MnDOT Fracture Critical System Analysis for Steel Bridges

Barritt Lovelace, Principal Investigator
Collins Engineers, Inc.

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16. Abstract (Limit: 250 words) MnDOT has identified the need to develop a methodology to quantify when a steel bridge system is considered fracture critical based on Federal Highway Administration recommendations. The overall goal of this project was to research existing applicable fracture critical research papers and publications, develop a methodology for determining whether a bridge is fracture critical or not based on this research, then apply this methodology to six different fracture critical steel pier caps as a refined analysis. This refined analysis used advanced computer modeling software to develop a 3-D model of the bridge, and then simulated localized failure modes at the fracture critical steel pier caps. These simulated failures were used to determine the adequacy of load path redundancy to the remaining portion of the bridge and whether a catastrophic failure of the entire bridge would occur.			
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MnDOT Fracture Critical System Analysis for Steel Bridges

Final Report

Prepared by:

Jennifer Zink
MnDOT Office of Bridges and Structures

Cory Stuber
Barritt Lovelace
Collins Engineers, Inc.

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EXECUTIVE SUMMARY

The current National Bridge Inspection Standards (NBIS) definition for a fracture critical member is "a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse." A fracture critical member lacks redundancy when there is no alternative load path or member to which the failed member can transfer its load.

When a bridge meets this definition, a special Fracture Critical inspection is needed. These inspections require significantly more time and resources compared to a typical routine inspection and are costly to state and municipal highway agencies.

To determine if a fracture critical member is redundant, the Federal Highway Administration (FHWA) allows for a refined analysis. From the FHWA Technical Memorandum, Clarification of Requirements for Fracture Critical Members, dated June 20, 2012:

> *Modern analytical techniques have provided a means for engineers to more accurately assess bridge redundancy and identify fracture critical members, with full consideration of 3-D system behavior in damage scenarios. It is no longer necessary to identify FCMs by simple checking for load path redundancy alone, unless the State chooses to maintain such criteria. To demonstrate that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a totally fractured FCM, a State must submit through the Division Office to the FHWA Office of Bridge Technology for review the detailed analysis and evaluation criteria that will be used to conduct the study. Once reviewed, these criteria can then be employed by the State systematically on their inventory.*

If the refined analysis can demonstrate that redundancy has been met, as defined by the evaluation criteria and without a partial or entire collapse of the bridge, then the member no longer needs to be identified as fracture critical.

However, AASHTO has not yet codified any method for this refined analysis. From Load Resistance Factor Design (LRFD) 6th Edition, section 6.6.2:

> *The criteria for a refined analysis used to demonstrate that part of a structure is not fracture-critical has not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of models and choice of element type should all be agreed upon by the Owner and the Engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the Owner and the Engineer.*

Thus, the Minnesota Department of Transportation (MnDOT) has identified the need to develop a refined analysis methodology to quantify when a steel bridge system is considered fracture critical based on loads, existing conditions, material properties, redundancy, and bridge configurations as recommended by the FHWA.

Specifically, this project took into consideration the redundancy of six fracture critical integral steel pier cap structures. Among these six, two different types of pier cap geometric configurations were evaluated. This included a "hammerhead" and "outrigger" style pier cap.

A hammerhead pier cap typically bears on a single column with several of the main bridge girders cantilevered out from the center. In contrast, outrigger pier caps generally have two columns, one at each end of the pier cap, creating a simple span, and are primarily used to span larger lengths. Outrigger pier caps are commonly associated with complicated roadway and bridge layouts where a ramp may interfere with a roadway directly below. The cross-sectional geometry of these six pier caps included a built-up steel box or I-girder shape.

Below is a summary of the six fracture critical pier caps analyzed for this project:

Bridge No.	Pier	Location	Configuration	Cross-sectional Geometry
27048	1	I-494 EB Ramp to Lindau Lane over TH 77	Hammerhead	Box Cap
27726B	1A	I-94 EB Ramp over Lyndale Ave and UP RR	Hammerhead	I-girder
27726B	2A	I-94 EB Ramp over Lyndale Ave and UP RR	Hammerhead	I-girder
27788	1	I-394 EB Ramp over TH 100 NB	Outrigger	Box Cap
62853	1	I-35W Ramp to TH 36 EB over TH 280 NB	Hammerhead	I-girder
62853	2	I-35W Ramp to TH 36 EB over TH 280 NB	Hammerhead	I-girder

Development of a methodology to analyze these fracture critical elements was largely taken from previous research that had been published, which discussed methods for determining redundancy of fracture critical structures. This included:

- NCHRP Report 406: Redundancy in Highway Bridge Superstructures
- NCHRP Report 458: Redundancy in Highway Bridge Substructures
- TechBrief: Evaluation of Member and Load-Path Redundancy on the US-421 Bridge over the Ohio River
- NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture Critical Details
- NCHRP Report 776 Bridge System Safety and Redundancy
- NCHRP 12-87A Fracture Critical System Analysis for Steel Bridges

After a review of these publications, NCHRP Report 406: Redundancy in Highway Bridge Superstructures was considered the best available method for determining structural redundancy, and was used as the primary resource in determining the methodology used for MnDOT. This study takes into account several load factors and defines a bridge as safe if the following four requirements are satisfied:

1. Reasonable safety levels against member failure.
2. The bridge does not reach ultimate system capacity under extreme loading conditions.
3. The bridge does not produce large deformations under expected loading conditions.
4. The bridge is able to carry some traffic loads after damage to a component.

To meet these four requirements, a finite element model of each bridge was built with STAAD.Pro V8i (Select Series 5) computer software. This was used to simulate different failure modes in the six integral steel pier cap structures. After these modes of failure were simulated, deflections and, if necessary, stresses of the bridge elements were evaluated. Results of this analysis determined whether the box cap could be considered sufficiently redundant or if a localized failure would result in a total or partial collapse of the bridge.

Based on the publication research and the refined analysis, the following conclusions were made:

- Bridge 27048 – Pier 1 (hammerhead) was sufficiently redundant and does not need to be classified as fracture critical.
- Bridge 27048 has a "Load Path Redundant" diaphragm adjacent to each side of the pier cap as part of the original construction of the bridge, which largely contributed to a redundant load path.
- Bridge 27726B – Pier 1A (hammerhead) was not sufficiently redundant and should remain classified as fracture critical.
- Bridge 27726B – Pier 2A (hammerhead) was not sufficiently redundant and should remain classified as fracture critical.
- Bridge 27788 – Pier 1 (outrigger) was not sufficiently redundant and should remain classified as fracture critical.
- Bridge 62853 – Pier 1 (hammerhead) was not sufficiently redundant and should remain classified as fracture critical.
- Bridge 62853 – Pier 2 (hammerhead) was not sufficiently redundant and should remain classified as fracture critical.

Based on the results of the analysis, the following recommendations are made:

- Two types of integral steel box caps were analyzed, this included a "hammerhead" and "outrigger" style. Outrigger style box caps have a very low probability of being redundant, therefore, hammerhead should be the primary focus of any future fracture critical analyses.
- Retrofitting current fracture critical pier caps with a load path redundant diaphragm, similar to Bridge 27048, may be an effective method for providing load path redundancy.

CHAPTER 1: INTRODUCTION

1.1 Purpose and Scope

The increased maintenance and inspection costs are a concern for existing fracture critical bridges in the State of Minnesota. These additional costs could be eliminated or minimized by re-examining fracture critical designations and reclassifying bridges where justified. Fracture critical bridges on both the state and local system could be reclassified as indicated in the June 20th, 2012 FHWA Technical Memorandum, *Clarification of Requirements for Fracture Critical Members*, which states on page 3:

> *Currently available refined analysis techniques have provided a means to more accurately define fracture critical members for new designs and to re-evaluate existing bridge members that were previously classified as fracture critical on the record design documents. If refined analysis demonstrates that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a totally fractured member (by structural redundancy), the member does not need to be considered fracture critical for in-service inspection protocol.*

The overall goal of this project is to utilize refined analysis techniques under the AASHTO Load Resistance Factor Design Manual (LRFD) Section 6.6.2 and the NCHRP Report 406 on specific structure types on the Minnesota bridge system, particularly steel pier caps, to determine structural redundancy. This refined analysis demonstrates if a structure has adequate strength and stability sufficient to avoid partial or total collapse and therefore does not need to be considered fracture critical any longer. Structures of this type included currently designated fracture critical bridges that likely exhibit structural redundancy, such as steel pier caps, steel arches, and/or two-girder steel systems.

1.2 Overview of Research

1.2.1 Literature Review

A critical review was conducted of existing specifications, technical literature, owners and industry experiences, NCHRP Report 406: Redundancy in Highway Bridge Superstructures, and ongoing NCHRP research, as well as the System Factors in the AASHTO Manual for Bridge Evaluation along with MnDOT manuals for bridge design and inspection, technical memorandums, and other related information supplied by the MnDOT Bridge Office.

1.2.2 Research Methodology

A research methodology was proposed to quantify when a steel bridge system should be considered fracture critical. The research methodology took into consideration the following: live load capacity before and after fracture, load combinations, fatigue, bridge age and existing conditions (e.g., corrosion, damage, and cracks), material properties, and transverse and longitudinal bridge configurations.

1.2.3 Proposed Specifications

Recommended MnDOT specifications were presented and are detailed in Chapter 4 of this report. These proposed specifications were used in the evaluation of existing bridges to determine if a bridge system was fracture critical in regards to in-service inspection protocol.

