Overload Permit Checking Based on Structural Reliability

GONGKANG FU[1] AND FRED MOSES[2]

Overload permit issuance for vehicles exceeding the legal weights is a common practice in many states. Most states face increasing pressure to allow heavier and greater number of overweight truck permits. The current method of permit checking in most states adopts the concept of allowable stress with operating strengths. It does not provide uniform level of structural reliability. A rational permit checking method is therefore desired. Little research has been reported which derives a practical and rational format for permit checking. This paper proposes a format to check permit overloads based on structural reliability. Weigh-In-Motion data is used to model live load. Frequent, one-trip, and escorted permits are considered. This checking format is compatible to both load factor checking procedure of current AASHTO specifications and load and resistance factor methods under development. It can be included in a bridge structure evaluation.

1. INTRODUCTION

A large number of bridges in this country are considered structurally deficient or functionally obsolete. On the other hand, truck traffic has been growing in both volume and weight. Commercial vehicles with weights above the legal limit have frequently been observed on the public highway system. Furthermore, overweight permit issuance for vehicles exceeding the legal weights has been a common practice in many states. Increasing overweight loadings on the current U.S. highway system have been reported by FHWA to the congress [1], which provides the latest overview of the situation. The impact on bridge safety of this trend deserves careful investigation.

The passage of an overweight vehicle on bridges becomes a legitimate activity by obtaining an overload permit. Such permits are now issued by most highway agencies of the nation. In fact, it has been noticed that most states are faced with increasing pressure to allow heavier and greater number of overweight trucks to travel in the present highway network. This is because of the obvious economic advantages of heavy freight transportation and recognized conservativeness in bridge structure design. However, how much reserve strength can be used to meet the growing requests remains a critical issue. Even if vehicles utilize a greater length and number of axles which helps mitigate pavement damage, there will be questions regarding the safety of bridge structures which must carry the total loads.

The current method in most states to check the permit issuance for bridge safety uses the concept of operating strengths. This is acceptable because the current AASHTO bridge code [2] adopts this concept. Nevertheless, this method is not rational regarding bridge structural reliability for the specific issue of permit overloading. First of all, the allowable stress checking methods associated with most bridge evaluation using the operating strength do not provide uniform safety level. Secondly, bridge structural safety is sensitive to the maximum load effect by extremely heavy loadings largely affected by permit traffic. This is not covered by the current code supposedly addressing only random, mostly legal, traffic flow. A rational permit checking procedure is therefore desired, which should be based on a more realistic modeling of the permit live loads, and should provide a more uniform level of structural reliability.

Cassano and LeBeau [3] addressed the issue of permit overload in the context of design criteria. They suggested that the permit overload be included in design as an additional load case. This suggestion was based on a deterministic example of design calculation. Bakht and Jaeger [4] suggested a procedure of calculating safe permit loads, which are based on the worst combination of maximum vehicle weights that a bridge is likely to have sustained during its life time. This procedure is not generally applicable, especially in the US, as the information on the worst loading ever carried by a bridge is not usually available to evaluation engineers. Ghosn and Moses [5] were the first researchers dealing with permit overload based on the criterion of structural reliability to quantitatively cover the uncertainties. They assessed structural reliability for permit checking with the operating stress. Permit checking equations were suggested for two cases of

1 New York State Dept. of Transportation
2 Case Western Reserve University

permit vehicle presence, namely, a permit vehicle alongside another permit vehicle and a permit vehicle alongside with a normal truck. However it was unknown to evaluation engineers just how possible those extreme cases would be.

Little research has been reported which derives a practical and rational format to check the permit loads in terms of structural reliability. This paper presents a study in this direction supported by the Ohio Department of Transportation and the New York State Department of Transportation in cooperation with the Federal Highway Administration. A format to check permit overloads based on structural reliability is proposed herein for possible inclusion in a bridge evaluation code. This format is compatible to both load factor checking procedures of current AASHTO specifications [6] and load and resistance factor methods under development [7]. Different types of permits such as frequent, one-trip and escorted permits are considered. The load factors illustrated herein are for the case of steel girder bridges regarding bending moment strength checking. Nevertheless, the concept of the derivation procedure can be applied to other cases such as shear strength, concrete or prestressed concrete bridges, etc.

2. TYPES OF OVERWEIGHT PERMITS

In this study, overweight permits are considered in three categories of travel, namely I) frequent or routine trips, II) one-trip or special trip, and III) escorted or controlled trip. Accordingly, the analysis methods to select load factors vary from one category to another in order to include various levels of uncertainties especially involved with live or vehicular load.

