

Redundancy Evaluation of Fracture Critical Bridges

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ABSTRACT

Cases of brittle fractures in major bridges prompted AASHTO to publish its first fracture control plan in 1978. It focused on material and fabrication standards, and required periodic 24-month hands-on inspection of bridges with fracture critical members. The practical result of this plan was to significantly increase the life cycle cost of these bridges, rendering them uneconomical. Apart from the Point Pleasant Bridge that failed in 1967, no other bridge has collapsed in the USA following a fracture, even though large fractures have been observed in many other bridges. All these bridges showed some degree of redundancy and therefore could be reclassified as non-fracture critical if detailed analyses are carried out.

The goal of this study is to provide guidance on redundancy evaluation of fracture critical bridges, specifically three girder bridges and twin box-girder bridges. The effect of various loading, analysis and geometric parameters on the post fracture response and the remaining load carrying capacity of the damaged bridge is evaluated through nonlinear finite element analysis of two well-documented structures: the Hoan Bridge and the twin box-girder bridge. Parameters such as damping definition, modelling of composite action, modelling of secondary elements, boundary conditions, and rate dependent material properties are found to be crucial in capturing the bridge response.

A two-step methodology for system redundancy analysis of fracture critical bridges is proposed, leading to a reclassification of these elements as non-fracture critical for in-service inspection. The first step evaluates bridge capacity to withstand collapse following fracture based on whether the residual deformation is perceivable to people on or off the bridge. If the bridge satisfies the first step requirements, then the reserve load carrying capacity of the damaged bridge is evaluated in the second step. The Hoan Bridge failed to satisfy the proposed requirements in the first step and therefore its girders could not be reclassified as non-fracture critical. The twin box-girder bridge successfully resisted the collapse in two out three loading scenarios and displayed reserve load carrying capacity following full depth fracture in the exterior girder, and therefore can be reclassified as non-fracture critical for in-service inspection.

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1 Introduction

On December 15, 1967, the Silver Bridge in Point Pleasant, West Virginia over the Ohio River collapsed during rush hour, resulting in the deaths of 46 people. The collapse was the result of brittle fracture of one of the non-redundant eye bars supporting the main span. Although this was an uncommon bridge type that was no longer being constructed, this tragic accident was the catalyst for many changes in material specifications, design, fabrication, shop inspection, in-service inspection, and maintenance of steel bridges. Many of these changes were embodied into Title 23, Part 650 of Code of Federal Regulations (United States. Dept. of Transportation, 2012), which was developed to provide guidelines for construction, inspection and rehabilitation of bridges.

On May 7, 1975 a large crack was discovered in one of the two main girders of the Lafayette Street Bridge in St. Paul, MN. The crack, which ran for about 95% of the girder's 143 in. depth, led to ~7 in. vertical deflection at the location of the crack. The crack initiated in a lack of fusion area in the weld of the lateral bracing gusset and transverse stiffener, which resulted in a brittle fracture of the girder web (Fisher, et al., 1977). In 1977, a 10 ft. long crack was discovered in the girder of the I-79 Glenfield Bridge at Neville Island, PA. The fracture initiated in the electroslog weld because of weld defect developed during the girder fabrication (Fisher, et al., 1984).

In response to fractures such as those in the Lafayette Street Bridge and the Glenfield Bridge, AASHTO published the first fracture control plan in 1978. It defined fracture critical members (FCMs) as "A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse". It focused on the control of structural flaws and specification of the material toughness, both required to provide low probability of brittle fracture (Dexter, et al., 2004). Standards and procedures for fabrication and inspection of fracture critical members were incorporated into the American Welding Standards Bridge Welding Code. In addition to the material and the fabrication controls, AASHTO required periodic 24 month hands-on inspection of bridges with fracture critical members. These requirements lead to increased cost for construction and maintenance, discouraging bridge designers and owners from use of fracture critical bridges in spite of the fact that these bridges might offer economical and efficient solution. It is interesting to note that no other country has the equivalent of 'fracture critical' provisions in

its bridge design codes and that thousands of similar bridges have been erected worldwide and have performed satisfactorily.

Apart from the Point Pleasant Bridge, no other bridge has collapsed in the USA following a fracture. On December 13, 2000, full web depth fractures were found in two out of the three steel girders on the Hoan Bridge. Partial depth web fractures were also found at three locations in the third girder. Visual inspection of the northbound bridge by WisDOT revealed a depressed area approximately 4-feet deep by 25-feet long, by about 50-feet wide across the roadway. Although the Hoan Bridge did not collapse following fracture, the large residual deflection left it unfit for service. There are cases in the literature where there was no/some perceivable deflection that would alert the drivers about an existing fracture and the bridge stayed in service. For example, the exterior girder of the I-95 bridge over the Brandywine River in Delaware developed a full depth crack (bottom flange and 80% of the web). The bridge stayed in service without showing perceivable deflection before a pedestrian walking in the park below the bridge noticed the crack (2005). All these damaged bridges displayed redundancy, and therefore could be reclassified as non-fracture critical. If we assume that some damage is acceptable, then the question becomes: *Is the damaged bridge capable of resisting external loads until the next inspection?* This question, which this thesis addresses, has not been studied carefully both because of the large computational power required and the lack of robust models to track collapse of structures.

1.1 Methodology

The goal of this study is to provide guidance on redundancy evaluation of fracture critical bridges, specifically three girder bridges and twin box-girder bridges, with and without composite action. The work was conducted within the scope of first phase of NCHRP Project 12-87. Guidelines will be developed primarily through use of advanced finite element analyses. The major steps in this study are as follows:

- a) Conduct a comprehensive review of knowledge relevant to this research including experimental and analytical redundancy evaluation methodologies used by researchers and practicing engineers, modeling of composite action at the steel-concrete interface, imperfection and residual stresses in steel components and their impact on member behavior, and live load models used for redundancy evaluation.

- b) Develop and calibrate modeling methodologies, material plasticity and fracture parameters in ABAQUS (2013) through comparison with the test data from simply supported slab tests by Jofriet and McNeice (1971) and two simply supported composite steel plate girder tests by Mans et al. (2001).
- c) Investigate the effect of different geometric and modeling assumptions on the post-fracture dynamic response through non-linear dynamic analysis of the Hoan bridge (Fisher, et al., 2001).
- d) Investigate the impact of fracture sequence and fracture location on the post fracture dynamic response using the Hoan Bridge model.
- e) Evaluate the capabilities of static and dynamic analysis methods with dynamic load amplification to capture the post fracture dynamic response using the Hoan bridge model.
- f) Investigate the effect of different geometric and modeling assumptions on the post-fracture dynamic response and the remaining load carrying capacity of the twin box-girder bridge tested at The University of Texas at Austin (Barnard, et al., 2010) through non-linear static and dynamic analysis.
- g) Evaluate the capability of simplified models to capture the reserve load carrying capacity of the damaged twin box-girder bridge tested at The University of Texas at Austin (Barnard, et al., 2010).
- h) Develop a system redundancy analysis methodology and its evaluation through analysis of the failure of a real bridge (Hoan Bridge) and simulated tests (Twin box-girder bridge).

1.2 Goals and outcomes

The principal goals and outcomes are:

- a) A state-of-the-art review of existing specifications and technical literature on both redundancy in fracture critical bridges and collapse analysis of steel bridges using finite element analysis.
- b) Development of a methodology for system redundancy analysis of fracture critical bridges using finite element analysis that can be used by department of transportations (DOTs) to make decisions on inspection, maintenance, repair and replacement of such structures.
- c) Validation of proposed system redundancy methodology through existing field and lab testing data.

1.3 Organization of dissertation

Chapter 2 provides a comprehensive review of experimental and analytical redundancy evaluation methodologies used by researchers and practicing engineers, modeling of composite action at the steel-concrete interface, imperfection and residual stresses in steel components and its impact on member behavior, and live load models used for redundancy evaluation. This chapter addresses the primary goal and outcome (a) in the previous section.

Chapter 3 evaluates the different aspects of modeling methodology through comparison with the test data from simply supported slab and simply supported composite steel plate girders. It discusses element selection to model different bridge components, gives an overview of implementation of material plasticity in ABAQUS (2013), and discusses modeling of varying level of composite action and fracture simulation methods for static and dynamic analysis.

Chapter 4 describes the Hoan bridge (Fisher, et al., 2001) case study used in the development of redundancy evaluation guidelines. It investigates the effect of geometric and modeling assumptions (damping definition and solution schemes, modeling of secondary bridge components, coefficient of friction, boundary conditions, slab concrete strength and reinforcement, rate dependent material behavior, composite action using rigid links) on the post-fracture dynamic response of the Hoan bridge (Fisher, et al., 2001) through non-linear dynamic analysis. It investigates the impact of fracture sequence and fracture location on the post fracture dynamic response; and evaluates the capabilities of static and dynamic analysis methods to capture the post fracture response using the Hoan bridge model. This chapter partially address the primary goal and outcome (b) in the previous section.

Chapter 5 describes the twin box-girder bridge (Barnard, et al., 2010) case study that will be used as the primary calibration study in the development of redundancy evaluation guidelines. It investigates the effect of different geometric and modeling assumptions (boundary condition, shear stud shear-tension models, rate dependent material behavior, steel plasticity definition, guardrail contribution, concrete shrinkage) on the post-fracture dynamic response and the remaining load carrying capacity of the damaged bridge through non-linear static and dynamic analysis. It also evaluates the capability of simplified models to capture the reserve load carrying capacity of the damaged bridge and the possibility of bridge shakedown. This chapter partially addresses the primary goals and outcome (b) in the previous section.

Chapter 6 proposes a methodology for system redundancy analysis of fracture critical bridges specifically three girder bridges and twin box-girder bridges primarily through advanced finite element analysis. The proposed system redundancy methodology is evaluated through existing field data (Hoan Bridge) and lab testing data (Twin box-girder bridge). This chapter addresses the primary goals and outcome (b) and (c) in the previous section.

Chapter 7 summarizes the important findings and provides recommendations for future research.

1.4 Original contributions

The expected original contributions of this thesis, aside from the more practical products listed under Section 1.2, include:

- Assessment of different geometric and modeling assumptions (damping definition and solution schemes, modeling of secondary bridge components and their contribution to bridge response, coefficient of friction, boundary conditions, slab concrete strength and reinforcement, rate dependent material behavior, composite action using rigid links, shear stud shear-tension models, steel plasticity definition, concrete shrinkage) on the post-fracture dynamic response immediately following fracture and the remaining load carrying capacity of the damaged bridge.
- Investigation of the impact of fracture sequence and fracture location on the post fracture dynamic response of bridges.
- Evaluation of the simplified methods to capture the post fracture dynamic response and the remaining load carrying capacity of fracture critical bridges.
- Development of finite element analysis guidelines for redundancy evaluation of three girder and twin box-girder bridges with static and dynamic analysis.
- Proposal for methodology for system redundancy analysis of fracture critical bridges using finite element analysis.

2 Literature review

This chapter provides a background on the topic of fracture critical bridges. It begins with a discussion of applicable standards, an important consideration given that all bridge design and construction is controlled by such documents (Section 2.1). It is followed by a description of recent analytical and experimental research on redundancy of bridges that can serve as the basis for the guidelines to be proposed as part of this work (Section 2.2). Section 2.3 describes research related to modeling of composite action, a key parameter, as a well-designed composite slab can play a key role in redistributing internal forces after a failure. Section 2.4 discusses the impact of imperfections and residual stresses on member capacity. Work done here assumes that crack is pre-existing and is not concerned with origins of fractures. The reasons for this type of fracture in bridges are well understood and are tied to poor detailing and steel material properties. Good summaries can be found in Dowling (2007), while specific treatment of bridges can be found in Barsom and Rolfe (1999)

2.1 Fracture critical practices and standards

Code of Federal Regulations (United States. Dept. of Transportation, 2012): Bridge inspection policies were implemented in the U.S. following the collapse of the Silver Bridge in Point Pleasant West Virginia in 1967. Title 23, Part 650 of the Code of Federal Regulations (United States. Dept. of Transportation, 2012) provides guidelines for construction, inspection and rehabilitation of bridges. The code requires routine inspections to be performed every 24 months on all bridges located on public roads including the state and the federally owned bridges. Certain bridges may require inspection at less than a 24 month interval based on age, traffic characteristics, and known deficiencies. It also indicates that certain bridges may be inspected at intervals greater than 24 months (up to 48 months) with written approval from Federal Highway Administration (FHWA).

The Code of Federal Regulations (United States. Dept. of Transportation, 2012) defines fracture critical members (FCM) as “A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.” It requires a hands-on inspection of a fracture critical member or member components. Hands-on inspection is defined as “Inspection within arm’s length of the components. Inspection uses visual techniques that may

be supplemented by nondestructive testing.” The Code of Federal Regulations (United States. Dept. of Transportation, 2012) requires:

- 1) Frequency of inspection for fracture critical members not to exceed 24 months. It recognizes that certain fracture critical members would require inspection at time interval less than 24 months.
- 2) Criteria for the level and frequency of hands-on inspection to be determined based on the age, traffic characteristics, and known deficiencies.
- 3) The state and federal authorities to identify the bridges with fracture critical members in their inspection records.
- 4) Identification of fracture critical member location and description of inspection frequency and procedure in the inspection records.
- 5) Inspection data for the bridges with fracture critical members to be entered in the Federal agency inventory database within 90 days of the date of inspection compared to 180 days for all other bridges.

AASHTO LRFD Bridge Design Specifications (AASHTO, 2012): Relevant provisions in the AASHTO LRFD Bridge Design Specification (AASHTO, 2012) related to analysis, redundancy, collapse and fracture critical members are provided in the sections listed below:

- Section 1.2 (AASHTO, 2012) defines collapse as a major change in the geometry of the bridge rendering it unfit for use.
- Section 1.3.1 (AASHTO, 2012) specifies that “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.” and that future improvements are anticipated.

AASHTO LRFD design philosophy is given by,

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \quad \text{Equation 2-1}$$

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \quad \text{Equation 2-2}$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \quad \text{Equation 2-3}$$

where,

γ_i	Load factor, a statistically based multiplier applied to force effects
ϕ	Resistance factor
η_i	Load modifiers
η_D	Factor relating to ductility
η_R	Factor relating to redundancy
η_I	Factor relating to operational importance
Q_i	Force effect
R_n	Nominal resistance
R_r	Factored resistance

Factor relating to relating ductility (η_D), redundancy (η_R) and operational performance (η_I) are limited to a range between 0.95 and 1.05. The commentary for this section (C1.3.2.1) states that using $\eta > 1.0$ relates to a reliability index of $\beta > 3.5$. The selection requirements for the η factor are somewhat arbitrary and future improvements are anticipated.

- Section 1.3.4 (AASHTO, 2012) on redundancy, states that “Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them. Main elements and components whose failure is expected to cause the collapse of the bridge shall be designated as failure-critical and the associated structural system as non-redundant. Alternatively, failure-critical members in tension may be designated fracture-critical. Those elements and components whose failure is not expected to cause collapse of the bridge shall be designated as nonfailure-critical and the associated structural system as redundant.” It provides redundancy factor (η_R) for different levels of redundancy but provides little to no guidance on methods to determine those levels of redundancy.

- Section 4 (AASHTO, 2012) discusses acceptable methods of structural analysis for bridges and provides general guidance for analysis and selection of computer programs. It recommends designers check the validity of assumptions made in the program and that the program is verified using benchmark problems or previously verified computer programs or experimental results.
- Section 6.6.2 (AASHTO, 2012) presents fracture limit state requirements and requires all primary longitudinal superstructure components, transverse floor beams and connections sustaining tensile force effects due to Strength Load Combination I to have Charpy V-notch fracture toughness as per Table 6.6.2-2 in the AASHTO LRFD Bridge Design Specification (AASHTO, 2012). Section 6.6.2 commentary provides discussion about the level of rigor needed in using refined analysis to demonstrate redundancy. It states that, “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.”
- Current AASHTO (2012) definitions of collapse and redundancy are subjective and do not provide guidance on the loads a bridge should sustain before collapse. The AASHTO (2012) does not provide specific details about the redundancy evaluation methodology for fracture critical bridges.

On June 20, 2012 the Federal Highway Administration (FHWA) published a memo with the subject “Clarification of Requirements for Fracture Critical Members” to provide clarification of the FHWA policy for the classification of fracture critical members. For the first time, structural redundancy demonstrated through refined analysis is formally recognized and may also be used for identification of fracture critical members for in-service inspection protocol. This clarification does not recognize redundancy from internal built-up details to impact the classification of fracture critical members. Therefore, it requires totally fractured fracture critical member for load path redundancy evaluation. For example, a bridge girder in bending has bottom flange and part of the web in tension. For redundancy evaluation, it requires the entire

member (tension flange, web and compression flange) failure. This clarification states that “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.”

NCHRP synthesis 354 (Connor, et al., 2005): This document is the main source for this project as it surveyed the existing literature on the fracture critical details and identified gaps in the literature; examined best practices and problems with how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details; and identified research needs. Surveys of DOTs conducted under this project indicated an increase in cost for fracture critical bridges of two to five times over that of conventional bridges. Most additional costs were associated with the access equipment, traffic control for lane closures, NDT testing, additional employee hours for detailed hands-on inspection and increased frequency of inspection for some agencies. The surveys also indicated a need for inspection based on the level of risk, average daily truck traffic (ADTT) and type of fracture and fatigue detail.

Connor et al. (2005) collected information from the literature, a survey of bridge owners and consultant inspectors, and from targeted interviews. Collected information showed that the initial cost premium for new bridges with FCMs is approximately 8% of the cost of fabricated steel. The major incurred expense is the additional mandate for hands-on, in-service inspection of FCMs. The FCMs definition used by DOTs was found to be consistent with the AASHTO LRFD Bridge design provisions, but its interpretation was variable. Agencies were conservative in their classification of fracture critical bridges, with all agencies classifying two-girder bridges as fracture critical. Table 2-1 shows the response by different agencies to the question “How would you categorize the following bridges?”

Table 2-1: Survey results indicating how states classify bridges to be fracture critical (Connor, et al., 2005)

Description	Fracture-Critical Bridges	
	Yes	No
Two-girder bridges	38	0
Three-girder bridges	9	28
Three-girder bridges with girder spacing	10	21
Multi-girder bridges with girder spacing	3	32
Truss bridges	34	3
Two-girder bridges fabricated using HPS 70W	31	1
Truss bridges fabricated using HPS 70W	28	2
Single-steel “tub” girder bridges	32	5
Twin-steel “tub” girder bridges	22	12
Multi-steel “tub” girder bridges	0	32
Other (post-tensioned, timber, steel cross girders, etc.)	13	3

2.2 Bridge redundancy evaluation

2.2.1 Analytical evaluation of bridge redundancy

NCHRP 406 (Ghosn and Moses, 1998) and Ghosn, et al. (2010): NCHRP 406 (Ghosn and Moses, 1998) defined bridge redundancy as “the capability of a bridge superstructure to continue to carry load after the damage or the failure of one of its members.” The authors recognize that the structural components do not behave independently but behave as a structural system. Design specifications at the time ignored the system interaction effects and used component design approach. This reports attempts to bridge the gap between a component-by-component design and the system effect by introducing system factors.

This report introduces system factor (ϕ_s) that relates to safety and redundancy of the entire bridge system. The system factor was applied as a multiplier to the nominal member resistance as shown below.

$$\phi_s \phi R' = \gamma_d D_n + \gamma_l L_n (1 + I) \quad \text{Equation 2-4}$$

where,

γ_d	Dead load factor
γ_l	Live load factor
ϕ	Resistance factor
ϕ_s	Resistance factor
D_n	Design dead load effects
L_n	Design live load effects
I	Dynamic impact factor
R'	Nominal material strength

The authors recommend the use of system factor to evaluate redundancy of existing bridge or design of new bridges. System factors were calibrated to the desired level of target system reliability index. The calibration was carried out using the load factors from four different limit states: member failure (LF_1), ultimate capacity of the undamaged bridge (LF_u), deflection (LF_f), and ultimate capacity of the damaged bridge (LF_d). LF_1 is expressed as the number of side-by-side HS-20 trucks, in addition to the dead load, a bridge can withstand before the first member failure. LF_1 can be calculated using linear elastic model and incrementing the truckload until first member failure. LF_u is expressed as the maximum possible truck load in addition to the dead load that can be applied before bridge collapse. LF_f is the capacity of the bridge to resist large displacements, which can be calculated by incrementing the two side-by-side HS-20 trucks in a nonlinear analysis until desired deflection limit is reached. The authors proposed displacement limit of (span length/100) for calculation of LF_f . LF_d is the ultimate capacity of the bridge after removal of main load carrying member. This study assumes a damage scenario that consists of complete removal of a main girder. Figure 2-1 (Ghosn, et al., 2010) provides a conceptual representation of all the load factors.

To develop the range of values for the load factors numerous, steel and concrete bridges were analyzed using analysis program NONBAN developed in house as a part of NCHRP 406 project (Ghosn and Moses, 1998). NONBAN used the grillage analysis method to analyze the bridge superstructure. NONBAN used the incremental nonlinear analysis with plasticity concentrated at beam ends. It only accounted for nonlinearity in the main (major) axis bending, while torsion,

axial deformation and bending about the minor axis are assumed to remain in the linear elastic range. The program was verified against a field test of a four-span four-girder rolled steel girder bridge in Tennessee, a three-girder simple span bridge tested at the University of Nebraska and a five-girder two-span continuous steel bridge tested in Ontario. Two side-by-side HS20 trucks were located for maximum effect and the load was incremented until collapse. This load is based on the load model developed by Nowak as part of NCHRP Report 368 (Nowak, 1999).

The bridge redundancy is determined based on the requirements for the system reserve ratios based on the capacity of the undamaged bridge (R_u), undamaged bridge deflection (R_f) and the capacity of the damaged bridge (R_d) calculated using Equation 2-5 through Equation 2-7. If the calculated system reserve ratios fail to satisfy the limit set in Equation 2-5 through Equation 2-7 then bridge should be strengthened (Ghosn and Moses, 1998), while those that exceed the reserve ratios may be designed less conservatively.

$$R_u = \frac{LF_u}{LF_1} \geq 1.30 \quad \text{Equation 2-5}$$

$$R_f = \frac{LF_f}{LF_1} \geq 1.10 \quad \text{Equation 2-6}$$

$$R_d = \frac{LF_d}{LF_1} \geq 0.5 \quad \text{Equation 2-7}$$

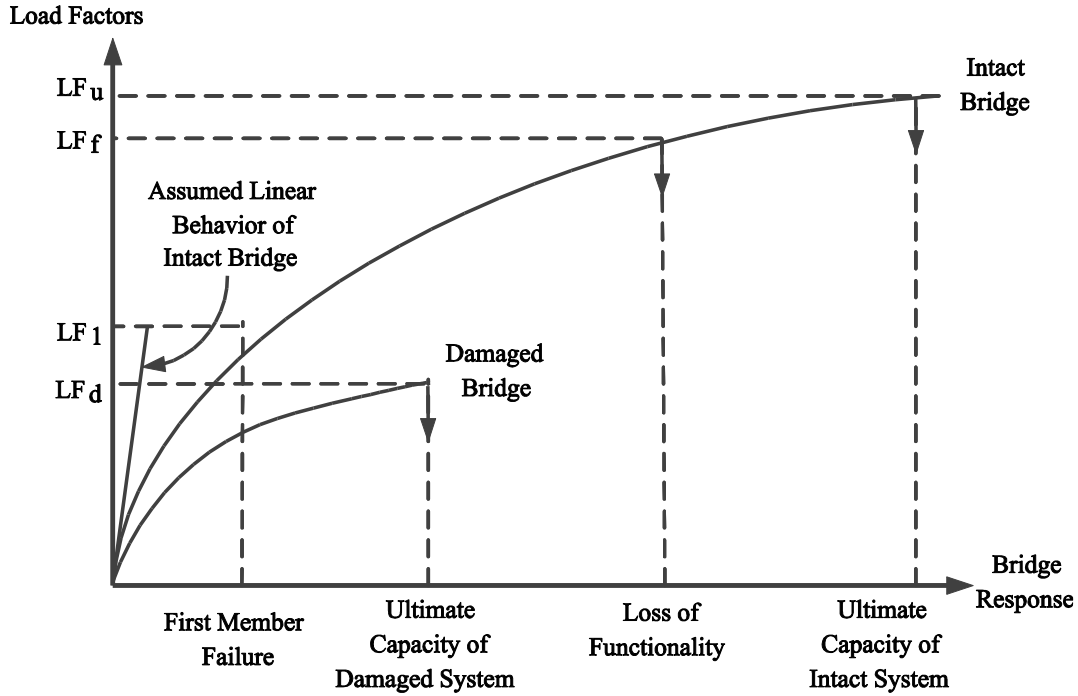


Figure 2-1: Conceptual representation of load factors (Ghosn, et al., 2014)

The report also quantifies redundancy in terms of relative reliability indices for undamaged bridge ($\Delta\beta_u$), undamaged bridge deflection ($\Delta\beta_f$) and the capacity of the damaged bridge ($\Delta\beta_d$). These relative reliability indices give measures of the relative safety provided to the bridge system compared to the nominal safety of first member failure.

$$\Delta\beta_u = \beta_{ult} - \beta_{member} \geq 0.85 \quad \text{Equation 2-8}$$

$$\Delta\beta_f = \beta_{funct} - \beta_{member} \geq 0.25 \quad \text{Equation 2-9}$$

$$\Delta\beta_d = \beta_{damaged} - \beta_{member} \geq -2.70 \quad \text{Equation 2-10}$$

This is the most comprehensive study available on the redundancy evaluation of bridges, but it has limitations for the redundancy analysis of fracture critical bridges. Bridges were modeled using grillage analysis. The study was calibrated based on the strength test results of undamaged bridges, therefore, the system reserve ratios and reliability indices limits might not be applicable for damaged bridge evaluation. In addition, this study did not consider typical fracture critical bridge configurations.

