \gamma_R) = \frac{\beta \sigma_t}{m} - 2\sigma_R \tag{30}$$

where

$$\sigma_t = \sqrt{(m\sigma_R)^2 + (\sigma_Q)^2} \tag{31}$$

$$\sigma_R = \sigma_{\log N} = \sqrt{0.4343\log(1+\delta_N^2)} \tag{32}$$

$$\sigma_Q = \sigma_{\log SE} = \sqrt{0.4343\log(1+\delta_{SE}^2)} \tag{33}$$

in which $\delta_N$ and $\delta_{SE}$ are the coefficient of variation of accumulated number of stress cycle $N$ and stress range $S_{RE}$ respectively. Since in fatigue design, *S-N* curves is drawn at mean minus two times the standard deviations to take care of variation in resistance $R$, the value of $\gamma_R$ is taken as 1 (Ranganathan 1999).

## 3. Modeling and analysis of bridge

The Finite Element modelling of steel truss bridge having a span of 67.5 m has been developed using SAP2000 version 14. The mechanical properties of all materials are summarized in Table 1. The bridge roadway deck was modeled with diaphragm constraints. Steel truss elements top and bottom chords, cross girders, diagonals, stringers, abutment walls, and abutment footings are model with frame elements. The frame element normally activates all the six degree of freedom at the both of its connected joints. The rigid links were used to connect between the cross girders and stringers, top laterals and portal beams, slab and cross girders, abutment wall and abutment footing. The bearing between the girders and cap beams were modeled using body constraints. Adjacent soil of the abutment and foundation has been model as Winkler spring. Stiffness of soil was calculated as sub-grade modulus reaction of soil (80,000 kN/m$^3$ for clayey medium dense sand) multiplied by mesh area (Thanoon *et al.* 2011). Abutment and foundation concrete have been chosen as Grade M20 corresponding to characteristic strength at 28 days as 20 MPa. Three basic analyses has been performed, static, modal and dynamic analysis using sap 2000 version 14.

Table 1 Sectional and material properties of the bridge

| Sl. No | Structural component | Sectional Properties of Truss Members (Bhavikatti 2009) | Yield strength $f_y$ for steel or characteristic strength $f_{ck}$ (MPa) for concrete | Modulus of elasticity $E_s$ (GPa) |
|---|---|---|---|---|
| 1 | Top chord | 2 ISMC 400, with 2 FLG PL. 120x12 mm THK (T&B) | 250 | 200 |
| 2 | Bottom chord | 2 ISMC 400, with 2 FLG PL. 120×12 mm THK (T&B) | 250 | 200 |
| 3 | Vertical members | ISMC 200, 400 mm B/B | 250 | 200 |
| 4 | Diagonal members | ISMC 250, 400 mm B/B | 250 | 200 |
| 5 | Stringer | ISMB 250 | 250 | 200 |
| 6 | Cross girder | ISMB 400 with 2 FLG PL 12 mm thick | 250 | 200 |

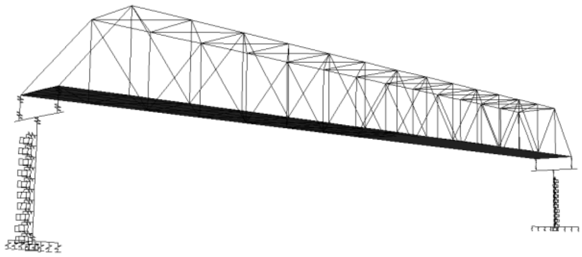


Fig. 1 Finite element modeling of bridge with soil spring

### *3.1 Static analysis*

Static analysis of steel truss bridge has been performed using SAP2000 version 14. The loading considered in the study are of three types: IRC 70 R, IRC Class AA wheeled vehicle (IRC 6: 2000) and AASHTO loading HS20-44 (AASHTO 2007) as mentioned earlier. The static analysis has been done to obtain the effect of Dead load by assigning material properties of each component.

### *3.2 Dynamic analysis*

Modal analysis has been performed to determine the vibration modes of a structure and its natural frequencies and damping ratios. These modes are useful to understand the dynamic behavior of the structure under dynamic loading and to bring out meaningful interpretation of many results. The most important factor to cause structural fatigue damage is stress fluctuation, which mainly induced by traffic loading.

