Guide Specification Strength Capacity Rating of Existing Girder Bridges

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The impact of the AASHTO Guide Specifications for Strength Evaluation of Steel and Concrete Bridges (STRENGTH method) on 40 steel and 33 concrete bridges in Missouri was investigated. The STRENGTH method is a reliability-based load and resistance factor rating procedure. The variable factors depend on levels of redundancy, deterioration, inspection, maintenance, truck volume, and weight enforcement, and selection of these factors is subjective, requiring considerable engineering judgment. The STRENGTH method considers site-specific loading and capacity characteristics to obtain consistent levels of safety over the bridge inventory. For bridges with good load and resistance characteristics, the STRENGTH method can significantly increase load ratings over current AASHTO load factor rating operating levels. However, deterioration and adverse traffic conditions can cause STRENGTH ratings to fall below load factor rating inventory levels. A method to evaluate the load capacity of concrete bridges that do not have detailed bridge plans is also investigated.

In the United States, federal law (*1*) requires that all bridges be evaluated periodically. The inspection process shall include a physical investigation of the bridge to ascertain the bridge's overall safety and operational characteristics and shall include a bridge load-carrying capacity evaluation (bridge rating). The governing authority over bridge inspections and load ratings is AASHTO. AASHTO's Manual for Maintenance Inspection of Bridges (*2*) (referred to as the maintenance manual) is used for guidance in the evaluation process and the current AASHTO Standard Specifications for Highway Bridges (*3*) (referred to as the design specs) is also used in conjunction with the maintenance manual. Although not used in this paper, there is also the new Manual for Condition Evaluation of Bridges, recently approved by AASHTO (*4*).

The process of bridge evaluation consists of two important operations: inspection and rating. Bridge inspection determines the actual condition of the bridge based on field inspection and field measurements. Results of the current inspection are compared with those of previous records to determine whether there are changes in the bridge condition. If there are substantial changes, or trends of deterioration are verified, then the load capacity is evaluated (bridge rating) for the new conditions.

Bridge rating is concerned with two major issues:

1. What vehicle, or group of vehicles, should be used for the load capacity evaluation?
2. How should the capacity of the bridge be evaluated?

Bridge rating is a mathematical exercise by which the strength of the bridge is evaluated. The specific outcome of the analysis is the rating factor (RF). The RF is the ratio of the calculated live load capacity of the bridge to the rating vehicle live load effects. Typically, the AASHTO rating vehicles, or state specific vehicles, are used to approximate the live load effects. The RF multiplied by the rating truck weight is the rating load. If RF is less than unity, then the bridge is judged to be deficient and some type of action is called for such as

1. Posting (reduce live load and/or speed),
2. Retrofitting the bridge,
3. Replacing the bridge, or
4. Closing bridge to traffic.

For most bridges, only a flexural capacity rating check is performed. However, there may be situations when a shear or bearing capacity check is warranted. Examples of these situations would be in deteriorated members with significant section loss and for older bridges. Also, fatigue rating (*5*) may be required in members with known high service load stresses. One important feature of the rating process is to subject the mathematical conclusions to the judgment of experienced bridge engineers.

With regard to structural analysis and load capacity limit states, as of 1994 there were three AASHTO methods for rating beam and girder bridges:

1. Allowable stress rating (ASR). For the ASR method, the nominal live loads on the structure and all other nominal loads shall not produce stresses in the member that exceed allowable stresses (*2*).
2. Load factor rating (LFR). For the LFR method, the criteria are that factored live loads and factored other loads must not exceed the (factored for concrete) nominal strength of the member (*2*).
3. Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges (STRENGTH method). The STRENGTH method is a load and resistance factor method using variable site-specific factors. Factored live loads and factored other loads must not exceed the factored member capacity (*6*).

There are also field testing rating methods where diagnostic or proof loads are physically applied to the bridge (*7*).

All three of these analytical rating methods use the maintenance manual as a guide for bridge inspection. The ASR and LFR methods are also contained in the maintenance manual. The STRENGTH method is similar to the LFR method; however, the load, resistance, and impact factors are variable and depend on site-specific characteristics. The nominal capacity is the same as the LFR maximum-strength capacity and both methods use the same level of structural usefulness (i.e., flexural hinge).

The ASR and LFR methods are direct extensions of their respective design procedures. In design, additional uncertainty needs to be incorporated in the process to meet the desired safety. Over the long design life, conservatism is warranted for changes in traffic volume and loads, deterioration, and material variabilities. Evaluating an

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existing bridge over short intervals removes many of the uncertainties inherent in the design process. When load rating an existing bridge, the uncertainty associated with the truck volume and expected weights, the level and rate of deterioration, and the as-built geometry and material properties are lower than at the design stage.