On June 26th 2014, Ghosn, et al. (2014) published the NCHRP 776 report which extends the work previously published by the author. NCHRP 776 report (Ghosn, et al., 2014) proposes a methodology to calibrate system factors that can be applied during the design and load capacity evaluation of highway bridges to account for bridge redundancy and system safety. The calibrated system factors can be used to assess bridges under distributed lateral loads or vertical vehicular loads. As mentioned before, the NCHRP 776 report was published very recently and cited here for the sake of completeness. At present, findings from this study are not incorporated in the methodology proposed as part of this research.

Daniels, et al. (1989): Daniels, et al. (1989) provided guidelines for design and rating of a redundant bracing system on new or existing two-girder steel bridges. They also investigated the after-fracture redundancy of simple span and continuous, composite and noncomposite, two-girder steel bridges. The investigation was done using an analytical model in which tension flange and full web depth fracture was assumed at any position along the length of one of the two girders. The bridge systems used in the analysis consisted of top and bottom lateral bracing and diaphragms. Results from the analysis showed that significant redundancy could be achieved through redundant bracing system.

McGormley, et al. (2000): McGormley, et al. (2000) evaluated retrofits to improve redundancy of fracture critical members on the I-26 Green River Bridge near Asheville, North Carolina and the I-70/55 Polar Street Complex Bridge in St. Louis, Illinois. Both of the bridges consist of a two girder system and are classified as fracture critical.

The I-26 Green River Bridge consists of two parallel structures with five spans with a total length of 1050 ft. Inspection revealed two transverse cracks in the bottom flange and numerous shorter transverse cracks in the web-to-flange plate fillet welds in all the girders. The numerous fillet weld cracks could not be removed by coring or drilling. Therefore, girder bottom flanges were bolted with cover plates 24 in. wide and 1 to 1.5 in. thick. The cover plates provided alternate load path if the cracked flange was to fracture. It is not stated if the retrofitted structure is still classified as fracture critical for future inspections.

The I-70/55 Polar Street Complex is an intricate system of elevated roadways and ramps with built-up steel girders that are non-composite with a concrete deck. Shortly after construction

inspection revealed numerous fatigue cracks in two-girder structures. Several of these cracks extended to become major fractures involving significant portions of the girder webs and girder flange at one location. Finite element analysis of a representative span was carried out to assess behavior following the complete fracture at critical locations. Analysis revealed that the structure's capacity to maintain its integrity with full depth girder cracks at some locations, while full depth cracks at other locations resulted in loss of serviceability or collapse. Details of the analysis methodology were not discussed in the paper. Based on the results from the analysis, the Iowa Department of Transportation implemented web reinforcement using HPS plates.

Bridge retrofit included bolting-on plates to increase capacity of members with low redundancy, and it also increased internal redundancy and deterred full member loss. The fracture criticality of the retrofitted members was reduced, but it is not clear which of those members were classified as non-fracture critical in future inspections.

Crampton, et al. (2007): Crampton, et al. (2007) performed redundancy analysis of the Polar Street Complex in Illinois and the I-435 Missouri River Bridge using finite element analysis and developed retrofit options to improve redundancy. This paper provides a general discussion on redundancy and limitations of current specifications. The authors indicate that there is no guidance on the load level that a damaged structure is expected to sustain, nor guidance on the dynamic amplification during fracture. The authors also indicate that the lack of guidance forces owners to establish their own definition of redundancy based on their judgment.

The Polar Street Complex consists of three and four span continuous two-girder non-composite bridge systems with built-up plate girders. Out-of-plane distortion cracking in the web gap region was observed two years after completion in 1973. In the subsequent years (1978-1983), brittle fractures through the web plate were observed at the floor beam to the bearing stiffener connection in negative moment region. The structure was still able to maintain serviceability after these major fractures. The retrofit scheme used high strength continuous web plates bolted to the girder web just above the bottom flange providing an alternate load path. The retrofit scheme was evaluated using finite element model developed in SAP 2000 (Habibullah and Wilson, 2005) with two side-by-side HS 20-44 truck loading with impact factor. The value of the impact factor used for the analysis is not specified in the paper. Nonlinear analysis indicated ultimate post fracture capacity of about 1.9 times the two side-by-side HS 20-44 truck loading.

The I-435 Missouri River Bridge consists of twin two-girder bridges each carrying three lanes. Inspection revealed out-of-plane distortion and fatigue cracking in the web plates of the main girders. Three dimensional finite element analyses revealed inability to sustain a full depth fracture in one of the girders, therefore, cover plates were installed in areas most likely to sustain fracture. Cover plates provided an alternate load path in the tension region for main girders and controlled crack growth. For both bridges, the fracture occurred between the inspection cycles, and the bridges maintained serviceability until the fractures were discovered during inspection.

Marquette Interchange Project (Elza, et al., 2004): The Marquette interchange project involved an elevated highway in a complex geometric layout. The project replaced deteriorated reinforced concrete box structures with a steel twin steel box-girder system over a portion of the structure. Design provisions classified the twin steel box-girder bridges as fracture critical. Therefore, HNTB, as part of Milwaukee Transportation Partners (MTP), performed redundancy analysis based on the recommendation by Ghosn and Moses (1998).

Redundancy evaluation was carried out using a 3-D finite element model developed in LARSA-4D (LARSA Inc., 2010). The bridge deck and the twin box-girders were modeled using four node shell elements. Top flanges were connected to the deck using rigid links to model the full composite action. The interior-box diaphragms were modeled using four node shell elements for web and beam elements for the flanges. Beam elements were released at the connections to reflect as-designed conditions. Parapets and bracing members were modeled using beam elements. The nonlinear structural behavior was simulated through step-by-step linear elastic analysis. To model cracking of deck concrete, its modulus of elasticity was adjusted manually to account for change in stiffness. Girder, diaphragms, and parapets section properties were also adjusted to account for plasticity. The fracture in one of the box girders was simulated by removing elements from top flanges, web, bottom flange and longitudinal stiffeners in a small region approximately 6 in. wide. Two side-by-side HS-20 trucks were placed on the bridge deck to cause the maximum load effects at the point of interest. The load was incremented in a series of load steps modifying the member properties or removing elements as necessary. This process was continued until onset of nonlinear behavior in the box girders.

Redundancy evaluation methodology based on the approach presented in NCHRP Report 406 (Ghosn and Moses, 1998) was implemented. A 3-D finite element model was used to evaluate

four limit states, member failure (LF_1), ultimate capacity of the undamaged bridge (LF_u), deflection (LF_f), and ultimate capacity of the damaged bridge (LF_d). For member failure limit state (LF_1), each member was evaluated using elastic analysis and against its design member capacity. The structure capacity was defined as the amount of both dead and live load the structure can support before the failure of any one member. Ultimate capacity of the undamaged bridge (LF_u) was evaluated with regard to the formation of the collapse mechanism or to the development of stresses in an element that might cause sudden failure of the overall system. The deflection limit state (LF_f) was evaluated by considering the maximum acceptable deflection. Ultimate capacity of the damaged bridge (LF_d) was determined by placing two side-by-side HS-20 trucks for maximum effect on the bridge with damaged primary load carrying member. The truck weight was incremented until the formation of a collapse mechanism or development of stresses in an element that might cause sudden failure.

On fracture, sudden redistribution of the energy happens throughout the system. In order to evaluate whether a bridge will be able to survive this initial redistribution, dynamic amplification of the response from fracture was calculated by modeling the bridge as a single degree of freedom system with 5% damping. The calculated dynamic amplification was applied to both the live load and dead load.

The calculated load factors were greater than the NCHRP Report 406 (Ghosn and Moses, 1998) recommended values allowing classification of the structure as a redundant structure and therefore as non-fracture critical. The Federal Highway Administration and the Wisconsin Department of Transportation also approved classification of the bridge as non-fracture critical for in service inspection (Elza, et al., 2004).

Hunley and Harik (2011): Hunley and Harik (2011) analytically investigated redundancy of twin steel box-girder bridges and proposed minimum design criteria for reclassification of a bridge as redundant. Their methodology was based on the approach presented in NCHRP Report 406 (Ghosn and Moses, 1998). The system reserve ratio (R_d) for the ultimate capacity of the damaged bridge was used as the redundancy criterion. The system reserve ratio (R_d) requires capacity of the damaged bridge to be at least 50 percent of the load that will result in the first member failure (Ghosn and Moses, 1998).

$$R_d = \frac{LF_d}{LF_1} \geq 0.50 \quad \text{Equation 2-11}$$

where LF_d is the ultimate capacity of the bridge after removal of one main load carrying member using nonlinear static analysis, and LF_1 is the capacity of the undamaged bridge calculated using linear elastic model.

Steel tub girder superstructure was designed using the grid analysis method in accordance with the AASHTO LRFD provisions (2004). In addition, nonlinear finite element models were constructed in ANSYS/CivilFEM (ANSYS, 2007). In the finite element model, steel tub girder webs and flanges were modeled using a Mindlin-Reisner type shell elements with five integration points through the thickness. Plasticity in girders was modeled using a stress-strain curve with parabolic post-yield response. All remaining structural steel components were modeled with elastic perfectly plastic material. Internal cross frames and top lateral bracing system were modeled with nonlinear Timoshenko beam elements. Top chords and diagonals of external K-type cross frames were modeled using truss elements, while the bottom chords were modeled using Timoshenko beam elements. The concrete deck was modeled using 4-node shell elements with Mindline-Reisner formulation overlaid with layered reinforcing steel elements. A multi-linear isotropic hardening material model with von Mises plasticity was used to represent the nonlinear behavior of deck concrete. The von Mises plasticity material model in ANSYS/CivilFEM (ANSYS, 2007) does not support material softening behavior. Therefore, deck stiffness degradation was modeled by reducing the stiffness of the individual finite elements to a near zero value when material reached crushing strain. Hunley and Harik's (2011) use of von Mises criteria for a pressure dependent material like concrete could lead to erroneous results. Stiffness provided by the guardrail was neglected to be conservative because of an increased interest in the use of open railing system on bridges. Shear studs were modeled using rigid beam elements. Hunley and Harik (2011) use of rigid beam elements to represent the shear studs could yield erroneous results as the studs in the fractured region are subjected to combined tension and shear.

Girder damage was simulated by reducing the element stiffness of a series of elements on the bottom flange and the web using the element birth and death feature in ANASYS/CivilFEM. Two side-by-side HS-20 trucks were placed at critical locations to cause maximum load effect at

the damaged girder location. This loading was consistent with the loading used in the development of system reserve ratio, R_d , by Ghosn and Moses (1998). Hunley and Harik (2011) also recommended a functionality or deflection limit of one hundredth of the span length which is identical to the deflection limit used by Ghosn and Moses (1998) for deflection load factor (LF_f).

A parametric study was carried out to study the effect of different geometric parameters: span length, girder design, bracing type, and continuity on bridge redundancy. The redundancy factor generally decreased with increase in span length. For curved bridges, designing inner and outer girders with equal flanges resulted in increased redundancy. External bracing served as a lateral load distribution mechanism increasing redundancy with solid diaphragms performing better than K-type cross-frames. Girder continuity also significantly increased the structural redundancy for continuous span bridges. The results of this study showed that the twin steel tub girder bridges designed with AASHTO LRFD designed code can be reclassified as redundant and non-fracture critical.

Yamaguchi, et al. (2011): Yamaguchi, et al. (2011) investigated post-member-failure analysis methods for steel truss bridges. Steel truss bridge models were analyzed using both the static and the dynamic analysis methods to study differences in the response. Analysis was performed using an in-house finite element program Y-FIBER3D. Critical members were determined by analyzing the truss under dead load. The critical member was then removed and the internal forces were applied at the nodes to simulate presence of the member. Another external load was applied in the opposite direction to simulate member failure. Member safety was evaluated using beam-column interaction equation proposed by Nagatani, et al. (2009). Static and dynamic analysis yielded different failure mechanisms. To reduce the computational time, the authors proposed a method based on the modal superposition. The proposed method yielded results consistent with the dynamic analysis with computational cost similar to a static analysis.

Kudsi and Fu (2001): Kudsi and Fu (2001) developed a new approach for redundancy analysis of structural systems in general with emphasis on truss bridges. The proposed approach represents the structural members on the bridge in series and parallel configurations building a block diagram to represent the entire bridge. In the block diagram, redundant members were

arranged in parallel configurations which were laid in series with the non-redundant members in the system. Multiple failure modes were defined in a series system layout and then equations were developed to calculate the system's pre-failure and post-failure reliability index and probability of failure. The system was defined as redundant or non-redundant by comparing the reliability index of the system after failure of a particular member with a target reliability index. The proposed methodology was applied to U.S Bridge Corporation Bridge 3000. A detailed finite element analysis was carried out using ANSYS 5.6 (ANSYS, 2001) identifying the redundant and non-redundant members. The entire truss bridge was divided into three subsystems and the block diagram was constructed to evaluate the pre-failure and post-failure reliability index. Target reliability index of 2.5 was used in this study. This paper does not provide adequate details about the finite element model of the truss bridge and member failure simulation methodology.

2.2.2 Experimental evaluation of bridge redundancy

Idriss, et al. (1995): Idriss, et al. (1995) conducted an analytical study and field testing of a two girder steel bridge on I-40 over the Rio Grande river in Albuquerque, New Mexico to determine the impact of different levels of damage on load redistribution, bridge capacity, and the potential for collapse. This bridge was taken out of service in 1993 and was scheduled for demolition. This allowed for destructive testing of the bridge prior to demolition. A three span continuous unit of the bridge was tested in both the undamaged and damaged condition under dead load and live load.

A three dimensional finite element analysis was carried out to assist with testing and simulate post fracture response of the bridge. During the test, the bridge was loaded in the positive moment region with 95.5% of the maximum New Mexico legal load (Idriss, et al., 1995). Four different levels of damage were introduced at the mid-span of the north plate girders by making various torch cuts in the web and flange. First a 2 ft deep cut was made in the web originating at the floor beam connection level. In the next step, the cut was extended to the bottom of the web. In the third state, the bottom flange was cut half way from each side directly below the web cut. In the fourth state, the bottom flange was severed completely. With a truck located right above the crack location, the fractured girder deflected 1-3/16 in. with no signs of yielding. In addition, there was no significant change in strains until the bottom flange was completely severed.

Analysis and testing showed longitudinal redistribution of the damaged girder to the interior support through cantilever action. In addition, the load was redistributed to the intact girder through deck, floor beams and bracing system through torsional stiffness of the system. This load transfer in the transverse direction was observed in the vicinity of the crack. These results indicated presence of secondary path in the event of fracture.

Barnard, et al. (2010): Barnard, et al. (2010) investigated the redundancy of fracture critical curved twin steel box-girder bridges through full-scale testing of a simply supported twin steel box-girder bridge and nonlinear finite element analysis. Three separate tests were performed on the bridge. In the first test, a fracture was simulated in the east side girder using an explosive shape charge that instantly cut the flange. The initial fracture did not cause the bridge to collapse. In the second test, bridge capacity to sustain release of potential energy following a full depth fracture of the girder web was evaluated. Full depth fracture was introduced by cutting the girder web while supporting it with temporary truss support. The live load equivalent of HS-20 truck was simulated by using concrete blocks placed at mid-span biased towards the east girder side. To simulate release of potential energy following the fracture, a tension tie in the temporary truss support was severed using explosives. In the third test, the reserve load carrying capacity of the damaged bridge was evaluated. This study is an important benchmark for redundancy analysis since it is one of the few where detailed experimental results are available.

Experimental response was compared with a detailed finite element model developed in ABAQUS (2007) by Kim (2010a). Box girder components were modeled using 8-node shell elements. The concrete deck was modeled using 8-node solid elements with reinforcement modeled using 2-node truss elements embedded within the concrete elements. Bridge rails were modeled using 8-node solid elements with contact at the expansion joint modeled using nonlinear spring elements. Internal and external brace members were modeled using 2-node truss and beam elements respectively. Structural steel and reinforcement plasticity were modeled using classical metal plasticity. Use of smeared cracking model and concrete damaged plasticity model in ABAQUS (2007) yielded numerical convergence problems. Therefore, the cast iron metal plasticity model was used to model nonlinear behavior of concrete. The cast iron metal plasticity model in ABAQUS (2007) uses von Mises criteria under compression. Barnard, et al. (2010) use of von Mises criteria for pressure dependent material like concrete

could lead to erroneous results. Behavior of shear studs under combined tension and shear was modeled using connector plasticity and damage definition in ABAQUS (2007). Further details about the modeling of shear studs are provided in Section 2.3.2 under “Review of research conducted at University of Texas at Austin”. Fracture was simulated by removing the connector elements initially binding the separated nodes along the predefined fractured path using “Model Change” option in implicit dynamic analysis. Removal of temporary truss support in the second test was simulated in a similar manner.

During the first test, the finite element model was able to capture the overall deformed shape of the fractured girder. Analysis predicted deflection 5.23 in. at mid-span compared to measured mid-span deflection of 5.64 in. Considering the level of accuracy of the displacement measuring instruments the authors deemed results from the analysis to be acceptable. During the second test, haunch separation from the top flange was observed with stud failure. Analysis predicted the haunch separation length along the inside of the fracture girder to be 96 ft compared to the measure separation of 67 ft In the third test, the bridge was loaded to find its reserved capacity. The bridge collapsed at the total load of 363 kips, which is somewhat lower than the capacity of 413 kips obtained from the finite element analysis. The gradual separation of the haunch observed in the finite element analysis results was not consistent with the sudden jump observed during test. Participation of the metal deck, stud shear-tension interaction model, rate dependent material and stud properties are possible reasons for the difference between the measured and predicted response.

Overall, the finite element analysis was able to predict the failure modes consistent with those observed during testing. Parametric study was carried out to determine the effect of stud length, barrier rail, bridge curvature, structural indeterminacy, and span length on load carrying capacity of twin steel box-girder bridges. Increased stud length reduced the haunch separation resulting in significant reduction in the vertical displacement of the fracture girder. Presence of the barrier rail in the model showed reduction in deck deflection under lower loads. However, after haunch separation along the outside of the intact girder, deck deflection increased. As the radius of curvature decreased, deflection in the fractured girder increased. The fractured girder showed increased in deflection with increase in span length. Dynamic amplification associated with sudden release of load was in the range of 1.36 to 1.58 for simply supported twin steel box-girder

bridges. The continuous span bridge showed increased load carrying capacity compared to the single span bridge. The continuous span bridge sustained the applied load without causing haunch separation along the outside of the intact girder. Based on the findings, one can conclude that the twin steel box-girder bridges are redundant and demonstrate alternate load path in the event of fracture for both the single span and continuous span bridges.

Park, et al. (2007): Park, et al. (2007) carried out this experimental and analytical study to evaluate the redundancy of simply supported two girder bridges. Previous tests by Idriss, et al. (1995) did not investigate the contribution of the lateral bracing system on the load distribution pre and post damage. Therefore, Park, et al. (2007) decided to test two 1/5th scale bridge specimens with and without lateral bracing.

A test bridge 50m long and 12m wide supporting 34cm thick prestressed concrete deck carrying a two lane highway on the top was designed as per Korean highway design code. Two scaled models were prepared from the prototype bridge configuration for testing. The 1/5th scale bridge specimens were 10 m long with two girders spaced 1.2 m center-to-center, supporting 1.8 m wide and 10.5 cm thick concrete deck on top. Bridge dimensions were selected to produce the same ratio of the moment of inertia of the prototype bridge before and after girders were made composite with the concrete deck. Concrete deck with 35 MPa concrete was used to replicate the prestressed concrete deck. The transverse prestressing in the prototype bridge configuration was excluded for simplicity. The deck was reinforced with 10 mm reinforcing bars 80 mm c/c at top and bottom in transverse and longitudinal direction. I-shape cross beams were installed at 1.25 m interval. For the bridge with lateral bracing, L-50 mm x 50 mm x 6 mm angles were used for X-type lateral bracing. Test specimens were fabricated as unshored composite girders with the shear connectors for composite action between the concrete deck and steel girders. No details about the shear stud configuration were provided in this paper. Specimens were simply supported using roller supports with load applied over a girder at mid-span.

A near full-depth crack was introduced through the lower flange and web of the loaded girder to simulate a severe fatigue crack. Prior to introducing the crack, the center of each girder was supported using temporary supports. The strain gauges and displacement transducers were zeroed after removal of the temporary supports before application of the additional load post damage. The additional vertical load was applied at a rate of 2.0 mm/min until the system was

unable to sustain any additional load. The specimen with lateral bracing was able to support ultimate load of 520 kN, while the specimen without lateral bracing sustained load of 190 kN. During testing, a punching-type failure of the concrete deck around the loaded area occurred when the load reached 430 kN and 160 kN for specimens with and without lateral bracing, respectively. In the damaged condition, lateral bracing assisted in the load redistribution in longitudinal and transverse direction to the undamaged girder. Strains in the cracked girder were less than 200 microstrains indicating that the load was carried by the uncracked girder in the presence or absence of lateral bracing. In the absence of lateral bracing, additional transverse moment was developed in the deck to redistribute the load to the uncracked girder after fracture. Results indicated enhanced role of crossbeams and concrete deck.

The system redundancy was also evaluated using the criteria developed by Ghosn and Moses (1998) as part of NCHRP 406 study for the damaged bridge. During testing and analysis, dead load was calibrated by applying additional load Δw_d to compensate for the difference between the dead load of the prototype bridge and test specimens. Nonlinear finite element analysis was carried out using ABAQUS (2004) in four stages: 1) additional load Δw_d was applied on the intact girders without concrete deck, 2) installation of concrete deck, 3) introduction of a crack in the girder, and 4) application of load at mid-span of the fractured girder. Analysis yielded live load capacity of 77 kN for the specimen without lateral bracing and 407 kN for the specimen with lateral bracing. The capacity predicted from finite element analysis was lower compared to the experimental capacity. Analysis yielded a system reserve ratio (R_d) of 0.14 and 0.76 for the bridge without and with lateral bracing, respectively, compared to the experimentally evaluated system reserve ratio (R_d) of 0.28 and 0.76, respectively. For the test bridge with lower lateral bracing, the system reserve ratio (R_d) was higher than the minimum value of 0.5 recommended by Ghosn and Moses (1998).

The results showed the importance of lateral bracing in load redistribution in the event of fracture. Lateral bracing enhanced the role of crossbeams and concrete deck in longitudinal and transverse direction. Therefore, the authors recommend providing lateral bracing in case of simply supported two girder bridges as a safety.

Park, et al. (2012): Park, et al. (2012) investigated post fracture redundancy of a two-girder three span continuous bridge through experimental testing and finite element analysis. This study also investigated at the contribution of lateral bracing system on load distribution pre and post damage.

The bridge specimen was 12.5 m long with two girders spaced at 1.2 m center-to-center, supporting 1.8 m wide and 10.5 cm thick concrete deck on the top. The reinforced concrete deck was reinforced with 10 mm reinforcing bars at the top and bottom with spacing of 80 mm c/c in transverse and longitudinal direction, respectively. I-shape cross beams were installed at 1.15 m interval. For the bridge with lateral bracing, L-50 mm x 50 mm x 6 mm angles were used for X-type lateral bracing. Two rows of 19 mm diameter shear connectors were used with spacing of 120 mm near the support and 150 mm in the remaining areas. Half of the three span continuous bridge was constructed as the test specimen and a reaction frame was installed at the end of the overhang span to approximately exert the longitudinal restraint offered by continuous span. Test specimens were supported on roller supports at the exterior and interior support with reaction frame at the overhang. An artificial fracture was introduced through the bottom flange and web at 0.4L from the end support using torch cut. Vertical load was applied using universal testing machine on the fractured girder at 3.43 m from the end support.

The specimen with lateral bracing was able to support an ultimate load of 510 kN, while the specimen without lateral bracing sustained load of 420 kN. Load displacement response of the girders for both test specimens indicated that the lateral bracing system helped in the load redistribution from the fracture girder to the intact girder. In the unbraced specimen, crossbeams experienced a rigid body like motion resulting in lower strains in the cross beams. The braced models behaved like a pseudo-box girder transferring load to the undamaged girder. In both specimens, the strains at the interior support were considerably higher for the fractured girder as compared to the intact girder indicating that the fractured girder behaved like a cantilever, transferring load to the interior support.

Nonlinear finite element analysis was carried out using ABAQUS (2004) to verify the post fracture experimental response and contribution of structural components based on the strain energy stored in each component. Concrete deck, steel girders, crossbeams and lateral bracing were modeled using 4-node shell elements. Park, et al. (2012) provide the stress-strain curves

used to model steel and concrete plasticity. However, no details about the plasticity models used to represent the three-dimensional plasticity are included in the paper. The authors also do not provide details about modeling of composite action between slab and girder. For a bridge without lateral bracing, the strain energy distribution indicates that the fractured girder and deck carried the majority of the load without significant load redistribution. For a bridge with lateral bracing, load was redistributed to the intact girder post fracture. The effect of crossbeam bending stiffness was investigated by increasing its stiffness by 6.2 times. No significant change in the response was observed for both test configurations. The load carrying capacity increased slightly in the braced system.

To investigate the system redundancy, finite element model of a two girder, three span continuous bridge 130 m long and 12 m wide was loaded with two side by side trucks each weighing 432 kN. Analysis yielded live load capacity of 3055 kN for the bridge without lateral bracing and 7720 kN for the bridge with lateral bracing. System redundancy was evaluated using the criteria developed for the damaged bridge by Ghosn and Moses (1998) as part of NCHRP 406 study. The bridge without lateral bracing yielded system reserve ratio (R_d) of 0.41 compared to 1.03 for the bridge with lateral bracing. Although, the system reserve ratio for the unbraced bridge was less than the recommended minimum value of 0.5, the redundancy factor ϕ_{req} was greater than the recommended value of 1.0 (Ghosn and Moses, 1998) for the bridge without lateral bracing ($\phi_{req} = 1.56$) and with lateral bracing ($\phi_{req} = 3.91$) indicating the desired level of post damage redundancy. The experimental and analytical response showed contribution of lateral bracing in the load redistribution in a damaged bridge. Based on the strain energy one can conclude that the lateral bracing carried a significantly more force than the typical design force demands.

