<center>**COMMENTARY**</center>

The attached document provides a commentary on the recommended modifications to Article 1.3 in Section I "Introduction" of the AASHTO LRFD Bridge Design Specifications. The modifications provide a method to include system factors that account for system ductility and redundancy during the design and safety evaluation of highway bridges.

C 1.3 DESIGN PHILOSOPHY

C1.3.1 General

The limit states specified herein are intended to provide for a buildable, serviceable bridge, capable of safely carrying design loads for a specified lifetime.

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications and structural design manuals due to incomplete knowledge of inelastic structural action. The use of system factors in the design equation is meant to account for the inelastic behavior and the presence of system reserve.

C1.3.2 Limit States

C 1.3.2.1 General

Eq. 1.3.2.1-1 is the basis of LRFD methodology.

Assigning resistance factor $\phi = 1.0$ to all non-strength limit states is a default, and may be over-ridden by provisions in other Sections.

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Ductility, redundancy and operational importance are significant aspects affecting the margin of safety of bridge structural systems and the presence of system reserve strength. While the first two directly relate to the physical strength, the last concerns the consequences of the bridge being out of service.

The system factor, ϕ_s of Equation 1.3.2.1-1 provides a measure of the system reserve strength as it relates to ductility, redundancy and operational importance, and their interaction. The system factor, ϕ_s is related to two other factors, ϕ_{su} and ϕ_{sd} which respectively account for system functionality and resistance to collapse conditions after the strength of the most critical member of an originally intact bridge is exceeded, and for the system strength of a bridge in damage state condition.

The system factors provided are calibrated based on LRFD principles and reliability techniques to provide bridges with adequate levels of overall safety and system reliability. Non-redundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. The aim of ϕ_s, is to add reserve capacity for non-redundant systems so that the overall system reliability is increased. If adequate redundancy levels are present, a system factor $\phi_s=1.0$ is used. In the instances where the level of redundancy is

<center>A.2-1</center>

more than adequate, a value of ϕ_s greater than 1.0 may be used. Upper and lower limits of 1.20 and 0.80 are recommended for ϕ_s until more experience is gained in the application of these factors in actual design situations.

Earlier editions of the AASHTO LRFD Specifications accounted for the system effects by applying load multipliers η on the load side of the equation. The system factor is applied on the resistance side of the equation because redundancy relates to the capacity of the system and thus should be applied on the resistance as is traditionally done in LRFD methods.

Unlike the load modifiers of previous editions, the system factor consists of one term only, tabulated (or calculated) for different superstructure and substructure configurations under vertical or horizontal load. Differences in member ductility and redundancy levels are considered by using different system factors depending on the ductility of the most critical members expressed by the ultimate curvature of bridge columns under horizontal loads or whether the main bridge members of multi-girder systems in negative bending have compact or noncompact sections. Instead of using a multiplier to account for operational importance, different system condition states are provided leaving the engineer in consultation with the bridge owners the option of choosing the appropriate condition states depending on the bridge's operational importance. This approach is consistent with current trends to develop performance-based design methods in bridge engineering.

The Strength I Limit State in the *AASHTO LRFD Design Specifications* has been calibrated for a target reliability index of 3.5 for members when the system factor $\phi_s=1.0$ with a corresponding probability of exceedance of 2.0E-04 during the 75-yr design life of the bridge. This 75-yr reliability is equivalent to an annual probability of exceedance of 2.7E-06 with a corresponding annual target reliability index of 4.6. Similar calibration efforts for the Service Limit States are underway. Return periods for extreme events are often based on annual probability of exceedance and caution must be used when comparing reliability indices of various limit states.