The truck volume and expected weights on individual bridges will have a large impact on the demands of the structure. For instance, a rural bridge with five trucks a day demands much less from a structure than an interstate bridge with 10,000 trucks a day. Likewise, a bridge with a 10 percent per year deterioration rate is of more concern than a bridge with virtually no deterioration rate. The ASR and LFR methods fail to fully consider these site characteristics. The methods will rate a high-volume severely deteriorating bridge and a low-volume bridge not deteriorating equally. Ignoring the differences in particular bridges leads to inconsistent safety.

An NCHRP project (*8*) was initiated in 1980 with the objective of developing improved techniques for evaluating the load-carrying capacity of reinforced concrete bridges. Another NCHRP project (*9*) was initiated in 1985 to extend and finalize the findings of NCHRP Project 10-15 (*8*) for reinforced concrete, prestressed concrete, and steel bridges. The researchers' goal was to produce "a flexible comprehensive approach to bridge evaluation that best utilizes the economic resources available and yet maintains consistent and definable criteria for ridge safety." To achieve this, a reliability framework was adopted that allowed a range of load and resistance factors (partial load factors) depending on site-specific bridge characteristics and the level of effort in the rating process. The STRENGTH method (*6*) is based on these two NCHRP projects.

The STRENGTH method yields only one rating factor corresponding to a strength limit state, whereas the LFR method has an operating rating and an inventory rating for both a maximum-strength capacity and a serviceability capacity. Ignoring serviceability limits (not including fatigue limits) for existing bridges is justified in the STRENGTH method by the fact that these bridges have survived these serviceability demands in the past.

OBJECTIVES

The AASHTO guide specs STRENGTH method (*6*) was released in 1989. However, not much is known on how the new procedures will affect the rating process. Barker et al. (*10*) investigated and compared the LFR method and the STRENGTH method with 73 steel and concrete girder bridges typical of state and rural bridges in Missouri. The study emphasized the impact of the STRENGTH method and the procedural changes from the LFR method.

This paper presents the following:

1. Comparisons of the STRENGTH method to the LFR maximum-strength operating and LFR serviceability operating ratings for 40 steel girder bridges,
2. Comparisons of the STRENGTH method to the LFR maximum-strength operating rating for 33 concrete girder bridges, and
3. An historically based method to evaluate concrete bridges with insufficient or nonexistent plans.

LOAD CAPACITY RATING EQUATION

For the LFR maximum-strength operating level and the STRENGTH method, the general load capacity rating equation is

$$\Gamma_D D_n + (RF)\Gamma_L L_n(DF)(1 + I) \leq \Phi M_n \tag{1}$$

or, solving for the rating factor,

$$RF = \frac{\Phi M_n - \Gamma_D D_n}{\Gamma_L L_n(DF)(1 + I)} \tag{2}$$

where

RF = rating factor (RF $\geq$ 1 is sufficient capacity),
Γ_D = dead load factor,
Γ_L = live load factor,
Φ = resistance factor,
M_n = nominal resistance,
D_n = nominal dead load,
L_n = nominal live load from the rating vehicle,
DF = lateral distribution factor, and
I = impact factor.

The nominal live loads are the same for both procedures. The lateral distribution factors of these loads is also identical except the STRENGTH method adjusts the factor if a method other than the AASHTO design specs S/D method is used. The nominal dead load is the same except that the STRENGTH method increases the nominal overlay thickness by 20 percent as a result of excessive uncertainty in overlay thickness estimations. The nominal resistance should consider the effects of deterioration with a reduced section analysis.

The major difference between the two methods is in the load and resistance factors and the impact factor as shown in Table 1. The STRENGTH method uses variable load, resistance, and impact factors, and the rating engineer must choose the values on the basis of site-specific information. To obtain consistent ratings, the use of engineering judgment is critical for selecting these subjective factors.

The LFR method also has a serviceability limit (excluding fatigue) for the operating level. For steel bridges the equation is

$$RF = \frac{M_s - D_n}{L_n\,(DF)(1 + I)} \tag{3}$$

The limit is basically a limited stress at service or nominal loads where M_s is the serviceability strength corresponding to the operating level and the other variables are defined above. The STRENGTH method has no such serviceability limit.

The LFR inventory ratings are 60 percent of the LFR operating ratings. This is simply the ratio of the operating and inventory live load factors (1.3/2.17) from Table 1.

