Operational Importance, Redundancy, & Ductility – Code Considerations for AASHTO LRFD

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Abstract

Load and Resistance Factor Design (LRFD) has, at its foundation, a rational probabilistic framework in which to base structural safety and serviceability. However, key aspects of safety, particularly redundancy and ductility, have not been adequately addressed in AASHTO LRFD provisions. This paper seeks to explore some aspects of ductility and redundancy in order to suggest potential code enhancements, as well as to suggest a simplified analysis procedure to better assess load demands on critical elements, particularly connections.

Introduction

At the outset, it is useful to consider the basic design philosophy in AASHTO LRFD[1], which explicitly considers three principal factors: ductility, redundancy, and operational importance together with the more familiar load and resistance factors. The equation that serves as the basis of the AASHTO LRFD methodology is as follows:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad (1)$$

where η_i is a multiplier that considers redundancy, ductility and operational importance to modify the load side of the equation (γ_i are load multipliers or factors that modify the force effects Q_i). For safe design, the factored resistance ϕR_n (right side of the equation) must exceed force effects as modified by the .

To compute the load modifier η_i , the component modifiers of η_D (ductility), η_R (redundancy) and η_I (operational importance) are multiplied together (each component multiplier ranges from a low of 0.95 to a high of 1.05). In the commentary, AASHTO LRFD describes redundancy, ductility and operational importance as "significant aspects affecting the margin of safety of the bridge…whereas the first two directly relate to physical strength (redundancy & ductility), the last concerns the consequence of the bridge being out of service…the grouping of these aspects on the load side of Eq. 1 is therefore arbitrary". It is the purpose of this paper to explore aspects of this design philosophy and to suggest at least notional improvements from a code perspective. What is of particular concern is this approach of load modifiers to account for operational importance, ductility

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and redundancy results in some unfortunate oversights that compromise system safety, particularly for non-redundant systems.

Operational Importance

To begin, operational importance and therefore the consequences of failure is clearly a principal component of risk, and a direct relationship with safety margin is appropriate. However, for this criterion to be meaningful, a more explicit definition of operational is appropriate. If we use the seismic provisions in AASHTO LRFD Section 3.10.5 to gauge the application of this load modifier, critical bridges are to be designed to be useable for emergency & safety / defense vehicles following a 2500 year return period event, whereas an essential bridge must meet the same criteria for a 1000 year return period event. The use of these two different return periods is inconsistent with a load modifier of 1.05 in force as equation 1 would suggest. What is more of a concern is the large difference between the eastern and western parts of the US in terms of probabilistic seismic hazard. By way of example, for San Francisco the ratio between the 2500 year and 1000 year return period PGA is .7728g / .6145g = 126% whereas for New York, the ratio is .2183g / .1024g = 213%, i.e. this would suggest that the difference in load modifier should be on the order of 1.26 (San Francisco) to as much as 2.13 (New York) in order to be risk consistent from an operational importance perspective.

This issue of a single load modifier for all factored loads becomes even less logical given the inconsistencies in return periods associated with the other AASHTO loads which are based upon much shorter return periods (50 year return period for wind, the 75 year maximum live load, and the 100 year flood for scour, to name a few). These return periods are more consistent with elastic design, whereas seismic resistant design takes explicit account for ductility. This difference is of fundamental importance as will be discussed below.

It is hard to imagine that a load modifier that ranges from 0.95 for less important bridges to 1.05 for critical bridges would give anything that approaches consistent reliability, and meaningful enhanced safety for important versus less important bridges. *NCHRP Report 489 Design of Highway Bridges for Extreme Events* [6] provides insights into some of the challenges associated with obtaining consistent reliability particularly, but does not address this issue of operational importance explicitly. Alternative strategies are presented in [4] and [5] with the premise of introducing system reduction factors on the capacity side; though operational importance is no longer explicitly considered.