Frequent or routine permits are those valid for unlimited passages without escort on specific routes in certain periods, varying from a few weeks to as long as 3 years. It is noted that this type of permit is the most beneficial one, in terms of redeeming the costs by reduced administration costs and added productivity for the carriers receiving the permits [1]. The presence of frequent permit vehicles on the highway system changes the live load spectrum of highway bridges from the normal traffic including illegal overloads. In turn, the probability distribution of maximum load effects experienced by these bridges is affected by these permits.

One-trip or special permits are valid only for single trip on a specific route without escort. They are usually issued for those vehicles which fit between frequent and escorted permit vehicles in terms of weight and/or size. The bridge live load in this case is analyzed herein by considering the permit vehicle plus whatever random vehicle may be alongside as needed. The maximum live load effect prediction concentrates on the possible alongside vehicle, since the permit vehicle is known with respect to its weight and configuration.

Escorted or controlled permits are issued to those overweight vehicles requiring escort. They are for passages of excessively heavy loads such as large power generators, machinery, aerospace equipment, etc. The information on those extremely heavy vehicles is usually available in terms of their axle weights and spacings. Additionally, no other vehicles will be allowed on the bridge alongside those escorted vehicles. Dynamic load impact can also be controlled by low speed if necessary. This presents the simplest case of live load modeling for permit checking.

3. PERMIT CHECKING EQUATION AND STRUCTURAL RELIABILITY

Current practice of structural strength checking in US is moving towards the use of load and resistance factors. As a matter of fact, current AASHTO bridge code [2] allows a load factor checking format evaluation. Furthermore, the format of load and resistance factors allows flexibility for adjusting those factors in order to reach a relatively uniform level of structural reliability, with respect to variation of loading distributions such as various ratios of dead to live load effects. A uniform checking format of load and resistance factors is therefore suggested for the three types of permits considered herein:

$$\phi R_n > \gamma_D D_n + \gamma_L L_p \qquad (1)$$

where ϕ, γ_D and γ_L are respectively factors of resistance or capacity reduction, dead (gravity) load and live (vehicle) load, and R_n, D_n and L_p are respectively nominal values of component capacity (resistance), dead load effect and permit load effect including dynamic impact. This checking procedure is similar to load factor design (or rating) in AASHTO specifications [2,6]. Determination of the factors for checking equation (1) is a major focus herein. Structural reliability is used as the criterion to make such decisions.

Structural engineering calculations in both design and evaluation are supposed to cover uncertainties involved. This well known fact is implicitly addressed in current and previous codes by safety factors in checking equations. Until recently those safety factors are determined by evaluation primarily based on engineering experiences. It should be noted that, therefore, these factors may not be rationally related to the uncertainties intended to be covered. Structural reliability theory quantifies these uncertainties in the context of relative probability. It therefore provides a realistic criterion for decision making in structural engineering problems. This approach is employed here.

Corresponding to the checking format using nominal values in Eq.(1) a safety margin z of a structural component being checked for permit issuance is

$$z = R - D - L \tag{2}$$

where R, D and L are respectively true values of component strength (resistance), dead load effect and live load effect in the component. A negative value of the safety margin indicates failure of the structural component. R, D and L are considered as random variables with uncertainties due to uncontrolled variations in design, construction, service condition, etc. They are assumed of lognormal distribution, and independent of each other.

The safety at component level is expressed by the so-called reliability index beta ß [8]

$$\beta = m_z / \sigma_z \tag{3}$$

where m_z and σ_z are mean value and standard deviation of the safety margin z, respectively. Beta covers the uncertainties by including the scatter described by the standard deviation. An advanced algorithm [8] is used to compute the reliability index. This index is utilized as the indicator of structural safety level, and is similar to the safety indices being developed for the new AASHTO LRFD code.

It is noted that engineers use Eq.(1) for component strength checking by using nominal, instead of true, values of resistance (R), dead load effect (D) and live load effect (L). Resistance and load factors ϕ, γ_D and γ_L are applied only to those nominal values as expressed in Eq.(1). Given statistical information on load effects D and L, the mean value of random variable R of Eq.(2) varies according to the load and resistance factors used in Eq.(1), and so does reliability index ß in turn. This mechanism allows adjustment of these factors in order to reach a target reliability index ß. Determination of the relative magnitudes between dead load factor and live load factor can be made to produce relatively uniform beta over the distributions of dead and live load, which are usually span dependent.

4. MODELING OF LIVE LOAD EFFECT

Bridges are designed to safely withstand the maximum load expected over the service lifetime of the structure. It is therefore critical to have a rational prediction of the maximum live load effect L of Eq.(2) The model of maximum bending moment introduced by Moses and Ghosn [9] is employed here:

$$L = M\ g\ i \tag{4}$$

where M is total static live load moment, or shear, due to vehicle combination, g is load effect distribution factor for individual girder and i is impact factor taking dynamic response into account. They are assumed of lognormal distribution. L_p of Eq.(1) is the nominal value of L, and is accordingly computed as follows

$$L_p = M_p\ g_n\ i_n \tag{5}$$

where M_p is static moment due to the permit vehicle, g_n and i_n are code specified nominal values of girder distribution factor and dynamic impact factor, respectively.

