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SESSION 2

Rating and Evaluation of Remaining Life of Bridges

Evaluation de la durée de vie des ponts

Schätzung der Lebenserwartung von Brücken

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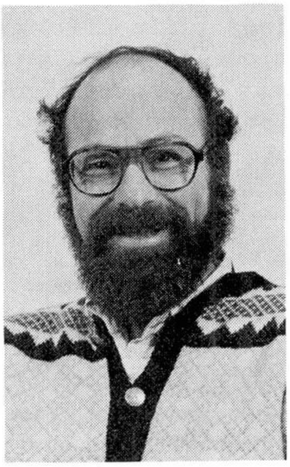
Load Spectra for Bridge Evaluation

Spectre de charges pour l'évaluation des ponts

Lastspektren für die Bewertung bestehender Brücken

Fred MOSES
Prof. of Civil Engineering
Case Western Reserve Univ.
Cleveland, OH, USA



Fred Moses received a civil engineering degree from CUNY in 1960 and a Ph. D. from Cornell in 1963. Since joining Case Institute, Professor Moses has been active in structural optimization and reliability with special applications to bridge and offshore structures including loading, safety analysis and system reliability studies.

SUMMARY

Evaluating existing bridges can be more complex then designing new structures. It is suggested herein that bridge inspections should include load history as well as bridge condition. A recently developed weigh-in-motion technology reduces uncertainty by accurately determining records of truck weights, bridge response and repetitive stress-spectra. Reliability predictions can further assist decision-making by modelling fatigue failure and overall fail-safe capacity. Applications include inspection, posting, legal limits, enforcement, rating and permit assessments. Such evaluation-related problems can all benefit from improved load modelling and site-specific loading statistics formulated into a reliability model.

RESUME

L'évaluation de ponts existants peut être plus complexe que le calcul de nouvelles constructions. L'inspection de ponts devrait inclure l'étude des cas de charges antérieures ainsi que de l'état du pont. Une technologie récente, nommée „weigh-in-motion", est basée sur la détermination exacte du poids des camions, le comportement du pont et le diagramme des charges répétitives. Des prédictions fiables facilitent la décision par la création de modèles de rupture à la fatigue et de capacité globale rupture-sécurité. La méthode tient compte de l'inspection, de la signalisation, des limites légales, et des charges autorisées. De tels problèmes d'évaluation peuvent être étudiés à l'aide d'un modèle de charge et de statistiques de charges exprimées en un modèle de sécurité.

ZUSAMMENFASSUNG

Die Bewertung bestehender Brücken kann umfassender sein als die Projektierung neuer Brückenbauten. Im vorliegenden Bericht wird vorgeschlagen in der Brückenüberwachung auch die Lastenentwicklung und den Brückenzustand einzuschliessen. Eine neu entwickelte „weigh-in-motion" – Technologie vermindert Unsicherheiten durch eine sorgfältige Bestimmung der Lastwagengewichte, der Antwort – und der Spannungsspektren. Zuverlässigkeitsvoraussagen können weiter zum Entscheid beitragen, indem Modelle für das Ermüdungsversagen und die umfassende „failsafe"-Kapazität geschaffen werden. Die Anwendungen beinhalten Überwachung, Standort, die gesetzlichen Grenzen, Durchsetzbarkeit, Bewertung und Einschätzungserlaubnis. Solche Bewertungsmodelle können profitieren von verfeinerten Lastmodellen und objektbezogenen Belastungsstatistiken, dargelegt in einem Zuverlässigkeitsmodell.

1. INTRODUCTION

Repair, posting and replacement of bridge structures requires high expenditures. Such decision must distribute limited available resources considering public economy, safety and utility. The decision process reflects past experience, current technologies, cost limitations and future needs. Because safety is implicitly involved, risk estimations are present. The limited data and cost of acquiring more information to assist decision-making is important. New developments in low-cost data gathering which reduce uncertainties must be explored.

Bridge evaluation and rating combines field information and calculation models. At present, strength estimates are compared to load calculations to check acceptable allowable stress levels. The assessment uncertainties and reliability may be different from such parameters in new designs. This paper suggests that the checking and calculations for rating, repair and strengthening of existing bridges be altered based on bridge site, geometry, traffic and loading conditions.

New technological developments in data gathering and broad philosophical changes in design codes of practice should now be considered in bridge assessment and evaluation. The data gathering refers to automated methods for rapidly and economically acquiring truck load information. The design technology includes reliability methods for calibrating acceptable safety margins. Advantages include a consistent basis for expressing load and strength uncertainties and improved economy for structures with high dead to live ratios typical of longer spans and older structures. AASHTO load factor design provisions were adopted to move towards these goals [1] and there is further study to refine design safety factors to reflect current heavy truck traffic and loads [2]. A reliability-based framework can produce significant benefits when assessing existing bridges. The issues to be resolved in these evaluation applications include the following:

- An existing bridge has a loading spectra that can be measured rather than extrapolated from planning models.

- Analysis assumptions such as load distributions and dynamic behavior may be verified by experimental observation. Also, self-weight can be estimated more accurately.

- Ultimate capacity rather than serviceability may be acceptable criteria for existing bridges.

- The optimum economic reliability changes for an existing structure compared to a new design. The cost of increased strength margins are usually much lower for new constructions and so the trade-off equations are different.

This paper primarily reviews two new developments to aid bridge assessment and rating:

1) The application of newly developed weigh-in-motion technology to obtain current traffic, loading and other bridge response data [3-6].

2) The use of reliability design methodology to aid the structural decision process [2, 7, 8].

2. LOADING ANALYSIS

For most short and medium span bridges, the critical loading is self-weight and heavy truck traffic. Self-weight can be accurately estimated from cores and recorded dimensions. Repetitive heavy vehicle load cycles, however, may induce fatigue damage, cracks and ultimately collapse. It is not uncommon for wheel load sensitive details to experience many millions of stress cycles. Main load carrying members also experience millions of load cycles as well as extreme occurrences that may cause instability, permanent displacement or collapse.

Each live load occurence depends on truck weight and dimensions, dynamic impact and intervals of adjacent vehicles (headways). In a critical component, stress range depends on load distribution and bridge dynamics which in design are estimated from simplified models. Present load specifications also reflect heavy truck traffic in existence many decades ago. Changes in truck traffic including heavier legal and permit vehicles and other modern trends are important. Comparisons should include:

- Increased gross weights. Unless accompanied by longer axle lengths, heavier vehicles induce greater longitudinal bending moments.

- Influence of closely spaced axles. Increased tandem and triaxial weight combinations significantly affect component stresses sensitive to concentrated wheel loads.

- Lighter bridges. Such recent designs are more prone to higher impact and dynamic response.

- Traffic increases. The frequency of platoons of closely spaced vehicles, superimposing their load effects, increases with higher volumes.

- Enforcement. There is concern that CB communication and by-pass options has decreased legal load enforcement. Little is also known about the efficiency of posting signs in restricting loads.

- Bridge lives. It is evident that initial estimates of 40-70 years for bridge lives are being surpassed. The current economic climate suggests little improvement in this regard.

2.1 Design Loads

Modern developments in bridge load modelling have produced changes in some design codes. The 1979 Ontario Highway Bridge Design Code completely revised the existing load tables [9]. Figure 1 compares the present AASHTO (U.S. - [1]) and Ontario design simple span longitudinal bending moments. The much larger Ontario mements were matched to loadometer data obtained in the early 1970's combined with a simulation model of truck headways [10] (The equipment available then did not permit undetected weighing or precise vehicle spacings. Such study is underway to further verify the loading models [11]). Other countries have also altered their loadings. In Great Britain, a fatigue spectra provided from field studies is used to check damage on a 120 year life estimate [12].

In the United States, several studies measured bridge stress spectra. Results were incorporated in the AASHTO fatigue checking provisions [13]. For example, a study for Ohio DOT and FHWA Surveyed 10 sites to give data on stress spectra, spacing behavior, dynamic response and girder analysis variations [14]. Recently, the electronic and computer equipment permits correlating bridge stresses with truck weight. This provides more accurate bridge loading data and the statis-

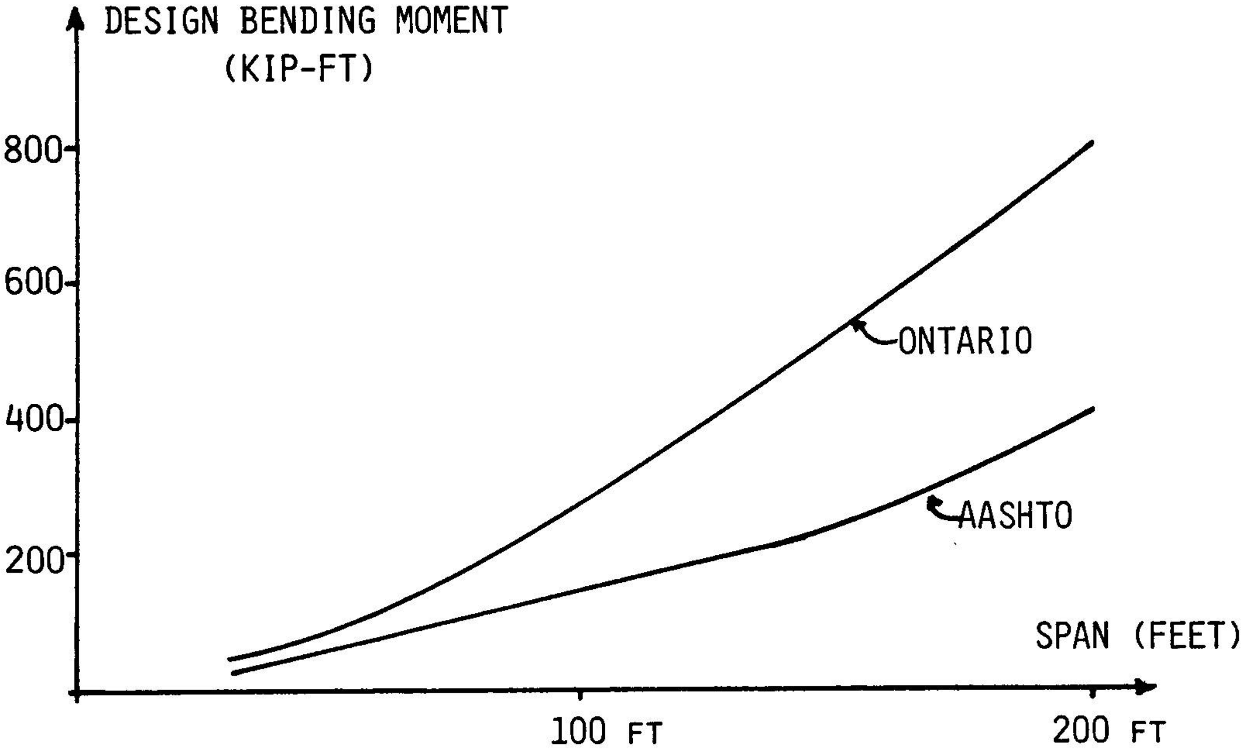


FIGURE 1 COMPARISON OF DESIGN BENDING MOMENTS FOR ONTARIO [9] AND AASHTO [1] BRIDGE SPECIFICATIONS.

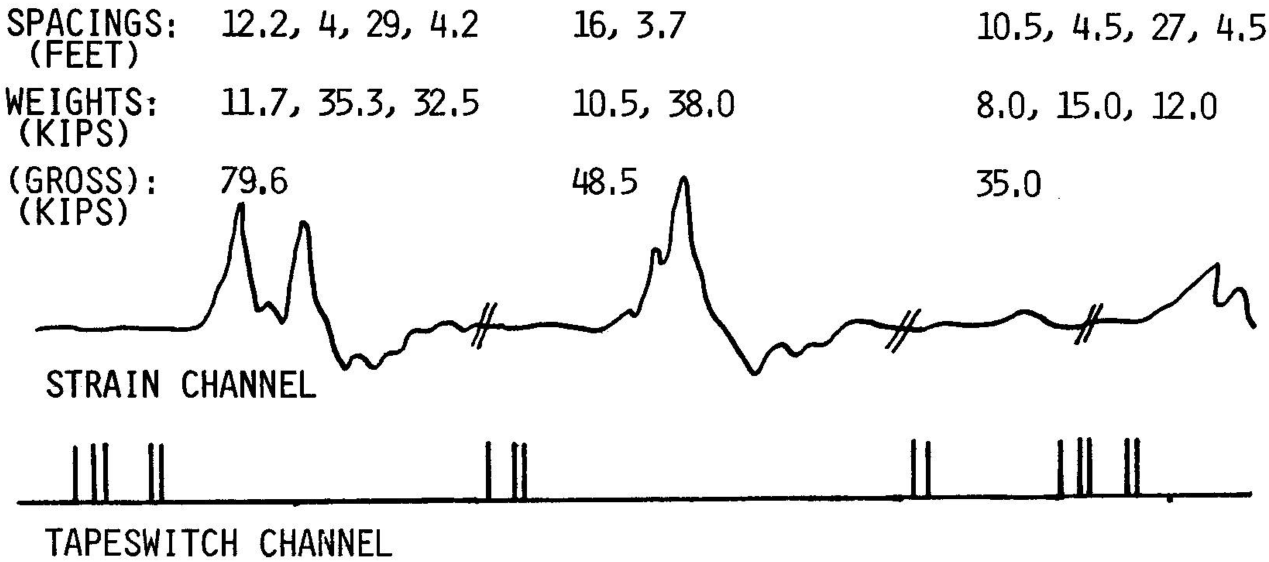


FIGURE 2 SAMPLE RECORD FROM WIM SYSTEM SHOWING BRIDGE STRAIN OUTPUT, TAPESWITCH SIGNALS AND AUTOMATICALLY PROCESSED TRUCK AXLE AND GROSS WEIGHTS, DIMENSIONS AND SPEEDS [4].

tics for reliability-oriented calculations. For bridge assessments, it gives a tool for specific on-site load spectra evaluation and to verify legal or posting conformance. This technique is described in the next section.

3. WEIGH-IN-MOTION TECHNOLOGY

For several years there has been world-wide interest in producing an undetectable system for automatically weighing moving trucks at normal highway speeds. A variety of pavement insert scales have been tested. These flexible plates respond to vertical forces and are calibrated to give histograms of recorded wheel loads. The problems encountered are due to scale flexibility and the "bounce" when a massive flexible vehicle moves on a rough pavement at high speeds. The vehicle is typically on the scale for only a portion of its natural period and large systematic errors may occur due to force oscillation. As a consequence, pavement scales are often restricted to low-speed sorting at busy weigh stations.

Avoidance of static scales is well recognized and by-pass routes makes most scales ineffective for obtaining accurate high-weight statistics [15]. As a consequence the author and his collegues extended the bridge measurement stress system to obtain truck weight information. The weighing system has reached the stage of relatively routine operation by the Ohio Department of Transportation [3, 4], the Federal Highway Administration [6] and other groups to monitor truck weights. Thus far, more than 50 sites have been surveyed.

Briefly, the weigh-in-motion (WIM) utilizes existing bridges as equivalent static scales. Trucks move at normal speeds and drivers cannot detect the weighing operations. Vehicle speeds and dimensions are obtained via tapeswitches bonded to the roadway. Bridge girder response comes from reuseable strain transducers clamped to steel flanges or bolted to concrete beams. The girder influence line provides a simulated strain record. By automatically matching the measured and simulated strains, the vehicle axle weights are obtained [3]. The data recording, monitoring and weight calculation is done by minicomputer in real-time in an instrument van usually parked beneath the bridge. To establish a relationship between strains and truck weight, a known calibration truck is used.

Sites monitored by this procedure have included single span and continuous steel girders and reinforced and prestressed concrete beams in all parts of the United States [6].

The WIM weighing accuracy has been verified by several studies comparing with static weighings. Also, at each site, repeatability is checked with the calibration truck and is usually less than 3%. The prediction accuracy for gross weights has shown standard errors less than 10%, which compares favorably with portable and other static weighing devices. It is most important for fatigue and bridge loading that the weight predictions are unbiased. The WIM surveys provide an important data source for load and fatigue spectra modelling.

Figure 2 shows a sample record from an Ohio site. The strain is actually a sum of several parallel girder responses. The vehicle combinations are also shown in Figure 2. A typical WIM loadometer survey is given in Figure 3. Weight spectra peaks correspond to loaded and empty vehicles. In addition to gross weights, the system outputs axle weights, vehicle axle dimensions, lane, speed and headway [4]. This data is important for constructing load models [2].

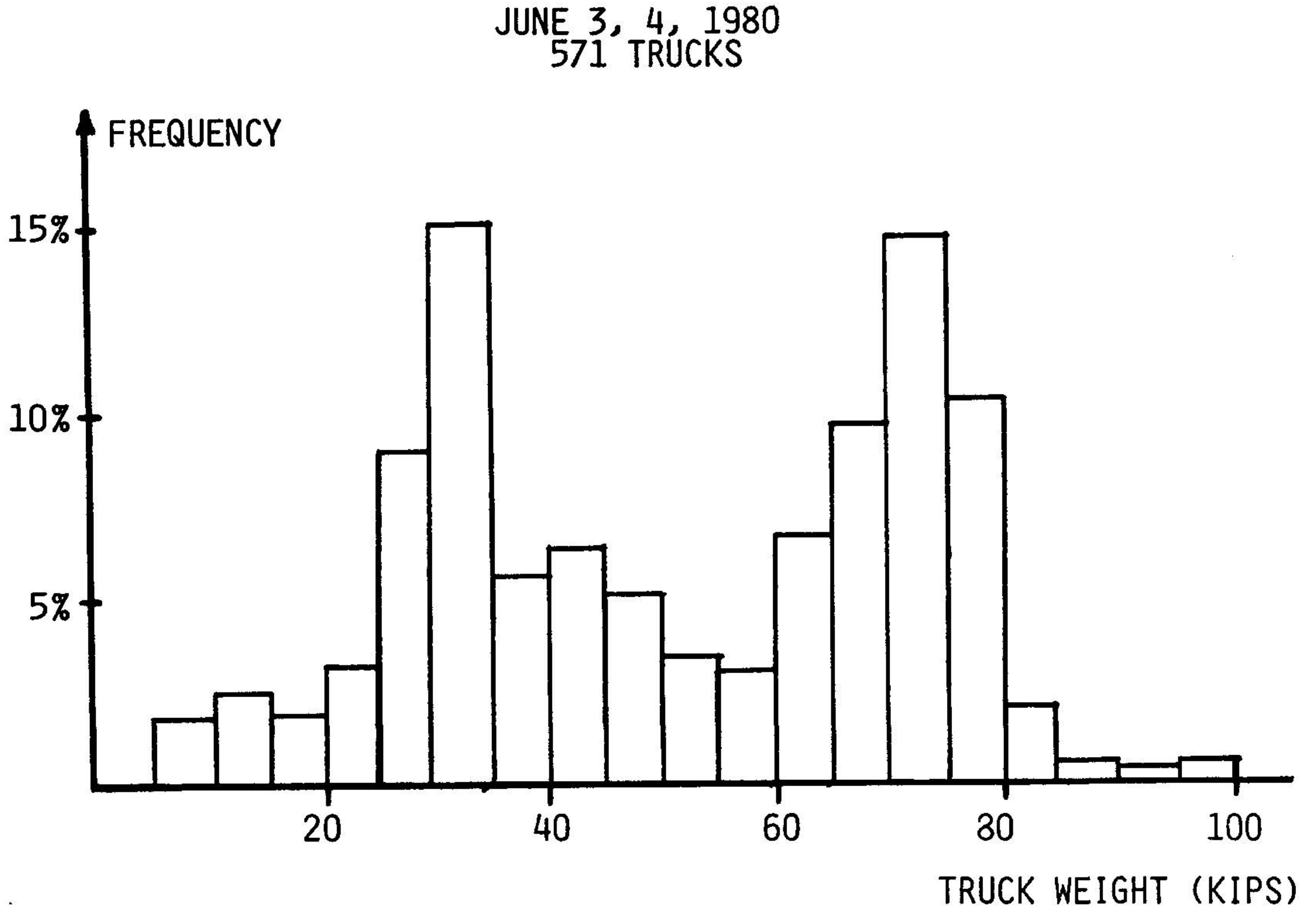


FIGURE 3 SAMPLE WIM GENERATED LOADOMETER SURVEY FOR GROSS TRUCK WEIGHTS [4].

4. FATIGUE ANALYSIS

The statistical data available from WIM technology can be utilized in several ways. For fatigue assessment the data can be expressed as a load spectra. Fatigue is a cumulative process in which each cycle adds damage until failure occurs. With several common assumptions the process can be incorporated in a risk evaluation. Assume a linear damage accumulation proportional to live load stress range. Thus,

Damage, $$D = \Sigma D_i \tag{1}$$

where: Σ - summation

D_i - damage due to single loading cycle

Using Miner's law, the damage is proportional to cycles to failure (N_i) to give:

$$D = \Sigma \frac{n_i}{N_i} \tag{2}$$

where: n_i - number of cycles of stress, S_i

Assuming N_i and S_i are related by a cubic damage rule gives [13]:

$$D = \frac{V}{c} \; \Sigma \; S_i^3 \; f(S_i) \tag{3}$$

where: c - constant from S-N fatigue curve intercept

V - truck volume

$f(S_i)$ - frequency of stress, S_i

Stress is proportional to truck weight so damage can be expressed in terms of the load variables. Thus, [5, 16]

$$D = \frac{V}{c} \, h \; I \; g \; m \; L \tag{4}$$

$f(S_i)$ - frequency of stress, S_i

where: h - superposition effect of closely spaced vehicles

I - dynamic overload

g - analysis variable (girder distribution)

m - stress of nominal design vehicle

and $$L = \Sigma\left(\frac{W_i}{W_n}\right)^3 \; f(W_i) \tag{5}$$

where: W_n - nominal design vehicle weight

For an existing structure the load variables can be measured with WIM equipment. Alternatively, V, L and h can be extrapolated from statistics at similar sites, while g, m, and I are estimated from similar bridge types and spans. The fatigue variable c is based on laboratory tests for appropriate structural details [13].

4.1 Reliability Estimation

The fatigue model can assess reliability in a notional rather than precise actuarial sense. This is satisfactory for comparing diverse bridge locations and incorporating past experiences. If the failure damage is denoted as D_f (mean is 1.0), the risk (P_f) can be written as:

$$\text{Risk, } P_f = Pr[D > D_f] \tag{6}$$

Pr means probability. The uncertainties include material variables, C and D_f traffic variables L, h and V and bridge variables I, m and g.

The complexity of combining all the data in frequency distributions means approximate risk assessments must be used. Second-moment reliability approximations utilize means and standard deviations to obtain safety index (Beta-β) measures [8]. Let the failure function, g, equal:

$$g = D - D_f \tag{7}$$

and safety index, $\beta = \frac{\bar{g}}{\sigma_g}$ [mean ÷ standard deviation] (8)

The reliability measure, β, is suitable for comparing fatigue risks [5, 16]. Recent reliability studies have improved the safety index mode for deriving bridge code safety factors in Canada and Great Britain and other structural codes in the U.S. [7, 12]. A calibration with acceptable structures assures that past practice is incorporated in attaining uniform reliability criteria. Strength as well as fatigue provisions have been studied utilizing lifetime predictions of maximum loading. Two limitations in these developments affect bridge applications.