1.2.4 Refined Analysis Results

Four bridges designated as fracture critical were chosen for further study. There were a total of six fracture critical pier caps within the chosen set of bridges. This set of bridges included a wide range of structure types and geometric layouts. Refer to Table 1 for an overview of the selected fracture critical bridges.

Table 1-1 Selected Fracture Critical Bridges

Bridge	Fracture Critical Element			Bridge Layout		
	Cap Type	Cap Location	Pier Type	Total # Spans	Total # Girders	Geometry
27048	Integral Steel Box Cap	Pier 1	Hammerhead	4	6	Curved
27726B	Integral Steel I-Girder Cap	Pier 1A & Pier 2A	Hammerhead	11	3	Curved
27788	Integral Steel Box Cap	Pier 1	Outrigger	2	3	Straight
62853	Integral Steel I-Girder Cap	Pier 1 and Pier 2	Hammerhead	3	4	Span 1 Curved, Spans 2 and 3 Straight

1.2.5 Discussion and Conclusions

A review of the research methodology was discussed with MnDOT. Based on these discussions and preliminary findings from the analysis, it was determined that the outrigger box caps had a very low potential for a change in the fracture critical designation. Thus, efforts were primarily focused on hammerhead style piers.

CHAPTER 2: LITERATURE REVIEW

The following is a critical review of existing specifications, technical literature, owners and industry experiences, National Cooperative Highway Research Program (NCHRP) reports, and ongoing NCHRP research, as well as the System Factors in the AASHTO Manual for Bridge Evaluation along with State manuals for bridge design and inspection, technical memorandums, and other related information supplied by the MnDOT Bridge Office.

2.1 NCHRP Report 406: Redundancy in Highway Bridge Superstructures

Current bridge design specifications design bridge components individually and tend to ignore system effects which results in conservative designs. NCHRP Report 406 attempts to address redundancy and provide a link between the system effects and individual component design. The objective of the study was to develop a basis for incorporating redundancy in the design and load capacity evaluation of highway bridge superstructures. It outlines the incorporation of system factors which determine the member capacity of the bridge system as a function of its level of redundancy. The system factors are statistically based multipliers applied to the nominal member resistance and are related to the redundancy of the bridge superstructure. The proposed methodology was calibrated using reliability techniques.

The study defines a bridge as safe if the following requirements are satisfied:

1. Reasonable safety levels against member failure.
2. The bridge does not reach ultimate system capacity under extreme loading conditions.
3. The bridge does not produce large deformations under expected loading conditions.
4. The bridge is able to carry some traffic loads after damage to a component.

These requirements are checked against several limit states including member failure, ultimate capacity of the bridge system, maximum acceptable live load displacement, and ultimate capacity of the bridge system after damage to a main load carrying component.

The framework can be applied during the design of new bridges or the load rating of existing bridges. Predetermined system factor tables were developed to cover typical configurations of prestressed concrete and steel bridges. For configurations not covered in the tables, a direct analysis approach was developed. This approach calculates redundancy factors from the results of a nonlinear incremental finite element analysis.

The following points highlight several conclusions of the report which are applicable to this project:

- A parametric analysis confirms that the redundancy of the bridges used for calibration of the proposed framework is a function of the geometric configuration and is not very sensitive to the variations in the section properties.

- Tables were created for a limited number of specific bridge types. For bridges that are not covered by the tables a direct analysis approach is recommended. Guidelines to perform this analysis are provided in the study.
- The study is concerned with the redundancy of highway bridge superstructures. The general framework is also applicable to substructures.
- Many bridges have unique permit load and posting requirements that would make the general database used for the calibration of design codes, and by extension the proposed system factor framework, unsuitable for use when performing the evaluation of existing bridges.

The recommendations from this study have been applied by several consulting firms in cooperation with the Wisconsin Department of Transportation for the safety analysis of existing fracture critical box girder bridges. The specifications developed as part of this study have not been adopted as part of the AASHTO LRFD Specifications pending more investigation to simplify the format and further confirm the validity of the approach.

2.2 NCHRP Report 458: Redundancy in Highway Bridge Substructures

The objective of the study was to develop a basis for incorporating redundancy in the design and load capacity evaluation of highway bridge substructures. The framework for system factors is similar as the one outlined in NCHRP Report 406. Predetermined system factor tables were developed to cover typical configurations of two-column and four-column bents.

The study considered several limits state to ensure redundancy in the substructure and structural safety. These limit states included the ultimate limit state, functionality limit sate, and the damaged condition ultimate limit state. The prediction of the ultimate strength of the substructure system is accomplished by performing a nonlinear analysis under incrementally applied loads. Plastic actions develop as the load is increased and the structural begins to behave nonlinearly. An example of this pushover analysis procedure is included in the study.

The following points highlight several conclusions of the report which are applicable to the current review:

- During evaluations of existing structures the difference between the as-provided strength (taking into account deterioration) and the required strength should be recognized and accounted for during the check of redundancy.
- Single-column bents are considered to be nonredundant and the lower limit of the system factor is recommended.
- If a substructure system failure is governed by shear it is considered to be nonredundant and the lower limit of the system factor is recommended.
- Integral-type connections between substructures and superstructures require special consideration.
- The study did not consider the possibility of soil failures but the effect of soil flexibility on the foundation was considered. (This study was produced before the Leo Frigo Bridge problems were encountered.)

- Four column substructures are considered adequately redundant and should be used as a target for designs.
- Most bridge substructures subjected to damage of one column do not exhibit redundancy since the pier cap is not typically designed with enough strength to transfer the loads to the remaining columns.

The specifications developed as part of this study have not been adopted as part of the AASHTO LRFD Specifications pending more investigation to simplify the format and further confirm the validity of the approach.

2.3 TechBrief: Evaluation of Member and Load-Path Redundancy on the US-421 Bridge over the Ohio River

The objective of the project was to study the after-fracture performance of a fracture critical two-line, simple-span truss bridge. The document was a technical summary of an unpublished Federal Highway Administration (FHWA) report. The project bridge was slated for demolition which offered the opportunity to study both internal and structural redundancy. The controlled demolition plan called to completely sever the bottom chord in a single location while monitoring the structural response.

The US-421 spanned the Ohio River and was also known as the Milton-Madison Bridge. One of the 149 foot approach Pratt trusses was selected for testing. The bridge span was then load tested and calibrated to a finite element model. Next the loaded span was subjected to a controlled demolition to sever the bottom chord of the truss. With 2/3 of the original design load the demolition of the bottom chord did not lead to the collapse of the structure. Based on the results of the study the authors called for the development of a modeling standard in order to avoid an unnecessary fracture critical designation. Some additional conclusions from the study include the following:

- The internal redundancy of a truss chord indicated that it might be effective at preventing collapse. That is, the instantaneous removal of an element from a built-up fracture critical member (FCM) did not overload the remaining parallel element enough to cause total member fracture.
- Total removal of an FCM in this particular truss did not result in the collapse of the structure. In fact, the bridge likely could have remained functional under normal service loads. However, based on the small vertical displacement observed, the deficiency is not perceivable to vehicular traffic and not likely to be noticed.
- A simple analysis model was able to conservatively predict the behavior of the truss in the faulted state.
- There was not enough instrumentation applied to the bridge to completely assess the load redistributed within the truss after the imposed fracture. Nevertheless, the filled grid deck and floor system presumably played a major role in secondary support of load, which was accurately demonstrated in the simple finite element analysis.

2.4 NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture Critical Details

The report is focused on the inspection and maintenance of bridges with fracture-critical members. The objectives were to survey the extent of and identify gaps in the literature, determine best practices for owners of bridges with FCMs, and identify research needs.

Surveys were sent to state DOT's and Canadian Provinces. Information was gathered regarding how bridge owners define, identify, document, inspect, and manage fracture critical bridges. This study presents the results of those surveys in an attempt to determine how states define fracture critical bridges, determine how they document collapses and fractures, determine the costs associated with inspection of fracture critical bridges, and identify areas of future research.

The following points highlight several conclusions of the report which are applicable to the current review:

- Modern bridges are much less susceptible to fatigue, corrosion, and fracture than bridges designed before 1975 (and 1985 for web-gap cracking) as a result of the fracture control plan and new fatigue specifications.
- The new high-performance steels (HPS) provide a level of toughness which research suggests could eliminate the need for special in-service inspection requirements for fracture-critical bridges.
- NCHRP Report 406 provides practical requirements for determining the capacity of damaged superstructures.
- Very few traditional fracture critical bridge members are being built.
- Hands-on fracture critical inspections have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. One example given is the fatigue crack discovered on the Lafayette Bridge in St. Paul, MN.
- Costs for fracture critical inspections are two to five times of routine inspections. These costs could be reduced by modifying inspection frequencies to account for things such as year built, traffic, and type of details.

2.5 NCHRP Report 776 Bridge System Safety and Redundancy

NCHRP Reports 406 and 458 developed guidelines for system safety and redundancy for bridge superstructures and substructures. NCHRP Report 776 Bridge System Safety and Redundancy was prepared in an effort to combine both previous studies into one system. This report also includes suggested specifications for the AASHTO LRFD Bridge Design Specifications and implementation examples.

The proposed system factors generated by this study can be used during the design and safety assessment of bridges. The system factors modify the LRFD design equation as follows:

$$\phi_s \phi R_n^N = \sum \gamma_i Q_i$$

Where
$R_n^N = Required\ Member\ Capacity$
$\phi_s = System\ Factor$
$\phi = Member\ Resistance\ Factor$
$\gamma_i = Load\ Factor$
$Q_i = Load\ Effect$

Equations for the system factors are presented with tables to aid calculation. Equations are given for bridges systems under distributed lateral load and vertical load and for both intact and damages systems. The results of this study simplify the equations and tables presented in NCHRP Report 406 and NCHRP Report 458. The study also further develops the specifications established in the previous studies and provides useful implementation examples.