The system factors in this set of specifications are calibrated to achieve a system reliability index higher than 3.5 by an additional reliability margin $\Delta\beta_u$ target for bridges subjected to vertical vehicular load. The target reliability margin was set at 0.85 for the collapse condition of originally intact bridge systems under vertical vehicular load. The target reliability margin was set at 0.50 for the collapse condition of originally intact bridge systems under horizontal load. The target reliability margin level $\Delta\beta_u$ was determined to match the system reliability of bridges that have been known to provide sufficient levels of redundancy should one of their members exceed its strength limit. Similarly, a target reliability margin $\Delta\beta_d =- 2.70$ value was set for bridges in damaged state condition under vertical vehicular load.

C1.3.2.2 Service Limit State

The service limit state provides certain experience-related provisions that cannot always be derived solely from strength or statistical considerations.

C 1.3.2.3 Fatigue and Fracture Limit State

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

The evaluation of the fatigue and fracture limit states is not affected by the system factor. The system factor is applied for bridges in damaged state condition to verify the system's capacity after a critical member exceeds its fatigue or fracture limits. The system factor accounts for the probability of collapse after a fatigue or fracture failure of a member while taking into consideration the low probability that such failure may take place.

C1.3.2.4 Strength Limit State

The strength limit state considers stability or yielding of each structural element. If the resistance of any element, including splices and connections, is exceeded, it is assumed that the bridge resistance has been exceeded. In fact, in multi girder cross-sections there is significant elastic reserve capacity in almost all such bridges beyond such a load level. The live load cannot be positioned to maximize the force effects on all parts of the cross-section simultaneously. Thus, the flexural resistance of the bridge cross-section typically exceeds the resistance required for the total live load that can be applied in the number of lanes available. Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained.

C1.3.2.5 Extreme Event Limit States

Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge.

C1.3.3 Ductility

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load carrying capacity immediately when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformations before any loss of load carrying capacity occurs. Ductile behavior provides warning of structural failure by large inelastic deformations. Under repeated seismic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structural survival.

If by means of confinement or other measures, a structural component or connection made of brittle materials can sustain inelastic deformations without significant loss of load carrying capacity, this component can be considered ductile. Such ductile performance shall be verified by testing.

In order to achieve adequate inelastic behavior the system should have a sufficient number of ductile members and either:

- joints and connections which are also ductile and can provide energy dissipation without loss of capacity: or

- joints and connections which have sufficient excess strength so as to assure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.

Statically ductile but dynamically non-ductile response characteristics should be avoided. Examples of this behavior are shear and bond failures in concrete members, and loss of composite action in flexural components.

Past experience indicates that typical components designed in accordance with these provisions generally exhibit adequate ductility. Connection and joints require special attention to detailing and the provision of load paths.

The Owner may specify a minimum ductility factor as an assurance that ductile failure modes will be obtained. The factor may be defined as:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{C1.3.3-1}$$

where:

Δ_u = deformation at ultimate

Δ_y = deformation at the elastic limit

The ductility capacity of structural components or connections may either be established by full or large scale testing, or with analytical models which are based on documented material behavior. The ductility capacity for a structural system may be determined by integrating local deformations over the entire structural system.

The special requirements for energy dissipating devices are imposed because of the rigorous demands placed on these components.

C1.3.4 Redundancy

Bridge redundancy is the capability of a bridge structural system to carry loads after damage or the failure of one or more of its members. Unlike commonly believed, neither the number of parallel elements in a system nor the provision of continuity guaranty structural redundancy. Redundancy is a complex system performance that depends on many parameters including the topological configuration of the bridge system such as the type of structure and the number and spacing of the members, the relative stiffnesses of different components, the ductility of the main members, and the strength and ductility of the members and connections.

Two types of structural redundancy are recognized: 1) the ability of the system to sustain high loads after a main member reaches its limit state capacity and 2) the ability of a system that has sustained some damage to continue to carry some level of live load. Because no criteria had been previously established to determine the acceptable level of overstrength or the strength requirements for damaged bridges, this set of specifications benchmarks these required levels relative to the expected performance of the most critical member in the system. The criteria were established to

provide an additional reliability margin beyond that established by previous sets of specifications that were concerned with individual member performance rather than the system's performance. (Ghosn and Moses, 1998, Liu et al, 2001, Ghosn and Yang, 2013).