1. Truck loads are evolving over time, so past practice is not a satisfactory calibration criteria.

2. Code oriented reliabilities are suitable for single component checks, but fail-safe capacity including redundancy is important for bridge assessment. That is, a single component weakness may not cause collapse but loads are redistributed and the bridge is still functional. A fail-safe investigation requires nonlinear behavior. Computer models to predict response are available and results have been verified by testing [17].

   An example of fail-safe implications are found in the AASHTO provisions which permit lower fatigue stresses for nonredundant designs [1]. This is intended to restrict situations in which single element fatigue failure leads to collapse.

   Studies of component and system reliability have been reported for bridges and other structural systems [2, 18]. In bridge assessment, system reliability models may have even greater decision-making potential. It is suited for environments with limited economic resources and when decisions must often categorize bridge deficiencies and rank investment priorities.

5. APPLICATIONS

The previous sections demonstrated reliability-based techniques to combine current truck traffic, bridge loading and laboratory and field data in strength and fatigue assessments. The following topics consider these new developments.

5.1 Inspections

Funds for bridge inspections are limited, requiring optimum schedules. Typical bridge inspection concentrates on physical condition giving important strength information. Inspections should alsoinclude load data since safety checking compares loads with strength. Load assessment may include truck volume, unbiased weight spectra, bridge dynamic response and data on behavior and load distribution within the structure. These parameters are potentially available from WIM technology. Costs may be reduced by not acquiring all information at each inspection.

In cases where posting or extensive rehabilitation seem necessary, additional physical testing to verify strength may be done. The Ontario Ministry of Transportation has been especially active in testing a variety of bridges and benefits from improved verification greatly exceed testing costs [10]. Such testing is more than proof-loading but is done in conjunction with structural analysis to verify predicted behavior. Combined with load assessments, the adequacy of strength margins can be predicted.

Reliability calculations also have potential for establishing inspection strategies. Although fatigue life calculations are often not part of assessment, the reliability predictions can identify potential flaws and provide guidance for detailed field inspection. In addition, components with small fail-safe system reliability margins for load redistribution should also receive frequent and detailed field inspections.

5.2 Posting

Weight posting is warranted if an assessment determines a bridge lacks adequate strength. This is a difficult decision since posting will be obeyed by buses, fire trucks and other critical services. Hence, there is pressure not to be overly restrictive. Some commercial operators, however, may violate posting so listed limits should be low. Specific WIM surveys should study whether the public is obeying posting limits. Tighter control is needed if significant violations are found. Otherwise, posting effectiveness to control extreme bridge loads introduces large uncertainties which reduces such reliability.

5.3 Legal Load Limits and Enforcement

The consequences to bridge safety of overloaded vehicles is well recognized. Large safety factors to cover this situation are uneconomic and may justify pressure by some commercial associations to press for higher legal loads. Instead, designers used strategies with hidden strength margins to cover load growth such as conservative analysis. With improved calculation models, these safety margins have been eroded and hence overloads utilize more of the available strength margin. This fact, combined with longer than anticipated bridge lives implys that stricter load enforcement is necessary. This requires political desire and an efficient technology. WIM displays output in real-time and is available in assisting enforcement to sort vehicles for subsequent portable scale weighing and ticketing. Enforcement is gaining political support as the public learns of road damage. Widespread load enforcement can extend bridge and pavement lives.

5.4 Rating and Permit Assessments

Rating and permit checks compare specified loads and allowable stresses. The latter are often increased above original design levels to reflect better control or less uncertainty in some of the behavior variables. The recent Ontario Code reflected the relative uncertainties in assessment compared to design [9]. Different limit state safety factors are used in assessment. To generalize such safety developments the following aspects should be included:

1) Exposure period. The load factors model load uncertainties and probability distributions of extreme occurrences. This distribution is a function of inspection interval, so shorter periods may have lower expected maximum loads.

2) The optimum reliability targets can be lower for assessment than design because of the trade-off between costs and risk present in such decisions. For new construction, the marginal costs to increase strength, and hence reliability, are much smaller than for an existing structure when strengthing is required.

3) A history of a particular bridge's acceptable performance reduces its modelling uncertainties. These required higher safety factors for new construcrion. Uncertainties include analysis, dimensional tolerances, fabrications and construction factors; in addition, if some simple strain or deflection measurements are made to rationalize the predicted behavior. This factor is recognized for example, in concrete bridges or foundations in which lack

of visual distress signs usually prevents posting in spite of assessment calculations.

4) Field observation of loading spectra at the site also justifies changes in assessment safety factors. New designs are dependent on (vague) forecasting of possible future load patterns. For the period between inspections, such extrapolations are unnecessary.

5) Material uncertainties normally increase with older bridges due to possible corrosion and fatigue weakness. On the other hand, the economic penalties of limiting permit vehicles or reducing capacity suggest that total structure system analysis and reliability be employed to justify increased capacity. For example, to recognize load redistribution. Nonlinear analysis verified by tests have shown significant reserve strength for bridges with adequate redundancy or parallel load paths. The cost of such analyses or testing is justified if it eliminates public inconvenience or costly unnecessary strengthening.

These items are merely an outline of the complex factors in assessing existing bridge structures. The cost, however, in these decisions are often major and hence, it is worth considering new approaches.

6. CONCLUSIONS

1) The decision process for rating, posting, strengthening or replacing existing bridges can be considerably more involved than designing new structures. This activity warrants further research including data on existing loads and predictions of total system performance.

2) Reliability analysis of fatigue spectra and extreme loads may broaden the scope for decision-making and provide a better measure for allocating critical resources. Component safety checks and associated partial factors for assessment should be separated from design safety factors in new construction. Some work exists but further development should incorporate ultimate strength capacities and system reliability models when redundant load paths can be verified by analysis or field testing.

3) New Weigh-In-Motion technology is available to provide better information on bridge loading spectra. For bridge assessment, the data gives appropriate site loading statistics. Bridge measurements in conjunction with weigh-in-motion can verify analyses assumptions, check dynamic response and determine stress distributions at critical fatigue-sensitive locations. This data may ultimately be incorporated in a reliability model for comparing alternative strategies and evaluating priorities for resource allocation.

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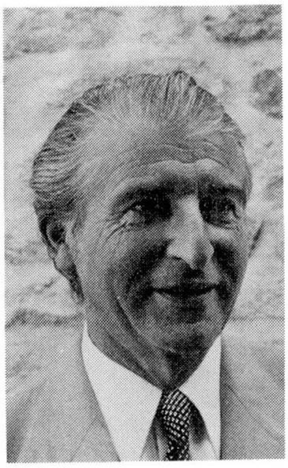
Rating and Evaluation of Remaining Life of Bridges

Classement et évaluation de la durée de vie restante de ponts

Bewertung und Berechnung der Restlebensdauer von Brücken

Guy GRATTESAT
Professor
Ecole Nat. des Ponts et Chaussées
Paris, France



Guy Grattesat is the author of several large bridges in Paris and elsewhere. As chief of the central office for bridges in the Ministry of Transportation he supervised numerous projects. Now as Inspecteur General, he is consulted at all stages of design, erection, maintenance and repairs of bridges.

SUMMARY
The remaining life of bridges depends on the condition of the structure, on its functional characteristics, and also on the foreseeable modifications. It depends mostly on the measures which are taken to extend it. It is generally more economical to repair and eventually to strengthen a bridge than to replace it. The decision must be taken in each particular case; this is sometimes difficult because of the remaining uncertainties. It is a complex domain which calls for a lot of research and international cooperation.

RESUME
La durée de vie restante des ponts dépend de l'état de l'ouvrage et de ses caractéristiques fonctionnelles, ainsi que des modifications à prévoir. Elle dépend surtout des mesures qui seront prises pour la prolonger. Il est généralement plus économique de réparer un pont et éventuellement de le renforcer que de le remplacer. La décision est à étudier dans chaque cas particulier; elle est quelquefois difficile à prendre à cause des incertitudes qui subsistent. C'est un domaine complexe qui nécessite de nombreuses recherches et une coopération internationale.

ZUSAMMENFASSUNG
Die verbleibende Lebensdauer von Brücken hängt vom Zustand des Bauwerks und seiner funktionellen Eigenschaft sowie von den vorzunehmenden Ausbesserungen ab. Vor allem hängt sie von den Massnahmen ab, die getroffen werden müssen, um die Lebensdauer zu verlängern. Es ist allgemein wirtschaftlicher, eine Brücke zu reparieren und eventuell zu verstärken als sie zu ersetzen. Die Entscheidung muss in jedem einzelnen Fall geprüft werden. Ein Entscheid ist oft nur schwer zu treffen wegen der bestehenden Unsicherheit. Dies ist ein komplexes Gebiet, das zahlreiche Untersuchungen und eine internationale Zusammenarbeit voraussetzt.

1. INTRODUCTION

It is widely believed in Europe that bridges have a very long life span. This opinion is based on the fact that several bridges and aqueducts built by the Romans 2 000 years ago are still surviving. In many European countries, a large part of the existing bridges are masonry bridges which appear to stand the test of time.

The other bridges, even if they are much lighter, generally produce an impression of strength and robustness which gives an illusion or long durability.

Another current idea is that the older a bridge is, the longer its remaining life, since it has resisted all aggressions. This may be partially true for stone bridges, but in most cases bridges, like other constructions and living beings, weaken and deteriorate with time.

Yet some spectacular accidents show that the safety of bridges is not absolute. The collapse of the Point Pleasant bridge in the United States in 1967, as that of the Reichsbrücke in Vienna in 1976, created a considerable stir in public opinion. In France, in 1978, the sudden collapse of several arches of a 18th century masonry bridge in Tours on the river Loire aroused a great emotion. By a miracle, there was no victim, but there could have been dozens of casualties if there had been many vehicles on the bridge when it collapsed.

Fortunately, bridge collapses are very unfrequent and their probability is very low, much lower than the probability of road accidents. But they are not admitted by the public opinion which considers them as unacceptable.

The engineers responsible for bridges are certainly aware that their life is far from being infinite. They know, which is most important, that their remaining life depends on their inspection and maintenance. It is sure that the reason why accidents of bridges are so rare is that those which appear dangerous are repaired or closed beforehand in order to avoid their collapse.

2. AVERAGE LIFE EXPECTATION

Is it possible to evaluate the expected life span of bridges ?

This question was raised during the elaboration and development of the new principles of structural safety based on probabilistic concepts. The evaluation of the life time of a structure is an important element for the assessment of the probability of failure, as for the determination of the mean return period of the different actions. It has been agreed that for bridges the expected life to take into account was about 100 to 120 years. But this value is more a "*reference period*" to be used in calculations than a real expected life span. It applies only to new bridges to be built, i.e. to structures which are very different from the previous ones.

This question appears also for the elaboration of a programme of replacement of existing bridges, in order to anticipate the annual corresponding cost. Some studies have been made on that subject in different countries. For example, the O.E.C.D. report on "*bridge maintenance*" published in 1981 mentions a study carried out in Germany in the Rhineland-Palatinate Land which calculates the number of bridges to be replaced every year until the year 2037. This study is based on an average life of 60 years. The remaining life is estimated as the difference between the average life and the age of the bridge. This method can give a rough evaluation of the replacements to be made, but the results would be markedly different if the average life had been chosen longer or shorter. The problem is that the available data are not sufficient to evaluate precisely the average life of bridges.

2.1 Average age of existing bridges

A first approach consists in analysing the stock of existing bridges in order to determine their age. In Europe, it is obvious that this age varies very greatly from the Roman bridges to the most recent ones. It is therefore essential to make distinctions between the different types of structures. From some surveys it appears that in France the average age of existing bridges is about 100 to 200 years for masonry bridges which compose 75 % of the total stock, about 100 years for metallic bridges and about 40 years for reinforced concrete bridges. But these surveys are too restricted up to now and it is not possible to derive precise information from them.

Obviously, there is a fundamental distinction to make between the average age and the average life span of a type of bridge.

In the history of masonry bridges, it appears that a large number of Roman and Middle Ages bridges have been destroyed, sometimes few years after construction. The average life of masonry bridges is therefore noticeably shorter than the average age of the existing ones.

Conversely, the oldest prestressed concrete bridges are approximately 40 years, and many new ones are built every year. So the average age of this type of bridges is less than 20 years, and it is fortunately certain than their average life span will be much longer than their present average age.

There is no doubt that it will be very useful to know better the average age of the different types of bridges, but it is only a partial element which does not enable by itself to evaluate their remaining life.

2.2 Annual rate of replacement

Another approach consists in considering the annual rate of replacement of bridges, that is the ratio between the number of bridges replaced every year and the total number of existing bridges. An inquiry has been made on this question in the O.E.C.D. countries and its results are given in the report on "*bridge maintenance*". The national rates extend from 0,02 to 1,6 %, with a majority of values between 0,2 and 0,4 %. Theoretically, it could be concluded from these last values that the average life of bridges, which is the inverse of the rate of replacement, is situated between 500 and 250 years. Obviously this conclusion does not mean anything, because the population of bridges is not at all homogeneous. As the life span of most of the bridges is certainly shorter, it appears that the annual cost of replacements is not determined by the age of the bridges, but by other considerations. However, as more and more bridges will need replacement, it is very likely that this annual cost will have to be considerably increased in the future in order to avoid traffic limitations or accidents on the road network.

2.3 Causes and reasons of replacement

In order to try to evaluate more precisely the remaining life, it is very instructive to analyse the causes and reasons of the death of bridges. They may be classified in different categories :

- collapses of the structure, due to errors in the design or during construction, or to the deterioration of materials ;
- collapses due to traffic ;
- collapses due to natural actions, such as scour of foundations, wind action on some metallic bridges ;
- collapses due to accidental actions, earthquakes, impacts by vessels or vehicles, landslides, avalanches, etc .. ;
- intentional demolition for structural reasons in order to avoid collapse ;
- intentional demolition for functional reasons, because the bridge is too weak or too narrow, or too low above a road or a navigable river, etc ..

In many countries, including France, a great number of bridges have been destroyed during the wars. So the diagrams representing the population of bridges by age-groups are very different from those of the regions which have not suffered from the wars.

It can be noted in the history of bridges that collapses due to traffic are relatively rare. They occurred only when there was a serious defect in the structure, or when a heavy vehicle tried to cross the bridge despite the load limitation.

Fortunately, the intentional demolitions are much more numerous than the accidental collapses.

These various factors work differently according to the type of the bridge.

The *timber bridges*, which have been very numerous in the past and which have almost completely disappeared, except in certain regions, have been destroyed by floods, ice pressure or fire, or by physical, chemical and biological attacks, or have been replaced because of their too weak carrying capacity.

Nearly all collapses of *masonry bridges* are due to failure of foundations, mainly because of scour, especially when they were supported by timber piles, as in the case of the Tours bridge. Many of them have been replaced because they were much too narrow or too low above a navigable river.

A large number of other bridges, some of them still young, have been destroyed because of defective *foundations*. Such accidents, which do not depend on the material of the superstructure but only on the type of foundations, still happen nowadays, as a result of scour and erosion during floods, or changes in the soil bearing capacity.

In certain regions, *earthquakes* have caused the collapse of many bridges.

The average life of the first *metallic bridges* which were built in cast iron has been rather short. They collapsed or have been replaced because of the brittleness of the metal. Many fractures appeared in the structure due to vibrations and temperature variations. In Paris, in 1939, a cast iron arch bridge on the river Seine was struck by a boat and collapsed instantaneously. After that, several other bridges of the same type have been replaced in order to avoid a similar accident.

In other types of bridges too, total or partial collapses were due to *impacts* by vessels or vehicles either against the supports or against the superstructure when these components were not robust enough.

Many accidents have occurred in the 19th century in *suspension bridges* because of corrosion and wind effects. It has been necessary to replace a number of those which had remained for it was not possible to strengthen them. Even in the 20th century, some accidents have been caused by wind. Every bridge engineer knows the adventure of the Tacoma bridge which collapsed in 1940 after a particularly short service life.

In *iron and steel bridges* built since the 19th century, the main cause of the disorders has been corrosion. The effects of corrosion have been very different depending on the various elements, on the detailing and on the possibility and quality of maintenance of the structure. It has been generally possible to repair or replace deteriorated elements and to avoid accidents. On the contrary, fatigue phenomena have caused some collapses without any warning.

Reinforced concrete bridges have suffered from the cracking of concrete and the corrosion of steel, especially those of the beginning of the century because of the shallow depth of concrete cover. When the deterioration was very serious, the bridge had to be replaced. In some cases, the concrete was greatly weakened, or was decayed due to chemical phenomena, when the bridge was situated in very aggressive weathering, or when the cement or aggregates were of bad quality. Some disorders came also from freezing and thawing and more rarely from alkali-reaction between the cement and certain types of aggregates.

Among the first *welded steel bridges*, some have collapsed because of phenomena of brittle fracture which are now overcome and can be avoided with some well known precautions concerning the quality of steel and the methods of welding.

It is too soon to analyse the reasons of the demolition of *prestressed concrete bridges*, and consequently it is not possible to predict their remaining life. In France, the first bridges built by Freyssinet over the river Marne from 1946 to 1950 are still in service and in good condition after some partial repairs. The very few bridges which have been demolished were more recent. Their defects came either from unsatisfactory design and a bad evaluation of the action-effects, or from the fact that the necessary precautions had not been taken during construction. Sometimes the quality of the concrete was not sufficient, or the position of the tendons and their protection against corrosion and especially against stress corrosion had not been carefully controlled.

It appears that water is one of the worst enemies of bridges. In many cases, whatever the material of the bridge, disorders were due to the seepage of water into the structure. That is why in most countries of Europe decks are protected by waterproofing layers. This precaution is considered as very important and necessary. The fact of the matter is that the observed degradations are much more serious when the waterproofing layer is of poor quality or deteriorated. On the contrary, when this layer operates correctly, the structure remains in a much better condition.

It would certainly be very useful and instructive to analyse in detail and quantitatively the reasons of the demolition of bridges in the past. Some studies have been made in this field, for example that of D.W. Smith quoted in the O.E.C.D. report "*evaluation of load carrying capacity of bridges*" in which are examined the causes of 143 collapses between 1847 and 1975. It appears that 60 % of the collapses are due to natural phenomena. A more complete inventory, including the intentional demolitions, would require long historic research for which much information would be lacking. But it is certainly possible and desirable to keep up to date in every country lists of annual replacements of bridges, with the precise reasons of each decision.

The conclusion of this short review is that it is not possible to take into account only the age of a bridge and an average life span in order to evaluate its remaining life. An overall assessment, like that which was made in Rhineland-Palatinate may be useful for drafting future programmes, but it would be quite unreasonable to order the retirement of a bridge when it reaches 60 or 100 years. It is absolutely necessary to examine the problem in each case.

2.4 Forecasts for the future

In order to anticipate the remaining life of bridges, it is not enough to analyse the experiences of the past. It must be considered that the aggressions they are exposed to will certainly get worse in the future.

The number of vehicles and the weight of freight vehicles have considerably increased in recent years. More and more permits for very heavy vehicles and exceptional transports are requested. It is possible to a certain extent to check the loading capacity and to evaluate the risks of fatigue deterioration of existing bridges under the present level of traffic. But it is not possible to predict the future. If the number and weight of vehicles still increase in the coming years, if new routes have to be adjusted to the transport of more heavy exceptional loads, it will be necessary to replace a lot of bridges which are now in good condition and the potential remaining life of which will be reduced. On that subject, one of the conclusions of the I.A.B.S.E. Symposium which took place in Cambridge in 1975 was that the authorities responsible for bridges had to warn the governments about the consequences of the increasing weight of very heavy vehicles as regards the safety and cost of strengthening of the structures.

It will be also necessary, if the traffic increases, to widen a certain number of bridges. In certain cases, it may be possible to widen the deck and to maintain and strengthen the bearing elements. Sometimes, a second bridge will have to be built along the existing one. But in other cases, the only solution will be to demolish and to replace it by a wider bridge. In this case too the remaining life will be voluntarily shortened.

As regards the other variable actions, especially wind actions, their effects are much better known than in the past. A great progress has been made when it has been recognized that they have a probabilistic and not a deterministic character. But the discussions which take place in the Joint Committee on Structural Safety and in the International Standardization Organization about the fixation of their characteristic values, and the discrepancies between the national codes, prove that our knowledge is still very insufficient on that subject.

The danger of some accidental actions, such as collisions, will increase with the traffic. It can be reduced by protective measures, which are never absolutely efficient and which cannot practically be implemented in all structures.

The chemical aggressions have increased in recent years, because of the weathering pollution and mainly of the generalized use of de-icing salts in order to ensure free traffic in winter time. These aggressions will probably still increase in the future, so that the deterioration of materials will be accelerated. Obviously their consequences will depend on the material and the location of the bridge.

Other new dangers already appear. For example, for some time past, extractions of materials from the river beds for the needs of construction and agriculture have considerably increased. If the volume of these extractions is not limited, the foundations of bridges can be severely attacked due to the lowering of the river bed and collapses may occur.

Conversely, it is possible that new protections and remedies are found against these dangers, thanks to the means of maintenance, repair and strengthening which can be improved, and to the protective measures which can be taken by the authorities.

Finally, it appears that the remaining life of a bridge is not at all fixedly determined : it depends essentially on what will be done intentionally or not to make it longer or shorter.

3. INDIVIDUAL DECISIONS

In practice the decisions are to be taken in each particular case, taking into account the present condition of the bridge and the foreseeable changes in the future.

3.1 Results of inspections

In most countries, bridges are systematically inspected. Generally, there are different levels of inspections : superficial inspections carried out permanently by the ordinary maintenance personnel, in order to detect the defects which may appear ; principal inspections carried out by trained personnel, at intervals of one or two years for general inspections, and of three to five years for major inspections ; special inspections made in unusual circumstances in view of a reassessment of the structure.

In recent years, these inspections have been carried out more regularly and carefully, as the engineers in charge of bridges became more aware of the dangerous condition of certain structures, when they heard of dramatic accidents which occurred in their country or abroad. On the other hand, the increase of requests for exceptional permits for heavy vehicles obliged them to check the load carrying capacity of many bridges. Moreover improvements in inspection methods have led to detect many defects which had not been discovered previously. For example, in France, underwater inspections by divers have been prescribed systematically over the past 25 years. These inspections of the lower parts of the supports are made regularly, sometimes with the aid of TV cameras, and have detected disorders which would have possibly caused the collapse of the bridge if they had not been discovered and repaired in time. As for the superstructure, thanks to the improvements of the means of access, and especially to the use of mobile inspection equipment operating from the bridge deck, it has been possible to inspect in detail those parts of the bridge which were previously very difficult to visit. So many visible deteriorations have been discovered, and more detailed inspections have been considered as necessary. And modern techniques of inspection have discovered other hidden deteriorations and obliged to take maintenance and safety measures.

This is the reason why the number of disorders has apparently increased rapidly during the recent years. In fact, many of these disorders existed already for a long time, but they had not been detected, and after all the bridges have been inspected, the number of disorders discovered every year will probably decrease.

Depending on the results of these inspections, the responsible authorities have to take decisions about the fate of the bridge. When there are no disorders and if no functional improvement is necessary, the only thing to do is to carry out regular maintenance of the bridge. When disorders are detected, there is a choice to make between repairs, eventually with a weight limit, and replacement of the bridge. When the disorders are serious, immediate measures are to be taken, traffic limitation or closure of the bridge, and possibly precautions against falling lumps under the bridge.

When functional improvements are necessary, the possibilities of strengthening or widening the bridge have to be examined.

In France, there are two annual programmes concerning existing bridges. The first one applies to the rehabilitation of those which need repairs, the second one to the improvement of functional characteristics of those located on special routes. In many cases bridges need both structural repairs and functional betterments.