2.6 NCHRP 12-87A Fracture Critical System Analysis for Steel Bridges

The objectives of this ongoing active research project are to (1) develop a methodology to quantify when a steel bridge system is considered fracture critical based on loads, existing conditions, material properties, and bridge configurations, and (2) recommend AASHTO specifications using the methodology in the design of new bridges and the evaluation of existing bridges. This study was started in September of 2014 and is scheduled to be complete in December of 2016. The progress of this study will be monitored throughout this project.

CHAPTER 3: RESEARCH METHODOLOGY

Redundancy is the ability of a bridge to continue to carry loads after failure of one of its members. This proposed methodology quantifies when a steel bridge system is considered to be fracture critical. By establishing that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a totally fractured member (structural redundancy), the member does not need to be considered fracture critical. While system factors may be used for common bridge types as outlined in NCHRP Reports 406 and 458, this methodology is for cases where a direct analysis approach is required. This method involves building a finite element structural model that considers elastic and inelastic behavior of the entire bridge system.

3.1 Limit States Summary

The following limit states should be considered in the analysis:

1. Member Failure
2. Ultimate Capacity
3. Functionality Conditions
4. Damaged Conditions

3.2 Safety Criteria

To be safe a bridge should satisfy the following:

1. Provide a reasonable safety level against first member failure.
2. Under extreme loading the ultimate capacity of the bridge system should not be reached.
3. Resist large deformations under expected traffic loads.
4. Be capable of carrying some traffic loads after damage or loss of a component.

3.3 Redundancy Check

3.3.1 Reliability Indices

As outlined in NCHRP Report 406, reliability indices are a measure of safety used in the development of the AASHTO LRFD specifications. They account for both the margin of safety of the design procedure and the uncertainties in establishing member strengths and applied loads. The relative reliability indices measure the relative safety provided by the bridge system compared with the nominal safety of first member failure. The relative reliability indices provide reliability based measures of redundancy and are defined for the ultimate limit state ($\Delta\beta_u$), functionality limit state ($\Delta\beta_f$), and the damaged limit state ($\Delta\beta_d$). The direct redundancy check procedure is based on satisfying minimum values of these relative reliability indices.

3.3.2 Load Factor Ratios

Load factor ratios are relative bridge capacities and are the ratio of system capacity with respect to member capacity. The required load factor ratios result from ensuring that the minimum

values of the reliability indices are exceeded a bridge has adequate levels of redundancy if all of the following conditions are met:

- For the ultimate capacity R_u greater than or equal to 1.3;
- For the functionality limit state R_f greater than or equal to 1.10; and
- For the damaged conditions R_d greater than or equal to 0.50.

These values assume the live load follows a log-normal distribution which covers live load capacity before and after fracture. Additionally, these values are valid for any number of lanes and for any load model including the HL-93 truck.

3.4 Step-by-Step Procedure

This section outlines a direct method to determine the redundancy level of a bridge system using a detailed nonlinear finite element analysis. These steps are similar to the procedure outlined in NCHRP Report 406: Redundancy in Highway Bridge Superstructures.

3.4.1 <u>Step 1</u>

Use AASHTO LRFD Specifications to find the member capacity (R_{req}) for all bridge elements identified for analysis, using the following equation:

$$\phi R_{req} = \gamma_d D_n + \gamma_l L_n(1 + I)$$

Where:
$\phi = Resistance\ Factor$
$\gamma_d = Dead\ Load\ Factor$
$\gamma_l = Live\ Load\ Factor$
$D_n = Nominal\ or\ Design\ Dead\ Load$
$L_n(1 + I) = Nominal\ or\ Design\ Live\ Load\ including\ Impact$

3.4.2 <u>Step 2</u>

Develop a structural model of the bridge to be used with a finite element package that allows static nonlinear analysis of the structure. Refer to Figure 3-1 for a typical view of a finite model of a bridge. In the model, use the best estimate of the nonlinear material properties of the structural members without applying any safety factors or strength reduction factors. Design plans and/or shop drawings should indicate steel used. The AISC Manual for Steel Construction can also be used to determine type of steel used based on year of construction. Apply the unfactored dead load based on design plans.
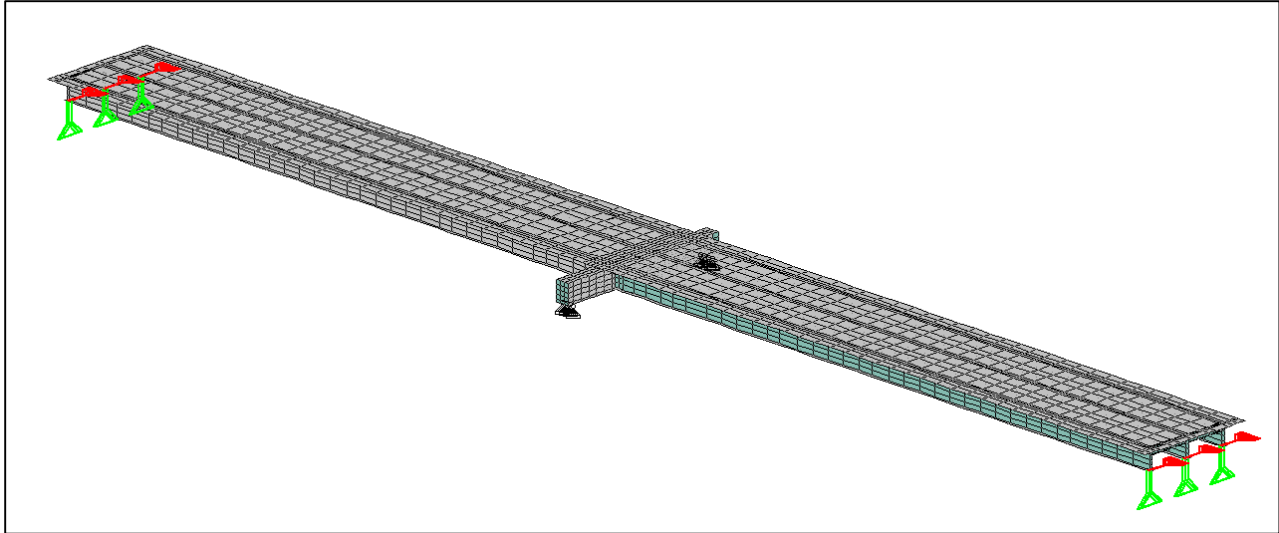
Figure 3-1 Bridge 27788 – TH 100 Ramp to TH 394 EB – Finite Element Model

3.4.3 Step 3

Identify transverse and longitudinal loading positions on the deck surface for the HL-93 AASHTO truck and lane loads to produce the most critical loading effect. Do not include impact factors.

3.4.4 Step 4

Apply the loads to the structure and perform a linear elastic analysis to calculate L_{HL-93} along with lane loads, which gives the effect of the AASHTO loading on the most critical member. Then, using the following equation, calculate the required member load factor LF_{1req}.

$$LF_{1req} = \frac{R_{req} - D}{L_{HL-93}}$$

3.4.5 Step 5

Increment the loads of the design vehicle until the first member reaches its limiting capacity. Note the load factor LF_1 by which the original HL-93 truck is scaled for the first member failure to occur. Then, using the following equation, calculate the member reserve ratio r_1.

$$r_1 = \frac{LF_1}{LF_{1req}} = \frac{R_{provided} - D}{R_{req} - D}$$

Where:
$R_{provided} = Provided\ Member\ Capacity$

3.4.6 <u>Step 6</u>

Using a nonlinear structural model and nonlinear material properties, increment the applied HL-93 truck until the maximum vertical deflection of a primary member reaches a deflection equal to span length/100. Note the load factor LF_f by which the original HL-93 truck are scaled to achieve the span length/100 displacement level. In the ratio $R_f = LF_f/LF_1$, if the ratio is greater than 1.1, then the bridge has a sufficient level of redundancy to satisfy the functionality limit state. Calculate the redundancy ratio for functionality r_f using the following equation:

$$r_f = \frac{R_f}{1.10}$$

3.4.7 <u>Step 7</u>

Increment the load further until the ultimate limit state is reached. The ultimate limit state is defined as the maximum possible truck load that can be applied on the structure before it collapse. Collapse is defined as the formation of a collapse mechanism or the point at which the structure is subjected to high levels of damage. A mechanism is the point at which the structure exhibits infinitely large levels of displacements rendering it unusable. In this section, damage is defined as the loss of the load-carrying capacity of a main member. The load factor calculated in this step is LF_u. If $R_u = LF_u/LF_1$ is larger than 1.30, then the bridge has a sufficient level of redundancy to satisfy the ultimate limit state. Calculate the redundancy ratio r_u using the following equation:

$$r_u = \frac{R_u}{1.30}$$

3.4.8 <u>Step 8</u>

Identify members whose failure might be critical to the structural integrity of the bridge. These scenarios may require consultation with the bridge owner. Examples include, but are not limited to the following: (a) members that can be damaged by an accidental collision by a vehicle, ship, or debris or (b) steel members that are prone to fatigue and/or are fracture critical.