According to Moses and Verma [10], the bias (ratio of mean value to nominal value) and coefficient of variation (COV) of lateral distribution factor g are set equal to 1.0 and 10%, respectively. The mean and COV of i are taken as 1.2 and 10%, respectively. The nominal values g_n and i_n are computed according to the simplified lateral distribution and impact formula in the AASHTO specifications [6].

Maximum bending moment M was predicted in this research by taking various factors into account, such as truck gross weight distribution (histogram), multiple presence of trucks on the bridge, traffic volume, site variation of traffic conditions, etc. The prediction is done for the three cases of permit loadings, and for various span lengths from 30 ft to 210 ft of simple and continuous span bridges. A time period of two years is covered in the prediction of maximum live load effect M for reliability assessment. This selection is consistent with the current inspection interval for bridge structures in the US.

4.1 Maximum Load Effect Prediction for Frequent Permits

Frequent permits are issued for a limited time period and a specific route. The presence of these overload vehicles has evident impact on the bridge live load spectrum. This influence is significant at the higher weight end of gross weight distribution, which in turn makes it sensitive to the structural reliability affected by maximum load effect. A simulation program is developed to find the mean and COV of maximum live load effect M for this case. This algorithm is based on the concept of convolution introduced by Moses and Ghosn [9]:

$$P[m]=\Sigma\Sigma\Sigma P[weights]P[locations]P[types] \quad (6)$$

where m is a realization of maximum moment M; P[m] is the probability of such realization, in the event of vehicle presence on a bridge; P[weights], P[types] and P[locations] are respectively the probabilities of vehicle occurrence characterized by their weights, locations on the bridge, and types with respect to vehicle configuration. These probabilities were obtained by Weigh-In-Motion data [9]. The triple summation in Eq.(6) is taken over all the combinations of weight, type and location that induce maximum moment of magnitude m. The probabilistic distribution of maximum moment due to one event of truck presence on a bridge is readily obtained by varying m in the convolution of Eq.(6). This distribution is then projected to cover the period of 2 years given traffic volume. The mean and COV of the maximum moment M are calculated based on this projected distribution.

Typical vehicle gross weight histograms based on Weigh-In-Motion data used in convolution Eq.(6) are displayed in Figs. 1a and 1b for sites of unenforced and enforced weight control, respectively. It is noted that the added permit vehicle weight histogram tail is according to statistics of Ohio Department of Transportation. For the simulation, possible locations of truck presence are depicted in Fig.2. Presence probabilities of 1, 2, 3, or 4 vehicles are based on headway statistics of several Weigh-In-Motion sites [9].

Load effects above the permit vehicle response is due to vehicles which may closely follow the permit vehicle in the same lane or more importantly be located in the adjacent lanes. The latter may especially be significant. It has been found that the effect due to permit issuance is much more significant on the sites with adequate weight control enforcement, which has few overweight vehicles in the normal traffic. The mean value and COV of maximum moment M, including site variation of live load, is contained in Table 1 for simple span bridges of various lengths. They are listed according to traffic condition. Light and heavy traffic volumes refer to average daily truck traffic (ADTT) lower and higher than 1000 trucks/day, respectively. Table 1 shows that bridges at weight control unenforced sites are expected to experience higher maximum moments than at enforced sites, as the former observe more overweight trucks. It also shows in Table 1 that heavy traffic volume increases the mean, and decreases the COV of maximum moment M. The higher the traffic volume, the higher the expected maximum moment could be, and the more certain maximum moment could be higher.

4.2 Maximum Load Effect Prediction for One-Trip Permits

Permit vehicles of one-trip without escort have much lower frequency than the frequent or routine permit vehicles to be present in the highway systems. Therefore, reliability analysis should consider only one passage instead of multiple passages as in the case of frequent permits above. The simulation for maximum load effect M is conducted with a focus on the load effect due to possible alongside vehicles, as the permit vehicle is known in terms of its weight and configuration. A simulation program similar to that for frequent permit case is also developed to obtain the mean and COV of the additional moment due to possible alongside vehicles. In this case, one of the four slots in Fig.2 is reserved for the permit vehicle. The mean and COV of the alongside vehicle induced maximum moment, including site variation of live load, is listed in Table 2 for simple span bridges of various lengths. It is noted that normal traffic volume is irrelevant in this case since an alongside vehicle of normal traffic has a very small probability to be present. This is also shown by the relatively large COV in Table 2, indicating higher uncertainty of such presence.

