# Redundancy in Highway Bridge Superstructures

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A framework for considering redundancy in the design and load capacity evaluation of highway bridge superstructures is proposed. Redundancy is defined in terms of the capacity of the bridge system to resist failure at high loads and to resist system serviceability distress compared with its capacity to resist first member failure. The consequences of damage of one member to the overall system capacity is also checked. The proposed framework consists of tables of load modifiers that can be used to assess the redundancy level of typical bridge configurations. The load modifiers are used during the design process to require that members of less redundant configurations be more conservatively designed than is allowed by current standards. On the other hand, highly redundant designs are rewarded by permitting less conservative member designs. For bridges with nontypical configurations that are not covered by the tables, a direct analysis approach is recommended. General guidelines explaining how to perform such an analysis are provided. These include the loads that should be applied, the limit states that should be checked for both intact and damaged conditions, and the target load factors that the bridge should sustain before these limit states are violated. System factors that provide a measure of the system's redundancy can be calculated from the results of the incremental analysis. The load modifiers obtained from the tables and the system factors obtained from the incremental analysis can be used for the design of new bridges or they can be used to calculate rating factors for the evaluation of existing bridges. Thus, bridges with lower levels of redundancy must have their member capacities increased or they will have lower ratings. An example illustrating the proposed procedures is provided.

Bridge redundancy as normally defined consists of the capability of a bridge to continue to carry loads after the damage or the failure of one or more of its members. Member failure can be either ductile or brittle. It could be caused by the application of large live loads or the sudden loss of one element as a result of brittle fracture or an accident such as collision of trucks, ships, or debris.

The framework proposed in this paper for implementing redundancy concepts in the design and evaluation of highway bridges consists of two parts. The first part presents tables of load modifiers that would be used to modify component strengths on the basis of the redundancy of bridge systems with typical geometric configurations. The second part presents guidelines for the redundancy analysis and evaluation of any bridge system using a nonlinear structural anlaysis program. This paper uses a system reliability model to calibrate the proposed load modifiers and provide the analysis guidelines.

## Bridge Safety

Although current bridge design and evaluation methods (*1*) have been successfully used for years, these are generally member-oriented procedures that do not provide adequate representations of the safety of the complete bridge system. In many instances, the failure of an individual member does not lead to the failure of the complete bridge system. On the other hand, because of possible large deformations, the bridge may be inadequate for truck traffic at loads that are lower than those that will cause a system failure.

Bridge members are often subjected to fatigue stresses that may lead to fracture and the loss of the load-carrying capacity of a main member. In addition, corrosion, fire, or an accident such as a collision by a truck, ship, or debris could cause the loss of a bridge member or the severing of the prestressing strands. To ensure the safety of the public, bridges should be able to sustain these damages and still operate, albeit at reduced capacity. Therefore, in addition to verifying the safety of the intact structure, the evaluation of a bridge's safety and redundancy should consider the consequences of the failure of critical bridge members.

In summary, a bridge should (*a*) provide a reasonable safety level against first member failure, (*b*) provide an adequate level of safety before it reaches its ultimate system capacity under extreme loading conditions, (*c*) remain functional under regular (or recurrent) traffic loads, and (d) be able to carry some traffic loads after damage or the loss of a component. These four critical limit states are described as follows (for convenient representation, a load model consisting of two AASHTO HS-20 vehicles is assumed to be acting on the structure).

### Member Failure

The capacity of the structure to resist first member failure, as defined herein, is expressed in terms of the number of AASHTO HS-20 trucks that it can carry before this first member failure limit state is violated. This HS-20 load multiplier will be referred to as $LF_1$. For two-lane bridges, $LF_1$ can be calculated by applying the dead loads and two AASHTO HS-20 vehicles using a linear elastic structural model of the bridge and then incrementing the loads until first member failure occurs.

### Ultimate Capacity

The ultimate capacity limit is defined as the maximum possible truck load that can be applied on the bridge before it collapses. The load factor (HS-20 load multiplier) corresponding to the ultimate limit state will be referred to as $LF_u$. $LF_u$ can be calculated by analyzing the bridge under the effect of the dead loads and two AASHTO HS-20 vehicles using a nonlinear structural model of the bridge and then incrementing the truck loads until the system collapses. Collapse is herein defined as the load level at which a mechanism forms or at which concrete bridge members begin to crush.

### System Serviceability Conditions

In a study of system serviceability conditions a maximum live load displacement limit of span length/200 is used as a system serviceability limit state. This displacement limit is based on best engineering judgment and is compatible with displacement limits used by other researchers (*2*). The capacity of a structure to withstand the maximum displacement limit can be expressed in terms of the number of a pair of AASHTO HS-20 trucks that can be placed on the structure before this system serviceability limit state is reached. $LF_s$ is defined as the load multiplier that will cause the violation of the serviceability limit state accounting for the nonlinear behavior of the bridge members. Because redundancy is concerned with the performance of the structure, the displacements are checked in the main members only. The displacements of the slab or secondary members are not checked for this serviceability limit state.

### Damaged Conditions

The damaged bridge condition consists of the removal from the structural model of a main load-carrying component that might be subject to brittle fracture or to accidental loss of capacity because of collisions or other causes. The load multiplier corresponding to the ultimate capacity of the damaged structure is defined as $LF_d$. $LF_d$ can be calculated by analyzing the damaged structure under the effect of the dead loads and two AASHTO HS-20 vehicles on a nonlinear structural model of the bridge and then incrementing the truck loads until the structural system collapses.

