Appendix B:
NSTM Hands-On Inspection Interval Assessment

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Introduction

The risk model described in this commentary may be used to establish hands-on inspection intervals for steel bridges with nonredundant steel tension members (NSTMs). The risk model is the result of a year-long research project which attempted to identify the attributes that influence the likelihood of brittle fracture as well as the overall need for arms-length inspection (e.g., corrosion damage) (Parr et al., 2010). Once these factors were identified by the research, a Reliability Assessment Panel (RAP) meeting was held at Purdue University on April 12th and April 13th of 2007. The meeting's purpose was to 1) seek expert input on the attributes identified, and 2) seek the RAP's input on additional attributes that may have been overlooked by the researchers. The RAP that conducted the analysis was comprised of a highly qualified and diverse group of experts with intimate knowledge of the topic area of inspection needs for NSTMs. Because the model is specifically focused on the damage mode of fatigue and/or fracture, specialized knowledge was required to identify suitable attributes and criteria. The weighting of attributes was completed based on engineering judgement, which was subsequently tested and calibrated (Parr et al., 2010). The original scoring of attributes assigned more points to attributes that reduced risk, and fewer points to attributes that increased risk. This scoring has been revised to score attributes in the opposite way, i.e., to assign more points to attributes that increase risk, to be consistent with the methodology applied in the National Cooperative Highway Research Project (NCHRP) Report 782, "*Proposed Guideline for Reliability-Based Bridge Inspection Practices"* (Washer et al., 2014).

While the general risk-based methodology described in the main body of this report allows for users to adjust the weights of various attributes, such flexibility is not intended with respect to the attributes and weights described in this appendix. This is because of the considerable effort put forth in the calibration phase during the model's development. As a result, the weights of the various attributes shall be locked and not modified by the user to avoid the possibility of compromising the integrity and overall philosophy built into the methodology. It should be noted that the original effort that developed the model envisioned a maximum hands-on inspection interval of 120 months, whereas the revised National Bridge Inspection Standards (NBIS) limits the maximum allowable interval to 48 months (FHWA, 2022a). Additionally, the revised NBIS allows for certain bridges meeting Method 1 criteria for NSTMs to have an extended 48-month interval. The risk model presented in this section is envisioned for addressing those NSTMs that do not meet the Method 1 criteria listed in the NBIS but are still suitable candidates for extended inspection intervals.

Background

During the 1970's, the material, design, fabrication, shop inspection, and in-service inspection requirements were improved for steel bridges in response to failures (Connor et al., 2005). Special design and fabrication provisions for fracture critical (FC) bridges (today bridges with NSTMs) were implemented in 1978 in reaction to earlier collapses of nonredundant bridges. These requirements were successful in transforming the industry and the design of modern bridges, so fatigue and fracture are very rare in bridges built after 1978. Unfortunately, at the time of the original study, about 76% of all bridges with NSTMs in the bridge inventory were fabricated prior to 1978. Hence, there are a significant number of bridges with NSTMs in service that were not designed for the fatigue limit state and with no consideration for material toughness (Connor et al., 2005).

The focus on shop inspection and fatigue design, as addressed by the more modern American Association of Highway and Transportation Officials (AASHTO) and American Welding Society (AWS) specification requirements, is appropriate. It has also been shown that the extra fabrication and material costs are small in comparison to the additional costs of hands-on inspection of NSTMs mandated by the FHWA's NBIS since 1988. The approximate initial cost premium for new bridges with NSTMs is about 8% to 12% of the cost of fabricated steel (Connor et al., 2005). Interestingly, there is also a hidden initial cost in some cases where more expensive superstructure designs have been used to maintain an acceptable reliability level because of restrictions associated with NSTMs, or a more subtle bias against bridges with NSTMs. For example, multi-girder bridges are commonly used where a two-girder bridge would have been more economical with respect to initial costs (Connor et al., 2005).

Although bridges traditionally classified as NSTMs are often *perceived* to be more susceptible to catastrophic failure by the typical practicing engineer, NCHRP Synthesis 354 and NCHRP Report 883 study could not find any examples of catastrophic failures in any "typical" bridges with NSTMs (Connor et al., 2018; Connor et al., 2005). Bridges with NSTMs have been shown, based on experience (i.e., in-situ failures), analysis, and full-scale testing, to possess considerable redundancy that is unaccounted for in practice.

For some owners, hands-on inspections of NSTMs consume a large fraction of the available inspection budget for a comparatively small number of structures in their inventory. This is because the cost of the hands-on inspection is typically 2 to 5 times greater than routine inspections. Hands-on inspections of closed sections, such as tub girders and tie members, are extremely expensive since inspectors must also enter confined spaces (Connor et al., 2005).

While NSTM inspections are intended to find small cracks before they result in brittle fracture, a recently complete probability of detection study has revealed that there is considerable variability in the ability of bridge inspectors to reliably detect fatigue cracks (Campbell, 2020). The study revealed that the crack size with a 50% probability of detection is about one in. long. In fact, the crack length that corresponded to a 90% detection rate was about 5.5 in. Since it is unknown (and in most cases unlikely) if the toughness of the steel members can sustain a crack of the size likely to be missed during the inspection, the benefits of the NSTM inspections are questionable in preventing brittle fractures. Certainly, the inspections are not capable of reliably identifying cracks that are much smaller than one inch, which may exceed critical crack lengths.

Not to be overlooked is the added risk to the public because hands-on inspection often requires special access equipment resulting in lane closures and traffic backups in which accidents are common. Finally, the inspectors themselves are placed at risk in such circumstances when working at heights under heavy traffic. Thus, it is unclear if the NSTM inspections provide the benefits presently perceived by the industry, particularly in finding small cracks before they become critical. Rather, the performance is more tied to the very rare occurrence of cracks in modern bridges and the improved defect tolerances of steel details (i.e., general linear elastic fracture mechanics calculations and the associated assumptions are conservative).

The NBIS generally requires that NSTMs be inspected using hands-on techniques at a frequency not to exceed 24 months (FHWA, 2022a). Risk-based inspection intervals of up to 48 months are allowed for bridges meeting certain criteria shown in Table B.1 using the Method 1 approach. The table lists the items from the "FHWA Recording and Coding Guide" (i.e., "Coding Guide") and the "Specifications for the National Bridge Inventory" (SNBI) (FHWA, 1995, 2022b). To qualify for a 48-month interval using Method 1, the superstructure and substructure must have a condition rating (CR) of 6 or greater and the bridge must have been built in 1979 or later.

In addition to the criteria listed in the table, the NSTM must be fabricated in accordance with the AASHTO Fracture Control Plan (FCP), which would be expected for a bridge built in 1979 or more recently. There is also a general requirement that no fatigue detail can have a finite life, and the NSTM cannot have a history of previous fatigue cracks. Bridges with pin and hanger connections are not eligible. For bridges not meeting these criteria, a Method 2 analysis can be performed by using a RAP as previously mentioned.

Table B.1. Listing of criteria for Method 1 analysis of NSTMs.

Item	Coding Guide Item	SNBI Item
Year Built	Year Built, Item 27 ≥ 1979	Item B.W.01 ≥ 1979
NSTM Inspection Condition	Superstructure CR, Item 59 ≥ 6, Substructure CR, Item 60 ≥ 6 Fracture Critical Details, Item 92A = Y24	Item B.C.14 ≥ 6
Inventory Load Rating Factor	Inventory Rating, Item 66 ≥ 1.0, when expressed as a rating factor.	Item B.LR.05 ≥ 1.0

It is noted that there was no quantitative engineering process used to substantiate the previous or current 24-month interval. Historically, hands-on inspections were often performed at intervals greater than 24 months when deemed appropriate by owners. This was because the maximum inspection interval for NSTMs bridges was not locked at 24 months and owners had more flexibility to justify a longer interval between hands-on inspections (with the routine biennial inspection still performed at 24 months).