3.4.9 <u>Step 9</u>

Remove or simulate a failure in one of the structural members identified in Step 8 from the finite element model and repeat the nonlinear analysis. Next, determine the load factor of the damaged bridge LF_d at ultimate. If the ratio $R_d = LF_d/LF_1$ exceeds 0.50, the bridge provides a sufficient level of redundancy. Finally, calculate the redundancy ratio for damage conditions r_d using the following equation:

$$r_d = \frac{R_d}{0.50}$$

3.4.10 Step 10

Place the member removed in Step 9 back into the model and removed another critical member. Repeat Step 9 until all the critical members identified in Step 8 are checked.

3.4.11 Step 11

Repeat Steps 3 through 10 to cover all critical loading cases. The scenario which produces the minimum value for LF_u, LF_f, LF_d, and LF_1 will control.

3.4.12 Step 12

If all the redundancy ratios r_u, r_f, and r_d obtained from the analysis are larger than 1.0, then the bridge has a sufficient level of redundancy.

If any redundancy ratio is less than 1.0, then the bridge does not have a sufficient level of redundancy and should be identified as fracture critical. In this case recommendations for retrofit should be made to increase the redundancy ratios.

CHAPTER 4: PROPOSED SPECIFICATIONS

The following proposed specifications include the recommended modifications to Article 1.3 of the AASHTO Design Specifications (AASHTO) and may be used for the LRFD MnDOT Bridge Design Manual. These modifications include additional language to define redundancy based on performing an advanced system analysis.

DESIGN PHILOSOPHY

4.1 General

Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety and serviceability, with due regard to issues of inspectability, economy and aesthetics, as specified in Article 2.5 of the AASHTO Specifications.

Regardless of the type of analysis used, Equation 1.3.2.1 -1 shall be satisfied for all specified force effects and combinations thereof.

4.2 Limit States

Note, the articles numbered below correspond to the articles from AASHTO.

1.3.2.1 General

Each component and connection shall satisfy Eq. 1.3.2.1-1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, for which the provisions of Article 6.5.5 shall apply. All limit states shall be considered of equal importance.

$$\sum \gamma_i Q_i \leq \varphi_s \varphi R_n = R_r \qquad (1.3.2.1\text{-}1)$$

Where:
φ_s=system factor: relating to ductility, redundancy and operational classification as specified in Article 1.3.6 for the design of structural components for strength and extreme event limit states. For all other limit states, the system factors shall be taken as 1.0
φ=resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections, 5, 6, 7, 8, 10, 11 and 12
γ_i=load factor: a statistically based multiplier applied to force effects
R_n = nominal resistance
R_r = factored resistance: $\varphi_s \varphi R_n$
Q_i = force effect

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1.3.2.2 Service Limit State

The service limit state shall be taken as restrictions on stress, deformation and crack width under regular service conditions.

1.3.2.3 Fatigue and Fracture Limit State

The fatigue limit state shall be taken as a set of restrictions on stress range due to a single fatigue truck occurring at the number of expected stress range cycles.

The fracture limit state shall be taken as a set of material toughness requirements of the AASHTO Material Specifications.

1.3.2.4 Strength Limit State

Strength limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

1.3.2.5 Extreme Event Limit States

The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle or ice flow possibly under scoured conditions.

1.3.3 Ductility

The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure.

Energy-dissipating devices may be substituted for conventional ductile earthquake resisting systems and the associated methodology addressed in these Specifications or in *the AASHTO Guide Specifications for Seismic Design of Bridges*.

1.3.4 Redundancy

The structural system of a bridge shall be configured and its members designed to ensure that it meets three system strength conditions: a) limited functionality and b) resistance to collapse if the strength of its most critical member is exceeded, and c) ability to carry some level of live load in a damaged state. Therefore, multiple-load-path, ductile and continuous structures should be used unless there are compelling reasons not to use them.

A system factor ϕ_s shall be applied during the design of bridge members to account for a bridge's system level of redundancy as specified in Article 1.3.6.

1.3.5 Operational Importance

This Article shall apply to the strength and extreme event limit states only.

The Owner may declare a bridge or any structural component and connection thereof to be of increased operational priority.

The Owner may also declare a structural component or connection to be damage-critical.

CHAPTER 5: REFINED ANALYSIS RESULTS

The refined analysis primarily considered integral steel integral or I-girder caps. However, the entire bridge was modeled and analyzed to determine how different load cases distributed as failure modes were simulated. The finite element model computer program STAAD.Pro V8i (Select Series 5) was used to analyze these failure modes. From the results of this analysis, it was determined whether the pier caps could be considered sufficiently redundant or if a localized failure would result in a total or partial collapse of the entire bridge.

Live loads were generated by the finite element model created in STAAD, using the BEAVA engine. BEAVA works by generating the worst case live load for the user identified areas whose failure may be critical to the structural integrity of the bridge.

Redundancy for the pier caps is accomplished if the value for R_d is greater than 0.50, as described in the Step-by-Step procedure in Chapter 3 of this report. This means that a live load factor equal to one-half of LF_1, for the damaged condition, must not deflect more than the maximum allowable deflection previously established for the analysis. This live load factor is used in the load case for the STAAD model damaged state:

- Live Load = 0.50 x LF_1 x Unfactored HL-93 Load Case from BEAVA
- Dead Load = Unfactored Deal Load of Model

5.1 Bridge 27048

5.1.1 <u>Summary</u>

Bridge 27048 is part of the off ramp which connects I-494 eastbound to Lindau Lane on the north end of the Mall of America and spans over TH 77. This structure is a continuous 4-span bridge comprised of six built up steel I-girders. Each span varies in length, with a maximum span length of 162 feet. Span 1 also has varied girder spacing. An integral steel box cap is utilized at Pier 1 and is bearing on a single reinforced concrete column. Two of the girders at this location bear directly over the column and the others are cantilevered out from the center. This style of pier is commonly referred to as a "hammerhead" shape. A "Redundant Load Path" diaphragm is offset 6'-6" on each side of the pier cap as part of the original design and construction of the bridge. The integral steel box cap is considered the only fracture critical element on the structure. Refer to Figure 5-1 for a view of the STAAD model. Refer to Figure 5-2 for a plan view of the "Redundant Load Path" diaphragm from the original design drawings.
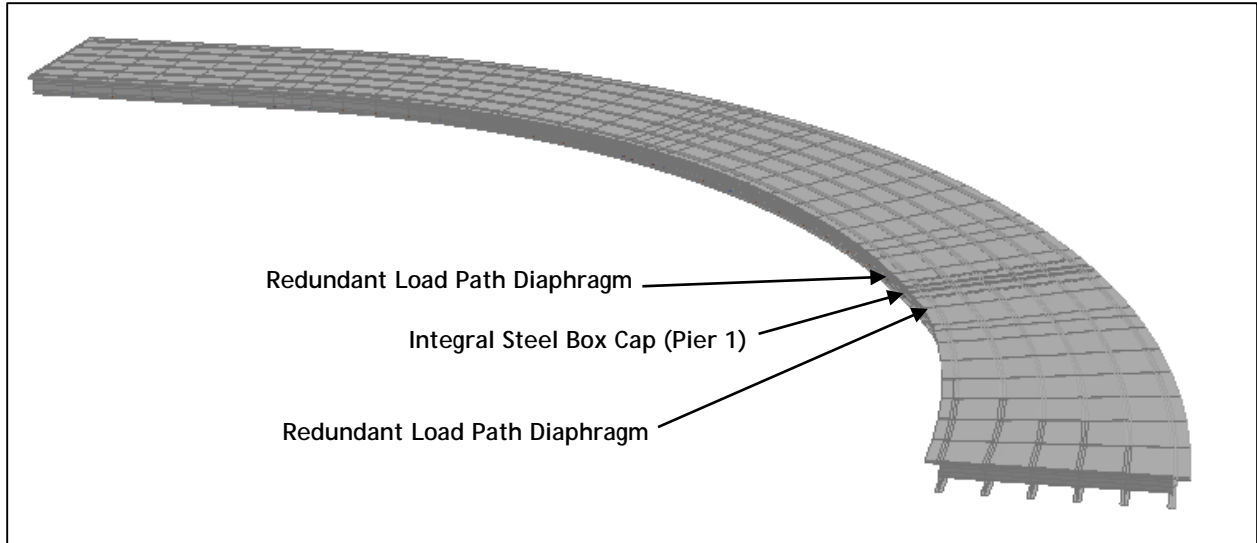
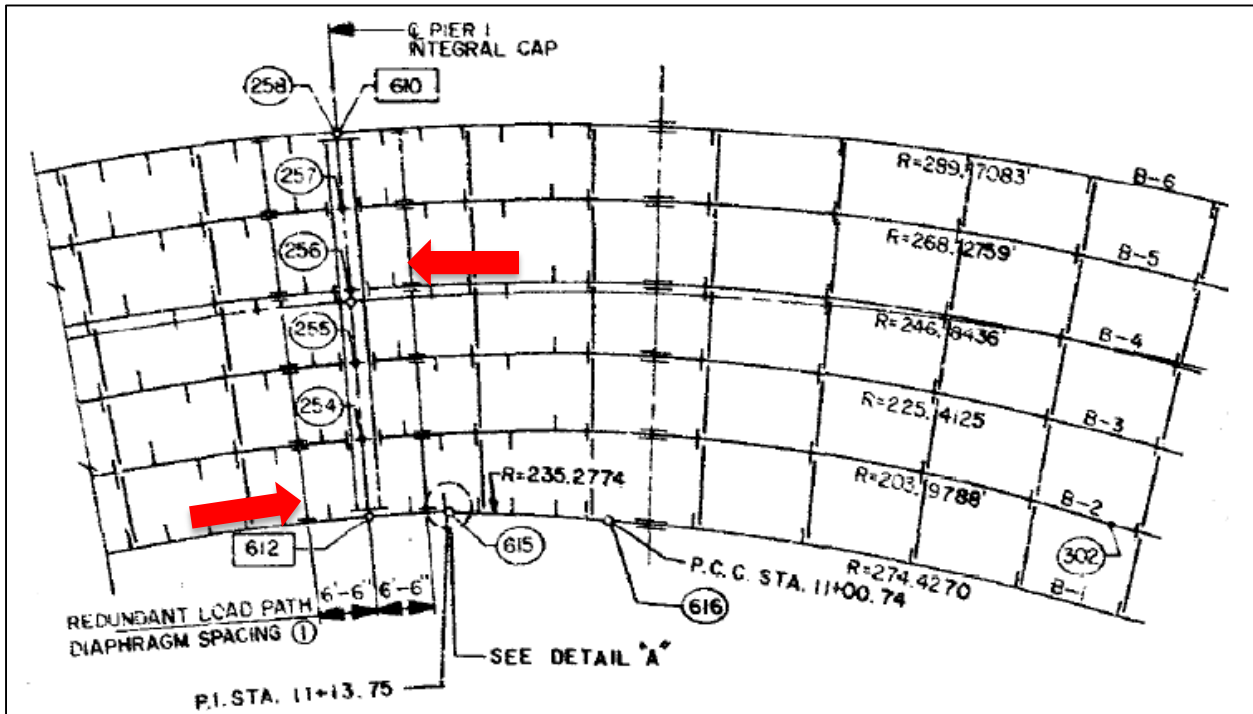*Figure 5-1 Bridge 27048 STAAD Model View*