C1.3.5 Operational Importance

Such classification should be done by personnel responsible for the affected transportation network and knowledgeable of its operational needs. The definition of operational priority may differ from Owner to Owner and network to network. Guidelines for classifying bridges to be of increased operational priority are as follows:

- Bridges that are required to be open to all traffic once inspected after the design event and are usable by emergency vehicles and for security, defense, economic, or secondary life safety purposes immediately after the design event.
- Bridges that should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the design event, and open to all traffic within days after that event.

Also, the Owner should identify bridges that are susceptible to brittle damage and severe deterioration. These include bridges with fatigue-prone details or bridges that may be subject to collisions with ships, trucks, or debris and ice carried by overflowing streams.

C1.3.6 System factor

The system factors are calibrated so that members of systems that do not provide adequate levels of redundancy are penalized by requiring that they be designed to more stringent criteria than the members of systems that provide adequate levels of redundancy. The reliability-based calibration process followed an approach that is consistent with the LRFD methodology that accounts for the uncertainties associated with estimating the load and hazard demands on the structure and the capacity of the system to resist these demands.

C1.3.6.1 System Factors for Bridge Superstructures under Vertical Loads

The interaction of structural members of a bridge superstructure should be considered in assessing the behavior of the system. Bridge superstructure redundancy is the capability of a superstructure system to carry vertical loads after damage to or the failure of one or more of its members. This provision focuses on vertical overloads because they cause most superstructure failures.

System factors are used in this article to maintain an adequate level of superstructure system safety. Less redundant systems are penalized by requiring their structural members to provide higher safety levels than those of similar superstructures with redundant configurations. The aim of ϕ_s is to add member and system capacity to less redundant systems so that the overall system reliability is increased. When adequate redundancy is present, a system factor, ϕ_s, greater than 1.0 may be used.

Two methods are provided to determine appropriate values for ϕ_s:

a) Equations for ϕ_s are provided for common superstructure configurations as listed in Tables 1.3.6.1-1 through 1.3.6.1-6.
b) An incremental non-linear analysis approach is recommended for superstructures with nontypical configurations and members. (Article 1.3.6.1.1).

The use of the more accurate non-linear incremental analysis for bridges classified to be of increased operational priority is consistent with current trends to use performance based design in bridge engineering practice.

The system factor equations provided in Tables 1.3.6. 1-1, through 1.3.6.1-6 are calibrated by Ghosn and Yang (2013) in NCHRP 12-86 to meet a target reliability index margin for a set of typical straight multi-girder superstructures of types (a) and (k), straight spread-box bridge superstructures of type (b) and (c), and straight single-cell and multi-cell box girder superstructures of type (d), This set includes simple span and continuous prestressed concrete I-girder bridges, simple span and composite steel I-girder bridges, simple span and continuous steel tub girder bridges, simple span and continuous spread prestressed concrete box girder bridges and single-cell and multi-cell box girder bridges. The analyses considered bridges with overdesigned members as well as underdesigned members with various levels of dead loads and dead load to resistance ratios. The analyses considered the ability of bridges to resist the collapse of originally intact systems and the collapse of damaged systems that have lost the contribution of an entire I-girder, the entire web of a box, or the fracture of two webs of a steel box. The systems were analyzed under the effect of two side-by-side AASHTO LRFD HL-93 design trucks and for two trucks in adjacent spans. Two trucks were used as the loading criterion because the AASHTO LRFD calibration found that this is the most probable overloading scenario.

The system factors are functions of the load carrying capacity of the bridge's main members which is represented by the Load factor LF_1 that is equivalent to a rating factor. LF_1 should be evaluated for all possible sections of the bridge including negative bending sections and positive bending section. The governing LF_1 value is the lowest of all possible values after considering all possible vehicular loading conditions. The stronger the individual members are, the stronger is the system.   However, in many cases, the additional strength may not be sufficient to meet the target reliability index margin.