As an alternative to this approach of a constant load multiplier based upon bridge importance, the following is proposed:

1. Establish a target Reliability Index for each operational importance class. This target reliability should be as consistent as possible across loads and load combinations (Strength & Extreme Event Limit States).

2. Develop a load specific modifier for each load type that is consistent with the reliability index target for each operational class. These load-specific modifiers can be readily incorporated into a revised definition for the conventional load multiplier, γ_i, i.e. there would be a different load factor for wind loads based upon operational importance.

3. For loads with adequate data, identify return periods consistent with each importance class (as opposed to the use of a load multiplier, the load multiplier is equal to 1 as in the case of seismic design).

4. Maintain a bridge damage / collapse database that updates bridge reliability indices by load type, with the particular emphasis on evaluating the benefit of code changes. It would be interesting to know how the reliability index of bridges designed using LRFD compare with the older provisions.

One key aspect of this implementation strategy is that it provides an explicit basis for evaluating other load types that are not currently part of the AASHTO LRFD design provisions. If loads such as flooding, wave effects, failure during construction, over-height vehicle impact, and fire are causing damage to or collapse of a statistically important number of bridges, it will become necessary to develop appropriate design provisions against this mode of damage / failure.

Given the work of Wardhana et al [8] in the *Analysis of Recent Bridge Failures in the United States*, there appear to be statistically significant loads that are not considered in AASHTO. In fact, the failure cause responsible for 33% of bridge

Failure causes and events	Number of occurrences	Percentage of total
Hydraulic	266	52.88
Flood	165	32.80
Scour	78	15.51
Debris	16	3.18
Drift	2	0.40
Others	5	0.99
Collision	59	11.73
Auto/truck	14	2.78
Barge/ship/tanker	10	1.99
Train	3	0.60
Other	32	6.36
Overload	44	8.75
Deterioration	43	8.55
General	22	4.37
Steel deterioration	14	2.78
Steel-corrosion	6	1.19
Concrete-corrosion	1	0.20
Fire	16	3.18
Construction	13	2.58
Ice	10	1.99
Earthquake	17	3.38
Fatigue-steel	5	0.99
Design	3	0.60
Soil	3	0.60
Storm/hurricane/tsunami	2	0.40
Miscellaneous/other	22	4.37
Total	503	100.00

Table 1 – Bridge Collapse by Type Period 1989-2000 [8]

failures over a 12 year period from 1989 to 2000, was flooding, for which design provisions do not currently exist. It is noted that the flooding of the Mississippi and Missouri Rivers in 1993 that accounted for a significant number of these failures. It would appear that this data, had it lead to the development of robust design provisions, could have alerted us to the potential for the tremendous damage to bridges associated with Hurricane Katrina in August, 29th 2005.

Redundancy

Table 1: Annual Target Probabilities (and Target β_T) from DNV Classification Note 30.6		
Class of Failure	Consequence of Failure	
	Less serious	Serious
I- Redundant Structure	$P_F = 10^{-3}$, $\beta_T = 3.09$	$P_F = 10^{-4}$, $\beta_T = 3.71$
II - Significant warning before the occurrence of failure in a non-redundant structure	$P_F = 10^{-4}$, $\beta_T = 3.71$	$P_F = 10^{-5}$, $\beta_T = 4.26$
III - No warning before the occurrence of failure in a non-redundant structure	$P_F = 10^{-5}$, $\beta_T = 4.26$	$P_F = 10^{-6}$, $\beta_T = 4.75$

Table 2 – DNV Annual Target Probabilities [7]

Redundancy has not been explicitly accounted for in the AASHTO LRFD design provisions, other than with the component modifier η_R outlined above. Though AASHTO LRFD specifies *"multiple load path and continuous structures should be used unless there are compelling reasons not to"*, the code specified penalty for the use of non-redundant members is not at all severe, η_R = 1.05 for non-redundant members, and 1.0 for conventional levels of redundancy (0.95 is permitted for exceptional levels of redundancy).