When the condition of the bridge is evidently so bad that no rehabilitation is feasible, the decision of replacement is imperative. But generally the choice between repair, strengthening and replacement is not evident and comparative studies must be undertaken.

3.2 Data and information

At first, the problem is to gather and assemble all data concerning the bridge, to find the original design and the documents about the repairs and modifications carried out since it was built. The archives are often lacking, or they are very incomplete and the drawings must be done again using the exterior dimensions of the accessible elements of the structure. Even when the existing documents seem to be accurate, they must be examined with caution, because it happens that the actual dimensions are different from those of the old drawings, and it is advisable to compare them as closely as possible.

All the disorders detected during the inspections, such as deformations, corrosion, cracks, etc .. must be analysed in order to obtain a first assessment of the condition of the bridge.

3.3 Evaluation of load-carrying capacity

If it appears possible to keep the bridge, its load-carrying capacity has to be evaluated. Except for recent bridges, the design specifications have changed since its construction. The loadings fixed in the codes have increased with the number and weight of vehicles. The permissible stresses have also increased, but generally the result of the calculation appears more unfavourable than the original one. So the safety level seems to be insufficient compared to that of new bridges. On that subject, it must be pointed out that there are considerable and unjustified differences between national codes, as well in the systems of traffic loadings as in the safety elements to be taken into account in the design.

Before taking the decision of strengthening or replacement, it is advisable to analyse the problem more completely. It is particularly useful in this field to resort to the new principles of safety. It has been shown that the concept of "*safety level*" is very complex and cannot be expressed quantitatively in a simple manner.

Firstly, a distinction must be made between ultimate and serviceability limit states. Thanks to this distinction which did not exist in the previous specifications it is possible to treat differently the effects of the actions which are really dangerous, and those which would have only minor consequences.

Secondly, it is also useful to refer to probabilistic concepts in order to evaluate the random elements to be taken into account, which are not the same as in a structure to be built. As the bridge exists, the permanent loads are known more precisely, provided that the real dimensions have been checked, and only the remaining uncertainties are to be taken into account. For the variable actions, it can be considered that the remaining life of the existing bridge will be shorter than the life time of a new bridge, hence the reference period and consequently the characteristic values of the actions may be reduced. But the evaluation of these life spans is not sure enough to arrive at precise conclusions. For the combinations of actions, the probability of simultaneous occurrence of unfavourable values of several independent actions is reduced, so that the values of some of the variable actions may be reduced, in accordance with J.C.S.S. and ISO documents. For instance, it would be unreasonable to combine the heaviest loading with the strongest wind action.

Concerning action-effects and resistances, it is important to know the actual condition of the structure. Strengths of materials can be measured by some removals of samples for laboratory tests. The mechanical behaviour can be appraised by the exterior condition and deflections of the load-carrying elements and also by close examination of the bearings and expansion joints. It may be necessary at this stage to carry out more complete investigations in order to determine the internal condition and the behaviour of the structure. Several non-destructive methods can give useful information : sclerometer (rebound-hammer), pachometer (magnetic detector), ultra-sonic devices, fissurometry, extensometry, gammagraphy, bearing pressure weighing devices, etc .. In certain cases, with recent techniques, it is possible to measure the total stresses of steel and concrete and not only the variations of stresses under loading.

These supplementary investigations are rather costly and must be used only for precise purposes, in close collaboration with the design office.

The structural analysis should apply to the bridge in its actual condition and eventually in its condition after repair or strengthening. As far as possible, the calculation should be carried out taking into account the real behaviour of the structure, including the influences of cracking, frictional forces, redistribution of stresses, etc .. With the present methods, the results of this calculation will be more accurate than the original ones.

According to its conclusions, an evaluation is then made of the load-carrying capacity. It must be recognized that this notion is not so simple as it may appear. It is not sure that the posting of a weight limit for one vehicle can prevent from dangerous stresses in certain elements due to random distributions of loads.

The results of the calculation can be confirmed by static full-scale loading tests which allow a comparison between theoretical deformations and those observed experimentally. A comparison may also be made between calculated and measured stresses in certain sections.

Dynamic tests, as those which are made in Belgium, will perhaps be able to give supplementary information on the condition and evolution of the structure.

These tests are useful to check the behaviour of the bridge as a whole and to detect the defects which have not been taken into account in the theoretical study. But they cannot allow to determine the maximum load-carrying capacity of the bridge, for the test loads are necessarily below the ultimate load, nor its resistance to a combination of several different actions. And of course they cannot say anything about the fatigue behaviour of the structure.

One of the problems encountered in this field is the fixation of permissible stresses or partial safety factors, which are not necessarily the same as for new structures. When the data concerning an existing bridge are sufficient to reduce the uncertainties, it may be considered that its "*level of safety*" is the same as if it was a new one, even if the numerical values of the safety factors used in the calculation are lower.

Up to now there is no international recommendation on that subject which deserves very useful research.

3.4 Disorders, repairs and modifications

When the conclusion of these studies is favourable, the design is completed for repairing and if necessary strengthening or modifying the bridge.

It is very important not to repair only the visible defects, but to find their causes in order to remedy them and avoid further deteriorations. It must be determined whether the degradations are due to the material itself, or to the detailing of the structure, or to movements of foundations, whether the cracks are normal or not, etc ..

It is also necessary to examine all the consequences of the planned rehabilitation work, some of which may be unfavourable or even dangerous. For example, when additional prestressing tendons are necessary, it must be checked not only that tensile stresses will be suppressed in certain sections, but also that dangerous stresses will not appear elsewhere.

The possible repairs and modifications are appreciably different according to the type of structure.

Masonry bridges are essentially exposed to disorders in their foundations. The vaults are generally very robust and able to carry very heavy loadings. It is often possible to place a reinforced concrete slab on the vault in order to widen the carriageway, provided that the foundations are strong enough. The remaining life of these bridges may be very long if their foundations are properly maintained and if necessary repaired and strengthened. In some cases, vibrating effects of traffic have caused cracks and loosening of stones, which are generally not worrying and can be easily repaired if the piers and abutments are sound.

Disorders in the *foundations* are dangerous for all types of bridges. They can be detected only in an indirect manner, through deformations in the supports and in the structure. It is often difficult to determine their cause and the corresponding repairs are generally very expensive. But they must be made without hesitation in order to avoid a possible collapse of the bridge. When the superstructure must be replaced for structural or functional reasons, it is sometimes suggested to re-utilize the supports which appear in good condition. This could be accepted only if it is ascertained that the existing foundations are strong and durable enough, for it would be unreasonable to build a new superstructure on defective foundations.

In *steel bridges*, the components which mainly suffer from corrosion can be quite often repaired and strengthened. It is possible to strengthen too weak elements with the aid of welded plates, or to replace them when they are too deeply deteriorated. So the remaining life of steel bridges can generally be extended and made very long. However some difficulties appear when some parts of the structure are not accessible or when metallic pieces are embedded in masonry or concrete. It is advisable in this case to demolish and rebuild those parts of the bridge which may be dangerous. Other serious problems arise from fatigue and corrosion fatigue phenomena and it is always necessary to strengthen or replace the elements subject to this menace.

Concerning *suspension bridges*, in spite of the improvements in the inductive-magnetic techniques and more recently in acoustic spying, it is not possible to evaluate accurately the amount of corrosion and of breakage of wires in the cables.

If it appears that the strength of the cable is too weak, the only solution is to replace it, which is difficult and expensive, or to replace the bridge by a new one. Similar problems are likely to appear in cable stayed bridges, that is why it is preferable to separate the cables in harp, in order to be able to replace them individually if necessary.

Reinforced concrete beams and girders can now be repaired rather easily and efficiently as a result of new techniques and various products created by modern chemistry. It is generally possible to rehabilitate the structure when the disorders are not too severe, but it is very difficult to strengthen and practically impossible to widen it. Recently in France a reinforced concrete bridge which it was not possible to close has been strengthened by prestressing cables, and the cost has been nearly the same as that of a new superstructure.

Some examples of repair and strengthening of *prestressed concrete bridges* exist already. In each case, the method must be designed in detail by specialists.

It is not possible to forecast the remaining life of these bridges, but it is certainly desirable to find new techniques of inspection and repair in order to extend it.

In all cases, in order to avoid new disorders, it is necessary to prevent seepage of water into the deck and for this purpose to protect the deck by an efficient and durable waterproofing layer.

3.5 Financial and safety criteria

When the design is completed and the cost of the works approximately known, the problem is either to repair or to replace the bridge.

The answer is immediate when the main elements of the structure are in good condition and when only partial repairs or strengthenings are necessary. In most cases, the proper solution is to increase the remaining life of the bridge by replacing or stengthening some deteriorated elements. Sometimes the load-carrying capacity can be notably raised by reducing the permanent load, for example by replacing a heavy deck slab by a lighter one, using such materials as lightweight concrete or aluminium.

The decision is more difficult when the cost of the works is very high. This cost must then be compared to that of a new bridge, to which are of course to be added the cost of demolition and all supplementary expenses due to disturbance of traffic. These supplementary expenses are sometimes so important that the total cost of reconstruction is much higher than the cost of the new bridge.

For this financial comparison, the concept of "*discounting*" is employed, which expresses the fact that future costs are of lower present value than present costs. Some economic studies have been made in this direction. For example, a calculation made in the O.E.C.D. report on maintenance shows that if the reconstruction of a bridge is postponed for 15 to 20 years, it is more economical to make repair works reaching 40 % of the replacement cost if the discount rate is higher than 3 %. Generally the discount rate is effectively higher than 3 %, and the remaining life due to so expensive works must be longer than 20 years. The conclusion is that from the economic point of view it is better to extend the remaining life of a bridge than to replace it, even if the corresponding cost is high.

But this point of view is secondary compared to the most important one which is to ensure safety. The main difficulty often encountered for the decision is that the results of investigations and calculations are not completely sure and that some uncertainties subsist. In fact, these studies can prove that the structure is unsafe, but they cannot totally ensure that it is safe, or will be safe after repair. Every engineer knows that absolute safety cannot be achieved.

It is then necessary to imagine danger scenarios and strategies against potential risks, and to look for the main determining factors and the major weaknesses which could threaten the bridge.

The main uncertainties concern those parts which cannot be inspected and are checked only by indirect ways. The principal danger is situated in the bearing elements the failure of which may lead to sudden collapse without any warning, for example :

- condition of foundations which cannot be assessed but very partially and indirectly ;
- condition of internal wires of suspension bridge cables and of their anchorages ;
- condition of elements subject to fatigue phenomena ;
- corrosion or rupture of prestressing tendons.

Methods and techniques of investigation have been very much improved in recent years. Yet further research is required and new improvements must be made in order to reduce the remaining uncertainties and to arrive at reliable qualitative and as much as possible quantitative data.

It is not enough to utilize at spaced intervals advanced instruments and methods. It is desirable to find more rapid and economical measures, such as acoustic spying or gammascopy for cables and tendons, which could allow a more complete and frequent inspection.

In spite of the improvements which may be awaited in this field, uncertainties will remain for a long time and a balance has to be achieved between the acceptable residual risks and the financial considerations which lead to extend the life of the bridge.

As the available funds are very generally limited, the responsible authorities have to make difficult choices when establishing programmes and priorities.

Some decisions are compulsory :

- The protection of the national heritage of bridges obliges to preserve as long as possible all the bridges with a historical or aesthetical character, even when the cost of their maintenance and rehabilitation is very high. Such is the case for almost all bridges prior to the 19^{th} century, and for several more recent ones which possess an architectural value or have represented a new step in the technical evolution.
- Conversely, those which are functionally insufficient and which cannot be modified, or are structurally in so bad a condition that they cannot be repaired, are to be replaced, as well as those which would necessitate very expensive repairs followed by permanent surveillance and maintenance expenses.

Between these two groups, there are a lot of bridges the remaining life of which must be extended for economic reasons. The necessary repairs or improvements must be carried out in order to ensure safety as much as possible, to avoid new disorders and to reduce further maintenance costs. In some cases, the loadings have to be limited.

Before the repairs are done, or when some doubts remain about their efficiency, supplementary inspection and warning measures are to be taken, in order to allow immediate emergency decisions concerning safety, including load limitations or even closure of the bridge if necessary.

4 CONCLUSIONS

In the present state of knowledge, it is not possible to evaluate precisely the remaining life of bridges. It is just a matter of very approximate engineering judgment.

For recent bridges, it is generally considered that their life time will be of about 100 years, but this prediction is obviously more subjective than rational. For the others, the remaining life depends essentially on the type and the basic material of each bridge, on its age and condition, and above all on the possibilities of repairs and improvements which can extend its service life.

Therefore it would be very useful to gather in each country inventories of existing bridges classified along their type and date of construction. It is also desirable to continue inquiries on the annual rate of replacement of bridges, indicating the reasons of each replacement and as far as possible to look for information on this subject in the past.

With the aid of various documentation systems using cards, registers, databanks, catalogues of defects, which allow to analyse the behaviour of the different types of bridges, it may be possible to prepare in a more precise manner the overall programmes of replacements for the years to come.

But the decision must be taken in each particular case either to extend the remaining life of the bridge or to demolish and replace it. This decision must take into account the actual condition of the structure and its foreseeable evolution as well as its functional capacities and the necessary improvements to be made for the needs of traffic.

The choice is sometimes difficult to make because, in spite of the new techniques developed in recent years, many uncertainties subsist in the appraisal of existing bridges.

Improvements are very desirable in this respect :

- in the techniques of inspection and investigations, especially in order to find out rapid and economical non destructive methods, allowing to check the internal condition of cables of suspension bridges and of tendons of prestressed concrete bridges, and the soundness of foundations ;
- in the design rules to be applied to existing bridges, which need special specifications ;
- in the techniques of repair, strengthening and widening.

Generally it is more economical to repair and modify the bridge than to replace it, and anyway the available funds are not sufficient to demolish and rebuild all defective ones. When the structural or functional condition is too bad, the reconstruction is unavoidable. But in many cases a balance is to be achieved between safety and economic considerations. Sometimes, when the bridge is kept in service, safety measures such as traffic limitations and increased surveillance must be taken.

It results from this policy that it is desirable to extend as much as possible the life expectation of new bridges, by different means : attention paid to durability aspects and eventually overdesigning of certain elements, protection devices especially for drainage and waterproofing, easy access for inspection and maintenance and possibilities of strengthening the structure if necessary.

It is much more economical to do so, because these precautions in the design can avoid expensive repairs in the future and substantially postpone the replacement of the bridge without appreciably increasing initial costs. In this field, prevention is certainly better than cure.

For new bridges as well as for existing ones, it is of the utmost importance that all relevant data information such as records of construction, inspections, repairs and alterations, as-built drawings, calculations, etc .. are collected and filed with continuous up-dating.

In every case, the remaining life depends obviously on the quality of inspection, maintenance and repairs, and on the future changes in traffic and environment of the bridge ; so it depends essentially on decisions the effect of which will make it longer or shorter.

These decisions involve a wide range of technical knowledge and a good engineering judgment, they call for much experience supplemented by advice from specialists of laboratories and design offices.

It is a very difficult problem which deserves a lot of studies and research in different fields in order to fill some serious gaps in our present knowledge and for which an active international cooperation is very desirable.

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Rating Bridges for Special Permit Loadings with Considerations of Future Life – Case Studies

Evaluation de la durée de vie des ponts et charges extraordinaires – Etudes de cas

Lebenserwartung von Brücken und ausserordentliche Lasten – Fallstudien

John M. KULICKI
Partner
Modjeski and Masters
Mechanicsburg, PA, USA

Dr. John Kulicki joined Modjeski and Masters in 1974 and entered the Partnership in 1980. He is currently in charge of technical development of structural projects in the Harrisburg office, serves as Supervisor of computer-related activities, and as Manager of design projects. He has recently developed load factor truss design procedures and computer programs for the Greater New Orleans Bridge No. 2

SUMMARY

Special permit loads continue to get heavier and more numerous with each passing year. This paper describes case studies of special analyses for heavy permit loads resulting either in 1) tables of permissible Gross Vehicle Weights (GVW) for vehicles of special interest and rules-of-thumb for particular bridges to be used by toll takers, or 2) tables of permissible Gross Vehicle Weights (GVW) for vehicles of special interest and bridge-specific computer programs to be used by permit officers. These studies have had considerations for fatigue of special details or typical details based on use history for the particular beidges as indicated in roll records or ADT and loadometer studies.

RESUME

Chaque année, les cas de charges extraordinaires deviennent de plus en plus nombreux et les charges de plus en plus élevées. L'article présente des études de cas pour la détermination de charges exceptionnelles admissibles. Il en résulte des tabelles de charges exceptionnelles admissibles pour des véhicules spéciaux, ainsi que des conseils pour le choix des ponts à franchir et des programmes spécifiques de calcul à l'ordinateur à l'intention de l'autorité exploitant les ponts. Ces études traitent les problèmes de fatigue de certains détails constructifs typiques ou spéciaux tels qu'ils apparaissent sur les protocoles de mesure de certains ponts, et sur la base de campagnes de mesures.

ZUSAMMENFASSUNG

Jedes Jahr werden die ausserordentlichen Verkehrslasten häufiger und grösser. Fallstudien wurden unternommen, um die zulässigen ausserordentlichen Verkehrslasten zu definieren. Daraus ergeben sich Tabellen zulässiger ausserordentlicher Verkehrslasten für Sonderfahrzeuge sowie Empfehlungen für die Auswahl von befahrbaren Brücken und spezielle EDV-Programme zu Handen der Brückenbehörden. Diese Studien behandeln Ermüdungsprobleme verschiedener typischer oder spezieller konstruktiver Details wie sie in Zustandsprotokollen gewisser Brücken erscheinen oder aufgrund von Messkampagnen resultieren.

CASE STUDY #1 - ST. GEORGES BRIDGE

INTRODUCTION

St. Georges Bridge is a fixed, high-level, four-lane highway bridge crossing the Chesapeake and Delaware Canal at St. Georges, Delaware. The main span is a 540 ft. (164.6 m) steel tied arch. The approaches consist of 3,714 feet (1132 m) of beam and girder spans of varying lengths and framing. The strucutre was designed under AASHO 1935 Specifications for a live load of H20 and subsequently rehabilitated in 1971 to HS20-44 loading in accordance with 1969 AASHO Specifications, as amended in 1970. At that time, a sidewalk was removed to widen the cartway. The original presence of a sidewalk resulted in a transversely unsymmetrical floor system. A rating analysis of the entire bridge was completed in 1973.

SCOPE OF WORK

The following is a summary of the Scope of Work for this Project. Some simplifications have been made in small details which do not affect the content of this paper.

o The floor systems and girders of the approach span and the floor systems and main members of the tied arch channel span were investigated. Splices in the stringers and girders and connections in the main bridge were evaluated only if a review of previous rating calculations indicated that they could control the evaluation. Previous rating calculations were used to eliminate sections which could not conceivably control permissible GVW. One hundred and fifty (150) possible sections remained to be checked for each loading.

o The girders, stringers and arch tie were evaluated for both bending and shear.

o The effects of loss-of-section were included.

o The latest edition of AASHTO Standard Specifications for Highway Bridges and Manual for Maintenance Inspection of Bridges was used modifications as indicated herein.

o Only AASHTO Group I loads were investigated and the "service load" method was used. The distance between the face-of-curb and the center of a wheel was 1.5 feet (0.457 m).

o Live and impact loads were in accordance with the latest AASHTO Specifications, except as follows. "Special" vehicles were used as determined from types found in the State of Delaware. Three transverse configurations of load were considered, with the vehicles located in parallel lanes:

(a) CASE I LOADING: Special vehicle, HS15, HS15, HS20 at normal design allowable stresses.

(b) CASE II LOADING: Special vehicle, special vehicle, HS15, HS20 at normal design allowable stresses.

A "Bottom Line" composite of Cases I and II was developed to represent the maximum GVW's of the special loads travelling in routine traffic.

(c) CASE III LOADING: Special vehicle with all other live loads prohibited. This loading case was analyzed with and without impact.

- Longitudinally, the loadings for CASE I consisted of two special vehicles separated by 30 feet (9.1 m) from front wheel of one vehicle to rear wheel of the other vehicle and the standard configurations of the AASHTO lane loading. For CASE II, the longitudinal loading consisted of the special vehicle and the standard configurations of the AASHTO lane loading. For CASE III, only the special vehicle was assumed to be on the bridge.

VEHICLE CONFIGURATIONS

Delaware motor vehicle regulations concerning weight limitations for trucks on Interstate routes follow the Federal formula, which is:

$$GVW = 500 \left[\frac{LN}{(N-1)} + 12N + 36 \right]$$

$$\left(GVW = 2.224 \left[3.28 \frac{LN}{(N-1)} + 12N + 36 \right] \right)$$

where: GVW = gross vehicle weight in pounds (kN)
L = distance in feet (m) between front and rear axles
N = number of axles

In addition to the limitations given by the above formula, there are specific limits placed on overall length, gross vehicle weight and maximum axle weight for both single and tandem axles, which apply to vehicles on all routes in Delaware. These limitations are shown below.

