Load Rating and Ultimate Capacity Evaluation of Compact Steel Girder Bridges

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Slab-on-steel girder bridges are highly redundant structures and show significant redistribution capacity and a large reserve capacity in the inelastic range. To achieve consistent levels of safety over the bridge inventory, consideration should be given to the ultimate capacity of the system. First-hinge and inelastic limit rating methods for a single-span and a three-span composite bridge were examined. The rating methods were the AASHTO load factor rating maximum-strength operating level rating, the AASHTO Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges rating, and the single-girder and the system shakedown limit ratings. Examination of limits beyond the first hinge yielded insight into the available reserve capacity. A comparison of the methods showed that even simple-span bridges have significant reserve capacity beyond the first hinge. The additional capacity was attributed to transverse redistribution of forces. However, the total reserve capacity was not uniform for all bridges. The three-span bridge example shows that, in addition to this transverse component, longitudinal redistribution of forces adds even more to the reserve capacity. The first-hinge rating methods do not reflect the relative ultimate load-carrying capacities of one-span and multispan bridges.

Bridges in the United States must be inspected periodically for maintenance reasons and to ensure bridge safety to the public. Along with the visual inspection, the load-carrying capacity (bridge rating) must be evaluated to determine the maximum truck loads allowed on the structure. The specific outcome of a bridge rating is the rating factor (RF), which is the ratio of the calculated live load capacity of the bridge to the rating vehicle live load effects (*1*). Typically, standard 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 greater than unity, the bridge is deemed adequate for the rating vehicle weight. If RF falls below one, the bridge is considered under capacity for that rating truck load, and the bridge needs to be posted for restrictive loading or speed, or both, or some other action must be implemented.

Currently there are three AASHTO methods for rating beam and girder bridges: the allowable stress rating (ASR) (*1*), the load factor rating (LFR), (*1*) and the Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges (*2*) (to be abbreviated herein as STRENGTH). For ASR, the nominal live loads on the structure and all other nominal loads shall not produce stresses in the member that exceed allowable stresses. For LFR, the criteria are that factored live loads and factored other loads must not exceed the nominal strength of the member. 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.

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All three of the above rating methods use the AASHTO Manual for Maintenance Inspection of Bridges (*2*) as a guide for bridge inspection. The ASR and LFR methods are also contained in the manual. Although not used in this paper there is also the newly approved AASHTO Manual for Condition Evaluation of Bridges (*3*). 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., single flexural hinge).

The STRENGTH method is a product of NCHRP reports by Imbsen et al. and Moses and Verma, (*4,5*). The researchers' goal was to produce "a flexible comprehensive approach to bridge evaluation which best utilizes the economic resources available and yet maintains consistent and definable criteria for bridge 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 result is a rating method that approaches a uniform level of safety for the first hinge limit state for steel bridges.

Slab-on-steel multigirder bridges are highly redundant structures and show significant redistribution capacity and a large reserve capacity in the inelastic range. To properly evaluate the ultimate safety of a bridge, this reserve capacity should be considered. Barker and Galambos (*6*) present a method to examine the ultimate load-carrying capacity of bridges on the basis of the inelastic system limit states. The ultimate limit is the maximum shakedown (incremental collapse) limit of the multigirder system. Galambos et al. (*7*) concluded in an NCHRP report that the shakedown limit of the system, coupled with the load and resistance factors developed for the STRENGTH method, is a rational and consistent method to rate existing bridges for the ultimate safety.

The shakedown limit state of a statically indeterminate structure subjected to variable repeated loads is that extreme load set that will just prevent incremental collapse (*8*). Consider a moving load, or a set of moving loads, that exceed the elastic limit load but are less than the plastic collapse load. As these loads cross an indeterminate structure, increments of inelastic rotation occur at sections along the structure. If the loads are less than or equal to the shakedown limit load (incremental collapse load), on further loadings the incremental increase in rotations and deflections decrease in magnitude and eventually vanish. After the permanent deformations stabilize, all future loadings not exceeding the shakedown limit load are resisted in an elastic manner without producing further damage

The shakedown limit state of the bridge system shows significant additional capacity over the single-girder first-hinge methods. This reserve capacity is from (*a*) more realistic elastic distribution of forces, (*b*) bridge system redistribution of forces in the longitudinal and transverse directions, and (*c*) implementation of inelastic system limit states. However, as expected, this additional capacity is not uniform for all bridges. Therefore, even though the single-girder first-hinge methods may yield the same rating factor for two particular bridges, the ultimate system capacity of the bridges may be very different. Although both bridges may meet or exceed intended safety requirements, this leads to inconsistency in the ultimate safety of the structures.