First four natural frequencies of the steel bridge under consideration are 1.7, 2.05, 2.9 and 5.08 Hz. The information of natural frequencies is useful to understand resonance behavior of the structure under forced vibration. Modal damping ratio ($\zeta$) has been ascertained from different natural frequencies according to Raleigh's damping criteria (Chopra 2007) and found to be 0.5% in the present case. In the present paper, however, direct integration scheme has been used instead of modal superposition technique (Bathe 1985) to find out response time history of the bridge component. Three different loadings, IRC 70 R, IRC Class AA wheeled vehicle and AASHTO loading (HS20-44), were employed. The time-history of stresses due to uniform vehicle speed in the range of 40 km/h to 100 km/h were obtained by Newmark's direct integration scheme (Bathe 1997). The time period corresponding to first natural frequency is a useful parameter to decide appropriate time step in Newmark scheme. In rating formula dynamic effect has been incorporated by using Impact factor obtained by dynamic simulation of three dimensional bridge models as {1+ ratio of maximum dynamic response to maximum static response}.

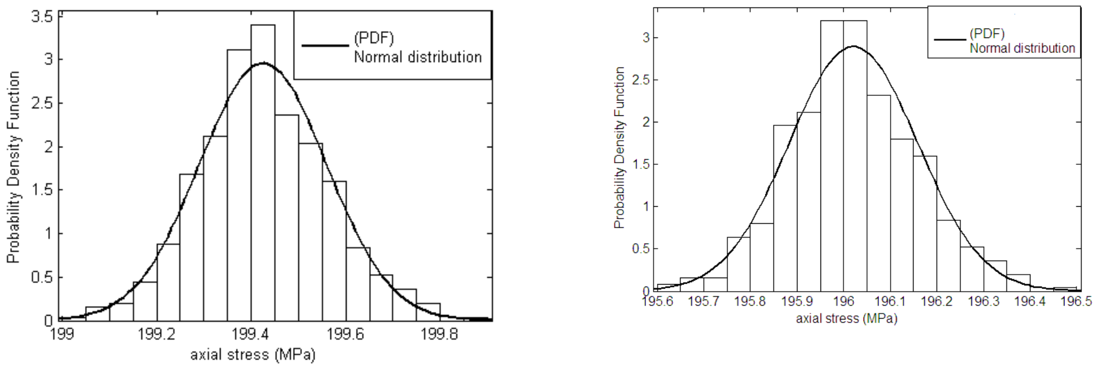
## 4. Result and discussion

In the present model, steel truss girders are main load carrying members. In a truss bridge under dead load and live load, the bottom members mostly remain in tension whereas top members carry major portion of the compressive loads. The diagonal members are subjected to the reversal of stress under moving load and therefore capacity should be checked for both compression and tension. When effect of random corrosion is considered, a reduction of sectional area will decrease the geometric properties and this change will occur in non linear fashion. Buckling capacity will be critically effected due to corrosion in the member. In rating formula presented in Eq. (3), one needs to obtain member capacity of the steel truss bridge. From the geometry the nominal member resistance ($R$) for axial - tensile and compressive stress has been calculated using Load and resistance Factor Design Method (*LRFD*). Condition factor ($\varphi_c$) = 0.95 (for fair condition), System factor ($\varphi_s$) = 0.9 (for truss bridge) have been considered. Strength reduction due to random corrosion of a member has been incorporate using *LRFD* resistance factor ($\varphi_n$) given in Eq. (5). The thickness reduction in a member due to random corrosion given in Eq. (6) makes the axial resistance of a member as random variable. The best probability function fit has been obtained and it has been found that normal distribution function is the best fit for all the members. Fig. 2 and Fig. 3 shows the probability function fit for a vertical member, diagonal member; top chord and bottom chord respectively. The resistance factor ($\varphi_n$) given in Eq. (5) has been calculated using normal distribution parameters -Mean value of resistance $R_m$, COV ($R_m$), which are shown in Table 2. Dead load factor ($\gamma_{DC}$) =1.25, Dead load factor for wearing surface and utilities ($\gamma_{DW}$) = 1.5, Live load factor ($\gamma_{LL}$) is taken as 1.35 and 1.75 for operating and inventory rating respectively.