2.3 Modeling of composite action

Composite construction with shear studs has been widely used to achieve economical structural designs. Shear studs are designed to transfer the shear force at the steel beam or truss and concrete deck interface. Shear studs provide connection by mechanical interaction between the stud and deck, in addition to the resistance provided by chemical bonding, and friction between the girder top flange and deck. In the event of steel beam or truss fracture, shear studs are subjected to combined tension and shear near the fractured region. If the tensile force on the

shear studs becomes too large, it could result in its failure from yielding of shear studs or breakout of surrounding concrete or both. Therefore, it is necessary to account for the shear-tension interaction of shear studs in the analysis to achieve realistic results. The following section discusses different mathematical models proposed by researchers and implementation of varying levels of composite action in finite element analysis.

2.3.1 Shear Studs Subjected to Shear

Oehlers, et al. (2000): Oehlers, et al. (2000) proposed a mathematical model for the composite action between the concrete deck and steel girder interface (Figure 2-2). It has two components: the shear resisted by shear studs (F_{dwl}) and the friction between the steel and concrete interface ($F_{friction}$). The horizontal shear resistance at the interface was calculated using Equation 2-12 and Equation 2-13 (Oehlers and Johnson, 1987).

$$K_{dwl} = \frac{P_{st}}{d_{sh} (0.16 - 0.0017f_c)} \quad \text{Equation 2-12}$$

$$P_{st} = 4.3A_{sh}f_u^{0.65}f_c^{0.35} \left(\frac{E_c}{E_s} \right)^{0.40} \quad \text{Equation 2-13}$$

where,

K_{dwl}	Shear stiffness of the shear stud (N/mm)
d_{sh}	Diameter of the shank of the shear stud (mm)
f_c	Compressive strength of the concrete (MPa)
P_{st}	Static strength of the shear stud (N)
A_{sh}	Area of the shank of the shear stud (mm ²)
f_u	Tensile strength of the shear stud (MPa)
E_c	Young's Modulus of concrete (MPa)
E_s	Young's Modulus of steel (MPa)

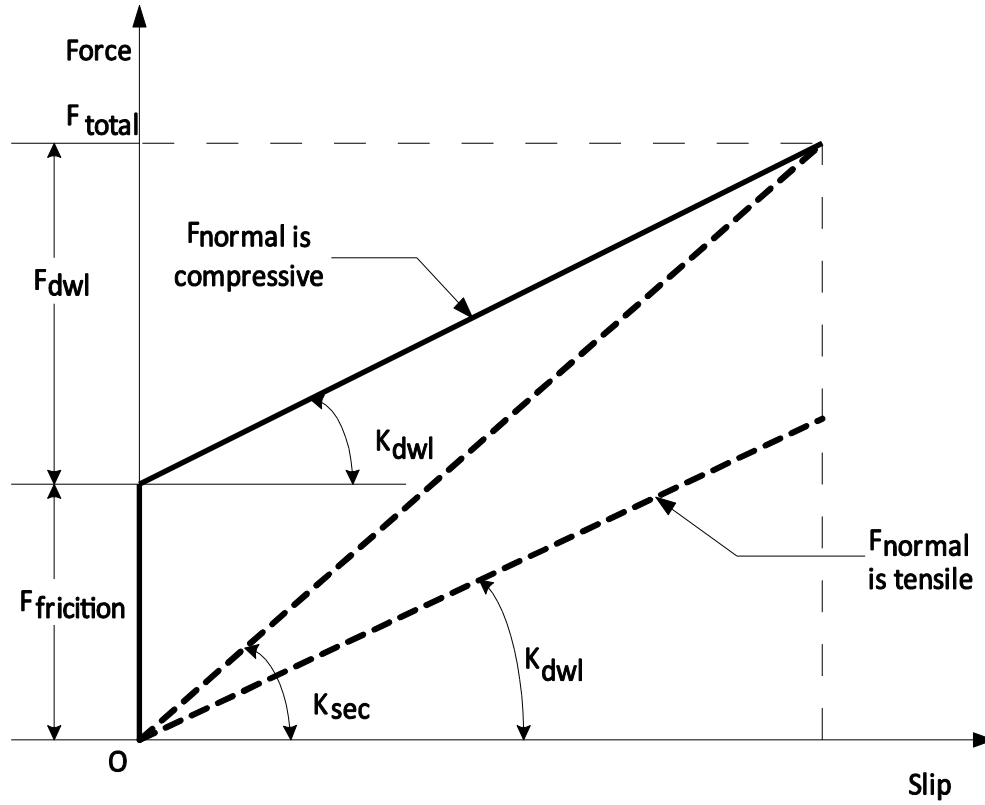


Figure 2-2: Load vs. slip response of shear stud by Oehlers et al. (2000), used under fair use.

Friction at the steel-concrete interface was calculated using the following equation,

$$F_{friction} = \mu F_{normal} \quad \text{Equation 2-14}$$

where,

$F_{friction}$	Frictional resistance (N)
F_{normal}	Normal force between the concrete deck and steel girder interface (N)
μ	Coefficient of friction between steel and concrete

The force resisted by dowel action of the shear stud was calculated using the following equation,

$$F_{dwl} = K_{dwl} \delta \quad \text{Equation 2-15}$$

where,

F_{dwl}	Shear force resisted by the shear stud (N)
K_{dwl}	Shear stiffness of the shear stud (N/mm)

δ Slip between the steel and concrete interface (mm)

The total resistance at the interface can be calculated using Equation 2-16.

$$F_{total} = F_{dwl} + F_{friction} \quad \text{Equation 2-16}$$

where,

F_{total} Total shear resistance (N)

$F_{friction}$ Frictional resistance (N)

F_{dwl} Shear force resisted by the shear stud (N)

Finally, the secant stiffness was calculated using Equation 2-17,

$$K_{sec} = \frac{F_{total}}{\delta} \quad \text{Equation 2-17}$$

This model results in an iterative procedure in order to determine the normal force acting at the interface.

Ollgaard, et al. (1971) : Ollgaard, et al. (1971) developed a relationships for strength and behavior of shear studs embedded in normal weight and lightweight concrete using a series of pushout specimens. Regression analysis of test results yielded Equation 2-18 for strength and Equation 2-19 for load vs. slip relationship of shear stud.

$$Q = Q_u (1 - e^{-18\Delta})^{2/5} \quad \text{Equation 2-18}$$

$$Q_u = 1.106 A_s f_c^{0.3} E_c^{0.44} \quad \text{Equation 2-19}$$

where,

f_c' Concrete compressive strength (ksi)

A_s Shear stud cross sectional area (in.²)

E_c Modulus of elasticity of concrete (ksi)

Q_u Shear strength of shear stud (kips)

Q Applied load (kips)

Δ Slip (in.)

Baskar, et al. (2002) : Baskar, et al. (2002) studied behavior of composite steel girders in negative bending and shear using nonlinear finite element analysis in ABAQUS (1998). Steel girders were modeled using eight node thin shell elements and the concrete slab was modeled using 20 node quadratic brick elements with reinforcing bars as a smeared layer. An elastic perfectly plastic relationship was used to model structural steel components, while the smeared crack model was used to model the concrete deck.

Two different techniques were used to model the composite action. In the first method, surface interaction technique in ABAQUS (1998) was used to model the composite action. This technique allows incompatible strains and slip between steel-concrete interface. More specifically, the bond strength at the steel and concrete interface and strength of shear stud were combined. A bilinear curve similar to the shear force vs. slip curve for the shear stud was used to model the slip. The surface behavior option that allows tensile stresses between two contact surfaces was used to model the vertical tensile strength of the stud. This method could not explicitly capture local effects, such as slab failure and stud connector failure. In the second method, general beam elements were used to model the shear studs and the area of the beam element was modified to account for strength of embedded shear stud in concrete.

Both techniques were evaluated by comparing test results from experimental evaluation of composite steel girders subjected to negative bending and shear by Allison, et al. (1982). The results obtained using the surface interaction technique underpredicted the ultimate strength within 10%. However, even though the second technique was able to match closely the results in the initial stage of the deformation, it had a slower convergence beyond peak load and required more mesh refinement than the first method to agree with the experimental results.

Abdullah and Easterling (2009): In this study, Abdullah and Easterling (2009) used explicit nonlinear finite element analysis to study a steel deck composite slab subjected to four point bending. In this study, a concrete slab was modeled using linear continuum elements and a steel deck was modeled using linear shell elements. Concrete plasticity was modeled using the brittle cracking model in ABAQUS/Explicit, which assumes compressive behavior to be linear elastic. Horizontal shear bond using the radial-thrust connector element in ABAQUS (2003). Horizontal shear stress vs. end slip relationship based on the bending test data performed by Abdullah (2004) was assigned to the radial component and a stiff value was assigned to the

thrust component (in direction perpendicular to the plane of composite slab) to avoid overlap between the concrete and steel elements. Interaction between the radial and thrust component was not considered in the analysis assuming that this effect would be included implicitly in the experimental shear stress vs. end slip relationship. Quasi-static analysis was carried out using the explicit solution method by gradually increasing displacement and eliminating any significant effect of inertia. The proposed method was able to predict load deflection response similar to the four point bending test response (Abdullah, 2004) up to ultimate load. The ultimate load analysis was unable to capture the descending branch of the experimental load deflection response.

Mirza and Uy (2010): Mirza and Uy (2010a) carried out this study to understand the long-term behavior of composite steel-concrete push-out tests for solid and profiled slabs using nonlinear finite element analysis. In this study, the concrete deck and steel girders were modeled using linear solid elements. Thirty node quadratic solid elements were used to model the shear connector. Linear shell elements were used to model the profiled steel sheeting and linear truss elements were used for reinforcing steel. Taking advantage of the symmetry in the experimental setup, only one quarter of the experimental setup was modeled in ABAQUS (2006) with appropriate boundary conditions and a static concentrated load at the web center. Shear studs were connected to the steel beam using a tie constraint. No discussion about the contact between the shear studs and concrete deck is provided in the paper. Concrete plasticity was represented using the damaged plasticity model in ABAQUS (2006). Creep and shrinkage of concrete was accounted for in an ad-hoc way by modifying the elastic modulus of concrete using a creep coefficient and aging coefficient of 0.5. Steel beam, profile sheeting and reinforcing bars were modeled as elastic plastic material with strain hardening. Shear studs were modeled in a similar fashion without strain hardening.

The modeling technique was verified by comparing with the results from the solid slabs tested by Lam and El-Lobody (2005). The finite element model was able to predict the force vs slip response closely with the predicted failure load approximately 2.6% lower than the experimental failure load. To study the effect of creep and shrinkage on failure load, analyses were carried out with modified concrete modulus for concrete age ranging from 0 to 2000 days. Creep and shrinkage of concrete resulted in reduced strength and stiffness of solid and profiled slabs with the solid slab exhibiting higher slip and strength as compared to the profiled slab. Analysis of

solid slabs showed an average reduction of 12% in the shear resistance, while profiled slab analysis showed 16% reduction in shear resistance at the end of 2000 days. The dominant failure mode in solid slabs was the stud fracture near the weld connecting the shear stud to the steel beam, while the profiled slab failed by formation of crack in the middle of the slab along the trough of profiled sheeting. Stud push-out test strength increased with higher concrete compressive strength. Parametric study showed change in failure mode from concrete failure at lower concrete strength to stud shearing failure at higher concrete strength. Also the smaller diameter stud showed lower capacity compared to larger diameter studs.

Kwon, et al. (2012): Kwon, et al. (2012) carried out research to develop preliminary design recommendations for development of composite action in existing noncomposite bridge girders using postinstalled shear connectors. A nonlinear finite element model was developed in ABAQUS (2007). Both the steel beam and concrete slab were modeled using linear shell elements. Reinforcing steel in the slab was modeled as a smeared layer in the shell elements. Contact between the steel beam and concrete slab was defined using contact interaction in ABAQUS (2007) with no limit on the pressure that could be transmitted between the two surfaces. Based on the experimental observations, separation at the steel-concrete interface was not allowed once they came in contact. Shear connectors were modeled using CARTESIAN connectors with nonlinear behavior and failure criteria. The connectors were removed from the analysis if the failure criteria were met. Load vs slip relationship developed by Ollgaard, et al. (1971) with the maximum slip based on the beam tests by Kwon et al. (2011) was used in the analyses. Pressure load was applied at the midspan to represent the test loading. Analysis was carried out using Riks method. Structural steel and reinforcement were modeled as elastic perfectly plastic. Concrete plasticity was represented using the smeared crack model.

The finite element model was verified by comparing against the tests by Kwon, et al. (2011). The finite element model was able to predict the peak load and load vs displacement response closely up to shear connector failure. After failure of the shear connectors, the model failed to capture the interface slip at the ends of specimen and load vs. displacement response. Using the same modeling technique, steel beams ranging from W27 to W36 spanning between 9 to 15m were analyzed with partial composite connection between 10 to 50% of full composite connection. Results from analysis were compared against the AISC design provisions (2010). AISC design

equations were able to predict the peak load and elastic stiffness of partially composite beams. Beam ductility improved with the increase in partial composite action. Deep steel beams showed lower ductility. Increased span length increased the slip demand at the steel concrete interface.

Nguyen and Kim (2009): Nguyen and Kim (2009) studied the behavior of large shear studs greater than 22 mm in diameter in composite beams with solid slabs using nonlinear finite element models of the push-out tests. One quarter of the push-out test arrangement was modeled taking advantage of the symmetry in the experimental setup. The concrete slab, steel beam and shear studs were modeled using eight node continuum/solid element. Reinforcement was modeled with truss elements which were embedded in the concrete slab. Shear studs were tied to the concrete surface eliminating the relative slip between two surfaces. The greased interface between the beam flange and concrete slab was modeled using frictionless contact interaction. Loading was applied by imposing displacement at the top of the steel beam. The analysis was carried out using Riks method with ABAQUS (2006).

Concrete plasticity was simulated using the damaged plasticity model with a material dilation angle of 20° and eccentricity of 0.1. Compressive behavior was modeled using uniaxial stress-strain curve in EN 1992 (2004) with an ultimate compressive strain of 0.01. Structural steel and reinforcement were modeled using elastic perfectly plastic material, while a tri-linear stress-strain curve was used to model shear studs with material damage and failure options in ABAQUS (2006). The proposed modeling technique was validated by comparing test results from Gattesco and Giuriani (1996), Loh, et al. (2004) and Lee, et al. (2005). Finite element response showed good agreement with failure load and failure mechanism. The proposed modeling technique was able to predict the experimental response with a mean accuracy of 99% (Coefficient of variation = 0.028), and, therefore, the same modeling technique was used to further study the effect of stud diameter and concrete strength on capacity of shear stud connection. A test setup similar to Lee, et al. (2005) was used with different stud diameters of 22, 25, 27, 30 mm and concrete strength between 25-60 MPa. Increase in stud diameter and concrete strength increased the failure load but reduced ductility.

Fu and Lu (2003) : Fu and Lu (2003) investigated the behavior of composite girders under service loads. Under service loads, steel girders were assumed to deform elastically, while concrete deck showed localized plasticity. Shear studs were modeled discretely by connecting concrete deck elements and top flange elements using bar elements. This is similar to using two independent linear springs with the axial stud stiffness given by Equation 2-20, and the tangential stiffness by based on Equation 2-21 the load-slip behavior of the shear stud proposed by Yam and Chapman (1972). The load-slip behavior of the shear stud (Yam and Chapman, 1972) is defined using Equation 2-22.

$$K_n = \frac{E_s A_s}{h_s} \quad \text{Equation 2-20}$$

where,

E_s Elastic modulus of shear stud
 A_s Cross sectional area of shear stud
 h_s Height of shear stud

$$K_t = abe^{-b\gamma} \quad \text{Equation 2-21}$$

$$P = a(1 - e^{-b\gamma}) \quad \text{Equation 2-22}$$

where,

$$a = \frac{P_1^2}{2P_1 - P_2} \quad \text{Equation 2-23}$$

And

$$b = \frac{1}{\gamma_1} \times \log_e \left(\frac{P_1}{P_2 - P_1} \right) \quad \text{Equation 2-24}$$

Where,

P_1 and P_2 Load
 γ_1 Interface Slip

The two load points P_1 and P_2 on the load slip curve were chosen such that the interface slip at P_2 is twice that at P_1 .

Jung (2006): Jung (2006) investigated the inelastic behavior of horizontally curved composite I-girder bridges using nonlinear finite element analysis. In the finite element model, concrete deck and girder webs were modeled using four node shell elements. Two node beam elements were used to model the girder flanges and transverse stiffeners.

Composite action between the concrete deck and girder top flange was modeled with two different techniques. In the first method, beam type multi-point constraints were used to model the composite action with no slip at the interface. In the second method, interface flexibility was modeled explicitly using the shear vs. slip relationship suggested by Ollgaard, et al. (1971). Shear vs. slip response was modeled using a combination of nonlinear rotational spring element and rigid beam at the location of each shear stud along the girder length. Multipoint constraint was used to connect the slab node to rigid beam element. A rotational spring was used to connect the top flange node to the rigid beam element to capture the interface flexibility. The proposed model is unable to capture the repeated loading tests, since it cannot represent the residual slip accumulated throughout the loading history. The proposed modeling techniques were verified by comparing its results with the experimental data from simply supported horizontally curved composite bridge tested at Federal Highway Administration laboratories. Load displacement curves using both methods yielded an identical response except at higher load levels where the second method yielded an additional vertical deflection. Because of simplicity in implementation of the first method, multipoint constraints were used in the research.

2.3.2 Shear Studs Subjected to Combined Tension and Shear

AISC (2010), ACI 318-11 (2011) and PCI Design Handbook, 6th edition (2004): AISC (2010), ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004) provide design guidelines for headed studs under tension, shear, or a combination of tension and shear.

Shear strength: Three failure modes considered for the failure of headed studs loaded in shear are as follows:

1. **Headed stud shear failure:** Headed studs failure occurs by formation of shear failure plane in the stud shaft. Stud shear failure is taken as the upper bound for connection strength, because once the steel shear strength has reached its maximum value no additional change in embedment, edge distance or reinforcing can increase the connection strength.

2. Concrete breakout failure: Concrete breakout failure occurs near the free edge by forming failure planes in the concrete surrounding the headed studs.
3. Concrete pryout failure: Concrete pryout failure occurs far from the free edge by formation of a concrete spall in the direction perpendicular to the applied shear force.

Pallarés and Hajjar (2010a) assessed headed stud shear strength design equations versus 391 monotonic and cyclic experiments from the literature. AISC (2010) stud shear strength equation predicted steel failure accurately if the resistance factor was included to ensure an acceptable level of reliability, compared to what is used in ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004). The authors deemed concrete pryout failure as governing failure over concrete breakout failure in composite girders. In ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004) concrete failure predictions were found to be conservative compared to AISC (2010). Pallarés and Hajjar (2010a, Pallarés and Hajjar, 2010b) concluded that strength under cyclic loading can be taken equal to 75% of the monotonic strength as long as the monotonic shear strength is predicted within reasonable statistical accuracy. The authors do not provide further explanation about the level of reasonable statistical accuracy.

Tensile strength: Four failure modes considered for the failure of headed studs in tension are as follows

1. Headed stud tension failure: Headed studs failure occurs when it reaches ultimate plastic strength in tension. This failure mode is taken as the upper bound for connection strength.
2. Pullout failure: Concrete directly under the stud head crushes reducing the connection stiffness and mobilizing the headed stud resulting in to pullout.
3. Side face blowout and splitting failure: Side face blowout and splitting occurs near free edge by formation of failure plane originating from the stud head. In the case of composite girders, this failure mode is unlikely.
4. Concrete breakout failure: Concrete breakout occurs when failure planes form in concrete surrounding the stud or stud group starting at the stud head mobilizing the separated concrete mass.

In the second paper, Pallarés and Hajjar (2010b) assessed tensile strength design equations versus 222 monotonic and cyclic experiments from the literature. The authors deemed the

nominal tension strength equation from ACI 318-11 (2011) with resistance factor of 0.75 as adequate to determine the strength of the headed stud with sufficient embedment to reach ductile failure. The concrete failure equations were assessed versus the 163 test results out of 222 tests previously. A subset was considered to eliminate tests influenced by group effect. PCI Design Handbook, 6th Edition (2004) and ACI 318-11 (2011) equations with resistance factors were found to be conservative with an average test-to-predicted ratio of 1.443.

Shear-tension interaction: PCI Design Handbook, 6th Edition (2004) and ACI 318-11 (2011) have adopted an elliptical (Equation 2-25) and a linear (Equation 2-26) equation, respectively, to evaluate the nominal strength of the headed stud under combined shear and tension. ACI 318-11 (2011) interaction assumes full stud capacity in shear or tension when shear-tension interaction ratio is less than 20% making interaction tri-linear.

$$\left(\frac{Q_{rt}}{\phi_t Q_{nt}} \right)^{5/3} + \left(\frac{Q_{rv}}{\phi_v Q_{nv}} \right)^{5/3} \leq 1.0 \quad \text{Equation 2-25}$$

$$\left(\frac{Q_{rt}}{\phi_t Q_{nt}} \right) + \left(\frac{Q_{rv}}{\phi_v Q_{nv}} \right) \leq 1.2 \quad \text{Equation 2-26}$$

where,

Q_{nt}	Nominal tensile strength (kips)
Q_{rt}	Required tensile strength (kips)
Q_{nv}	Nominal shear strength (kips)
Q_{rv}	Required shear strength (kips)
ϕ_t	Strength reduction factor in tension
ϕ_v	Strength reduction factor in shear

Pallarés and Hajjar (2010b) assessed headed shear-tension interaction equations versus 54 test results from the literature and found the elliptical interaction equation by PCI Design Handbook, 6th Edition (2004) to be more accurate compared to tri-linear interaction equation by ACI 318-11 (2011). Pallarés and Hajjar (2010b) also proposed the following requirements to ensure ductile failure in the stud shank rather than in the surrounding concrete for headed studs subjected to tension and shear-tension interaction:

- a. Minimum stud length to diameter ratio of 8

- b. Minimum stud head diameter equal to 1.63 times stud diameter
- c. Minimum edge distance of 1.5 times the effective embedment length of stud
- d. Minimum spacing of 3 times the effective embedment length of stud

Review of research conducted at University of Texas at Austin: Barnard, et al. (2010) investigated the redundancy of fracture critical twin steel box-girder bridges using nonlinear finite element analysis. Box girder components were modeled using 8-node shell elements, while concrete deck and barrier rail were modeled using 8-node solid elements. Concrete plasticity was modeled using cast iron metal plasticity, while structural steel and reinforcement plasticity were modeled using classical metal plasticity. More details about the modeling methodology are provided in Section 2.2.2. The following sections discuss behavior of shear studs in shear and tension followed by the shear-tension interaction model used in this study by Barnard, et al. (2010).

a) Shear strength and load slip behavior: Topkaya et al. (2004) proposed equations for the ultimate shear strength (Q_u) and load slip behavior of shear stud based on the push-out tests. They are as follows:

$$Q_u = 2.5A_{sc}(f'_c E_c)^{0.3} \quad \text{Equation 2-27}$$

where,

A_{sc}	Cross-sectional area of shear stud (in. ²)
f'_c	Concrete compressive strength (ksi)
E_c	Elastic modulus of concrete (ksi)

$$Q = Q_d \frac{3\left(\frac{\Delta}{0.03}\right)}{1 + 2\left(\frac{\Delta}{0.03}\right)} \quad \text{Equation 2-28}$$

$$Q_d = 1.75A_{sc}(f'_c E_c)^{0.3} \quad \text{Equation 2-29}$$

where,

Δ	Slip of shear stud (in.)
Q_d	Shear load at 0.03 in. slip (kips)

b) Pull-out strength and load-deflection behavior: Sutton (2007) and Mouras (2008) studied the pull-out strength of shear studs using small deck specimens designed to represent the transverse bending of a concrete bridge deck between the points of inflection of a twin box girder. Response varied depending on the haunch depth, shear stud length, number of studs and their arrangement. Mouras (2008) proposed the following modifications on ACI 318-11 (2011) anchor strength equation to quantify the stud pull-out strength.

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{g,N} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} N_b \quad \text{Equation 2-30}$$

$$N_b = k_c \sqrt{f'_c} h_{ef}'^{1.5} \quad \text{Equation 2-31}$$

$$h_{ef}' = h_{ef} - h_h \geq \frac{w_h}{3} \quad \text{Equation 2-32}$$

$$\Psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5h_{ef}'} \quad \text{Equation 2-33}$$

$$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2e_N'}{3h_{ef}'}\right)} \leq 1.0 \quad \text{Equation 2-34}$$

where,

N_{cbg}	Design concrete breakout strength of a stud group of studs (lb.)
A_{Nc}	Projected concrete cone failure area of a stud group ($= 3h_{ef}' w_h$ for $w_h \leq 3h_{ef}'$) (in. ²)
A_{Nco}	Projected concrete cone failure area of a single stud ($= 9h_{ef}'^2$) (in. ²)
$\Psi_{g,N}$	Group effect modification factor 1 stud : 1.00 2 studs spaced transversely: 0.95 3 studs spaced transversely : 0.90 Stud spaced longitudinally : 0.80
$\Psi_{ec,N}$	Eccentric load modification factor
$\Psi_{ed,N}$	Edge distance modification factor ($=1$ when $c_{a,min} \geq 1.5h_{ef}'$)

$\Psi_{c,N}$	Cracked concrete modification factor Cracked concrete with a stud installed : 1.00 Uncracked concrete : 1.25
N_b	Concrete breakout strength of a single isolated stud (lb.)
k_c	24 (cast-in-place shear studs)
h_{ef}	Effective height (in.)
h_{ef}'	Effective height, excluding the haunch height (in.)
h_h	Haunch height (in.)
w_h	Width of haunch in the cross-section of a bridge span (in.)
$c_{a,min}$	Smallest edge distance measured from center of stud to the edge of concrete (in.)
e'_N	Eccentricity of resultant stud tensile load (in.)