4.3 Maximum Load Effect Prediction for Escorted Permits

Escorted permits are usually issued to those extremely heavy overload vehicles. Speed control may be exercised to reduce dynamic effect. This makes live load simulation in the reliability assessment unnecessary, since the true maximum moment, M, is known and no other vehicles are allowed to be alongside for certain.

5. PROBABILISTIC DESCRIPTION OF DEAD LOAD EFFECT AND RESISTANCE

In addition to the statistical information on live load described above, it is also important to establish an appropriate database for dead load effect and resistance. This information is needed to assess the reliability level of bridge structures in Eq.(3). Significant work of developing such database has been done by Moses and Verma [10], and Nowak and Zhou [11].

Bias and COV of dead load effect D are respectively taken equal to 1.0 and 10% according to Moses and Verma [10]. The unbiased D indicates that dead load effect can usually be computed with relatively good accuracy. The nominal value of dead load effect D_n is estimated for the derivation by an empirical equation suggested by Hansell and Viest [12]:

$$D_n = 0.0132\ \text{SpanLength(ft)}\ M_d\ g_n\ i_n \qquad (7)$$

where M_d is design live load effect and is assumed to be that due to HS-20 vehicle of AASHTO code [6].

The uncertainties in steel member behavior are due to variation in material yield, fabrication and accuracy of strength prediction theories. The bias and COV of resistance R are set equal to 1.1 and 12%, respectively [10]. The nominal value is based on material and section properties for the component being examined.

6. LOAD AND RESISTANCE FACTORS FOR PERMIT CHECKING

Current bridge evaluation in the US is performed according to AASHTO code [2,6]. The resistance factor and dead load factor of checking equation (1) are therefore chosen equal to 0.95 and 1.20 respectively, to be consistent with AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges [13]. For the three cases of permit issuance discussed above, live load factors γ_L are suggested in order to reach respective target reliability values.

6.1 Load Factors for Frequent Permits

For this case a target beta ß equal to 2.3 is selected. This target value is consistent with a recent study to prepare a new version of AASHTO bridge evaluation guidelines under development [7]. This target beta ß was obtained by a calibration of current practice according to AASHTO [2] using operating stress levels for checking [14,15].

The derived live load factor is listed in Table 3 for four cases of traffic conditions. The traffic condition addresses weight control status and traffic volume. It includes cases of weight control enforced - heavy traffic, weight control enforced - light traffic, weight control unenforced - heavy traffic and weight control unenforced - light traffic. A site with unenforced weight control may observe a significant number of illegal overweight passages. This will obviously affect the safety of a bridge structure on the site by higher expected maximum moment (see Table 1). Therefore a higher live load factor is suggested (1.20). As indicated in Table 3 the traffic volume also has influence on the bridge safety, i.e. a higher traffic volume reduces structural reliability since risk is increased thereby. The traffic volume is defined by the number of ADTT. A site is considered of light traffic volume if its ADTT is lower than 1000 trucks/day, and of high traffic volume otherwise. The load factors proposed in Table 3 allows the evaluation engineer to make decisions based on information on traffic condition. In Table 3, a nominal lane load is introduced for spans longer than the length of permit vehicle + 30 ft. It reflects load effects of vehicles in the front or to the rear of the permit vehicle. This lane load is chosen such that a relatively uniform safety index is reached over span lengths.

6.2 Load Factor for One-Trip Permits

This type of permit is usually issued to those overweight vehicles heavier than frequent permit vehicles. Therefore failure implications of this type of passage may likely be more severe. At the same time the economic benefits of an infrequent permit vehicle is much lower than for high volume permit traffic. Accordingly it is suggested that the target reliability index beta ß is raised to the range of 3.0 to 3.5 for this case. This target beta level is similar to that of new bridge designed by AASHTO specifications [6], since this type of permit may exceed any previous load on the bridge structure.

Table 4 presents the derived live load factor for this type of permit. It is noted that single lane distribution factor g_n should be used as the permit vehicle is checked in loading only one lane. This factor, for example, for a steel and prestressed concrete girder is equal to girder spacing (ft) / 7 according to AASHTO code [6]. This value contrasts with the multilane distribution factor, girder spacing (ft) / 5.5, to be used for frequent permit case. The load effect due to possible presence of other vehicles is covered herein by the suggested load factor. It should also be noted that the traffic condition (weight control and traffic volume) has negligible effect on the live load factor for this case. This is because the frequency of the permit vehicle being checked is so low that normal traffic flow may unlikely be present alongside the permit vehicle. A lane load is also prescribed in Table 4 for this case. It is consistent with that for frequent permit.