While non-redundant superstructures are comparatively rare, single column substructures are used ubiquitously throughout the United States, even in high seismic and hurricane prone regions as well as for substructures in navigable waters that are subject to vessel impact loading. From the perspective of redundancy, there tends to be more of a focus on superstructures than substructures.

As a basis for comparison Det Norsk Veritas (DNV) has developed target reliabilities for redundant versus non-redundant structures based upon failure consequences. Given the AASHTO definition "*main elements and components whose failure is expected to cause collapse of the bridge shall be designated failure critical and the associated structural system as non-redundant*", the consequences of failure consistent with DNV definitions would be categorized as "serious". In a similar manner, for redundant superstructures in typical bridges, the consequence of failure would be considered "less serious" in most cases.

The difference in target reliability as described in the commentary [AASHTO LRFD C1.3.2.1] associated with the total load modifier of 1.0 to 1.05 is 3.5 (redundant) to

3.8 (non-redundant) respectively for girder type bridge superstructures. The comparative DNV target reliability index would be 3.09 (redundant, less serious) to 4.26 (non-redundant, serious) and 4.75 (for non-redundant, serious, without warning prior to failure). If AASHTO were to adopt reliability indices consistent with DNV requirements, we would expect a substantially higher load multiplier for non-redundancy.

Redundancy in AASHTO LRFD appears to remain entrenched in the idea of fracture criticality, i.e. that member/connection failure is related to corrosion and fatigue of steel structure systems, the clear contributors to the major collapses of the Mianus and Silver bridges in the last century that had a major impact on code development. While fatigue and brittle fracture remain a significant concern particularly for the large inventory of steel bridges built in the US prior to the 1970's, it results in a focus on routine inspection of existing bridges and member design, detailing, and fabrication of new bridges instead of the global structural system design. This fatigue and fracture-based framework results in code specified requirements for Charpy V-notch fracture toughness of steels used for members identified as FCM's (fracture critical members) instead of holistic requirements for system redundancy in design.

In fact, fracture is discussed only in AASHTO LRFD Section 6: Steel Structures, with the surprising commentary [Section C6.6.2]:

> *"The criteria for a refined analysis used to demonstrate that part of the structure is non-fracture critical has not yet been codified. Therefore, the loading cases to be studied, the location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and the fineness of the models and choice of element type should be agreed upon by the Owner and the Engineer…"*

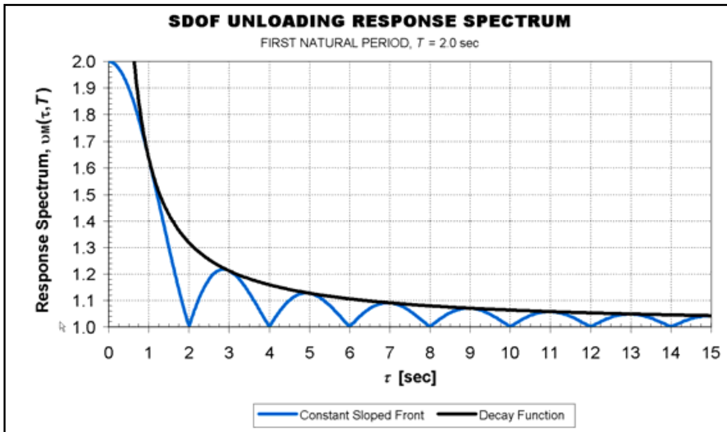


Figure 5 – Unloading Response Spectrum

Though this is a complex topic worthy of future research to be treated comprehensively, there must be more definitive guidance on what constitutes fracture criticality, or more broadly failure criticality given that this assessment methodology should not be restricted to steel tension members only, and therefore should be moved from section 6 of the specifications. The ability to conduct such analyses in a comprehensive way will promote improved structural systems with enhanced robustness. This will result in a much more direct assessment of which members in a structural system are non-redundant and what strategies may be used to minimize the impact of or potentially eliminate member non-redundancy. A more detailed presentation of this approach is given in [9].

