APPENDIX C

ELIGIBILITY SCREENING CRITERIA FOR SYSTEM ANALYSIS

Not all bridge attributes can be explicitly included in the finite element analysis (FEA) methodology. Some attributes such as corrosion cannot be implemented because there are no verified models that relate inspection aspects, such as element level inspection categories, to mechanical behavior, such as tensile strength reduction. Implementation of other attributes that are localized phenomena, like existing fatigue cracks, require finite element analysis procedures and techniques that are complex, computationally expensive and cannot be reliably benchmarked in the context of finite element models of large multi-component assemblies. Additionally, certain outdated design or fabrication practices, such as pin and hangers, or discontinuous back-up bar splices, that have historically been problematic, should not be permitted in a bridge which redundancy is evaluated per the current FEA methodology. The impact of such attributes on system behavior in the faulted state is nearly impossible to include with confidence. Hence, in the development of the FEA methodology, it was assumed that every bridge member is capable of reaching its nominal ultimate capacity. This requires that the geometry and the material of each member are as described in the design drawings, in sufficiently good condition, and lacking any certain problematic details.

Prior to performing any system analysis, the user shall perform an assessment of the structure using the criteria described in the current appendix in order to identify if any specific negative predetermined characteristics exist. By default, any fracture critical members (FCMs) in a bridge that fails to meet any of the necessary criteria from the screening phase cannot be re-designated as system redundant members (SRMs), regardless of the results of a redundancy evaluation whether such evaluation is performed or not. While the FEA methodology is not intended to be applied to bridges that fail the screening criteria, it may be prudent to perform redundancy evaluations to gain insight into the expected response after the failure of a primary steel member. The rationale for this screening process is to ensure that bridges with certain characteristics do not, under any circumstance, see their inspection requirements lessened. The criteria were established through NCHRP Project 12-87a and based on the work by Parr et al. (2010) [1] and NCHRP Report 782: Proposed Guideline for Reliability-Based Bridge Inspection Practices (2014) [2].

C.1 Application

The screening criteria are only to be applied to the fracture critical members and not necessarily the entire bridge. However, while the results of system analysis are not likely available during the screening phase, consideration should be given to the anticipated response of the bridge in the *faulted* state (i.e., after the initial failure of primary steel tension member). Therefore, some members that would not be classified as FCMs, such as compression members in trusses, may carry tension after failure of one of the members classified as a FCM on the design plans due to load redistribution. The screening criteria discussed below and the principles discussed are intended to be applied to these members as well in some circumstances.

Bridges which satisfy all the criteria examined in the screening phase are acceptable candidates for system analysis which may result in extended arm’s length inspection intervals. There are nine screening criteria that have been identified. However, Owners should consider including additional criteria that are specific to their region or inventory, a specific structural configuration under evaluation, or based on their experience.

C.2 Screening Criteria

The specific screening criteria are as follows:

1. New / recently retrofitted or rehabilitated FCMs.
2. Presence of pin and hangers.
3. Presence of non-redundant eyebars.
4. Presence of plug welds or discontinuous back-up bar splices.
5. Presence of active fatigue cracks.
6. Susceptibility to constraint induced fracture.
7. Presence of existing maintenance problems / load posting.
8. Unreliable or unavailable field inspection data.
9. Condition of FCMs.

Each of the screening criteria is discussed in detail below. The reason for each screening criterion and guidance on how to assess or evaluate a structure for the associated criteria are also provided.

C.2.1 New / recently retrofitted or rehabilitated FCM

C.2.1.1 Reason(s) for Screening:

In order for this assessment to be utilized, an initial arms-length inspection must first be performed. New bridges or bridges recently retrofitted or rehabilitated will thus automatically be screened out. These bridges are required to receive an inspection within 24 months of completion of the work (either newly built or following retrofit / rehabilitation work).

This screening criterion is intended to ensure that all new or “altered” bridges containing a member traditionally classified as a FCM(s) receive an arms-length inspection within 24 months of construction are properly archived and documented. This inspection is intended to identify that the initial construction, fabrication or retrofit/repair work was properly performed and that other problems have not occurred since the work was completed. The inspection is intended to identify defects that the structure may contain that may disqualify the bridge from being a candidate for system analysis.