6.3 Load Factor for Escorted Permits

Table 5 proposes a live load factor for this case, based on the target safety beta ß similar to that for one-trip permits. No lane load is required here. Single lane checking should be performed as for the case of one-trip permit. This load factor is lower than that for one-trip permit above, since some of the uncertainties are eliminated. The checking is no longer influenced by uncertain traffic condition as the normal traffic will be isolated when the permit vehicle crosses the bridge. The dynamic effect is reduced as speed control can be very effective. Additionally the path of vehicle across the structure may be made to reduce member load effect, for example, by straddling girders [16].

7. EXAMPLES AND IMPACT OF PROPOSED FORMAT

This section contains examples of permit checking using the proposed format. This format is compared with currently used method, namely, AASHTO operating strength by allowable stress method [2] (referred to as current AASHTO method thereafter). It is noted that more examples are examined in [16].

Bridge:
simple span of 60 ft, 2 lanes, 6 uniform steel girders with common spacings S of 8 ft.
Analysis:
i_n=1+50/(125+SpanLength)
=1+50/(125+60)=1.27(<1.30)
g_n=S/5.5
=1.45(for multiple lane checking)
g_n=S/7
=1.14 (for single lane checking)
Dead load effect (estimated here by assumption Eq.(7)):
M_d=403 (kips-ft) (due to wheel load of HS-20)
D_n=.0132*60*403*1.45*1.27=590(kips-ft)

Checking for frequent permit:

Permit vehicle:
gross weight of 115 kips and axle configuration shown in Fig.3.
Distribution factor:
g_n=S/5.5=1.45(multiple lane checking)
Live load effect:
maximum wheel load effect M_p by the permit vehicle = 495 (kips-ft) (by moving the permit vehicle over the bridge); no lane load is needed since 60 ft (span length) < 41 (vehicle length) + 30 = 71 ft.
L_p=495*1.45*1.27=912(kips-ft)

Table 6 displays the required strengths corresponding to traffic conditions by using load and resistance factors in Table 3. It also contains the checking result by current AASHTO method for comparison. The AASHTO result is comparably expressed by plastic moment capacity assumed 13 percent higher than elastic one. It can be seen in Table 6 that the proposed method requires lower strength and allows flexibility by taking traffic condition into account.

Checking for one-trip permit:

Permit vehicle:
gross weight of 230 kip-ft and axle configuration shown in Fig.3 (100% higher than the frequent permit vehicle above).
Distribution factor:
g_n=S/7=1.14(single lane checking)
Live load effect:
maximum wheel load effect M_p by the permit vehicle = 990 (kips-ft) (by moving the permit vehicle over the bridge); no lane load is needed due to the same reason as frequent permit case above
L_p=990*1.14*1.27=1433(kips-ft)

Table 7 contains the required strength by load and resistance factors of Table 4. It also gives the result by current AASHTO method for comparison. The AASHTO result is again comparably expressed by plastic moment capacity, and requires a higher strength of members.

Checking for escorted permit:

Permit vehicle:
gross weight of 299 kips and axle configuration shown in Fig.3 (30% heavier than the one-trip permit vehicle above).
Distribution factor:
g_n=S/7=1.14(single lane checking)
Live load effect:
maximum wheel load effect M_p by the permit vehicle = 1287 (kips-ft) (by moving the permit vehicle over the bridge); no lane load is needed due to the same reason as frequent permit case above
L_p=1287*1.14*1.27=1863(kips-ft)

Table 8 contains the required

strength by load and resistance factors of Table 5, compared with the result by current AASHTO method. The AASHTO result is again comparably expressed by plastic moment capacity, and shows a higher requirement.

A general comparison of the proposed format of prescribed load factors with current AASHTO method is made as follows. A rating factor (R.F.) is used here as the indicator:

$$R.F. = (0.95\ R_n - 1.2\ D_n)/\ \gamma_L L_p \qquad (8)$$

where R_n is the plastic moment capacity assumed 13 percent higher than elastic moment capacity. If the Operating Strength requirement has been satisfied, i.e.

$$R_n = 1.13\ (D_n + L_n)\ /\ 0.75 \qquad (9)$$

R.F. can be rewritten as follows by substituting Eq.(9) into Eq.(8)

$$R.F. = [1.43(1+r)-1.2]/\gamma_L\ r \quad \text{(Frequent Permit)} \qquad (10)$$

where r is the ratio of live to dead load L_p/D_n. R.F. indicates the times of higher load effect a bridge can take than by current AASHTO method. It is shown in Table 9 given γ_L for traffic conditions and r in an exhaustively wide range. It is seen there that at least 19 percent higher strength can be used for permit checking using the proposed format. Tables 10 and 11 provide similar information for the cases of one-trip and escorted permit based on the following relations, respectively:

$$R.F. = [1.43(1+1.27r)-1.2]/(1.55\ r) \quad \text{(One-Trip Permit)} \qquad (11)$$

$$R.F. = [1.43(1+1.27r)-1.2]/(1.45\ r) \quad \text{(Escorted Permit)} \qquad (12)$$

The additional factor 1.27 for r in Eqs.(11) and (12) is due to the different distribution factors specified by the proposed method and current AASHTO method (ratio of S/5.5 to S/7). Tables 10 and 11 show that generally the proposed method allows higher permit loads (R.F. greater than 1.0), and respectively at least by 17 and 25 percent.