The system factor is also a function of a member's dead load to resistance ratio D/R. The dead load should also include the effect of prestressing forces for continuous prestressed concrete systems. Although the system factor should be applied to all the members of the system affected by the vertical load, the system factor will change from member to member based on its individual D/R ratio. The D/R ratio in the system factor equation is the result of the calibration process which applies the system factor on the entire equation 1.3.2.1-1 including dead load and live loads.

The system factor for bridges in damaged state condition is also a function of the spacing between the I-girders. As the spacing increases, the system's ability to redistribute the load to the remaining bridge members is reduced.   Thus, bridges with members at 12-ft spacing are less redundant than a system with 4-ft spacing regardless of the number of beams.

The contribution of the bracing and diaphragms to help transfer the load of bridges damaged by losing an external girder is considered through the parameter $\gamma_{transverse}$ in Tables 1.3.6.1-4 and

1.3.6.1-5. The equations for calculating the equivalent moment capacity for diaphragms and cross bracings are:

    a. $M_{transverse} = M_{slab} + M_{br/L}$ (C.1.3.6-1)

Where $M_{transverse}$ = combined moment capacity for lateral load transverse per slab unit width

               M_{slab} = moment capacity of slab per unit width

               $M_{br/L}$ = contribution of the bracing and diaphragms to transverse moment capacity calculated using C.1.3.6-2 or C.1.3.6-3

    b. Equivalent transverse moment capacity for cross bracing as defined in FHWA Steel Bridge Design Handbook: Bracing System Design (2012):

$$M_{br/L} = \frac{F_{br}h_b}{L_b}$$ (C1.3.6-2)

    c. Equivalent transverse moment capacity for diaphragm:

$$M_{br/L} = \frac{M_{br}}{L_b}$$ (C1.3.6-3)

where M_{br} = Moment capacity of diaphragms contributing to lateral transverse distribution of vertical load between adjacent main bridge girders.
F_{br} = bracing chord force determined from the applicable limit state for the bolts (see AISC Steel Construction Manual Part 7), welds (see AISC Part 8), and connecting elements (see AISC Part 9).
L_b = spacing between cross frames or diaphragms.
h_b, = distance between the bracing top and bottom chords.

The analyses were limited to slabs with transverse moment capacity in the range of 6.69 kip-ft/ft up to 27 kip-ft/ft. This capacity includes the contributions of well distributed diaphragms at 25-ft spacing. There are indications that the contribution of the slab reaches an upper limit on the order of 1.10 to 1.20. Also, the relationship for $\gamma_{transverse}$ was developed based on the analysis of I-girder bridges, additional analyses are needed to verify its applicability to bracing placed inside box girder bridges.

The effect of the dead weight that was originally carried by the damaged member and then transferred to the remaining system was also considered and accounted for through the factor γ_{weight} in Tables 1.3.6.1-4.

The system factor should be applied to all the members affected by the applied vertical load. A minimum value ϕ_s=0.80 and a maximum value of ϕ_s=1.20 are recommended until further experience is gained in the implementation of the process.

The system factors given in Tables 1.3.6.1-1 through 1.3.6.1-6 did not include some of the very narrow bridges as they are not typical. However, Tables C.1.3.6.1-1 and C.1.3.6.1-2 are provided in this set of commentaries in case the engineer wishes to evaluate the system safety of old existing bridges.

Tables C.1.3.6.1-1 Additional system factors for straight I-girder superstructures for system functionality and resistance to collapse conditions under vertical loading

Bridge cross section type	System factor
Simple span 4 I-beams at 4-ft	$\phi_{su} = 0.80 + 0.16\, D/R$
Simple span 4 I-beams at 6-ft	$\phi_{su} = 0.90 + 0.09\, D/R$
Simple span 6 I-beams at 4-ft	$\phi_{su} = 0.95 + 0.05\, D/R$
Continuous span 4 I-beams at 4-ft with compact members	$\phi_{su} = 0.93 + 0.07\, D/R$

Table C.1.3.6.1-2 Additional system factors for straight I-girder superstructures in damaged state condition under vertical loading.