OBJECTIVES

To achieve consistent levels of safety over the bridge inventory, consideration should be given to the ultimate capacity of the system or, stated relative to current rating methods, consideration must be given to the reserve capacity beyond the first hinge limit. This paper examines rating methods for two existing bridges comprising compact sections: a single-span composite bridge and a three-span composite bridge. The rating methods are the LFR maximum-strength operating level rating, the STRENGTH rating, and the single-girder and the system shakedown limit ratings using STRENGTH method load and resistance factors. The specific objectives are to

1. Compare LFR and STRENGTH single-girder first-hinge ratings to single-girder shakedown limit state ratings. These comparisons will illustrate the reserve capacity available in the longitudinal direction,
2. Compare LFR and STRENGTH single-girder first-hinge ratings to system shakedown limit state ratings. These comparisons will illustrate the system reserve capacity available in the longitudinal and transverse directions, and
3. Examine the relative reserve capacities between the single-span bridge and the three-span bridge. This will illustrate the inconsistency inherent in the current single-girder first-hinge rating methods when considering ultimate load capacities.

LFR AND STRENGTH RATING PROCEDURES

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

$$\Gamma_D D_n + (RF)\Gamma_L L_n(DF)(1 + I) = \Phi M_n \quad (1)$$

or, solving for the rating factor,

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

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.

Table 1 shows the respective factors for the LFR and STRENGTH rating equations. For the STRENGTH ratings, the factors are selected from site-specific load and resistance characteristics (*2*). Because both the LFR and STRENGTH methods are first hinge limits, the RF is determined by the critical first hinge section.

SHAKEDOWN PROCEDURES

As a moving load that exceeds the elastic limit but is less than the plastic-collapse limit crosses a statically indeterminate structure, increments of inelastic rotation occur at various sections in the structure. If this load is less than the incremental collapse load, the incremental inelastic rotations during each load pass decrease in magnitude and eventually vanish. After the incremental inelastic rotations vanish, all future loadings are resisted in an elastic manner.

In a statically indeterminate structure, inelastic behavior is characterized by internal residual moments that remain after removal of the load (similar to a support settlement). Thus, after loading and unloading, the structure contains internal forces and moments that are self-equilibrating. For this structure to resist the subsequent load elastically, the applied elastic dead and live load moments plus the internal residual moment must remain in the elastic range at each section. Using the assumption of an ideal elastic-plastic moment-curvature relationship (to represent work-hardening of the compact section) and including the rating factor, this criterion can be written:

$$(RF)M_L^+ + M_D + m_r = M_P^+ \quad \text{for positive moment, and} \quad (3a)$$

$$(RF)M_L^- + M_D + m_r = M_P^- \quad \text{for negative moment} \quad (3b)$$

everywhere in the structure, where

RF = rating factor,
$M_L^{+,-}$ = positive and negative live load moments,
M_D = dead load moment,
m_r = residual moment, and
$M_P^{+,-}$ = positive and negative moment capacity.

TABLE 1 Load and Resistance Factors for Rating Methods

RATING METHOD	Resistance Factor Φ	Dead Load Factor Γ_D	Live Load Factor Γ_L	Impact Factor I
LFR Operating	1.00	1.30	1.30	50/(L+125)≤0.3 based on span length (ft)
STRENGTH and Shakedown Limit	0.90 slight deterioration with vigorous maintenance	1.20 with 20% additional depth on the overlay thickness	1.30 low truck volume and effective weight enforcement	0.10 smooth riding surface

At the maximum shakedown limit, the left and right sides of Equation 3 are equal for the critical moment direction at the critical sections. Shakedown is a limit state controlled by variable repeated loads or moving loads; thus, shakedown is a major concern for bridges.