Every joint is riveted connection that falls into detailed category 'D' as per AASHTO-LRFD Bridge Design Specification. In order to obtain mean and coefficient of variation for Random Accumulated Number of Stress Cycles ($N$), the random variable of $ADTT_0$ and $r$ have been generated using distribution parameters given in section 2.3.3, which were substituted in Eq. (23).

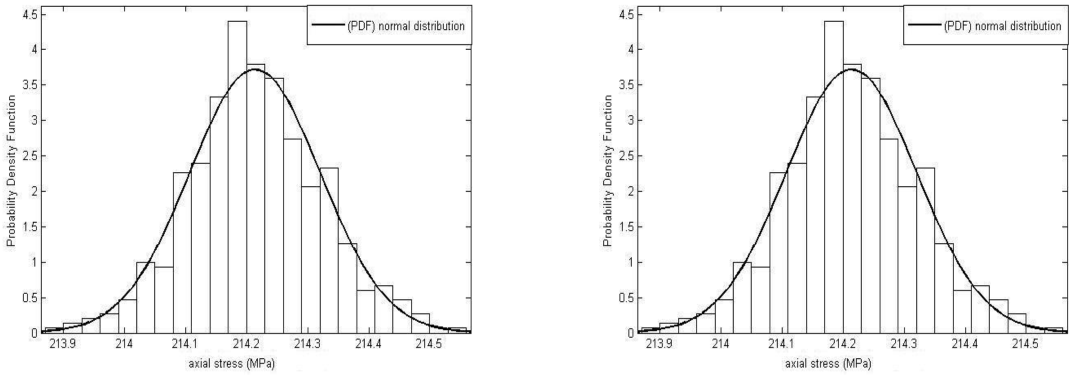
Table 2 Resistance factor ($\varphi_n$) for a different member

| Member | Mean value of resistance ($R_m$) (MPa) | COV ($R_m$) | Resistance factor ($\varphi_n$) |
|---|---|---|---|
| Verticals | 199.425 | 0.0067 | 0.957 |
| Diagonals | 196.021 | 0.0075 | 0.951 |
| Top chord | 214.214 | 0.0054 | 0.994 |
| Bottom chord | 214.214 | 0.0054 | 0.994 |




(a) Vertical member (b) Diagonal member

Fig. 2 Probability function fit for axial stress




(a) Top chord (b) Bottom chord

Fig. 3 Probability function fit for axial stress

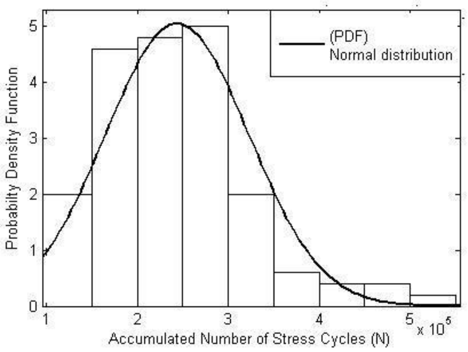
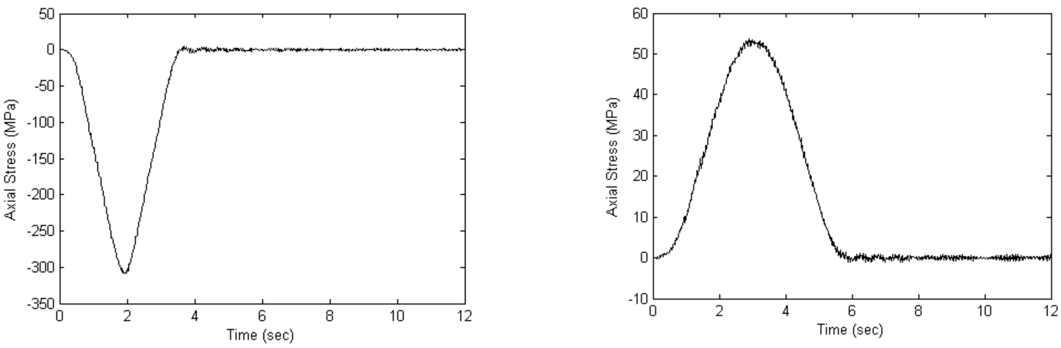


