Introduction

Based on the comments received, it appears that an overarching issue with the report was a lack of clarity related to why the reported ratings differed from those developed using a more common single-line girder (SLG) approach (e.g. the ratings reported by STV using Bar7). In an attempt to add clarity, the three bridges for which cross-sectional dimensions were provided (namely 041, 061, and 138) were rated using version 2.3.0.0 of the STLRFD program developed by PennDOT. The use of STLRFD was selected over the use of the STV Bar7 ratings since the primary methodology of the THMPER System is LRFR, and thus using the STLRFD software permitted a direct, one-to-one comparison. Table 1 shows a side-by-side summary of the dead load (DL) demand, live load (LL) demand (for the governing HL93 live load model), and moment capacity computed by both STLRFD and the Finite Element (FE) rating approach (assuming non-composite behavior).

Table 1 – Summary of LRFR Unfactored Demands and Capacity for both the Single-Line Girder (SLG) and the Finite Element (FE) Approaches (assuming Non-Composite Behavior) – Ratings Factors are shown in Table 2 Below

			LRFR Strength I Flexure (Governing Limit State)								
			Total DL Demand (kip*ft)			Total LL Demand (HL93 w/IM) (kip*ft)			Capacity (kip*ft)		
Bridge	Girder	Skew	SLG	FE	% Diff	SLG	FE	% Diff	SLG	FE	% Diff
041	Int	60^o	234	190	-19%	270	109	-60%	603	603	0%
	Ext		238	177	-26%	313	115	-63%	603	603	0%
063	Int	0^o	150	144	-4%	253	195	-23%	492	492	0%
	Ext		144	164	13%	310	210	-32%	492	492	0%
138	Int	0^o	278	286	3%	276	275	0%	855	855	0%
	Ext		275	283	3%	382	270	-29%	855	855	0%

In comparing DL demands for bridges 063 and 138, it is apparent that the FE modeling approach generates consistent (within 5%) or conservative results as compared with the SLG model. The only difference that exceeds 5% is for the exterior girders of Bridge 063 and is due to the localization of barrier mass in the FE model. This phenomenon is found when comparing the FE- to SLG-derived superimposed dead load for most multi-girder bridges; however it is not apparent in bridge 138 because of two compensating factors. First, while Bridge 063 has a deck overhang of 27 in., Bridge 138 have relatively small deck overhang of 16 in.. In addition, the magnitude of the barrier weight is much greater in Bridge 063 than in the other two structures. Bridge 063 has large reinforced concrete parapets and a small curb while the other two structures have a small curb topped by w-rail type safety barriers.

In comparing the DL demands for Bridge 041, it is observed that the FE model produces DL estimates that are 19% to 26% less than those produced by the SLG model. This is a result of the large skew angle (60^o), which is ignored for DL computations by the SLG approach.

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For LL demands the FE model generally produces lower moments than the SLG model. This can be traced to the distribution factors which aim to approximate both how LL demands distribute transversely and the influence of skew. Given the approximate nature of these factors, it is not uncommon for them to display conservatism upwards of 100% (especially for small girder spacings).

However, this conservatism is not uniform. For example, in the case of Bridge 138 both the FE and SLG models produce identical LL demands for interior girders. It is this variability in accuracy (between 0% and 63% for the three bridges analyzed) which is the most significant shortcoming of the SLG modeling approach. In contrast, the use of FE models to compute LL demands removes this source of variability as it explicitly simulates the influence of skew as well as the transverse distribution of load.

Finally, on the resistance side, both the STLRFD and FE-based ratings use the same moment capacity, since this is explicitly defined by the AASHTO MBE. In going through and developing this table, it was noticed that the capacity we used in our report (dated 5 May 2015) for the girders of Bridge 041 was approximately 5% larger than the capacity shown in the STLRFD software. This was traced to a difference between the moment of inertia used for the historic rolled section identified by STV and a direct calculation of the moment of inertia using the cross-sectional dimensions of this cross-section.

Table 2 provides the updated ratings for Bridge 041 using STV's moment of inertia. In addition, this table provides updated ratings for Bridge 076 in responses to comments below.

Table 2 – Updated LRFR Strength I Load Rating Results without Composite Action

	Updated Model w/o Composite Action	
	Inv	Op
Bridge 041	2.44	3.16
Bridge 063	0.81	1.06
Bridge 076	1.08*	1.41*
Bridge 138	1.10	1.42
Bridge 200	1.04	1.35

*See the "Responses to Comments in the Report" section below for details

In summary, the difference in rating factors between the SLG and the FE ratings is primarily due to the computation of LL demands (with some influence associated with DL demands in cases of large skews).

Response to Comments Received Via Email

Comment 1: *Tom will call you with more details. But as I look at this it seems to be leading us to utilization of FEA to rate bridges. The THMPR does a test – obtains a FEA – modifies it to take out certain items that we might not or should not count on – then indicates the posting. So in the end this is FEA of a bridge to get a better posting. There are issues that come with that. Also this gets down to differences between S/over factors and FEA as well as the appropriate factor of safety.*

Response: This is correct, as discussed above, the differences in the ratings are due to differences in the computed demands (which come from the model chosen – either a SLG or an FE model). The only advantage that THMPER provides over conventional refined analysis is that it is able to validate that the model is representative of the bridge in question through comparison with field captured response data.

Comment 2: *So a further question related to findings would be to ask why we should not go to a higher FS to match what other states do. And why a FEA is better to use than a S/over factor. This gets back to what the goal is in using the THMPR – is it just to get a higher rating. OR is it to insure something is working properly?*

Response: The goal of the THMPER System in regards to ratings is to provide ratings with a uniform safety level across populations of bridges. Although in cases where the S/over (or distribution factors) have an additional level of arbitrary conservatism (e.g. Bridges 041 and 063 – see Table 1 above) the ratings often increase, it is not the goal of the system to produce higher rating factors.