Two-lane bridges using the HS-20 load model are used as the basis of the calibration performed in this study. This is based on the observation that maximum lifetime load effects are dominated by the presence of two heavy trucks side-by-side on a bridge and on the observation that two-lane loads produce the most critical loading condition in the linear elastic range for many bridge configurations (*3*). The final results obtained in this study are generalized to be applicable for any number of lanes and any truck load model by requiring that one-lane bridges and bridges with three or more lanes as well as bridges designed with other than the HS-20 load model must satisfy the same safety criteria derived herein for two-lane bridges.

## Redundancy Measures

Redundancy is defined as the capability of a structure to continue to carry loads after the failure of one or more of its members; in particular it should continue to carry load after the failure of a main member. The failure of a main member is thus used herein as the basis of the proposed measures of redundancy. Therefore, a comparison between $LF_u$, $LF_s$, $LF_d$, and $LF_1$ would provide a measure of the level of bridge redundancy. The system reserve ratios for the ultimate limit state $R_u$, for the serviceability limit state $R_s$, and for the damaged condition $R_d$ are defined as follows:

$$R_u = \frac{LF_u}{LF_1}$$

$$R_s = \frac{LF_s}{LF_1}$$

$$R_d = \frac{LF_d}{LF_1} \tag{1}$$

The system reserve ratios $R_u$, $R_s$, and $R_d$ are nominal (deterministic) measures of bridge redundancy. For example, when the ratio $R_u$ is equal to 1.0 ($LF_u = LF_1$), the ultimate capacity of the bridge system is equal to the capacity of the bridge to resist failure of its most critical member. Such a bridge is considered nonredundant. As $R_u$ increases, the level of bridge redundancy increases. Similar observations can be made about $R_s$ and $F_d$. These two ratios, however, may under certain circumstances have values lower than 1.0. A value of $R_s$ less than 1.0 means that the bridge will exhibit a deformation equal to span length/200 at a load level smaller than the load that will cause the first member failure. This situation might occur in certain bridges because $LF_1$ is calculated using a linear elastic model while $LF_s$ accounts for the nonlinear behavior of the bridge. Similarly, $R_d$ less than 1.0 means that a damaged bridge will be able to carry less live load than the load that will cause the first member failure in the intact structure.

To check whether a bridge system has adequate levels of redundancy it is sufficient to use a structural analysis program to calculate $LF_u$, $LF_s$, $LF_d$, and $LF_1$ and to verify that $R_u$, $R_s$, and $R_d$ are adequate. Minimum acceptable values of $R_u$, $R_s$, and $R_d$ should be established by examining the results of bridges that are clearly redundant. In addition, these minimum acceptable values should account for the uncertainties associated with determining the loads and the resistances of bridge superstructures. Minimum acceptable values of $R_u$, $R_s$, and $R_d$ are determined in this study using a sysem reliability model similar to that used in development of the AASHTO Load and Resistance Factor Design (LRFD) specifications (*4*).

## Reliability Model

The safety index $\beta_{\text{member}}$ for the failure of the first member is expressed herein using the following lognormal format:

$$\beta_{\text{member}} = \frac{ln\left(\dfrac{\overline{LF_1}}{\overline{LL_{75}}}\right)}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{2}$$

where $\overline{LF_1}$ is the mean value of the load factor that will cause the first member failure in the bridge assuming elastic analysis. $\overline{LF_1}$, which is the mean value of $LF_1$, is related to the nominal value by a bias $\lambda_{LF}$. $LF_1$ is a function of the strength capacity of the member represented by the nominal resistance, $R$, and the nominal dead load, $D$. $\overline{LL_{75}}$ is the mean value of the maximum expected lifetime live load including impact. The same HS-20 load model is used to express $LF_1$ and $\overline{LL_{75}}$. A 75-year lifetime is used on the basis of work done elsewhere (*3*). $V_{LF}$ is the coefficient of variation of $LF_1$, whereas $V_{LL}$ is the coefficient of variation of the maximum expected live load $LL_{75}$. The denominator in Equation 2 gives an overall measure of the uncertainty in estimating the resistance, the dead load, and the live load including dynamic impact.

The safety index of the system for the ultimate limit state is defined herein with respect to the extreme loading condition as

$$\beta_{\text{ult}} = \frac{ln\left(\dfrac{\overline{LF_u}}{\overline{LL_{75}}}\right)}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{3}$$

where $\overline{LF_u}$ is the mean value of the load factor corresponding to the ultimate limit state. $LF_u$ relates to the strength capacity of the system and the dead load. $\overline{LL_{75}}$ and $V_{LL}$ are the same values used to calculate $\beta_{\text{member}}$. A 75-year exposure period is also used herein for the ultimate limit state. Because of insufficient data, it is assumed that $LF_u$, $LF_s$, and $LF_d$ have the same bias value and the same coefficients of variation used for $LF_1$. The statistical data base used in this study is provided elsewhere (*5*).

For the serviceability limit state, the performance of the complete system can also be measured in terms of a system serviceability safety index, $\beta_{serv.}$, defined as

$$\beta_{serv.} = \frac{ln\left(\frac{\overline{LF_s}}{\overline{LL_2}}\right)}{\sqrt{V_{LF}^2 + V_{LL}^2}} \quad (4)$$

where $\overline{LF_s}$ is the mean load factor to reach the serviceability limit state. $\overline{LF_s}$ relates to the capacity of the bridge system to resist large deformations and the applied dead load. $\overline{LL_2}$ is the mean applied live load for a 2-year exposure period expressed in terms of the HS-20 load model. The 2-year exposure period is used herein to reflect the fact that if a bridge has serviceability problems, these will be noticed during the biennial mandatory bridge inspection period.