Vehicle Type	Max. Length (ft)	Gross Vehicle Wt. (k)	Max. Axle Weight (k) Single (Tandem)
1	40 (12.2 m)	40 (178 kN	20 (89 kN)
2	40 (12.2 m)	65* (289 kN)	None
3	40 (12.2 m)	73.28 (326 kN)	None
4	65 (19.8 m)	60 (267 kN)	20 (89 kN)
5	65 (19.8 m)	70 (311 kN)	20/40 (89/178 kN)
6-10	65 (19.8 m)	80 (356 kN)	20/40 (89/178 kN)
11	60 (18.3 m)	80 (356 kN)	20/40 (89/178 kN)

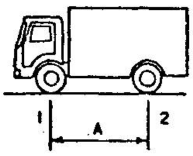
*70 (311,000 N) with Special Annual Permit

Any vehicle which operates in the State of Delaware and exceeds the limitations set forth by the State for size or weight must request and receive a Special Hauling Permit. Information listed on the permit includes gross vehicle weight, legal vehicle weight, vehicle length, and the route which the vehicle will take. No information on axle spacing and axle

NOTE: Table in Feet and Kips
Feet x 0.3048 = m
Kips x 4.4482 = kN

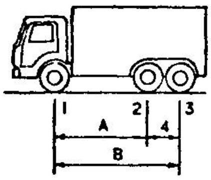
TYPE

1



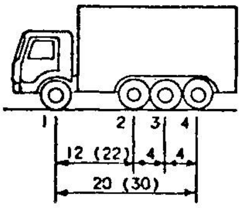
TYPE	SPACING A (FT)	AXLE WEIGHT (K) 1	2	TOTAL	NOTES
1A	15	7	20	27	(6)
1B	10	20	20	40	(1)
1C	30	20	40	60	(3)
1D	35	40	50	90	(3)

2



TYPE	SPACING (FT) A	B	AXLE WEIGHT (K) 1	2	3	TOTAL	NOTES
2A	15	19	16	17	17	50	(5)
2B	12	16	18	26	26	70	(2) (6)
2C	12	16	14	28	28	70	(2)
2D	26	30	14	28	28	70	(2)

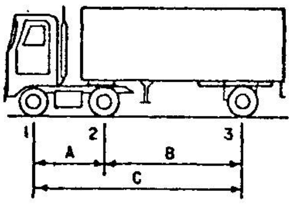
3



TYPE	AXLE WEIGHT (K) 1	2	3	4	TOTAL	NOTES
3A	13.28	20	20	20	73.28	(2)
3B	8	21.76	21.76	21.76	73.28	(2)
3C	18.32	18.32	18.32	18.32	73.28	(2) (6)
3D	13.28	20	20	20	73.28	(2)

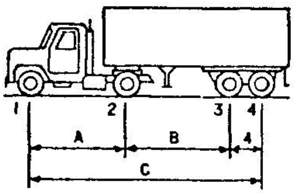
( ) DENOTES SPACING FOR TYPE 3D

4



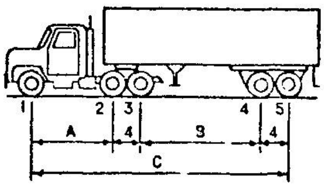
TYPE	SPACING (FT) A	B	C	AXLE WEIGHT (K) 1	2	3	TOTAL	NOTES
4A	10	10	20	11	20	20	51	(1)
4B	10	25	35	11	20	20	51	(1) (6)
4C	12	20	32	20	20	20	60	(1)
4D	12	38	50	20	20	20	60	(1)

5



TYPE	SPACING (FT) A	B	C	AXLE WEIGHT (K) 1	2	3	4	TOTAL	NOTES
5A	11.5	24	39.5	5	20	12.5	12.5	50	(1) (6)
5B	10	18	32	10	12	20	20	62	(1)
5C	10	18	32	10	20	16	16	62	(1)
5D	10	20	34	10	14	20	20	64	(1)
5E	10	20	34	10	20	17	17	64	(1)
5F	12	28	44	10	20	20	20	70	(1)
5G	12	34	50	10	20	20	20	70	(1)

6

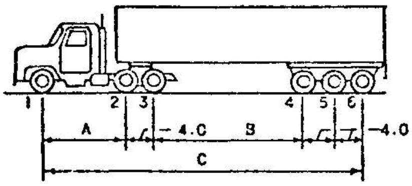


TYPE	SPACING (FT) A	B	C	AXLE WEIGHT (K) 1	2	3	4	5	TOTAL	NOTES
6A	12	12	32	8	16	16	16	16	72	(2)
6B	11	22	41	10	15.5	15.5	15.5	15.5	72	(5)
6C	10	28	46	10	15	10	20	20	76	(1)
6D	11.5	22	41.5	16	16	16	16	16	80	(6)
6E	10	34	52	10	18	12	20	20	80	(1)
6F	11.5	29.5	49	15	27	27	27	27	123	(4)
6G	11.5	29.5	49	18	23	28	23	28	130	(3)

FIGURE 1 - VEHICLES FOR ANALYSIS (1/2)

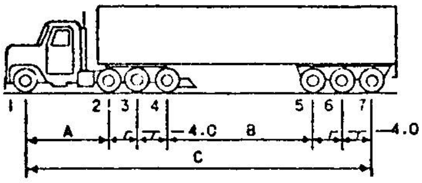
TYPE

7



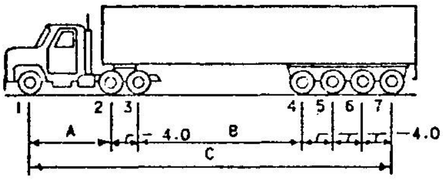
TYPE	SPACING (FT)			AXLE WEIGHT (K)				NOTES
	A	B	C	1	2-3	4-6	TOTAL	
7A	10	26	46	8	2 at 15	3 at 14	80	(1)
7B	12	26	50	15	2 at 27	3 at 27	150	(4)
7C	12	30	54	12	2 at 33	3 at 24	150	(3)

8



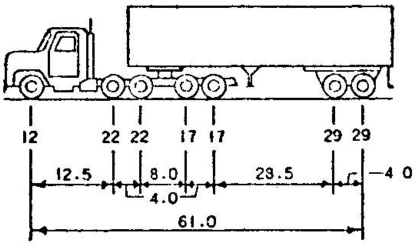
TYPE	SPACING (FT)			AXLE WEIGHT (K)				NOTES
	A	B	C	1	2-4	5-7	TOTAL	
8A	9.33	24.67	50.0	8	3 at 12	3 at 12	80	(1)
8B	9.33	31.67	57.0	9	3 at 22	3 at 25	150	(3)
8C	9.33	24.67	50.0	15	3 at 27	3 at 27	177	(4)

9



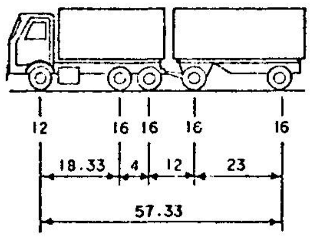
TYPE	SPACING (FT)			AXLE WEIGHT (K)				NOTES
	A	B	C	1	2-3	4-7	TOTAL	
9A	12	22	50	8	2 at 20	4 at 8	60	(1)
9B	11.5	34	61.5	12	2 at 25	4 at 22	150	(3)
9C	12	22	50	15	2 at 27	4 at 27	177	(4)

10, 11



(KIPS)

(FT.)



NOTES:

(1) GROSS VEHICLE WEIGHT LESS THAN OR EQUAL TO MAXIMUM ALLOWED BY FEDERAL BRIDGE FORMULA.

(2) GROSS VEHICLE WEIGHT FROM DELAWARE DMV LAWS.

(3) AXLE WEIGHTS AND SPACING FROM DELAWARE DMV SPECIAL PERMITS.

(4) AXLE WEIGHTS FROM PENNDOT SPECIAL PERMITS.

(5) AXLE WEIGHTS AND SPACING FROM AASHTO MANUAL FOR MAINTENANCE INSPECTION OF BRIDGES - 1978.

(6) AXLE SPACING AND WEIGHT DISTRIBUTION FROM UNPUBLISHED NATIONWIDE STUDY.

FIGURE 1 - VEHICLES FOR ANALYSIS (2/2)

weight is required for vehicles with a gross weight between 80 kips (356 kN) and 120 kips (534 kN). For vehicles exceeding 120 kips (534 kN), a sketch is attached to the permit which shows axle spacings and axle weights.

A search was made through all of the nearly 25,000 Special Hauling Permits issued for 1979. Almost 400 vehicles which exceed the appropriate legal weight followed a route which could take them over the St. Georges Bridge. Approximately 100 vehicles exceeding 120 kips (534 kN) received permits in 1979, and all the information given on the permit sketch was recorded regardless of whether or not the vehicle crossed the St. Georges Bridge. Representatives of many of these vehicles are included in the set of proposed special vehicles. Sixteen vehicles exceeding 120 kips (534 kN) were routed over the St. Georges Bridge, and every one of these 16 vehicles is represented in the set of special vehicles.

From the information compiled from these studies, 41 vehicle configurations were developed which represented the axle combinations, spacings and axle weight ratios found. The 41 vehicle configurations are shown in Figure 1.

COMPUTER ANALYSIS

The computer program written to execute the Scope of Work uses data from two sources. The first is disk files of influence lines and member load and property data which are stored by several small "service" programs. The second source of data is to be provided by the user for each truck to be investigated. These data include:

(1) Three lines of descriptive titles.

(2) A description of the loading in each lane given as (1) any negative number to indicate a special vehicle, or (2) the "HS number" (e.g. "20" for HS20, "15" for HS15, etc.).

(3) The multiple of the design allowable stresses to be used, i.e., the provision to use 110% etc. of design allowable stress.

(4) The longitudinal distance between two trucks in a lane.

(5) The number of axles, axle spacing and weights for the special vehicle.

If the inputted distance between tandem vehicles is not zero, a duplicate set of axle weights and distances is appended to the original set such that the rear axle of the first vehicle and the front axle of the second vehicle are the inputted distance apart. The complete train of axle weights and distances are then copied in reverse.

The two (i.e. forward and reverse) axle trains are then moved along previously stored influence lines for the moment, shear, reaction or axial force being investigated. Each wheel of both axle trains is positioned over selected points on the influence line and the result is compared with previous results and retained if it is a maximum or minimum.

If Case I or Case II loading has been described (i.e. more than one lane of traffic), then the AASHTO lane load uniform load of 640 #/ft per lane (9.34 kN/m) is placed on the influence line. The uniform load is placed only as appropriate to add, algebraically, to maximums and minimums after allowance is made for the location of the special vehicle.

There are six permissible analytical lane positions on the bridge. Lanes 1-4 are the normal traffic lanes; 5 and 6 are additional positions for use with loading Case III. Lane 5 is in the middle of normal traffic Lanes 1 and 2, Lane 6 is in the middle of Lanes 3 and 4. (Note that there is a permanent median barrier on the bridge.) The user defines what vehicles are in the lanes using the following rules:

- The special vehicle must be in at least one lane.
- Lane 5 must not be used if Lane 1 and/or Lane 2 is used, and likewise for Lanes 3, 4 and 6.
- Any other combinations of lanes is permissible.
- The special vehicles should be in the left lanes of any lane pattern chosen.

The previously stored disk files contain the force in the member being studied corresponding to HS20 which are scaled to "HSXX", a distribution factor is applied to the member force corresponding to live load in each lane, impact is applied and the member forces corresponding to each lane rank-ordered. If stringers for which the S/11 distribution factor is applicable in design are being studied with a loading involving only one loaded lane, then the distribution factor will be taken as S/14. (S is the stringer spacing.)

Once the maximum and minimum live load forces are found, they are combined with the dead load forces and stresses and interaction values are computed using data on the disk files. The stresses resulting from the combined loads using the original axle weights are saved for output.

The computation of allowable GVW proceeds by a straight-forward algebraic calculation for stringers for which the S/11 distribution factor is applicable. For all other approach and floor system members, a trial and error process based on the classical interval halving procedure is used to find the scaled GVW for which an interaction value is equal to 1.000 ± 0.001. The special vehicle lane force is scaled up or down, as required, and the live and dead loads are recombined and stresses and interaction values are re-evaluated for 1 or 2 lanes at 100%, 3 lanes at 90% and 4 lanes at 75%. A trail and error procedure is used so that any possibility that a different combination of lanes would create higher combined live load forces for varying weights of the special vehicle is accounted for.

The main members of the arch have the additional complication of having an axial load and two end moments, each of which could be maximized by a different position of the special vehicle and the HSXX vehicles in the other lanes, if any. This was handled by looping through the entire process three times for each main bridge member processed. Each pass through the loop maximized and minimized one of the three forces (axial load and two moments) caused by the special vehicle. The maximum and minimum of the other two forces were found corresponding to the positions of the special vehicles required by the first force. Combining lanes, computation of stresses and interaction values and trial and error solution for allowable GVW proceeded. The second force was maximized and minimized and the process repeated. Finally, the third force was maximized and minimized and the process repeated again. The output for the given member contained one set of controlling values from all three loops.

The result of an analysis is a set of stress tables for floor system and main bridge members as shown in Figure 2. The tables for the floor system contain a description of each section of the floor system which was studied, the allowable stress, dead load stress, positive and negative live load stress, total stress and the allowable gross vehicle weight. The live load stresses and total stress are computed using the special truck exactly as inputted. The allowable gross vehicle is the scaled weight of the original vehicle such that the total stress is equal to the allowable stress.

The output for the main bridge members is essentially the same as the output for the floor system, except that yield point of the material is printed instead of the allowable stress, and an interaction value is printed. The allowable gross vehicle weight is scaled such that the interaction value is $1.0 \pm 0.1\%$. It is possible that the live load stresses

ST. GEORGES BRIDGE
LOADING IS: SPECIAL, HS15, HS15, HS20
7-9-80 JMF MAX GVW REPORTED = 3 X ORIGINAL GVW

HS NUMBERS -1.0 15.0 15.0 20.0 0.0 0.0 STRESS SCALE 1.0000
SINGLE VEHICLE IMPACT INCLUDED
AXLE WT 7.00 20.00
DISTANCES 0.00 15.00

MEMBER IDENTIFICATION		ALLOW STRESS	DL STRESS	+LL STRESS	-LL STRESS	TOTAL STRESS	ALLOW GVW
FASCIA STRINGERS	CENTERLINE	18.00	2.18	5.37	0.00	7.56	81.0
	SHEAR	13.50	1.24	3.49	0.00	4.73	81.0
STRINGERS(S3,S53)	CENTERLINE	18.00	3.43	7.29	0.00	10.72	55.1
STRINGER(S4)	CENTERLINE	18.00	3.45	7.04	0.00	10.49	57.1
STRINGERS(S5,S55)	CENTERLINE	18.00	3.53	6.89	0.00	10.42	58.3
INT R B STRG(SS2-4,S2-54)	CENTERLINE	18.00	7.02	4.91	0.00	11.93	69.4
INT R B STRG(SS5,6,55,56)	CENTERLINE	18.00	6.60	6.26	0.00	12.86	55.2
MAIN SPAN STRINGER(S101)	CENTERLINE	18.00	5.84	6.66	0.00	12.51	52.4
	FATIGUE-WELD SHEAR	15.66	0.52	0.80	0.00	1.32	81.0
MAIN SPAN STRINGER(S102)	CENTERLINE	18.00	6.02	6.17	0.00	12.19	56.0
	FATIGUE-WELD SHEAR	15.66	0.54	0.80	0.00	1.34	81.[illegible]
MAIN SPAN STRINGER(S103)	CENTERLINE	18.00	5.36	5.51	0.00	10.87	66.9
MAIN SPAN STRINGER(S104)	CENTERLINE	13.00	6.35	6.38	0.00	12.73	52.5
	FATIGUE-MOM.(9.13)	18.00	3.12	4.85	0.00	7.97	81.0
	FATIGUE-WELD SHEAR	15.66	0.62	0.87	0.00	1.49	81.0
MAIN SPAN STRINGER(S105)	CENTERLINE	20.00	4.39	6.40	0.00	11.29	69.0
MAIN SPAN STRINGER(S105C)	CENTERLINE	18.00	6.02	5.57	0.00	11.59	62.5
	FATIGUE-SHEAR	15.66	0.40	0.75	0.00	1.15	81.0
END FLOORBEAMS(FB1,FB51)	FIRST CUT-OFF	18.00	3.74	8.71	0.00	12.45	81.0
	(P3) CENTERLINE	18.00	3.50	7.98	0.00	11.48	81.0
	(P4) CENTERLINE	18.00	3.60	8.37	0.00	11.97	81.0
END FLOORBEAMS(FB2,FB52)	FIRST CUT-OFF	18.00	2.88	7.62	0.00	10.50	81.0
	SECOND CUT-OFF	18.00	3.48	8.10	0.00	11.58	81.0
	(P3) CENTERLINE	18.00	3.46	7.89	0.00	11.35	81.0
	(P4) CENTERLINE	18.00	3.56	8.27	0.00	11.84	81.0
INT FLOORBEAMS(FB3,FB53)	FIRST CUT-OFF	18.00	5.63	7.88	0.00	13.52	81.0
	(P3) CENTERLINE	18.00	5.14	6.84	0.00	11.99	81.0
	(P4) CENTERLINE	18.00	5.29	7.18	0.00	12.47	81.0
INT FLOORBEAMS(FB4,FB54)	FIRST CUT-OF	18.00	5.57	7.52	0.00	13.08	81.0
	(P3) CENTERLINE	18.00	5.60	7.23	0.00	12.83	81.0
	(P4) CENTERLINE	18.00	5.77	7.59	0.00	13.35	81.0
INT FLOORBEAMS(FB5,FB55)	FIRST CUT-OFF	18.00	7.42	7.02	0.00	14.44	81.0
	SECOND CUT-OFF	18.00	7.53	6.65	0.00	14.18	81.0
	THIRD CUT-OFF	18.00	6.74	5.87	0.00	12.60	81.0
	(P3) CENTERLINE	18.00	6.31	5.45	0.00	11.76	81.0
	(P4) CENTERLINE	18.00	6.48	5.72	0.00	12.19	81.0
	SHEAR	11.00	4.06	3.33	0.00	7.39	81.0
MAIN SPAN END FLOORBEAMS	FIRST CUT-OFF	18.00	6.11	6.29	0.00	12.40	81.0
	AT STRINGER P4	18.00	5.09	5.27	0.00	10.36	81.0
	AT STRINGER P5	18.00	5.78	6.31	0.00	12.09	81.0
	AT STRINGER P6	18.00	5.89	6.60	0.00	12.50	81.0
	AT STRINGER P7	18.00	5.39	5.62	0.00	11.02	81.0
MAIN SPAN INTERMED FLBMS	FIRST CUT-OFF	18.00	8.56	5.50	0.00	14.06	81.0
	SECOND CUT-OFF	18.00	8.02	5.10	0.00	13.13	81.0
	AT STRINGER P4	18.00	6.47	4.17	0.00	10.64	81.0
	AT STRINGER P5	18.00	7.34	4.76	0.00	12.09	81.0
	AT STRINGER P6	18.00	7.49	4.98	0.00	12.46	81.0
	AT STRINGER P7	18.00	6.86	4.34	0.00	11.20	81.0
BUILT-UP STRG(SS1,SS51)	FIRST CUT-OFF	18.00	6.20	5.22	0.00	11.42	67.9
	SECOND CUT-OFF	18.00	6.47	5.04	0.00	11.52	68.7
	CENTERLINE	16.00	6.49	5.09	0.00	11.58	70.2
	SHEAR	10.64	2.48	2.44	0.00	4.92	81.0
	SECOND PANEL SHEAR	6.54	2.27	2.14	0.00	4.40	60.1
BUILT-UP STRG(SS7,SS57)	FIRST CUT-OFF	18.00	9.44	4.92	0.00	14.36	50.9
	SECOND CUT-OFF	18.00	9.19	4.72	0.00	13.91	55.4
	CENTERLINE	18.00	8.05	4.44	0.00	12.49	69.6
	SHEAR	10.64	2.48	2.46	0.00	4.93	81.0
	SECOND PANEL SHEAR	6.54	2.27	2.15	0.00	4.42	59.6
BUILT-UP STRG(SS8,SS58)	FIRST CUT-OFF	18.00	6.28	5.19	0.00	11.47	67.8
	SECOND CUT-OFF	18.00	5.96	4.45	0.00	10.41	81.0
	THIRD CUT-OFF	18.00	6.07	4.66	0.00	10.73	79.0
	CENTERLINE	18.00	5.77	4.53	0.00	10.30	81.0
	FOURTH CUT-OFF	18.00	6.10	4.65	0.00	10.75	77.8
	SECOND PANEL SHEAR	3.90	1.94	1.83	0.00	3.77	81.0
BUILT-UP STRG(SS9,SS59)	FIRST CUT-OFF	18.00	9.46	4.62	0.00	14.08	54.6
	SECOND CUT-OFF	18.00	7.87	3.98	0.00	11.85	77.9
	THIRD CUT-OFF	18.00	7.64	4.05	0.00	11.69	79.3
	CENTERLINE	18.00	7.35	3.93	0.00	11.28	81.0
	FOURTH CUT-OFF	18.00	9.06	4.45	0.00	13.51	60.7
	SECOND PANEL SHEAR	8.90	1.94	1.84	0.00	3.79	81.0

FIGURE 2 - SAMPLE OUTPUT (1/3)

ST. GEORGES BRIDGE
LOADING IS: SPECIAL, HS15, HS15, HS20
7-9-80 JHF MAX GVW REPORTED = 3 X ORIGINAL GVW

HS NUMBERS -1.0 15.0 15.0 20.0 0.0 0.0 STRESS SCALE 1.0000
SINGLE VEHICLE IMPACT INCLUDED
AXLE WT 7.00 20.00
DISTANCES 0.00 15.00

MEMBER IDENTIFICATION		ALLOW STRESS	DL STRESS	+LL STRESS	-LL STRESS	TOTAL STRESS	ALLOW GVW
80 FT. GIRDERS(G4,G54)	FIRST CUT-OFF	18.00	8.83	4.12	0.00	12.95	81.0
	SECOND CUT-OFF	18.00	10.71	4.79	0.00	15.50	75.5
	CENTERLINE	18.00	10.40	4.68	0.00	15.08	81.0
	SHEAR	9.65	3.64	2.83	0.00	6.47	81.0
	SECOND PANEL SHEAR	6.89	3.53	2.59	-0.00	6.11	55.3
98 FT. GIRDER(G56)	FIRST CUT-OFF	18.00	7.69	4.19	0.00	11.87	81.0
	SECOND CUT-OFF	18.00	8.84	4.47	0.00	13.31	81.0
	THIRD CUT-OFF	18.00	8.93	4.57	0.00	13.50	81.0
	CENTERLINE	18.00	9.12	4.57	0.00	13.68	81.0
105 FT. GIRDER(G3)	FIRST CUT-OFF	18.00	8.14	4.18	0.00	12.32	81.0
	SECOND CUT-OFF	18.00	8.68	4.23	0.00	12.91	81.0
	THIRD CUT-OFF	18.00	8.99	4.33	0.00	13.32	81.0
	FOURTH CUT-OFF	18.00	9.11	4.39	0.00	13.50	81.0
	CENTERLINE	18.00	9.10	4.32	0.00	13.42	81.0
	SECOND PANEL SHEAR	8.40	3.93	2.18	-0.00	6.11	81.0
120 FT. GIRDERS(G2,G52)	FIRST CUT-OFF	18.00	8.94	3.93	0.00	12.87	81.0
	SECOND CUT-OFF	18.00	9.72	4.10	0.00	13.82	81.0
	THIRD CUT-OFF	18.00	9.33	3.99	0.00	13.33	81.0
	FOURTH CUT-OFF	18.00	9.55	3.99	0.00	13.54	81.0
	CENTERLINE	18.00	9.61	3.94	0.00	13.55	81.0
130 FT. GIRDERS(G1,G51)	FIRST CUT-OFF	24.00	12.77	5.20	0.00	17.97	81.0
	SECOND CUT-OFF	24.00	12.51	5.22	0.00	17.73	81.0
	THIRD CUT-OFF	24.00	12.83	5.23	0.00	18.06	81.0
	CENTERLINE	24.00	12.68	5.10	0.00	17.78	81.0
CONT. GIRDER(61.67 SPAN)	FIRST CUT-OFF	18.00	-6.97	4.74	-5.91	-12.88	81.0
	SECOND CUT-OFF	18.00	-10.68	1.58	-5.10	-15.78	76.1
	THIRD CUT-OFF	18.00	-11.07	0.34	-4.51	-15.58	81.0
CONT. GIRDER(INT SUPPORT)	INTERIOR SUPPORT	18.00	-10.88	0.00	-4.33	-15.22	81.0
CONT. GIRDER(105.67 SPAN)	FIRST CUT-OFF	18.00	-10.21	0.14	-3.96	-14.17	81.0
	SECOND CUT-OFF	18.00	-8.91	0.59	-3.52	-12.44	81.0
	THIRD CUT-OFF	18.00	-9.22	1.37	-3.95	-13.18	81.0
	FOURTH CUT-OFF	18.00	7.67	6.32	-1.63	13.99	81.0
	FIFTH CUT-OFF	18.00	7.73	5.32	-1.06	13.05	81.0
	SIXTH CUT-OFF	18.00	9.46	5.33	-0.67	14.79	81.0
	SPAN CENTERLINE	18.00	8.70	4.92	-0.52	13.61	81.0
	62.94 FROM INT SUPP.	18.00	9.53	5.09	-0.42	14.62	81.0
	73.43 FROM INT SUPP.	18.00	8.56	4.74	-0.31	13.31	81.0
	SEVENTH CUT-OFF	18.00	9.36	5.15	-0.32	14.51	81.0
	EIGHTH CUT-OFF	18.00	9.51	5.19	-0.27	14.69	81.0
	NINTH CUT-OFF	18.00	8.23	5.00	-0.23	13.23	81.0
END FAS BRKTS(B1,2,51,52)	MOMENT	13.50	3.57	4.53	0.00	8.10	59.3
END FAS BRKTS(B2A,52A)	MOMENT	26.00	6.77	8.61	0.00	15.38	60.5
INT FAS BRKTS(B3,53)	MOMENT	26.00	12.07	9.45	0.00	21.51	42.3
INT FAS BRKTS(B4,54)	MOMENT	26.00	12.20	9.55	0.00	21.75	41.3
INT FAS BRKTS(B4A,54A)	MOMENT	18.00	6.84	5.35	0.00	12.19	61.9
	SHEAR	13.50	5.46	4.77	0.00	10.23	49.0
SPECIAL BRKTS(B5,55)	MOMENT	26.00	9.38	3.48	0.00	12.86	81.0
	SHEAR	13.50	6.60	3.98	0.00	10.59	50.2
INT FAS BRKTS(B6,56)	MOMENT	13.50	5.38	4.63	0.00	10.01	50.3

FIGURE 2 - SAMPLE OUTPUT (2/3)

ST. GEORGES BRIDGE
LOADING IS: SPECIAL, HS15, HS15, HS20
7-9-80 JMF MAX GVW REPORTED = 3 X ORIGINAL GVW.