Shakedown Limit for Single-Girder Analyses

For the single-girder shakedown limit analyses, the lateral distribution factor is used to estimate the loads applied to the girder and the shakedown analysis assumes only longitudinal redistribution of forces and no lateral or transverse system interaction.

The shakedown limit upper-bound mechanism (*6*) method can be employed to find the shakedown limit state rating factor. The equation is based on virtual work of moments working through a mechanism motion. Using Figure 1, the mechanism equation can be written:

$$(RF)[M_L^1\theta^1 + M_L^2\theta^2 + M_L^3\theta^3] + [M_D^1\theta^1 + M_D^2\theta^2 + M_D^3\theta^3] = [M_P^1\theta^1 + M_P^2\theta^2 + M_P^3\theta^3] \qquad (4)$$

or, solving for the rating factor,

$$RF = \frac{[M_P^1\theta^1 + M_P^2\theta^2 + M_P^3\theta^3] - [M_D^1\theta^1 + M_D^2\theta^2 + M_D^3\theta^3]}{[M_L^1\theta^1 + M_L^2\theta^2 + M_L^3\theta^3]} \qquad (5)$$

where the mechanism rotations θ are shown in Figure 1. The controlling shakedown rating factor is the minimum calculated from assumed kinematically admissible incremental mechanisms.

The STRENGTH method load, resistance, and impact factors are used in this paper for the shakedown limit analyses. Substituting in the STRENGTH method factors, the governing rating factor equation becomes

$$RF = \frac{\Phi[M_n^1\theta^1 + M_n^2\theta^2 + M_n^3\theta^3] - \Gamma_D[D_n^1\theta^1 + D_n^2\theta^2 + D_n^3\theta^3]}{\Gamma_L[L_n^1\theta^1 + L_n^2\theta^2 + L_n^3\theta^3](DF)(1+I)} \qquad (6)$$

Shakedown Limit for System Analyses

Determining the shakedown limit of a single isolated girder entails finding the moment envelopes and solving Equation 6. In a bridge system, however, the longitudinal girders are no longer isolated and the interaction of the girders, slab, and transverse diaphragms make up a complicated load-resisting system. The bridge system shakedown limit state model is derived from the shakedown requirements, the bridge system elastic behavior, and global equilibrium equations (*5*). To develop the method, the bridge is assumed to have a global incremental collapse mechanism similar to that shown in Figure 1. For two-lane bridges under typical truck loading, Grundy (*9*) shows that a global mechanism controls for the collapse limit state. Only in a wide multilane bridge would one expect a local collapse mechanism.

The system shakedown limit equation involves condensing the system's elastic response and resistance into a global kinematic incremental collapse mechanism. Across a critical global bridge section, each girder must reach the Equation 3 shakedown condition. Summing the individual requirements across the transverse section:

For positive moment:

$$(RF)\sum_i M_L^+ + \sum_i M_D^+ + \sum_i m_r = \sum_i M_P^+ \qquad (7a)$$

For negative moment:

$$(RF)\sum_i M_L^- + \sum_i M_D^- + \sum_i m_r = \sum_i M_P^- \qquad (7b)$$

where Σ is the summing of the various moment quantities across the transverse section of the bridge.

In Equation 7, the individual girder residual moment fields, m_r, adjust to attain individual girder shakedown, but they are not necessarily in equilibrium in the individual girder sense. Transverse residual forces change the single-girder equilibrium requirements to meet system equilibrium. However, the system residual moment field, Σm_r, must still be in equilibrium with no applied loads.

The shakedown limit upper-bound mechanism method can again be employed to find the shakedown limit state rating factor of the entire bridge system. Using Figure 1, the mechanism equation can be written as follows:

$$(RF)\sum_i [M_L^1\theta^1 + M_L^2\theta^2 + M_L^3\theta^3] + \sum_i [M_D^1\theta^1 + M_D^2\theta^2 + M_D^3\theta^3] = \sum_i [M_P^1\theta^1 + M_P^2\theta^2 + M_P^3\theta^3] \qquad (8)$$

or, solving for the rating factor and substituting in the STRENGTH method factors as was done for Equation 6:

$$RF = \frac{\Phi\sum_i [M_n^1\theta^1 + M_n^2\theta^2 + M_n^3\theta^3] - \Gamma_D\sum_i [D_n^1\theta^1 + D_n^2\theta^2 + D_n^3\theta^3]}{\Gamma_L\sum_i [L_n^1\theta^1 + L_n^2\theta^2 + L_n^3\theta^3](1+I)} \qquad (9)$$