Figure 5-2 Location of Redundant Load Path Diaphragm

5.1.2 Results

Refer to Figure 5-3 and Table 2 for an overview of results from the refined analysis. Two critical areas of the box cap were evaluated. The first assumed that a localized failure will occur at the approximate centerline of the bridge, between the two interior girders (A-A). The other assumed a localized failure occurs along the cantilever portion of the cap, at maximum negative moment (B-B). Each of these areas were selected using engineer judgment based on the maximum stresses produced by the BEAVA engine live load results. From this analysis, the following conclusions were made:

1. A localized failure along plane A-A will not result in a catastrophic collapse of the entire structure.
2. A localized failure along plane B-B will not result in a catastrophic collapse of the entire structure.

Since a catastrophic collapse will not occur at the critical locations located at the pier cap, and it meets redundancy criteria, then the pier cap does not need to be designated as fracture critical.
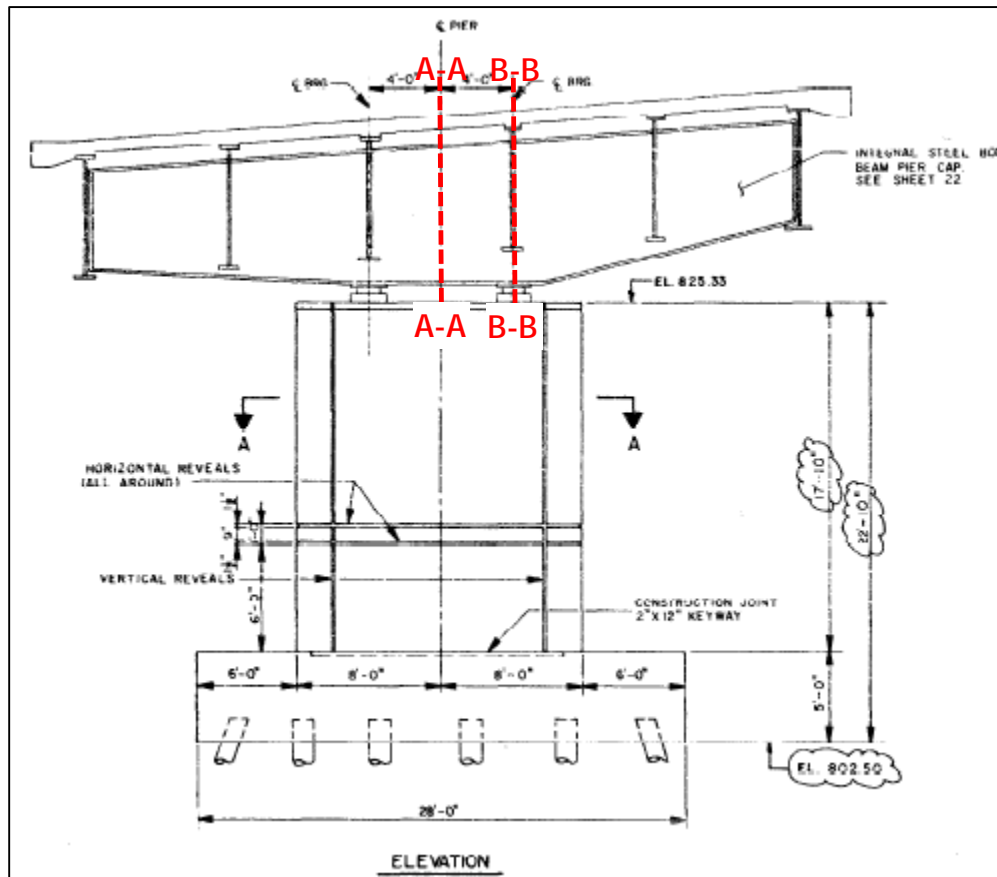
***Figure 5-3 Bridge 27048 Pier 1 Elevation with Failure Planes***

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Table 5-1 Bridge 27048 Analysis Results Overview

Failure Plane	Component	Live Load Factor	Deflection (in)	Allowable Deflection (in)	Maximum Stress (ksi)	Allowable Stress (ksi)
A-A	Web	11.74	1.38	4.80	29.5	65.0
A-A	Top Flange	11.74	1.39	4.80	56.4	65.0
A-A	Entire Section	11.74	1.64	4.80	21.3	65.0
B-B	Entire Section	11.74	1.36	4.80	19.3	65.0

5.2 Bridge 27726B

5.2.1 <u>Summary</u>

Bridge 27726B is the off ramp which connects I-94 eastbound to I-394 west bound ramp and spans over a BNSF railroad track, the Cedar Lake Trail, and Lyndale Ave North. This structure is a continuous 11-span bridge with three hinge locations. It is comprised of three built up steel I-girders. Each span varies in length from approximately 68 feet to 137.5 feet. An integral I-girder cap anchored to a single reinforced concrete column is utilized at Pier 1A and Pier 2A. This style of pier is commonly referred to as a "hammerhead" shape. The integral steel I-girder caps are considered the only two fracture critical elements on the structure. Refer to Figure 5-4 for a view of the STAAD model.


Steel I-Girder Cap (Pier 1A)

Steel I-Girder Cap (Pier 2A)
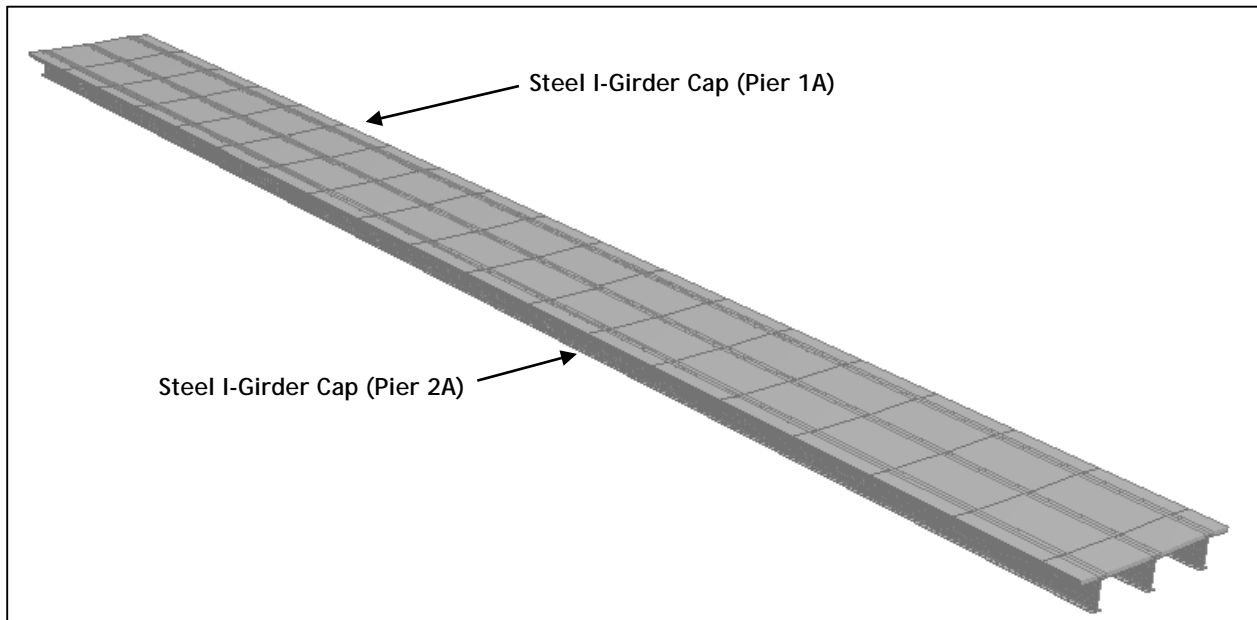
Figure 5-4 Bridge 27726B STAAD Model View

5.2.2 Results

Refer to Figure 5-5, Figure 5-6, and Table 3 for an overview of results from the refined analysis. Based on these results the following conclusions were made:

1. A localized failure along plane A-A for Pier cap 1A will <u>not</u> result in a catastrophic collapse of the entire structure based on deflection criteria only.
2. A localized failure along plane B-B for Pier cap 1A will likely result in a catastrophic collapse of the entire structure based on deflection criteria.
3. A localized failure along plane C-C for Pier cap 2A will likely result in a catastrophic collapse of the entire structure based on deflection criteria.
4. Since a localized failure along plane C-C will likely result in a catastrophic collapse, analyzing section D-D is not necessary.