To begin there are two types of element failures (member loss) scenarios:

- Abrupt element failure (member loss) with significant dynamic response of the system

- Gradual element failure (member loss), with near quasi-static behavior anticipated.

The linear elastic response spectrum for an undamped single-degree-of-freedom (SDOF) system provides useful insights into this distinction. Figure 5 shows peak response versus unloading duration for a SDOF system with a natural period of 2 seconds. For a loss duration that is a fraction of the fundamental period, the maximum dynamic impact of 2.0 is anticipated. For durations that are 4 to 6 times the natural period of the structure, quasi-static response is anticipated with little or no dynamic impact.

For abrupt member loss, the following recommendations are applicable for use with standard finite element software with static and /or dynamic time history analysis capabilities:

1. Load Factors consistent with Extreme Event I shall be used for fracture criticality evaluations

2. Static Analysis Method – To assess element failure criticality, the safety of the structure shall be evaluated with the assumption of impact force(s) that are 2.0 times the static force(s) in the element prior to failure. This methodology is appropriate for simple structural systems.

3. Dynamic Analysis Method – To assess element failure criticality in a dynamic environment, a time history analysis must be conducted. First the force(s) in the subject member shall be determined from a static analysis. Next the member shall be removed from the model; and a time history be developed that consists of three components; the first component is to replace the force effects in the removed member on the structure over a rise time τ, such that the original configuration (displacement, force distribution, etc) prior to member loss is achieved, the next is the steady state component whereby the force effects of the member are kept constant, and the third component is the unloading component (rapid removal) of these forces over a short duration. A schematic representation of this is given in the figure below. This is appropriate for more complex structural systems.

4. The load application duration τ shall be no less than twice the fundamental period of the structure, and the load shall be applied using the cyclic front, given in the adjacent equation, with ξ_c the magnitude of the force at time t.

$$\xi_{.cf}(t,\tau) := \begin{cases} \left[\dfrac{\xi_{.c}}{2\cdot\pi}\cdot\left(\dfrac{2\cdot\pi\cdot t}{\tau} - \sin\left(\dfrac{2\cdot\pi\cdot t}{\tau}\right)\right)\right] & \text{if } t \le \tau \\ \xi_{.c} & \text{otherwise} \end{cases}$$

5. The steady state portion shall be maintained for a duration of no less than the fundamental natural period of the structure

6. The unloading duration shall not exceed ¼ of the natural period for the highest mode(s) of interest. Peak response for all critical members should occur within a duration of twice the bridge fundamental period. Analysis beyond that point is unnecessary.

7. Cracking or section loss that results in redistribution of the forces within the element in question to other parts of the structural system prior to member failure can be considered explicitly when calculating response magnitude.