In light of this, questions have arisen with respect to inspection requirements for newer bridges which were built using superior steels (especially using any type of High-Performance Steel (HPS)), subjected to modern shop inspection techniques including nondestructive testing (NDT), and fabricated using higher quality welding processes than in the past. In addition, those not fabricated using the present FCP, not designed with fatigue-resistant details, and subjected to high Average Daily Truck Traffic (ADTT) should be inspected with greater frequency than those fabricated using the FCP, designed with more fatigue-resistant details, and subjected to low ADTT. Equally important is the need to justify the interval of 24 months, as this interval was not based on any quantitative data nor any formal RAP process.

There is no doubt that the past and current hands-on inspection requirements have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. However, considering the potential for under- or over-inspection and the substantial cost, both in terms of finances and effort, the question arises, “Should all bridges traditionally classified as nonredundant be inspected at the same interval regardless of condition, material type, ADTT, fatigue details, etc.?” For example, should a newly built bridge fabricated to the modern fracture control plan using HPS, highly fatigue resistant details, and carrying little or no truck traffic (e.g., a high occupancy vehicle ramp) be inspected at the same interval as an older two girder bridge fabricated from lower quality material and fabrication processes, high ADTT, and poor fatigue resistant details. Many other industries, including aerospace, offshore, and ship building have established inspection intervals using more rational criteria than simple “calendar based” criteria, as it provides a higher level of reliability and provides a more rational approach to inspection. Furthermore, using such criteria generally results in an inspection program that is more economical.

The current version of the NBIS applies a standard interval of 24-months with some options for extending that interval using risk-based inspection (RBI). The procedures contained herein are intended to provide a risk-based approach to setting the maximum inspection interval for NSTMs based on a set of attributes criteria known to influence the likelihood of fatigue and fracture. As mentioned previously, the attributes and criteria described herein are the product of a RAP meeting focused specifically on NSTMs. Originally, the maximum interval envisioned by the risk model was 120 months. However, the maximum inspection interval under current NBIS requirements is only 48 months.

The scoring approach has been adjusted to provide a similar format to the risk models for routine inspections described in the main body of the report. However, the weighted sum model is applied differently to provide a simple scoring process that sums the point values assigned for the different attributes, with numerical thresholds to define the risk levels (i.e., inspection interval) for a given bridge being analyzed. The following commentary presents the assessment criteria and detailed examples of the methodology. It should also be noted that the NBIS includes certain criteria for requiring hands-on inspection of NSTMs at less than 24 months. The scope of this risk model does not modify those requirements but does provide a means for assessing NSTMs that require inspection intervals of less than 24 months.

Assessment Procedure

From this point forward, the reference to an inspection refers to a hands-on inspection of a NSTM unless otherwise noted. This is differentiated from the routine biennial inspection carried out on all bridges. It is assumed that when the NSTM inspection is increased beyond 24 months, the routine inspection will continue at the defined interval for routine inspection of a given bridge.

Presently, the general requirement is for NSTMs to be inspected at an interval not to exceed 24 months unless the bridge meets the Method 1 criteria listed in Table B.1, or criteria developed by a RAP using Method 2. However, there is no objective data that can be cited which confirms that 24 months is the *appropriate* interval for NSTM inspection. Hence, the strategy of the assessment procedure presented herein is to first determine if a given bridge *qualifies* to be inspected at an interval *as long as* 24 months. Part 1 of the assessment, which is referred to as the "Screening Phase," attempts to address this issue. If the bridge "fails" this first assessment phase it is described as *Category I* and a shorter interval (e.g., 12 months) may be appropriate. The reduced interval will be established through the "Scoring Phase" (Part 2). However, if it is demonstrated that the bridge meets the screening set of criteria, it is described as *Category II*, and inspection intervals of 24 months or greater may be considered. Increasing the interval beyond 24 months during the Category II Scoring Phase requires that the bridge achieve a certain score.

The overall assessment procedure is summarized in the flowchart in Figure B.1. The procedure begins with scoring a selected bridge according to Part 1 - Screening Phase (Attachment C). If any of the attributes in Part 1 are not satisfied, the bridge is defined as a Category I bridge and its maximum inspection interval is limited to 24 months. The bridge is then scored according to Part 2 - Scoring Phase. The total number of points scored for the bridge based on rating all attributes (Part 1 and Part 2) are then summed to determine the final score for the bridge. The final score for the bridge is compared with the threshold values for Category I bridges shown in Table B.2. If the total number of points scored for the bridge is less than 130 pts, a 24-month inspection interval is assigned, unless the member has a CR of 4 or less. The NBIS requires any NSTM with a CR of 4 or less to have a maximum interval of 12 months (FHWA, 2022a). If the total score is in the range of 130 points to 149 points, a reduced interval of 12 months may also be appropriate. If the total score is 150 points or greater, an inspection interval of 6 months should be considered for the bridge.

If the bridge passes all the screening criteria it is defined as a Category II bridge and the total score will depend only on the Part 2 Scoring Phase (Attachment B). As shown in Table B.2, if the bridge scores 130 points or more in the Part 2 Scoring Phase, the assigned interval remains as 24 months. However, if the member scores less than 130 points, a 48-month interval may be assigned. It should be noted that the original formulation of this risk model had additional thresholds for defining an interval of 72 months and 120 months, but the revised NBIS does not allow inspection intervals of greater than 48 months for any NSTM.

It must be noted that the assessment proposed herein shall apply to typical steel bridges in the general inventory including:

- Typical plate girders (curved included).

- Typical box girders (curved included).
- Typical trusses.
- Tied arches.

Bridge types NOT covered by this assessment include:

- Suspension bridges.
- Cable-stayed bridges.

Engineering judgement may be used to determine other “atypical” bridges to which this assessment may not apply.

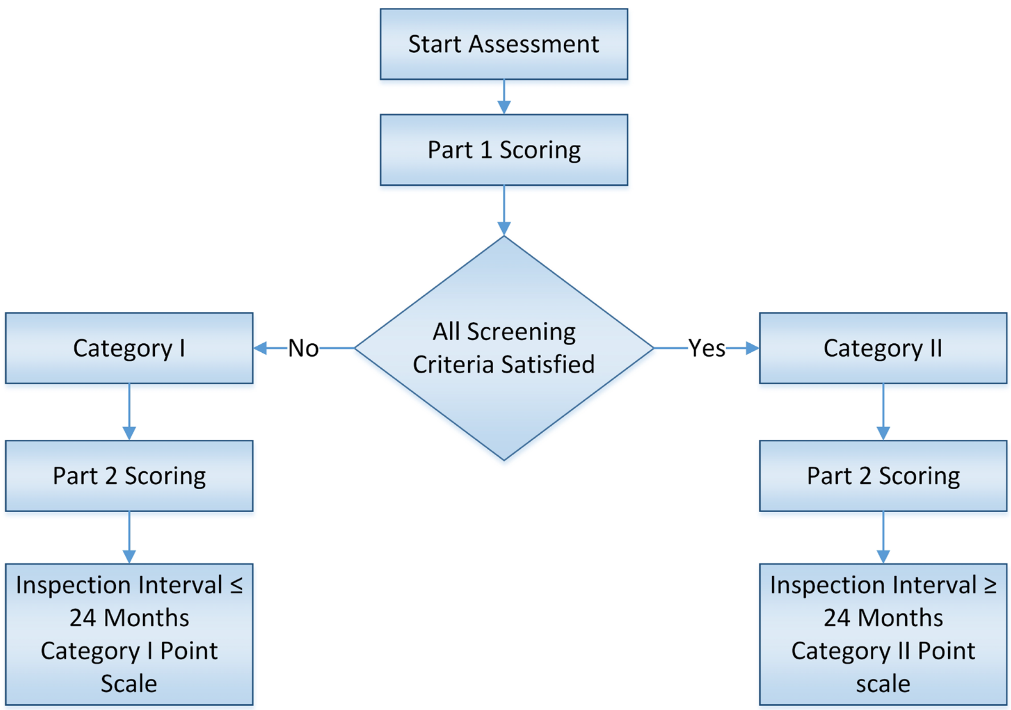

Figure B.1. Assessment procedure flowchart for risk assessment of NSTMs.