Bridge cross section type	Redundancy ratio R_d	System factor
Simple span and continuous prestressed concrete I-girder bridges with 4 beams at 4-ft	$R_d = 0.56\gamma_{transverse}\gamma_{weight}$	
Continuous non-compact steel I-girder bridges with 4 beams at 4-ft	$R_d = 0.58\gamma_{transverse}\gamma_{weight}$	$\phi_{sd} = \dfrac{R_d}{0.47 - (0.47 - R_d)\dfrac{D}{R}}$
Simple span and continuous compact steel I-girder bridges with 4 beams at 4-ft	$R_d = 0.64\gamma_{transverse}\gamma_{weight}$	

The Tables 1.3.6.1-1 through 1.3.6.1-6 are for straight bridges. The system behavior of curved bridges and that of bridges with large skews exceeding 40° requires a special analysis to account for the large torsional deformations and different load redistribution patterns on both member safety and system safety. For such cases, the incremental non-linear analysis procedure of Articles 1.3.6.1.1 should be used.

The incremental analysis described in Section 1.3.6. 1.1 " Incremental Non-linear Redundancy Analysis for Bridges under Vertical Loads" should be used for bridge systems with configurations not covered in Tables 1.3.6.1-1 through 1.3.6.1-6 and for bridges classified to be of increased operational priority to obtain more precise values of ϕ_s for all configurations.

C 1.3.6.1.1 Incremental Non-linear Redundancy Analysis for Bridges under Vertical Loads

The nonlinear incremental analysis of bridges should be performed following the procedures described by Ghosn and Moses (1998) and Ghosn and Yang (2013) accounting for the nonlinear behavior of the structural components.

The analysis requires the availability of a nonlinear incremental analysis program that provides a vertical load versus vertical deflection curve and that adequately models the nonlinear behavior of the components up to crushing of concrete, rupture of steel or the formation of a collapse mechanism. Commercially available finite element packages can be used for such purpose.

The ratio of the vertical force needed to cause the failure of the originally intact bridge system to the force causing the failure of any structural member is defined as the system reserve ratio for resistance to collapse condition, R_u.

The ratio of the vertical force needed to cause a vertical deflection equal to span length/100 to the force causing the failure of any structural member is defined as the system reserve ratio for the functionality condition, R_f.

The ratio of the vertical force needed to cause the failure of a damaged bridge superstructure to the force causing the failure of any structural member of the undamaged bridge is defined as the system reserve ratio for the damaged condition R_d.

The engineer in consultation with the bridge Owner must determine the appropriate damage scenarios. Possible damage scenarios include the loss of a critical single member, a failure-critical or fracture-critical member, or a portion of a beam or a connection.

The nonlinear incremental load analysis is executed by applying the unfactored vertical loads (vertical dead and prestressing loads and the design truck load with no impact) on a structural model of the superstructure and incrementing the truck loads until the failure of the first member. It is recommended to place two LRFD design trucks side-by-side in one span as a primary load case and incrementing these loads. Another loading scenario consisting of one truck in each of two adjacent spans maybe used as an alternate scenario. Two trucks are used because the LRFD calibration determined that this load represents the most probable maximum vehicular loading condition. The factor by which the original vertical live load is multiplied to cause the failure of the first member is defined as LF_1.

The nonlinear analysis is then continued beyond the failure of the first member until the maximum vertical displacement reaches a value equal to span length/100. This displacement limit is defined as the functionality condition. The factor by which the original truck load is multiplied to reach this functionality condition is defined as LF_f.