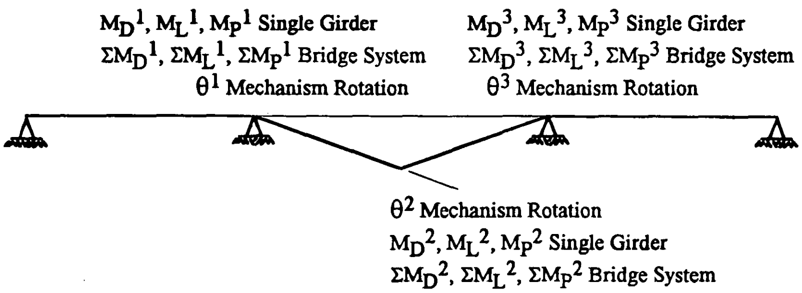

FIGURE 1 Single-girder and system shakedown mechanism.

where the global mechanism rotations, θ, are illustrated in Figure 1. Because the system analysis calculates the elastic live load moment in each girder, the equation no longer contains the lateral distribution factor. Equation 9 illustrates that the system shakedown limit state can be reduced to an equivalent single-girder analysis where the equivalent elastic moment envelope is the summation of all the individual girder elastic envelopes across the bridge section. Likewise, the equivalent dead load and resistance moments are the respective sums of the individual moments across the section. The controlling shakedown rating factor is the minimum calculated from assumed global kinematically admissible incremental mechanisms.

BRIDGE RATING EXAMPLES

Following are LFR maximum-strength operating, STRENGTH, single-girder shakedown limit, and system shakedown limit ratings for two example bridges (*10*). The first is a single-span composite bridge with a 13.4-m (44-ft) span, five girders spaced at 2.24 m (7.33 ft), and a 160-mm (6.25-in.) structural concrete deck with a 28-day compressive strength of 27.6 MPa (4,000 psi). The interior girders are W24 × 84 and the exterior girders are W24 × 76, both of 248-MPa (36-ksi) material. The second is a three-span continuous composite bridge with spans of 12.5, 16.2, and 12.5 m (41, 53, and 41 ft), five girders spaced at 2.24 m (7.33 ft), and a 171-mm (6.75-in.) structural concrete deck with a 28-day compressive strength of 27.6 MPa (4,000 psi). All the girders are W24 × 68 made of 248 MPa (36 ksi) material. The reinforcing steel over the interior pier supports is assumed to act compositely with the steel sections for both bridges. The bridges were selected to be similar in construction except for the longitudinal redundancy (three-span versus simple-span). Because the objective of this paper is to compare relative capacity ratings, for brevity, detailed specifics are not presented.

The AASHTO Type 3 rating vehicle is used because it yields the lowest rating factors for both bridges. For the STRENGTH method and the shakedown limit procedures, the factors are assumed as follows (Table 1): $\Phi = 0.90$, $\Gamma_D = 1.2$ with an increase of 20 percent on the overlay thickness, $\Gamma_L = 1.3$, and $I = 0.10$. The AASHTO S/D lateral distribution factors are used for all the single-girder analyses.

A grillage analysis routine developed specifically to analyze bridge systems for first hinge and shakedown limit states (*11*) is used for the single-girder and system dead load and live load force effect analyses. A post processor uses the results of the analysis to apply the rating equations (Equations 2, 6, and 9). The grillage program discretizes each span into tenth points and assumes admissible mechanism-positive rotations at each tenth point of each span.

Single-Span Bridge Example

The simple-span bridge has an LFR impact factor of 0.296, a lateral distribution factor of 1.257 (exterior girder controls for single-girder analyses), and a uniform dead load of 12.3 kN/m (0.84 kips/ft) of which 1.0 kN/m (0.07 kips/ft) is attributed to the wearing surface. Therefore, for the ratings using the STRENGTH factors, the nominal dead load moments are multiplied by a factor of $(11.3 + 1.2 * 1)/12.3 = 1.02$ to adjust for the 20 percent surface thickness increase.