It must be noted that the assessment provided herein relies heavily on knowledge regarding the limit states of fatigue and fracture coupled with some engineering judgment. Hence, the procedure should only be performed by an experienced licensed professional engineer familiar with steel bridges and a solid understanding of the fatigue and fracture limit states.

Table B.2. Scoring scale for NSTM inspection intervals.

Category I Scale	Inspection Interval	Category II Scale	Inspection Interval
Score < 130 points	24 Months	Score ≥ 130 points	24 Months
135 points ≤ Score < 150 points	12 Months	Score < 130 points	48 Months
Score ≥ 150 points	6 Months	-	-

When using the assessment, it is not necessary to possess all the information about the bridge being analyzed. The procedure was purposefully crafted and calibrated in this fashion. For example, if the Average Daily Truck Traffic for a single lane ($ADTT_{SL}$) for the bridge is unknown, then that section of the assessment should be assigned the maximum score, and a conservative score will result. This applies for all criteria in the assessment in both Part 1 and Part 2.

The assessment procedure is intended to be applied to individual NSTMs and not the entire bridge. However, for simplicity, bridges with multiple NSTMs can be addressed by analyzing the "worst case" NSTM and the resulting inspection interval may be applied to all NSTMs on the bridge. Alternatively, an owner could decide to address each NSTM in the bridge individually. The decision of whether to apply the assessment to all NSTMs or the bridge's "worst case" NSTM is left up to the owner or engineer.

Part 1 - Screening Phase

The goal of the screening phase is to establish whether a bridge possesses the characteristics that would justify an inspection frequency *as great as* 24 months. This phase attempts to "screen out" and identify those bridges (or NSTMs) that should be inspected with a frequency of 24 months or less. Bridges failing to meet any of the necessary criteria from the screening phase are automatically limited to a maximum inspection interval of 24 months. Attachment B includes details for each of the eight screening attributes. Following the screening phase, the bridge is assessed using the Part 2 scoring procedure to establish the maximum permitted inspection interval. It should also be re-emphasized that the screening criteria and the assessment itself may be applied to the NSTMs and not necessarily the entire bridge.

Bridges which satisfy <u>all</u> the criteria examined in the screening phase are classified as Category II. If an owner does not wish to proceed with the Part 2 scoring, the evaluation may be terminated at this point and a 24-month interval assigned to the NSTM. However, by proceeding with the Part 2 scoring, an extension of the maximum inspection interval to 48 months may be realized.

Five points are associated with each of the eight attributes in the screening phase. For each attribute, 5 points are added for each negative or detrimental attributes the bridge/NSTM possesses. The score from the screening phase (Part 1) is eventually combined with the score from the Scoring Phase (Part 2) to give the final total score for the assessment.

Part 2 - Scoring Phase

The Scoring Phase of this assessment is intended to account for the different factors that are established through engineering principals and/or expert opinion to influence fatigue and fracture of primary load carrying members and hence their relationship to the inspection interval. Each of the attributes and associated criteria are described in detail in Attachment C.

In the overall scoring method, more points are assigned to those qualities or attributes considered less favorable, i.e., reduce the reliability of the NSTM. This approach is consistent with the overall RBI methodology previously developed where a higher score results in a shorter inspection interval. While the absolute score is important, it is also important to consider the relative difference between each qualitative description. For example, in this assessment, an optimum quality is typically assigned zero points. However, a quality that is viewed as an unfavorable quality may be assigned anywhere from 5 to 20 points, depending on how it was weighted in terms of negative impact during the calibration process. Further, some items are binary in terms of application. As an example, when comparing if a bridge was or was not fabricated to the AASHTO/AWS Fracture Control Plan, there are only two choices: either it was or it was not. While zero points are given to bridges built to the FCP, a bridge not fabricated to the FCP is assigned 20 points. However, this does not mean that such bridges should be viewed as being expected to perform poorly in service. Rather, this factor was simply weighted heavily by the RAP during the development of the procedure.

Each point scale may also be divided into smaller intervals for individual attributes that require multiple levels of scoring. For instance, when the ADTT is measured, the level most benefiting the bridge, (almost zero ADTT) receives 0 points, low ADTT receives 5 points, the intermediate level (moderate ADTT) receives 13 points, and the least beneficial level (high ADTT) receives 20 points. Further details associated with each of these general classifications are discussed individually in the context of the specific attributes under consideration in the attachments.

Attachment A:
Assessment Commentary
Part 1 – Screening Phase

**NOTE: THE FOLLOWING ASSESSMENT COMMENTARY MAY BE APPLIED TO INDIVIDUAL NSTMs, AND DOES NOT NEED TO BE APPLIED TO THE ENTIRE BRIDGE

1.1 - New or Recently Retrofitted or Rehabilitated NSTM

Reason(s) for Screening:

For this assessment to be utilized, an initial detailed hands-on inspection must first be performed. New bridges or bridges recently retrofitted or rehabilitated are automatically screened out until such inspections are conducted. This screening criterion is intended to ensure that all new or "altered" bridges containing NSTM(s) receive a hands-on inspection within 24 months of construction and that all defects or changes are properly documented and, if needed, defects are mitigated.

Assessment Procedure:

The current NBIS states, "Perform an initial inspection in accordance with Section 4.2, AASHTO Manual …. for each new, replaced, rehabilitated, and temporary bridge as soon as practical, but within 3 months of the bridge opening to traffic." (FHWA, 2022a). Therefore, it is implied that the NSTM should be inspected, and the data recorded, within three months of completion. A second inspection within 24 months of the initial inspection is required before an extended inspection interval can be implemented (FHWA, 2022a).

This screening criterion is not intended to override these current regulations but is simply to ensure new NSTMs or bridges containing a NSTM(s) that have undergone retrofit get inspected within the 24-month interval. 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 fatigue cracking. For example, a deck replacement could inadvertently damage a girder flange and retrofitted cracks may continue to grow, or new cracks may be introduced. It is the objective of this screening attribute to consider the potential for defects in newly constructed or retrofitted/rehabilitated bridges.

Table B.3. Criteria and scoring for new or recently retrofitted or rehabilitated NSTMs.

Screening Citeria	Interval and Points
New or rehabilitated NSTMs not previously subjected to a detailed hands-on inspection since the completion of construction	≤ 24 months & 5 points
Existing NSTMs subjected to a detailed hands-on inspection within the past 24 months after the completion of construction	0 points

1.2 - Pin & Hanger Connections

Reason(s) for Screening:

The 1983 collapse of the Mianus River Bridge due to a pin and hanger failure is the reason this criterion is in the Screening Phase of this assessment (Demers, 1989). Bridges with these details should not be inspected at intervals greater than 24 months because these components are generally viewed as being non-redundant and susceptible to catastrophic failure. The interval should be applied to all NSTMs in the subject bridge, not only the pin and hanger connections. The reasoning for this restriction is related to the questionable response of the entire structural system should a failure anywhere on the bridge occur. For example, consider the case of a riveted built-up member. Unless an internal redundancy evaluation of the entire bridge is performed using the AASHTO guide for internal redundancy, it would be unknown if the failure of a single cover plate would be expected to result in failure of the member at that location (AASHTO, 2022). As the load is redistributed throughout the bridge following such an event, the highly uncertain response of the pin and hangers could lead to total collapse. In contrast, the outcome may not be as detrimental in a continuous girder where pin and hangers are not present.