The analysis is further continued beyond this point until the structure reaches its ultimate capacity which can be considered as the peak of the load versus vertical deformation curve or when a hinge collapse mechanism occurs. This defines the collapse condition. The load factor by which the original vertical truck load is multiplied to reach the ultimate capacity is defined as LF_u.

The same process is repeated for a model of a damaged bridge whenever consideration of the survival of a damaged bridge is required by the Owner (e.g. when the bridge is classified as increased operational priority or when a damage situation is considered likely). The damage scenario must be realistic and must be chosen in consultation with the bridge Owner. Damage scenarios may include the loss of a member that may be subjected to risk of fatigue fracture, loss of prestressing, or brittle failure from collisions by ships, vehicles, ice or flooding debris, or major deterioration from corrosion and other environmental factors as well as the possibility of the loss of a column due to impact, scour or foundation failure. The analysis of the damaged bridge is executed in the same manner outlined above for the originally intact structure. But, only the ultimate capacity of the damaged bridge need to be checked. The load factor by which the original vertical live load is multiplied to reach the ultimate capacity of the damaged structure is defined as LF_d.

Bridge superstructures that produce redundancy ratios $R_u= 1.3$, $R_f=1.1$ and $R_d=0.5$ or higher are classified as adequately redundant. Those that do not satisfy these criteria will have system factors ϕ_s, less than 1.0 and require higher component safety levels. If bridge superstructure redundancy is sufficiently high, a system factor greater than 1.0 may be used.

Satisfying the criteria for R_u, R_f and R_d is recommended for bridges classified to be increased operational priority and are susceptible to brittle damage. Satisfying the criteria for only R_u, and R_f is recommended for increased operational priority bridges not susceptible to brittle damage. Satisfying the criteria for only R_u, is sufficient for all other bridges.

The check of R_u verifies that a bridge's ultimate system capacity is at least 30% higher than the load level that will cause the failure of one member. The 1.30 value was obtained based on the analysis of systems that have been historically considered to provide adequate levels of redundancy.

The check of R_f verifies that a bridge's vertical deflection during the application of expected maximum vertical loads is still acceptable allowing the bridge to remain functional for emergency situations.

The check of R_d verifies that a damaged bridge is still capable of carrying 50% of the live load that a non-damaged bridge can carry before one member fails. This would reduce the probability of collapse of the structure in damaged condition should a failure-critical or fracture-critical member fail.

Tables 1. 3.6. 1 -1 and 1.3.6.1-2 do not provide values for the functionality limit state, because for the typical cases tabulated, the results from the functionality limit state are similar to those for the ultimate capacity. This observation is not necessarily true for other bridge

configurations such as trusses and arch bridges. Hence, a check on the functionality limit state must be undertaken for all bridges classified to be of increased operational priority.

C 1.3.6.2 System Factors for Bridge Substructures under Horizontal Loads

The interaction of structural members of a bridge should be considered when assessing the behavior of the substructure system accounting for the superstructure /substructure interaction. Bridge substructure redundancy is the capability of a substructure system to carry loads after damage to or the failure of one or more of its members. For substructures, the most common failures are the result of horizontal loads from earthquakes, wind, ice, stream flow or accidental collisions of vehicles or vessels. Consequently, this Article focuses on horizontal loads from any of the above sources.

System factors are used in this article to maintain an adequate level of substructure system safety. Less redundant systems are penalized by requiring their structural members to provide higher safety levels than those of similar substructures with redundant configurations. The aim of ϕ_s, is to add member and system capacity to less redundant systems such that the overall system reliability is increased. When adequate redundancy is present, a system factor, ϕ_s, greater than 1.0 may be used.

It is recommended that the system factor be limited to a maximum value of 1.20 until sufficient experience the application of high system factors is gained.

Two methods are provided to determine appropriate values for ϕ_s,

a) Eq. 1.3.6.2-2 with the parameters provided in Table 1.3.6.2-1can be used for common straight bridges with typical substructure configurations with equal column heights.
b) An incremental non-linear analysis approach is recommended for substructures with nontypical configurations and members. (Article 1.3.6.2.1).