HS NUMBERS -1.0 15.0 15.0 20.0 0.0 0.0 STRESS SCALE 1.0000
SINGLE VEHICLE IMPACT INCLUDED
AXLE WT 7.00 20.00
DISTANCES 0.00 15.00

MEMBER IDENTIFICATION			YIELD STRESS	DL STRESS	+LL STRESS	-LL STRESS	MAX STRESS	INTER VALUE	ALLOW GVW
HANGER	U1-L1	INTERACTION	45.00	13.26	2.91	-1.11	16.17	0.6534	81.0
	U2-L2	INTERACTION	33.00	10.74	3.61	-1.90	14.35	0.7904	81.0
	U3-L3	INTERACTION	33.00	9.20	4.45	-3.14	13.65	0.7522	81.0
	U4-L4	INTERACTION	33.00	8.48	5.13	-3.90	13.62	0.7503	81.0
	U5-L5	INTERACTION	33.00	7.89	5.73	-4.52	13.62	0.7504	81.0
	U6-L6	INTERACTION	33.00	7.35	6.97	-5.65	14.21	0.7832	81.0
	U7-L7	INTERACTION	33.00	6.78	7.31	-5.95	14.00	0.7712	81.0
ARCH RIB	L0-U1	INTERACTION	45.00	-15.25	0.47	-2.39	17.57	0.8747	81.0
	U1-U2	INTERACTION	45.00	-14.51	0.95	-2.68	16.64	0.8253	81.0
	U2-U3	INTERACTION	45.00	-15.74	1.12	-2.96	18.35	0.9168	81.0
	U3-U4	INTERACTION	45.00	-15.60	1.34	-3.18	18.45	0.9152	81.0
	U4-U5	INTERACTION	45.00	-15.97	1.31	-3.22	18.92	0.9250	81.0
	U5-U6	INTERACTION	45.00	-16.13	0.96	-3.07	18.60	0.9125	81.0
	U6-U7	INTERACTION	45.00	-16.42	0.57	-3.07	19.10	0.9244	81.0
	U7-U7'	INTERACTION	45.00	-15.31	0.23	-2.82	19.55	0.9458	81.0
TIE GIRDER	L0-L1	INTERACTION	45.00	10.90	3.82	-2.90	14.72	0.5943	81.0
		SHEAR	45.00	0.96	1.68	-1.23	2.64	0.1776	81.0
	L1-L2	INTERACTION	45.00	11.30	5.87	-4.79	17.15	0.6928	81.0
		SHEAR	45.00	0.53	1.22	-0.91	1.75	0.1130	81.0
	L2-L3	INTERACTION	45.00	11.17	6.35	-5.22	17.50	0.7072	81.0
		SHEAR	45.00	0.57	0.98	-0.83	1.55	0.1045	81.0
	L3-L4	INTERACTION	45.00	11.48	6.40	-5.27	17.75	0.7172	81.0
		SHEAR	45.00	0.50	0.90	-0.87	1.40	0.0943	81.0
	L4-L5	INTERACTION	45.00	11.90	6.51	-5.14	18.09	0.7309	81.0
		SHEAR	45.00	0.46	0.99	-1.10	1.45	0.0978	81.0
	L5-L6	INTERACTION	45.00	12.06	5.82	-4.25	17.64	0.7126	81.0
		SHEAR	45.00	0.40	1.18	-1.29	1.58	0.1063	81.0
	L6-L7	INTERACTION	45.00	13.86	5.56	-3.49	19.20	0.7759	81.0
		SHEAR	45.00	0.33	1.31	-1.37	1.64	0.1106	81.0
	L7-L7'	INTERACTION	45.00	13.90	4.59	-2.66	18.31	0.7397	81.0
		SHEAR	45.00	0.30	1.37	-1.37	1.67	0.1122	81.0

SUMMARY -	STRINGERS	FLOORBEAMS	GIRDERS	BRACKETS	MAIN BR
	52.426	81.000	50.922	41.331	81.000
CONTROLLING VALUE =		41.331			

FIGURE 2 - SAMPLE OUTPUT (3/3)

and dead load stress printed in the output tables will not add up to the printed value of the total stress. This is because the dead load stress and the maximum and minimum live load stresses may be computed at different locations along the member. The total stress is computed using dead load and live load stresses which occur at the same locations.

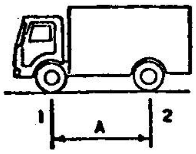
The final table is a summary of all the preceeding tables and contains the controlling GVW for the stringers, floorbeams, girders, brackets and main bridge members, and the final controlling value.

The computer program was used repeatedly to analyze all 41 vehicle configurations for each of the loading cases indicated in the Scope of Work. As an example of the final product, Figure 3 shows the final results for loading Cases I and II. These summary figures are to be used as permit-issuance guides. Vehicles not adequately represented by one or a combination of the 41 configurations are to be analyzed using the program.

NOTE: Table in Feet and Kips
Feet x 0.3048 = m
Kips x 4.4482 = kN

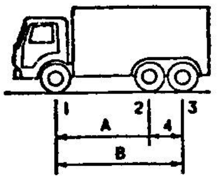
TYPE

1



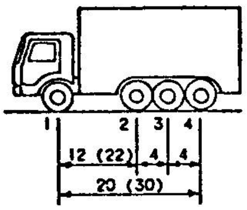
TYPE	SPACING A (FT)	ORIG. AXLE WT. (K) 1	2	TOTAL	ALLOW. GVW (K)
1A	15	7	20	27	41.3
1B	10	20	20	40	43.1
1C	30	20	40	60	51.2
1D	35	40	50	90	61.4

2



TYPE	SPACING (FT)		ORIGINAL AXLE WT. (K)				ALLOW. GVW (K)
	A	B	1	2	3	TOTAL	
2A	15	19	16	17	17	50	49.7
2B	12	16	18	26	26	70	44.7
2C	12	16	14	28	28	70	43.3
2D	26	30	14	28	28	70	47

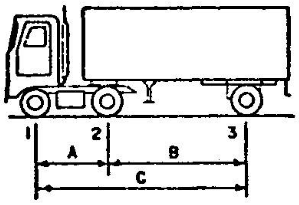
3



TYPE	ORIGINAL AXLE WT. (K)					ALLOW. GVW (K)
	1	2	3	4	TOTAL	
3A	13.28	20	20	20	73.28	46.6
3B	8	21.76	21.76	21.76	73.28	44
3C	18.32	18.32	18.32	18.32	73.28	50.5
3D	13.28	20	20	20	73.28	49.8

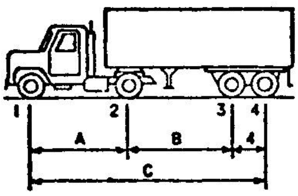
( ) DENOTES SPACING FOR TYPE 3D

4



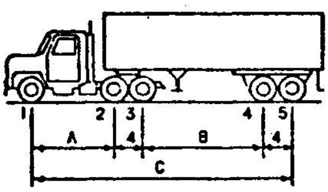
TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)				ALLOW. GVW (K)
	A	B	C	1	2	3	TOTAL	
4A	10	10	20	11	20	20	51	51
4B	10	25	35	11	20	20	51	68.4
4C	12	20	32	20	20	20	60	64.3
4D	12	38	50	20	20	20	60	70.3

5



TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)					ALLOW. GVW (K)
	A	B	C	1	2	3	4	TOTAL	
5A	11.5	24	39.5	5	20	12.5	12.5	50	68.6
5B	10	18	32	10	12	20	20	62	57.6
5C	10	18	32	10	20	16	16	62	62.8
5D	10	20	34	10	14	20	20	64	60.9
5E	10	20	34	10	20	17	17	64	64.1
5F	12	28	44	10	20	20	20	70	65.6
5G	12	34	50	10	20	20	20	70	65.6

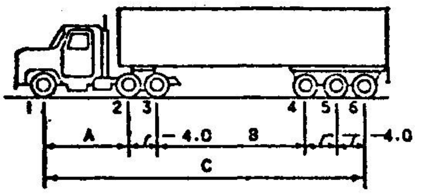
6



TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)						ALLOW. GVW (K)
	A	B	C	1	2	3	4	5	TOTAL	
6A	12	12	32	8	16	15	16	16	72	62
6B	11	22	41	10	15.5	15.5	15.5	15.5	72	72.1
6C	10	28	46	10	16	10	20	20	76	71.3
6D	11.5	22	41.5	16	16	16	16	16	80	74.5
6E	10	34	52	10	18	12	20	20	80	75
6F	11.5	29.5	49	15	27	27	27	27	123	80
6G	11.5	29.5	43	18	23	23	23	20	130	80.7

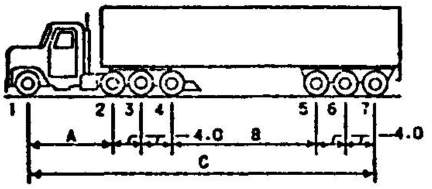
FIGURE 3 - COMBINED RESULTS FOR CASE I AND CASE II (1/2)

7



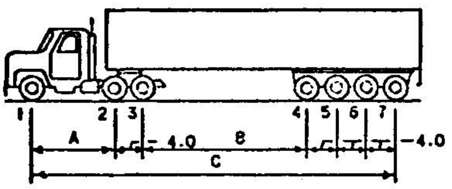
TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)				ALLOW. GVW (K)
	A	B	C	1	2-3	4-6	TOTAL	
7A	10	26	48	8	2 at 15	3 at 14	80	76.4
7B	12	26	50	15	2 at 27	3 at 27	150	75.6
7C	12	30	54	12	2 at 33	3 at 24	150	83.1

8



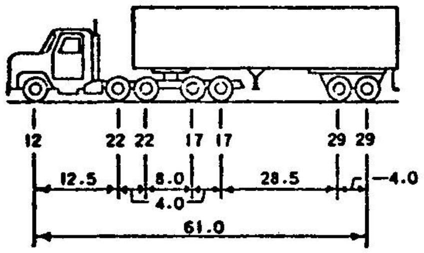
TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)				ALLOW. GVW (K)
	A	B	C	1	2-4	5-7	TOTAL	
8A	9.33	24.67	50.0	8	3 at 12	3 at 12	80	81.6
8B	9.33	31.67	57.0	9	3 at 22	3 at 25	150	81.5
8C	9.33	24.67	50.0	15	3 at 27	3 at 27	177	80.6

9



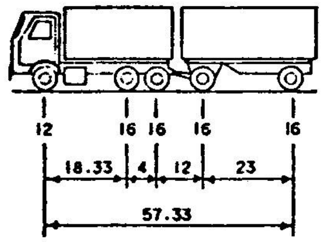
TYPE	SPACING (FT)			ORIGINAL AXLE WT. (K)				ALLOW. GVW (K)
	A	B	C	1	2-3	4-7	TOTAL	
9A	12	22	50	8	2 at 20	4 at 8	80	74.5
9B	11.5	34	61.5	12	2 at 25	4 at 22	150	74.7
9C	12	22	50	15	2 at 27	4 at 27	177	74.4

10, 11



ORIGINAL GVW - 148^K
ALLOWABLE GVW - 90.6^K

(KIPS)
(FT.)



ORIGINAL GVW - 76^K
ALLOWABLE GVW - 79.3^K

FIGURE 3 - COMBINED RESULTS FOR CASE I AND CASE II (2/2)

FATIGUE ANALYSIS

Information provided by DelDOT indicated that in 1978 the ADTT was 1,881 trucks per day in one direction, and that the ADTT projected for 1995 was 3,128 trucks per day in one direction. The distribution of gross vehicle weights was assumed to be represented by the total of the 1977 and 1979 Delaware Loadometer Surveys for the stations near the bridge, given below. The DelDOT surveys found no vehicles with a GVW over 120 kips (534 kN). The permits reviewed by Modjeski and Masters indicated that in 1979, 16 vehicles weighing up to 150 kips (667 kN) traveled a route which could have taken them over the St. Georges Bridge. Assuming an ADTT of 1,954 trucks per day in one direction of 1979, these 16 vehicles could constitute 0.00112% of the traffic if they crossed only once per permit evenly divided between northbound and southbound traffic. Most of the permits reviewed were for one-way trips. Even if 10 times as many of these vehicles, each weighing 150k (667 kN), crossed the bridge in both directions, they would affect the percent of HS20 equivalent stress cycles only about 1/2%. Therefore, the DelDOT loadometer data is considered sufficiently representative of all traffic using the St. Georges Bridge, including the occasional extremely heavy permit vehicles. Application of Miner's Rule with an exponent of 3 as shown in Reference 1 yields 41.6% equivalent HS20 trucks. This compares to 35% equivalent HS20 trucks found in the 1970 FHWA Nationwide Loadometer Survey.

Wt. Range	No. of Vehicles	Average Wt. Kips (kN)	No. of HS20
0 to 5	323	4.3 (19.1)	0.1
5 to 10	204	6.8 (30.2)	0.2
10 to 15	72	12.2 (54.3)	0.3
15 to 20	83	17.6 (77.8)	1.2
20 to 25	87	22.5 (100.1)	2.7
25 to 30	127	27.6 (122.8)	7.1
30 to 35	83	32.4 (144.1)	7.6
35 to 40	103	37.8 (168.1)	14.9
40 to 45	59	42.2 (187.7)	11.9
45 to 50	42	47.6 (211.7)	12.1
50 to 55	29	52.4 (233.1)	11.2
55 to 60	46	57.6 (256.2)	23.6
60 to 65	53	63.0 (280.2)	35.5
65 to 70	66	67.4 (299.8)	54.1
70 to 75	92	72.5 (322.5)	94.0
75 to 80	79	77.3 (343.8)	97.7
80 to 90	101	85.3 (379.4)	168.0
90 to 100	72	93.6 (416.3)	158.1
100 to 110	5	102.7 (456.8)	14.5
110 to 120	1	117.6 (523.1)	4.4
Total	1727		719.0
Percentage of Total			41.6

Use of the ADTT data and the DelDOT loadometer data resulted in a calculation of 7.4 million equivalent HS20 vehicles crossing the bridge in 70 years in each direction. This is based on an average ADTT of 2,033 and a projected ADTT of 4,555 at the end of a 70 year life. By comparison, the current AASHTO Specification requires design for 2.0 million cycles (longitudinal members) and over 2.0 million cycles (transverse members) for an ADTT of over 2,500.

Initial calculations indicated that the stress range in some riveted details and the shear range in stud connectors of composite members were so high relative to the current AASHTO allowable stress ranges that more sophisticated calculation procedures had to be found to avoid placing undue restrictions on the use of the bridge. Research was reviewed and several experts were contacted for opinions and advice. It was decided that the current specifications, while suitable for the design of new structures, contain many assumptions which can be quite conservative in some cases, particularly as it relates ADTT to design cycles of equivalent HS20 trucks. It also reflects some editorializing by the committees involved with developing the specifications. The following considerations were evaluated in developing criteria to be used on the St. Georges Bridge:

- The current AASHTO fatigue specification is generally believed to result in safe designs despite its many simplifications and assumptions. There are, however, other technically acceptable, presumably more sophisticated procedures which can be used to relate random loading to design cycles and allowable stress ranges. One approach would be to evaluate the most extreme loading possible, and if the stress range corresponding to this loading is below the "runout limit" for the detail under consideration, fatigue cracks will not propagate regardless of the number of cycles. Alternatively, it is possible to compute an "effective stress range" using either Miner's rule or a root-mean-square approach. The "effective stress range" is the stress range corresponding to the total number of cycles which will occur during the design life of the structure, i.e. ADTT x 365 x design life in years. If the point corresponding to the effective stress range and the total number of cycles is below the lower confidence limit for a given fatigue category by some reasonable margin, failure due to fatigue crack propagation is not to be expected during the design life.

- The values of α , a variable used to relate measured stress to calculated design stress, built into the AASHTO Specifications are also conservative. The chosen values were 0.7 for longitudinal members and 0.8 for transverse members. Values somewhat less than this could be used in evaluating existing structures, 0.6 was used for longitudinal members and 0.7 was used for transverse members when computing the effective stress range, and 0.9 was used when computing the extreme value.

- Stresses caused by vehicles were assumed to be proportional to their gross vehicle weight. While not precisely correct, this assumption is necessary to convert the random loading indicated by loadometer histograms into equivalent cycles of constant amplitude loading so that allowable stress ranges may be determined from published data.

- Applications of assumptions above and the equation below results in an effective stress range equal to 44.8% and 52.3% of the HS20 stress range for longitudinal and transverse members, respectively.

$$\sigma_{eff} = \left[\alpha^3 \Sigma \phi_i \left(GVW/GVW\text{-}HS20\right)_i^3\right]^{\frac{1}{3}}$$

where: ϕ_i = % of a given GVW range in a loadometer survey

α = Stress ratio defined above

- There is no technical reason to evaluate existing structures using the criteria for non-redundant members. These criteria were established somewhat arbitrarily by AASHTO with the intent of penalizing certain details, particularly Category "E" details, so badly that designers would choose other details.

- Riveted details can be evaluated using Category "C" instead of "D" if there is reason to believe the rivets are tight.

- In the case of the St. Georges Bridge, use of four loaded lanes as a fatigue loading is unduly restrictive. The "extreme value" case was based on all lanes loaded without the AASHTO multiple lane reduction factors, but the "effective stress range" was based on single vehicle loadings with some allowance for multiple occurrences.

- In the particular case of shear stress range in welded studs, the values in the AASHTO Specification were developed from tests of relatively small specimens which contained only four studs, and which were loaded so as to pry the concrete slab off of the flange of the specimen. Thus, the specification values do not take into account the bond and friction between the concrete deck and the flange. This bond and friction significantly reduces the stress range in the stud connector. The result is that the design values are again, quite conservative. Furthermore, the failure of stud connectors is not a catastrophic event and, if it occurred repeatedly, would lead to slip of the deck relative to the beam which would be detectable during annual inspections. Finally, unless the deck is of modern construction utilizing deck protection systems, the deck will probably have to be replaced before the stud connectors fail. Additional stud connectors could be added when the deck is replaced.

- Impact is a statistical quantity and the AASHTO impact may be regarded as an extreme value. It was thought that statistical analysis of actual impacts might lead to an average impact of about 1/3 of the AASHTO impact value.

- In some cases, design stresses are computed using distribution factors calculated by crowding the vehicles to one side or the other of their design lanes and/or crowding the design lanes into a position of maximum effect. The actual position of vehicles is also a statistical quantity and all cycles of loading will not occur with the same distribution factors.

Implicit in some of the assumptions above is the replacement of the deck slab in the near future, i.e. about 1990 or before. Stress cycles accumulated by the stud connectors (added in 1974) will be on the order of 1.1 million equivalent HS20 cycles by that time (average ADTT = 2,172, 1974-1990). If the actual shear stress was further reduced by only 25 percent due to bond and friction, this would be equivalent to about the 0.5 million cycles for which they were designed. When the deck slab is replaced, shear studs can be added and other members can be upgraded to the then existing AASHTO requirements if so desired.

The findings of fatigue analyses are summarized below.

Item	Acceptable by AASHTO As Amended	Acceptable by Effective Stress Range
Rolled Stringers	Yes	Yes
Built-up Stringers	Yes	Yes
Simple Span Girders	Yes	Yes
Floorbeams	No	Yes
Continuous Girders	Yes	Yes
Brackets	No	No
Main Bridge Members	Yes	Yes

Only the floorbeams will be discussed further.

Stress ranges in floorbeams resulting from three lanes of AASHTO HS20 vehicles at 100% were computed and exceed the 10 ksi (68.9 MPa) AASHTO allowable stress range for Category "C" details on transverse members in 1/3 of the cases investigated. (By comparison, if the riveted details were considered Category "D", the AASHTO allowable stress range would be only 7.0 ksi (48.3 MPa). Accounting for the multi-lane reduction factors, only two of the 30 floorbeam sections investigated met this criterion.)

Some of the stresses computed for 3 lanes of loading were higher than the Category "C" runout limit of 10 ksi (68.9 MPa), so the extreme value concept could not be used. The floorbeams were investigated further using the effective stress range concepts. When only one exterior lane is loaded, the maximum HS20 stress range is 3.3 ksi (22.7 MPa) in controlling floorbeams. The single exterior lane truck loading is the most common form of loading for the floorbeams. This corresponds to an effective stress range of 1.73 ksi (11.9 MPa) which is clearly acceptable. If both exterior lanes were loaded simultaneously, the HS20 stress range would be 4.02 ksi (27.7 MPa) which results in an effective stress range of 2.10 ksi (14.47 MPa) and is still obviously acceptable.

A more severe loading results when both lanes on one side of the bridge are loaded. A maximum HS20 stress range of 8.92 ksi (61.5 MPa) and a corresponding effective stress range of 4.67 ksi (32.2 MPa) were computed for this loading. If the full one directional ADTT was applied to two vehicles at a time, there would be about 25.6 million cycles of loading with an allowable stress range of 6.0 ksi (41.3 MPa). Considering the statistical nature of impact and the vehicle position within lanes, this is acceptable even for so severe a loading.

Another evaluation of floorbeams could be undertaken by assuming some hypothetical distribution of vehicles to account for multiple occurrences such as: (1) 50 vehicles crossed the bridge in the exterior lanes such that 25 were in each exterior lane, corresponding to 50 cycles of loading, (2) 20 vehicles (10 pairs) crossed in adjacent southbound lanes, (3) 20 vehicles (10 pairs) crossed in adjacent northbound lanes, (4) 20 vehicles (10 pairs) crossed in opposing exterior lanes, and (5) 12 vehicles (3 quads) crossed with all lanes loaded. This traffic distribution results in a total of 83 cycles of load per 122 trucks crossing the bridge. Assuming that this distribution applies to the entire histogram of vehicle weights results in an effective single vehicle stress range of 3.34 ksi (23.0 MPa) in the controlling floorbeams. The number of cycles is 83/122 times the total number of vehicles in both directions or approximately 69.7 million cycles. The allowable stress range would then be about 4.3 ksi (29.6 MPa) which is acceptable.

It was therefore concluded that, while the floorbeams do not satisfy the current AASHTO criteria, as amended herein, they are adequate for a 70 year life.