<u>Assessment Procedure:</u>

This criterion should apply ONLY to non-redundant bridges with pin and hanger assemblies. Bridges with multiple girder lines containing pin and hanger assemblies are not considered nonredundant, and these procedures therefore do not apply. For example, a bridge with three or more girders with pin and hanger details is typically not considered to be nonredundant.

Table B.4. Criteria and scoring for pin and hanger assemblies.

Screening Citeria	Interval and Points
Pin and Hangers assemblies	≤ 24 months & 5 points
NO Pin and Hanger assemblies	0 points

1.3 - Nonredundant Eyebars

<u>Reason(s) for Screening:</u>

The 1967 collapse of the Silver Bridge due to a fracture in an eyebar is the reason this criterion is included in the Screening Phase of this assessment (NTSB, 1970). Because of the lack of redundancy and potential for catastrophic failure, bridges with eyebar assemblies should not be inspected at intervals greater than 24 months if there are fewer than four eyebars carrying primary loads in any connection.

<u>Assessment Procedure:</u>

This should apply ONLY to non-redundant bridges with NSTMs that have fewer than 4 eyebars in any individual segment of an eyebar chain. (*Note, an individual eyebar segment is defined to be between any two nodes connected by eyebars. For example, in a lower chord between nodes L1 and L2, there would need to be at least 4 eyebars for this provision not to be applied.)*

Table B.5. Criteria and scoring for bridges with eyebars.

Screening Citeria	Interval and Points
NSTM with < 4 Eyebars	≤ 24 months & 5 points
NO Eyebars or ≥ 4 Eyebars	0 points

1.4 - Plug Welds or Discontinuous Back-Up Bars

<u>Reason(s) for Screening:</u>

These types of details have a history of problems with respect to fatigue and/or fracture that are significant enough to justify a 24-month maximum inspection interval.

<u>Assessment Procedure:</u>

In general, NSTMs containing plug welds or discontinuous back-up bars should not be inspected at intervals greater than 24 months. However, if these types of details have been thoroughly assessed (such as through NDT and/or some form of rational fracture mechanics evaluation) and are deemed "safe," then this criterion may not need to be applied. However, an evaluation must be performed and documented for this criterion not to apply. For example, if plug welds are known to exist in a NSTM but they have been inspected or evaluated with NDT methods and the engineer considers them to be safe, or the plug welds have been removed, then this may not apply, and the five points need not be assigned for this screening criterion.

Also, plug welds shown to be in a compression zone of a NSTM may be eliminated from this criterion. For this to apply, the "worst load case" (maximum factored load case) must be shown to generate NO 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. Plug welds subjected to ANY live load tensile stress ranges should NOT be permitted to be exempt from this screening criterion without sufficient evaluation (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.

Table B.6. Criteria and scoring for Plug Welds.

Criteria	Interval and Points
NSTM(s) with plug welds or discontinuous back up bars	≤ 24 months & 5 points
NO plug welds or discontinuous back up bars	0 points

1.5 - Active Fatigue Cracks

<u>Reason(s) for Screening:</u>

Active fatigue cracks can grow quickly and potentially fracture an entire member. The presence of active fatigue cracks in a NSTM is not acceptable and thus the maximum inspection interval should be set at 24 months.

<u>Assessment Procedure:</u>

An active fatigue crack is defined as a crack discovered during previous inspection(s) that has not yet been proven to be arrested. A fatigue crack may be considered "inactive" when it has been shown that no growth has occurred during the previous seven years. (*The requirement of 'no crack growth in the past seven years' was selected during the development of this procedure and may be modified at the discretion of the owner if sufficient justification is provided and included in the documentation associated with the assessment procedure as a permanent record. Although it may appear to be a severe criterion, it is also intended to discourage the practice of permitting un-retrofitted fatigue cracks to remain in NSTMs.*)

When a crack is retrofit, it must be inspected again within 12 to 24 months of the repair and demonstrated to have arrested the crack before it may be considered repaired <u>and</u> inactive. Tack welds that have cracked but show no signs of additional crack growth into the base metal of the NSTM are not considered active fatigue cracks. In other words, cracked tack welds themselves are not a concern regarding this criterion if crack growth into the base metal is not present. Further, this criterion shall apply to any NSTM containing Condition State (CS) 3 for defect element 1010 (Cracking) (AASHTO, 2019). This defect element indicates the presence of cracking in steel elements that have not been repaired or arrested.

This screening criterion should also include distortion-induced fatigue cracking. An example of distortion-induced fatigue cracking is shown in Figure B.2 (Roddis & Zhao, 2003). This type of cracking is caused by an abrupt change in stiffness in the web in the region between the flange and transverse stiffener (Roddis & Zhao, 2003). There are many excellent references on this type of cracking available in the literature and the reader is encouraged to examine these documents. A bridge that simply possesses details that are susceptible to distortion cracks, but where no cracks have been observed, is exempt from this criterion.

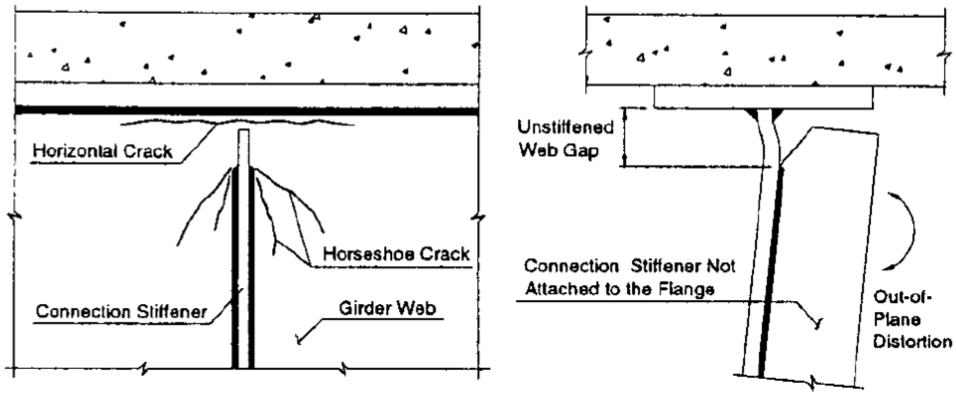

Figure B.2. Diagram of details susceptible to distortion-induced fatigue cracking (Roddis & Zhao, 2003).

Table B.7. Criteria and scoring for Active Fatigue Cracks.

Criteria	Interval and Points
Active Fatigue Cracks In NSTM(s)	≤ 24 months & 5 points
No Active Fatigue Cracks in NSTMs	0 points

1.6 - Susceptibility to Constraint-Induced Fracture

Reason(s) for Screening:

The failure of the Hoan Bridge in December 2000 in Milwaukee, Wisconsin due to constraint-induced fracture (CIF), and similar fractures in other bridges is the reason this criterion is in the Screening Phase of this assessment (Fisher et al., 2001). Other bridges with similar details as the Hoan Bridge are considered as potentially at risk for this form of brittle fracture. It is realized that frequent inspection will not prevent these failures from occurring (the Hoan Bridge was inspected three days prior to the failure). However, during the Purdue University Workshop cited earlier, it was agreed that extending the inspection interval beyond 24 months for bridges possessing these details cannot be justified. Hence, the inspection interval resulting from this assessment is limited to a maximum of 24 months. This criterion shall accordingly act as an incentive and encourage owners to repair/retrofit bridges with these types of details.