Deflection criteria was first established, and based on the results, a catastrophic failure would likely occur along each failure plane (A-A, B-B and C-C). Given that deflection criteria failed, it is not necessary to consider stress values. Therefore, based the criteria used for redundancy in this analysis, and the likely hood of a catastrophic collapse occurring from a localized failure at Pier caps 1A and 2A, this bridge should remain classified as fracture critical.
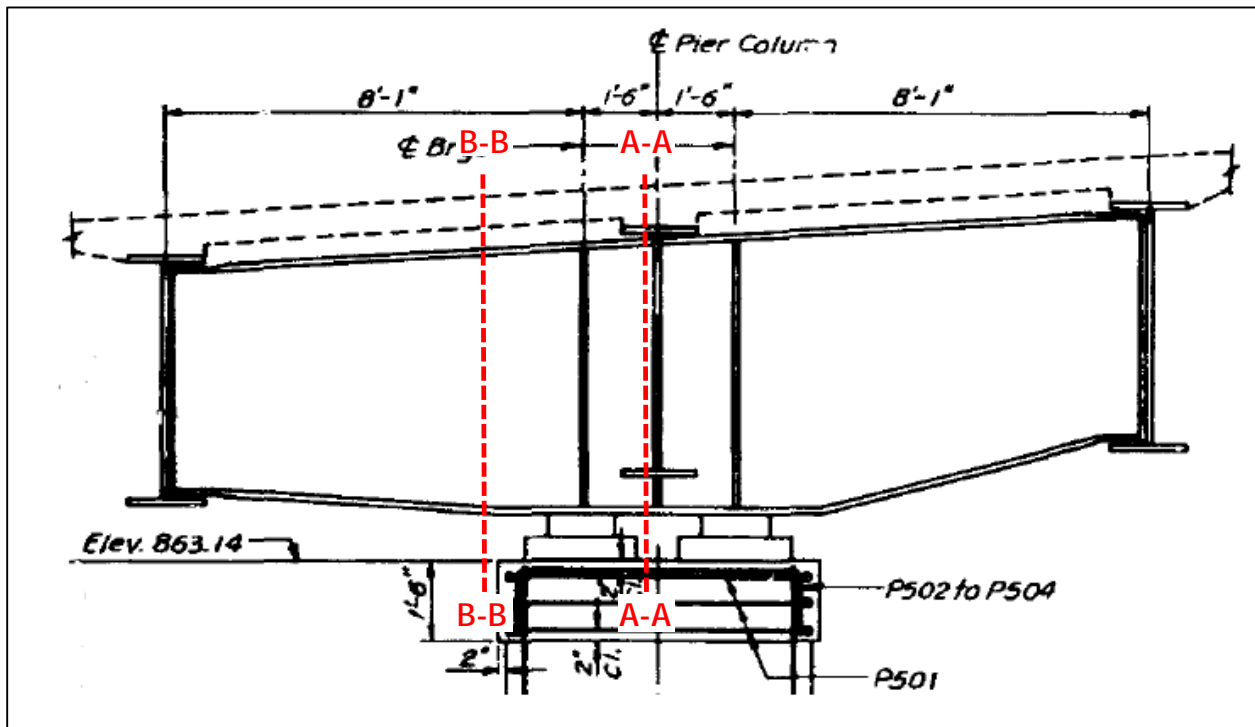
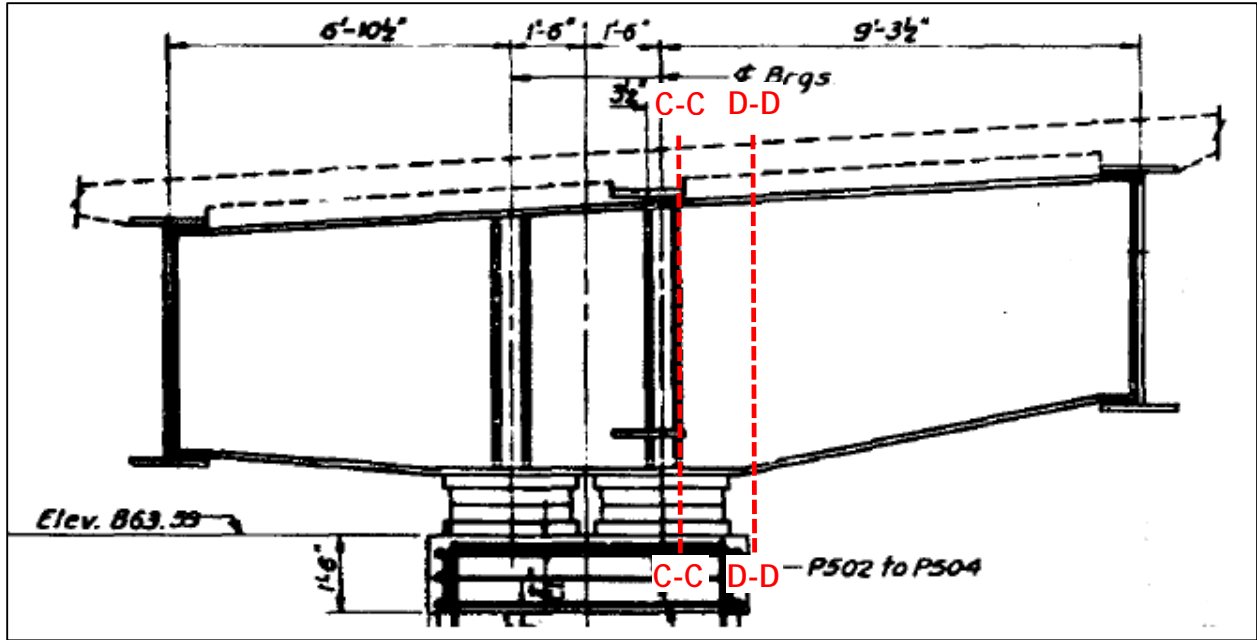*Figure 5-5 Bridge 27726B Pier 1A Elevation with Failure Planes*

Figure 5-6 Bridge 27726B Pier 2A Elevation with Failure Planes

Table 5-2 Bridge 27726B Analysis Results Overview

Failure Plane	Component	Live Load Factor	Deflection (in)	Allowable Deflection (in)	Maximum Stress (ksi)	Allowable Stress (ksi)
A-A (Pier 1A)	Entire Section	3.91	1.68	2.30	NA	NA
B-B (Pier 1A)	Entire Section	3.91	**4.60**	2.30	NA	NA
C-C (Pier 2A)	Entire Section	5.36	**2.71**	2.30	NA	NA
D-D (Pier 2A)	NA	NA	NA	NA	NA	NA

5.3 Bridge 27788

5.3.1 <u>Summary</u>

Bridge 27788 connects Hwy 100 NB to I-394 EB. The structure is two spans with three continuous steel I-girders and an outrigger pier. Each span measures 142 feet from centerline of bearing to the centerline of Pier 1. An integral steel box cap is utilized at the pier to span an onramp which connects to northbound Hwy 100. The span length of the box cap is 36.5' from centerline of bearing to centerline of bearing. The integral steel box cap is considered the only fracture critical element on the structure. Refer to Figure 5-7 for a view of the STAAD model.
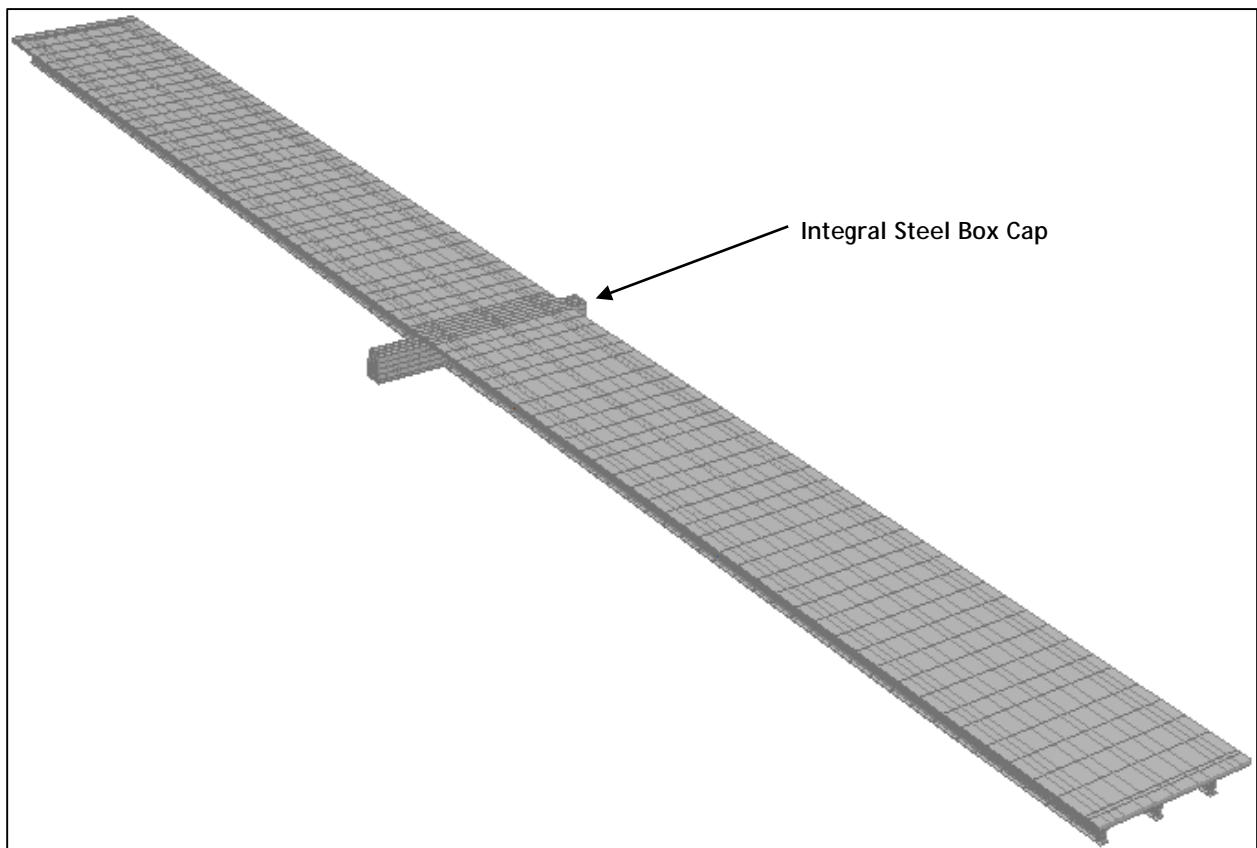