C.2.1.2 Assessment Procedure:

The current NBIS standards state, *“For routine, in-depth, fracture critical member, underwater, damage and special inspections enter the Structural Inventory and Appraisal (SI&A) data into the State or Federal agency inventory within 90 days of the date of inspection for State or Federal agency bridges.”* [3]. Therefore it is implied that the FCM shall be inspected and the data recorded within three months of completion.

This screening criterion is not intended to override these current regulations but is simply to ensure new FCMs or bridges containing a FCM(s) that have been retrofitted receive an inspection within the 24 month time period. Also note that this criterion applies to the entire bridge, and would be required after any major retrofit or rehabilitation project, such as the replacement of the deck or retrofits performed to mitigate or repair fatigue cracks. Deck replacement could inadvertently damage a girder flange and cracks that have been improperly retrofit may continue to grow. It is the objective of this screening criterion to identify defects in newly constructed or retrofitted/rehabilitated bridges.

It is also noted that bridges which previously underwent system analysis and that passed the current screening criteria in the past are subjected to this criterion following any major retrofit or rehabilitation project. For these bridges, an appropriate inspection is required following the work to identify defects that could have been introduced as a result of the construction work that may void any system analysis previously completed. However, updated system analysis need not be performed as long as no structural modification have been made which would alter the findings of the initial evaluation.

C.2.2 Presence of Pin and Hangers

C.2.2.1 Reason(s) for Screening:

The collapse of the Mianus River Bridge (Greenwich, CT) in 1983 due to a pin and hanger failure is the reason this criterion is in the screening phase of this assessment [4]. Through relatively simple in terms of modeling effort for overall behavior while intact, modeling a pin-and-hanger *system* is rather complex when considering failure modes. As a result, there are basically two types of systems addressed by this section:

- <u>Pin and Hanger in Truss Bridges</u>: Truss type hangers in which an entire truss span is suspended by (typically four) individual pin-ended vertical truss members at each corner of the span. Failure of a truss hanger is presumed to be the result of fracture along any location along the member, failure of the pin itself, failure at the net section of the truss member or in the pin plates on the supporting truss members. Presuming one of the hanger systems were to fail (i.e., at any of the locations cited), the integrity of the remaining hangers and the global and local forces in the structure and connections are difficult to quantify due to the development of significant racking forces and resulting large deflections.
- <u>Pin and Hangers in Girder Bridges</u>: Pin and hanger systems employed in girder systems where a separate hanger plate is located on each side of the girder web. A single pin passes through the web of each of the girders at the joint. Realistically, it is unlikely that both plates on a given girder would fail simultaneously due to brittle fracture. However, although brittle fracture of both hanger plates may not occur, failure of one plate will produce a tremendous imbalance and dynamic impact load on the pin and likely result in the remaining hanger slipping off the pin, as was the case in the Mianus River Bridge. Hence, it would generally be required to assume that both hanger plates have "failed" at a given girder end. Similarly, if the pin were to fail, the complete assembly would obviously be considered failed.

  However, similar to the truss hangers, it is the integrity of the remaining hangers in the other girder and locations that are of concern. In a two girder system, the remaining hanger would be subjected to a significant twisting force in addition to a significant increase in vertical force. The ability of the cap plates which secure the hanger plates remain in place is questionable. If the cap plates were capable of securing the hanger plates in place, the hangers would be subjected to significant out-of-plane bending in addition to the increased axial forces. Further, the resulting local stresses and deflections in the girder web are difficult to accurately model. For example, effects of localized plastic deformations and fracture (ductile or brittle) would need to be considered.

In light of these concerns and uncertainties, bridges with pin and hanger systems <u>do not</u> qualify for system analysis and it is <u>not</u> recommended that they see their inspections requirements lessened.

C.2.2.2 Assessment Procedure:

This criterion shall only be applied to non-redundant bridges with pin and hanger systems. It is noted that this screening criteria only applies to the pin and hanger assembly and not to other tension members traditionally classified as FCMs. For example, a pin and hanger system may support a truss span. In such a case, the redundancy of the truss itself could be evaluated per the proposed FEA methodology and other primary steel tensions members in that truss could be designated as SRMs. In other words, system analysis may be performed to evaluate the members of the suspended span, such as failure of a lower chord. If it is shown through system analysis that the truss remains stable with the lower chord failed, future arms-length inspection need only be conducted on the pin and hanger system. It is noted that the analysis must explicitly include the pins, hangers and their connections, however the pin and hanger assembly is to remain as a FCM.