Assessment Procedure:

A detailed hands-on inspection is required to identify the presence of details susceptible to CIF. Details susceptible to CIF are discussed in the following references (Coletti, 2021; Connor et al., 2007; Fisher et al., 2001; Mahmoud et al., 2005).

Table B.8. Criteria and scoring for CIF details.

Criteria	Interval and Points
Details susceptible to CIF	≤ 24 months & 5 points
NO Details susceptible to CIF	0 points

1.7 - Existing Maintenance Problems / Posted

Reason(s) for Screening:

Bridges with existing maintenance problems or concerns may lead the owner to believe inspection intervals greater than 24 months are not appropriate. The presence of existing maintenance problems in bridges with NSTMs is perceived to result in lower reliability. Bridges may also be screened out based on the engineer's judgment with this attribute. This attribute may be used at the owner's discretion and is provided to be a "catch all." This simply means that a bridge with "problems" should be screened out by this criterion and thus prevented from being inspected at an interval exceeding 24 months. While this screening criterion does not necessarily apply to NSTMs or fatigue and fracture (it applies to the entire bridge), the intent is to prevent "troubled" bridges that possess NSTMs from being inspected at too great of an interval.

Assessment Procedure:

This should apply to bridges with NSTM(s) that have existing maintenance problems. The following list provides some examples of maintenance problems, but others may be included at the engineer's discretion. Examples of maintenance problems that may require screening include:

- Damaged or non-functioning bearings.
- Collision damage to NSTMs.
- Corrosion deemed unsafe (may also apply to part 1.8).
- 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 based on the judgement of the owner and engineer. Therefore, a bridge with what is determined to be "minor" damage to bearings or "minor" collision damage does not necessarily have to be screened out. Again, this criterion is simply to provide a means for the engineer to limit the inspection interval to 24 months or less based on experience, engineering judgment, and common sense.

Along with this criterion, bridges with posted weight limitations should also be screened out. It was decided by the members of the Purdue workshop that posted bridges should have hands-on inspection intervals of 24 months or less.

Table B.9. Criteria and scoring for Existing Maintenance Problems / Posted.

Criteria	Interval and Points
Bridges with Maintenance issues or Posted Bridges	≤ 24 months & 5 points
Bridges with NO Maintenance issues	0 points

1.8 - Condition of NSTM

Reason(s) for Screening:

Bridges with NSTM(s) in poor condition are to be screened out. This criterion is to prevent, for example, excessively corroded NSTMs (i.e., those with section loss) from going uninspected for more than a 24-month interval. It is recognized that in many cases, the condition of the NSTM 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 “1.7 Existing Maintenance Problems” as that criterion relates to the entire bridge span while 1.8 relates to the individual NSTM. It is noted that the phrase “Condition of NSTM” also includes the connections. For example, corrosion, pack-out, or distortion in gusset plates should be considered in this appraisal. Other detrimental issues related to connections of NSTMs must be included.

Assessment Procedure:

The determination of the NSTM condition is based on the Coding Guide or SNBI condition assessment. (FHWA, 1995, 2022b). According to the NBIS, NSTMs with a CR of 4 or less require an inspection interval of less than 24 months.

This assessment allows and encourages the use of element level data (AASHTO, 2019). Therefore, an alternative or supplement to the National Bridge Inventory (NBI) rating is the element level data; any NSTM with quantities of CS 4 for the member should be screened out by this criterion, even for NSTMs with CR ≥ 5. This criterion is not intended to be applied to coatings on the member.

Any NSTM with a CR ≥ 5 may “pass” this screening criterion.

Table B.10. Criteria and scoring for Condition of NSTMs.

Criteria	Interval and Points
NSTM(s) with NBI rating ≤ 4 OR any quantity of CS 4	≤ 12 months & 5 points
NSTM(s) with NBI rating > 4 OR CS ≤ 3 (any quantity)	0 points

Attachment B
Assessment Commentary - Part 2 – Scoring Phase

2.1 – Fracture Control Plan (FCP)

<u>Reason(s) for Consideration:</u>

Bridges designed and fabricated to the AASHTO/AWS FCP are believed to be less likely to experience a brittle fracture during their service life than those fabricated prior to the adoption of these requirements. These bridges meet more stringent material specifications and shop inspection requirements and are therefore perceived to be less susceptible to brittle fracture (Connor et al., 2005).

<u>Assessment Procedure:</u>

The FCP criterion should apply to bridges that are known to be designed and fabricated to meet the AASHTO/AWS FCP. If there is doubt or no documented proof that the bridge was constructed under the FCP, then 20 points shall be awarded for this attribute.

Table B.11. Criteria and scoring for Fracture Control Plan.

Criteria	Scoring (points)
Bridges fabricated to the FCP	0
Bridges NOT fabricated to the FCP	20

<u>Relative Importance:</u>

The excellent performance record of NSTMs that were designed and fabricated according to the FCP suggests that the plan is serving its intended purpose. Hence, it seems reasonable that bridges designed and fabricated to the FCP should be distinguished from those which are not. Because the FCP controls materials, fabrication, and shop inspection, and occurs on the "front end" of the bridge design process, this attribute is viewed as an optimum quality to possess. As a result, the RAP determined that this attribute should be weighted heavily. Therefore, bridges not meeting the FCP criteria are assigned 20 points.

2.2 – Temperature Zone

<u>Reason(s) for Consideration:</u>

As is well known, temperature affects the toughness of steel and its resistance to brittle fracture. Bridges constructed in warmer climates will never be subjected to the conditions experienced by bridges in colder climates. It is for this reason that minimum service temperature, as determined from the AASHTO LRFD Bridge Design Specification temperature zones, is included as an attribute in the assessment.

Although the AASHTO/AWS FCP addresses temperature zones through requiring higher toughness of steels in colder climates, it would only apply to those bridges which were designed and fabricated after the plan was implemented. Hence, it was agreed that older bridges not meeting the FCP in colder climates should be assessed differently than those located in warmer climates.

One could argue that this factor should not be considered for those bridges which meet the FCP. However, using the same reasoning as above, it was agreed that although a given bridge may have been designed and fabricated to the FCP, those located in cold climates are still generally more susceptible to brittle fracture in the presence of flaws.

Assessment Procedure:

The three AASHTO temperature zones across the United States should be used to determine the temperature score of the bridge, see pg. 6-64, Table 6.6.2.1-2 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2020).

Table B.12. Criteria and scoring for Temperature Zone.

Criteria	Scoring (points)
Bridges located in Zone 1	0
Bridges located in Zone 2	5
Bridges located in Zone 3	10

Relative Importance:

Although colder climates are less advantageous, it was not deemed to be overly influential. Therefore, a bridge in Zone 3 is assigned 10 points. This is attributed to the fact that brittle fracture can still occur during warmer temperatures. For example, the SR 422 Bridge in Pennsylvania failed during the month of May when the minimum temperature was in the mid 50's (Kaufman, 2004). Nevertheless, it was believed the climate where the bridge is located should be accounted for and included in the assessment. No points are added for bridges located in Zone 1. Zone 2 was assigned 5 points by taking the average of Zone 1 and Zone 3.