Bridges being evaluated for seismic extreme event limit state can be evaluated using the displacement-based approach using the AASHTO (2011) Guide Specifications for LRFD Seismic Bridge Design or the traditional force-based approach. The displacement-based approach directly accounts for the system's capacity and therefore a system factor=0.75 is applied. Damage-critical bridges should still be evaluated to verify their capacity to carry some level of vehicular load should one member fail. However, further research is needed to propose a lower level of seismic displacement demand for damaged bridges.

The use of more stringent criteria for seismic hazards associated with high levels of uncertainty in their intensities and the response of the bridge is consistent with current trends to use risk-based design in bridge engineering practice.

The system factors provided in Eq. 1.3.6.2-2 and Table 1.3.6.2-1 are calibrated in NCHRP 12-86 by Ghosn and Yang (2013) to satisfy Equation 1.3.2.1-1 for a set of typical bridge substructure configurations. This set includes straight two-column and multi-column bridges of with concrete columns that have the same height per system. Bridges with various column heights, diameters, lateral and longitudinal reinforcement ratios and foundation stiffnesses were analyzed. Bridges with integral column/superstructure connections as well as superstructures supported on bearings were

considered. All columns in the system are assumed to have similar dimensions and detailing. The nonlinear pushover analysis described in Section 1.3.6.2.1 "Incremental Non-linear Analysis of Bridges under Horizontal Load" should be used for substructures with other configurations.

Bridges with members that will fail in shear as well as all as those with detailing that does not allow the columns to reach their plastic moment capacities are assigned a system factor of 0.75 for seismic events and 0.85 for other hazards. The difference is due to the larger uncertainties associated with estimating the seismic demand and capacity of bridges. This system factor was calibrated to produce a system reliability index higher than the member reliability index by a target margin $\Delta\beta_u$=0.50. This target was used to match the performance of unconfined columns in systems with four-column bents subjected to non-seismic hazards.

The implementation of Eq. 1.3.6.2-2 requires the determination of the risk factor, $\mathfrak{R}$, which reflects the load demand and capacity evaluation uncertainty, the multi-column factor, F_{mc}, which is based on the number of columns engaged in the load application, and the ultimate curvature of the columns which is obtained based on the evaluation of its bending capacity. Eq. 1.3.6.2-2 assumes that member detailing and the members connected to the columns will allow for the ultimate column capacity to be reached before failure in shear or before the failure of the connecting elements. If the detailing does not allow the columns to reach their plastic moment capacities, the system is considered nonredudant and a low system factor is assigned to all the members affected by the horizontal load.   If the shear capacity, connecting members and detailing allow the columns to reach their plastic moments but are not sufficient to allow them to reach their full ductile capacity, a reduction factor, γ_φ, is applied to reduce the ultimate curvature capacity of the columns. This reduction factor is obtained by interpolation using Eq. 1.3.6.2-4 based on the moment that the detailing or connecting members can provide compared to the ultimate moment capacity of the columns.   The ultimate moment capacity can be obtained using widely used available software such as SAP2000 or Xtract.

Systems evaluated using the displacement-based approach are not redundant because unlike the traditional force-based approach, the displacement-based approach directly accounts for the system's performance.

The system behavior of curved bridges, bridges with large skews exceeding 40° and bridges with uneven column heights requires a special analysis to account for the large torsional deformations and different load redistribution patterns on both member safety and system safety. For such cases, the incremental non-linear analysis procedure of Articles 1.3.6.2.1 should be used.