It has been shown that the proposed method requires generally lower strength or allows higher permit loads than current AASHTO method. It is attributed to the low frequency of permit load appearance, which is taken into account here.

8. SUMMARY AND CONCLUSIONS

Overload permit issuance has become routine practice in most states of the US. Current methods for permit checking are based on the higher operating stress in the allowable stress format. This method does not contrast normal traffic flow with relatively small number of permit loads. This paper presents a method of load and resistance factors for such permit checking. The load factors are derived against the criterion of structural reliability. Permit issuance for multiple trip, single trip and escorted trip are covered. Flexibility is permitted to use the live load factors to control the safety index for such cases as nonredundant structural types or permit vehicles of extraordinary important economic consequences. This checking method also allows consideration on site traffic conditions and overall truck weight enforcement. Examples using the proposed format are included for illustration and usually higher vehicle load is allowed by this method than current one using operating strength in allowable stress. The results are suitable for inclusion in AASHTO Manual for Maintenance Inspection of Bridges [2].

9. ACKNOWLEDGEMENTS

The Ohio Department of Transportation and Federal Highway Administration sponsored the study. The support of the New York Department of Transportation in cooperation with Federal Highway Administration is acknowledged. Dr.Y.liu of Case Western Reserve University assisted in part of the calculation. The comments of reviewers on the draft version of this paper are appreciated.

REFERENCES

[1] FHWA "Overweight Vehicles - Permits and Penalties: An Inventory of State Practices for Fiscal Year 1987" FHWA-MC-89-050, Jan. 1989

[2] AASHTO Manual for Maintenance Inspection of Bridges, Washington, D.C. 1983

[3] Cassano,R.C. and LeBeau,R.J. "Correlating Bridge Design Practice with Overload Permit Policy", Transportation Research Record 664, Sept.25-27, 1978, pp.230-238

[4] Bakht,B. and Jaeger,L.G. "A Rational Procedure for Overweight Permits", Transportation Research Record 950, Vol.I, 1984, pp.59-70

[5] Ghosn,M. and Moses,F. "Calibration of A Bridge Rating Formula for Overload Permits" ASCE Structures Congress'87, Aug.17-20, 1987, Bridges and Transmission Line

Structure, Proc. (Ed.) by L.Tall, pp.15-25
[6] AASHTO Standard Specifications for Highway Bridges, Washington, D.C. 13th Ed. 1983
[7] Moses,F. and Verma,D. "Load Capacity Evaluation of Existing Bridges, Phase II", Civil Engineering Department, Case Western Reserve University, Cleveland, OH, Draft Final Report to NCHRP, July 1989
[8] Ang,A.H-S. and Tang,W.H. Probability Concepts in Engineering Planning and Design, Vol.1 and Vol.2, New York: John Wiley & Sons, 1984
[9] Moses,F. and Ghosn,M, "A Comprehensive Study of Bridge Loads and Reliability", Civil Engineering Department, Case Western Reserve University, FHWA/OH-85/005, Final Report to Ohio Department of Transportation, Jan. 1985
[10]Moses,F. and Verma,D. "Load Capacity Evaluation of Existing Bridges" NCHRP Report 301, Washington, D.C., Dec. 1987
[11]Nowak,A.S. and Zhou,J. "Reliability Models for Bridge Analysis", Dept. of Civil Engineering, University of Michigan, Ann Arbor, March 1985
[12]Hansell,W.C. and Viest,I.M. "Load Factor Design for Steel Highway Bridges" AISC Engineering Journal, Vol.8, No.4, Oct. 1971, pp.113-123
[13]AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, Washington, D.C., 1989
[14]Verma,D. and Moses,F. "Calibration of Bridge-Strength Evaluation Code", ASCE Journal of Structural Engineering, Vol.115, No.6, June 1989, pp.1538-1554
[15]Ghosn,M. and Moses,F. "Reliability Calibration of Bridge Code" ASCE Journal of Structural Engineering, Vol.112, No.4, April 1986
[16]Moses,F., Fu,G. and Liu,Y. "A Reliability Analysis of Permit Loads on Bridges", Draft Final Report to ODOT, Case Western Reserve University, Cleveland, OH, Sept. 1990