This criterion does not apply to bridges with multiple girder lines containing pin and link assemblies (three or more girders or truss lines with pin and hanger details are typically not considered to contain any FCMs).

C.2.3 Presence of Non-Redundant Eyebars

C.2.3.1 Reason(s) for Screening:

The collapse of the Silver Bridge (Point Pleasant, WV) in 1967 due to a fracture in an eyebar is the reason this criterion is included in the screening phase of this assessment [5]. Because of the non-redundancy and potential for catastrophic failure, some bridges with eyebar assemblies may not be candidates for system analysis. Criteria for identifying situations where the number and configuration of eyebars would disqualify a bridge from being a candidate for a redundancy evaluation are discussed below. It is important to view eyebars and their connections as assemblies when considering redundancy evaluations. For example, the two eyebars shown in the left of Figure C-1 are intended to serve as a hanger support for a floor beam in this railroad bridge. When only two (or less) eyebars are present at a joint, it must be assumed that the both eyebars have failed. The type of detail is illustrated by the member with two eyebars shown left in Figure C-1. Failure of one of the eyebars will likely result in significant twisting, leading to the other intact eyebar slipping off the pin due to failure of the cap plate. Thus, the system analysis could proceed if the entire hanger (i.e., both eyebars) is assumed to have failed. However, there are other cases where the imbalance would result in multiple members slipping off of a pin and engineering judgement is required in such cases to determine if the bridge should be considered for system analysis.

In other cases, multiple eyebars are used together to serve as one member, as shown to the right of Figure C-1 where the lower chord of this bridge is comprised of eight separate eyebars (near side) or six eyebars (far side). In such a case, experience has shown that failure of a single eyebar does not result in catastrophic failure of the entire eyebar assembly and the joint remains intact. In such a case, the effect of failure of one of the members can be evaluated with confidence. In a way, the overall lower chord shown in Figure C-1 is internally redundant.

Figure C-1. Photographs of various eyebar configurations.

However, there could be other situations where a member made up of two eyebars may be considered acceptable. For example, the two eyebars that make up the hanger in the left of Figure C-2 could likely be considered separate members that would fail independently. The eight eyebars of the chord member are able to easily resist the twisting imbalance that would be created by failure of one of the eyebar hangers (the opposite end of the two eyebar member must be evaluated to ensure similar restraint is provided). Another case where twisting may be prevented is where one member is made up of two eyebars, but the other members are fabricated from multiple eyebars or built-up members, as shown in the right of Figure C-2. In this case, the restraint provided by the built-up member, multiple eyebar lower chord, and floor beam is sufficient to prevent twisting of the pin so the other eyebars would not slip off.

It is noted that this screening criteria *only* applies to the eyebars and does not apply to pins. Provisions for reliable pin inspection must be developed separately from these criteria used to evaluate eyebars. Generally, damage in pins does not become detectable using visual inspection until either the pin has failed or there is significant movement due to wear. Clearly, failure of the pin would be detrimental to the performance of the bridge.

Figure C-2. Examples where failure of one eyebar of the two-eyebar will not result in excessive pin rotation.

C.2.3.2 Assessment Procedure:

This criterion will generally apply to non-redundant bridges with two or fewer eyebars in any segment of an individual eyebar chain. It does not apply to the pins used to connect eyebars and appropriate inspection strategies, including interval and scope shall be developed for pins. However, as discussed above, there are some configurations where it may be overly conservative to exclude systems comprised of two eye bars from redundancy evaluations. Hence, for all eyebar systems, individual circumstances should be considered when evaluating the anticipated performance of a given eyebar configuration.

C.2.4 Presence of Plug Welds or Discontinuous Back-up Bar Splices

C.2.4.1 Reason(s) for Screening:

These types of details have a history of problems with respect to fatigue and or fracture that are significant enough to justify exclusion of bridges containing such details from system analysis unless they have been mitigated. This is primarily due to the difficulty in detecting plug welds during visual inspection, susceptibility of these details to sudden brittle fracture, fatigue cracking, and the fact they can be distributed throughout a given structure. There are also many examples where multiple visual inspections have failed to detect discontinuous backup bars inside of box girders and other structures, suggesting initial detection is often unlikely.