2.3 – Average Daily Truck Traffic – Single Lane ($ADTT_{SL}$)

Reason(s) for Consideration:

It is well established that ADTT plays a significant role in the probability of brittle fracture occurring through the initiation and propagation of fatigue cracks due to the high number of cycles applied. However, ADTT is viewed as a distinct attribute that is independent of fatigue *resistance*. In other words, ADTT is considered on the "load" side of the equation, while the fatigue detail category (i.e., category A, B, C, etc.,) is on the "resistance" side. It is also important to note that although a bridge may possess details with low fatigue resistance (e.g., E or E') due to how the member was detailed; these locations may not necessarily be subjected to high stress ranges, even though the ADTT may be significant.

The primary purpose of this attribute is to consider the frequency of loading. The $ADTT_{SL}$ attribute is simply to account for how quickly damage may accumulate and to categorize bridges into "heavy," "moderate," and "light" levels of loading frequency due to $ADTT_{SL}$.

Assessment Procedure:

The $ADTT_{SL}$ score depends on whether the data were obtained through estimates/calculations or actual field traffic counts (e.g., weigh-in-motion data). Field measured values of $ADTT_{SL}$ receive fewer points because of the greater certainty associated with these data. Since estimates are merely approximations, confidence in the data decreases and therefore greater points are awarded. To receive the lower "field measured" score, a traffic count at the bridge must be performed. If the count is performed "upstream" of the bridge and yet represents the actual $ADTT_{SL}$ the bridge experiences, then the engineer shall note the location and distance of the traffic count relative to the bridge and justify the accuracy of the count. In other words, if no on-ramps or off-ramps exist between the count location and the bridge, then the count will be accurate because the same traffic must use the bridge and no further (uncounted) traffic may access the bridge.

The calculation of the $ADTT_{SL}$ for all cases should use Section 3.6.1.4.2 in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2020). For bridges where only the Average Daily Traffic (ADT) is known, it is permitted to estimate the $ADTT_{SL}$ using Table C3.6.1.4.2 (AASHTO, 2020). However, it is intended that the readily available data from the NBI be used to estimate the $ADTT_{SL}$.

Like section 2.4, bridges with extremely low $ADTT_{SL}$ (15 trucks per day (tpd) or fewer); receive even less points because of the improbability of fatigue cracking due to the fact there are such a small number of trucks crossing the bridge. In other words, it is to account for the conditions in those bridges (e.g., rural bridges) where the fatigue limit state essentially could not control. It is noted that for this criterion to apply, the bridge must continue to have routine biennial inspection. Any future increase in the $ADTT_{SL}$ above the threshold of 15 tpd would require a re-assessment using this procedure.

The $ADTT_{SL}$ limit of 15 tpd comes from the fact that for bridges where the ADT is less than 100 tpd, the ADT is typically not reported in the NBIS. During the previously mentioned Purdue University workshop, it was agreed that an ADTT of 15% (of the ADT) was a reasonably conservative estimate of the proportion of trucks crossing a typical low volume bridge. Hence, 15% of the lowest ADT reported in the NBIS (ADT = 100 vehicles per day (vpd) yields an ADTT of 15 tpd. The required ADT data are readily available in the NBIS and there is no distinction made between field measured and estimated data for bridges with an ADT less than 100 tpd.

Table B.13. Criteria and scoring for estimated ADT and ADTT.

Criteria	Scoring (points)
Bridges with $ADTT_{SL} \leq 15$ tpd or ADT < 100 tpd	0

Table B.14. Criteria and scoring for field-measured ADTT.

Criteria	Scoring (points)
Bridges with measured 15 tpd < $ADTT_{SL} \leq$ 100 tpd	5
Bridges with measured 100 tpd < $ADTT_{SL}$ < 1000 tpd	13
Bridges with measured $ADTT_{SL} \geq$ 1000 tpd	20

Table B.15. Criteria and scoring for estimated or calculated ADTT.

Criteria	Scoring (points)
Bridges with measured 15 tpd < $ADTT_{SL} \leq$ 100 tpd	10
Bridges with measured 100 tpd < $ADTT_{SL}$ < 1000 tpd	15
Bridges with measured $ADTT_{SL} \geq$ 1000 tpd	20

Relative Importance:

The relative importance of $ADTT_{SL}$ was based upon the overall consensus based on engineering judgment of the RAP. The lower bound value of 100 tpd was set to separate bridges in rural areas versus “moderately” traveled bridges. The upper bound value of 1000 tpd was obtained by simply increasing the lower bound limit of 100 tpd by an order of magnitude (x 10). It was included herein to create a boundary between “heavily” and “moderately” traveled bridges. The amount of truck traffic a bridge experiences typically plays a significant role with respect to crack propagation and is weighted more heavily.

2.4 – Bridges With Zero ADTT

Reason(s) for Consideration:

The assessment procedure accounts for the influence of various levels of ADTT in Section 2.3 However, due to the improbability of fracture of a member without truck traffic, i.e., ADTT = 0 tpd, bridges that have truck traffic are assigned additional points in the assessment. The likelihood of fatigue cracking and fracture (as a result of fatigue) is significantly greater for a bridge carrying trucks when compared with a bridge that does not carry truck traffic. zero.

Assessment Procedure:

If the bridge is on a roadway restricted to high occupancy vehicles, a parkway (where trucks are not permitted), or if truck traffic is permanently prohibited from the bridge, then the structure will satisfy this attribute. The classification would not be affected if emergency vehicles were occasionally permitted to cross the structure. Regular bus traffic generally does not produce significantly large stress ranges and may still be included in this category if deemed appropriate by the engineer. To be clear, a bridge with zero truck traffic shall be assigned 0 points from section 2.3 as well as assigned 0 points from section 2.4.

Table B.16. Criteria and scoring for Bridges with Zero ADTT.

Criteria	Scoring (points)
Bridges with ADTT = 0	0
Bridges with ADTT > 0	20

Relative Importance:

This binary attribute has been weighted as either zero or 20 points. This relatively large difference in weight has been assigned due to the previously mentioned unlikelihood of the occurrence of fatigue and fracture with such low stress ranges (zero truck traffic). This attribute could be added as a fourth category under the criteria used to assess the ADTT. However, it was believed that bridges with no trucks are special cases and should be considered separately. Thus, although additional points have been added for bridges with truck traffic, which compound points and may appear to "skew" the final score, the effect has been addressed in the calibration of the final assessment score. This large score is further justified when comparing bridges with truck traffic with bridges along high-occupancy vehicle roadways, where trucks are not allowed, because they are designed as typical highway bridges with truck traffic despite never experiencing such loading.

2.5 – High Performance Steel (HPS)

Reason(s) for Consideration:

Generally, brittle fracture is far less likely to occur in newer bridges fabricated using HPS in tension members designated as NSTMs. Furthermore, cracks are more likely to arrest in members fabricated from HPS.

Assessment Procedure:

If the primary tension members designated as NSTMs are not fabricated using HPS, this criterion should be assigned 20 points. All NSTMs members built from HPS should receive zero points for this attribute.

Table B.17. Criteria and scoring for the HPS attribute.

Criteria	Scoring (points)
NSTM(s) fabricated using HPS	0
NSTM(s) NOT fabricated using HPS	10

<u>Relative Importance:</u>

Since in most cases, steels classified as HPS will exceed, sometime significantly, the minimum levels specified in American Society for Testing and Materials (ASTM) A709 specifications, it is deemed to be an optimum quality. The toughness associated with steels that are not HPS, but still meet the specifications for non-redundant steel members is still considered to be a "very desirable" quality with respect to a member's integrity. For this reason, only 10 points are to be added for NSTMs that *do not* meet this requirement. During the development of the scoring methodology, it was deemed that there is no need to have a third category (e.g., adding 15 points) for steels that do not meet the FCP and hence, do not necessarily meet the A709 requirements for non-redundant members. Doing so was viewed as applying a double penalty for such cases.