CASE STUDY #2 - BLUE WATER BRIDGE

INTRODUCTION

The Blue Water Bridge connects the State of Michigan with the Province of Ontario at Port Huron, Michigan. The main bridge is an 871 foot (266 m) cantilever truss. The Michigan approach structures consist of a concrete beam and column supported slab approach (not in project), 1,731 feet (528 m) of steel beam and girder approach spans of varying spans, and 508 feet (155 m) of approach truss spans. The Canadian approach structures consist of 2,100 feet (640 m) of steel beam and girder spans and 508 feet (155 m) of approach truss spans. In addition, there is a flared toll plaza area on structure on the Michigan side, and a flare-on-structure approaching the Ontario toll plaza. The bridge was opened in 1938 and both toll plazas were widened in 1954, and the Ontario plaza was widened again in 1974. The bridge carrys a three lane cartway with passing permitted on the up-grade of the approach structures.

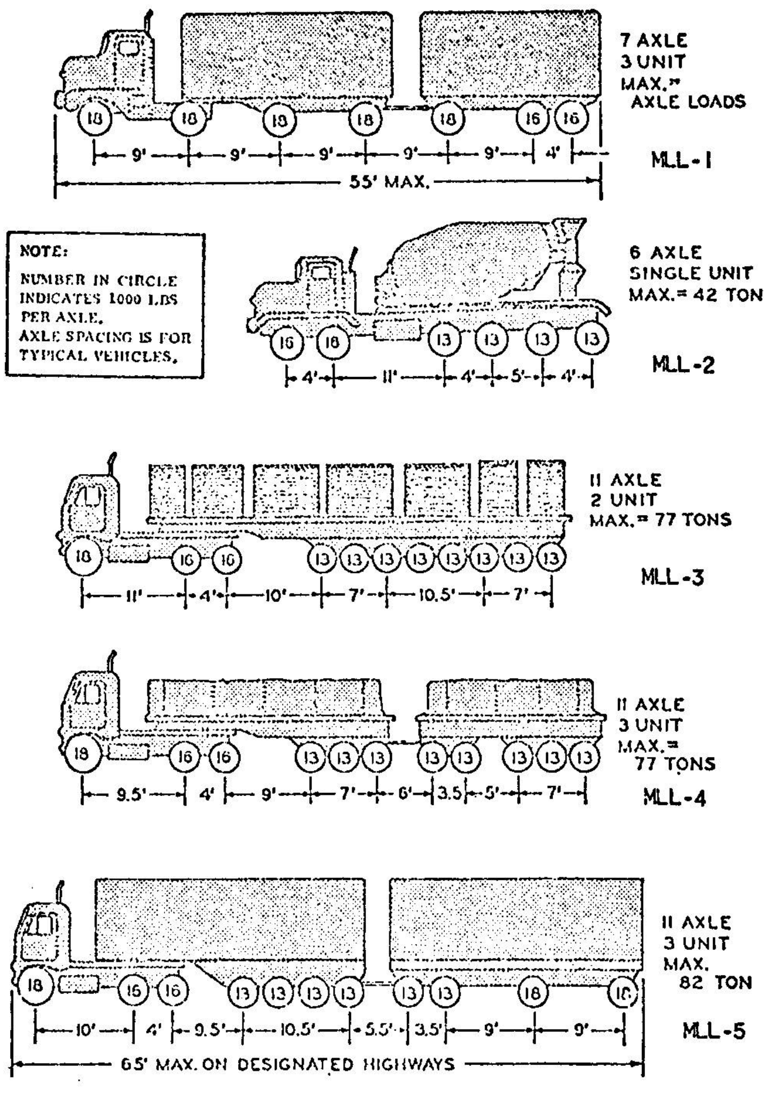
SCOPE OF WORK

This project involved the analysis of fifty-eight vehicles, in two levels of operation, applied to 56 components the steel floor system from end-to-end of the bridge. The fifty-eight vehicle configurations are shown schematically in Figure 4 and consist of:

- Seventeen vehicles, MPL-1 to MPL-17, described in the Michigan Department of Transportation's "Table of Overloads Permissible on Bridges" dated 6/30/78.
- Five vehicles, MLL-1 to MLL-5, shown on the Department's figure "Maximum Gross Vehicle Weights in Michigan in 1970".
- Four vehicles, MMSL-1 to MMSL-4, selected by the Department from the list of Special Vehicles studied prior to 1968.
- Thirty-two vehicles, BWBA-1 to BWBA-32, submitted by the Blue Water Bridge Authority's consultant, which were developed from a study of vehicles crossing the bridge during a three-day period in February, 1979.

The two levels of operation were called the "maintenance condition" which simulated traffic patterns during closure of an exterior lane during maintenance operations, and the "closed bridge condition" in which traffic would be limited to passenger vehicles only to maximize the permissible weight of a special vehicle.

In the "maintenance condition", distinctions were made for operation in the toll plaza areas. Except in the toll plaza areas, two special vehicles were centered in the worse exterior lane and the center lane. Girders and stringers were to be evaluated using 120 percent of the design allowable stress; floorbeams were to be evaluated using 130 percent of the design allowable stress. Stringers in the toll plaza areas were also to be evaluated using 120 percent of design allowable stress. Floorbeams and girders in the plaza areas were to be evaluated using the more critical of (1) a single special vehicle positioned for maximum effect at 110 percent of design allowable stress, or (2) a special vehicle positioned for maximum effect, and adjacent HS20 vehicles centered in 12 foot (3.7 m) lanes, at 80 percent of yield stress.



MAXIMUM GROSS VEHICLE WEIGHTS IN MICHIGAN IN 1970

MICHIGAN DEPARTMENT OF STATE HIGHWAYS AND TRANSPORTATION
OVERLOADS PERMISSIBLE ON BRIDGES

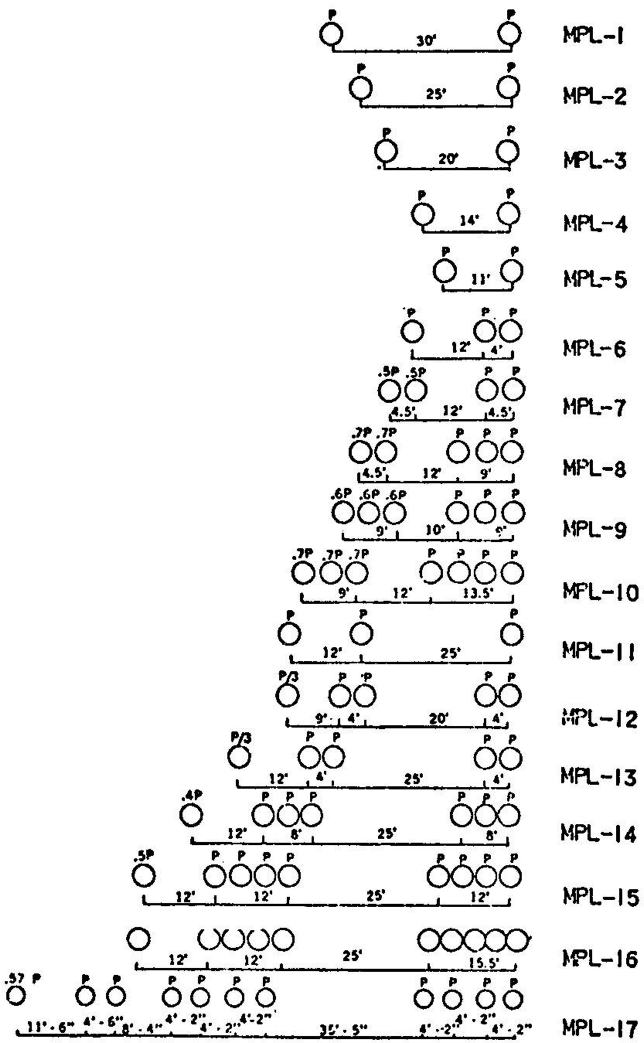


FIGURE 4 - SPECIAL VEHICLES - BLUE WATER

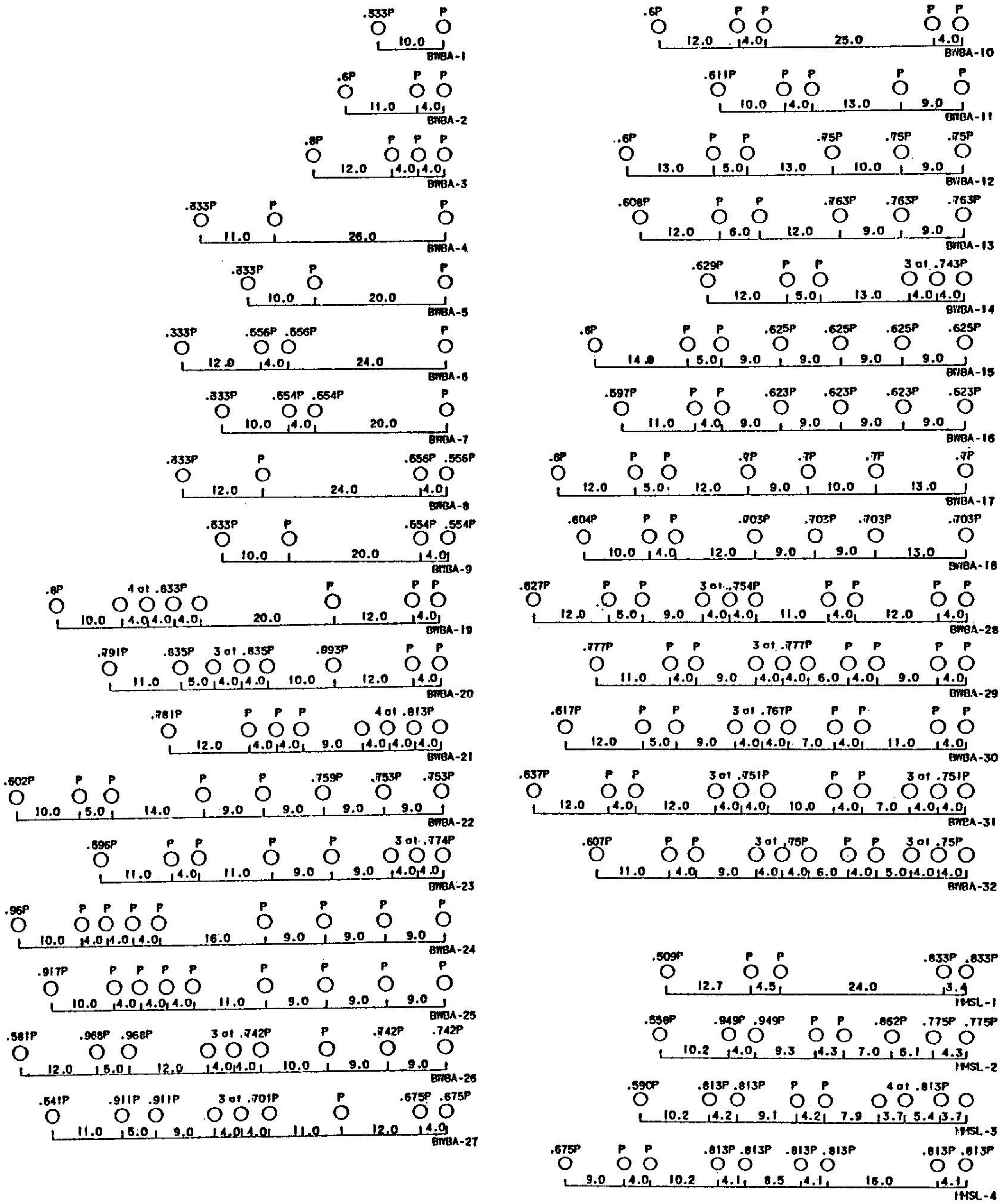


FIGURE 4 - SPECIAL VEHICLES - BLUE WATER

For the "closed bridge condition", one special vehicle was positioned in the center of the middle lane, or centered on either one of the middle lane stripes. In the plaza areas, the special vehicle was assumed to be centered along the projected centerline of bridge, or centered 5 feet (1.4 m) from the projected bridge centerline. All members were evaluated using 110 percent of the design allowable stress.

The objectives of this project were:

- o To develop a set of tables to be used by the MDOT Bridge Section to define the maximum permissible axle weight and corresponding gross vehicle weight for all 58 vehicle configurations for both the "maintenance condition" and the "closed bridge condition". The maximum permissible axle weights and gross vehicle weights were those determined by rigorous analysis.
- o Developed a set of tables based on simple rules-of-thumb, to define permissible gross vehicle weights for given length vehicles to be used by the MDOT Permit Section. The rules-of-thumb were established for both "maintenance condition" and the "closed bridge condition" using only the vehicle configurations developed by the Blue Water Bridge Authority's consultant (BWBA-1 to BWBA-32).
- o The impact of strengthening selected floor system members on the permit load capacity was to be evaluated. (This requirement not discussed herein.)

SUMMARY OF RESULTS

The allowable GVW's for each of the 58 vehicles for both conditions of operation are shown in Figure 5; the top band of points correspond to the "closed bridge condition", the lower set to the "maintenance condition". The source of the individual loads is also indicated by the symbols in the legend of Figure 5. These data points, tabulated by vehicle name, satisfied all requirements of the Scope of Work calling for data obtained by rigorous analysis.

Also shown on Figure 5 are three bi-linear curves which are the "rules-of-thumb" required for use by the Department's Permit Section. The lowest curve is defined as:

$$L < 20 \quad W = 40{,}000 + 1{,}000\,L$$
$$(L < 6.1 \quad W = 178 + 14.6\,L)$$
$$L > 20 \quad W = 36{,}000 + 1{,}200\,L \leq 120{,}000$$
$$(L > 6.1 \quad W = 160 + 17.5\,L \leq 534)$$

"W" is the gross vehicle weight in pounds (kN) and "L" is the distance between the centers of the front and rear wheels in feet (m).

The intermediate lines shown in Figure 5 represent an upper bound for gross vehicle weights for the "maintenance condition" based on all fifty-eight vehicle configurations and is given by the following rule-of-thumb.

$$L < 20 \quad W = 35{,}500 + 2{,}100\,L$$
$$(L < 6.1 \quad W = 158 + 30.7\,L)$$
$$L > 20 \quad W = 55{,}500 + 1{,}100\,L \leq 130{,}000$$
$$(L > 6.1 \quad W = 247 + 16.1\,L \leq 578)$$

Vehicles falling above the highest lines can be authorized passage only after review by the Bridge Section using the rigorous data points corresponding to the "closed bridge condition".

The upper curve is for the "closed bridge condition", developed from the thirty-two vehicle configurations submitted by the Blue Water Bridge Authority's consultant. This curve is defined as:

$$L < 50 \quad W = 55{,}000 + 1{,}500\,L$$
$$(L < 15.2 \quad W = 245 + 21.9\,L)$$
$$L > 50 \quad W = 30{,}000 + 2{,}000\,L \leq 170{,}000$$
$$(L > 15.2 \quad W = 133 + 29.2\,L \leq 756)$$

To further expedite issuance of routine permits, the rules-of-thumb given above were at one-foot (.3048 m) intervals. These tables are the "working format" for the Permit Section. The rigorous data are the "working format" for the Bridge Section.

FATIGUE CONSIDERATIONS

The Scope of Work for this project limited fatigue investigations to one detail which induced a weld between filled grid deck and stringers on the main bridge. This weld was considered a Category "C" detail.

A load spectrum was provided by the Michigan Department of Transportation and was reduced to 29.0% equivalent HS20 vehicles based on Miner's Rule. The average ADTT was 286.2 for 70 year life based on the toll records of previous truck traffic and the Department's projections of future traffic. The data above lead to a projection of 644,100 cycles of an HS20 stress range of 14.2 ksi (97.9 MPa) over 70 years. The comparable allowable stress range was estimated to be 18.3 ksi (126.2 MPa). Therefore, fatigue considerations did not control the capacity of stringers supporting the filled grid deck.

REFERENCE

1. Fisher, J. W., "Bridge Fatigue Guide Design and Details", American Institute of Steel Construction, New York, 1977.

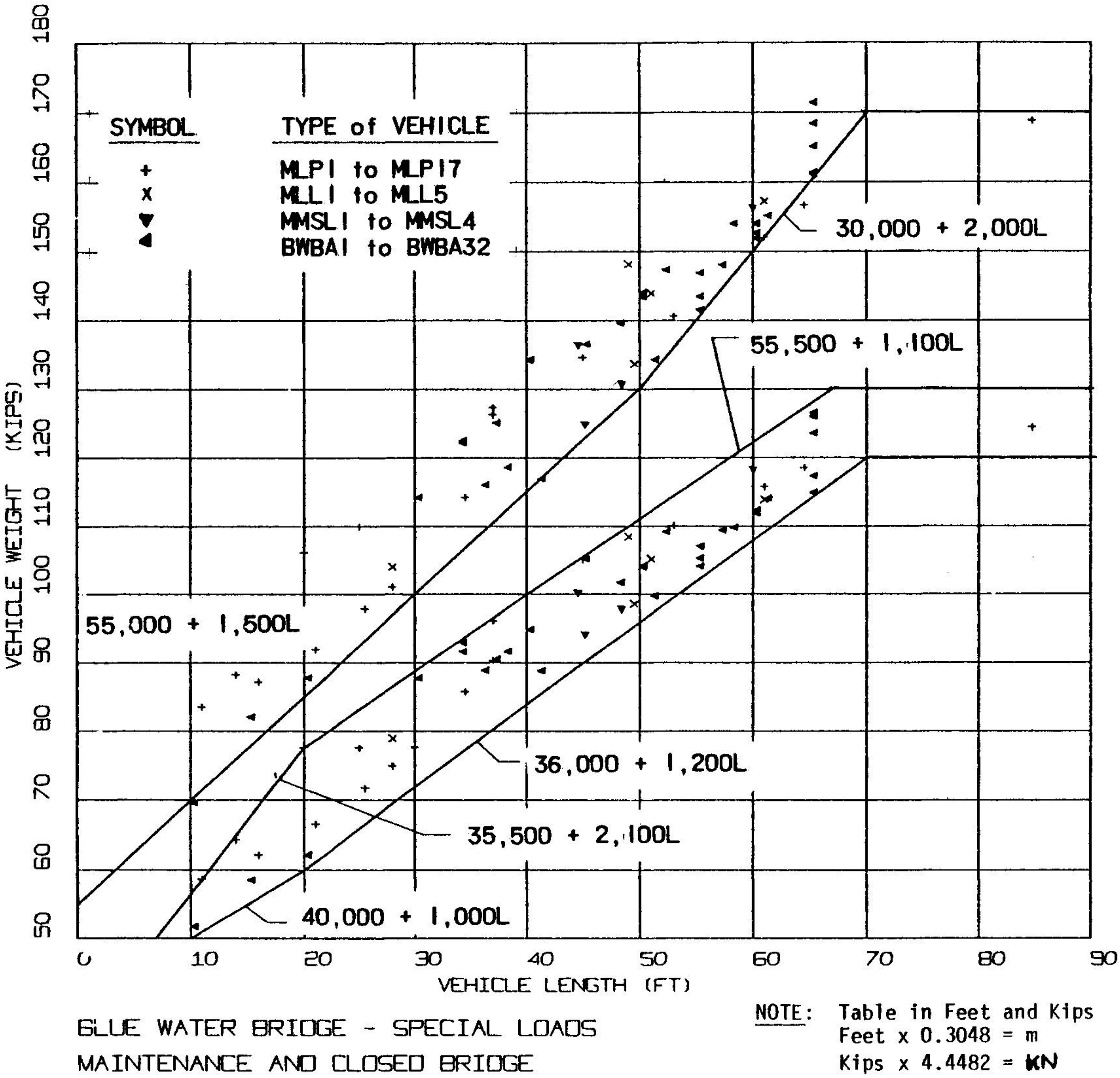


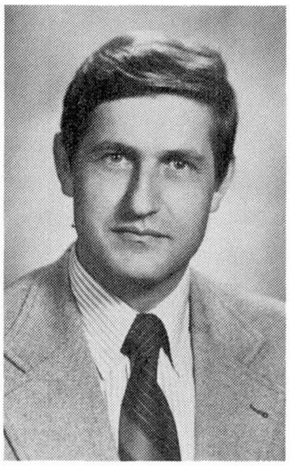
FIGURE 5 - GVW Vs Vehicle Length

Remaining Fatigue Life of Bridges

Durée de vie résiduelle des ponts

Restlebensdauer von Brücken

Manfred A. HIRT
Professor
Swiss Fed. Inst. of Technology
Lausanne, Switzerland



A graduate of ETH Zürich, Manfred A. Hirt received his doctorate from Lehigh University, Bethlehem, PA, USA. He has worked with consultants in Zurich and New York City before joining the Swiss Federal Institute of Technology in Lausanne. He is chairman of the Swiss committee on loads and the ECCS committee for fatigue design of steel structures.

SUMMARY
The evaluation of the remaining fatigue life of an existing structure involves the following important steps. Firstly, two load models, representing past load history and future traffic have to be established. Then the static and dynamic structural performance must be assessed by either computer analysis or in-situ stress measurement or both. Having also established an appropriate fatigue strength curve, the theoretical remaining fatigue life may be evaluated using probabilistic methods. These assessments are becoming increasingly more important as many existing structures are exceeding their design lives.

RESUME
L'estimation de la durée de vie résiduelle d'une structure existante soumise à la fatigue comprend les principales étapes suivantes. En premier lieu il faut établir deux modèles de charges, l'un représentant les charges antérieures supportées par l'ouvrage et l'autre la prévision du trafic à venir. Il faut ensuite déterminer le comportement statique et dynamique soit par une analyse à l'aide de l'ordinateur, soit par les deux moyens. Après avoir également choisi la courbe de fatigue appropriée, la durée de vie résiduelle théorique peut être déterminée à l'aide de méthodes probabilistes. Ces évaluations deviennent d'autant plus nécessaires que de nombreux ouvrages existants ont dépassé leur durée de vie prévue lors du dimensionnement.

ZUSAMMENFASSUNG
Die Abschätzung der Restlebensdauer von bestehenden Konstruktionen umfasst die folgenden wichtigen Schritte: Zuerst werden zwei Lastmodelle aufgestellt und zwar einerseits für die Lastgeschichte und anderseits für den zukünftigen Verkehr. Dann muss das statische und dynamische Tragverhalten durch Computersimulation und/oder durch Spannungsmessungen am Objekt erfasst werden. Liegen die relevanten Ermüdungsfestigkeitswerte ebenfalls vor, kann dann die theoretisch vorhandene Restlebensdauer mit Hilfe von Wahrscheinlichkeitsüberlegungen abgeschätzt werden. Dieses Vorgehen gewinnt zunehmend an Bedeutung, da viele bestehende Konstruktionen ihre Bemessungslebensdauer schon überschritten haben.

1. INTRODUCTION

There are different reasons why an evaluation of the remaining fatigue life might become necessary. The most obvious need occurs when cracks are found in a structure. Another reason for evaluation arises when significant changes have happened during the life of the structure. A third and economically important aspect, particularly due to the large number of cases involved, concerns structures approaching their theoretical design life.

This paper tries to identify the basic parameters needed for the evaluation of the remaining fatigue life. Each parameter is discussed and its data base and importance in the evaluation is considered. Based on this and using commonly accepted rules for cumulative damage, simplified methods of evaluation are shown. It must be added that generalized rules are neither available nor have been agreed upon, as yet.