STRENGTH METHOD LOAD AND RESISTANCE FACTORS

The following are general guidelines (*6,10*) for determining the factors for the STRENGTH method. For a more detailed explanation, and for variances to the conditions that follow, the reader is referred to the guide specs (*6*).

Dead and Live Load Factors

The dead load factor is a constant 1.2. However, the live load factor ranges from 1.30 to 1.80. A factor of 1.30 (same as LFR operat-

TABLE 1 Load and Resistance Factors

Factor	LFR	STRENGTH
Γ_D	1.30	1.20 plus 20% additional thickness on the wearing surface
Γ_L	1.30 Operating 2.17 Inventory	1.30 low volume and enforced weight limits 1.45 high volume and enforced weight limits 1.65 low volume and unenforced weight limits 1.80 high volume and unenforced weight limts
Φ	1.00 steel 0.90 concrete	0.55-0.95 based on redundancy, deterioration, inspection, and maintenance
I	50/(L+125)≤0.3 based on span length	0.10 smooth deck 0.20 significant deck roughness 0.30 major deficiency in riding surface

ing) represents a low-volume bridge with good weight enforcement. As the volume increases or enforcement decreases, or both, the live load factor increases. The volume is deterministic, but the level of enforcement is subjective. The rule that the site is considered enforced if less than 5 percent of the trucks exceed legal limits can be used if this information is available and dependable. Of course the rating agency could be conservative and categorically assume insufficient enforcement; however, this would defeat the objective of having uniform safety over the bridge inventory.

Impact Factor

The impact factor depends on the riding surface roughness. The dynamic effects range from 0.10 for smooth surfaces to 0.30 for surfaces with serious deficiencies. Inspection procedures will need to be developed and a new appraisal rating will need to be incorporated into the inspection program to determine this subjective factor. The deck appraisal did not correlate well with the perceived dynamic effects for the bridges used in this study. The engineer should consider the design specs impact factor when choosing from the STRENGTH method options. Perhaps for shorter bridges, 0.20 should be used unless there are serious deficiencies. For longer bridges with smooth conditions, a value of 0.10 could be justified.

Resistance Factor

The resistance factor has a large impact on the load rating because it can vary from 0.55 to 0.95. The basic resistance factor for a member in good condition (0.95 for steel and 0.90 for concrete) is significantly decreased if there is deterioration. If there is deterioration, the resistance factor can be increased if a careful inspection is executed and either increased or decreased, depending on whether maintenance will inhibit future section losses or deterioration is uninhibited, respectively. Although the resistance factor is subjective, there seems to be adequate information to determine the adjustments for deterioration and inspection effort (*10*).

ASSUMPTIONS USED FOR BRIDGE DATA BASE

This study used a uniform set of assumptions for determining the live load, resistance, and impact factors for the STRENGTH method. For the live load factor, all sites were assumed enforced. This was done because a high percentage of the bridges should have less than 5 percent of the trucks exceeding legal limits. However, for posted bridges this may not be true. There would be a tendency for bridges with restrictive loading to have a higher percentage of weight violators. Future studies are needed to examine weight characteristics on posted bridges. If the sites are unenforced, the STRENGTH ratings would decrease by approximately 20 percent from the ratings assuming enforced conditions.

The resistance and impact factors were chosen conservatively. Deck and superstructure appraisal ratings from inspection reports were used to determine the level of deterioration according to the STRENGTH method guidelines. The resistance factor was also decreased for intermittent maintenance, and it was assumed that there was not a careful inspection. The impact factor was based on the deck appraisal rating. Thus, the majority of the STRENGTH ratings reported herein could be increased significantly (upwards of 10 percent) with a careful inspection adjustment and a vigorous maintenance adjustment.

BRIDGE DATA BASE

This paper compares the LFR method and the STRENGTH method for 73 bridges typical of state and rural girder bridges in Missouri. Of the 40 steel girder bridges examined, 10 are simple-span composite (SC), 5 are continuous-span composite (CC), 20 are simple span noncomposite (SNC), and 5 are continuous span noncomposite (CNC). Of the sections checked for the capacity ratings for both the composite and noncomposite bridges, there is a mix of compact, noncompact, braced, and unbraced sections. The dates the bridges were built range from 1932 to 1968.

Of the 40 steel girder bridges, two have high truck volume (Γ_L = 1.45), and the remaining 38 have low truck volume (Γ_L = 1.30). Six of the bridges have slight deterioration (Φ = 0.80), whereas the rest have insignificant section loss (Φ = 0.90). Three of the bridges have lower deck appraisal ratings, which resulted in an impact factor of 0.20, whereas the remaining bridges have high deck appraisal ratings, which resulted in an impact factor of 0.10.