2.6 – Condition of NSTMs

<u>Reason(s) for Consideration:</u>

The condition of the bridge's NSTMs is included in the assessment to account for the age and overall integrity of the bridge's load-carrying NSTMs. The current condition of the NSTMs is believed to significantly affect the general reliability of the bridge. For instance, assume two identical bridges exist, "Bridge A" and "Bridge B." The bridges are identical except that "Bridge A" has a NSTM with CR of 4 and "Bridge B" has a NSTM with CR of 8. In this scenario, "Bridge A" would be assigned points because of its poorer condition.

<u>Assessment Procedure:</u>

The value of the CR or element CS rating should be obtained from the most recent inspection report. It is noted that a hands-on inspection of the NSTM is required <u>prior</u> to the implementation of the assessment procedure described herein. This initial inspection is intended to identify any problems or conditions with NSTMs on the bridge that would put the integrity of the bridge in question. This inspection must be completed prior to making any considerations for extending the inspection interval beyond 24 months and is required for both new bridges and existing bridges.

It is well established that bridges exposed to deicing chemicals or harsh environments are more susceptible to corrosion-associated problems. As a result, more points are added when the bridge is exposed to chemical deicing agents due to the likelihood of accelerated corrosion and section loss. A "harsh environment" includes coastal regions and any other region the engineer deems a higher corrosive environment.

Table B.18. Criteria and scoring for NSTMs not exposed to deicing chemicals or harsh environments.

Criteria	Scoring (points)
NSTMs with a CR ≥ 7 or CS 1, 2	0
NSTMs with CR = 6 OR element CS 3 quantities	5
NSTMs with CR = 5 OR element CS 4 quantities	15

Table B.19. Criteria and scoring for NSTMs exposed to deicing chemicals for harsh environments.

Criteria	Scoring (points)
NSTMs with a CR ≥ 7 or CS 1, 2	5
NSTMs with CR = 6 OR element CS 3 quantities	10
NSTMs with CR = 5 OR element CS 4 quantities	15

Relative Importance:

The condition of NSTMs is considered a key factor when establishing the inspection interval. For this reason, it has been weighted 15 points. This relative importance was again established by the RAP. A CR of 5 or below has at best "minor section loss, cracking, spalling, or scour," (FHWA, 1995). NSTMs with even minor issues as just described should receive 15 points. An NSTMS with a CR of 7 and above is in good condition and is awarded 0 or 5 points, depending on whether the members are exposed to deicing agents.

2.7 – Internal Redundancy

Reason(s) for Consideration:

Uninhibited brittle fracture propagation in a steel member can obviously lead to complete failure of the component cross-section. In welded built-up shapes, there have been several examples of such failures in which fracture in one component propagated through a weld into another portion of the member. Recent research has shown that riveted or bolted built-up members provide significant resistance to the ability of a brittle fracture in one component to propagate entirely through a member and is therefore considered in this assessment. Thus, the likelihood of complete loss of load carrying capability for such built-up members is very low.

Assessment Procedure:

Internal redundancy is deemed to exist when a member is comprised of multiple elements that are mechanically fastened and a fracture that forms in one element cannot propagate directly into adjacent elements. Examples include riveted or bolted built-up girders, tie girders, tension members of a truss, split box sections with longitudinal bolted splices, the bracing system, laterals, and cross frames within a box member, and bolted continuous plates or shapes (AASHTO, 2022). Internal redundancy may also be applied to box caps that have beams framing into them if the longitudinal beams are composite with the deck.

All riveted members or any other members with internal redundancy as discussed above may receive the 8 points as shown below.

The user is made aware of the AASHTO "*Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members*" (AASHTO, 2022). This document provides a codified methodology on how to evaluate whether a member should be classified as an internally redundant member (IRM) or remain classified as an NSTM. The AASHTO IRM also contains explicit guidance on how to set a rational inspection interval for members which satisfy the provisions contained therein. Therefore, if a member is classified as an IRM, the inspection interval *and* inspection strategy for such members would not be governed by the provisions contained in this herein. This is because they are exempt from the traditional NSTM inspection requirements since they are not classified as NSTMs but rather IRMs.

Table B.20. Criteria and scoring for Internal Redundancy.

Criteria	Scoring (points)
Bolted or riveted built-up member. (NO analysis performed)	8
NSTM(s) – all other steel members	15

Relative Importance:

> *User Note:*
>
> *It is noted that only the interval of the inspection may be extended by the methodology herein. A hands-on inspection is still required for built-up NSTMs that have not been evaluated using the AASHTO IRM specifications. The advantage of utilizing the AASHTO IRM specification is that it may show that failure of an entire component can be tolerated. The member is no longer classified as an NSTM but rather an IRM in such cases. As a result, a traditional hands-on inspection is no longer required. Obviously, it is also far easier to detect completely broken components rather than small cracks which must be detected by the traditional hands-on NSTM inspection. A more economical inspection strategy and interval as calculated with the AASHTO IRM provisions can be realized when members are classified as IRMs.*

Built-up members were determined to be very desirable with respect to the bridge's integrity when compared to a welded built-up member. However, as stated, there is a distinction made between built-up members in general and those identified as *IRM* using the AASHTO IRM provisions. Built-up members that meet the AASHTO IRM provisions are no longer classified as NSTMs as stated above.

While a built-up member is viewed as advantageous even when an assessment was not performed, zero points are not assigned to these members since their actual in-service behavior is unknown. Therefore, at least eight points are assigned to all mechanically fastened built-up members when no calculations are made (i.e., the AASHTO IRM provisions were not used). For all other members, 15 points are assigned (e.g., welded built-up members, rolled shapes, etc.).

2.8 – Structural Redundancy

Reason(s) for Consideration:

One of the main problems with the NSTM designation is determining (with certainty) whether a bridge is justly classified as nonredundant. The NBIS defines an NSTM as "A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse" (FHWA, 2022a).However, redundancy is defined in the AASHTO LRFD Bridge Design Specifications as "The quality of a bridge that enables it to perform its design function in a damaged state" (AASHTO, 2020).

It can be argued that if a bridge truly possesses structural redundancy, then its members should not be classified as NSTMs, meaning it will not collapse upon the failure of one of its primary load-carrying members. However, many two girder bridges have continuous spans which would have traditionally been classified both as having NSTMs (the loss of one of the two girders would probably result in collapse) as well as structurally redundant (external static indeterminacy). This paradox is considered in the assessment, simply because in-service performance has demonstrated that structural redundancy decreases the potential for complete collapse after failure of a NSTM (Connor et al., 2005). In fact, no collapses were found in the research cited in the referenced NCHRP Report (Connor et al., 2005). Despite violating the definition of an NSTM, structural redundancy is ultimately considered because of the obvious benefits. For example, consider two identical multi-span bridges each with two main girders. One bridge has all simply supported spans and the other has continuous spans. While both are designated as having NSTMs, the bridge with continuous spans has a greater chance of surviving the fracture of one of its girders than the simply supported bridge.

Assessment Procedure:

Structural redundancy may exist for bridges with external static indeterminacy. This can occur in a two or more span continuous girder or truss. Note that only part of an end span between the fracture and the pier may be supported by structural redundancy and that the part of the end span at the abutment could theoretically collapse. However, the end spans of these bridges are still considered to benefit from this attribute per consensus among RAP members. This means that all spans, including the end spans, on a continuous span girder or truss may be considered structurally redundant in this assessment.

In the absence of any rigorous analysis, structural redundancy should be applied only to "typical" bridges. For instance, a bridge with 400-foot spans should not benefit from continuous span plate girders because of the extreme span lengths. Engineering discretion and judgment is to be used and justified for this attribute when determining what is and is not "typical." Because this attribute is only worth 10 points and the calibration for the scoring is set to be slightly conservative, engineering discretion will suffice in this situation.