It should also be recognized that one of the most important aspects of the evaluation procedure is the insight into the problem, and the ensuing possibility for correctly rating the structure. Deterministic approaches are generally used, sometimes introducing statistical values for the fatigue strength. More research is under way to establish clear lines for assessing the probability of survival using modern safety concepts. Such procedures are hindered by lack of knowledge of the effects of loading and the need to calibrate with experience.

A closely related problem is the rating of a complete set of structures, for example all railway bridges on a given stretch of line, or all highway bridges in a county or state. This aspect will become more and more important since the number of "old" bridges increases every year. Therefore, decisions have to be made whether to keep these bridges in service beyond their theoretical design life, to replace them, or to strengthen them. Priorities to carry out this work also need to be established.

2. MOTIVATION AND GOALS

The main purpose of the evaluation of the remaining fatigue life resides in the rating of the structure. This rating has to include decisions on various actions such as inspection, retrofit, repair, strengthening and replacement of elements or even the whole structure.

There are three distinct circumstances where such a rating is needed :

1.- cracks are found in a structure,
2.- the structure approaches its design life,
3.- it is recognized that important changes have occurred.

In the first case, immediate action has to be taken in order to decide whether or not a structure has to be closed to traffic. The investigation, very often based on modern methods of fracture mechanics, will reveal what type of repairs or retrofit procedures are needed to keep the structure in service.

The second case involves an increasingly large number of structures. New codes commonly define design lives of the order of 30 years for crane gantry girders, 50 years for highway bridges, and 100 years or more for railway bridges. The public and even many engineers relate these ("arbitrarily" chosen design) values to existing structures, although they may never have been designed for fatigue. Approaching this design life is often equated to an "unsafe" condition. As a consequence, transport authorities have to define priorities in the replacement of these "overdue" structures, or produce evidence that they may be kept in service. The tendency, for economic reasons, is to hang on to existing structures unless other conditions such as maintenance problems or operational requirements become predominant.

The third case encompasses a large variety of structures which have experienced major changes. These changes are not always obvious and they may be of quite different natures, such as :

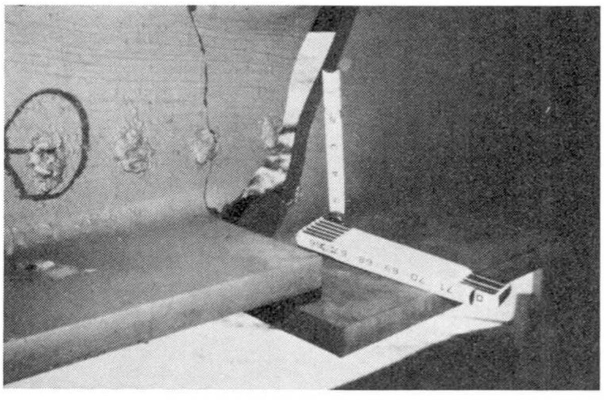
- physical modifications to the structure,
- improvement of knowledge,
- increase in traffic.

Physical modifications may include :

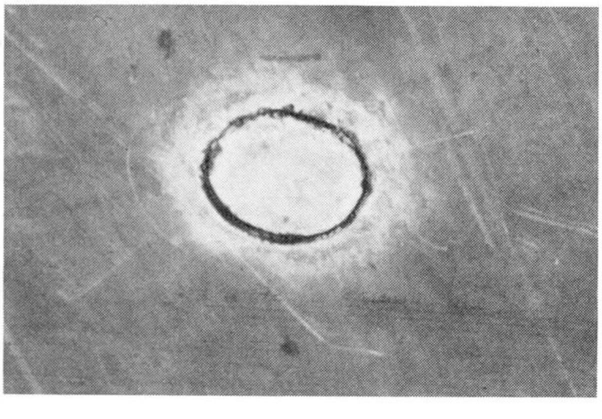
- changes due to fabrication and erection procedures which were not accounted for in the design. Welded lifting attachments left in place, bolt holes or flame cut notches filled with weld material, cut-out elements rewelded in place (FIGURE 1), etc. ;
- changes due to strengthening or widening of a structure in order, for example, to accomodate increased traffic volume or loads ;
- attachments added to hold utility lines (gas, water, sewer, etc.) ;
- replacement or repair of corroded elements or parts thereof by, for example, fillet-welding doubler-plates.

Improvement of knowledge recognizes the fact that new insight has been gained into the fatigue behavior of structures. For example, better information on the fatigue strength of typical welded details is now available todate than compared with 20 to 30 years ago. In countries where modern fatigue clauses have not yet or only recently been introduced, it is quite probable that severe details have been built into the structures. These details were not recognized as serious at the time of the design, in the same way that physical modifications are often not recognized to be detrimental. In addition, the widespread introduction of high strength steels and new welding procedures (electroslag welding) was sometimes undertaken without much knowledge of their behavior and performance. Design rules were mainly based on static strength.

The most widely recognized change, but not always the most important, is the



a) Fatigue crack emanating from bolt holes filled with weld material.



b) Flame-cut plate element before welding back into its original position.

FIGURE 1 : Examples of the possible effects of fabrication and erection procedures.

increase in traffic over the past twenty years. One of the most disturbing aspects of this observation is the great difficulty to model future traffic, both in load intensity and traffic volume. Connected to that is the question of how the change of legal load limits will affect the remaining fatigue life of structures.

Before discussing the different parameters and assumptions needed for an evaluation of the remaining fatigue life, it is necessary to point out that no unique or generally applicable method exists or has been agreed upon. The methods used should necessarily reflect the specific goals for the given structure. Considering also the time and money available, it would appear sensible to :

- proceed in steps, going from a rough approximation to more detailed and refined approaches (different levels) ;
- start from the safe side, that is overestimating stresses and underestimating strength ;
- rapidly conclude, whether a problem exists or not, and only if there is an indication of a possible problem proceed to a higher level of approximation.

In proceeding this way, one may also obtain an idea of the influence of improved assumptions on the resulting estimate of the remaining fatigue life. This appreciation of sensitivity may be useful in the judgement of the result and, hence, in the rating of the structure.

The evaluation and the ensuing rating, irrespective of motivation, should identify the most critical elements within a given structure, provide guidance for inspection intervals during the remaining fatigue life and allow priorities to be established for replacement or inspection in a given set of structures. Another important goal is to forsee an answer to the economic impact due to the possible increase of legal load limits.

3. BASIC PARAMETERS

The following section tries to identify possible steps, or levels of precision, in the definition of significant parameters. It is generally done by starting with simplified assumptions before going into detailed considerations, which require a large amount of calculations, or the evaluation of statistical data, or even tests.

3.1. Load History

Step 1 : Traffic model based on present traffic.

A fatigue load model may consist of a set of typical load cases described by the disposition of the loads and their intensities including the relative occurence of each load case. Such load models have been proposed by a few modern codes or specifications [1] [2] [3]. FIGURES 2 and 3 show the fatigue load models [4] [5] implicitly used in the Swiss steel specifications. Both models give the load intensities and geometries of trains or trucks and the relative occurrence of the different types. A comparison with the actual traffic and the use of these fatigue load models is further described in paragraph 3.8.

Step 2 : Evaluation of past traffic conditions.

Changes in past traffic may have occurred at different periods of time during the life of the structure, either gradually or rather quickly. One might consider for example :

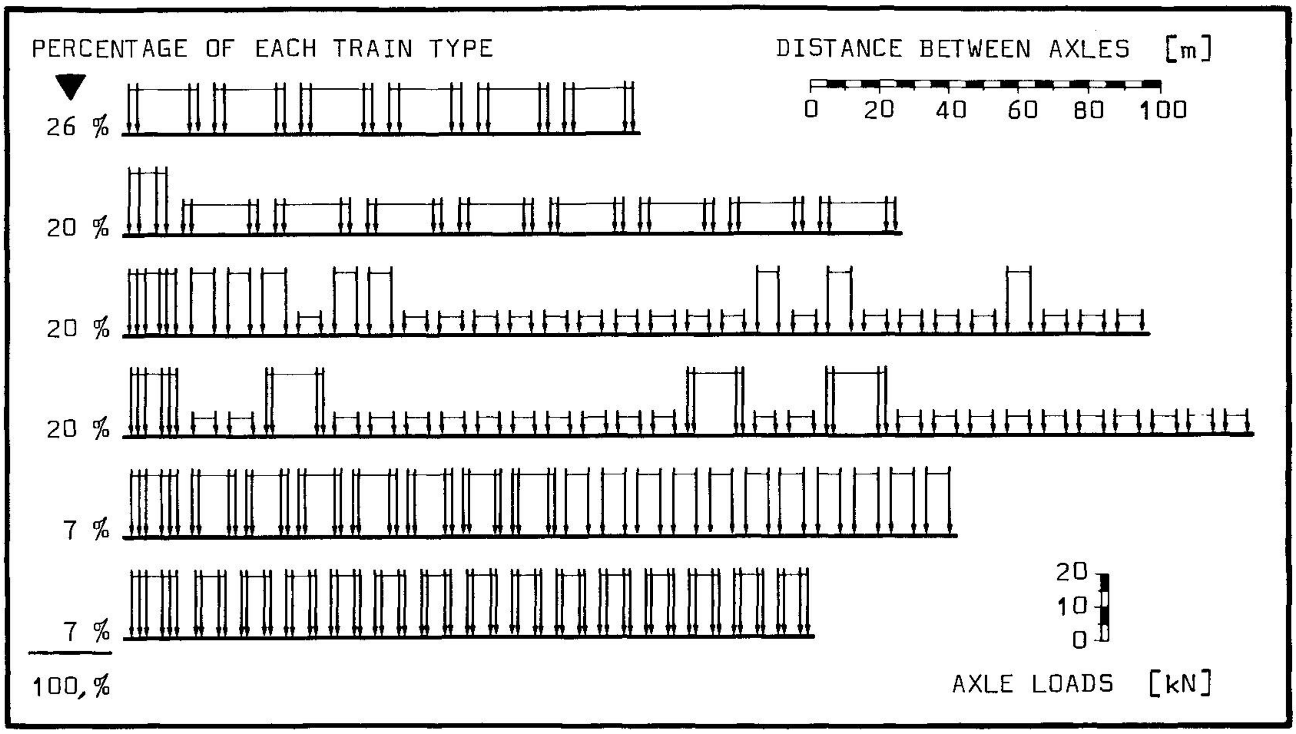


FIGURE 2 : Fatigue load model (Swiss Federal Railways) representing actual traffic on railway bridges.

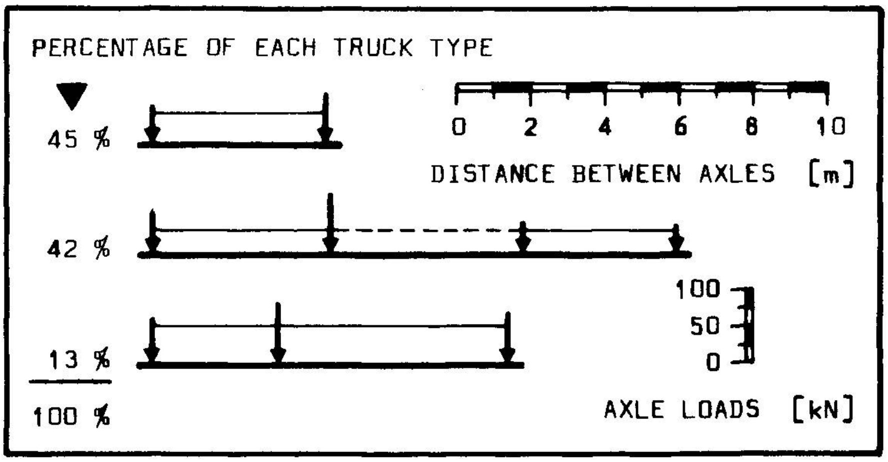


FIGURE 3 : Fatigue load model representing actual traffic on Swiss highway bridges.

- technical development of the vehicles, such as at the time of electrification of railway lines, or use of diesel locomotives ;
- change in traffic pattern when new lines or highways are opened ;
- effect of war-time loads ;
- change of design codes or maximum legal limits on loads, which incidentally might have resulted in strengthening the structures.

3.2. Future Loadings

Step 1 : Present traffic situation.

Using the present traffic situation to describe future loading, therefore neglecting possible increases, does obviously not represent a conservative assumption, but it has the merit of being simple. It might even be accurate enough when the calculated remaining fatigue life is short. If this should not be the case, one might then still proceed to an educated guess of the future traffic situation.

Step 2 : Estimated future loads.

Traffic development, particularly the increase of axle loads or total truck or car weight, is likely to be influenced by political decisions and economic factors. One example is the effect of Common Market agreements in Europe which tend to adjust legal load limits in the various countries. It is apparent from FIGURE 4 that the legal load limit influences directly the position of the peak in the probability density functions of heavy truck traffic [6]. Another pressure calling for heavier truck weights comes from the ecology movements and fuel efficiency in order to reduce the number of trucks needed to transport the same total tonnage.

Step 3 : Extreme load situation limited by maximum capacity or operational limits.

It should be noted that steps 2 and 3 are open guesses, particularly if the traffic evolution over a period of 20 years or more must be estimated.

As a conclusion, it is preferable to operate on a resonable level of knowledge, for example the present traffic condition including scheduled increases. This type of evaluation might appear not to be on the safe side. However, safeguards can be provided by specifying the traffic conditions for which, when reached, a renewed evaluation becomes obligatory.

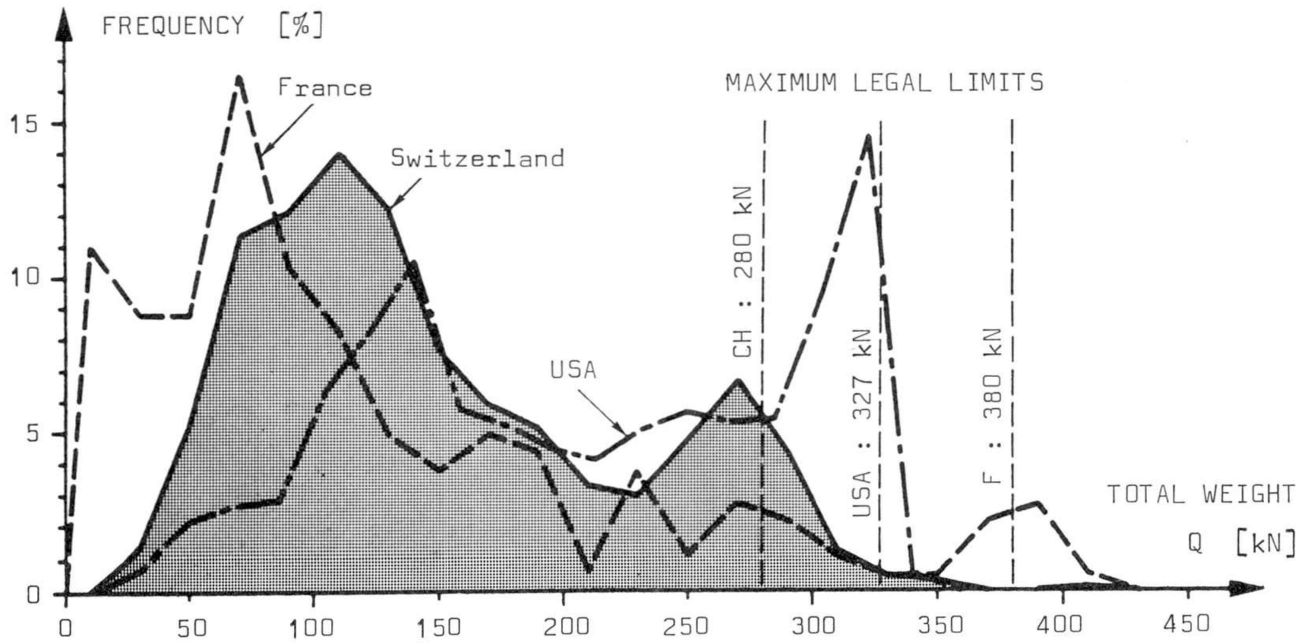


FIGURE 4 : Comparison of truck weight histogrammes from three contries.

3.3. Static Stresses and Strains

Step 1 : Simple stress analysis according to common engineering practice.

Step 2 : Detailed analysis based on computer methods.

Step 3 : In-situ measurements under well defined loads.

Generally, a difference is observed between measured and calculated stresses. This difference may vary from one element to another, and from one point to another depending on the type of structural system. Also, the difference becomes smaller with improved approximation of the structural system.

The measured stresses at midspan are generally smaller than computed although this does not hold for the support region. Typically, a floor beam calculated as a simple beam might behave more like a fixed-ended beam under service conditions due to the structural detailing of the support region, even though at ultimate load the statical system is more like a simple beam.

Hence, support regions are likely to be more highly stressed that calculated. In addition, they might impose stresses and strains to the supporting elements, which might lead to strain induced cracking [7] [8].

Finally, one must check whether changes have occurred in the static behavior of the structure or its elements. This may be due to strengthening of the structure, changes in the superstructure, support settlements, "frozen" bearings due to corrosion or dirt, etc.

3.4. Impact Factor for Dynamic Behavior

Step 1 : Impact factor according to design codes.

Step 2 : Information based on experimental evidence from similar structures.

Step 3 : In-situ measurements of the live load stresses.

Design impact factors, as a rule, are on the high side ; this is particularly true for short span elements. Some codes, like UIC [9], give additional values for the assessment of bridges introducing for example parameters such as : length of the influence line, natural frequency, track condition. It is obvious that a ballasted railway bridge deck will have less dynamic impact than when the rails are directly connected to the structural system composed of floor beams and stringers.

Similarly, the road surface condition affects dramatically the impact values for highway bridges, particularly when pot holes are present. Jumps at expansion joints or those due to the settlement of the approach slab may also impose additional dynamic effects. As a consequence, and due to the relatively great influence of impact factors on the result of the evaluation, measurements may often be very useful.

3.5. Constant Amplitude Fatigue Strength

Step 1 : Assumptions based on design curves.

The most important aspect is the correct identification of the severe details and their relationship to a given classification system. FIGURE 5 shows S-N curves proposed by ECCS [10] with a double-logarithmic scale. Having parallel lines

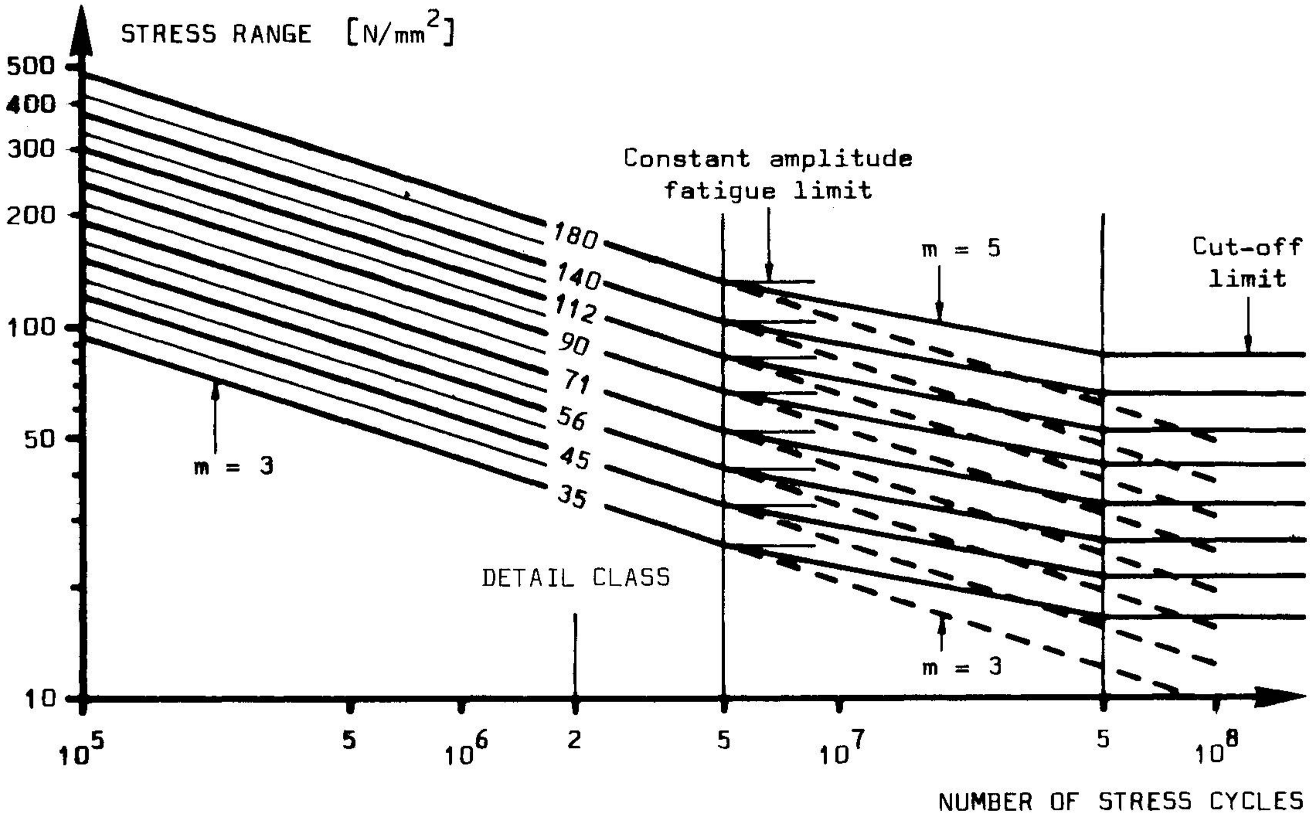


FIGURE 5 : **ECCS proposal for "European Fatigue Strength Curves" representing mean minus two standard deviations.**

greatly simplifies the designers work and the definition of equidistant curves sets the level of accuracy needed. This is particularly important in avoiding over-precision of parameters of lesser importance.

It should be noted that stress range is the governing factor for the fatigue life of a given structural detail. Minimum stress, stress ratio, and even grade of steel do not significantly affect the fatigue strength [11].

It is possible that particular details of a given structure will not be identified in a classification system. It should be possible, based for example on fracture mechanics considerations, to conservatively introduce such details in the system. Important parameters for the evaluation of details are stress concentration due to the general stress field and defect size.

Special attention has to be paid to built-in defects. Such defects are commonly created by incomplete penetration welds at the crossing of different (secondary) elements. This type of defect, where the lack of penetration is perpendicular to the stress field, is not contained in the usual classifications systems.

Step 2 : Use of published data.

When using test data, for example for old riveted structures, one should recognize that test data obtained with small test specimens will overestimate the fatigue resistance. The effect of grade of steel is often presented on the basis of machined base material specimens which obviously do not reflect the stresses and notch conditions of real connections.

Another parameter, historically important since most early design codes used it, is the mean stress or stress ratio. In practice, this effect is generally no longer considered since :

- the mean stress in the structural element is not known due to the influence of thermal stresses, stresses due to misfits, or effect of support movements ;
- the fatigue strength curves for the various mean stresses are not available ;
- large welded elements simply do not show the effect of mean stress ;
- cumulative damage rules considering mean stress are not well established.

Step 3 : Tests on structural elements.

In case of the evaluation of a large set of structures, for example riveted bridges built before the turn of the century using wrought iron, this approach might be justified. It is sometimes possible to remove typical details from an existing structure or use material from a similar structure being dismantled. When interpreting the test data, it is important that fractographic examinations are made in order to check whether small fatigue cracks had already existed in the test specimens at the onset of the tests.

3.6. Counting Method

Step 1 : Major stress cycles only.