Finally, the system's ability to sustain loads after damage can be expressed in terms of a system safety index for damaged conditions, $\beta_{damaged}$, defined as

$$\beta_{damaged} = \frac{ln\left(\frac{\overline{LF_d}}{\overline{LL_2}}\right)}{\sqrt{V_{LF}^2 + V_{LL}^2}} \quad (5)$$

$\overline{LF_d}$ is the mean load factor to reach the ultimate capacity of the damaged system. $\overline{LF_d}$ relates to the residual capacity of the system after one member is damaged and the dead load. A 2-year exposure period is also used for the damaged conditions as with the serviceability conditions.

Redundancy is defined as the capability of a bridge system to continue to carry load after the damage or the failure of one or more of its members. Hence, to study the redundancy of a system, it is useful to examine the difference between the safety indexes of the system expressed in terms of $\beta_{ult.}$, $\beta_{serv.}$ and $\beta_{damaged}$ and the safety index of the most critical member, expressed in terms of $\beta_{member}$. The relative safety indexes are defined as follows:

$$\Delta\beta_u = \beta_{ult.} - \beta_{member}$$
$$\Delta\beta_s = \beta_{serv.} - \beta_{member}$$
$$\Delta\beta_d = \beta_{damaged} - \beta_{member} \quad (6)$$

These relative safety indexes give measures of the relative safety provided by the bridge system compared with the nominal safety of first-member failure. The relative safety indexes provide reliability-based measures of redundancy. Thus, a bridge system will provide adequate levels of system redundancy if the relative safety indexes are adequate.

## Determination of Target Safety Indexes

The object of this study is to calibrate a set of load modifiers that can be used with the typical design equations to account for the redundancy of typical bridge superstructures. In addition, this study calibrates a step-by-step procedure to check the redundancy of nontypical structures using a nonlinear finite element analysis. To perform the calibration of the load modifiers and the step-by-step procedures, minimum target $\Delta\beta_u$, $\Delta\beta_s$, and $\Delta\beta_d$ values that a bridge should satisfy must be obtained. In this study, these target values are extracted on the basis of a review of the performance of existing redundant designs.

To perform the reliability calibration, a large number of common-type multigirder steel reinforced concrete T-beam and prestressed concrete I-beam bridges were analyzed. Values of the load factors $LF_1$, $LF_u$, $LF_s$, and $LF_d$, the safety indexes $\beta_{member}$, $\beta_{ult.}$, $\beta_{serv.}$, and $\beta_{damaged}$ as well as $\Delta\beta_u$, $\Delta\beta_s$ and $\Delta\beta_d$ were calculated for each bridge configuration. An earlier work (*5*) gives detailed descriptions of the bridges analyzed and the results obtained. The extraction of the target relative safety indexes is performed for the ultimate limit state, serviceability limit state, and the damaged condition on the basis of bridge designs that are known to be redundant. The conclusions obtained for the typical bridge configurations that were studied revealed that a bridge will provide adequate levels of redundancy if all the following conditions are satisfied:

1. It gives a value of $\Delta\beta_u$ greater than or equal to 1.0;
2. It gives a value of $\Delta\beta_s$ greater than or equal to $-1.0$; and
3. It gives a value of $\Delta\beta_d$ greater than or equal to $-0.5$.

## Load Modifiers for Bridge Design

An earlier work (*4*) defines a load modifier $\eta$ as a "factor relating to the ductility, redundancy and the operational importance of a bridge." Many formats can be used to apply the load modifier in the LRFD design check equation. Because redundancy as defined in this study relates to the load factors LF, which are a function of the live load margin (R-D), it is proposed to apply the load modifier factor on the live load of the LRFD equation such that the design equation becomes the following:

$$\phi R'_{req.} = \gamma_d D_n + \eta\gamma_l L_n(1 + I) \quad (7)$$

where

$\eta$ = live load modifier relating to bridge redundancy;
$\phi$ = resistance factor;
$\gamma_d$ = dead load factor;
$\gamma_l$ = live load factor;
$R'_{\text{req.}}$ = required member capacity accounting for the bridge system's redundancy;
$D_n$ = nominal or design dead load; and
$L_n = (1 + I)$ is the nominal or design live load including the dynamic impact factor ($I$).

Equation 7 has a general format that can be used for any AASHTO criteria. For example, for working stress design (WSD) criteria, $\gamma_d$ and $\gamma_l$ are given as 1.0 and $\phi$ is equal to 1/0.55. For the LFD criteria, $\phi$ will depend on the type of member being analyzed, $\gamma_d$ is equal to 1.3, and $\gamma_l$ is 2.17. For the LRFD criteria, $\phi$ will depend on the type of member being analyzed, $\gamma_d$ is equal to 1.25 and $\gamma_l$ is 1.75.

When $\eta$ is equal to 1.0 Equation 7 becomes the regular design check equation used in current member-oriented practice. A value of $\eta$ greater than 1.0 indicates that the bridge structure is not adequately redundant, and thus this bridge's members are penalized by requiring higher member capacities than are currently permitted. On the other hand, a value of $\eta$ less than 1.0 indicates that the bridge is sufficiently redundant and that its members' capacities can be reduced without jeopardizing overall system safety.

Because redundancy is related to maximum system capacity, Equation 7, including the load modifier $\eta$, should be applied only when checking the strength limit state of bridge components. The equations for member serviceability limit states should not include $\eta$.

The derivation of $\eta$ values for typical bridge configurations was performed in this study such that typical bridge configurations satisfy the target safety index values determined in the previous section. Values of the load modifier $\eta$ for typical simply supported prestressed concrete I-beam bridge configurations with identical parallel beams are given in Tables 1 through 3 as a function of the number of beams and the beam spacings. The values in the tables are given for typical simply supported bridges with 120-ft span lengths. Corrections are specified in the tables when the load modifier is influenced by changes in span length. Ghosn and Moses (5) also provide additional tables for multigirder steel bridges and concrete T-beam bridges. They are applicable for any type of specifications (WSD, LFD, or LRFD) with any number of lanes and any load model. The tables are given separately for each system limit state. The final value that should be used is the maximum value obtained from the three limit states. A minimum value of 0.75 is herein recommended as a conservative lower bound on the load modifier.