Figure 2 illustrates the nominal dead and live load moments for the span. The single-girder LFR operating rating and STRENGTH rating from Equation 2 and Table 1 are:

$$RF = \frac{1605 - 1.3(275)}{1.30(268)1.257(1 + 0.296)}$$

$$= 2.20 \qquad \text{(LFR operating rating), and}$$

$$RF = \frac{0.90(1605) - 1.2(1.02 * 275)}{1.30(268)1.257(1 + 0.10)}$$

$$= 2.30 \qquad \text{(STRENGTH Rating)}$$

The STRENGTH rating is 4.6 percent higher than the LFR operating rating. This is expected because the STRENGTH factors represent a bridge that has good load and resistance characteristics. Barker et al. (*10*) compare the LFR and STRENGTH method on a data base of existing bridges elsewhere, and the results are not repeated here. Both methods are included here for reference purposes.

The single-girder shakedown limit is identical to a first hinge limit because there is no redundancy in the girder. This can be shown by Equation 5:

$$RF = \frac{0.90[0 + 1605(2\theta) + 0] - 1.2(1.02)[0 + 275(2\theta) + 0]}{1.3[0 + 268(2\theta) + 0](1.257)(1 + 0.10)}$$

$$= 2.30 \qquad \text{(single-girder shakedown rating)}$$

As can be seen, a free hinge has no moment resistance or applied moment and, therefore, does not resist load or cause work in the equation. Because a simple-span isolated girder is not redundant, the first hinge is a failure and, thus, the rating is identical to the STRENGTH rating.

A simple-span bridge system, however, is redundant. A single hinge in one girder does not cause failure because the system has the capacity to redistribute forces in the transverse direction. Using the equivalent girder global moment summations in Figure 2 and Equation 9, the shakedown limit capacity of the system is

$$RF = \frac{0.90[0 + 8522(2\theta) + 0] - 1.2(1.02)[0 + 1326(2\theta) + 0]}{1.3[0 + 1491(2\theta) + 0](1 + 0.10)}$$

$$= 2.84 \qquad \text{(system shakedown rating)}$$

The one-span bridge system contains redundancy in the transverse direction. As shown by the system shakedown rating factor, the system has additional capacity beyond the first hinge.

Three-Span Bridge Example

The three-span bridge has LFR impact factors of 0.30 in the outer two spans, 0.29 over the interior pier supports, and 0.28 in the center span. The lateral distribution factor is 1.333 (interior girder controls for single-girder analyses) and a uniform dead load of 14 kN/m (0.96 kips/ft) of which 1.2 kN/m (0.08 kips/ft) is attributed to the wearing surface. Therefore, for the ratings using the STRENGTH factors, the nominal dead load moments are multiplied by a factor of $(12.8 + 1.2 * 1.2)/14 = 1.02$ to adjust for the 20 percent wearing surface thickness increase.

Figure 3 illustrates the nominal dead and live load moments for the span. The controlling first hinge location for the LFR operating rating and the STRENGTH rating is at the centerline of the center span. From Equation 2 and Table 1, the ratings are as follows:

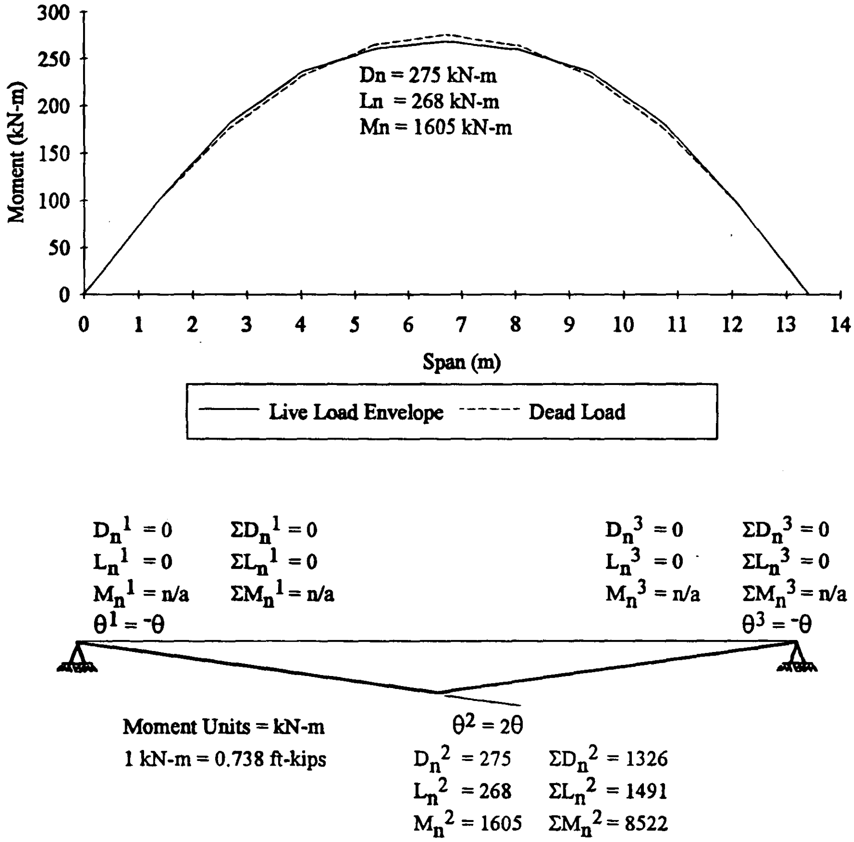

FIGURE 2 Simple-span nominal moments and incremental collapse mechanism.

$$RF = \frac{1483 - 1.3(187)}{1.30(221)1.333(1 + 0.28)}$$

$$= 2.53 \qquad \text{(LFR operating rating), and}$$

$$RF = \frac{0.90(1483) - 1.2(1.02 * 187)}{1.30(221)1.333(1 + 0.10)}$$

$$= 2.63 \qquad \text{(STRENGTH Rating)}$$

Again the STRENGTH rating is slightly higher (4 percent) than the LFR operating rating.

Unlike a simple-span bridge, a three-span single-girder bridge has redundancy in the longitudinal direction and, therefore, has reserve capacity beyond the first hinge. For this example, the critical incremental collapse mechanism is in the center span as shown in Figure 3. According to Equation 5, the single-girder shakedown limit rating is

$$RF = \{0.90[1466\theta + 1483(2\theta) + 1466\theta] - 1.2(1.02)[270\theta + 187(2\theta) + 270\theta]\}/\{1.3[129\theta + 221(2\theta) + 129\theta](1.333) \times (1 + 0.10)\} = 3.14 \text{ (single-girder shakedown rating)}$$

For the redundant center span, all three hinges in the mechanism do work. The single-girder shakedown limit is considerably higher (19.4 percent) than the STRENGTH method first hinge rating. However, the structure still has the capacity to redistribute forces in the transverse direction. According to Equation 9 and the same critical global incremental collapse mechanism shown in Figure 3, the system shakedown limit is

$$RF = \{0.90[7446\theta + 7416(2\theta) + 7446\theta] - 1.2(1.02)[1253\theta + 929(2\theta) + 1353\theta]\}/\{1.3[754\theta + 1273(2\theta) + 754\theta] \times (1 + 0.10)\} = 3.65 \text{ (system shakedown rating)}$$

The three-span bridge system is a highly redundant system and, as shown by the system shakedown rating factor, it has a large reserve capacity over the first hinge.

COMPARISON OF RATING METHODS

Table 2 shows the ratings for the different methods. For both bridges, the STRENGTH ratings exceed the LFR operating ratings by a few percent. For the STRENGTH factors chosen, this is typical. If the bridges showed significant deterioration or higher truck volume, the results would be reversed (*10*).