Figure 5-7 Bridge 27788 STAAD Model View

5.3.2 Results

Refer to Figure 5-8 and Table 4 for an overview of results from the refined analysis. Based on these results the following conclusions were made:

1. A localized failure along plane A-A will <u>not</u> result in a catastrophic collapse of the entire structure.
2. A localized failure along plane B-B will likely result in a catastrophic collapse of the entire structure based on excessive stresses.
3. A localized failure along plane C-C will likely result in a catastrophic collapse of the entire structure based on excessive stresses and deflection criteria.

Failure along C-C, or the failure of the east column would likely not be taken into account for the purposes of determining collapse in regards to a fracture critical failure. The column in this location is protected from traffic, and is not a steel element or in tension. However, this failure is similar to one simulated for B-B, where a crack forms in the bottom flange, then propagates to the web and top flange, thus, creating a crack through the entire cap. Therefore, based on this localized failure and the criteria used for redundancy in this analysis, the bridge should remain classified as fracture critical.
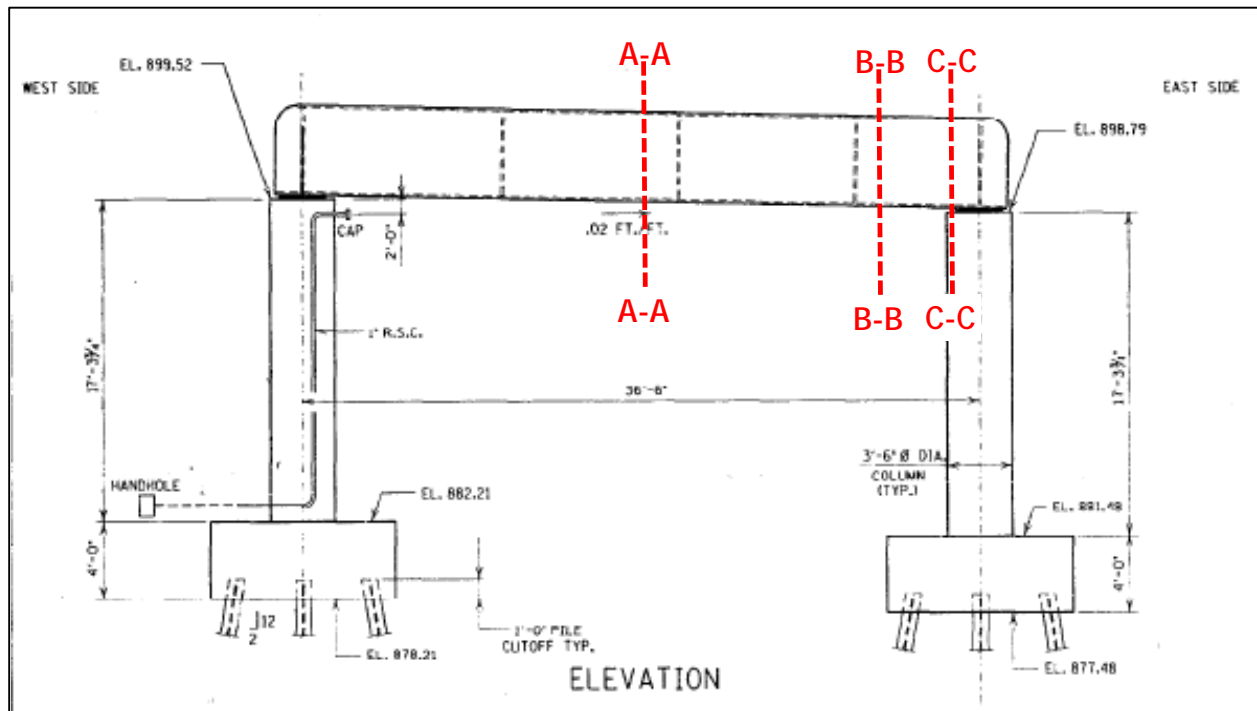
*Figure 5-8 Bridge 27788 Pier 1 Elevation with Failure Planes*

Table 5-3 Bridge 27788 Analysis Results Overview

Failure Plane	Component	Live Load Factor	Deflection (in)	Allowable Deflection (in)	Maximum Stress (ksi)	Allowable Stress (ksi)
A-A	Web	6.34	1.38	4.38	**65.8**	65.0
A-A	Bottom Flange	6.34	1.39	4.38	**> 800**	65.0
A-A	Bottom Flange and Webs	6.34	1.64	4.38	46	65.0
B-B	Bottom Flange	6.34	1.36	4.38	**> 300**	65.0
B-B	Bottom Flange and Webs	6.34	1.86	4.38	**> 300**	65.0
C-C	East Support	6.34	**-420.21**	4.38	**>400**	65.0

5.4 Bridge 62853

5.4.1 <u>Summary</u>

Bridge 62853 connects 35W northbound to TH 36 eastbound and crosses over TH 280 in Roseville, MN. This structure is a continuous 3-span bridge comprised of four built up steel I-girders. Spans 1 and 3 are similar, with a span length of approximately 88 feet, and Span 2 is approximately 115 feet. An integral steel I-girder cantilevered pier cap is utilized at Pier 1 and Pier 2 and is bearing on a reinforced concrete column. Two of the girders at this location bear directly over the column and the others are cantilevered out from the center. This style of pier is commonly referred to as a "hammerhead" shape. These two integral steel I-girder caps are considered the only fracture critical elements on the structure. Refer to Figure 5-9 for a view of the STAAD model.


Integral Steel I-Girder Cap (Pier 1)

Integral Steel I-Girder Cap (Pier 2)
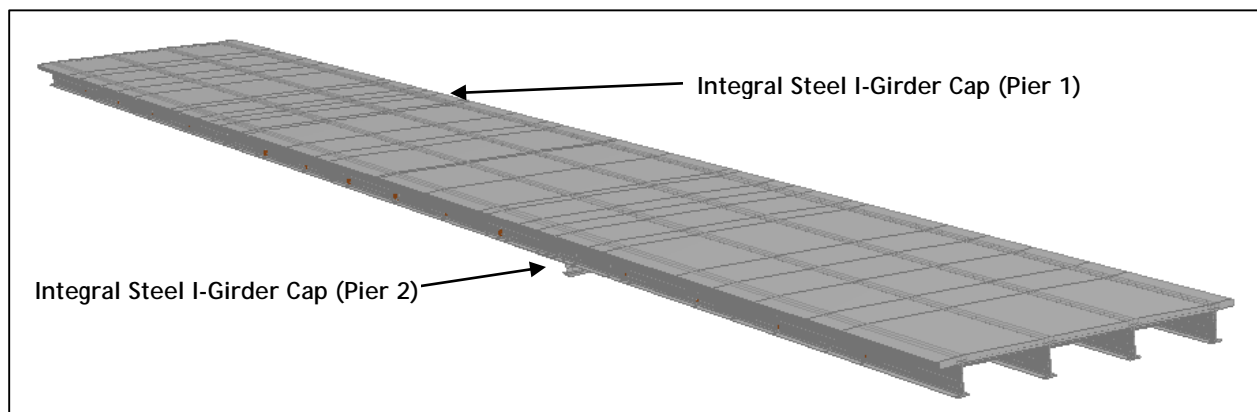
Figure 5-9 Bridge 62853 STAAD Model View

5.4.2 Results

Refer to Figure 5-10, Figure 5-11, and Table 5 for an overview of results from the refined analysis. Based on these results the following conclusions were made:

1. A localized failure along plane A-A for Pier cap 1 will likely result in a catastrophic collapse of the entire structure based on deflection criteria.
2. A localized failure along plane B-B for Pier cap 2 will likely result in a catastrophic collapse of the entire structure based on deflection criteria.