Bridges that have load modifiers greater than 1.0 do not provide sufficient levels of redundancy. These should be strengthened by increasing their required member capacity using Equation 7. Existing bridges that cannot be strengthened should be assigned lower rating factors.

Equation 7 provides one possible format to include a load modifier (or redundancy) factor in the design equation. The possibility of using other formats will be investigated in future phases of this study.

**TABLE 1 Load Modifiers for Prestressed Concrete I-Beam Bridges for Ultimate Limit State**

| | Load Modifier by Beam Spacing (ft) | | |
|---|---|---|---|
| No. of Beams | 4 | 6 | 8 |
| 4 | 1.20 | 1.15 | 1.05 |
| 6 | 1.00 | 0.80 | 0.65[a] |
| 8 | 0.80 | 0.65[b] | |

[a]For bridges less than 120 ft, increase the load modifier shown by 0.05 for every 10 ft. The final load modifier should not be less than 0.75.
[b]For bridges less than 120 ft, increase the load modifier shown by 0.05 for every 10 ft.

**TABLE 2 Load Modifiers for Prestressed Concrete I-Beam Bridges for System Serviceability Limit State**

| | Load Modifier by Beam Spacing (ft) | | |
|---|---|---|---|
| No. of Beams | 4 | 6 | 8 |
| 4 | 0.95[a] | 0.95[b] | 0.85[a] |
| 6 | 0.85[a] | 0.75 | 0.75 |
| 8 | 0.75 | 0.75 | |

[a]For bridges less than 120 ft, decrease the load modifier shown by 0.05 for every 10 ft.
[b]For bridges less than 120 ft, decrease the load modifier shown by 0.10 for every 10 ft. The final load modifier should not be less than 0.75.

**TABLE 3 Load Modifiers for Prestressed Concrete I-Beam Bridges for Damaged Conditions**

| | Load Modifier by Beam Spacing (ft) | | |
|---|---|---|---|
| No. of Beams | 4 | 6 | 8 |
| 4 | 1.05 | 0.95 | 0.90 |
| 6 | 0.90 | 0.75 | 0.75[a] |
| 8 | 0.75 | 0.75 | |

[a]For bridges less than 50 ft long, use a load modifier equal to 0.85.

## Direct Analysis Approach

Given the target safety indexes determined above and Equations 1 through 6, work by Ghosn and Moses (5) illustrates how the values of the system reserve ratios, $R_{u\ \mathrm{req.}}$, $R_{s\ \mathrm{req.}}$, and $R_{d\ \mathrm{req.}}$, that are required to satisfy a minimum level of bridge redundancy are calculated. These required values are summarized in Table 4. A particular bridge system will provide adequate levels of redundancy if the values of $R_u$, $R_s$, and $R_d$ calculated for that bridge are higher than the required values given herein.

Therefore, to verify the redundancy level of a bridge with a configuration that is not covered in Tables 1 through 3, a nonlinear finite element analysis should be performed and the values of $LF_1$, $LF_u$, $LF_s$, and $LF_d$ should be calculated. If the values of $R_u = LF_u/LF_1$, $R_s = LF_s/LF_1$ and $R_d = LF_d/LF_1$ obtained are greater than the required values shown in Table 4, then the bridge is sufficiently redundant. If $R_u$, $R_s$, or $R_d$ is less than the values shown in Table 4, the bridge has low levels of redundancy and measures should be taken to improve the safety of this bridge. The system reserve ratios are thus defined as

$$r_u = \frac{R_u}{R_{u\ \mathrm{req.}}}$$

$$r_s = \frac{R_s}{R_{s\ \mathrm{req.}}}$$

$$r_d = \frac{R_d}{R_{d\ \mathrm{req.}}} \qquad (8)$$

Thus, if $r_u$, $r_s$, or $r_d$ are all greater than 1.0, the system is redundant.

The check of $R_u$, $R_s$, and $R_d$ is a check on the redundancy of the system. Bridges that are not redundant may still provide high levels of system safety if their members are overdesigned. Therefore, the redundancy check should always be performed in conjunction with a member safety check. This is achieved by comparing the actual capacity of the bridge members to the capacity required by the current member-oriented specifications. In this case, $R_{\mathrm{req.}}$ is defined as the member capacity required to satisfy the current AASHTO specifications. Any acceptable member design criteria can be used. For example, the required member capacity $R_{\mathrm{req.}}$ is calculated for the most critical member using AASHTO's design and evaluation equations:

$$\phi R_{\mathrm{req.}} = \gamma_d D_n + \gamma_l L_n(1 + I) \qquad (9)$$

where

$\phi$ = resistance factor;
$\gamma_d$ = dead load factor;
$\gamma_l$ = live load factor;
$D_n$ = nominal or design dead load; and
$L_n(1 + I)$ = nominal or design live load including impact.

The required member load factor, $LF_{1\ \mathrm{req.}}$ is defined as

$$LF_{1\ \mathrm{req.}} = \frac{R_{\mathrm{req.}} - D}{L_{HS\ 20}} \qquad (10)$$

where $D$ is the dead load effect on the most critically loaded member and $L_{HS\ 20}$ is the effect of a pair of AASHTO HS-20 vehicles on the most critical member. To provide a measure of the adequacy of the actual member capacity represented by $LF_1$ to that required by the AASHTO specifications, the member reserve ratio $r_1$ is defined as the following:

$$r_1 = \frac{LF_1}{LF_{1\ \mathrm{req}}} \qquad (11)$$

Bridge members that are designed to exactly match the AASHTO specifications will produce a member reserve ratio of 1.0. Members that are overdesigned will produce $r_1$ values higher than 1.0.