Because the shakedown rating analyses use the STRENGTH method load and resistance factors, only comparisons between these methods are presented. The one-span single-girder shakedown rating is the same as the STRENGTH first-hinge rating. The simple-span bridge has no redundancy to redistribute forces longitudinally after this hinging. However, the three-span single-girder bridge has longitudinal redundancy at the negative pier regions. This redistribution capacity results in a load capacity increase of 19.4 percent

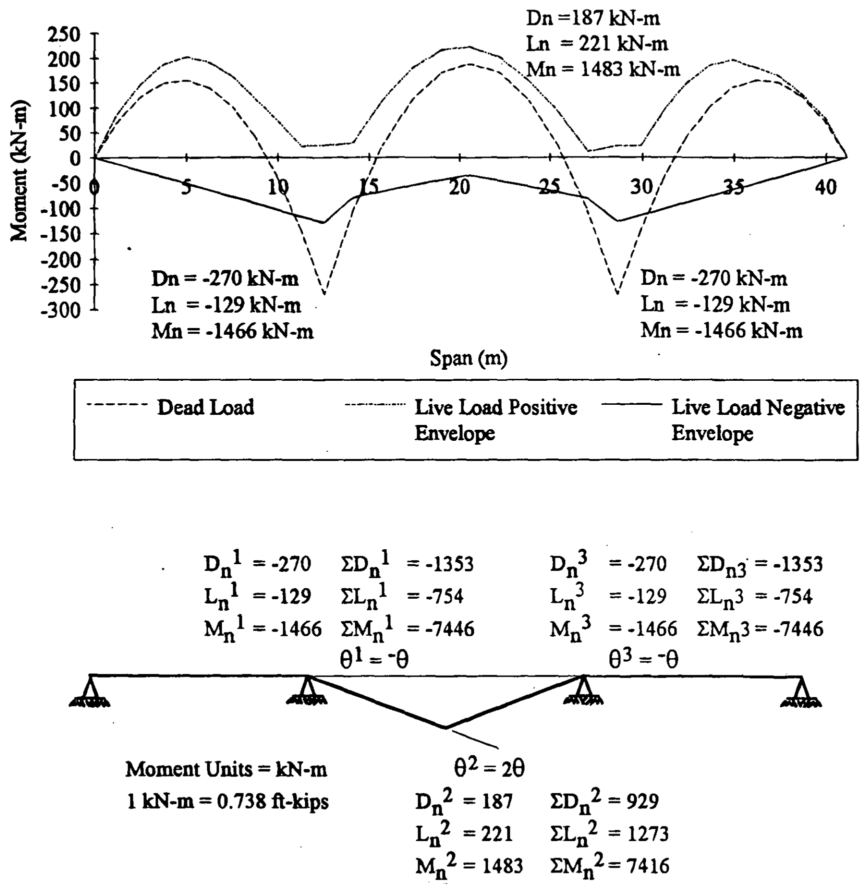

FIGURE 3 Three-span nominal moments and incremental collapse mechanism.

over the STRENGTH rating. Therefore, for single-girder analyses, although there is no additional capacity over the first hinge for simple-span bridges, there is significant reserve capacity for multi-span bridges.

The system shakedown limit considers both longitudinal and transverse redundancy in the structure. The results are as follows: the single-span system shakedown rating is 23.5 percent larger than the first-hinge STRENGTH rating and the three-span system shakedown rating is 38.8 percent larger than the first-hinge STRENGTH rating. The STRENGTH method infers consistent safety for both bridges for a first hinge limit. However, Table 2 shows that the safety is no longer consistent when examining the ultimate shakedown limit rating.

Both bridges show a significant load capacity increase caused by transverse redundancy: 23.5 percent for the simple-span bridge and 16.2 percent for the three-span bridge as illustrated by the percent increase of the system shakedown rating over the single-girder shakedown rating. This value represents the reserve capacity inher-

TABLE 2 Summary of Ratings

Rating Method	One-Span Bridge Example	Three-Span Bridge Example
LFR Operating Rating	2.20	2.53
STRENGTH Rating	2.30	2.63
Single-Girder Shakedown Rating	2.30	3.14
System Shakedown Rating	2.84	3.65
Single Girder Shakedown % Increase Over STRENGTH	0%	19.4%
System Shakedown % Increase Over STRENGTH	23.5%	38.8%
System Shakedown % Increase Over Single-Girder Shakedown	23.5%	16.2%

ent in any multigirder structure. Therefore, if this reserve capacity is similar for all multigirder bridges, it does not reflect inconsistency in the ultimate safety between the bridges.

The ultimate safety difference stems from the longitudinal redistribution characteristics. This is directly illustrated by the single-girder shakedown rating increase over the STRENGTH ratings. The three-span girder shows 19.4 percent additional strength and the one-span girder has no additional strength.