Presuming an initial fracture was to initiate at such a detail, the likelihood of a secondary fracture occurring at a similar detail in another main member, either immediately or shortly thereafter, is deemed unacceptably high. As a result, members with these details disqualify a structure from being considered for system analysis unless all such details are repaired, retrofitted, deemed of acceptable quality, or deemed not susceptible to fatigue crack growth leading to fracture.

C.2.4.2 Assessment Procedure:

In general, bridges with FCMs containing plug welds or discontinuous back up bars should not be considered for system analysis. However, if these types of details have been thoroughly inspected (such as nondestructive testing, and/or some form of rational fracture mechanics evaluation) and are deemed to be of acceptable quality, then this criterion may not necessarily apply. For example, if plug welds are known to exist on a FCM but they have been inspected, evaluated through nondestructive testing methods or fitness-for-service techniques, and the Engineer considers them to be safe, then this criterion may not apply and the bridge can be considered for system analysis.

Also, plug welds and discontinuous backup bar splices known to be located in a compression zone of a FCM may be eliminated from this criterion. For this to apply, the “worst load case,” (*maximum factored load case*) must be shown to generate no net tension at the location of the plug weld. It must also be shown that no stress reversals (compression to tension, or tension to compression) will occur at the location of the plug weld. If these details are subjected to any net live load tensile stress ranges, the bridge should not be considered for system analysis unless a sufficient evaluation has been made (as discussed in the previous paragraph). This is because it has been shown that fatigue damage can still accumulate despite the detail being subject to relatively low tensile stress ranges.

C.2.5 Presence of Active Fatigue Cracks

C.2.5.1 Reason(s) for Screening:

The presence of active fatigue cracks in an FCM disqualifies the structure from consideration for system analysis. The likelihood that fatigue cracks will be present in other similar members (that may not be detectable) is considered to be unacceptably high. For example, if a crack is found in one girder of a two girder system, it is likely a crack has formed or will form in the other girder at the same location. It is important to recognize that cracks may exist that have not been detected using traditional visual inspection methods simply due to the probability of detection associated with visual inspection.

Assuming a fatigue crack results in fracture, there are two other concerns. First, existing cracks in the remaining members could be expected to grow at an accelerated rate after the first member fails since the live load stress range will almost certainly increase. This is important to recognize as it is well established that crack growth rates are not linear and are proportional to the stress range cubed. Thus, a new stress range that is one a half times the original stress range would result in the new remaining life being less than 30% ($1/1.5^3$) of the original remaining life. If there were no obvious indicators of the initial fracture, such as no significant displacement, existing fatigue cracks would grow unchecked at an increase rate in the remaining members until they too fractured. The second scenario is similar to the first, except there is no stable fatigue crack growth in the remaining members with existing cracks. Upon fracture of the first member, the increase in loading on the remaining member(s) is such that the existing cracks become unstable either instantly or at some short time in the future (say during cold weather) and the remaining member fractures.

Prevention of both scenarios would require an inspection at sufficient detail to identify the initial fracture before secondary failures occur. A bridge that successfully met the requirements of system analysis will likely have its primary steel tension members from the FCM list and FCM inspections would no longer occur. Therefore, there is increased potential for the initial fracture to go unobserved for an extended period. Also there is greater potential for fatigue cracks to go unobserved in all members prior to the fracture since the inspections will not be as detailed.

This screening criterion also includes all distortion-induced fatigue cracking. There are many excellent references on this type of cracking available in the literature and the reader is encouraged to examine these documents. The concern is that in the faulted conditions, significant forces in secondary members may be generated. Cracks where these members attached to main girders, truss members, etc. could compromise the load carrying capacity of the connections where these members frame into the primary members. Quantification of the capacity of a cracked connection, under various levels of toughness, crack size, etc. are deemed to be highly uncertain and hence, the estimated capacity of these members is unreliable.

It is also not acceptable to “assume” these members are not present and rely upon those with uncracked connections or to assume other components, such as the concrete deck, to transfer the loads in the faulted condition as these members will carry load in the real bridge regardless of the load path assumed during the evaluation. For example, consider the situation where out-of-plane distortion cracking has resulted in classic web-gap cracking in the webs of both girders of a two girder bridge. If one girder were to fracture, large out-of-plane forces will be produced in the cross frames. Depending on the magnitude of this force, type of cracking, toughness, etc. tearing or even fracture of the web and possibly flange of the *remaining* girder is a viable failure scenario. This would of course compromise the capacity of the remaining girder. Even if it were assumed in the system analysis that the cross frame(s) was not present and the concrete deck transmits all of the load to the remaining girder, the fact remains the cross frame *is* present and *will* carry load. Hence, damage to the remaining girder remains a real possibility.