The 33 concrete girder (T-beam) bridges analyzed for comparison in this study are typical state bridges built between 1922 and 1961. Of the 33, five of the bridge plans have general member dimensions but do not have reinforcement details. This is a problem with many concrete bridges across the nation. However, as will be discussed, a method to estimate the reinforcement details on the basis of limited historical data was used to determine the load capacities. This method, with future verification and refinement, could be an asset for rating concrete bridges that have no plans.

Of the 33 concrete girder bridges, 30 have low truck volume ($\Gamma_L = 1.30$) and three have high truck volume ($\Gamma_L = 1.45$). Whereas all the steel bridges in this study have good or fair structural appraisal ratings, 8 of the concrete bridges have superstructure conditions of good or fair ($\Phi = 0.85$), 16 are classified deteriorated ($\Phi = 0.75$), and 9 are heavily deteriorated ($\Phi = 0.65$). Deterioration significantly reduces the STRENGTH rating as evidenced by the wide range of the resistance factors. The concrete bridge impact factors are distributed as follows: 25 have impact factors of 0.10, 5 have impact factors of 0.20, and 4 have impact factors of 0.30 (major deficiency in the riding surface).

IMPACT OF THE STRENGTH METHOD

Table 2 shows the average rating factors for controlling vehicles and controlling spans for the steel and concrete bridges. For the steel data base, the STRENGTH method average ratings (1.33) were significantly greater than the LFR operating levels considering serviceability limits (1.19). However, when considering only the maximum-strength LFR limit, the average ratings were nearly identical (1.33 STRENGTH and 1.34 LFR). When serviceability controls, the LFR rating is lowered from what the maximum-strength limit ratings dictate and, therefore, the difference between the STRENGTH and LFR methods increases.

This difference is illustrated in Figures 1 and 2. In these figures, the ratio of the STRENGTH RF to the LFR operating RF is plotted against the controlling span length. In Figure 1, the ratio varies considerably because of the serviceability limit. All the points above the dashed line, and the two points indicated below this line, had serviceability controlling the LFR rating. It is clear that large increases in the rating could be realized if the rating agency switches from LFR serviceability limits to the STRENGTH method limits, especially for SC bridges. This would be in agreement with the STRENGTH method philosophy that these bridges have survived these serviceability demands in the past. However, if the LFR method ignores serviceability limits and uses only maximum strength limits, the ratio is much more uniform, as shown in Figure 2. Here all the points are uniform except when one or more of the factors change.

In Figure 2, the majority of the RF ratios are between 1.0 and 1.1. These points correspond to a low-volume roadway with good weight enforcement ($\Gamma_L = 1.30$), a good riding surface ($I = 0.10$), and members in good shape ($\Phi = 0.90$). As the load factors increase or the resistance factor decreases, the STRENGTH ratings will decrease and, thus, give a relatively lower rating compared with the LFR method. In summary, the STRENGTH ratings are, on average, approximately equal to or above the average LFR operating ratings.

This is not the case for the concrete bridges. For the concrete bridges shown in Table 2, the STRENGTH method average ratings (1.06) were well below the LFR operating level maximum-strength ratings (1.30). In fact, they were about midway between the LFR operating and the LFR inventory levels (0.78). This means that the STRENGTH method would require more restrictions on the concrete bridges of this data base relative to those imposed on the steel bridges.

In Figure 3, the ratio of the STRENGTH RF to the LFR operating RF is plotted against the controlling span length. There are three distinct regions that correspond to the Φ factor or the level of deterioration. With heavier deterioration, the STRENGTH method will give a low rating compared with the LFR method. Variations in the other factors affect the ratio to a lesser extent. Examination of the STRENGTH method resistance factors, live load factors, and impact factors reveals why the concrete bridges did so much more poorly than the steel bridges when LFR ratings were compared.

The concrete data base had an average resistance factor of 0.75, whereas the average was 0.89 for the steel bridges. This difference greatly exceeds the 0.05 difference for steel and concrete members in good condition. The disparity occurred because the concrete bridges had consistently lower structural appraisal ratings. Visual inspection of the bridges, however, showed no apparent condition differences. Because the STRENGTH method considers deterioration in the rating process and because of the subjective factors, an important aspect of the STRENGTH method is the inspection. This apparent discrepancy between steel and concrete bridge inspection appraisals will hinder the consistency of the STRENGTH ratings.