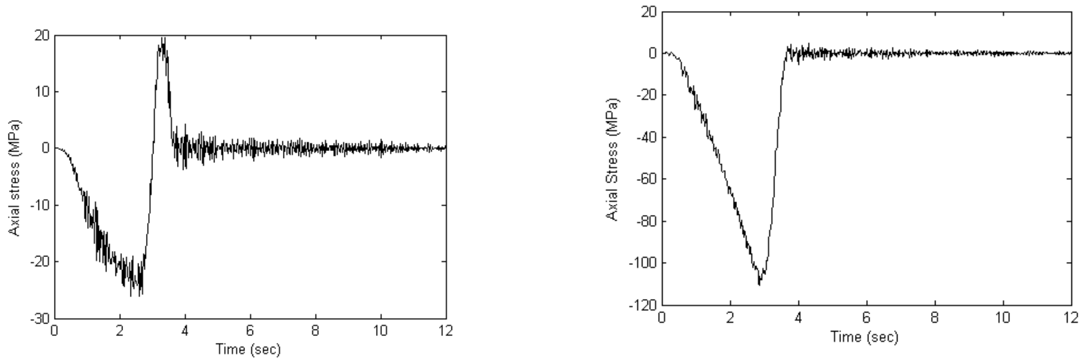

Fig. 4 Probability function fit for a ccumulated number of stress cycles ($N$)




(a) Top chord (b) Bottom chord

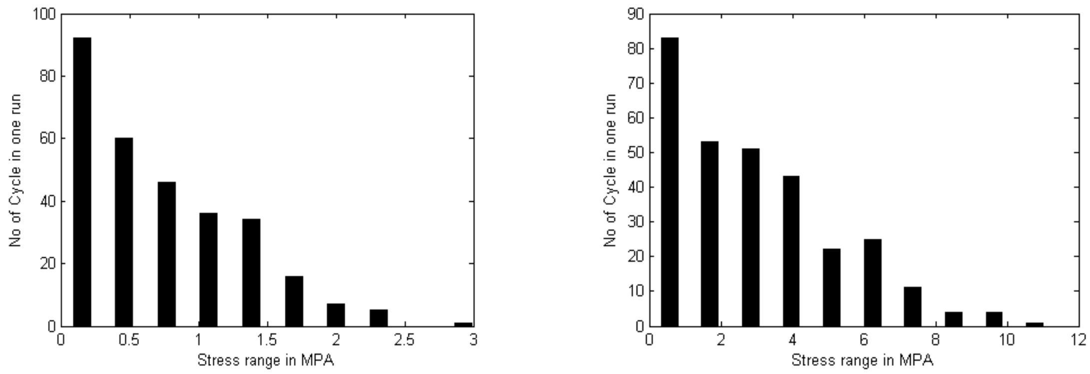
Fig. 5 Axial force time history due to passage of IRC 70 load

The best probability distribution function fit for Accumulated Number of Stress Cycles ($N$) has been found and presented in Fig. 4. Result shows that it follows normal distribution with mean value ($\mu_N$) as $2.47\times10^5$ and coefficient of variation ($\delta_N$) as 0.032.