Table B.21. Criteria and scoring for Structural Redundancy.

Criteria	Scoring (points)
Bridges with structural redundancy	0
Bridges without structural redundancy	10

Relative Importance:

> *User Note:*
>
> *In lieu of utilizing the provisions herein, owners may wish to utilize the AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members. The results of the evaluation shall define members as one of two types. Specifically, a Fracture Critical Member (NSTM) or a System Redundant Member (SRM). A SRM is defined as a steel primary member subject to tension for which the redundancy is not known by engineering judgment, but which is demonstrated to have redundancy through a refined analysis. SRMs need not be subject to the hands-on inspection protocols and intervals for NSTMs as described in the NBIS.*

The experience in steel plate girder bridges where girders have failed suggests that complete collapse is not the most likely outcome. This is even the case when end-spans of multi span units have experienced a fracture. Even though the actual performance of bridges where fractures have occurred, when there has been no thought given to the performance in the faulted state, has been exceptional, the lack of "perceived" redundancy was not weighted heavily. Therefore, the score assigned when the level of structural redundancy is unknown is comparatively low (10 points). Also, because this assessment is attempting to be a "one size fits all" approach to many different bridges, the effectiveness of structural redundancy will vary from bridge to bridge unless it is fully quantified.

2.9 – Remaining Fatigue Life

Reason(s) for Consideration:

Knowing the remaining fatigue life of a bridge enables engineers to understand the potential for the bridge to experience brittle fracture due to fatigue crack growth. It is for this reason included in the assessment criteria for inspection intervals.

Assessment Procedure:

Similar to the ADTT criterion, the fatigue life score depends on whether the fatigue life is calculated, or field measured. Calculated fatigue life is often very conservative and inaccurate and accordingly assigned fewer points. Fatigue life established through field testing is awarded more points due to the higher accuracy associated with field measurements. Assigning more points for field measured life may seem counter intuitive since predicted fatigue lives are generally longer when field data are used. However, because calculated life is notable for being overly conservative and can even result in negative life values, it makes sense to assign more points to the more accurate (field measured) values.

The 25-year cutoff point was obtained from a NYSDOT study to establish criteria for evaluating inspection intervals for FC bridges. Despite the arbitrary nature of this cutoff point, it was determined to suffice for this assessment based on the consensus of the attendees at the Purdue University workshop. Again, this attribute is intended to distinguish between bridges with sufficient remaining life and those thought to be "questionable," which here is called below 25 years.

Table B.22. Criteria and scoring for Remaining Fatigue Life established through field testing.

Criteria	Scoring (points)
Bridges with Remaining Fatigue life ≥ 75 years	0
Bridges with 25 years < Remaining Fatigue life < 75 years	5
Bridges with Remaining Fatigue life ≤ 25	10

Estimated or Calculated:

Table B.23. Criteria and scoring for Remaining Fatigue Life determined by calculation.

Criteria	Scoring (points)
Bridges with Remaining Fatigue life ≥ 75 years	0
Bridges with 25 years < Remaining Fatigue life < 75 years	4
Bridges with Remaining Fatigue life ≤ 25 years	7

Relative Importance:

While infinite fatigue life is a desirable quality, during the expert panel discussion it was agreed that it is not a factor that should be overly weighted. Hence, it is only assigned a total range of 10 points. Fatigue life is separate from ADTT for the cases that experience large ADTT values with infinite remaining life versus cases with low ADTT with finite remaining life. In these cases, the two criterion scores will "offset" each other, but they can also compound with one another. This assessment was calibrated such that fatigue life and ADTT (cycles (N) vs. stress range (S_r)) each play independent roles and can thus account for bridges subjected to varying levels of truck traffic and magnitude of stress range.

2.10 – Fatigue Detail Category

Reason(s) for Consideration:

The type of Fatigue Detail Category for a bridge's primary load-carrying NSTMs is somewhat related to the probability of the occurrence of brittle fracture. Very poor details are more likely to develop fatigue cracks on a given bridge. This is implied in the current AASHTO LRFD Bridge Design Specifications, which discourages the use of details lower than Category C and encourages design for infinite life (AASHTO, 2020). Fortunately, since the introduction of the modern AASHTO fatigue provisions in the early 1970's, the use of poor details, such as D, E, and E' has decreased substantially. Hence, in newer bridges, these details will likely be less common. Nevertheless, fatigue cracks, if left unchecked and permitted to grow, can result in brittle fracture (Connor et al., 2005).

Assessment Procedure:

The "worst" type of detail used on the NSTM under consideration shall be used.

Table B.24. Criteria listing the detail category and scoring for Fatigue Detail Category.

Criteria	Scoring (points)
A, B, OR B'	0
C	5
D	10
E OR E'	15

Relative Importance:

Detail classification was viewed as being an important factor in influencing the likelihood of fatigue cracking that may lead to fracture. Hence, a range of 15 points has been assigned for this attribute. The number of points earned increases with decreasing fatigue resistance. The performance and integrity of bridges vary widely depending on their type of connection details. It was believed that riveted connections, Detail Category D, should receive some points because of its possible member redundancy characteristics. Hence ten points should be assigned to all Category D details, including holes that are not rivet holes. Also note that plug welds and discontinuous backup bars should have been screened out at this point in the assessment and receive zero points for the detail category attribute.

2.11 – Tack Welds

Reason(s) for Consideration:

The existence of tack welds on NSTMs, although common, is deemed to be a quality that is perceived as having a negative impact on the fatigue performance of the NSTM. NCHRP Report 721 has shown that tack welds may be classified as Category C details and hence, the actual performance is quite well (Bowman, 2012). Nevertheless, due to the perception associated with tack welds and the wide range in the actual quality, the expert panel agreed that some points should be assigned when tack welds are present in addition to those associated with the normal Detail Category in Item 2.10. Therefore, this attribute is evaluated separately.

Assessment Procedure:

NSTMs with no tack welds receive zero points while those with tack welds receive five points.

Table B.25. Criteria and scoring for Tack Welds.

Criteria	Scoring (points)
NSTMs with NO tack welds	0
NSTMs with tack welds	5

Relative Importance:

A NSTM that possess tack welds is assigned five points to reflect the relatively low influence of this attribute.

2.12 – Owner's / Engineer's Discretion

An additional five points can be removed to the total assessment score at the owner's discretion. Because of the qualitative scoring procedure that is calibrated to be slightly conservative, the subtraction of five points based on engineering judgment is allowed. These points are intended to apply to those "borderline" cases where only a few points are needed to bump the inspection interval into the next bracket. However, it is noted that the subtraction of these points must be justified and clearly documented by the owner or his representative (i.e. a consultant that should be licensed). Some examples of justification for the subtraction of five "free" points are listed below:

- In cases where the engineer has inspected the bridge multiple times in the past and feels that reducing points is justified based on his past experience with the bridge: In a case such as this, it would be documented that there are no major issues with the bridge and explain how past experience and engineering judgment justifies the subtraction of the five points.
- In cases where a bridge has been inspected at the same interval, say every 24 months, for a substantial period (*say 10 years*) with no apparent problems. However, using the assessment provided herein indicates a recommended inspection interval *less* than 24 months. If the subtraction of five points enables the bridge to be inspected at what it has "historically" been inspected at (i.e., 24 months), the owner may wish to subtract five points to maintain the same inspection interval. Again, the reasons and rationale for subtraction of these points must be fully documented.

Table B.26. Criteria and scoring at Owner's / Engineer's discretion.

Criteria	Scoring (points)
NSTMs with a justification for subtraction of points	0
NSTMs without a justification for subtraction of points	5

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