For example, four of the concrete bridges have structural appraisal ratings of 3. According to the inspection, these bridges

TABLE 2 Rating Factors for the 73-Bridge Data Base

RATING METHOD	STEEL BRIDGES		CONCRETE BRIDGES	
	AVG	RANGE	AVG	RANGE
Guide Spec STRENGTH Rating	1.33	0.58-3.77	1.06	0.26-2.23
LFR Operating Max. Strength & Serviceability Rating	1.19	0.63-2.99	NA	NA
LFR Operating Maximum Strength Only	1.34	0.63-3.98	1.30	0.78-2.68
LFR Inventory Max. Strength & Serviceability Rating	0.71	0.38-1.79	NA	NA
LFR Inventory Maximum Strength Only	0.81	0.38-2.39	0.78	0.47-1.61

Serviceability not Considered for Concrete Bridges

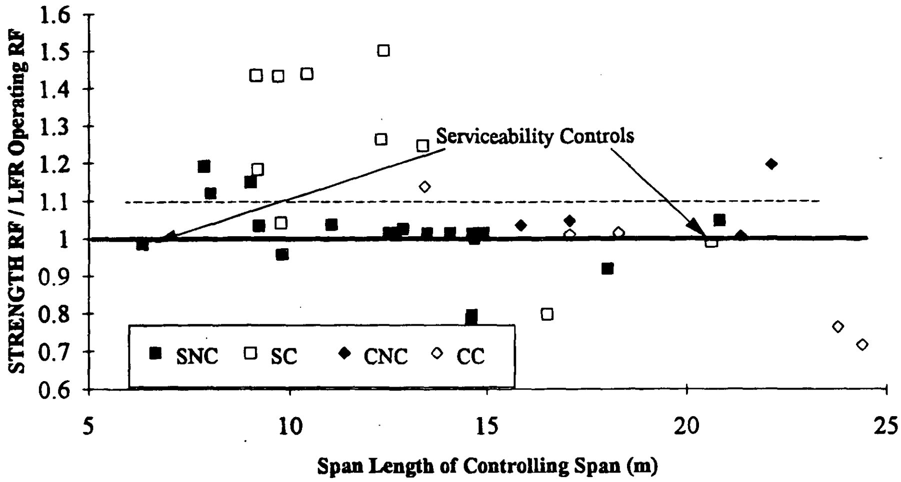

FIGURE 1 Ratio of STRENGTH and LFR operating maximum strength and serviceability rating factors for steel bridges.

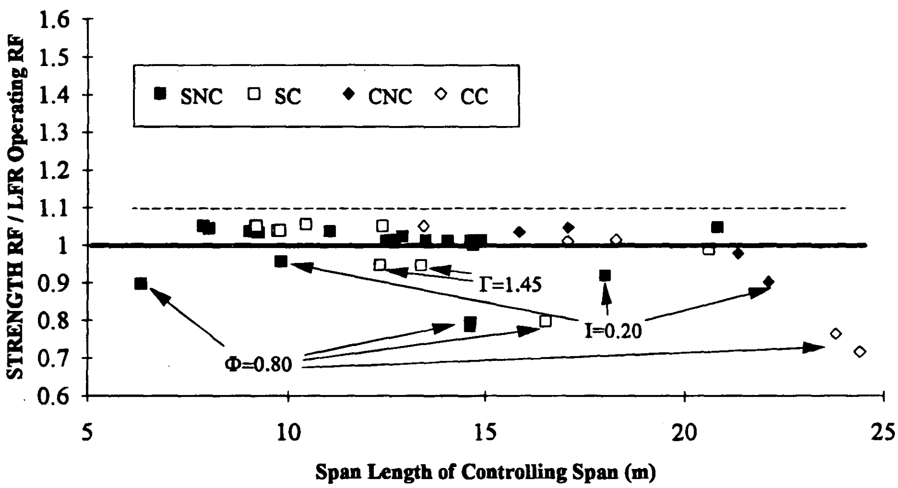

FIGURE 2 Ratio of STRENGTH and LFR operating maximum strength rating factors for steel bridges.