The experimental data indicated a linear load-displacement response until the stud reaches pull-out strength. Beyond the pull-out strength, the load dropped quickly because of brittle failure in concrete. This behavior was idealized as bilinear where tensile force increased linearly up to the stud pullout strength and then reduced to zero at a displacement 12 times the displacement at the stud pullout strength. Sutton (2007) and Mouras (2008) tested specimens with one, two and three studs per row. However, due to limited amount of test data available to quantify the effect of the number of studs per row on the displacement at the stud pullout strength, results for the three studs per row were used in the regression analysis to construct Equation 2-35.

$$U_m = \frac{\sqrt{f'_c} h_{dh}}{5700} \Psi_{h,U} \quad \text{Equation 2-35}$$

$$\Psi_{h,U} = 0.038 + 0.346 \frac{h_{ef}'}{h_d} \quad \text{Equation 2-36}$$

where,

U_m	Relative displacement at pull-out strength (in.)
$\Psi_{h,U}$	Haunch height effect modification factor
h_d	Deck height (in.)
h_{dh}	Deck height including haunch (in.)

c) Stud shear-tension interaction: A linear damage model was proposed in this study, which reduced the shear resistance of the connector elements when the vertical tension force on the shear studs exceeds studs tensile strength. After damage initiation, shear and tension resistance was reduced linearly to zero. This model does reduce the shear and tension resistance of the stud once it reaches its tensile strength, but it does not account for shear-tension interaction.

Barnard, et al. (2010) implemented this model in the redundancy investigation of twin box girder bridges. During the second test when the bottom flange and web of one of the twin box girders was fractured, the finite element model over predicted the separation length on the inside of the fractured girder and its deflection. The observed cracking pattern suggested haunch failure due to the combination of tension and shear. Exclusion of shear-tension interaction could have modified force redistribution resulting in a difference between the measured and predicted response. Further details about the experimental and analytical investigation are summarized in Section 2.2.2.

Mirza and Uy (2010b): Mirza and Uy (2010b) carried out study to understand the behavior of shear connection in composite beams with solid and profiled slabs subjected to combined shear and tension using nonlinear finite element analysis. In this study, the concrete deck and steel girders were modeled using linear solid elements. Quadratic solid elements were used to model the shear connectors. Linear shell elements were used to model the profiled steel sheeting, and linear truss elements were used for reinforcing steel. Reinforcement was embedded in the concrete deck assuming perfect bond between the surrounding concrete and reinforcement.

Concrete plasticity was represented using the smeared crack model in ABAQUS (2006). In tension, concrete strain increased linearly up to cracking. After cracking, the tensile stress was decreased linearly to zero at a strain 10 times the strain at cracking. Steel beam, profile sheeting and reinforcing bars were modeled using elastic plastic material with strain hardening. Shear studs were modeled in a similar fashion without strain hardening. Mirza and Uy (2010b) does not provide details about the interaction between the shear studs and concrete, concrete slab and top of the beam flange, shear studs and top of the beam flange. Interaction between surfaces could have significant impact on the response. Taking advantage of the symmetry in the experimental setup, only one quarter of experimental setups were modeled in ABAQUS (2006) with

appropriate boundary conditions to represent the experimental setup. Analysis was carried out using RIKS method in ABAQUS (2006).

The modeling technique was verified by comparing with the experimental test results from Saari, et al. (2004). The proposed model was able to predict the failure load within 5% of the test results and predicted the load vs. displacement response with reasonable accuracy. Saari, et al. (2004) studied the effect of combined shear and tension on two steel reinforcement configurations. The confined reinforcement scheme showed better ductility in pure shear, while the cage reinforcement scheme that provided confinement around the shear stud showed better ductility in combined shear and tension. Additionally, the cage reinforcement scheme yielded higher failure load. Mirza and Uy (2010b) investigated the effect of concrete strength, stud height and diameter, and thickness of concrete slab on response. Shear strength increased with increase in slab thickness for the model subjected to shear. Longer and large diameter shear studs showed increase in capacity for both slab types. With increase in the concrete strength, failure mode changed from concrete failure at lower concrete strength to stud shear failure at higher concrete strength. Results from the parametric study were conservative when compared against the interaction equation proposed by McMackin, et al. (1973) for steel anchors in solid slabs.

2.3.3 Non-composite Action

Behavior of non-composite bridges is considerably different compared to fully or partially composite bridges. In the absence of shear studs, interface resistance comes from the frictional contact at the slab and top flange interface that is a function of the normal force at the interface. The literature available on the analytical and experimental investigation of non-composite bridges under service loads or ultimate loads is limited. This section reviews some of the literature on the live load performance and ultimate load capacity of non-composite bridges.

Breña, et al. (2012): Breña, et al. (2012) performed a field evaluation of a damaged non-composite steel girder bridge. The bridge selected for study was a three span continuous steel girder bridge, 214.5 ft long and 30 ft wide, supporting 7.5 in. thick non-composite reinforced concrete deck. Bottom flanges of three out of five girders were damaged from impact of an over-height vehicle. Therefore, live load testing of the bridge was carried out to study the effects of damage on bridge response. For live load testing, the central span was instrumented with strain gauges in the regions where maximum positive and negative moments were expected

under different truck positions. Testing was carried out using two loaded dump trucks moving at a speed of 5 mph to avoid any dynamic response. During live load testing, moments, strains and neutral axis location were close to values calculated using elastic composite section properties.

The composite action observed by Breña, et al. (2012) during testing might be because of the frictional resistance at the slab and steel interface. Additionally, comparison of strains in the bottom flange of damaged girders with adjacent undamaged girders indicated development of alternate load paths in the bridge superstructure.

Zhou, et al. (2004): Zhou, et al. (2004) studied the effect of superstructure flexibility on deck strength through non-linear finite element analysis. Experimental data from Petrou, et al. (1996) tests were used to validate the model and study effect of composite action, girder spacing to slab thickness denoted as slab slenderness ratio, and presence of diaphragms on bridge deck strength. Petrou, et al. (1996) tested 1/6.6th scale physical models of non-composite bridge under concentrated monotonically increasing static wheel load applied at different location. Results from the test when load was applied at the mid span were used to validate the finite element model developed in ANSYS (2001). ANSYS elements Shell63, Link 8 and Solid 65 were used to model steel girders, diaphragms and concrete deck respectively. Reinforcement was modeled as a smeared layer with elastic perfectly plastic material properties. Concrete plasticity was modeled using Drucker-Prager plasticity with multi-linear isotropic hardening. The non-composite actions at the steel-concrete interface, vertical and transverse degrees of freedom of the coincident nodes were coupled, while allowing independent motion in the longitudinal direction. Analysis encountered convergence problems beyond load 5500 lb., but up to that load, the finite element showed good agreement with the experimental results. In addition, analysis showed good correlation with the cracking pattern. Although the analysis model had convergence problems at higher loads, it predicted the cracking pattern reasonably well and therefore was used in the parametric study at load levels below 5500 lb.

Models were reanalyzed by assuming composite action at the interface by coupling all degrees of freedom at the coincident loads. The composite slab showed lower deflection and supported more load compared to the non-composite model. In addition, the cracking pattern was confined between the loading point and adjacent girders for the composite deck. For the non-composite deck the cracking was not confined and propagated in longitudinal and transverse directions.

Diaphragms stiffened the response of the non-composite slab considerably more compared to the composite slab. In both cases, local slab failure near the loading point was observed when diaphragms were present and it limited the amount of cracking. Higher slab slenderness ratio yielded higher deflection and cracking in case of non-composite slabs. The composite and non-composite slabs with higher slenderness ratio showed cracking at lower load levels.

2.4 Imperfections and residual stresses

Residual stresses are stresses that arise in hot-rolled, welded, or flame-cut structural steel components due to differential cooling of cross section during manufacturing. Presence of residual stresses results in early yielding of cross section, resulting in loss of stiffness and reduction in load carrying capacity. Therefore, residual stresses can have significant impact on the brittle fracture, fatigue, stress corrosion and buckling strength of structural steel sections (ECCS, 1976). During fabrication and erection procedures, imperfection in the form of out of straightness of the member and its attachments, out of flatness of panels, twist of local stiffeners or flanges are introduced in the structure. Imperfections along with the residual stresses can have significant impact on the overall behavior of the structure. The following section provides a summary of imperfection and residual stress distributions specified in the design standards and literature.

2.4.1 Design Standards

ECCS (1976): ECCS (1976) provides a summary of residual stress distribution and imperfections in steel components.

Residual Stresses: ECCS (1976) provides simplified equations to represent the residual stress distribution developed during manufacturing processes.

- a) Rolled plates: Mean residual stress distribution across rolled plated was found to be parabolic with compression at the edges and tension in the center (Figure 2-3.a). For a rolled plate of thickness t (mm) and width b (mm), the value of the compressive stress (σ_c) and tensile stress (σ_t) in MPa is given by the following equations

$$\sigma_c = 220 \left[1 - \frac{b}{37t} \right] \quad \text{Equation 2-37}$$

$$\sigma_t = 0.5\sigma_c \quad \text{Equation 2-38}$$

These equation are only valid in the range $0 < b < 28t$.

- b) Flame cut plates: Flame cutting induces tensile stress equal to the yield stress in the narrow strip of plate next to the cut. For a plate of thickness t (mm) and yield stress f_y (MPa), which is flame cut along longitudinal axis, the width of tension block c (mm) is given by

$$c_f = \frac{1100\sqrt{t}}{f_y} \quad \text{Equation 2-39}$$

For plate cut along one edge, the longitudinal residual stresses (Figure 2-3.b):

$$\sigma_t = f_y \frac{c \times (2b + c)}{(b - c)^2} \quad \text{Equation 2-40}$$

$$\sigma_c = f_y \frac{c \times (4b - c)}{(b - c)^2} \quad \text{Equation 2-41}$$

For a plate with cuts along both edges, the uniform longitudinal residual stress (Figure 2-3.c):

$$\sigma_c = f_y \frac{2c}{(b - 2c)} \quad \text{Equation 2-42}$$

- c) Welded plates and shapes: During welding, a tension block is produced next to the welding in the same way as the flame cut. The stress distribution from the plate welding along one or both plate edges is similar to the stress distribution in flame cut plates (Equation 2-43). The stress distribution in a center weld plate (Figure 2-3.d) is given by

$$c_w = \frac{12000\rho A_w}{f_y \sum t} \quad \text{Equation 2-43}$$

where

ρ	Process efficiency factor
A_w	Cross sectional area of the added weld metal (mm ²)
$\sum t$	Sum of the plate thickness meeting at the weld (mm)
f_y	Plate yield stress (MPa)

The process efficiency factor depends on the welding process used. Efficiency factors for different welding processes are tabulated in Table 2-2.

Table 2-2: Welding processes efficiency factors

Process	ρ
Submerged arc	0.9
Cored wire CO ₂	0.85
Manual	0.8
Fusearc	0.75
MIG: spray	0.62
Dip	0.42

The width of tension block for welding of previously flame cut edge does not result in algebraic addition of tension block widths, because welding heat tends to relieve the tensile stresses from flame cutting. The final width of tension block (c_{fw}) is estimated as

$$c_{fw} = (c_f^4 + c_w^4)^{0.25} \quad \text{Equation 2-44}$$

Equivalent tension block (c') produced by intermittent welds is given by

$$c' = c \frac{w}{w + m} \quad \text{Equation 2-45}$$

where, w is the weld length (mm) and m is the miss length (mm).

The effective width of tension block width for two welds spaced d (mm) apart is given by

$$c_2 = c + 0.5d, d \leq 2c \quad \text{Equation 2-46}$$

Residual stresses were assumed constant through the thickness for thin plates with thickness less than 25 mm.

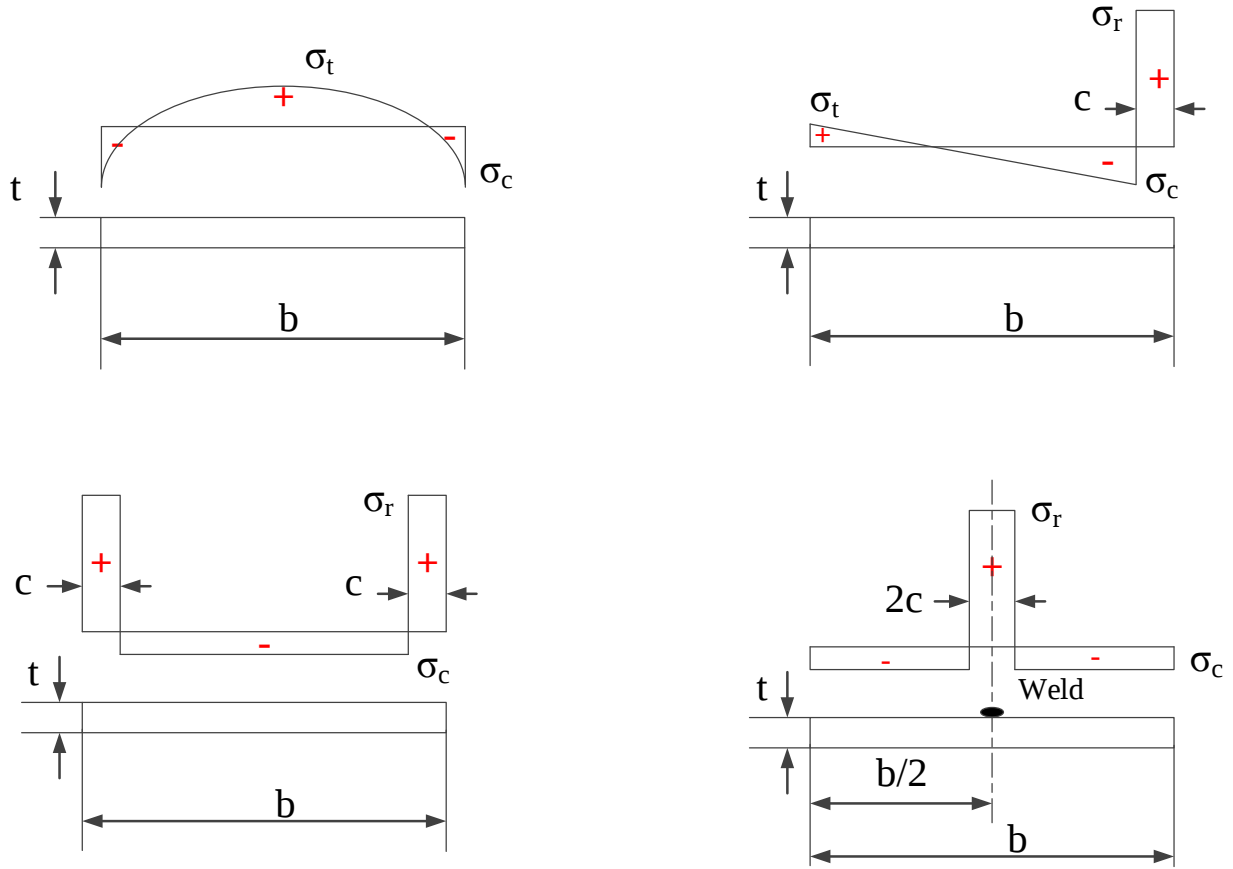


Figure 2-3: Residual stress distributions for flame cut, welded and rolled plates (ECCS, 1976), used under fair use.

- d) Rolled shapes: The magnitude and distribution of residual stresses in hot rolled shape (Figure 2-4) depend on the section geometry. Absolute values of residual stress were found to be unaffected by the yield stress. These equations satisfy the longitudinal equilibrium provided the material is below yield. The residual stress distribution in the rolled shapes is given by Equation 2-47 through Equation 2-49

$$\sigma_{c1} = 165 \left[1 - \frac{ht_w}{2.4Bt} \right] \quad \text{Equation 2-47}$$

$$\sigma_{c2} = 100 \left[1.5 + \frac{ht_w}{2.4Bt} \right] \quad \text{Equation 2-48}$$

$$\sigma_t = 100 \left[0.7 + \frac{ht_w}{2.4Bt} \right] \quad \text{Equation 2-49}$$

where

σ_{c1}	Compressive stress at the flange toes (MPa)
σ_{c2}	Compressive stress in the center of the web (MPa)
σ_t	Tensile stress at the web to flange junction (MPa)
h	Web depth (mm)
t_w	Web thickness (mm)
B	Flange breadth (mm)
t	Flange thickness (mm)

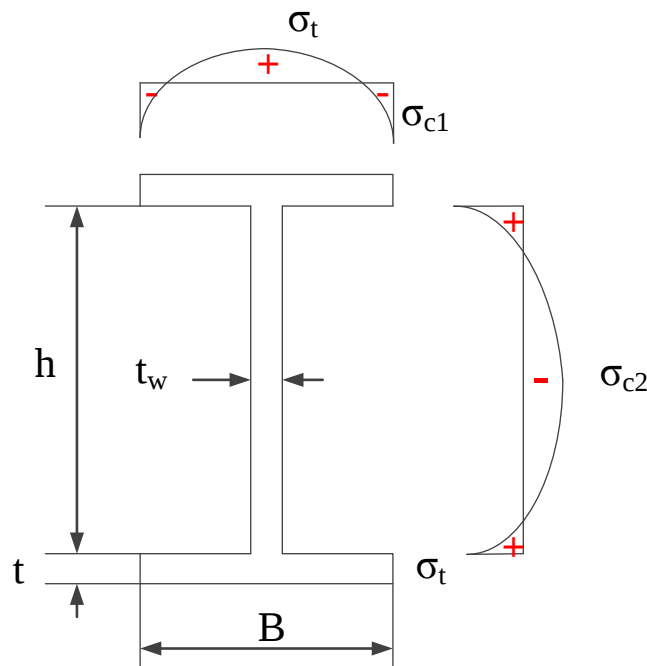


Figure 2-4: Residual stress distributions for rolled sections (ECCS, 1976), used under fair use.

Imperfections: ECCS Manual on Stability of Steel Structure (ECCS, 1976) represents the effect of member imperfection through two parameters, load eccentricity at the member ends and initial out of straightness of members. ECCS experimental program yielded values from 0 to 2.0 mm for load eccentricity at the member ends and initial out of straightness equal to $1/500$ to $1/3350$ of total member length.

EN 1993-1-5 Annex C (2005): EN 1993-1-5 Annex C (2005) provides guidance for the use of finite element methods for evaluation of strength and stability of plated steel structures. Specification recommends including geometrical imperfections and residual stresses in finite element method to evaluate strength and stability of plated steel structures. The specification leaves the choice of residual stress distribution to engineering judgment. In the absence of more refined analysis of the geometric and structural imperfections, it recommends use of equivalent geometric imperfections values tabulated in Table 2-3 (EN 1993, 2005). The equivalent geometric imperfections include out of straightness of the member and its attachments, out of flatness of panels, and twist of local stiffener or flange. Combining these imperfections, it recommends including leading imperfection with full magnitude and the accompanying imperfections reduced to 70% of full magnitude resulting in several combinations for analysis.

Table 2-3: Equivalent geometric imperfections (EN 1993, 2005)

Type of Imperfection	Component	Shape	Magnitude
Global	Member with length l	Bow	$l/350 - l/100$
Global	Longitudinal stiffener with length a	Bow	$\min(a/400, b/400)$
Local	Panel or subpanel with short span a or b	Buckling shape	$\min(a/200, b/200)$
Local	Stiffener subjected to twist	Bow twist	$1/50$

BSK99 (National Board of Housing Building and Planning, 2003): Swedish regulation for steel structures BSK99 (National Board of Housing Building and Planning, 2003) recommends considering the effect of residual stresses on stiffness and resistance of structure. Figure 2-5 shows the schematic residual stress distribution that can be used in the analysis and design of rolled and welded I shaped girders with maximum element thickness of 40 mm and welded rectangular hollow sections with the same wall thickness.

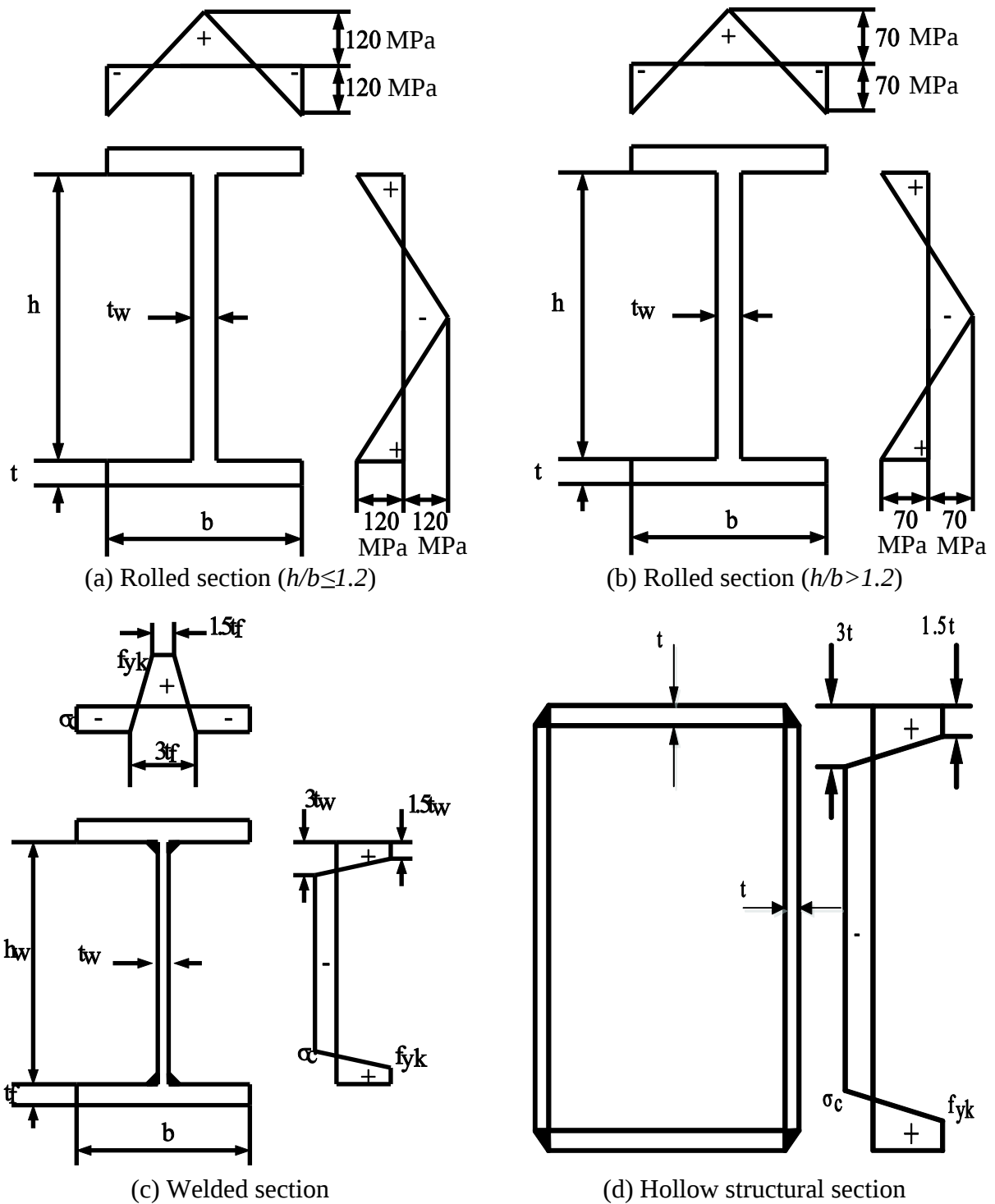


Figure 2-5: Residual Stress Distribution (National Board of Housing Building and Planning, 2003), used under fair use.

where,

f_{yk} Material yield strength (MPa)
 σ_c Compressive stress (MPa) based on the condition that the residual stresses on the cross section do not lead to a resulting normal force or moment.

t, t_f, t_w Component thickness

Imperfections: Swedish regulation for steel structures, BSK99 (National Board of Housing Building and Planning, 2003) in sub clause 8:55 sets tolerances on deviations from intended shape of the finished structures. They are as follows:

- a) The inclination from the intended direction for compression member should not exceed 0.005 times the member length.
- b) The deflection of a member in compression or flexure should not exceed 0.0015 times the member length, however, for hollow sections, the deflection should not exceed 0.002 time the member length.
- c) A buckle in the web of a member of I, U or Z section, or in the parts of the cross section of a box section, should not exceed the following values

$$e_w \leq \frac{b_w}{200} \text{ if } \frac{b_w}{t_w} \leq 80 \quad \text{Equation 2-50}$$

$$e_w \leq \frac{b_w^2}{16000t_w} \text{ if } 80 < \frac{b_w}{t_w} < 200 \quad \text{Equation 2-51}$$

$$e_w \leq \frac{b_w}{80} \text{ if } \frac{b_w}{t_w} \geq 200 \quad \text{Equation 2-52}$$

where,

b_w Depth of web (mm)
 t_w Thickness of web (mm)
 e_w Maximum value of buckle or out of flatness in the web (mm)

For a web with a longitudinal stiffener, these limits apply for each panel.

d) The deflection of a free edge in compression should not exceed the following values

$$e_f \leq \frac{L_b}{250} \text{ if } \frac{b_f}{t_f} \leq 10 \quad \text{Equation 2-53}$$

$$e_f \leq \frac{L_b b_f}{2500 t_f} \text{ if } \frac{b_f}{t_f} > 10 \quad \text{Equation 2-54}$$

where,

t_f	Thickness of edge in compression (mm)
b_f	Width of free edge (mm)
L_b	Deflected edge length (mm)
e_f	Deflection of free edge (mm)

2.4.2 Literature

White, et al. (1997): White, et al. (1997) developed a simplified moment rotation relationship over the interior pier supports for inelastic design of continuous span steel girder bridges using nonlinear finite element analysis. The finite element model included physical characteristics, such as initial geometric imperfection, residual stresses, and material plasticity that can influence the moment rotation behavior at the pier. Results from the analysis were compared against the experimental data available in the literature. Parametric studies were also carried out to study the effect of different design parameters on the moment rotation relationship. The following section discusses the residual stress distribution and imperfections included in the study and its impact on moment rotation relationship.