(a) diagonal member (b) vertical member

Fig. 6 Axial force time history due to passage of IRC 70 load

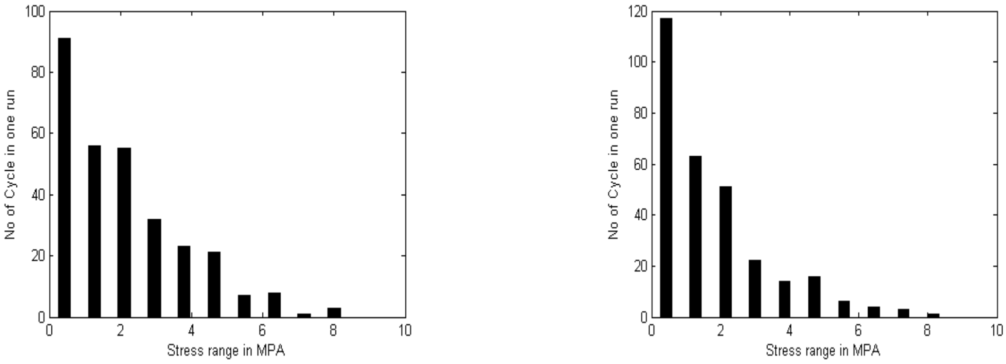



(a) top chord (b) bottom chord member

Fig. 7 Axial stress range histogram subjected to IRC 70 R loading with 80 km/h speed

Dynamic time-history analysis subjected to a standard vehicle (as mentioned earlier) moving at uniform speed in the range of 40 km/h to 100 km/h has been performed to obtain axial stress time history for different member of the bridge. An axial stress time history for a bottom chord, top chord, diagonal and vertical members have been shown in Fig. 5 and Fig. 6 subjected to IRC 70 R loading traveling along the span at 80 km /h speed.

As expected, bottom members and top chord members remain in tension and compression respectively, whereas stress reversal has been noticed in diagonal members. For this vehicle speed, time span of 12 sec has been considered for predicting fatigue damage. It may be noted after vehicle leaves the bridge, free vibration takes place at diminishing amplitude. Free vibration time around 9 seconds is also captured in the time history. Cycle counting has been conducted using Rainflow method (Dowling 1972) and stress range histogram for a critical bottom chord, top chord, diagonal as well as vertical members have been presented in Fig. 7 and Fig. 8.




(a) diagonal member (b) vertical member

Fig. 8 Axial stress range histogram subjected to IRC 70 R loading with 80 km/h speed

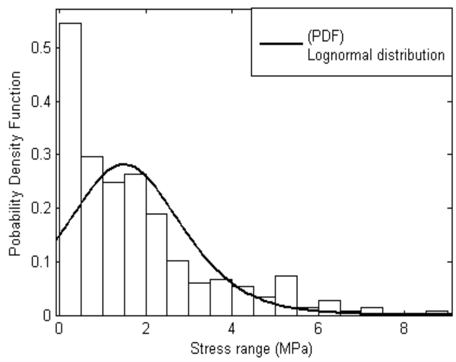



Fig. 9 Stress range histogram and PDF for vertical member subjected to IRC 70 R with vehicle speed 80 km/h

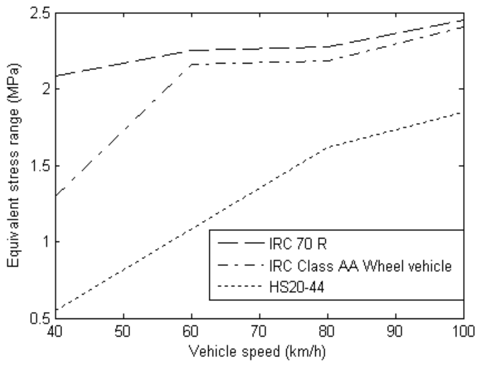



Fig. 10 Equivalent stress range for vertical member loaded with three standard vehicles with different speed in the range of 40 km/h to 100 km/h

Equivalent stress ($S_{RE}$) range for different member of a bridge has been calculated using Eq. (12) for different loading with different vehicle speed. Fig. 9 shows the stress range histogram and

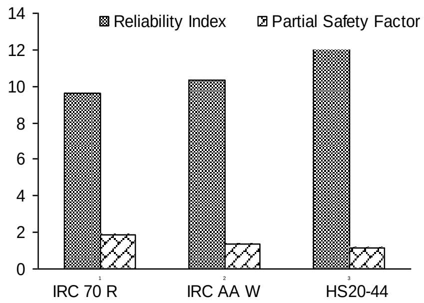
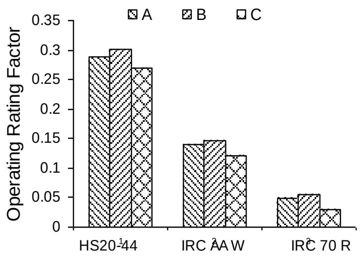
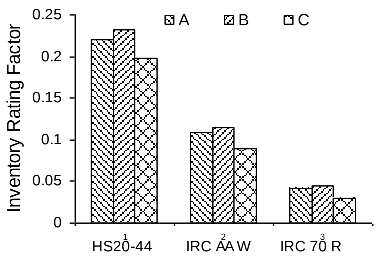