C.2.5.2 Assessment Procedure:

An active fatigue crack is defined as a crack discovered during previous inspection(s) that has not yet been mitigated. When a crack is mitigated by retrofitting, it must be inspected again within 24 months of the repair and demonstrated to have arrested the crack before it may be considered repaired and inactive. Tack welds that have cracked but are not growing are not considered to be active fatigue cracks. In all cases it must be verified that cracking has not extended into the base metal.

This screening criterion also includes all distortion-induced fatigue cracking. A bridge that simply possesses details that are susceptible to distortion cracks, but where no cracks have been observed, is exempt from this criterion. While performing the assessment, the likelihood of finding cracks (i.e., POD) should also be considered when evaluating this screening criterion. If fatigue calculations have indicated that a negative remaining life exists, this information should be incorporated into the evaluation in some way.

C.2.6 Susceptibility to Constraint Induced Fracture

C.2.6.1 Reason(s) for Screening:

Fractures observed in several bridges due to constraint induced fracture (CIF) are the reason this criterion is in the screening phase of this assessment [6], [7]. Other bridges with similar details as the Hoan Bridge are considered to be at risk for this form of potential brittle fracture. Further, it is realized that frequent inspection will not prevent these failures from occurring (the Hoan Bridge was inspected three days prior to the failure).

In every case where CIF details have been observed, they are distributed randomly throughout the structure. Presuming an initial fracture was to initiate at such a detail, the likelihood of a secondary fracture occurring at a similar detail in another main member, either immediately or shortly thereafter, is deemed unacceptably high. As a result, the presence of CIF details disqualify a structure from being considered for system analysis unless all such details are repaired, retrofit, deemed of acceptable quality, or deemed not susceptible to fatigue crack growth leading to fracture.

C.2.6.2 Assessment Procedure:

A detailed arms-length inspection is required to identify the presence of details susceptible to CIF. Details susceptible to CIF are discussed in Fisher et al. (2001) and Mahmoud et al. (2005) [6], [7].

C.2.7 Presence of Existing Maintenance Problems / Load Posting

C.2.7.1 Reason(s) for Screening:

Bridges with existing maintenance problems that result in the analyst having low confidence in the results of the system analysis shall be screened out. The level of reliability on which the current system analysis procedures were developed may not be met if a bridge with FCMs has significant existing maintenance needs. Bridges may also be screened out based on the Engineer’s judgment with this specific screening criterion. This criterion may be used at the owner’s discretion and is provided to be a “catch all” to allow for other factors to be included. This criterion is included to ensure a bridge with problems should be screened out and thus prevented from re-designating any FCM as SRM. While this screening criterion does not necessarily apply to FCMs or fatigue and fracture (it applies to the entire bridge), its intent is to prevent “troubled” bridges that possess FCMs from being inspected at too great of an interval. Further, in some cases, the overall condition may be such that various components cannot be relied upon to carry load reliably. For example, a composite deck may be so severely deteriorated that it cannot be reliably counted on to transfer load from the fractured girder to the intact girder.

C.2.7.2 Assessment Procedure:

This should apply to bridges with FCM(s) that have existing maintenance problems. The following list provides some examples of "maintenance problems" but may include others at the Engineer's discretion:

- Damaged or non-functioning bearings;
- Collision damage to FCMs;
- Corrosion deemed unsafe;
- Severe drainage issues (causing corrosion etc.);
- Deck in very poor condition;
- Any issue identified as providing sufficient cause to limit inspection interval to no more than 24 months.

Note that this criterion is subjective to the owner and engineer. Therefore, a bridge possessing what is determined to be "minor" damage to bearings or "minor" collision damage does not necessarily have to be screened out. Along with this criterion, bridges with posted weight limitations should be screened out.

C.2.8 Unreliable or Unavailable Field Inspection Data

C.2.8.1 Reason(s) for Screening:

In some cases, available inspection data is found to be unreliable. For example, if it has been shown that information pertaining to cracking or other damage is highly variable from inspection report to inspection report. There are known cases were cracks found during one inspection could not be identified or were reported to have decreased in size during subsequent inspections. Clearly, in such cases the inspection data are unreliable.