Residual Stresses: White, et al. (1997) used the residual stress distributions summarized in the ECCS Manual on Stability of Steel Structure (1976) in their finite elements models with some modification. Analysis used nine node Lagrangian shell elements with 2x2 Gauss integration rule to model the girder web and flanges. The residual stress distribution applied at the Gauss points is as shown in Figure 2-6. The residual stresses are in equilibrium only for perfectly flat plates. In this study, no load was applied in the first analysis step and the residual stresses were allowed to equilibrate, which caused a small change in the distribution.

Imperfections: Three type of geometric imperfection were considered in this study: out of flatness of web, tilt of compression flange, and lateral sweep of the compression flange. The

values used for the analysis were similar to the values permitted by AWS Code (1995) and are tabulated in Table 2-4.

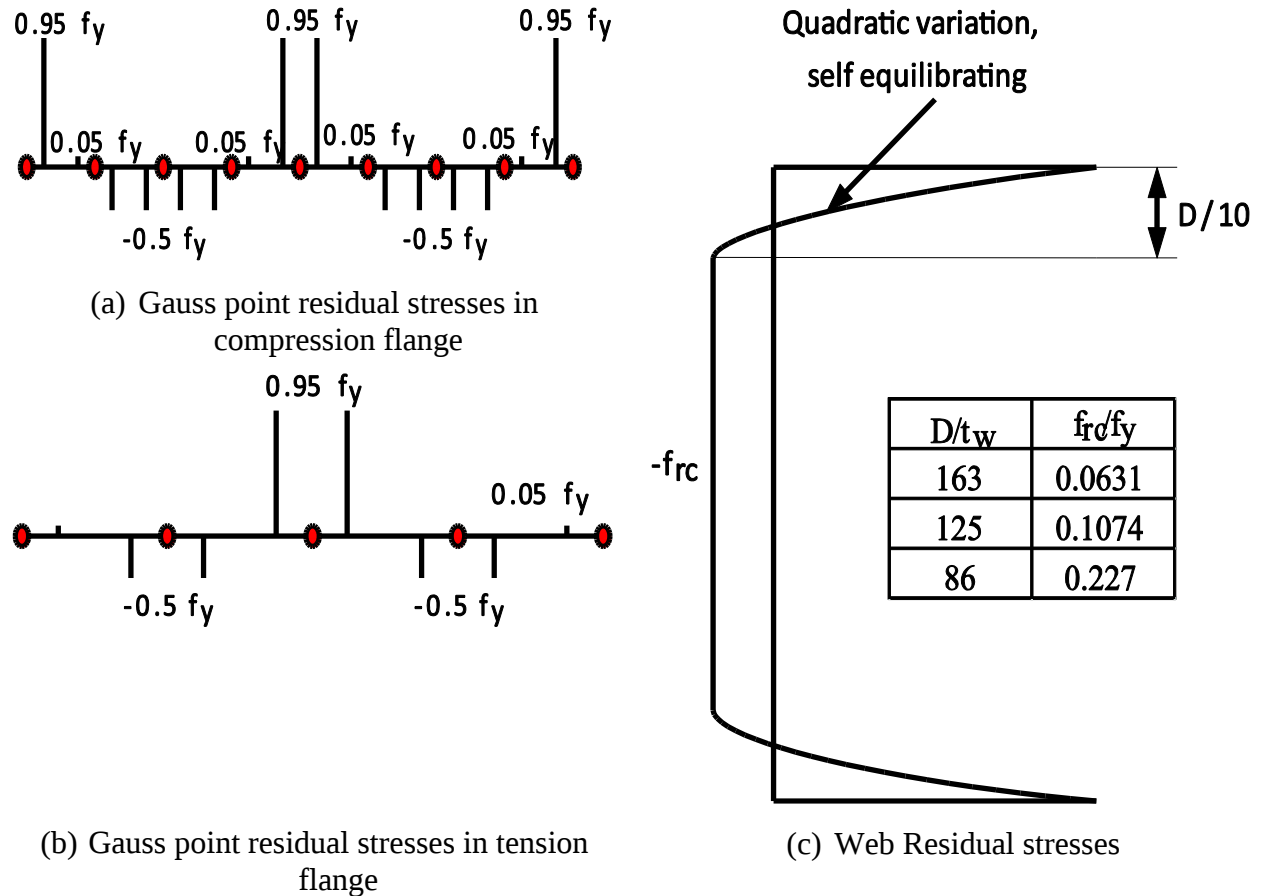


Figure 2-6: Initial Residual Stresses (Barth and White, 1998), used under fair use.

Table 2-4: Geometric Imperfections

Type of imperfection	Shape	Magnitude
Out of flatness of web		
a) If $d_o < D$	Sinusoidal	$d_o/100$
b) If $d_o \geq D$		$D/100$
Tilt of compression flanges		
a) If $b_{fc} < 0.3 d_o$	Sinusoidal	$b_{fc}/150$
b) If $b_{fc} \geq 0.3 d_o$		$d_o/500$
Lateral sweep of compression flange	Sinusoidal	$L_b/1500$

The initial geometric imperfection changed slightly during the first analysis step because of the small changes in residual stress distribution. The initial imperfections were set such that the maximum imperfections at the end of first analysis step were still slightly less than the maximum AWS tolerances. The predominant effect that residual stresses and imperfection had on the response was the early yielding and gradual softening of the pre-peak moment rotation curves. The effect on the maximum moment capacity and on the post-peak inelastic rotations was found to be small (Barth and White, 1998).

Jung (2006): Jung (2006) investigated the inelastic behavior of horizontally curved composite I-girder bridges using nonlinear finite element analysis. As a part of this research, inelastic behavior of the full-scale simply supported horizontally curved composite test bridge at the FHWA Turner-Fairbank Highway research center was evaluated. Jung (2006) used the residual stress patterns proposed in ECCS Manual on Stability of Steel Structure (ECCS, 1976). The residual stress pattern was applied at the Gauss integration points of the shell elements used to model the girder web and flanges. The residual stress values at the integration points were adjusted to obtain zero net force on the girder cross section. The test bridge model was analyzed with and without residual stress distribution. Both models yielded virtually identical load vs. displacement response with a slight difference in the nonlinear part of the curve. Therefore, Jung (2006) concluded that the residual stress pattern can be excluded from the analysis. The finite element model of the test bridge was modified by restraining the girder ends to investigate inelastic behavior in the negative moment region. Residual stresses pattern was included in this model because it is likely to have significant effect on the behavior of the bridge I-girders in the negative moment regions. Jung (2006) did not explicitly investigate the impact of residual stress pattern on inelastic behavior in the negative moment region.

Chacon, et al (2009, 2012): Chacon, et al. (2009) and Chacon, et al. (2012) investigated the influence of initial conditions on the ultimate loading capacity of steel plate girders when subjected to patch loading using experimental and analytical evaluation. Four simply supported girders were loaded to failure by applying patch load at mid-span. Before loading, initial imperfection in all girders was measured by means of a 3D coordinated measuring device. Test results were compared against the analytical results from ABAQUS (2008). Steel plate girder flanges, web and stiffeners were modeled using four node shell elements.

Residual Stresses: Chacon, et al. (2012) and Chacon, et al. (2009) modified the residual stress pattern proposed by White, et al. (1997). Figure 2-7 shows the modified residual stress pattern used in this study. Along with the modified pattern residual stress patterns proposed by ECCS manual of stability of steel structures (1976), White, et al. (1997), Swedish regulation for steel structures, BSK99 (National Board of Housing Building and Planning, 2003) were used to investigate its impact on the ultimate strength and load deflection response of plate girders.

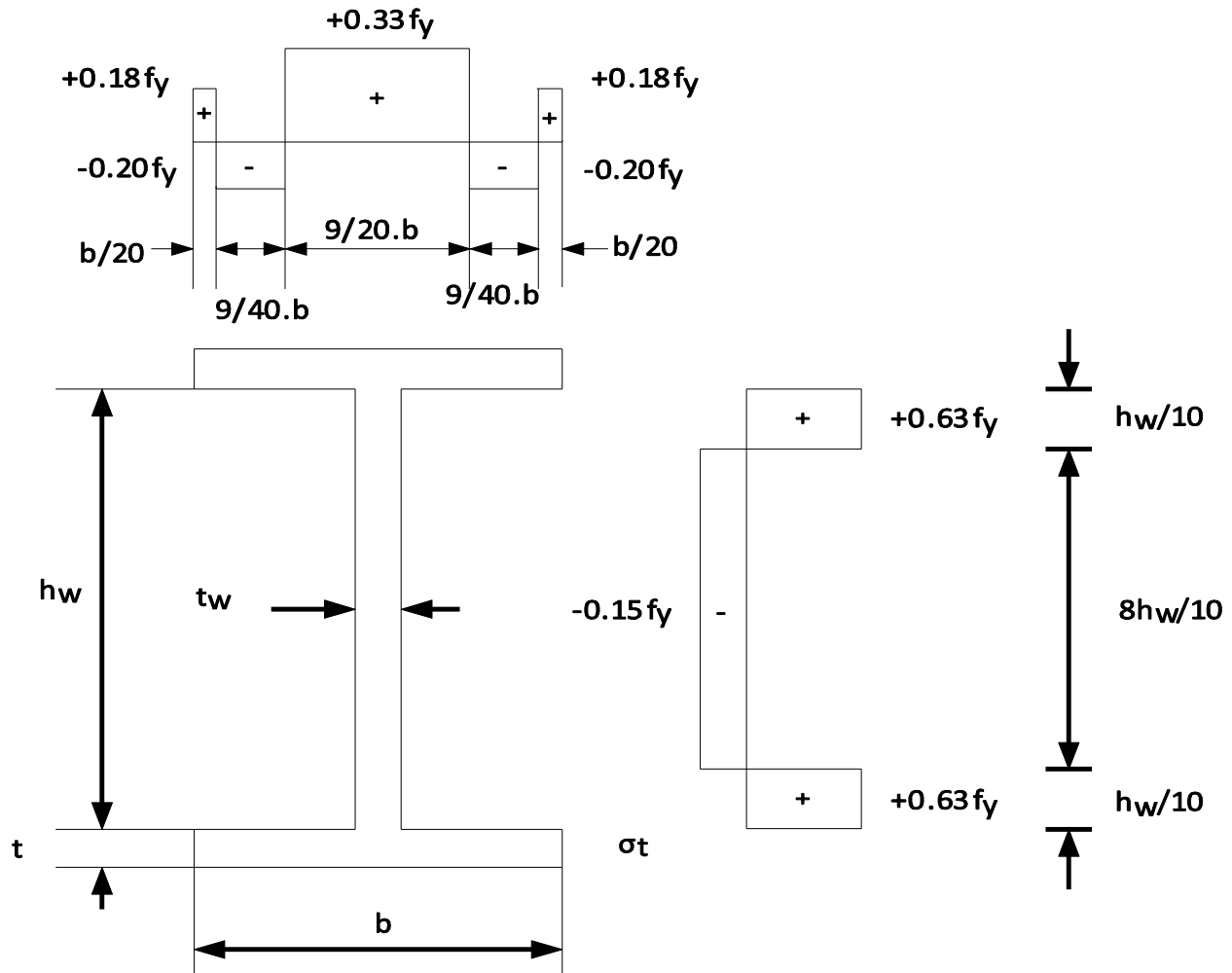


Figure 2-7: Residual stresses in steel plate girders Chacon, et al. (2012), used under fair use.

Residual stress variation across the girder cross section was applied as the initial stress condition. The residual pattern was assumed constant along the longitudinal direction regardless of the presence of transverse stiffeners. An initial step with no additional load was included after the application of residual stresses to attain equilibrium. Chacon, et al. (2012) concluded that the use

of different residual stress patterns yields similar results for ultimate strength and load deflection response.

Imperfections: Chacon, et al. (2009) used the geometric imperfection values measured during testing and the equivalent geometric imperfections specified in EN 1993-1-5 Annex C (EN 1993, 2005). Chacon, et al. (2009) used the residual stress pattern shown in Figure 2-7. These imperfections were incorporated in the finite element model in three different ways. In the first method, experimental geometric imperfection data were used to develop precise girder geometry. A dense mesh was required to model the precise girder geometries. In the second method, the geometric imperfections were applied by scaling the Eigen modes. Initially straight plates were assembled and then perturbed with Eigen value analysis. Four Eigen modes were used to represent different geometric imperfection shapes and were then scaled to the largest value of imperfection. In the third method, geometric imperfections were modeled using the equivalent geometric imperfections specified in EN 1993-1-5 Annex C (EN 1993, 2005).

A slight difference in the ultimate strength and the shape of the load displacement curve was observed for the first and third method. Use of first Eigen mode yielded results identical to experimental results, while higher Eigen modes yielded considerably higher ultimate strengths and different load deflection curves. As parametric study was carried out by varying the value of geometric imperfections ranging from 0% to 250% of the maximum allowable web out of flatness according to fabrication tolerances. Results of the parametric study showed that stocky web panels were more sensitive to the magnitude of web out of flatness than girders with slender web panels. Based on the results, Chacon, et al. (2009) concluded that use of 80% of the fabrication tolerances for maximum values for geometric imperfections would yield conservative results.

The study by Chacon, et al. (2009) and Chacon, et al. (2012) suggests that the geometric imperfections and residual stresses do not play a decisive role in the ultimate load capacity and load vs. displacement response of girders, and their exclusion from the analysis would yield slightly higher strength for girders subjected to patch loading.

Kim (2010b) and Sanchez (2011): Kim (2010b) developed a design procedure for design of frames with welded prismatic and web-tapered members. Sanchez (2011) investigated the effect of the bracing system on the performance of curved and skewed I-girder bridges during construction.

Residual Stresses: Kim (2010b) and Sanchez (2011) used the residual stress pattern (Figure 2-8).

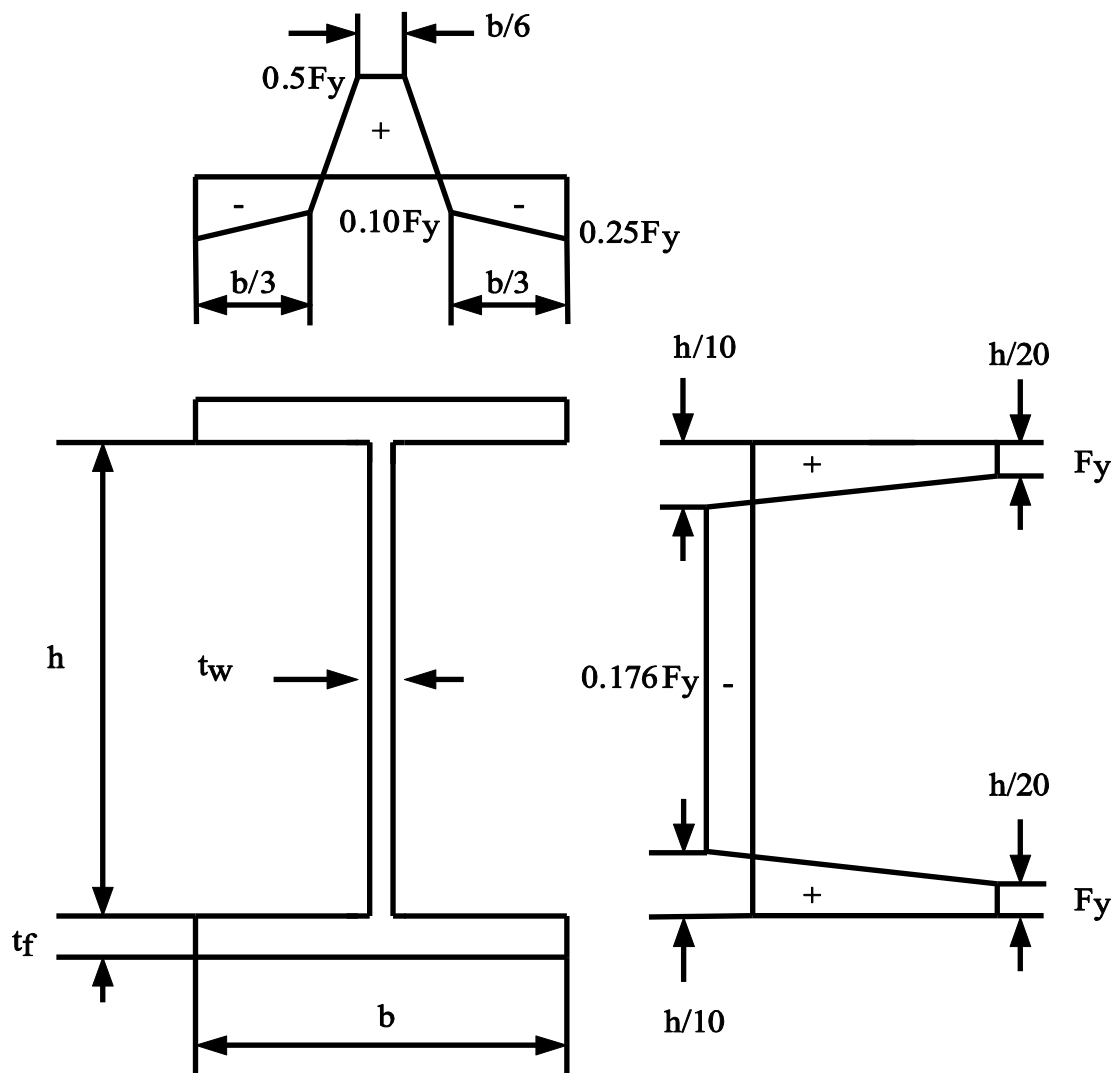


Figure 2-8: Residual stresses in steel I-sections (Kim, 2010b), used under fair use.

Imperfections: Based on the erection tolerances in the Manual of Steel Construction (AISC Committee, 2010), the maximum value of $L_b/1000$, where L_b is the unbraced length, was used for out of straightness in the parametric study by Kim (2010b). Other local forms of geometric imperfections, such as out of flatness of panels, twist of local stiffener or flange, were neglected

in this study. For the horizontally curved and skewed I-girder bridges investigated by Sanchez (2011), lateral-torsional deflection during construction was expected to be significantly larger than the initial geometric imperfections, therefore, initial geometric imperfections were not included in this research. Both Kim (2010b) and Sanchez (2011) did not explicitly investigate the effect of inclusion of residual stresses and imperfection on the response.

2.5 Live load

Nowak (1999): Nowak (1999) developed a load and resistance model for AASHTO LRFD design provisions using the reliability analysis methods as part of NCHRP Report 368. As a part of this research Nowak (1999) also developed live load models based on the available truck survey data. Live load model covers a range of forces produced by vehicles moving over the bridge. Nowak (1999) considered the effect of live load in terms of positive moment, shear force, and negative moment in case of continuous spans. Based on the weigh-in-motion data, the maximum expected load effects for various periods during a day were determined. For a single lane bridge single truck or two (or more) trucks following one behind the other in a lane produced maximum load effects. For longer spans, two vehicles per lane, one behind the other, governed. For two lane bridges, the maximum load effect was determined through simulations. Two trucks placed side-by-side, each weighing about 0.85 of the maximum 75-year truck, produced the maximum load effects. Based on this study, different models were considered and analyzed to develop AASHTO LRFD live load model. The selected model combined the HS20 truck with a uniformly distributed lane load of 640 lb. /ft. The HS-20 truck has three axles, the front axle weighs 8 kips, while the back two axles weight 32 kips each. For shorter spans, a tandem of two equal axles, each weighing 25 kips, spaced 4 ft apart with a uniformly distributed load of 640 lb. /ft, was selected. The lane load is assumed to act uniformly over a 10 ft width.

Ghosn and Moses (1998) used the two trucks side-by-side live load model in the NCHRP 406 redundancy study. The methodology proposed by Ghosn and Moses (1998) has been used widely, making the two trucks side-by-side load model the most common live load model for redundancy evaluation. Crampton, et al. (2007), Marquette Interchange Project, Hunley and Harik (2011) have used this live load model in their redundancy evaluation study.

AASHTO (2012): The AASHTO (2012) HL-93 live load model consists of a combination of the:

- Design truck or design tandem, and
- Design lane load.

The design truck is a HS-20 truck with three axles. The front axle weighs 8 kips, while the back two axles weigh 32 kips each. The first two axles are spaced 14 ft apart. The spacing between the back two axles varies between 14 ft and 30 ft to produce maximum load effect at the desired location. The design tandem consists of a pair of 25 kips axles spaced 4 ft apart. The transverse spacing of 6.0 ft is taken between the wheels. Design truck or design tandem is a notional load and it does not relate directly with escorted permit loads, illegal overloads, or short duration special permits, only the spectrum of loads and their associated load effects (AASHTO, 2012).

The design lane load consists of a load of 640 lb. /ft uniformly distributed in the longitudinal direction. Transversely, the design lane load is distributed uniformly over a 10 ft width. Dynamic effects associated with a moving truck are considered through the dynamic load allowance. The dynamic load allowance is only considered for the design truck load and the design tandem load. Effect of live load in multiple lanes is incorporated through use of multiple presence factor that accounts for the probability of multiple lanes being loaded simultaneously. Barnard, et al. (2010) used the HS-20 design truck in the redundancy evaluation of twin box girder bridges. The rear axle spacing of 14 ft was used to maximize the positive bending moment response.

2.6 Conclusion

Section 2.1 summarizes fracture critical standards in the United States. Title 23, Part 650 of the Code of Federal Regulations (United States. Dept. of Transportation, 2012) defines fracture critical members (FCM) as “A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.” This definition was found to be widely accepted across bridge owners, bridge inspectors and other government agencies, but its interpretation was found to be variable (Connor, et al., 2005). The inspection cost for fracture critical bridges was found to be two to five times higher compared to regular bridges (Connor, et al., 2005), making those systems unattractive to bridge owners and other agencies.

Section 2.2 provides overview of the redundancy evaluation methodology used by researchers and practicing engineers. The redundancy evaluation approach presented by Ghosn and Moses (1998) with two side-by-side HS-20 truck load model has been adopted widely for redundancy evaluation. However, no consideration has been given to the capacity of the load distribution path and strength of secondary member connections.

From the literature reviewed in Section 2.3, it is clear that the interaction at the steel-concrete interface with or without shear studs is a complex phenomenon and is necessary to capture a realistic structural response. The proposed interface models have been calibrated to match the researcher specific problems. Selection of modeling technique depends upon the problem in hand and forces at the interface.

Section 2.4 provides an overview of geometric imperfection and residual stress distribution used over the years. Literature shows that the geometric imperfection and the residual stresses does not play a significant role in the ultimate load capacity and load displacement response of the structure as long as its response is not governed by local behavior. Its inclusion or exclusion from the analysis depends on the anticipated failure mode. Inclusion of imperfections in a multi-member structure would result in multiple analysis runs to find the combination of imperfections in different members that cause the highest impact and would make it unreasonable for the practicing engineer. In addition, the actual imperfection data are not available for the majority of in-service bridges, and one would expect the second order effects arising from the fracture to have significant impact on the load capacity and the global response of the structure. Residual stresses and geometric imperfections will not be considered for this research.

Section 2.5 summarizes the live load models used in the redundancy analysis of fracture critical bridges. Two HS-20 trucks side-by-side placed for maximum load effect is the most common live load model used for bridge redundancy evaluation. The model is based on the study by Nowak (1999) which was carried out for the development of AASHTO LRFD Bridge Design provisions. This study showed that the two HS-20 trucks side-by-side live load produced maximum load effect typically on two lane bridges, and therefore the current HL-93 load model was developed to encompass all other live load combinations. The HL-93 live load model will be used for the current research.

3 Modeling methodology and verification

3.1 Introduction

With the advancement of computational resources over the last two decades, three dimensional finite element modeling is becoming a usable choice for the analyses of steel bridges. Existing modeling techniques for simulating the structural behavior of steel bridges vary in their level of complexity and accuracy. Therefore, the first important step in finite element modeling is the selection of appropriate modeling techniques and elements. The objective of this chapter is to discuss and verify the modeling methodology that will be used in the current research.

The first half of the chapter provides a discussion about modeling methodologies. The chapter begins with a discussion on element selection to model different bridge components. This is followed by an overview of concrete and steel plasticity material models and their implementation in ABAQUS (2013). Then methodologies implemented to model varying levels of composite action and to simulate fracture for static and dynamic analyses are discussed. This section ends with a description of the method used to simulate the live load on bridge decks. In the second half of the chapter, test results from a corner supported two way slab tested by Jofriet and McNeice (1971) and two full-scale composite plate girders tested by Mans et al. (2001) are used to verify some of the modeling methodologies discussed in first half of the chapter.

3.2 Modeling methodology

3.2.1 Modeling of bridge components

Finite element models are constructed using the ABAQUS CAE pre-processor. The modeling hierarchy in ABAQUS CAE allows individual components to be constructed separately and assembled together with appropriate connections and boundary conditions. The reinforced concrete slab is modeled using general-purpose four node shell elements with reduced integration and six degrees of freedom per node (S4R). The S4R is a general purpose conventional shell element which uses a thick shell formulation (Mindlin shell formulation) as the shell thickness increases and uses a discrete Kirchhoff shell formulation as shell thickness decreases (ABAQUS, 2013). Longitudinal and transverse reinforcement is modeled using smeared rebar layers at the correct position through the slab thickness. Interface effects such as

bond slip and dowel action are modeled indirectly through tension stiffening model which is used to model the post cracking response of concrete in tension. The main girders, floor beams and stringers are modeled using S4R shell element. The web is modeled using shell elements at its mid-surface while the flanges are modeled at the flange to web connection with an offset equal to half the flange thickness to get the correct sectional properties. Transverse stiffeners and longitudinal stiffeners are also modeled using S4R shell elements. Cross bracing components, lower lateral bracing and guardrail are modeled using linear beam element (B31) at their centroids. B31 uses Timoshenko beam formulation with lumped mass matrix.