Fig. 11 Comparison among the vertical member's Fatigue reliability index ($\beta$) and partial safety factor ($\gamma_{fat}$) with three standard vehicles IRC 70 R, IRC AA Wheel vehicle and HS20-44




(a) Operating rating factor



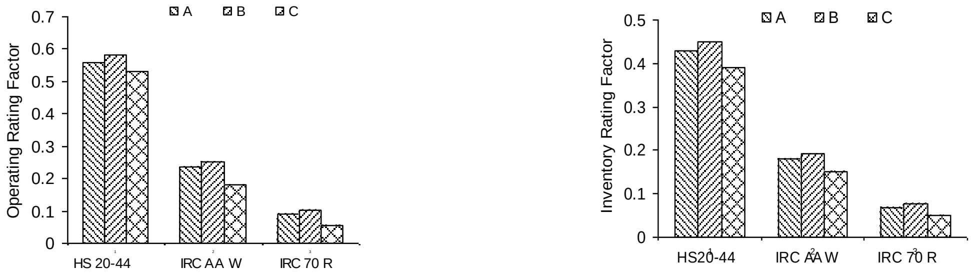

(b) Inventory rating factor

Fig. 12 Rating factor for top chord member using impact effect from (A) IRC (B) AASHTO (C) Present dynamic analysis

probability density functions for vertical member subjected to IRC 70 R with vehicle speed 80 km/h and Fig. 10 shows that the comparison among different equivalent stress range for vertical member loaded with three standard vehicles IRC 70 R, IRC Class AA Wheel vehicle and AASHTO loading (HS20-44) having the gross weight of 700 kN, 400 kN, 320 kN respectively with different speed in the range of 40 km/h to 100 km/h.

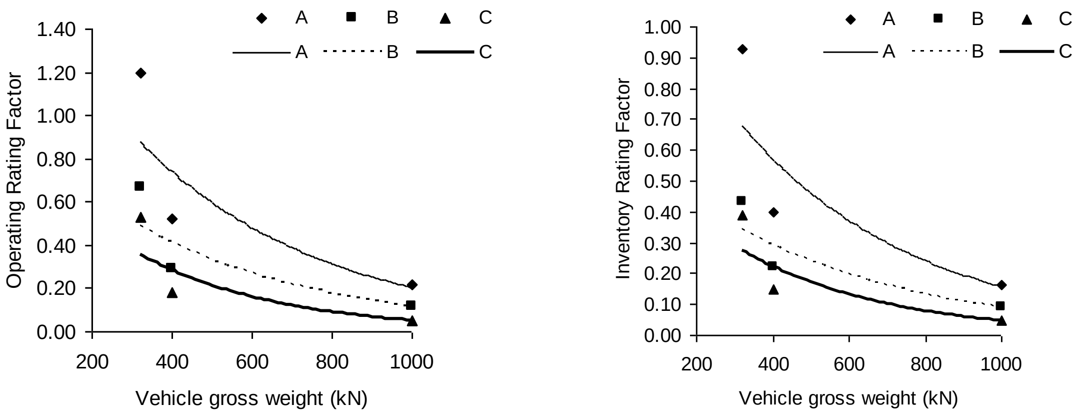
Fatigue reliability index ($\beta$) and partial safety factor has been calculated using Eq. (26) and Eq. (30) respectively. Fatigue reliability index ($\beta$) and partial safety factor ($\gamma_{fat}$) of vertical member loaded with three standard vehicles has been compared in Fig. 11.