Using the results of the nonlinear incremental analysis, a system factor $\phi_s$ is defined as follows:

$$\phi_s = \min(r_1 r_u,\ r_1 r_s,\ r_1 r_d) \le r_1 \times 1.35 \qquad (12)$$

The value of 1.35 is used in Equation 12 as a conservative upper limit.

If $\phi_s$ is less than 1.0, it indicates that the bridge under consideration has an inadequate level of system safety. A system factor greater than 1 indicates that the level of system safety and redundancy is adequate. To improve the redundancy of a bridge system, the geometric configuration of the bridge should be changed by either adding members or providing continuity at the supports. If this cannot be achieved, nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations.

**TABLE 4 Required Load Factor Ratios for Direct Analysis**

| Configuration | System Reserve Ratio | Value |
|---|---|---|
| Ultimate limit state | $R_{u\ \mathrm{req.}} = (LF_u/LF_1)_{\mathrm{req.}}$ | 1.3 |
| Serviceability limit state | $R_{s\ \mathrm{req.}} = (LF_s/LF_1)_{\mathrm{req.}}$ | 0.7 |
| Damaged condition | $R_{d\ \mathrm{req.}} = (LF_d/LF_1)_{\mathrm{req.}}$ | 0.8 |

Note: These values are valid for any number of lanes and for any load model, including HS-20 or HS-25 trucks.

One possible way to increase the capacity of bridges is to increase their member reserve capacities ($R - D$) by a factor of $1/\phi_s$. For example, if a bridge member has a resistance $R$ and a dead load $D$ and a system factor $\phi_s$ less than 1.0, it should be penalized by requiring a new resistance $R'$, such that

$$R' - D' = \frac{R - D}{\phi_s} \tag{13}$$

where

$R'$ = updated member resistance (after application of the system factor),
$D'$ = updated dead load, and
$R$ and $D$ = original values of the resistance and the dead load.

Values of ($R - D$) for a bridge that has a system factor greater than 1.0 can be reduced by a factor of $1/\phi_s$, and the bridge will still maintain adequate levels of system safety.

In principle, the same system factor, $\phi_s$, could be applied to all the members of the bridge system. In reality, some members may contribute less than other members toward the overall system capacity, and using the same $\phi_s$ factor for all the members may be inefficient. To be more efficient, the system factor $\phi_s$ may be applied to the most critical member(s) only and the full analysis may be repeated until the system redundancy requirements are satisfied.

Application of a system factor $\phi_s$ will improve the bridge members' strengths represented by $LF_1$ and will also improve system strength expressed in terms of $LF_u$, $LF_s$, and $LF_d$. Thus, the system ratios $R_u$, $R_s$, and $R_d$ may remain unchanged and a nonredundant bridge will remain nonredundant. However, by applying the system factor $\phi_s$ the safety index for one member as well as the system safety indexes will be increased. Thus, nonredundant designs are penalized by requiring higher member safety levels than similar bridges with redundant configurations.

## Load Rating of Existing Bridges

As developed earlier, the proposed redundancy framework is used for the design of new bridges or the load capacity evaluation of existing bridges by modifying the strengths of the members using Equations 7 or 12. It is often difficult to change the member capacities of existing bridges because this may require costly rehabilitations. Therefore, instead of changing the member capacities, the evaluating engineer may simply choose to account for bridge redundancy and system safety by changing the load rating.

According to the 1992 AASHTO specifications (*1*), rating an existing bridge is currently performed by calculating a rating factor RF as shown in Equation 14:

$$\phi R_{\text{exist.}} = \gamma_d D_n + \gamma_l \text{RF} L_n(1 + I) \tag{14}$$

where

RF = rating factor,
$\phi$ = resistance factor,
$\gamma_d$ = dead load factor,
$\gamma_l$ = live load factor,
$R_{\text{exist.}}$ = existing member capacity,
$D_n$ = nominal dead load, and
$L_n(1 + I)$ = nominal or design live load including the dynamic impact factor ($I$).

To account for bridge redundancy during the load rating of existing bridges, the rating factor $RF$ can be expressed as a function of the existing capacity $R_{\text{exist.}}$ and the required capacity $R'$, such that

$$\text{RF} = 1. + \frac{\phi(R_{\text{exist.}} - R')}{\gamma_l L_n(1 + I)} \tag{15}$$

$R'$ in Equation 15 can be calculated using either Equation 7 for typical bridge configurations or Equation 13 for nontypical bridges.

## Example

A 100-ft prestressed concrete bridge that satisfies AASHTO's LFD criteria with nominal HS-20 loading is to be checked for redundancy. The cross section of the six-girder simply supported prestressed concrete bridge is shown in Figure 1. The girders are spaced 8 ft center to center, and the deck is 7 in. The longitudinal members are Type IV AASHTO girders with 4.80 in.$^2$ of grade 270 prestressing steel at an effective depth of 57 in. from the top of the slab. The same effective depth is assumed for the whole span length. The effective prestressing force is equal to 726 kips. The section's concrete strength is 5,000 psi, whereas the slab's strength is 3,000 psi. According to AASHTO's specifications, the nominal ultimate moment capacity ($R$) of each girder section was found to be 5,810 kip-ft. Assuming Type IV AASHTO girders, the dead load moment $D$ for every member is equal to 1,970 kip-ft.