Deflection criteria was first established, and based on the results, a catastrophic failure would likely occur along both failure planes (A-A and B-B). Given that deflection criteria failed, it is not necessary to consider stress values. Therefore, based the criteria used for redundancy in this analysis, and the likely hood of a catastrophic collapse occurring from a localized failure at Pier caps 1 and 2, this bridge should remain classified as fracture critical.
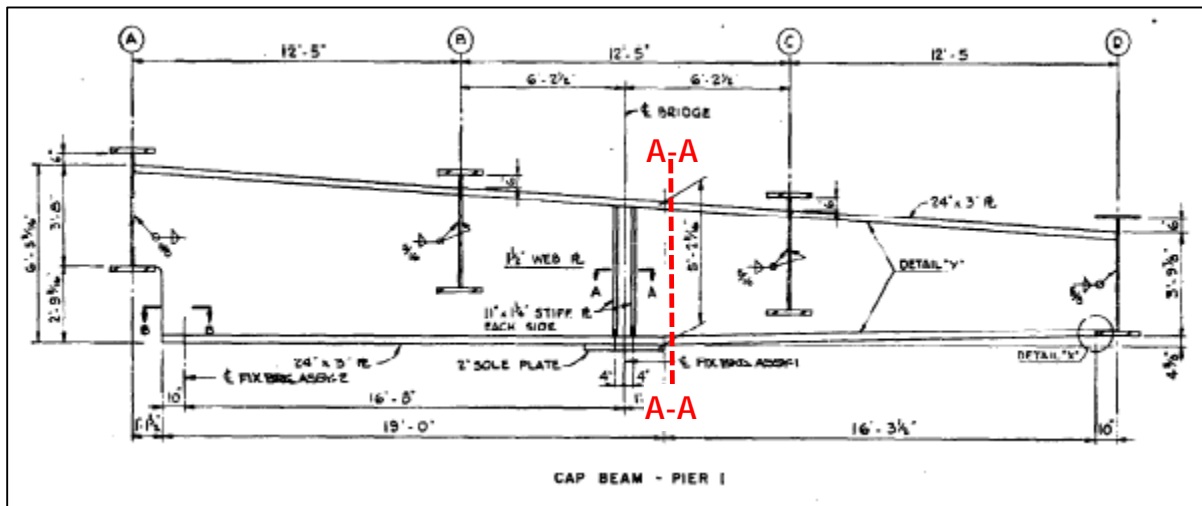
Figure 5-10 Bridge 62853 Pier 1 Elevation with Failure Planes

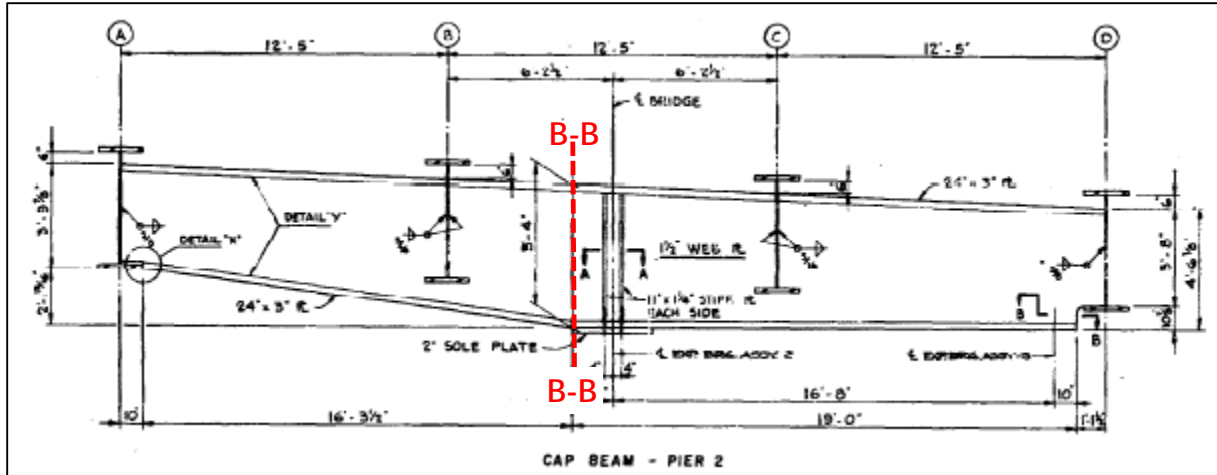*Figure 5-11 Bridge 62853 Pier 2 Elevation with Failure Planes*

Table 5-4 Bridge 62853 Analysis Results Overview

Failure Plane	Component	Live Load Factor	Deflection (in)	Allowable Deflection (in)	Maximum Stress (ksi)	Allowable Stress (ksi)
A-A (Pier 1)	Entire Section	3.91	**22.1**	4.24	NA	NA
B-B (Pier 2)	Entire Section	3.91	**7.23**	4.24	NA	NA

Results Summary

Table 6 presents a complete overview of the fracture critical pier caps for each bridge.

Table 5-5 Summary of Refined Analysis Results

Bridge	Cap Type	Pier Type	Location	Designation
27048	Integral Steel Box Cap	Hammerhead	Pier 1	Redundant
27726B	Integral Steel I-Girder Cap	Hammerhead	Pier 1A	Fracture Critical
27726B	Integral Steel I-Girder Cap	Hammerhead	Pier 2A	Fracture Critical
27788	Integral Steel Box Cap	Outrigger	Pier 1	Fracture Critical
62853	Integral Steel I-Girder Cap	Hammerhead	Pier 1	Fracture Critical
62853	Integral Steel I-Girder Cap	Hammerhead	Pier 2	Fracture Critical

CHAPTER 6: DISCUSSION AND CONCLUSIONS

Based on the publication research and the refined analysis, the following conclusions were made:

- Bridge 27048 – Pier 1 was sufficiently redundant and does not need to be classified as fracture critical
- Bridge 27048 has a "Redundant Load Path" diaphragm adjacent to each side of the pier cap as part of the original construction of the bridge, which largely contributed to a redundant load path.
- Bridge 27726B – Pier 1A was not sufficiently redundant and should remain classified as fracture critical
- Bridge 27726B – Pier 2A was not sufficiently redundant and should remain classified as fracture critical
- Bridge 27788 – Pier 1 was not sufficiently redundant and should remain classified as fracture critical
- Bridge 62853 – Pier 1 was not sufficiently redundant and should remain classified as fracture critical
- Bridge 62853 – Pier 2 was not sufficiently redundant and should remain classified as fracture critical
- As outlined in NCHRP Report 406, to calculate the initial reserve ratio values, a nonlinear analysis of the finite element model is needed as opposed to a linear elastic analysis originally used to calculate LF_1. Ideally for a redundancy analysis, the live load is incrementally increased until the maximum deflection of the model equals span length/100. If the factor used to increment the live load is at least 1.10 x LF_1, then R_f has been adequately satisfied. Using this same nonlinear analysis, the live load is then incrementally increased until the structure reaches ultimate capacity. If this factor is at least 1.30 x LF_1, then R_u has been adequately satisfied. However, for the purposes of this refined analysis, STAAD computer software was utilized. This program does not sufficiently provide a method of incrementing the live load in this way, nor does it adequately indicate when the structure has reached its ultimate capacity.

Based on the results of the analysis, the following recommendations are made:

- Two types of integral steel box caps were analyzed, this included a "hammerhead" and "outrigger" style. Outrigger style box caps have a very low probability of being redundant, therefore, hammerhead should be the primary focus of any future fracture critical analyses.
- Retrofitting current fracture critical hammerhead pier caps with a load path redundant diaphragm, similar to bridge 27048, may be an effective method for providing load path redundancy.

Given the live load incrementing and ultimate capacity limitations of STAAD, as discussed in the conclusions above, a modification to the methodology outlined in NCHRP Report 406 is recommended. If STAAD is used for the refined analysis, conservatively multiply LF_1 by 1.30, which is the factor typically used for R_u, and instead use this value as the live load factor for the

nonlinear analysis. If the model has not reached a maximum deflection of L/100, typically the threshold for R_f, then it can be assumed that this will likewise be sufficient and that the undamaged model will be considered to have provided an adequate level of redundancy.

REFERENCES

M. Ghosn and F. Moses (1998) NCHRP Report 406: Redundancy in Highway Bridge Superstructures, Nation Cooperative Highway Research Program, Washington, DC

W. Liu, A. Neuenhoffer, M. Ghosn and F. Moses (2001) NCHRP Report 458: Redundancy in Highway Bridge Substructures, National Cooperative Highway Research Program, Washington, DC

FHWA (2013) TechBrief: Evaluation of Member and Load-Path Redundancy on the US-421 Bridge over the Ohio River, Federal Highway Administration, McLean, VA

R. Connor, R. Dexter, and H. Mahmoud (2005) NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture Critical Details, National Cooperative Highway Research Program, Washington, DC

C. Jenks, C. Hedges, W. Dekelbab, D. Powell, E. Delaney and H. Freer (2014) NCHRP Report 776 Bridge System Safety and Redundancy, National Cooperative Highway Research Program, Washington, DC

W. Wright (2013) NCHRP 12-87A Fracture Critical System Analysis for Steel Bridges, National Cooperative Highway Research Program, Washington, DC

Minnesota Department of Transportation, *Bridge and Structure Inspection Program Manual*, MnDOT Office of Bridges and Structures, Oakdale, MN

Federal Highway Administration. Internet. Bridges and Structures, June 20, 2012, https://www.fhwa.dot.gov/bridge/120620.cfm.

AASHTO (2013) *LRFD 6th Edition with 2013 Interim Revisions,* American Association of Highway and Transportation Officials, Washington, DC