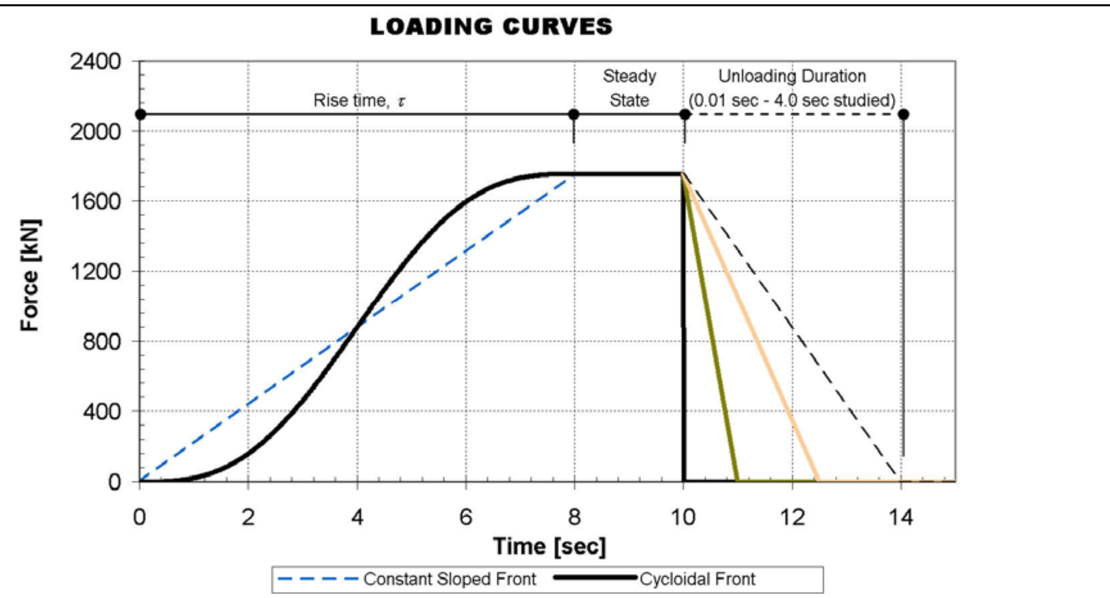


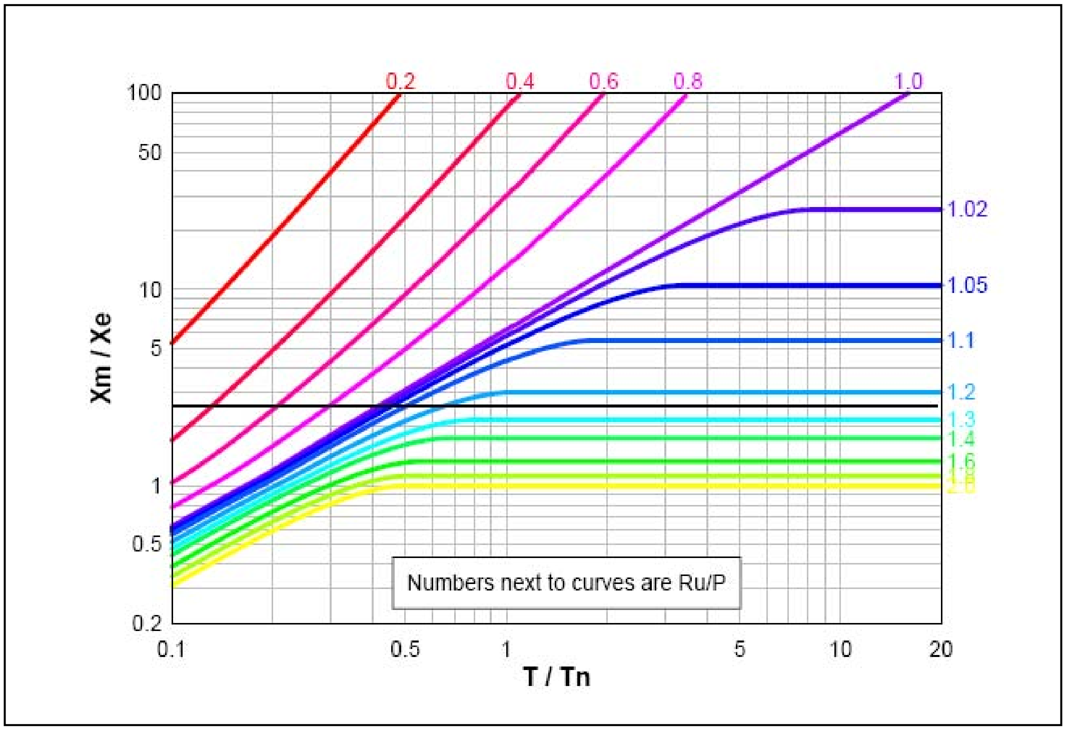
Figure 3 Proposed Loading Curve – Dynamic Analysis

Note, the cyclic front is recommended in order to minimize dynamic response of the structural system in question during reapplication of the loads in the removed member, which serves to minimize the time required for the steady state component, particularly for lightly damped structural systems.

It is felt that this dynamic analysis strategy will lead to direct insights into system force demands, and will help the designer assess potential failure modes and identify strategies to enhance system safety rather than to simply design the fracture critical element to be 5% stronger.