Such inconsistencies may be due to number of factors including:

- Inadequately trained inspectors;
- Poor vision of the inspector;
- Insufficient lighting;
- Insufficient cleaning;
- Errors in data entry;
- Subjectivity of the inspector;
- Attitude and work ethic of the inspector(s).

When such is evident in the inspection data, the engineer/analyst should be alerted to the fact that assumptions regarding actual condition of the bridge are highly suspect. Thus, incorporating such inspection data into a finite element model or when attempting to perform the screening process contained herein would be questionable.

The other concern is related to cases where key fracture critical elements or portion thereof are simply not possible to inspect. For example, the condition of some nested eyebars cannot be determined with confidence. Some structures contain tension members that are embedded in concrete, the condition of which is obviously unknown. Another example may be related to some type of welds where quality has been known to be an issue based on experience (*i.e., early electroslag welds or butt welds in A514*). In the absence of recent nondestructive examination to verify the quality of the weld, testing may be prudent in some cases. Thus, when conditions cannot be known with confidence, assumptions regarding the capacity of remaining members are clearly questionable. Failure of one element may produce failure of another element resulting to reduced capacity from unknown damage.

C.2.8.2 Assessment Procedure:

This should apply to bridges with FCM(s) containing elements that are not possible to inspect, or when the reliability of inspection data is questionable. However, if accurate inspection data is obtained and data pertaining to the condition of all elements is available through the use of nondestructive examination or some other appropriate means, system analysis is acceptable.

It is emphasized that regardless of the desire to perform system analysis, it is advisable to accurately ascertain the condition of all FCMs. Where such is not possible, consideration should be given the consequence of member failure.

C.2.9 Condition of FCM

C.2.9.1 Reason(s) for Screening:

Bridges with FCM(s) in relatively "poor" condition are to be screened out. This criterion is to prevent excessively corroded or damaged FCMs from going un-inspected for more than a 24 month interval. It is recognized that in many cases, the condition of the FCM is not directly related to the potential for fatigue and fracture, however, from a practical inspection perspective severely corroded members should be limited to a 24 month inspection interval. It is also noted that this criterion is differentiated from Criterion 7 "Existing Maintenance Problems / Load Posting" as that criterion relates to the entire bridge span while this relates to the individual FCM. It is noted that the phrase "Condition of FCM" also includes the connections. For example, corrosion, pack-out, or distortion in gusset plates in trusses should be considered in this appraisal.

C.2.9.2 Assessment Procedure:

The determination of the FCM condition may be based on the 1995 "*Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*" published by the FHWA [8] and all relevant updates. The rating scale, which ranges from 0 to 9 with each number corresponding to a condition description, is intended to apply to a bridge's superstructure, substructure, or deck. However, these predefined definitions may be used and applied to the FCMs on the bridge for this assessment. This criterion should screen out FCM(s) with a NBI rating of 4 or below.

This assessment allows and encourages the use of element level data. Therefore, an alternative or supplement to the NBI rating is the CoRe Element Guide [8]. Smart flag #357 and # 363 in a condition state of 3 or 4 should be screened out by this criterion. These flags are for members containing excessive pack rust or section loss. Also, any FCM with the lowest (worst) condition state (may be 3, 4, or 5 depending on the element) should be screened out by this criterion.

An evaluation or stress analysis may be performed to show that a FCM is "unaffected" or still provides sufficient capacity. For example, a corroded member's section loss may be measured and analysis performed. If the analysis demonstrates that the section loss is minimal and the member has adequate capacity, then this criterion may be ignored. Note that all calculations must be attached to this evaluation procedure for record keeping purposes when used to show the adequate capacity of the FCM.

Any FCM with an NBI condition rating of 5 or above may "pass" this screening criterion. It is further recommended that any FCM with a NBI condition rating of 4 or below be screened out by this criterion. However, as mentioned above, if a strength analysis is performed and clearly demonstrates the member's adequate capacity, then the member may "pass" this criterion.

The presence of members that are classified as Element Level Condition 4 which affect the system performance would result in the structure being screened out from being a candidate for system analysis until evaluated by the engineer. In some cases Element Level Condition 3 may also result in a bridge being screened out, but again, this would need to be determined by the engineer.

C.3 List of References

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