SUMMARY AND CONCLUSIONS

Multigirder steel bridges are highly redundant structures and show a large reserve capacity in the inelastic range over the capacity calculated from first hinge limit methods. To achieve consistent levels of safety over the bridge inventory, consideration should be given to the ultimate capacity of the system. This paper examines rating methods for a single-span and a three-span bridge. The rating methods are the LFR maximum strength operating level rating, the STRENGTH rating, and the single-girder and the system shakedown limit ratings. The following are conclusions from this work.

1. The three-span single-girder shakedown rating is 19.4 percent higher than the first-hinge STRENGTH method rating. This is because of longitudinal redundancy. There is no increase for the one-span single-girder ratings because the girder is not redundant. The current LFR and STRENGTH methods do account for some longitudinal redistribution by allowing a 10 percent redistribution of negative pier moments for bridges comprising compact sections. However, this adjustment is arbitrary and does not apply to this three-span bridge because the positive moment region is critical for the first hinge rating.
2. The system shakedown ratings were significantly higher than the STRENGTH first hinge ratings. The additional capacity can be divided between longitudinal and lateral redundancy. The lateral redundancy shows a somewhat uniform increase for the two bridges and is not responsible for the higher overall increase for the three-span bridge. This larger increase is a result of longitudinal redistribution as discussed earlier. However, it is important to note that even the simple-span bridge shows a 23.5 percent reserve capacity over that of the first-hinge rating.
3. The important rating comparison for examining the consistency of rating methods is the additional load capacity caused by the longitudinal redistribution. This is because the transverse redistribution is nearly uniform for the two bridges, thus making this contribution irrelevant for comparing reserve capacities at ultimate limits. However, the increase as a result of longitudinal redundancy is pronounced with the three-span bridge and nonexistent for the one-span bridge. This illustrates inconsistency in ultimate capacity when using single-hinge limit rating methods.

FUTURE WORK

To consider the ultimate load capacity of bridges in a rating procedure, either inelastic limit state procedures must be standardized or a longitudinal redundancy adjustment must be incorporated. In addition, this paper presents results for bridges comprising compact sections. To encompass all types of bridges, the redistribution characteristics for bridges comprising noncompact sections need to be investigated. To study these topics, a current research project (*12*) is testing three large-scale composite bridge girders: 1 three-span girder with compact sections and 2 two-span girders with noncompact sections. One major objective of this work is to develop comprehensive inelastic design and rating procedures for steel-girder bridges. In addition, this work assumes a global incremental collapse mechanism. Transverse contributions in a local mechanism may occur in wider bridges.

REFERENCES

1. *Manual for Maintenance Inspection of Bridges.* AASHTO, Washington, D.C., 1983.
2. *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges.* AASHTO, Washington, D.C., 1989.
3. Manual for Condition Evaluation of Bridges, AASHTO, Washington, D.C., 1994.
4. 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.
5. Moses, F., and D. Verma. *NCHRP Report 301: Load Capacity Evaluation of Existing Bridges,* TRB, National Research Council, Washington, D.C., 1987.
6. Barker, M. G., and T. V. Galambos. Shakedown Limit State of Compact Steel Girder Bridges. *Journal of Structural Engineering,* ASCE, Vol. 118, No. 4, April 1992, pp. 986–998.
7. Galambos, T. V., R. T, Leon, C. E. French, M. G. Barker, and B. E. Dishongh. *NCHRP Report 352: Inelastic Rating Procedures for Steel Beam and Girder Bridges.* TRB, National Research Council, Washington, D.C., 1993.
8. Neal, B. G. *The Plastic Methods of Structural Analysis,* John Wiley & Sons, New York, 1956.
9. Grundy, P. Shakedown Design of Bridges. Presented at 1st National Structural Engineering Conference, Institute of Engineering, Australia, Aug. 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. Barker, M. G. *The Shakedown Limit State of Slab-On-Girder Bridges.* Thesis, University of Minnesota, Minneapolis, June 1990.
12. Barker, M. G. *Development and Experimental Verification of Inelastic Design Procedures for Steel Bridges Comprising Noncompact Sections.* National Science Foundation, Washington, D.C., 1993–1996.

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