Two levels of rating- operating and inventory has been obtained for each truss member loaded with three different standard loadings: HS20-44, IRC Class AA wheeled and IRC 70 R vehicle, only top chord and vertical member's rating results have been shown in Fig. 12 and Fig. 13. Dynamic effect on rating has been examined by incorporating impact factor from: (i) IRC code (ii) AASHTO provisions (iii) present dynamic analysis. The simulated results have been used to calculate impact factor by estimating a ratio of maximum total stress (static + dynamic) to that of static stress for each member. However, other codal provisions viz., IRC and AASHTO ignore vehicle speed in impact factor formulae, although AASHTO has provision to incorporate surface roughness condition (*The Manual for Bridge Evaluation*, AASTO 2008). IRC impact factor for wheeled vehicle in steel bridges is 25% up to a bridge span of 23 m and beyond 23 m span, Impact factors are to be read from the chart based on the span only (IRC 6: 2000)




(a) Operating rating factor (b) Inventory rating factor

Fig. 13 Rating factor for vertical member using impact effect from (A) IRC (B) AASHTO (C) Present dynamic analysis




(a) Operating rating factor (b) Inventory rating factor

Fig. 14 Regression curves for vertical member rating factor with three different conditions (A) neglecting both the effects of corrosion and fatigue, (B) considering only corrosion effect and (C) considering both the effects

A regression analysis of the rating factor (both operating and inventory) versus vehicle gross weight, as explained earlier, has been carried out for three conditions: (1) neglecting both corrosion and fatigue effect, (2) considering only corrosion effect and (3) considering both the effects. Fig. 14(a) and Fig. 14(b) show the operating and inventory rating factor regression curves obtained for vertical member only. The analysis showed that rating factors decrease as the vehicle gross weight increase. In addition to this, at the same vehicle gross weight, rating factor is considerably reduced when the effect of fatigue and corrosion are considered in the rating. The correlation between rating factor (*RF*) and the vehicle gross weight (*W* in kN) for operating rating has been found as: (1) neglecting both corrosion and fatigue effect, *RF*= 1.739 exp(-0.0021W), ($R^2$ = 0.848); (2) considering only corrosion effect, *RF*= 0.964exp(-0.0021W), ($R^2$ = 0.848); and (3) considering both the effects, *RF*= 0.892 exp(-0.0029W), ($R^2$ = 0.863). Similarly, regression analysis has yielded the correlation of inventory rating with vehicle gross weight as *RF*= 1.342exp (-0.0021W), ($R^2$ = 0.848); *RF*= 0.64 exp (-0.002W), ($R^2$ = 0.892) and *RF*= 0.632exp (-0.0026W), ($R^2$ = 0.869) for the three conditions respectively

## 5. Conclusions

In the present paper, rating factor of the steel truss bridges has been found incorporating dynamic effect of moving vehicle in the form of impact factors under the consideration of fatigue due to random stress produced by moving vehicles at different speeds. The corrosion of exposed steel member has also been incorporated in the analysis assuming randomness in the reduction of thickness. The dynamic analysis has been carried out in Finite Element frame work with three standard classes of vehicles. It has been found that rating factor is higher when the codal provision of impact factor has been used in the rating formula compared with those obtained from the time history of stresses in present dynamic analysis. Comparison of rating factors shows that present dynamic analysis yields an average of 10% to 30% lower values compared to that obtained by using impact factors from codal provisions. Strength reduced in the truss member due to corrosion has been incorporated in the rating formula using LRFD reduction factor. Fatigue has been taken in to account in the rating by partial safety factor which is calculated from the fatigue reliability index. Fatigue reliability index has been found for each member by considering random variables, assuming appropriate probability density function, on -fatigue detail parameters ($A$), Miner's critical damage index ($\Delta$) and accumulated number of stress cycle ($N$) which is obtained from the consideration of random traffic growth rate and random stress cycle per truck passage. Result shows that reliability index decreases with increasing vehicle gross weight while the partial safety factor increases with increasing vehicle gross weight. Comparison of rating factor shows that considering the effects of corrosion on the member and fatigue yields an average of 56% to 70% lower values compared to that obtained by neglecting both the effects. Regression of rating factor versus vehicle gross weight has been obtained which reveals that both the operating and inventory rating factor decreases with the increase in vehicle gross weight.

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