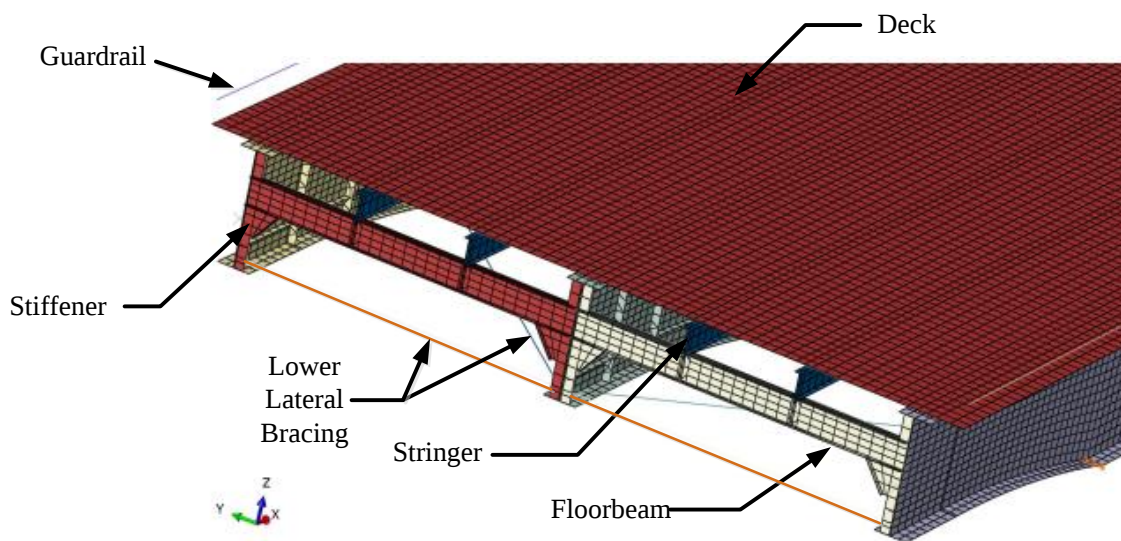


Figure 3-1: Bridge component model.

3.2.2 Material plasticity

ABAQUS plasticity models uses “ incremental” theories in which the mechanical strain rate is decomposed into an elastic part and a plastic part (ABAQUS, 2013). Incremental plasticity models are formulated in terms of three parts:

- A yield surface defines the elastic limit of the material in a multi-axial state of stress

$$F = F(\sigma_{ij}, \varepsilon_{ij}^p, \dots) = 0 \quad \text{Equation 3-1}$$

where, $F(\sigma_{ij}, \varepsilon_{ij}^p, \dots)$ is the yield surface.

- b. A flow rule defines the plastic potential function and direction of incremental plastic strain vector.

$$d\varepsilon_{ij}^p = d\lambda \frac{\partial g}{\partial \sigma_{ij}} \quad \text{Equation 3-2}$$

where, $g(\sigma_{ij}, \varepsilon_{ij}^p, \dots)$ is the plastic potential function, $d\lambda$ is the scalar measuring the amount of the plastic flow rate and $d\varepsilon_{ij}^p$ is the plastic strain increment vector.

- c. A hardening law defines evolution of subsequent yield surfaces or loading surface and/or flow rule changes. Hardening variables are included in the definition of yield surface and plastic potential function.

To define these three parts, the user needs to input a uniaxial true stress-logarithmic strain curve along with other parameters that will be discussed in detail in the subsequent sections. The uniaxial engineering stress-strain curve can be converted to true stress-logarithmic strain curve using following expressions

$$\sigma_{true} = \sigma_{engineering} (1 + \varepsilon_{engineering}) \quad \text{Equation 3-3}$$

$$\varepsilon_{logarithmic} = \ln(1 + \varepsilon_{engineering}) \quad \text{Equation 3-4}$$

3.2.2.1 Concrete

ABAQUS (2013) offers three different constitutive models to model concrete plasticity at low confining pressures:

- An smeared crack model in *ABAQUS/Standard* intended for concrete subjected monotonic straining.
- A brittle cracking model in *ABAQUS/Explicit* intended for concrete behavior dominated by tensile cracking and linear compressive behavior.
- A damaged plasticity model in both *ABAQUS/Standard* and *ABAQUS/Explicit* intended for concrete subjected to monotonic, cyclic, and/or dynamic loading.

Concrete plasticity in bridge slabs would involve both tensile and compressive response and therefore, the smeared crack model and the damaged plasticity model will be evaluated as part of this study. These constitutive models require a number of input parameters to define the three-

dimensional concrete plasticity. These parameters can be obtained from extensive testing, which is not feasible in most cases. In the absence of test data, typical values based on the literature can be used. This section discusses the compressive and tensile behavior of concrete followed by the fundamentals associated with the concrete plasticity models and the typical ranges for the parameters required to define three-dimensional concrete plasticity.

3.2.2.1.1 Compressive behavior

The ABAQUS (2013) concrete plasticity definition needs uniaxial stress-strain data in compression as one of the inputs to define concrete plasticity in compression. This section discusses the uniaxial-stress curves considered in this research.

Hognestad (1951): Hognestad (1951) proposed a uniaxial stress-strain relationship for concrete in compression. The σ - ε curve was assumed to be parabolic (Equation 3-5) up to compressive strength (f'_c) and then reduces linearly to $0.85f'_c$ at ultimate strain of 3.8%.

$$f_c = f'_c \left[2 \frac{\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o} \right)^2 \right] \quad \text{Equation 3-5}$$

$$\varepsilon_o = \frac{2 f'_c}{E_t} \quad \text{Equation 3-6}$$

$$E_t = 1.8 \times 10^6 + 460 f'_c \quad \text{Equation 3-7}$$

where,

f'_c	Compressive strength (psi)
E_t	Initial tangent modulus (psi)
f_c	Stress (psi)
ε_c	Strain

EN 1992-1-1 (2004): EN 1992-1-1 (2004) uses a parabolic stress-strain curve (Equation 3-8) to define uniaxial compressive behavior of concrete up to ultimate strain ε_{cu1} .

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta} \quad \text{Equation 3-8}$$

$$\eta = \frac{\varepsilon_c}{\varepsilon_{c1}} \quad \text{Equation 3-9}$$

$$\varepsilon_{c1} = \min\left(\frac{0.7 f_{cm}^{0.31}}{1000}, \frac{2.8}{1000}\right) \quad \text{Equation 3-10}$$

$$k = 1.05 E_{cm} \frac{|\varepsilon_{c1}|}{f_{cm}} \quad \text{Equation 3-11}$$

$$E_{cm} = 22 \left[\frac{f_{cm}}{10} \right]^{0.3} \quad \text{Equation 3-12}$$

$$\varepsilon_{cu1} = 2.8 + 27 \left[\frac{(98 - f_{cm})}{100} \right]^4 \text{ if } f_{ck} \geq 50 \quad \text{Equation 3-13}$$

$$\varepsilon_{cu1} = \frac{3.5}{1000} \text{ if } f_{ck} < 50$$

where,

E_{cm}	Concrete secant modulus (GPa)
f_{cm}	Mean concrete cylinder compressive strength (MPa)
ε_{cu1}	Nominal ultimate strain
ε_{c1}	Strain at peak stress
σ_c	Stress (MPa)
ε_c	Strain

CEB-FIP model (De Schutter, 1999): CEB-FIP Model Code 1990 stress-strain curve (De Schutter, 1999) is the earlier version of the EN 1992-1-1 (2004) model. Barth and Wu (2006) rearranged the original expression to get a simplified expression (Equation 3-14).

$$f_c = \frac{0.85 f'_c (a - 206,000 \varepsilon_c) \varepsilon_c}{1 + b \varepsilon_c} \quad \text{Equation 3-14}$$

$$a = 6193.6 (0.85 f'_c + 1.015)^{-0.953} \quad \text{Equation 3-15}$$

$$b = 8074.1 (0.85 f'_c + 1.450)^{-1.085} - 850 \quad \text{Equation 3-16}$$

where,

E_{cm}	Concrete secant modulus (GPa)
f'_c	Compressive strength (MPa)
ϵ_{cu1}	Nominal ultimate strain
ϵ_{c1}	Strain at peak stress
f_c	Stress (psi)
ϵ_c	Strain

Figure 3-2, shows the comparison between the uniaxial stress-strain models for a 5500 psi concrete in compression.

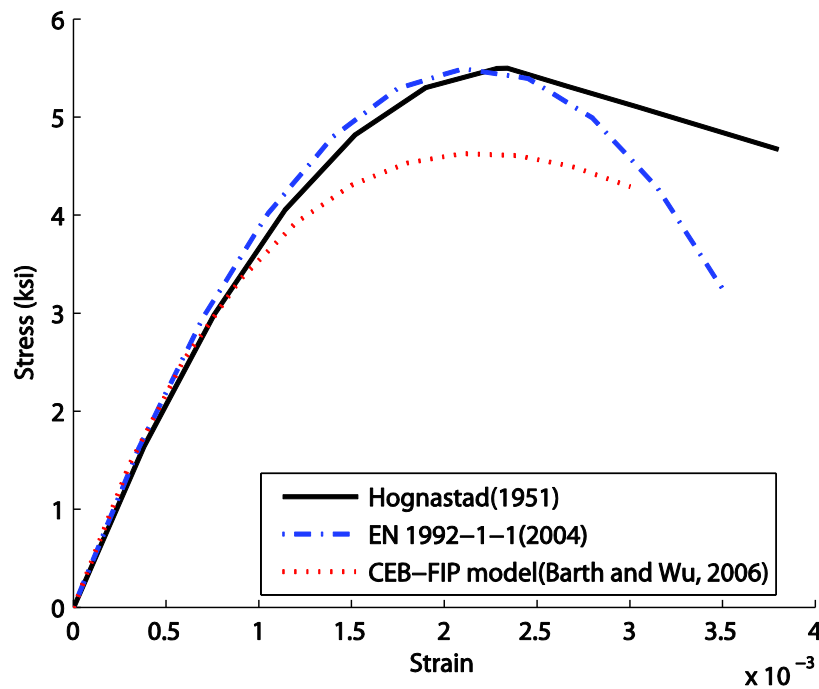


Figure 3-2: Concrete uniaxial stress-strain curve.

The Hognestad (1951) and EN 1992-1-1 (2004) stress-strain curves are similar up to the concrete compressive strength (f'_c). Beyond the post peak stress, the EN 1992-1-1 (2004) model shows a sharper stress drop when compared to the Hognestad (1951) model. The CEB-FIP Model Code

1990 (Barth and Wu, 2006) shows reductions in both peak stress and the stress-strain response because of the 15% reduction on the concrete compressive strength (f'_c).

3.2.2.1.2 Tensile behavior

Tensile strength of concrete is about 8-10% of its compressive strength. Usually this strength is ignored during design. Reinforced concrete members show degradation in tensile stress value as it exceeds the tensile strength of concrete.

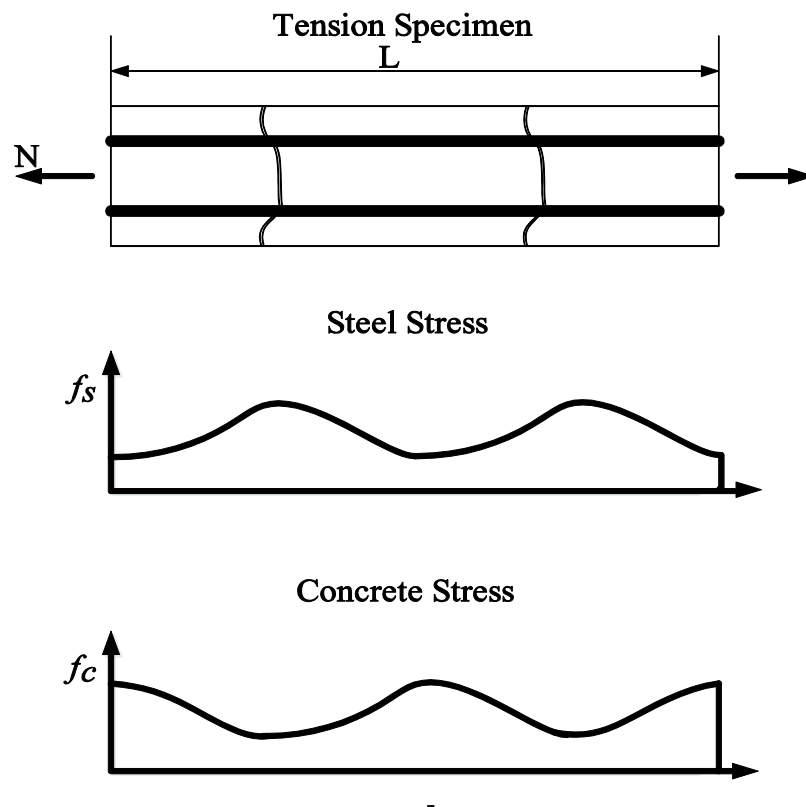


Figure 3-3: Distribution of steel stress and concrete stress in tension specimen (Chung, 2003), used under fair use.

Consider a reinforced concrete specimen (Figure 3-3) subjected to tension so that the large cracks form in the specimen. At the crack location, a relative movement (slip) between the reinforcement and the concrete takes place. However, the intact concrete between the two adjacent cracks hangs on to the reinforcement and contributes to the overall stiffness of the specimen. This stiffening effect is called as tension stiffening and can be quite significant in certain components. In order to realistically model the behavior of cracked members, many researchers have included tension stiffening as the descending branch of the concrete stress-

strain curve in tension beyond maximum tensile strength of concrete as shown in Figure 3-4. Tension stiffening can also be included in terms of the fracture energy and the crack width or as an increase in reinforcing steel stiffness and stress by increasing the elastic modulus of the reinforcing steel. In this research, tension stiffening is included as the descending branch of the concrete stress-strain curve in tension. The following section discusses some of the tension stiffening relationships proposed by researchers.

Vecchio and Collins (1986): Vecchio and Collins (1986) proposed a stress-strain relationship based on the testing of thirty reinforced concrete panels subjected to uniform biaxial stresses. Average principal tensile stress (f_{c1}) vs. strain (ε_1) relationship is linear prior to cracking (Equation 3-17) and then decreases after cracking following Equation 3-18.

$$f_{c1} = E_c \cdot \varepsilon_1 \quad \text{Equation 3-17}$$

$$f_{c1} = \frac{f_{cr}}{1 + \sqrt{200 \cdot \varepsilon_1}} \quad \text{Equation 3-18}$$

where, E_c is the modulus of elasticity of concrete (ksi/MPa), f_{cr} is the concrete cracking stress (ksi/MPa).

Collins and Mitchell (1991): Collins and Mitchell (1991) also proposed a model to capture the tensile behavior of concrete before (Equation 3-19) and after cracking (Equation 3-20). This model accounts for bond characteristics and loading time period.

For $\varepsilon_{cf} \leq \varepsilon_{cr}$

$$f_c = E_c \cdot \varepsilon_{cf} \quad \text{Equation 3-19}$$

For $\varepsilon_{cf} > \varepsilon_{cr}$

$$f_{c1} = \frac{\alpha_1 \alpha_2 f_{cr}}{1 + \sqrt{500 \cdot \varepsilon_{cf}}} \quad \text{Equation 3-20}$$

where,

ε_{cf}	Average principal tensile strain
f_{cr}	Concrete cracking stress (ksi/MPa)
ε_{cr}	Concrete cracking strain
α_1	Factor accounting for bond characteristics of reinforcement = 1.0 for deformed bars = 0.7 for plain bars, wires and bonded strands = 0 for unbounded reinforcement
α_2	Factor accounting for loading time period = 1.0 for short-term monotonic loads = 0.7 for sustained and/or repeated loads

Abrishami and Mitchell (1996): Abrishami and Mitchell (1996) studied the influence of splitting cracks on the tension stiffening. This was done through testing of ten tension specimens similar to the specimens shown previously in Figure 3-3.

For $\varepsilon_{cf} \leq \varepsilon_{cr}$

$$f_c = E_c \cdot \varepsilon_{cf} \quad \text{Equation 3-21}$$

For $\varepsilon_{cf} > \varepsilon_{cr}$

$$f_{c1} = \frac{\alpha_1 \alpha_2 \alpha_3 f_{cr}}{1 + \sqrt{500 \cdot \varepsilon_{cf}}} \quad \text{Equation 3-22}$$

where,

α_3	Factor accounting for splitting cracks = 1.0 for $\frac{c}{d_b} > 2.5$ = $0.8 \frac{c}{d_b} - 1$ for $1.25 \leq \frac{c}{d_b} \leq 2.5$ = 0 for $\frac{c}{d_b} < 1.25$
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c Concrete cover
 d_b Bar diameter

Rest of the variables are same as defined by Collins and Mitchel (1991).

Rex and Easterling (2000): Rex and Easterling (2000) used a multi-linear representation of the tension stiffening model proposed by Collins and Mitchel (1991) with tensile stress reaching zero at a strain of 0.1.

Table 3-1: Concrete tension stiffening stress-strain response by Rex and Easterling (2000).

Strain	Stress
f_{cr}/E_c	f_{cr}
0.001	$0.6f_{cr}$
0.002	$0.4f_{cr}$
0.008	$0.3f_{cr}$
0.1	0

where, E_c is the modulus of elasticity of concrete (ksi/MPa), and f_{cr} is the concrete cracking stress (ksi/MPa).

Hsu and Zhang (1996): Hsu and Zhang (1996) proposed tensile stress-strain relationship for concrete based on the test results of fifty-five reinforced concrete panels subjected to uniaxial and biaxial loading at the University of Houston. This model assumes a linear stress-strain relationship up to cracking (Equation 3-23). The post cracking stress reduces using the power law defined by Equation 3-24.

For $\varepsilon_{cf} \leq \varepsilon_{cr}$

$$f_{c1} = E_c \cdot \varepsilon_{cf} \quad \text{Equation 3-23}$$

For $\varepsilon_{cf} > \varepsilon_{cr}$

$$f_{c1} = f_{cr} \left(\frac{\varepsilon_{cr}}{\varepsilon_{cf}} \right)^{0.4} \quad \text{Equation 3-24}$$

where,

ε_{cf}	Strain
f_{cr}	Concrete cracking stress(MPa)
E_c	Modulus of elasticity of concrete(MPa)
ε_{cr}	Concrete cracking strain

Izumo et al. (1992): The Izumo et al. (1992) tension-stiffening model assumes a linear stress-strain relationship up to cracking (Equation 3-25). The post cracking stress remains equal to the cracking stress value up to a strain equal to twice the cracking strain (Equation 3-26). Then stress reduces using the power law defined by Equation 3-27.

For $\varepsilon_{cf} \leq \varepsilon_{cr}$

$$f_{c1} = E_c \cdot \varepsilon_{cf} \quad \text{Equation 3-25}$$

For $\varepsilon_{cr} < \varepsilon_{cf} < 2\varepsilon_{cr}$

$$f_{c1} = f_{cr} \quad \text{Equation 3-26}$$

For $\varepsilon_{cf} > 2\varepsilon_{cr}$

$$f_{c1} = f_{cr} \left(\frac{2\varepsilon_{cr}}{\varepsilon_{cf}} \right)^{0.4} \quad \text{Equation 3-27}$$

where,

f_{c1}	Average principal tensile stress (ksi/MPa)
ε_{cf}	Average principal tensile strain
f_{cr}	Concrete cracking stress (ksi/MPa)
E_c	Modulus of elasticity of concrete (ksi/MPa)
ε_{cr}	Concrete cracking strain

Along with these different tension stiffening models, ABAQUS (2013) suggests that assuming reduction of stress linearly to zero at a total strain about 10 times the strain at failure will be a reasonable starting point. Figure 3-4 shows the comparison between different tension stiffening

models for 5500 psi concrete. For the purpose of comparison modifications factors α_1 , α_2 , α_3 used by Collins and Mitchell (1991), and Abrishami and Mitchell (1996) are taken as 1.0.

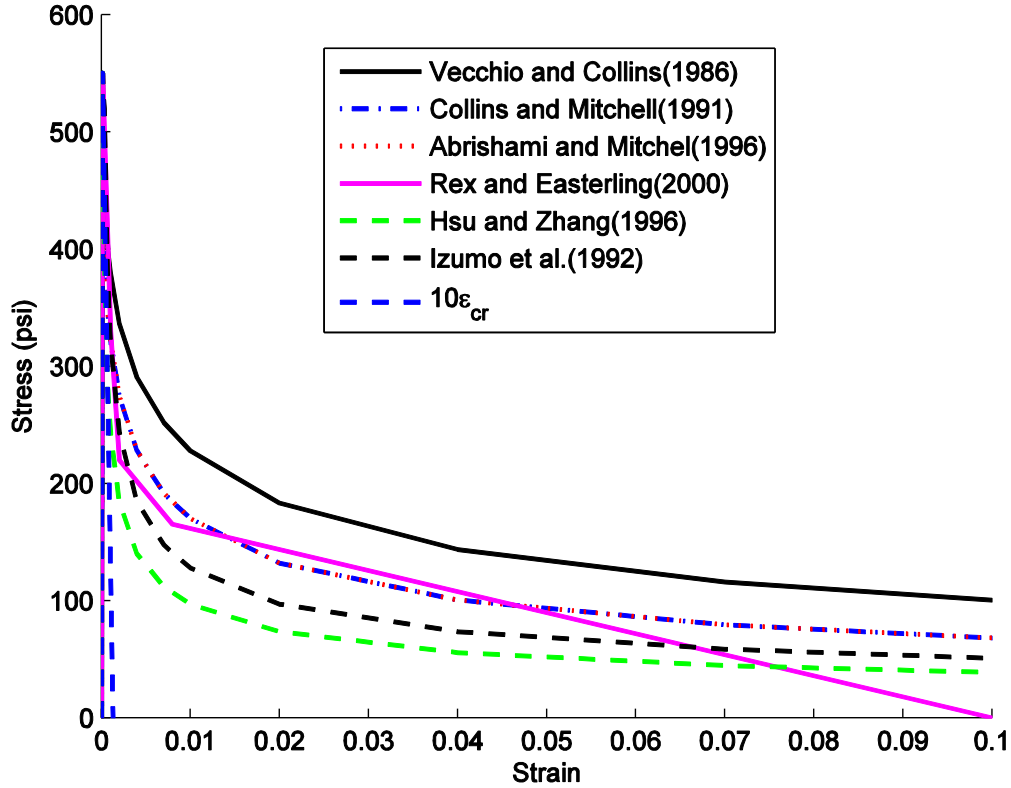


Figure 3-4: Comparison of tension stiffening models.

3.2.2.1.3 Concrete smeared crack model

The concrete smeared crack model is intended for concrete subjected to monotonic loading under low confining pressures (less than four to five times the largest compressive stress that can be carried by concrete in uniaxial compression). This model uses linear elastic model to define the elastic properties.

$$\bar{\sigma} = D^{el} : \bar{\varepsilon} \quad \text{Equation 3-28}$$

where, $\bar{\sigma}$ is the stress tensor, $\bar{\varepsilon}$ is the strain tension and D^{el} is the initial elastic stiffness tensor

Compression response: Concrete response in compression is modeled by an elastic-plastic theory with associated flow and isotropic hardening. It uses incremental plasticity theory in which total mechanical strain rate ($\dot{\epsilon}$) is decomposed into elastic strain rate ($\dot{\epsilon}^{el}$) and plastic strain rate ($\dot{\epsilon}_c^{pl}$) associated with the compression yield surface.

Yield surface: The compression yield surface (f_c) is defined using Equation 3-29

$$f_c = q - \sqrt{3}a_o p - \sqrt{3}\tau_c \quad \text{Equation 3-29}$$

$$p = -\frac{1}{3} \text{trace}(\bar{\sigma}) \quad \text{Equation 3-30}$$

$$q = \sqrt{\frac{3}{2} \bar{S} : \bar{S}} \quad \text{Equation 3-31}$$

$$a_o = \sqrt{3} \frac{1 - r_{bc}^{\sigma}}{1 - 2r_{bc}^{\sigma}} \quad \text{Equation 3-32}$$

where,

a_o	Constant needed to define the shape of surface
$\bar{S}$	Deviatoric stress tensor ($\bar{S} = \bar{\sigma} + p\bar{I}$)
p	Effective pressure stress/ Hydrostatic stress
q	Mises equivalent deviatoric stress
τ_c	Hardening parameter
r_{bc}^{σ}	Ratio of ultimate biaxial compression to uniaxial compression

Flow rule: The associated flow rule is defined as below:

If $f_c = 0$ and $d\lambda_c > 0$,

$$\dot{\epsilon}_c^{pl} = d\lambda_c \left(1 + c_o \left(\frac{p}{\sigma_c} \right)^2 \right) \frac{\partial f_c}{\partial \bar{\sigma}} \quad \text{Equation 3-33}$$

Otherwise,

$$\dot{\epsilon}_c^{pl} = 0 \quad \text{Equation 3-34}$$

Where,

f_c	Compression yield surface
$d\lambda_c$	Scalar measuring the amount of the plastic flow rate in compression
$\dot{\varepsilon}_c^p$	Plastic strain increment vector in compression

And,

$$c_o = 9 \frac{r_{bc}^\varepsilon (\sqrt{3} - a_o) + \left(a_o - \frac{\sqrt{3}}{2}\right)}{r_{bc}^\varepsilon (a_o - \sqrt{3}) + (r_{bc}^\sigma)^2 (2\sqrt{3} - 4a_o)} \quad \text{Equation 3-35}$$

where,

r_{bc}^ε	Ratio of the magnitude of a principal component of plastic strain at ultimate stress in biaxial compression to the plastic strain at ultimate stress in uniaxial compression
$d\lambda_c$	Scalar measuring the amount of the plastic flow rate at yield
c_o	Constant

In uniaxial compression, $p = \frac{1}{3}\sigma_c$, $q = \sigma_c$ and $S_{11} = -\frac{2}{3}\sigma_c$ so the associate flow rule definition becomes

$$\dot{\varepsilon}_c^{pl} = d\lambda_c \left(1 + \frac{c_o}{9}\right) \left(\frac{a_o}{\sqrt{3}} - 1\right) \quad \text{Equation 3-36}$$

Integrating this equation gives total plastic strain ε_c^{pl} ,

$$\varepsilon_c^{pl} = \lambda_c \left(1 + \frac{c_o}{9}\right) \left(\frac{a_o}{\sqrt{3}} - 1\right) \quad \text{Equation 3-37}$$

Knowing the total plastic strain ε_c^{pl} , a_o and c_o , λ_c value can be computed.