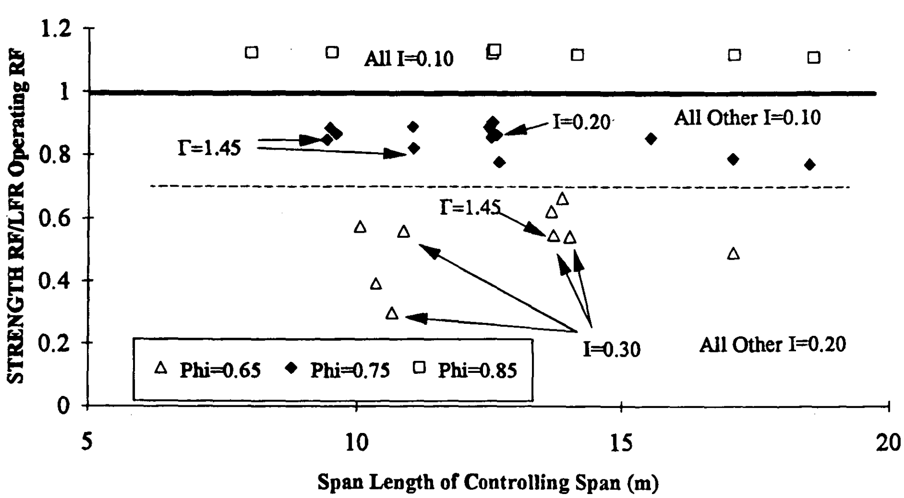

FIGURE 3 Ratio of STRENGTH and LFR operating maximum strength rating factors for concrete bridges.

are in serious condition. A superstructure appraisal of 3 means the following (*11*):

> Serious Condition: Loss of steel section, deterioration, or spalling has seriously affected primary structural components. Repair or rehabilitation required as soon as possible. Damage or disintegration of a structural support element which requires shoring, auxiliary splices, or substitute members. Severe disintegration of concrete. Diagonal shear cracks. Wide flexural cracks. Delamination from primary steel.

If these concrete bridges are truly in the shape that the appraisal ratings indicate, then the LFR method is not adequately representing the condition of the bridge or the seriousness of the deterioration. The LFR method would give nearly the same ratings to these seriously deteriorated bridges and a new bridge. However, the STRENGTH method reduces the load-carrying capacity to reflect the heavy deterioration.

The average live load factor for the concrete bridges is 1.314, and the average is 1.307 for the steel bridges. Therefore, the live load factor did not cause much difference between the steel and concrete data bases. However, the average impact factor for the concrete bridges is 0.14, whereas the average for the steel bridges is 0.108. This factor has a direct effect on the ratings and the relative differences between the steel and concrete bridges.

CONCRETE BRIDGES WITH INSUFFICIENT PLAN DETAILS

An ongoing problem for many states is how to estimate the load-carrying capacity of concrete bridges that lack reinforcement details. One solution is to field test the bridge with applied diagnostic or proof loading (*7*). However, if load testing is not feasible, the maintenance manual states that a bridge that shows no signs of distress need not be posted. This clearly is not acceptable in many situations. There is no definitive definition of distress and, if there is distress, there are no guidelines for posting limits. There are also no permitting guidelines should the situation arise.

There is a need for a more consistent strategy for making rational estimations of the load-carrying capacity for these bridges. The information required to calculate the section capacity is the structural depth (d), the area of steel, represented by the reinforcement ratio (ρ), and the concrete strength, (f'_c), which can be estimated with respect to age in the maintenance manual. Following is a method based on historical data from existing concrete bridges with known reinforcement configurations (*10*). Figure 4 plots the normalized depth against the age for the 28 concrete bridges in the data base that had full plans including reinforcement details. The normalized depth is simply the structural depth divided by the overall depth (d/H). Figure 5 is a similar plot for the normalized reinforcement ratio (ρ/ρ_{max}) where ρ_{max} is 75 percent ρ_{bal}.

The normalized depth shows little scatter for the 28 bridges. The normalized reinforcement ratio shows more scatter. The averages and standard deviations for the two variables are shown on the figures. The proposed strategy for selecting the reinforcement area and effective depth is based on these statistical values. To incorporate a margin of safety, target values lying roughly 1.3 standard deviations below the mean values were selected. This distance from the mean corresponds roughly to a probability that the assumed value will be conservative 90 percent of the time.

The results of these calculations yield the following target values: $d/H = 0.84$, and $\rho/\rho_{max} = 0.66$. These numbers are rounded to convenient values to produce the following recommended strategy for bridges with unknown reinforcement details: $d = 0.85\ H$, and $\rho = 2/3\ \rho_{max} = 0.5\ \rho_{bal}$. Although this procedure does not in any way ensure the actual reinforcement configuration, it does provide the rating engineer with a consistent and rational method for selecting reinforcement properties and dimensions for bridges without reinforcement details.

Before implementing the specific values in the above example, a larger data base should be surveyed. Further refinement of the target values could also be obtained by examining the data with respect to design-specific factors such as year built, material properties, and span length.