### Load Modifiers

Equation 7 can be directly used to estimate the required member capacity for this bridge if it were to be designed

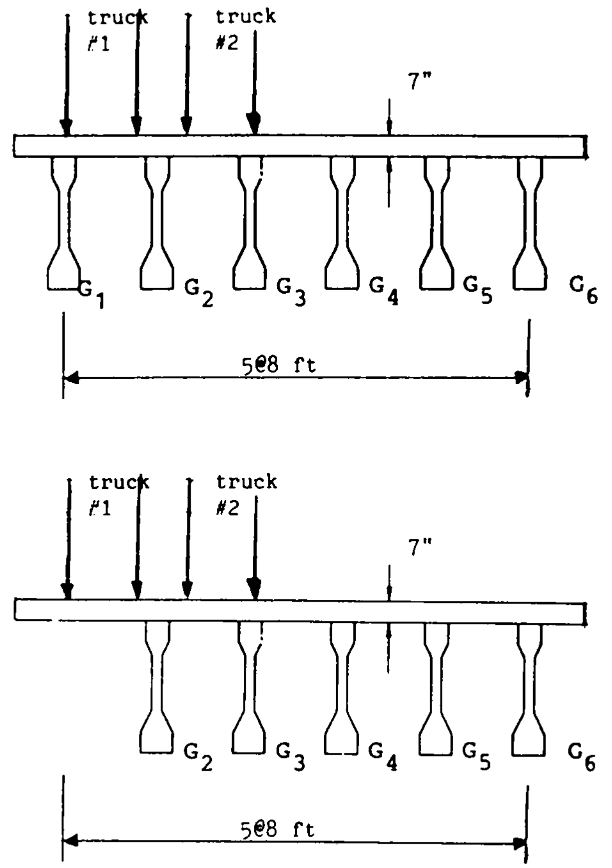


**FIGURE 1 Layout of example of prestressed concrete bridge.**

taking into consideration its redundancy. Using the LFD criteria, the resistance factor $\phi$ is 0.95, the dead load factor $\gamma_d$ is 1.3, and the live load factor $\gamma_l$ is 2.17. $D_n$ is the dead load moment and for this six-girder prestressed bridge is given as 1,970 kip-ft. $L_n$ is the nominal live load and for this 100-ft bridge is given as 1.45 × 762.5 kip-fit (distribution factor times moment due to one wheel load) or 1,105 kip-ft. The impact factor is given as 0.22. The load modifier $\eta$ is taken as the highest value from Tables 1 through 3 for six beams at 6-ft spacings for a 100-ft span. The highest value is 0.80. Plugging these figures into Equation 7, the design check equation becomes the following:

$$0.95\ R'_{\mathrm{req.}} = 1.3 \times 1970 + 0.80 \times 2.17 \times 1105 \times 1.22 \quad (16)$$

Equation 16 gives a required updated member capacity of 5,159 kip-ft. This means that because of the high level of redundancy of this bridge configuration, the member capacity can be reduced from 5,810 to 5,159 kip-ft while still providing an acceptable level of system safety.

## Direct Analysis

In a first stage, a linear elastic analysis is performed for two AASHTO HS-20 trucks without impact factor applied on the bridge, as indicated in Figure 1. The total moment due to the two HS-20 trucks is 3,050 kip-ft. The most heavily loaded member is the external girder G1. G1 carries a live load $L_{\mathrm{HS\text{-}20}}$ = 945 kip-ft constituting 31 percent of the total live load. This value shows that the 1992 AASHTO distribution factor is conservative. The dead load moment ($D$) was found to be 1,970 kip-ft. Using the results of the elastic analysis, the projected load factor $LF_1$ that will lead to the failure of the most heavily loaded member can be calculated as follows:

$$LF_1 = \frac{R - D}{L_{HS\ 20}} \quad (17)$$

where

$R$ = actual member capacity given as 5,810 kip-ft,
$D$ = dead load effect equal to 1,970 kip-ft, and
$L_{HS\ 20}$ = 945 kip-ft = effect of the two HS-20 trucks on the most heavily loaded member.

Substituting into Equation 17 leads to a load factor $LF_1$ of 4.06. This indicates that, by projecting the results of a linear elastic analysis, the first member of the bridge will fail when the pair of HS-20 trucks is incremented by a factor of 4.06.

In a second analysis stage, the AASHTO loads are incremented using a nonlinear model of the bridge structure. The maximum vertical deflections in the longitudinal girders are computed for every load step as the truck load is incremented. Figure 2 gives a plot of load factor versus displacement obtained for this bridge example. A maximum deflection of 6.00 in., corresponding to the span length/200 criterion, is obtained when the load factor is 3.94. This load factor is defined as $LF_s$. The load was further increased until concrete crushing occurred in external girder G1. This was reached at a value $LF_u$ of 6.42.

The calculation of the capacity of the bridge to sustain load under damaged conditions is also performed. For example, the same incremental analysis is repeated assuming that the external girder G1 was completely removed from the model. Girder G1 was chosen as the damaged girder in this example because it was the most critical member of the intact structure. Figure 1 shows a cross section of the damaged model with the loading pattern used in this analysis. The critical loading pattern in this case was assumed to be the same as that of the intact bridge; this, however, may not necessarily always be true. Different loading patterns should be checked