Hardening law: The hardening parameter τ_c can be calculated by evaluating the compression yield surface definition (f_c) at uniaxial compression.

$$\tau_c = \left(\frac{1}{\sqrt{3}} - \frac{a_o}{3} \right) \sigma_c \quad \text{Equation 3-38}$$

Tension response: To model the concrete response in tension, the total mechanical strain rate ($\dot{\epsilon}^{el}$) is decomposed into an elastic strain rate ($\dot{\epsilon}_d^{el}$) and a plastic strain rate ($\dot{\epsilon}_t^{pl}$) associated with the crack detection surface.

Yield surface: The tension yield surface or crack detection surface (f_t) is defined as,

$$f_t = \hat{q} - \left(3 - b_o \frac{\sigma_t}{\sigma_t^u} \right) \hat{p} - \left(2 - \frac{b_o}{3} \frac{\sigma_t}{\sigma_t^u} \right) \sigma_t = 0 \quad \text{Equation 3-39}$$

$$b_o = 3 \frac{1 + (2 - f) r_t^\sigma - \sqrt{1 + (f r_t^\sigma)^2 + f r_t^\sigma}}{1 + r_t^\sigma (1 - f)} \quad \text{Equation 3-40}$$

where,

σ_t^u	Failure stress in uniaxial tension
b_o	Constant
F	Ratio of the tensile principal stress at cracking, in plane stress, when the other principal stress is at the ultimate compressive value, to the tensile cracking stress under uniaxial tension
r_t^σ	Ratio of the uniaxial tensile stress at failure to the ultimate uniaxial compressive stress
$\sigma_t(\lambda_t)$	Hardening parameter

Concrete response under tension uses the crack detection surface to determine when the cracks are formed but it does not track individual cracks. Rather cracking introduces anisotropy affecting the stress and the material stiffness during computation. Once the crack is detected, its orientation is stored for subsequent calculations restricting subsequent cracks orthogonal to stored crack orientation. Cracks remain for the rest of the calculation but may open or close in subsequent iterations.

Flow rule: The associated flow rule for crack detection is defined as below:

If $f_t = 0$ and $d\lambda_t > 0$,

$$\dot{\epsilon}_t^{pl} = d\lambda_t \frac{\partial f_t}{\partial \sigma} \quad \text{Equation 3-41}$$

Otherwise,

$$\dot{\epsilon}_t^{pl} = 0 \quad \text{Equation 3-42}$$

where,

f_t	Tension yield surface or crack detection surface
$d\lambda_t$	Scalar measuring the amount of the plastic flow rate in tension
$\dot{\epsilon}_t^p$	Plastic strain increment vector in tension

Hardening law: Hardening is defined by specifying the magnitude of stress σ_t in a uniaxial tension test as a function of inelastic strain. In case of uniaxial tension, $S_{11} = \frac{2}{3}\sigma_t$ and $q = \sigma_t$, $\dot{\epsilon}_t^{pl}$ reduces to

$$(\dot{\epsilon}_t^{pl})_{11} = d\lambda_t \left(2 - \frac{b_o}{3} \frac{\sigma_t}{\sigma_t^u} \right) \quad \text{Equation 3-43}$$

Upon integrating the above equation we get λ_t from $(\dot{\epsilon}_t^{pl})_{11}$ and therefore the relationship between σ_t and λ_t is obtained from the tension stiffening input data. Tension stiffening not only allows for defining the strain softening behavior post cracking but also allows for the concrete reinforcement interactions such as bond slip and dowel action.

Concrete cracking results in shear stiffness reduction, which is defined by a reduction in shear modulus. The shear modulus is reduced linearly as a function of crack opening strain across the crack (Equation 3-44). The model also assumes a reduced shear modulus where the cracks have closed in the subsequent steps (Equation 3-45).

$$\rho = \left(1 - \frac{\varepsilon}{\varepsilon^{\max}}\right) \text{ for } \varepsilon < \varepsilon^{\max} \quad \text{Equation 3-44}$$

$$\rho = 0 \text{ for } \varepsilon \geq \varepsilon^{\max}$$

$$\rho = \rho^{close} \text{ for } \varepsilon < 0 \quad \text{Equation 3-45}$$

This model simplifies the behavior of concrete in compression to improve the computational efficiency. However, the associated flow assumption results in over predicting inelastic volumetric strain. The yield surface does not match accurately with the tri-axial tension and compression data. In addition to that, the model does not capture the inelastic response of concrete when subjected to high-pressure stresses (hydrostatic stress).

3.2.2.1.4 Concrete damaged plasticity model

The damaged plasticity model is intended for concrete subjected to monotonic, cyclic, and/or dynamic loading. This model allows for different yield strength in tension and compression, tension softening and compression hardening, elastic stiffness degradation during cyclic loading, and strain rate hardening. The damaged plasticity model in ABAQUS (2013) is based on the model proposed by Lubliner et al. (1989) and Lee and Fenves (1998) and uses isotropic linear elastic material model to define the elastic properties.

$$\bar{\sigma} = D^{el} : \bar{\varepsilon} \quad \text{Equation 3-46}$$

where, $\bar{\sigma}$ is the stress tensor, $\bar{\varepsilon}$ is the strain tensor and D^{el} is the initial elastic stiffness tensor

In incremental plasticity the total strain rate ($\dot{\varepsilon}$) is decomposed into elastic part ($\dot{\varepsilon}^{el}$) and plastic part ($\dot{\varepsilon}^{pl}$) of the strain rate. The linear elasticity with the scalar degradation damage is given by

$$\bar{\sigma} = (1 - d) D_o^{el} : (\varepsilon - \varepsilon^{pl}) \quad \text{Equation 3-47}$$

$$\bar{\sigma} = (1 - d) \bar{\sigma} \quad \text{Equation 3-48}$$

where,

$\bar{\sigma}$	Effective stress tensor
D_o^{el}	Initial(undamaged) elastic stiffness
ε	Total strain

ε^{pl}	Plastic strain
d	Scalar degradation damage variable
$\bar{\sigma}$	Cauchy stress tensor

Yield surface: The yield function $F(\bar{\sigma}, \tilde{\varepsilon}^{pl})$ represents a surface in the effective stress space to determine the material failure or damage.

$$F(\bar{\sigma}, \tilde{\varepsilon}^{pl}) = \frac{1}{1-\alpha} (\bar{q} - 3\alpha\bar{p} + \beta(\tilde{\varepsilon}^{pl}) \langle \bar{\sigma}_{\max} \rangle - \gamma \langle -\bar{\sigma}_{\max} \rangle) - \bar{\sigma}_c(\tilde{\varepsilon}_c^{pl}) \leq 0 \quad \text{Equation 3-49}$$

where,

$$\bar{p} = -\frac{1}{3} \bar{\sigma} : \bar{I} \quad \text{Equation 3-50}$$

$$\bar{q} = \sqrt{\frac{3}{2} \bar{S} : \bar{S}} \quad \text{Equation 3-51}$$

$$\bar{S} = \bar{p}\bar{I} + \bar{\sigma} \quad \text{Equation 3-52}$$

$$\beta(\tilde{\varepsilon}^{pl}) = \frac{\bar{\sigma}_c(\tilde{\varepsilon}_c^{pl})}{\bar{\sigma}_t(\tilde{\varepsilon}_t^{pl})} (1-\alpha) - (1+\alpha) \quad \text{Equation 3-53}$$

where,

$\bar{\sigma}$	Effective stress tensor
$\langle \bar{\sigma}_{\max} \rangle$	Maximum eigenvalue of $\bar{\sigma}$
α, β	Dimensionless material constants
$\bar{\sigma}_t(\tilde{\varepsilon}_t^{pl})$	Effective tensile cohesion stress
$\bar{\sigma}_c(\tilde{\varepsilon}_c^{pl})$	Effective compression cohesion stress

The parameters in the yield function are evaluated so that the yield function accurately represents the experimental data. The coefficient α is determined from the initial equi-biaxial stress (σ_{bo}) and uniaxial compressive yield stress (σ_{co}), as

$$\alpha = \frac{\sigma_{bo} - \sigma_{co}}{2\sigma_{bo} - \sigma_{co}} \quad \text{Equation 3-54}$$

The coefficient γ is determined by comparing the yield conditions along the tensile and compressive meridians under state of triaxial compression.

$$K_c = \frac{\bar{\bar{q}}_{Tensile_Meridian}}{\bar{\bar{q}}_{Compressive_Meridian}} \quad \text{Equation 3-55}$$

$$\gamma = \frac{3(1-K_c)}{2K_c-1} \quad \text{Equation 3-56}$$

Flow rule: The damaged plasticity model assumes a non-associated flow,

$$\dot{\epsilon}^{pl} = \dot{\lambda} \frac{\partial G(\bar{\bar{\sigma}})}{\partial \bar{\bar{\sigma}}} \quad \text{Equation 3-57}$$

where, G is the plastic potential function, $\dot{\lambda}$ is the scalar measuring the amount of the plastic flow rate and $\dot{\epsilon}^p$ is the plastic strain increment vector.

It uses a Drucker-Prager hyperbolic function as the plastic potential function,

$$G = \sqrt{(\sigma_{t0} \tan \psi)^2 + \bar{\bar{q}}^2} - \bar{\bar{p}} \tan \psi \quad \text{Equation 3-58}$$

where, ψ is the dilation angle measured in p-q plane, σ_{t0} is the uniaxial tensile stress at failure and ϵ is the eccentricity. Eccentricity defines the rate at which the flow potential function approaches the asymptote. This flow potential ensures a uniquely defined flow direction. The default flow potential eccentricity is 0.1, which implies the same dilation angle over wide range of confining stress values (ABAQUS, 2013). Increasing the value of eccentricity provides more curvature to the flow potential, implying that the dilation angle increases more rapidly as the confining pressure decreases.

Hardening law: Damaged states in tension and compression are characterized independently by two hardening variables, $\tilde{\epsilon}_t^{pl}$ and $\tilde{\epsilon}_c^{pl}$, which are referred to as equivalent plastic strain in tension and compression, respectively. Based on Lee and Fenves (1998), the equivalent plastic strain rates are defined as

$$\dot{\tilde{\epsilon}}_t^{pl} = r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right) \cdot \hat{\epsilon}_{\max}^{pl} \quad \text{Equation 3-59}$$

$$\dot{\tilde{\epsilon}}_c^{pl} = -\left(1 - r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right)\right) \cdot \hat{\epsilon}_{\min}^{pl} \quad \text{Equation 3-60}$$

where, $\hat{\epsilon}_{\max}^{pl}$ and $\hat{\epsilon}_{\min}^{pl}$ are maximum and minimum eigenvalues of the plastic strain rate tensor ($\dot{\epsilon}^{pl}$). The stress weight factor $r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right)$ is expressed as

$$r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right) = \frac{\sum_{i=1}^3 \left\langle \frac{\hat{\sigma}}{\bar{\sigma}} \right\rangle}{\sum_{i=1}^3 \left| \frac{\hat{\sigma}}{\bar{\sigma}} \right|}; 0 \leq r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right) \leq 1 \quad \text{Equation 3-61}$$

$$\dot{\tilde{\epsilon}}^{pl} = \begin{bmatrix} \dot{\tilde{\epsilon}}_t^{pl} \\ \dot{\tilde{\epsilon}}_c^{pl} \end{bmatrix} = \hat{h}\left(\frac{\hat{\sigma}}{\bar{\sigma}}, \tilde{\epsilon}^{pl}\right) \cdot \hat{\epsilon}^{pl} \quad \text{Equation 3-62}$$

$$\hat{h}\left(\frac{\hat{\sigma}}{\bar{\sigma}}, \tilde{\epsilon}^{pl}\right) = \begin{bmatrix} r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right) & 0 & 0 \\ 0 & 0 & -\left(1 - r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right)\right) \end{bmatrix} \quad \text{Equation 3-63}$$

where,

$\dot{\tilde{\epsilon}}_t^{pl}$ Equivalent plastic strain rate in tension

$\dot{\tilde{\epsilon}}_c^{pl}$ Equivalent plastic strain rate in compression

$\hat{\epsilon}^{pl}$ Eigenvalues of plastic strain rate tensor

The equivalent plastic strain can be obtained by integrating the equivalent plastic strain rate.

$$\tilde{\epsilon}_t^{pl} = \int_0^t \dot{\tilde{\epsilon}}_t^{pl} dt \quad \text{Equation 3-64}$$

$$\tilde{\epsilon}_c^{pl} = \int_0^t \dot{\tilde{\epsilon}}_c^{pl} dt \quad \text{Equation 3-65}$$

Stiffness degradation: This model assumes isotropic elastic stiffness degradation and is characterized by single scalar degradation variable d . Materials damaged elastic stiffness is defined as

$$D^{el} = (1-d)D_o^{el}; 0 \leq d \leq 1 \quad \text{Equation 3-66}$$

where,

D^{el} Damaged elastic stiffness tensor

D_o^{el} Initial(undamaged) elastic stiffness tensor

The scalar degradation variable d is the function of stress state and uniaxial damage variables, d_t (tensile damage) and d_c (compressive damage),

$$(1-d) = (1-s_t d_c)(1-s_c d_t), 0 \leq s_t, s_c \leq 1 \quad \text{Equation 3-67}$$

$$s_t = 1 - \omega_t r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right); 0 \leq \omega_t \leq 1 \quad \text{Equation 3-68}$$

$$s_c = 1 - \omega_c \left(1 - r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right)\right); 0 \leq \omega_c \leq 1 \quad \text{Equation 3-69}$$

where, ω_t and ω_c are the weight factors on the stress weight factor $r\left(\frac{\hat{\sigma}}{\bar{\sigma}}\right)$. $\omega_t = 0$ and $\omega_c = 1$ are the default values for the weight factors. The weight factor $\omega_c = 1$ corresponds to a complete recovery of the compressive stiffness upon crack closure as the load changes from tension to compression. Meanwhile, a weight factor $\omega_t = 0$ corresponds to no recovery of the tensile stiffness as the load changes from compression to tension once cracks have developed. If the damage variables d_t (tensile damage) and d_c (compressive damage) are not specified, the slope of the unloading curve is equal to the slope of the elastic portion of the stress-strain curve.

3.2.2.1.5 Input Parameters

Three dimensional concrete plasticity models need input parameters based on the test data. In the absence of experimental data, typical values can be used. The following section discusses the typical values used by researchers in the past.

Smearred crack model: The shape of the failure surface is defined using ratios of stresses under uniaxial and multiaxial state. These ratios are called failure ratios. Chen(1982) provides ranges of values expected for the first two failure ratios.

Table 3-2: Typical failure ratio values.

Failure ratios	Chen (1982)	Abaqus default
Ratio 1 (r_{bc}^{σ})	1.10-1.16	1.16
Ratio 2 (r_t^{σ})	0.08-0.10	0.1
Ratio 3 (r_{bc}^{ε})	-	1.28
Ratio 4 (f)	-	0.333

where,

r_{bc}^{σ} Ratio of the ultimate biaxial compressive stress to the ultimate uniaxial compressive stress

r_t^{σ} Absolute value of the ratio of the uniaxial tensile stress at failure to the ultimate uniaxial compressive stress

r_{bc}^{ε} Ratio of the magnitude of a principal component of plastic strain at ultimate stress in biaxial compression to the plastic strain at ultimate stress in uniaxial compression

f Ratio of the tensile principal stress at cracking, in plane stress, when the other principal stress is at the ultimate compressive value, to the tensile cracking stress under uniaxial tension

Damaged plasticity model: The dilation angle (ψ) is the angle made by the plastic strain increment vector with the failure surface. It represents an increase in volume relative to the initial state. For $\psi = 0$ the inelastic deformation is incompressible; if $\psi \geq 0$, the material dilates. This angle can be set equal to the angle of internal friction for the associated flow rule. Based on the evaluation of the Modified Coulomb criteria under uniaxial compression, the angle of internal friction can be calculated using Equation 3-70 (Nielsen and Linh, 2010). Nielsen and Linh(2010) present a plot (Figure 3-5) showing the relationship between the angle of internal friction and the compressive strength of concrete. The internal friction angle is around 37° for low strength concrete and decreases to a constant value of 27° for high strength concrete.

$$f_c = 2c \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad \text{Equation 3-70}$$

where,

ϕ	Internal friction angle (Degrees)
f_c	Concrete compressive strength (MPa)
c	Cohesion (MPa)

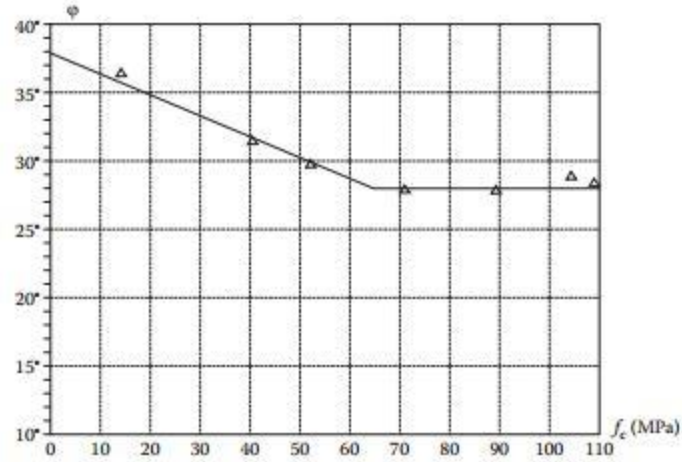


Figure 3-5: Internal friction angle (ϕ) vs. concrete compressive strength (f_c) (Nielsen and Linh, 2010), used under fair use.

Goodman(1980) suggested that the internal friction angle for concrete is between 36 ° to 45 °. Li and Sung(2003) proposed a simplified relationship between the internal friction angle and compressive strength of concrete,

$$\phi = 36 + 1 \left(\frac{f_c'}{35} \right) \leq 45 \quad \text{Equation 3-71}$$

where, ϕ is the internal friction angle (degrees) and f_c is the concrete compressive strength (MPa). Equation 3-71 shows increase in internal friction angle with increase in compressive strength. The Li and Sung (2003) relationship shows trend opposite to that observed by Nielsen and Linh(2010). In this research, the internal friction angle is selected based on Figure 3-5 by Nielsen and Linh(2010).

The eccentricity (ε) defines the rate at which concrete flow potential in the meridian plane approaches the linear Drucker-Prager flow potential asymptotically. ABAQUS (2013)

recommends default value of 0.1 for eccentricity. When $\varepsilon = 0$, the flow potential becomes linear Drucker-Prager flow potential.

The parameter K_c is defined as the ratio of Mises equivalent effective stress on the tension meridian to the compression meridian. It governs the shape of the yield surface and yield surface becomes a circle in the deviatoric plane for K_c equal to 1.0. Lubliner et al. (1989) suggested that the typical value of K_c ranges from about 0.64 to 0.80. ABAQUS (2013) recommends a default value of 0.67 for K_c . When K_c is set to unity and dilatation angle set equal to the angle of internal friction, flow potential reduces to the original Drucker-Prager plasticity model.

The ratio of the ultimate biaxial compressive stress to the ultimate uniaxial compressive stress is used in the determination of the coefficient α used in the yield surface definition. This ratio falls in the range of 1.10 to 1.16, yielding values of α between 0.08 and 0.12 (Lubliner, et al., 1989). ABAQUS (2013) uses default value of 1.16 for this ratio.

3.2.2.2 Structural Steel and Reinforcing Steel

3.2.2.2.1 Uniaxial stress-strain curve

The classical metal plasticity model in ABAQUS (2013) uses uniaxial stress-strain data as one of the inputs for plasticity definition. It is assumed that the material stress-strain (Figure 3-6) behavior can be characterized using the following parameters,

- Elastic modulus (E)
- Yield ratio: ratio of yield stress(F_y) to ultimate stress (F_u)
- Ratio of strain at strain hardening(ε_{st}) to yield strain(ε_y)
- Strain hardening modulus(E_{st})
- Material modulus post ultimate stress(E_{st1})
- Ratio of strain at fracture(ε_f) to yield strain(ε_y)

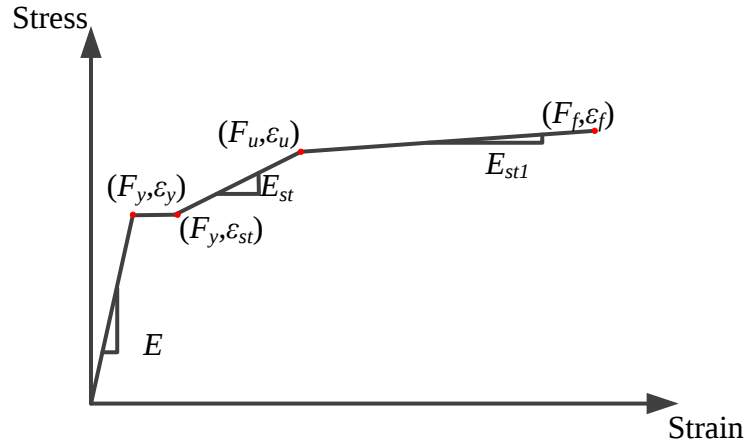


Figure 3-6: Steel uniaxial stress-strain curve.

Based on the research by Salmon et al. (2009), Barth et al. (2000) and White et al. (1997) , the following values (Table 3-3) for the parameters shown in Figure 3-6 are selected for different grades of steel.

Table 3-3: Stress-strain parameters for structural steel.

Input parameters	A 36	A572 G50	HPS 70W
Elastic modulus (E)	29000 ksi		
Yield ratio (F_y/F_u)	0.55-0.62	0.77	0.83
Ratio of strain at strain hardening to yield strain (ϵ_{st}/ϵ_y)	10	10	1.9
Strain hardening modulus (E_{st})	650 ksi	720 ksi	280 ksi
Modulus post ultimate stress (E_{st1})	10 ksi	10 ksi	14.5 ksi
Ratio of strain at fracture to yield strain (ϵ_f/ϵ_y)	80-100		

Similar parameters are presented in Chen (1982) to classify reinforcement stress-strain curve and are tabulated in Table 3-4. Figure 3-7 shows the stress-strain curve used by Barth and Wu(2006) to predict the ultimate load carrying capacity of the composite steel girder bridges.

Table 3-4: Stress-strain parameters for reinforcement.

Input parameters	Grade 40	Grade 60
Elastic modulus (E)	29000 ksi	
Yield stress (F_y)	40 ksi	60 ksi
Yield ratio (F_y/F_u)	1.55	
Ratio of strain at strain hardening to yield strain (ϵ_{st}/ϵ_y)	8-15	
Strain hardening modulus (E_{st})	700 ksi	
Modulus post ultimate stress (E_{st1})	10 ksi	
Ratio of strain at fracture to yield strain (ϵ_f/ϵ_y)	50	

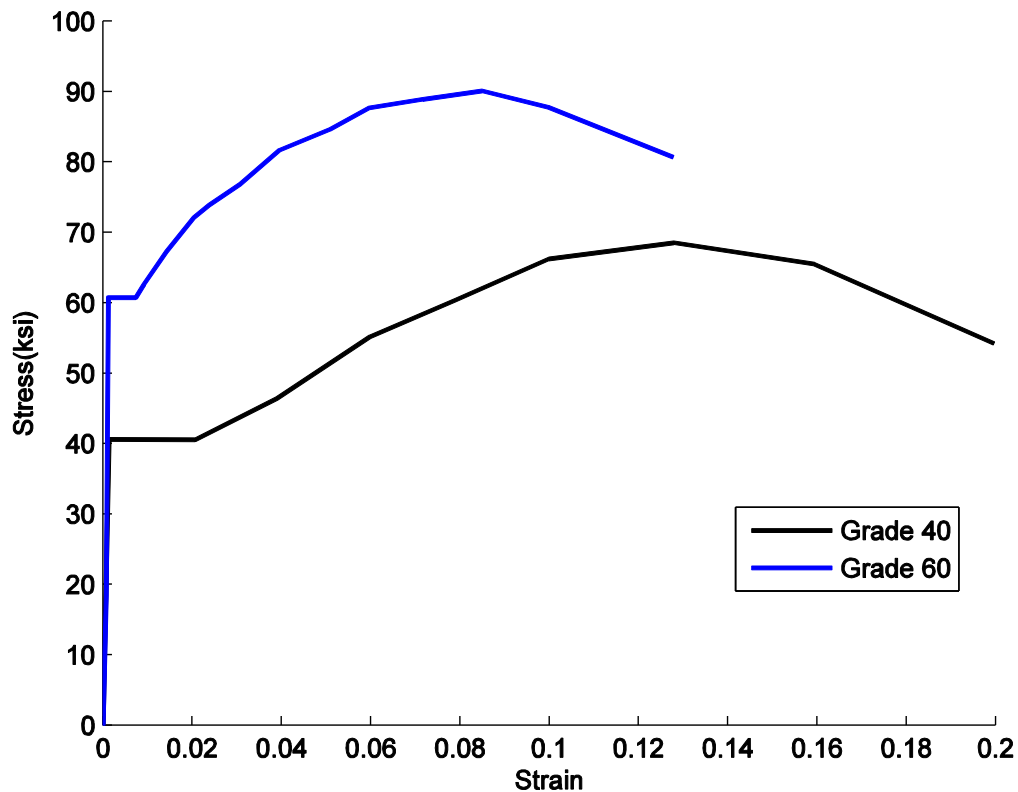


Figure 3-7: Reinforcement stress-strain curve (Barth and Wu, 2006), used under fair use.