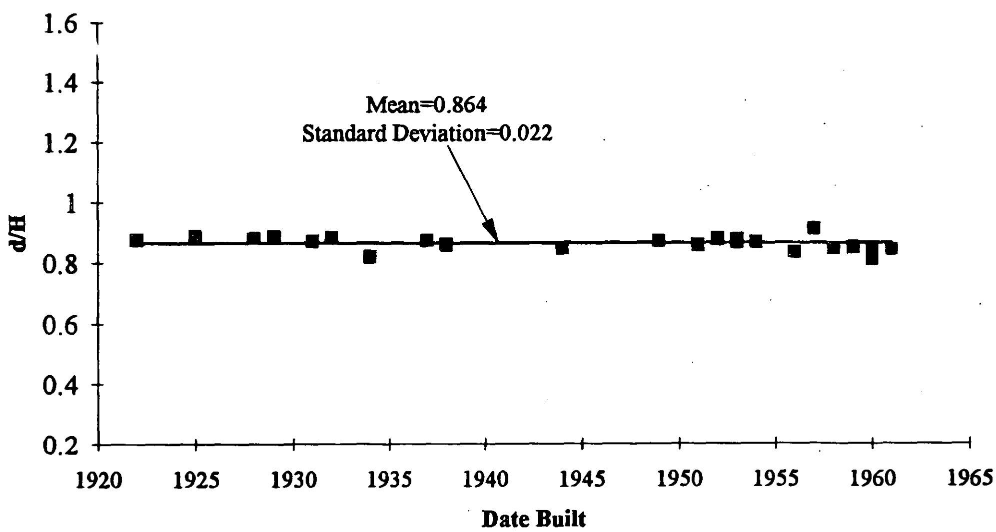

FIGURE 4 Normalized structural depths (*d/H*) for concrete bridges.

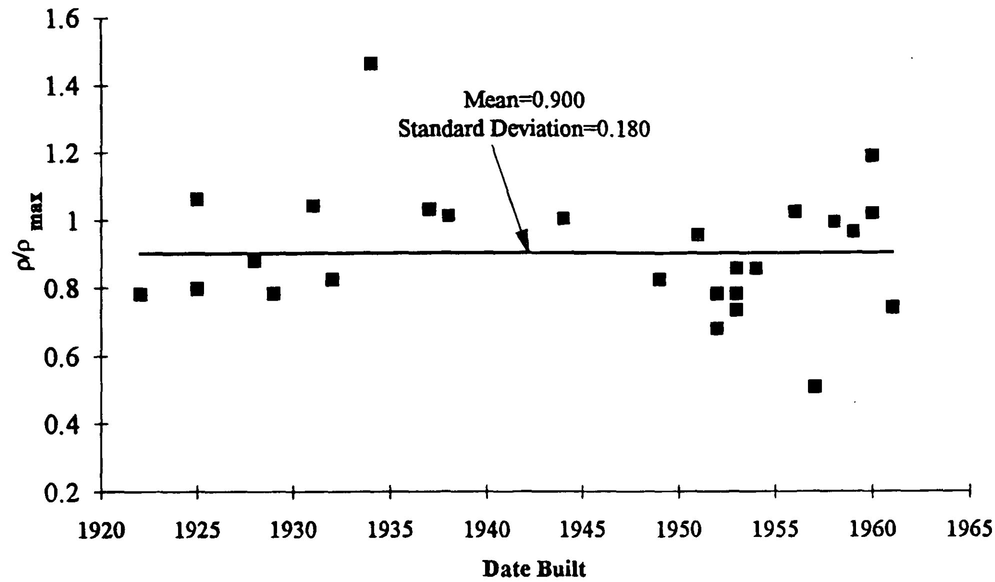

FIGURE 5 Normalized reinforcement ratios (ρ/ρ_{max}) for concrete bridges.

SUMMARY AND CONCLUSIONS

The STRENGTH method is a reliability-based load and resistance factor method that uses variable site-specific live load, impact, and resistance factors. The procedure considers redundancy, weight enforcement, and truck volume, deterioration, inspection effort, and level of maintenance in the rating. The STRENGTH method has several advantages over the current LFR method, including the following (*9,10*):

1. Implicitly recognizes the difference between design and evaluation,
2. Provides consistent level of safety for all bridges,
3. Uses site-specific load and resistance characteristics,
4. Incorporates engineering judgment in the rating process,
5. Uses familiar form—similar to LFR method,
6. Permits potential improvements in ratings through extra efforts in inspection and maintenance,
7. Encourages better inspection and maintenance programs, and
8. Eliminates much of the variation in state posting practices.