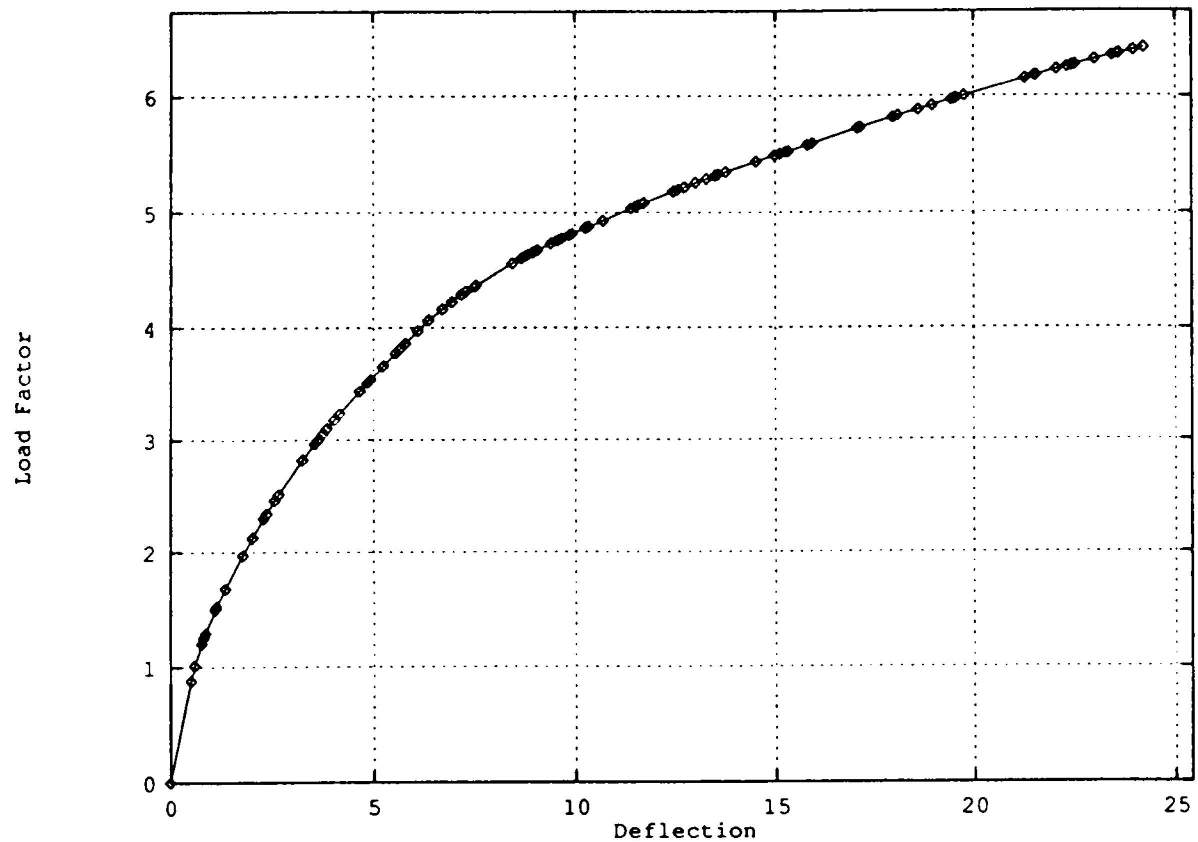


**FIGURE 2 Load deflection curve for example of prestressed concrete bridge.**

when performing the incremental analysis. The HS-20 loads applied on the damaged bridge are incremented until concrete crushing occurred. This occurred at a load factor $LF_d$ of 5.52. In this case, the member that failed is G2. In this analysis in which the nonlinear behavior of the slab in the transverse direction was considered, the slab is assumed to have high levels of ductility.

The values of $LF_u$, $LF_s$, and $LF_d$ obtained from the nonlinear incremental analysis of the prestressed concrete bridge are compared with the member factor $LF_1$ = 4.06. The system reserve ratios obtained are $R_u = LF_u/LF_1 = 1.58$, $R_s = LF_s/LF_1 = 0.97$, and $R_d = LF_d/LF_1 = 1.36$. These system ratios are compared with the required system reserve ratios $R_{u\,\mathrm{req}}$, $R_{s\,\mathrm{req}}$, and $R_{d\,\mathrm{req}}$ given in Table 1. The required values are respectively 1.30, 0.7, and 0.8. The redundancy ratios are obtained as $r_u = R_u/R_{u\,\mathrm{req}} = 1.21$, $r_s = R_s/R_{s\,\mathrm{req}} = 1.38$, and $r_d = R_d/R_{d\,\mathrm{req}} = 1.70$. Because all the redundancy ratios are greater than 1.0, this bridge's geometric configuration is considered to be adequately redundant.

A bridge system that is equally redundant may still be inadequate for truck traffic if its members are inadequately designed. Redundancy recognizes the reserve strength of the bridge system and not individual member strengths. Therefore, the redundancy ratios should be combined with a measure of member safety to verify that the overall system safety is adequate. Checks of member safety can be performed according to any currently acceptable AASHTO criteria, including WSD, LFD, or the proposed LRFD methods. For example, because the bridge was originally designed to satisfy the AASHTO LFD criteria, using Equation 11, the member reserve ratio $r_1$ for the LFD criteria is 1.0. The system factor $\phi_s$ is then calculated using Equation 12 as follows:

$$\phi_s = \min\,(r_1 r_u,\ r_1 r_s,\ r_1 r_d)$$

$$\phi_s = \min\,(1.0 \times 1.21,\ 1.0 \times 1.38,\ 1.0 \times 1.70) < 1.0 \times 1.35 \tag{18}$$

$$\phi_s = \min\,(1.21,\ 1.38,\ 1.70) = 1.21 < 1.35$$

Because $\phi_s$ is greater than 1.0, this system is redundant and its member capacities can be reduced by a factor of 1.21 without jeopardizing the overall system safety. Using Equation 13 with $D' = D$ = 1,970 kip-ft and $R$ = 5,810 kip-ft a value of $R'$ = 5,143 kip-ft is obtained. This is compared with the value of 5,159 kip-