Ductility

AASHTO has identical values for the component modifier for ductility, η_D 1.05 for non-ductile components and connections, 1.0 for components and connections that



meet the specifications and 0.95 for components and connections that exceed the specifications.

Redundancy and ductility are clearly interrelated and a separate load modifiers for redundancy and ductility do not effectively account for this. It is clear in the DNV target reliability indices given in table 2, there is a substantial difference in reliability index targets for an element that demonstrates significant warning prior to failure as compared to an element that fails abruptly.   While this makes intuitive sense, particularly from the perspective of the potential to avoid collapse since the signs of distress will promote the need to unload the structure, there is a more subtle and potentially more important interaction between ductility and redundancy, particularly from the perspective of system design.

Insights can be again gained from the response of SDOF systems, though in the case of ductility, it is necessary to consider a non-linear system, in the simplest case a system with bilinear response (elastic, perfectly plastic response). The graph above depicts the relationship of ductility (y axis) with load duration as a function of natural period for a rectangular pulse. Each colored curve represents a constant ratio of element strength to applied load. For elements designed to remain elastic, the element resistance must be twice the applied (i.e. the dynamic impact is 2.0). For members with inherent ductility, significant reductions in dynamic impact are noted; the black line depicts an element with a ductility of 2.5; it sees an impact of only 1.25. This demonstrates that if members adjacent to a failed element behave with ductility, the dynamic impact force may be reduced significantly.

What becomes fundamentally clear in terms of system response is that enhanced ductility of members adjacent to a failure critical element is a highly effective design strategy to promote safety and robustness. In seismic design philosophy, ductile detailing is utilized for members to enhance response, and their connections are designed to be capacity protected, i.e. the connection is adequate to develop the plastic strength of the member to which it is attached. In accordance with *AASHTO Section 1.3.3 Ductility*, *"the requirements for ductility are satisfied for a concrete structure in which the resistance of a connection is not less than 1.3 times the maximum force effect imposed on the connection by the inelastic action of the adjacent components"*.

What is clear is a similar strategy is appropriate for connections of failure critical members, which is in direct conflict with the requirements of *AASHTO LRFD Section 6.13 Connections and Splices*. This requirement, which has been carried over from earlier AASHTO specifications, requires that connections be designed for the larger of 75% of the strength of the member or the average of the applied load and the factored capacity of the member. From the perspective of system safety, this is clearly inadequate, particularly given the potential for dynamic amplification associated with element failure outlined above.

Worse is that bracing members for straight or horizontally curved flexural members are excluded from even these minimum requirements since "*these details tend to become so large as to be unwieldy resulting in large eccentricities and force concentrations. It has been decided that the negatives associated with these connections justifies the exceptions permitted herein*" AASHTO LRFD Section C6.13.1. Given that the diaphragms / lateral bracing are the primary means for redundancy for two and three girder systems, rather than these minimum requirements being waved, instead provisions similar to the seismic requirements for connections should be in place.

Of most concern are primary truss, arch, and cable stayed girder connections of failure critical elements, with truss connections clearly the most problematic. The combination of failure critical elements connected to what are likely under-designed connections to members that have little ability to respond with ductility and thereby reduce impact forces is as poor a combination as possible from the perspective of structural safety.

Summary & Conclusions

A critical review of the load modifier approach outlined in AASHTO is presented, with the intent to highlight its ineffectiveness at providing enhanced reliability / structural safety. It is recommended that load-specific multipliers be developed consistent with target reliability indices that are dependent upon operational importance.

Similarly, the use of redundancy and ductility load modifiers obscures key aspects of structural safety, particularly for connections of non-redundant members, which may be significantly under-designed using the current AASHTO LRFD provisions. An analysis strategy for assessing member redundancy that explicitly considers member loss dynamics is briefly presented with the intent that it replaces the load modifier for redundancy, and the interaction between redundancy and ductility is explored to provide further insights into structural safety.

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