Unfortunately, the STRENGTH method also has disadvantages (*10*):

1. Load, impact, and resistance factors are subjective;
2. Some terms, such as "careful," "estimated," "vigorous," and "intermittent," are subjective;
3. States may choose to categorically use conservative factors, defeating the purpose of the STRENGTH method;
4. Additional inspection information is required; and
5. It changes established bridge rating programs.

The STRENGTH method is similar to the LFR maximum-strength method in form. For bridges in good shape with reasonable traffic enforcement, the STRENGTH ratings will be similar to LFR maximum-strength operating levels. The STRENGTH method can greatly improve ratings when serviceability limits control the LFR. With increased impact, deterioration, and truck volume, the STRENGTH method ratings decrease significantly and can fall below LFR inventory levels.

The concrete data base definitely shows the effect of deterioration (Figure 3). With 16 out of 33 bridges classified as slightly deteriorated and 9 classified as heavily deteriorated, many of the STRENGTH ratings fell below LFR inventory level. Although the results may seem startling, if these bridges are in this much disrepair, the LFR method does not indicate the seriousness of the situation. In other words, if the STRENGTH method yields consistent levels of safety for these bridges, the LFR method is not achieving desirable safety levels. It is suggested that, since the deterioration was based on appraisals, inspection procedures should be reviewed to determine if these bridges are truly in such poor structural condition.

Unlike the LFR method, the STRENGTH method uses variable load and resistance factors depending on site-specific information. This creates subjectivity in the rating process, and the inspection and rating personnel must use judgment in choosing values. The consistency and accuracy in which these personnel choose the factors determines the success of the STRENGTH method in meeting its objective of consistent reliability in rating.

RECOMMENDATIONS

It is first recommended that the AASHTO Guide Specification for the Strength Evaluation of Existing Steel and Concrete Bridges (*6*) be used to determine alternative posting ratings for girder bridges that are currently posted according to ASR or LFR methods. The STRENGTH method has the potential to increase or remove posted limits on many bridges, and an effort by the state Department of

Transportation (DOT) will quickly determine the usefulness of the procedure. To apply the STRENGTH method consistently, there will need to be modifications to the current inspection procedures to collect additional information. This will inherently require better communication between inspectors and bridge rating engineers. This study examined only reinforced concrete and steel girder bridges. The STRENGTH method also applies to prestressed girder and truss bridges. These types of bridges should be examined in conjunction with concrete and steel girder bridges.

Once states establish application procedures and gain confidence in the STRENGTH method with these posted bridges, the provisions should be applied to all girder bridges for a consistent evaluation over the bridge inventory.

During routine rating of concrete bridges, the state DOT should keep a tally of reinforcement ratios and structural depths to build a data base for implementing a strategy for concrete bridges without reinforcement details similar to the limited strategy shown in this paper.

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REFERENCES

1. National Bridge Inspection Standards, *Code of Federal Regulations*, Vol. 23, Section 650.3. Government Printing Office, Washington, D.C., 1988.
2. *Manual for Maintenance Inspection of Bridges*. AASHTO, Washington, D.C., 1983.
3. *Standard Specifications for Highway Bridges*. AASHTO, Washington, D.C., 1989.
4. *Manual for Condition Evaluation of Bridges*. AASHTO, Washington, D.C., 1994.
5. *Guide Specifications for Fatigue Evaluation of Existing Bridges*. AASHTO, Washington, D.C., 1990.
6. *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*. AASHTO, Washington, D.C., 1989.
7. Lichtenstein, A. G. *Bridge Rating Through Nondestructive Load Testing*. Draft Report. NCHRP Project 12-28(13A). TRB, National Research Council, Washington, D.C., 1993.
8. Imbsen, R., W. Liu, R. Schamber, and R. Nutt. NCHRP *Report 292; Strength Evaluation of Existing Reinforced Concrete Bridges*. TRB, National Research Council, Washington, D.C., 1987.
9. Moses, F., and D. Verma. *NCHRP Report 301: Load Capacity Evaluation of Existing Bridges*. TRB, National Research Council, Washington, D.C., 1987.
10. Barker, M. G., D. M. Koenig, and L. M. Magruder. Load Rating Steel and Concrete Girder Bridges in Missouri. Missouri Cooperative Highway Research Program Report 91-1. Missouri Highway and Transportation Department, Jefferson City, 1994.
11. Minor, J., K. White, and R. Busch. *NCHRP Report 312*: Condition Surveys of Concrete Bridge Components Users Manual, TRB, National Research Council, Washington, D.C., 1988.

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