C 1.3.6.2.1 Incremental Non-linear Redundancy Analysis for Bridges under Horizontal Loads

The nonlinear pushover analysis of bridges classified to be of increased operational priority should be performed as described by Liu et al (2001) and Ghosn and Yang (2013) accounting for the nonlinear behavior of the structural elements of the substructure and considering soil/foundation flexibility. The analysis requires the availability of a nonlinear analysis program that provides a horizontal load versus horizontal deflection curve and that adequately models the nonlinear behavior of the substructure components up to crushing of concrete, shearing failure, rupture of steel or the formation of a collapse mechanism. The program should also be able to model the stiffness and the

soil/foundation system by either the use of equivalent springs or actual modeling of the nonlinear foundation. Commercially available programs can be used for such purpose.

The ratio of the horizontal force causing the failure of the system to the force causing the failure of any member is defined as the system reserve ratio for resistance to collapse condition, R_u.

The ratio of the horizontal force causing a horizontal deflection equal to clear column height/50 to the force causing the failure of any member is defined as the system reserve ratio for the functionality limit, R_f.

The ratio of the horizontal force causing the failure of a damaged bridge substructure to the force causing the failure of any member of the undamaged bridge is defined as the system reserve ratio for the damaged state condition, R_d.

Possible damage scenarios include the loss of a single column or a connection and should be considered in consultation with the bridge Owner.

The nonlinear pushover analysis is executed by applying the unfactored loads (horizontal and vertical live and dead loads) on a structural model of the substructure and incrementing the horizontal loads until the failure of the first member. It is recommended that only 20% of the AASHTO HL-93 load be applied vertically during the pushover analysis to account for the low probability of having an extreme horizontal loading event in combination with a maximum vertical loading situation. The factor by which the original horizontal load is multiplied to cause the failure of the first member is defined as, P_{p1}.

The nonlinear analysis is then continued beyond the failure of the first member until the maximum horizontal displacement reaches a value equal to average clear column height/50. This displacement limit is defined as the functionality limit. The factor by which the original horizontal load is multiplied to reach this functionality limit is defined as P_f.

The analysis is further continued beyond LF_f until the ultimate capacity of the system or until a hinge collapse mechanism occurs. The load factor by which the original lateral load is multiplied to reach the ultimate capacity is defined as P_u.

The same process is repeated for a model of a damaged bridge whenever consideration of the survival of a damaged bridge is required by the Owner (e.g. when the bridge is classified to be of increased operational priority or when a damage situation is considered likely). The damage scenario must be realistic and must be chosen in consultation with the bridge owner. Damage scenarios may include the complete loss of a column that may be subjected to risk of brittle failure from collisions by ships, vehicles, or flooding debris, or severe corrosion or scour. The analysis of the damaged bridge is executed in the same manner outlined above for the originally intact structure. But, only the ultimate capacity of the damaged bridge need to be checked. The load factor by which the original horizontal load is multiplied to reach the ultimate capacity of the damaged structure is defined as LF_d. For severe damage scenarios in the substructure, the analysis of the damaged bridge should also be performed for the entire system under the effect of vertical loads following the incremental non-linear analysis procedure described in Article 1.3.6.1.1.

Bridge substructures that produce redundancy ratios R_u=1.2, R_f=1.2 and R_d=0.5 or higher are classified as adequately redundant. Those that do not satisfy these criteria will have system factors ϕ_s less than 1.0 and require higher component safety levels. If bridge redundancy is sufficiently high a system factor greater than 1.0 may be used.

Satisfying the criteria for R_u, R_f and R_d is recommended for bridges classified to be of increased operational priority and those that are susceptible to damage. Satisfying the criteria for only R_u and R_f is recommended for other increased operational priority bridges. Satisfying the criteria for only R_u is sufficient for all other bridges.

The check of R_u verifies that a bridge's ultimate system capacity is at least 20% higher than the load level that will cause the failure of any one member.

The check of R_f verifies that a bridge's horizontal deflection during the application of high loads is still acceptable allowing the bridge to remain functional for emergency situations.

The check of R_d verifies that a damaged bridge is still capable of carrying 50% of the load that a nondamaged bridge can carry before one member fails. This will reduce the risk of collapse should one member be damaged.

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