Fig. 1a. TYPICAL COMBINATION VEHICLE WEIGHT HISTOGRAM (AT UNENFORCED SITES) WITH PERMIT TAIL

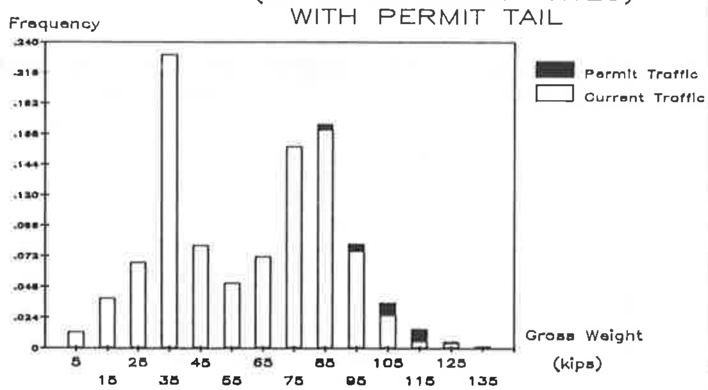

Fig. 1b. TYPICAL COMBINATION VEHICLE WEIGHT HISTOGRAM (AT ENFORCED SITES) WITH PERMIT TAIL

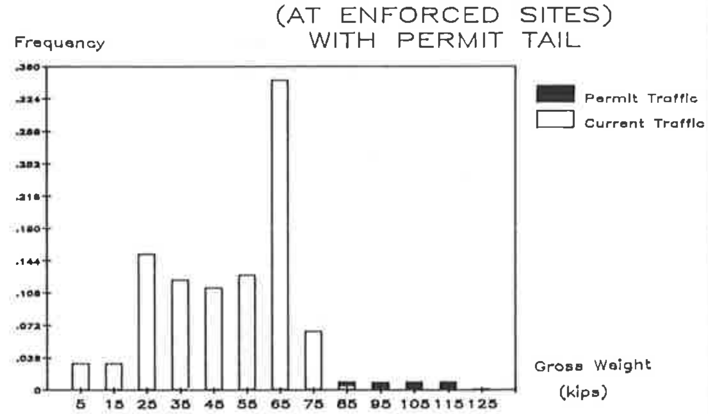

Fig. 2. VEHICLE LOCATIONS ON A BRIDGE

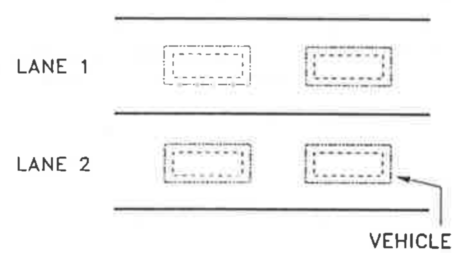

Fig.3 Example Permit Vehicles

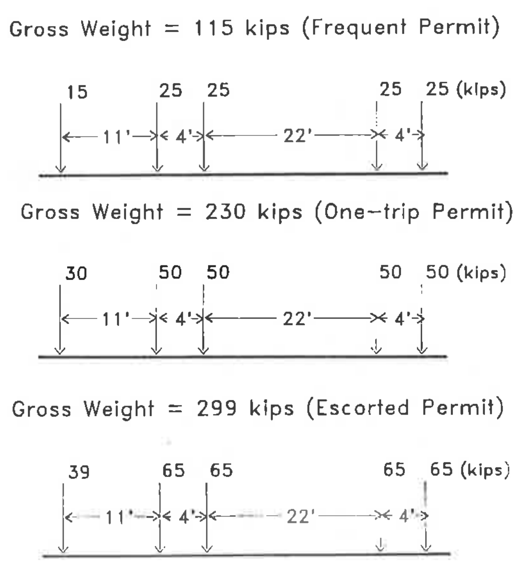

Table 1. Mean(kips-ft) and COV(%) of Maximum Moment M - Simple Spans

Span(ft)	30	60	90	120	150	180	210
Enforced Light-Traffic	641 6.2%	1758 5.4%	3307 5.1%	4890 5.4%	6499 5.6%	8244 6.7%	10085 7.9%
Enforced Heavy-Traffic	681 5.0%	1849 4.2%	3481 4.2%	5155 4.2%	6859 4.4%	8735 5.4%	10719 6.7%
Unenforced Light-Traffic	699 6.8%	1904 6.5%	3569 7.1%	5269 6.9%	6967 7.2%	8728 7.4%	10533 7.9%
Unenforced Heavy-Traffic	743 4.8%	2023 5.2%	3800 5.6%	5595 5.6%	7407 5.8%	9281 6.1%	11216 6.4%

Table 2. Mean(kips-ft) and COV(%) of Additional Maximum Moment due to Alongside Trucks (Permit Truck Effect Excluded) - Simple Spans