In our opinion, the primary issue with distribution factors is not their "blanket" conservatism but rather the variability of their conservatism. That is, they provide very different levels of conservatism depending on the characteristics of the bridge. For example, for Bridge 041 which had tightly spaced girders and a significant skew, the distribution factors over-predicted the LL demands by more than a factor of two. In other cases (e.g. Bridge 138), the distribution factors can produce LL demands that are very close to FE model results.

The issue of "Target FS" is a difficult one, but it is not directly related to the type of model used within the analysis. Rather, the use of distribution factors within the rating process implicitly imposes different FS based on its accuracy for a given bridge (which is based on the bridges characteristics, e.g., span length, girder spacing, skew, etc.). This is illustrated by the significant conservatism associated with the SLG rating for Bridge 041 and the very small level of conservatism for the rating of Bridge 138. As a result, regardless of the desired "Target FS", FE analysis (given its superior accuracy) is better suited to apply the "Target FS" uniformly across a population of bridges.

Comment 3: *The dead load stresses reported as substantially lower than the dead load stresses calculated in BAR7.*

Response: If this comment was referring to Bridge 041, then the lower DL demands (see Table 1) were due to the fact that the FEA implicitly considers the skew whereas the SLG approach ignores skew for DL demand computation. As shown in Table 1, the DL demands produced by the FEA for Bridges 061 and 138 were with either very similar (within 5%) or larger than the SLG model demands.

Comment 4: *The structural capacities cited in your report exceed the AASHTO Overload provision capacity and even exceed the plastic capacity of the steel section.*

Response: The specific capacities pointed out in relation to the overload provisions further on in the report were associated with composite yield and composite ultimate moment capacities. The non-composite capacities used within the load rating analyses were the same as employed by STLRFD software as shown in Table 1 above. These capacities have been added to the summary tables in the appendices of the revised report.

Response to Comments in the Report

Comment 1: *Verify, LFR or LRFR*

Response: The load rating analyses was carried out using LRFR, LFR, and ASR methodologies.

Comment 2: *Appears that the ratings account for composite action.*

Response: All load rating analyses were carried out both with and without composite action. Both rating factors are reported for each bridge.

Comment 3: *This is not a conservative assumption. It is unconservative. Revise accordingly.*

Response: The authors believe that the assumption that all super-imposed dead load and live load is carried only by the concrete (with no contribution from the embedded steel girder) is a conservative assumption. That is, the assumption is that the moment associated with concrete cracking (neglecting any contribution of the encased steel girder) is the moment capacity of the section. It is common practice for designers to rely only on the encased steel girder in the tension zone, so by ignoring the contribution of the encased girder a much smaller capacity is estimated.

It is acknowledged that this is not a standard approach to perform a load rating. This approach was not intended to produce an actual rating factor (which is why no value was originally reported), but rather to provide some insight into the likely rating to support engineering judgement (given the lack of information available on the embedded steel section). The thought process was that if the bridge has sufficient capacity (or close to sufficient capacity) while ignoring the embedded steel section (aside from its contribution to resist DL) then it is very likely that this structure has significant reserve capacity. If this approach is not satisfactory, then information about the dimensions of the encased steel girder will need to be obtained so that this approach can be abandoned and a more conventional analysis carried out.

Comment 4: *Provide an example of this calculation for review.*

Response: In this case, the maximum super-imposed dead load (SDL) and LL stresses were pulled directly from the model and compared against the modulus of rupture of concrete. The modulus of rupture was taken as that provided by ACI for an assumed concrete compressive strength of 3000 psi (e.g. $f_r = 7.5\sqrt{f_c'}$). The following expressions provide an example of this computation.

$$RF = \frac{Capacity - Factored\ SDL}{Factored\ LL}$$

$$RF = \frac{410\ psi - 81\ psi}{304\ psi} = 1.08$$

Comment 5: *Provide basis that this modal testing is permitted in the AASHTO NBE.*

Response: In Section 8.1.2 of the AASHTO MBE, it states that "Diagnostic tests are performed to determine certain response characteristics of a bridge, its response to loads, or validate analytical procedures or mathematical models." It is this latter use (to validate analytical models) that is consistent with the use of THMPER. That is, THMPER does not use modal testing to directly develop ratings, but rather to validate that the FE model used to generate the ratings is representative of the bridge in question. The AASHTO MBE does reference various dynamic testing techniques, including the type of modal impact testing used by THMPER, but it does not provide specific examples of how such tests may be used. However, since THMPER uses an FE model (validated through modal testing) as the basis for the rating computations, it is also consistent with the "refined analysis" rating procedure within the AASHTO MBE.

Comment 6: *Verify, this probably exceeds the AASHTO overload provision or plastic moment capacity of the beam. Clarify.*

Response: The table in question provided rating factors based on the LRFR methodology for the Strength I Limit State. The reason for the large increase in rating factors is due, principally, to the reduction in Live Load moment that results from better load sharing (as identified through the FEA) than assumed by the distribution factor expressions. The capacity used in computing this rating factor is the same as in previous STV ratings.

Comment 7: *This statement is not definitive. It should be deleted.*

Response: This statement has been deleted and the actual rating factors computed based on the assumption that the moment capacity coincided with the concrete cracking moment were added.

See Table 2 for the resulting rating factors based on the assumptions described in the response to Comments 3 and 4.

Comment 8: *The rating values should be updated based on comments provided in this report relative to dead load stresses, structural capacity of steel sections (AASHTO overload Provisions and Plastic Moment Capacity)*

Response: See Table 2