The fact that stress range is the predominant parameter for fatigue strength implies that stress ranges have to be identified in a given stress-time diagram, be it calculated or measured. In the first step, the major stress ranges are counted using for example peak counting (might be overconservative) or peak-to-peak counting. All stress ranges smaller than about 30 % of the major stress ranges can be neglected, provided their number (frequency of occurrence) is of the same order of magnitude. This can be verified using the equivalent stress range concept given by Equation 2 in paragraph 3.7.

Step 2 : Rainflow counting (Reservoir method).

Rainflow and range-pair count theoretically give the same results provided that the level of the neglected stress cycles for the range-pair count is kept very small. This indicates one advantage of rainflow, where the decision on the suppression of small cycles is not needed before the counting. On the other hand, the computer programming of rainflow is not very convenient. Also, one has to be aware that many rainflow programs are not prepared to handle stress excursions with changing sign, as for example for the influence line of a continuous beam. For manual counting the representation using the reservoir model is more attractive.

Note : In order to have a common basis for comparison and discussion, ISO, Eurocode, ECCS and IIW propose to use rainflow counting [12] [13] [14] [15].

3.7. Cumulative Damage Calculations

It is proposed and assumed that cumulative damage will be calculated according to Palmgren-Miner's rule equating the total sum of damage to unity. If this is not done it should be clearly stated. It is also important to indicate which fatigue strength curve (mean or %-fractile) has been used for the cumulative damage calculation.

If all stress cycles fall below the fatigue limit, it is assumed that no fatigue damage occurs or has occurred [14]. When the stress spectrum is such that a part of it lies above the fatigue limit, three steps of approximation are possible :

Step 1 : The fatigue limit is disregarded.

All stress ranges are considered to be fatigue damaging. Based on the equation of the fatigue strength curves,

$$N = C\,\Delta\sigma^{-m}\,, \tag{1}$$

and using Palmgren-Miner's rule, it is possible to express the stress spectrum by an equivalent stress range $\Delta\sigma_e$ [16], which would yield the same number of constant amplitude cycles $N = \Sigma\, n_i$ as contained in the stress spectrum :

$$\Delta\sigma_e = \left[\frac{\Sigma\, n_i\, \Delta\sigma_i^m}{\Sigma\, n_i}\right]^{1/m} . \tag{2}$$

This has been experimentally verified on a large series of test beams submitted to programmed loading [17]. With these programmed loadings, the validity of the equivalent stress range, or in other words, the cumulative damage rule is checked.

True stress-time histories do not correspond to block loadings or to random loadings. In addition, a counting method has first to be used in order to identify each stress range cycle. Pilot studies on test beams subjected to recorded stress time histories from either highway traffic or railway traffic have shown that the equivalent stress range may be used in conjunction with rainflow counting (FIGURE 6). A parametric study is under way to evaluate the effect of other counting methods and mean stress.

FIGURE 6 shows that, for purposes of design or evaluation of the remaining fatigue life, the equivalent stress range concept is quite adequate. It is noted (FIGURE 6 a) that the scatter of the stress history data is about the same as

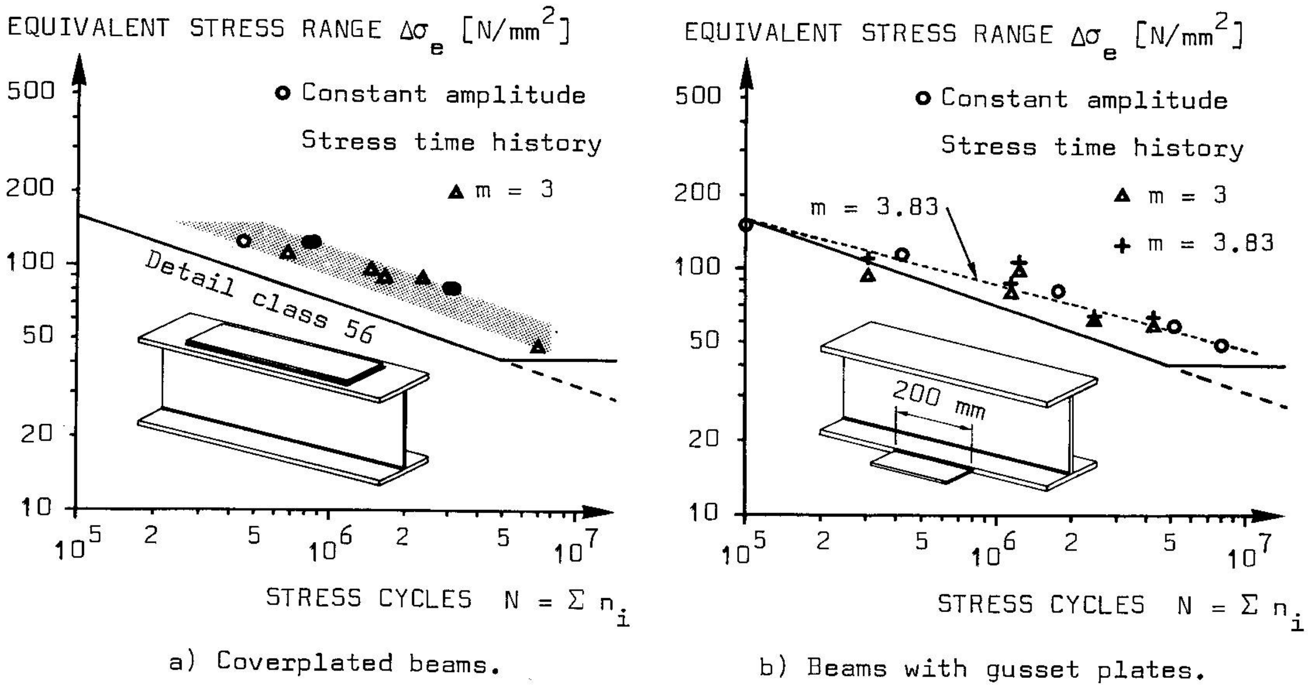


FIGURE 6 : Data from fatigue tests with stress-time histories analyzed by rainflow counting and equivalent stress range (detail class 56 denotes the fatigue strength at $2 \cdot 10^6$ cycles and is identical to AASTHO category E).

that for the constant amplitude data. At any rate, it should not be expected that the variable amplitude data would show smaller scatter than the basic data.

FIGURE 6 b shows that the exponent m (slope of the S-N curve) has little effect on the fit of the data. Even though the observed slope of m = 3.83 gives a better fit for this particular detail and small sample size, the use of the common slope of m = 3 is still satisfactory. Generally, larger test data samples tend toward an exponent of 3 for the lower bound.

Step 2 : Fatigue strength curves with a knee point.

It can be concluded from step 1 that the equivalent stress range concept may be used for a rapid evaluation. However, when a large portion of the stress spectrum falls below the constant amplitude limit, this procedure may give over-conservative estimates. Different ways to account for stress ranges smaller than the fatigue limit have been proposed. A simple approach consists of introducing a bi-linear S-N line with a knee (for example at 5 million cycles) below which a smaller slope (for example (2 m - 1) or (m + 2), as shown in FIGURE 5) is introduced [11] [18]. Tests are under way in various laboratories to further investigate this proposal, which is at present quite commonly accepted.

Step 3 : Fracture mechanics analysis.

Simplified or sophisticated fracture mechanics procedures are generally not recommended for the evaluation of the remaining fatigue life of structures, unless a crack has been observed. In such a case, the location and possibly the dimensions and shape of the crack are known as well as the stress field surrounding it. A detailed analysis then becomes possible or even necessary in order to define retrofit or repair needs.

3.8. Composition of Traffic

Over the past twenty years, extensive research has been carried out on the fatigue strength of details. The description of loads has long been a neglected part of the problem, and it seemed impossible to compare loads in or between different countries. However, it has been shown recently that a comparison is possible provided it is not made on loads (intensity, geometry, etc.) but on their cumulative damage [6] [19] [20].

In order to make a comparison, one needs a well defined reference load which might be the same as the standard live load used for static design. An example for railways is shown in FIGURE 7. The extreme maximum and minimum stresses due to this load are calculated and used to obtain a reference (design) stress range $\Delta\sigma_d$. Before showing examples, it is necessary to recall some important characteristics of the S-N diagram (FIGURE 8) using the equivalent stress range concept.

Assuming that the equivalent stress range $\Delta\sigma_e$ and the corresponding number of stress range cycles $N = \sum n_i$ is known, a damage line can be introduced in

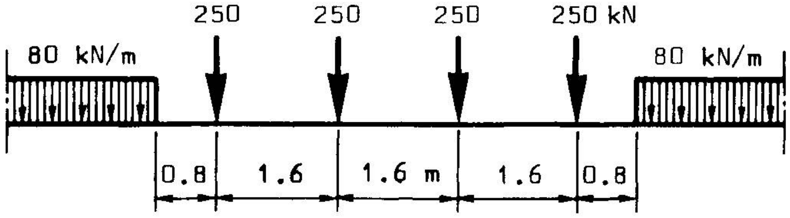


FIGURE 7 :

UIC standard design live load for rail bridges (UIC : Union Internationale des Chemins de fer).

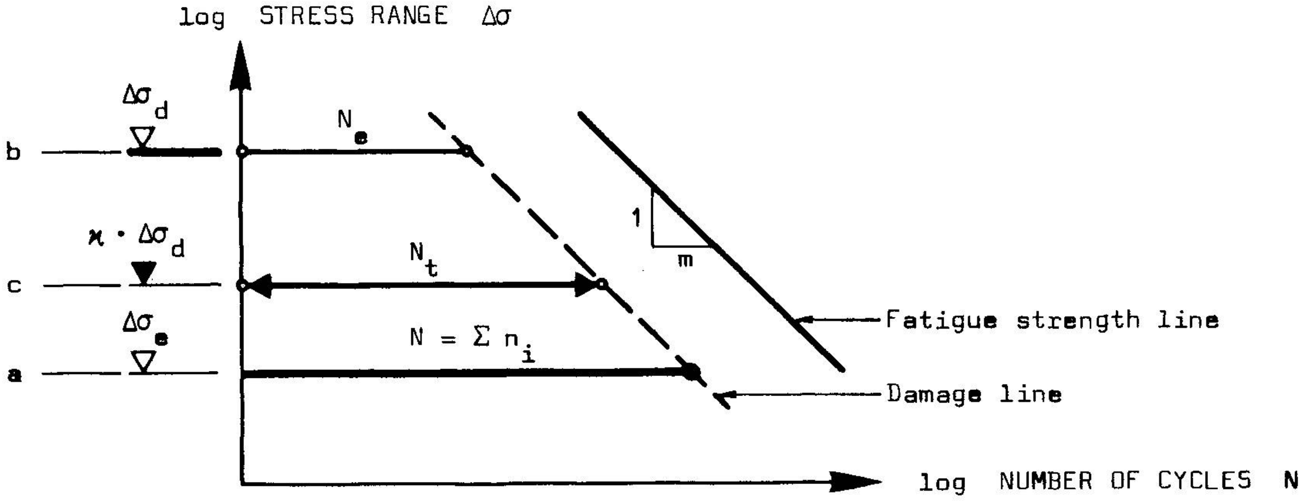


FIGURE 8 : Definition of three levels of stress range with the corresponding number of cycles yielding the same cumulative damage.

FIGURE 8 parallel to the fatigue strength line. Two other points may now be defined on this same damage line. One is fixed at the level of the reference (design) stress range $\Delta\sigma_d$ and the other by a preselected number of stress cycles. Therefore, three distinct levels are defined by the damage line among which certain correlations may be retained :

- Level a is based on the stress spectrum and expressed by its equivalent stress range $\Delta\sigma_e$ (Eq. 2) and the corresponding number of stress cycles $N = \Sigma n_i$.
- Level b is defined by the stress range $\Delta\sigma_d$ due to a reference load and related to level a by an equivalent number of stress cycles N_e, where

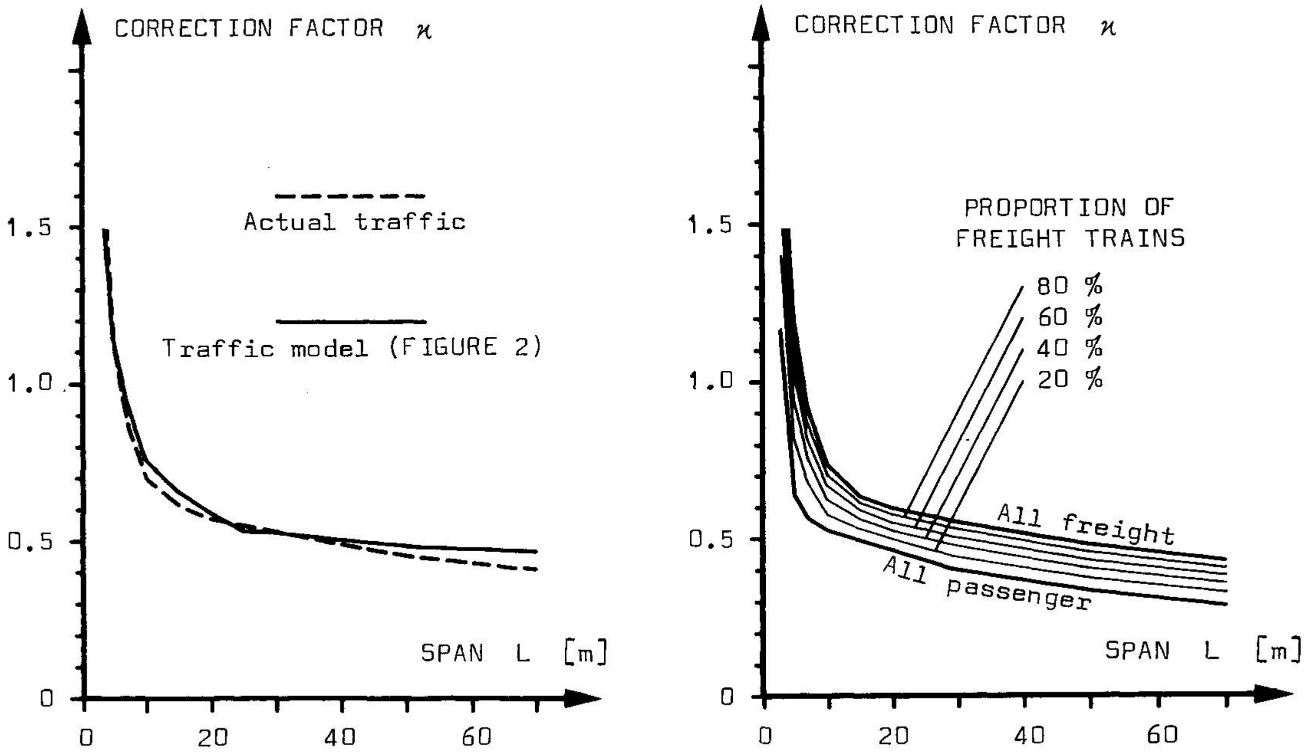
$$N_e = \left(\frac{\Delta\sigma_e}{\Delta\sigma_d}\right)^m N . \tag{3}$$

- Level c is given by an arbitrarily fixed number of cycles N_t. Say for example the number of trains which would lead to the same value for all bridge elements, as opposed to the number of stress cycles which vary with influence length. The corresponding level of $\kappa \cdot \Delta\sigma_d$ is expressed in terms of a correction factor κ, multiplied by the design stress range $\Delta\sigma_d$ for ease of comparison, where

$$\kappa = \left(\frac{\Delta\sigma_e}{\Delta\sigma_d}\right) \left(\frac{N}{N_t}\right)^{1/m} . \tag{4}$$

When these relationships are applied to railway bridges [16] [20], it becomes possible to verify whether a load model (FIGURE 2) represents the fatigue effects of real traffic (FIGURE 9 a). Since a common European load model (FIGURE 7) is used, all countries can thus compare their load models or the effect of their actual traffic in one and the same way.

The effect of traffic composition, showing for example different proportion of freight trains, may be studied in the same way. FIGURE 9 b has been established on the basis of about 150 measured trains [21]. Finally, it should be noted that this correction factor has been introduced in the Swiss Steel Specification [1] and UIC Recommendations [3] where it is called α and λ_T, respectively.



a) Comparison of the computed fatigue effects of a traffic model and actual traffic.

b) Effect of the proportion of freight trains in the traffic.

FIGURE 9 : Possible applications of the correction factor $\varkappa$ for railway bridges.

Another example relates to highway bridges [6]. Considering the great difference in the histogrammes of the truck weights for different countries (FIGURE 4), it seems impossible to find a common denominator. Even more so when the differences in type of trucks and their geometry is observed. However, it has been found [5] that the cumulative fatigue damage of a given truck traffic, including the variation in weight and geometry of all trucks, can be expressed by a correction factor.

The procedure is as follows. First, the equivalent stress range $\Delta\sigma_e$ is computed using the stress ranges of all individual trucks. Then, a stress range due to a single concentrated load, represented by the equivalent weight Q_e

$$Q_e = \left[\frac{\Sigma\, n_i\, Q_i^m}{\Sigma\, n_i}\right]^{1/m} . \qquad (5)$$

is also computed. The correction factor is the ratio between the equivalent stress range $\Delta\sigma_e$ and the stress range due to Q_e. This ratio is shown in FIGURE 10 [22] in terms of span length and for three different countries.

It is very suprising to note the small difference in this correction factor even between countries having a different traffic pattern. The fact that the "average" truck might be heavier in one country than in another is reflected by the numerical value of the equivalent weight. Based on these observations, it now appears possible to define a harmonized traffic model for fatigue design using the correction factor applied to the equivalent weight.

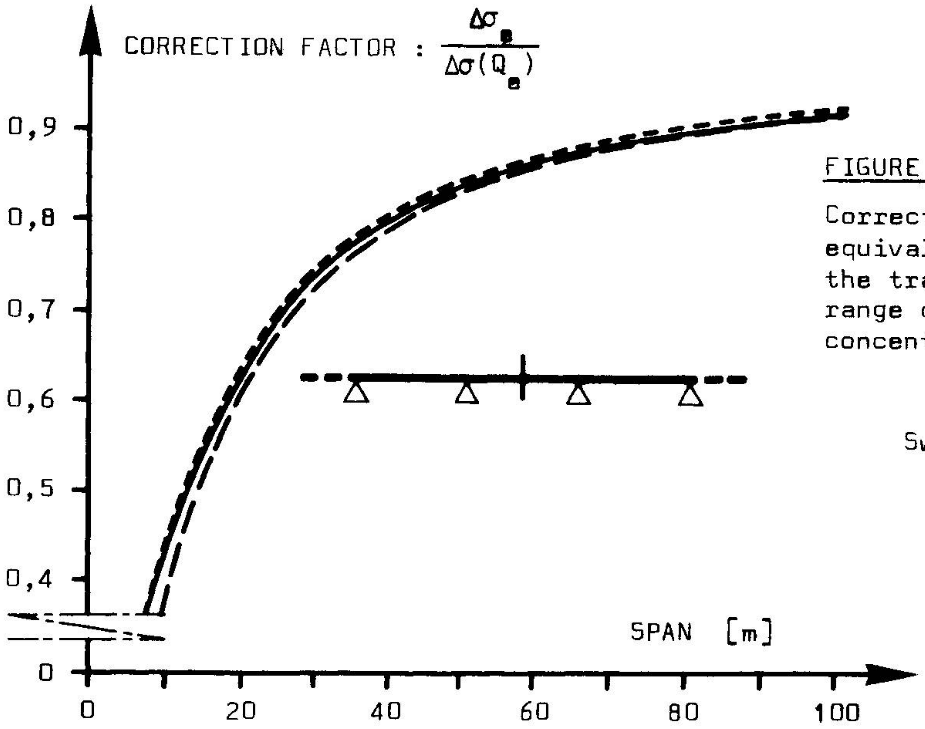


FIGURE 10 :

Correction factor relating the equivalent stress range from the traffic $\Delta\sigma_e$ to the stress range due to the equivalent concentrated weight Q_e.

Switzerland : Q_e = 182 kN

France : Q_e : 230 kN

Germany : Q_e = 232 kN

4. METHODS OF EVALUATION

In addition to the basic parameters discussed in the previous section, safety considerations or margins of safety must be introduced for the evaluation of the remaining fatigue life. Some basic concepts may be distinguished [23] [24] [25].

Level 1 : Deterministic approach.

In this approach fixed values are assigned to all parameters, for example mean or fractiles. When the fatigue strength curves are extended below the fatigue limit an analytical solution using the equivalent stress range concept is possible. In any case one should always proceed in steps in order to identify the effect of the individual assumptions on the resulting fatigue life estimation. A safety factor can be introduced on this life estimation depending on the degree of precision of the individual parameters introduced.

Level 2 : Pseudo-probabilistic approach.

This has recently been introduced by the Swiss steel specification and the UIC recommendation where the scatter of the fatigue strength is represented by a log-normal probability density function. Also, the equivalent stress range has a log-normal distribution assigned to it. The effects of load and strength are thus "separated" by means of counting method and cumulative damage rule.

Usual safety considerations, as developed for ultimate strength design, can be used by introducing a safety index β. Incidentally, this is only possible if the log-normal distribution of the fatigue strength, which was originally obtained on the horizontal (number of cycles) axis, is transformed into the vertical (stress range) axis. However, the result may still be expressed in terms of life. The major problem remains the calibration with experience in order to define the numerical value of β.

Level 3 : Probabilistic approach.

All parameters must be introduced with their statistical distribution. A major problem in the analytical methods resides in the fact that strength is not independant of stress spectra. Cumulative damage rules have to be expressed in a different way and the result will be in terms of probability of survival.

Research to establish these analytical methods is under way [26] and numerical procedures using for example Monte Carlo simulations also seem possible. In order to reduce the number of parameters that have to be introduced in such computations, the results shown in paragraph 3.8 might be of interest.

5. CONCLUSIONS

This paper summarized the reasons and circumstances which might lead to an evaluation of the remaining fatigue life. The basic parameters needed for such an evaluation have been enumerated and discussed. The main purpose of the evaluation procedure is the rating of the structure. Unfortunately, no clear or agreed upon procedures exist and more work is urgently needed to establish such methods. Nevertheless, a certain number of ideas might be retained.

1.- When a crack is found in a structure, this generally indicates that many more cracks are present. Repair and retrofit procedures must be established using for example fracture mechanics analysis. It has to be stressed that the remaining fatigue life is generally very short once the cracks are easily visible, and thus found.

   Repairs are often very costly, hence, small span structures might most economically be replaced by adequately designed structures. Long span structures very rarely suffer fatigue damage in the principal structural elements, unless cracks in the secondary elements have grown into them. Whenever retrofit of a superstructure is needed one should also try to reduce impact factors by, for example, changing the load path of directly introduced loads.

2.- The rating of a structure obviously needs a clear evaluation procedure and the necessary information on the basic parameters. If the calculated remaining life is negative, then two possibilities exist : one, the assumptions are too conservative (impact factor, stresses in a highly redundant structure, loads) or two, the problem is real, in which case fatigue damage is very probable.

   If the calculated remaining life is positive, an appropriate safety factor is needed (level 1 or 2) on the remaining life (not stress) of each element whilst taking into account its importance for the entire structure. In other words, the redundancy of the structure becomes a significant factor to judge the importance of possible cracking ; for example, a multibeam bridge will be not as critical as a two girder bridge. Based on this, inspection procedures and intervals have to be defined.

3.- The rating of a given set of structures places less importance on the choice of the safety margin since the primary goal of the evaluation is to establish priorities for inspection of replacement.

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