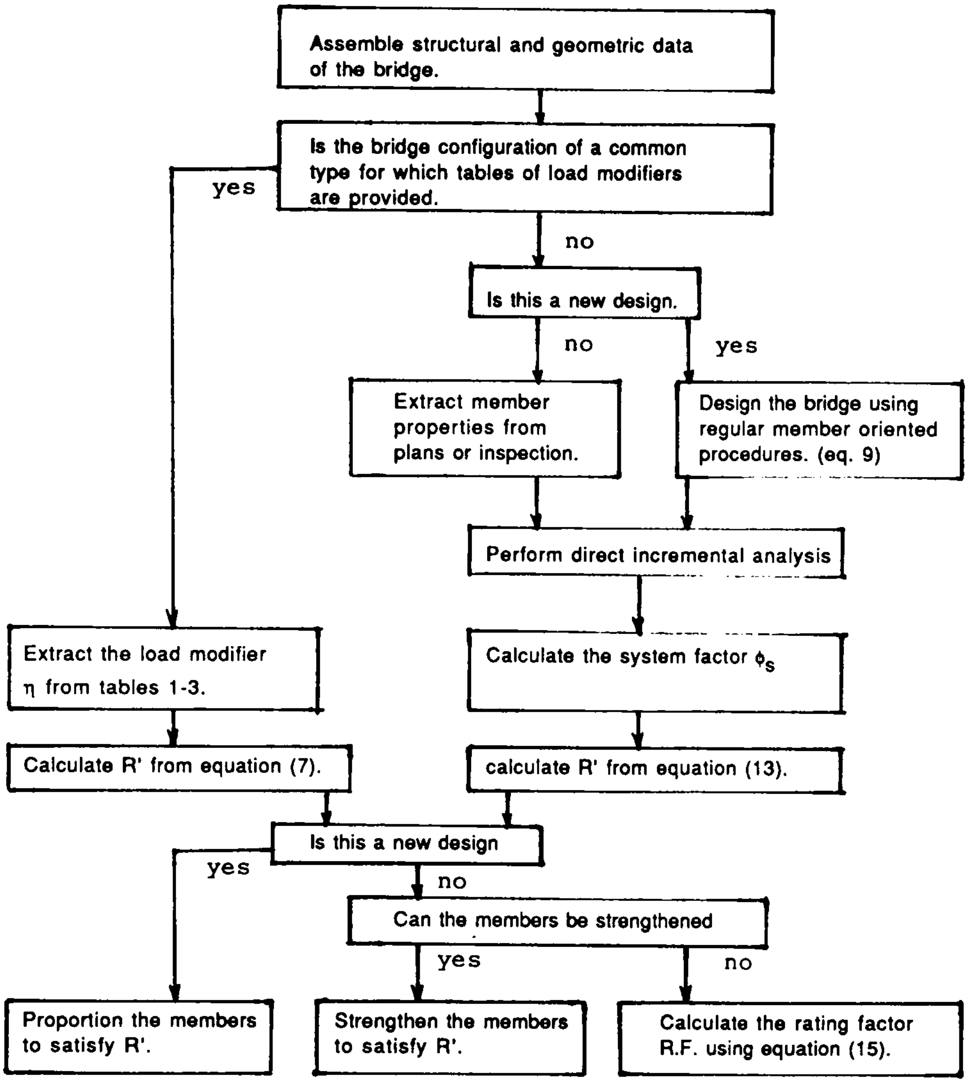


**FIGURE 3** **Flow chart of proposed framework for redundancy evaluation of bridge systems.**

ft obtained using the load modifier tables. The difference between the two methods is less than 1.0 percent.

Other examples illustrating the proposed procedures are provided elsewhere (*5*). The framework proposed in this study to include system redundancy in the bridge design and evaluation of process is summarized in the flow chart of Figure 3.

## Conclusions

The objective of this study was to develop a framework for considering redundancy in the design and load capacity evaluation of highway bridge superstructures. This goal was achieved by proposing a method to reward adequately redundant designs by permitting less conservative member designs than is allowed by current standards. On the other hand, designs with insufficient redundancy are penalized by requiring that their members be more conservative. This could be achieved by applying load modifiers during the routine bridge design and evaluation process.

Tables of load modifiers have been developed for typical bridge configurations. For bridges with configurations that are not covered by the tables, a direct analysis approach is recommended. This requires the performance of a nonlinear incremental load analysis to verify whether acceptable behavior, unserviceable conditions, or collapse states occur under maximum expected truck loading. Guidelines necessary to perform such an analysis are provided. The proposed framework could be readily integrated into future editions of the AASHTO standard and LRFD specifications.

The proposed methods are calibrated using reliability techniques. Redundancy is defined in terms of the difference between the reliability index (or safety index) of the bridge system and the reliability index of the members. The load modifier tables and the load factors recommended for the direct analysis approach are calibrated to ensure that highway bridges will provide adequate levels of system safety.

Although reliability techniques are used during the development of the methodology, the reliability model is transparent to the end user. To consider redundancy during the design and evaluation of a bridge structure, the bridge engineer can simply utilize the proposed system factors without referring to reliability theory.

The calibration process investigated the performance of typical simple-span concrete T-beam, prestressed concrete 1-girder bridges, and multigirder steel bridges. A parametric analysis verified that the redundancy of these simple-span bridges is a function of the geometric configuration and is not sensitive to variations in the section properties. The tabulated load modifiers were calculated only for these bridge configurations, assuming that all the members of a bridge are identical. Load modifiers for other configurations can be easily included in the future.

Continuous bridges are not included in the tables pending further investigation. A sensitivity analysis showed that continuous bridges produced higher redundancy levels than did simple span bridges if the sections in negative bending have sufficient levels of ductility. This means that steel sections in negative bending should be compact, and the concrete section should satisfy AASHTO's requirements. On the other hand, if the sections in negative bending are not ductile, continuous bridges may show low levels of redundancy. Future research on bridge redundancy should carefully consider the relationship between member ductility and the redundancy of continuous bridges.

## ACKNOWLEDGMENT

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