Span(ft)	30	60	90	120	150	180	210
Enforced	131 46.6%	358 46.9%	668 48.7%	979 49.1%	1290 49.6%	1600 49.7%	1911 49.9%
Unenforced	170 49.5%	463 49.4%	864 50.9%	1266 51.5%	1667 51.8%	2069 52.1%	2471 52.2%

Table 3. Proposed Live Load Factor γ_L for Frequent Permits (with $\phi = 0.95$ and $\gamma_D = 1.20$ for Eq.(1)) To Be Used with Multilane Distribution Factor g_n

	Heavy Traffic	Light Traffic
Weight Control Enforced	1.15	1.10
Weight Control Unenforced	1.20	1.15

Load Effect L_p = permit vehicle load effect + lane load effect
lane load effect =
0, for span length < permit vehicle length + 30 ft
200 lb/ft, for span length > permit vehicle length + 30 ft over as many spans as required produce maximum effect

Table 4. Proposed Live Load Factor γ_L for One-Trip Permits
(with ϕ = 0.95 and γ_D = 1.20 for Eq.(1))
To Be Used with Single Lane Distribution Factor g_n

γ_L = 1.55

Load Effect L_p = permit vehicle load effect + lane load effect
lane load effect =
0, for span length < permit vehicle length + 30 ft
200 lb/ft, for span length > permit vehicle length + 30 ft
over as many spans as required produce maximum effect

Table 5. Proposed Live Load Factor γ_L for Escorted Permits
(with ϕ = 0.95 and γ_D = 1.20 for Eq.(1))
To Be Used with Single Lane Distribution Factor g_n

γ_L = 1.45

Load Effect L_p = permit vehicle load effect

Table 6. Required Resistance (kips-ft) for Frequent Permit Issuance (Example)

	by Proposed Method (Using Table 3) $0.95\ R_n = 1.2\ D_n + \gamma_L\ L_p$ (Plastic Moment $F_yZ=R_n$)	by AASHTO Operating Strength of Allowable Stress $0.75\ R_n = D_n + L_p$ (Plastic Moment $F_yZ=1.13*R_n$)
Enforced-Light (γ_L=1.10)	1801	
Enforced-Heavy (γ_L=1.15)	1849	1.13*2003=2263
Unenforced-Light (γ_L=1.15)	1849	
Unenforced-Heavy (γ_L=1.20)	1897	

Table 7. Required Resistance (kips-ft) for One-Trip Permit Issuance (Example)

by Proposed Method (Using Table 4) $0.95\ R_n = 1.2\ D_n + 1.55\ L_p$ (Single Lane g_n = S/7) (Plastic Moment $F_yZ=R_n$)	by AASHTO Operating Strength of Allowable Stress $0.75\ R_n = D_n + L_p$ (Multiple Lane g_n = S/5.5) (Plastic Moment $F_yZ=1.13*R_n$)
3083	1.13*3217=3635

Table 8. Required Resistance (kips-ft) for Escorted Permit Issuance

by Proposed Method (Using Table 5) $0.95\ R_n = 1.2\ D_n + 1.45\ L_p$ (Single Lane $g_n = S/7$) (Plastic Moment $F_yZ=R_n$)	by AASHTO Operating Strength of Allowable Stress $0.75\ R_n = D_n + L_n$ (Multiple Lane $g_n = S/5.5$) (Plastic Moment $F_yZ=1.13*R_n$)
3589	1.13*3941=4460

Table 9. Impact of Proposed Method on Frequent Permit Checking
(R.F. by the proposed method if the strength satisfies Operating Strength)

$r = L_p/D_n$	0.50	0.75	1.00	1.50	2.00	5.00	10.0	100	∞
	R.F. by the proposed method								
Enforced-Light (γ_L=1.10)	1.72	1.58	1.51	1.44	1.40	1.34	1.32	1.30	1.30
Enforced-Heavy (γ_L=1.15) and Unenforced-Light	1.64	1.51	1.44	1.38	1.34	1.28	1.26	1.26	1.24
Unenforced-Heavy(γ_L=1.20)	1.58	1.45	1.38	1.32	1.29	1.23	1.21	1.19	1.19

Table 10. Impact of Proposed Method on One-Trip Permit Checking
(R.F. by the proposed method if the strength satisfies Operating Strength)

$r = L_p/D_n$	0.50	0.75	1.00	1.50	2.00	5.00	10.0	100	∞
R.F.(γ_L=1.55)	1.47	1.37	1.32	1.27	1.25	1.20	1.19	1.18	1.17

Table 11. Impact of Proposed Method on Escorted Permit Checking
(R.F. by the proposed method if the strength satisfies Operating Strength)

$r = L_p/D_n$	0.50	0.75	1.00	1.50	2.00	5.00	10.0	100	∞
R.F.(γ_L=1.45)	1.57	1.46	1.41	1.36	1.33	1.28	1.27	1.25	1.25