3.2.2.2.2 Classical metal plasticity

The classical metal plasticity model is intended for modeling rate-independent or rate-dependent material behavior subjected to static or dynamic, cyclic loading. It can be used in conjunction with the progressive damage and failure models. It uses linear elastic material model (Equation 3-72) to define elastic properties.

$$\bar{\sigma} = D^{el} : \bar{\varepsilon} \quad \text{Equation 3-72}$$

where, $\bar{\sigma}$ is the stress tensor, $\bar{\varepsilon}$ is the strain tensor and D^{el} is the initial elastic stiffness tensor. ABAQUS (2013) allows plasticity definition with or without work hardening. For material with work hardening definition ABAQUS (2013) allows users to select between Isotropic hardening model, Johnson-Cook isotropic hardening model, Kinematic hardening with linear and nonlinear isotropic or kinematic hardening models.

Yield surface: ABAQUS (2013) uses a Mises yieldad surface for isotropic materials like steel, which assumes that yielding is independent of equivalent pressure. This is true for metals except when subjected to conditions of high triaxial tension when voids may nucleate and grow in the material. These conditions might arise in stress field near crack tips and in some extreme thermal loading (ABAQUS, 2013). ABAQUS (2013) uses incremental plasticity theory in which total mechanical strain rate ($\dot{\varepsilon}$) is decomposed into elastic strain rate ($\dot{\varepsilon}^{el}$) and plastic strain rate ($\dot{\varepsilon}^{pl}$) associated with the yield surface.

Isotropic hardening: The yield surface for isotropic hardening is defined as

$$f(\bar{\sigma}) = \sigma^0(\varepsilon^{pl}, \theta) \quad \text{Equation 3-73}$$

where,

$\bar{\sigma}$	Stress tensor
$\sigma^0(\varepsilon^{pl}, \theta)$	Equivalent(uniaxial) stress
ε^{pl}	Plastic strain
θ	Temperature

Kinematic hardening: The yield surface for kinematic hardening is defined as

$$f(\bar{\sigma} - \bar{\alpha}) = \sigma^0(\varepsilon^{pl}, \theta) \quad \text{Equation 3-74}$$

where,

$f(\bar{\sigma} - \bar{\alpha})$	Equivalent von Mises or Hill's potential surface
$\bar{\alpha}$	Backstress
$\sigma^0(\varepsilon^{pl}, \theta)$	Size of yield surface
ε^{pl}	Plastic strain

θ Temperature

Flow rule: This model assumes associated plastic flow rule (Equation 3-75)

$$\dot{\varepsilon}^{pl} = \dot{\lambda} \frac{\partial f}{\partial \sigma} \quad \text{Equation 3-75}$$

where,

f Yield surface

$\dot{\varepsilon}^{pl}$ Plastic flow rate

$\dot{\lambda}$ Scalar measuring the amount of the plastic flow rate

Hardening law: Isotropic hardening assumes that the yield surface changes size uniformly in all directions. ABAQUS (2013) uses the equivalent stress ($\sigma^o(\varepsilon^{pl}, \theta)$) as a function of plastic strain to define isotropic hardening. The stress at a given state is interpolated from the input data and it remains constant for the plastic strains exceeding the last input value (ABAQUS, 2013).

Kinematic hardening assumes that during plastic deformation, the loading surface translates as a rigid body in stress space, maintaining the size, shape, and orientation of the initial yield surface. Kinematic hardening is the simplest rule that captures Bauschinger effect and anisotropy induced by work hardening (ABAQUS, 2013). The backstress tensor ($\bar{\alpha}$) specifies the total translation of the center of the yield surface from the origin. The user can define the backstress tensor using a linear or nonlinear isotropic or kinematic hardening model.

3.2.2.3 Rate dependent material properties

Structural elements subjected to dynamic load from earthquake, fracture, blast loading exhibit higher strength than similar elements under static loading. This increase in strength results from increased material strength under rapid loading rates. UFC 3-340-02 (2008) provides guidelines for the selection of rate dependent properties for structural steel and reinforced concrete. UFC 3-340-02 (2008) provides dynamic increase factor (DIF) i.e. the ratio of the dynamic stress to the static stress. Following section provides rate dependent properties for structural steel and reinforced concrete that will be used in this research.

3.2.2.3.1 Reinforced concrete

Reinforced concrete shows increase in compressive strength with increase in strain rate. UFC 3-340-02 (2008) provides design curves for the DIF for the ultimate unconfined

compressive strength of concrete between 2,500 – 5,000 psi. Bridge decks are typically constructed with 3,000 – 5,000 psi concrete and are unconfined. Therefore, the design curves provided in UFC 3-340-02 (U.S. Department of Defense, 2008) are used in present research. The DIF values used to model rate dependent behavior of concrete are tabulated in Table 3-5.

Table 3-5: DIF for ultimate unconfined compressive strength of concrete (2,500 – 5,000 psi).

Concrete	
$\dot{\epsilon}$ (1/sec)	DIF
0.005	1.09
0.01	1.11
0.05	1.15
0.1	1.19
0.5	1.30
1	1.37

As the strain rate increases, the secant modulus of elasticity of concrete increases slightly, and the strain at maximum stress and rupture remain nearly constant. Schuler et al. (2006) compiled data from tests and literature about the increased tensile strength and fracture energy of concrete at high strain rates. The data clearly indicates an increase in tensile strength and fracture energy of concrete at high strain rates especially in the range between 10^1 and 10^2 1/s. For simplicity no increase in tensile strength and fracture energy is assumed at higher strain rates in the UFC 3-340-02 provisions (2008). The ultimate strength of the reinforcement is less sensitive to the strain rate compared to the yield strength of the reinforcement. The modulus of elasticity and the rupture strain of the reinforcement stays the same at different strain rates. The DIF values from UFC 3-340-02 provisions (2008) are used to model the rate dependent behavior of reinforcement and are tabulated in Table 3-6.

Table 3-6: DIF for reinforcement.

$\dot{\epsilon}$ (1/sec)	DIF					
	Grade 40		Grade 60		Grade 75	
	f_y	f_u	f_y	f_u	f_y	f_u
0.0001	1.00	1.00	1.00	1.00	1.00	1.00
0.001	1.10	1.03	1.06	1.02	1.04	1.01
0.010	1.22	1.05	1.14	1.04	1.08	1.03
0.10	1.34	1.09	1.21	1.06	1.12	1.04
1.0	1.47	1.12	1.30	1.08	1.16	1.05
10.0	1.62	1.15	1.38	1.10	1.20	1.07

3.2.2.3.2 Structural steel

Structural steel rate dependent behavior is similar to that of reinforcement. The yield strength increases substantially with increase in strain rate. The modulus of elasticity and rupture strain remains insensitive to the loading rate. The ultimate strength increases slightly. However the percentage increase is less than that for the yield stress. UFC 3-340-02 (2008) recommends average strength increase of 10% for all steel below 50 ksi yield strength along with the dynamic increase factor (DIF). UFC 3-340-02 (2008) provides DIF at different strain rates for A36 and A514 steel based on the experimental data from the literature and it also provides estimate for the A588 steel. The DIF values used to model rate dependent behavior of structural steel are tabulated in Table 3-7.

Table 3-7: DIF for structural steel.

$\dot{\epsilon}$ (1/sec)	DIF					
	A36		A588		A514	
	f_y	f_u	f_y	f_u	f_y	f_u
0.02	1.19	1.1	1.12	1.05	1.05	1
0.05	1.24		1.15		1.05	
0.1	1.29		1.19		1.09	
0.3	1.36		1.24		1.12	

3.2.2.3.3 Implementation in ABAQUS (2013)

ABAQUS (2013) implements rate dependent plasticity only for the isotropic hardening model, the isotropic component of the nonlinear isotropic/kinematic plastic model and the Drucker-Prager plasticity model. It can be implemented using two methods. In the first method, the user

provides true stress vs. equivalent plastic strain data at different strain rates. In the second method, strain rate behavior is assumed to be separable so that the stress-strain dependence is similar at all strain rates. Strain rate behavior is implemented through use of yield stress ratio (R). The yield stress ratio (R) is defined as the ratio of yield stress at non-zero strain rate to the static yield stress. The damaged plasticity model for concrete uses true stress vs. equivalent plastic strain data at different strain rates to define concrete behavior in tension and compression.

3.2.3 Imperfection and residual stresses

Chapter 2 presented a review of design standards and research publications pertaining to imperfections and residual stresses. The literature showed that the geometric imperfection and residual stresses do not play a significant role in the ultimate load capacity and load displacement response of the structure as long as its response is not governed by local behavior. To understand its impact, one need to consider different combinations of imperfection patterns for different members and this would result in large number of analyses runs. Furthermore, in the event of fracture one would expect the second order effects and member plasticity to dominate the global response and bridge capacity and therefore residual stresses and geometric imperfections are not considered for this research.

3.2.4 Modeling of composite action

From the literature reviewed in Chapter 2, it is clear that the interaction at the steel-concrete interface with or without shear studs is a complex phenomenon and is necessary to capture the realistic analytical response of a fractured bridge. This section discusses different methods that will be used to model the composite action.

3.2.4.1 Shear studs subjected to shear

Two methods are considered to model shear studs subjected to shear. The first method uses tie constraints or rigid links simulating full composite action or rigid stud behavior at the steel and concrete interface. The tie constraint imposes a multi-point constraint between master and slave nodes, eliminating the degrees of freedom at the slave nodes. The beam connector elements kinematically constrain all degrees of freedom at two connector nodes without eliminating any degrees of freedom. Eliminating degrees of freedom in the tie constraint reduces the size of stiffness matrix and thus improving efficiency of the analysis but constrains users from directly applying external loads at the degrees of freedom of slaved nodes.

In the second method, composite action at the interface is modeled using empirical load vs. slip relationships, such as that proposed by Ollgaard et al. (1971). Interface behavior is modeled using a pair of connector elements in series by introducing additional node at the top surface of the top flange. A beam connector is used to connect the deck node to the additional node while a (Radial-Thrust +Align) connector is used to connect the additional node to the top flange node. The Radial-Thrust connector is used to define the interface translational properties while the Align connector is used to define the interface rotational properties. The Align connector constrains all three rotational degrees of freedom between the additional node and the top flange node ensuring rotational compatibility between them. The radial component of the Radial-Thrust connector is assigned with the load vs. slip model (Equation 3-76 and Equation 3-77) proposed by Ollgaard et al. (1971). The connector definition in ABAQUS (2013) needs the load vs. slip response to be divided into elastic and plastic response. The initial stud stiffness is calculated by assuming elastic behavior up to slip of 0.01 in. The plastic load vs. slip response is obtained by subtracting the elastic response from the total load vs. slip response. The thrust component of the Radial-Thrust connector is made analytically rigid to prevent any separation at the steel-concrete interface.

$$Q_u = 1.106 A_s f_c'^{0.3} E_c^{0.44} \quad \text{Equation 3-76}$$

$$Q = Q_u (1 - e^{-18\Delta})^{2/5} \quad \text{Equation 3-77}$$

where,

f_c'	Concrete compressive strength (ksi)
A_s	Shear stud cross sectional area (in. ²)
E_c	Modulus of elasticity of concrete (ksi)
Q_u	Shear strength of shear stud (kips)
Q	Applied load (kips)
Δ	Slip (in.)

3.2.4.2 Shear Studs Subjected to Combined Tension and Shear

Shear studs are installed to transfer the shear force at the steel-concrete interface. However, in the event of fracture in steel beam or truss member, the shear studs are subjected to combined

shear and tension near fractured region making it necessary to account for shear-tension interaction in the finite element model to capture the realistic structural response.

Shear studs subjected to combined shear and tension would result in family of curves in shear and tension for different shear-tension ratios. This kind of data is not available in the literature but interaction equations for combined strength check are available in ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004). The PCI Design Handbook, 6th Edition (2004) equation (Equation 3-78) is used to model the shear-tension interaction in this reserach. Further details about this equation are provided in Section 2.3.2.

The shear-tension failure envelope is modeled using connector damage initiation definition. The axial behavior of the shear stud is defined using Macauley bracket (ABAQUS, 2013), which reduces the tension-shear interaction equation to shear only failure criteria under compression-shear loading condition. Once the connector element reaches the interaction envelope connector fails. Post failure stud behavior is defined using damage evolution definition. ABAQUS (2013) offers displacement based and energy based damage evolution criterias.

$$\left(\frac{Q_{rt}}{\phi_t Q_{nt}} \right)^{5/3} + \left(\frac{Q_{rv}}{\phi_v Q_{nv}} \right)^{5/3} \leq 1.0 \quad \text{Equation 3-78}$$

where,

Q_{nt}	Nominal tensile strength (kips)
Q_{rt}	Required tensile strength (kips)
Q_{nv}	Nominal shear strength (kips)
Q_{rv}	Required shear strength (kips)
ϕ_t	Strength reduction factor in tension a) Stud governed by strength of a ductile steel element $\phi_t=0.75$ b) Stud governed by a brittle steel element $\phi_t=0.65$
ϕ_v	Strength reduction factor in shear a) Stud governed by strength of a ductile steel element $\phi_v=0.65$ b) Stud governed by a brittle steel element $\phi_v=0.60$

3.2.4.3 *Non-composite Action*

In case of non-composite steel girders with concrete deck, friction at the steel-concrete interface provides the interface shear resistance. Surface-to-surface contact with finite-sliding was used to simulate the non-composite action. Surface-to-surface contact in ABAQUS (2013) is capable of simulating both the normal and tangential behavior at the interface. The normal behavior is modeled using the hard contact that minimizes penetration of the slave surface into the master surface and does not allow for the transfer of tensile stresses across the interface.

Hard contact is implemented using penalty method in ABAQUS (2013) that approximates the behavior using linear or nonlinear penalty stiffness at the interface. ABAQUS (2013) by default sets a penalty stiffness (K_{in}) equal to 10 times the stiffness of representative underlying elements (K_e) for its linear penalty method. For the nonlinear penalty method, the penalty stiffness increases exponentially with over closure (ABAQUS, 2013). The default initial penalty stiffness (K_i) is equal to the representative underlying element stiffness (K_e). The stiffness increases exponentially with the final penalty stiffness (K_f) equal to 100 times the stiffness of representative underlying element. With the nonlinear penalty method the low initial penalty stiffness typically results in better convergence, while the higher final stiffness keeps the over-closure at an acceptable level as the contact pressure builds up (ABAQUS, 2013).

The tangential behavior is modeled using an isotropic coulomb friction model. The isotropic coulomb friction model requires a coefficient of static friction at the interface. Table 3-8 tabulates values of the coefficient of static friction at the steel-concrete interface from the literature. Rabbat and Russell (1985) test setup resembles non-composite action at the top flange and slab interface. Rabbat and Russell (1985) recommend value of 0.57 for coefficient of static friction at dry interface. Impact of coefficient of static friction value on load-displacement response is investigated in Section 3.3.2.4.

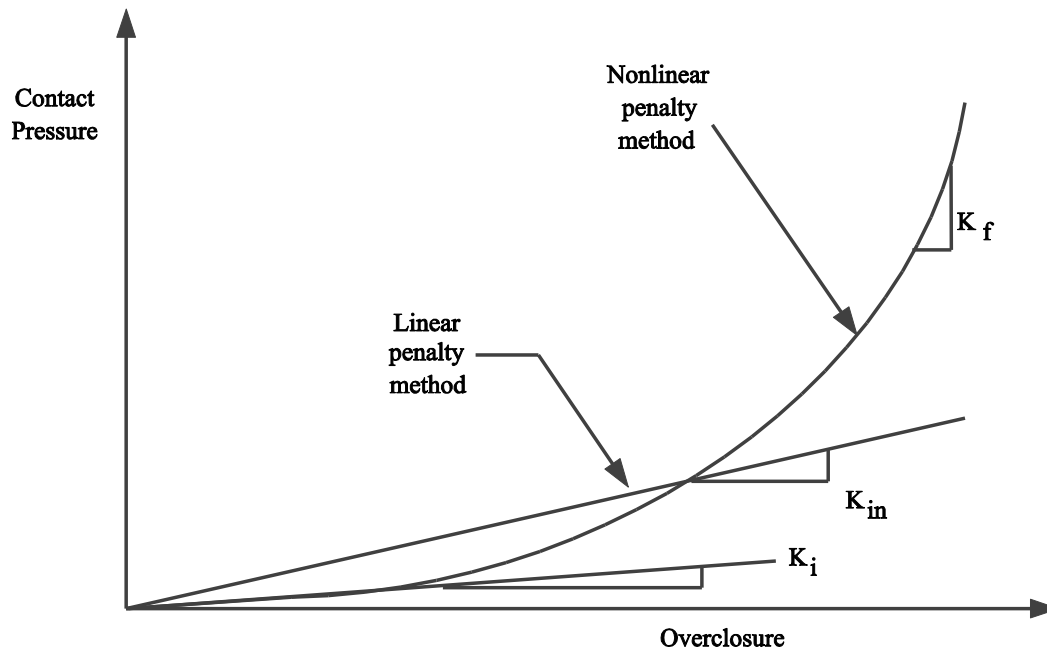


Figure 3-8: Nonlinear penalty pressure-overclosure relationship (ABAQUS, 2013), used under fair use.

Table 3-8: Coefficient of static friction between concrete and steel.

Literature	Coefficient of static friction (μ_k)
Chiewanichakorn et al.(2004)	0.30 - 0.80
Rabbat and Russell(1985)	0.57 - 0.70
Baltay and Gjelsvik(1990)	0.20 - 0.60

3.2.5 Fracture simulation methods

The choice of a fracture simulation method depends on the level of accuracy expected from the analysis. In the first method, fracture is modeled by disconnecting the elements along the fracture path by introducing duplicate nodes before the start of the analysis. This is the simplest method to study the impact of fracture on the structural response but it does not capture the change in member forces and displacements from fracture and dynamic effects associated with the member fracture. This method can be used to evaluate the post fracture load carrying capacity of the bridge if the deformations and strains following a fracture event are small and do not influence the load carrying capacity significantly.

In the second method, fracture is also modeled by introducing duplicate nodes at the predefined fracture path (Figure 3-9). At the start of the analysis, the duplicate nodes are connected using

zero length beam/weld connector elements to mimic intact condition. Fracture is simulated by removing the connector elements using the “Model Change” option in a very short transient step (~ 0.001 sec). “Model Change” option stores the forces in the region prior to removal of connector elements and ramps down forces to zero over the step to simulate element removal. No additional loads are applied during that step and the structure is allowed to reach a deformed configuration. This method allows for simulation of fracture in static and dynamic analysis in *ABAQUS/Standard*. Compared to the first method, this method captures change in member forces and displacement post fracture.

To simulate fracture in *ABAQUS/Explicit*, the analysis is carried out using the intact and the fractured model of the bridge. First the intact model of the bridge is analyzed and the forces in the connector elements at the location of fracture are recorded. These forces are then applied as equal and opposite nodal forces at the fracture location in the fractured bridge model and then ramped down over a short time to simulate fracture. Both methods capture behavior following a fracture assuming crack is arrested to the predefined length. They are incapable of capturing any fracture growth or subsequent fracture initiation in other members.

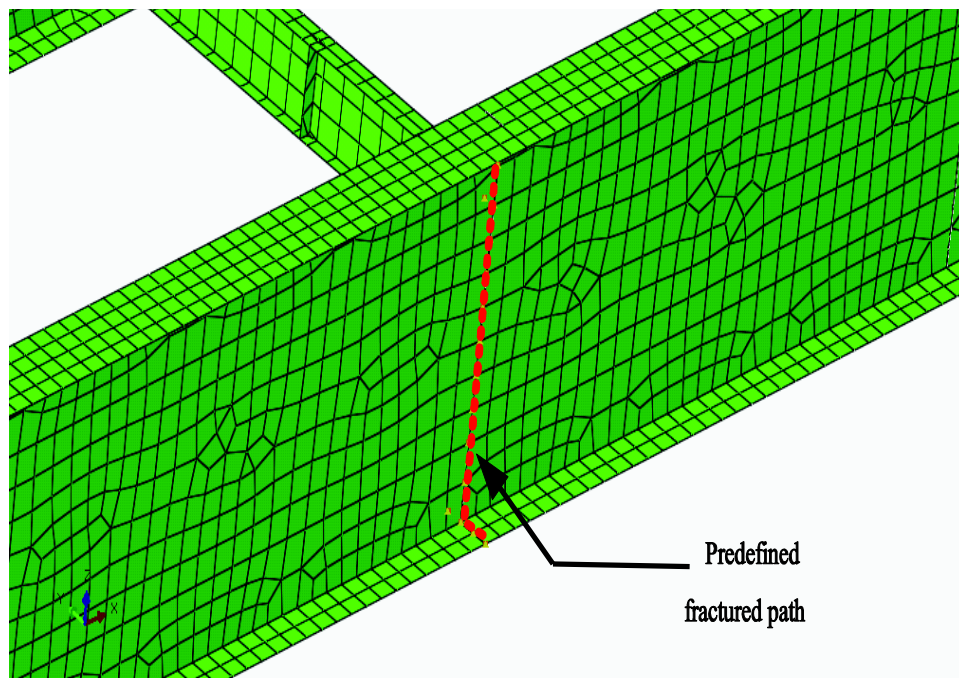


Figure 3-9: Predefined fracture path connected using beam connectors.

3.2.6 Live Load

AASHTO (AASHTO, 2012) represents each wheel load as a patch load with tire pressure uniformly distributed over the contact area. Dimensions of the tire patch can be calculated using the following expressions given in AASHTO C3.6.1.2.5 (AASHTO, 2012).

$$Tire\ width = \frac{P}{0.8} \quad \text{Equation 3-79}$$

And

$$Tire\ length = 6.4\gamma \left(1 + \frac{IM}{100}\right) \quad \text{Equation 3-80}$$

where,

γ	Load Factor
IM	Dynamic load allowance percent
P	Design wheel load (kips)

Application of truck loading using patch loads to represent each wheel is not trivial and often requires a fine mesh be used in the deck such that contact area dimensions of the patch matches the dimensions of the element. This method increases the computational cost and the complexity of deck meshing. Current research is focused on the global bridge response following a fracture in a fracture critical member; therefore the following simplified approach is used for application of truckload. Each wheel load is applied as a series of point loads on the nodes of elements occupied by the tire patch. Load on each node is proportioned using Equation 3-81 which is based on the node position compared to the center of tire patch, and position of the all the nodes of elements occupied by the tire patch. This method of load application does not conserve work done by the load as in the case of equivalent nodal loads but ensure application of total wheel load and avoids local plasticity arising from the application of truck load as a single point load.

$$Load_i = (Wheel_load) \times \frac{r_i}{\sum_{j=1}^n r_j} \quad \text{Equation 3-81}$$

where,

$Load_i$	Load at node i
$Wheel_load$	Total load applied by each wheel over the tire patch
r_i	Distance between the center of tire patch and node i
$\sum_{j=1}^n r_j$	Sum of distance between the center of tire patch and node j for all the nodes of elements occupied by tire patch.

3.3 Verification

Finite element models of the simply supported slab tested by Jofriet and McNeice (1971) and two simply supported composite steel plate girders tested by Mans et al. (2001) were developed using the proposed modeling methodology and compared against the experimental results. The simply supported slab model and composite girder model were further used to evaluate the impact of parameters used to define concrete and reinforcement plasticity on load carrying capacity of the slab. The simply supported composite girder model was also used to evaluate the modeling of composite action at the steel-concrete interface. This section first provides general attributes of the tests conducted by Jofriet and McNeice(1971) and Mans et al.(2001) followed by comparison between the experimental data and finite element results.

3.3.1 Corner supported two-way slab (Jofriet and McNeice, 1971)

Jofriet and McNeice (1971) tested a corner supported two-way slab (Figure 3-10), 36 in. square by 1.75 in. thick loaded with a point load at its center. The slab is reinforced in both directions with a reinforcement ratio of 0.85% in each direction. The reinforcement is placed at 75% of the slab depth measured from the top of the slab. Because of symmetry along both axes only one quarter of the model is analyzed with symmetric boundary condition along planes of symmetry. The corner support is modeled by restraining vertical movement of the slab. The slab is modeled using four node shell element S4R at its mid plane with reinforcement modeled as a smeared layer at the correct depth through slab thickness. The material properties from ABAQUS Example Problems Manual (ABAQUS, 2013) are used in this study and are tabulated in Table 3-9. These material parameters are taken from Gilbert and Warner (1978). The superscript “*” in Table 3-9 indicates the assumed values. Figure 3-11 shows the comparison between the experimental and analytical response. The analytical response is calculated using the smeared

crack model (Model-1) and damaged plasticity model (Model-2) for concrete and elastic perfectly plastic model for reinforcement. The input parameters used to define material plasticity are summarized in Table 3-9. A linear relationship is used to model the post yield response of concrete in compression. The behavior of concrete in tension is defined using tension stiffening model that defines a linear loss of strength beyond cracking failure of concrete. The concrete tensile strength is linearly reduced to zero at a strain of 10×10^{-4} . In Model-2, the dilation angle is set equal to the angle of internal friction. Based on the relationship between the angle of internal friction and concrete strength (Nielsen and Linh, 2010), a dilation angle of 32° is used in this study. The other input parameters used to define damaged plasticity are summarized in Table 3-9. Analysis is carried out using the Newton-Raphson method.

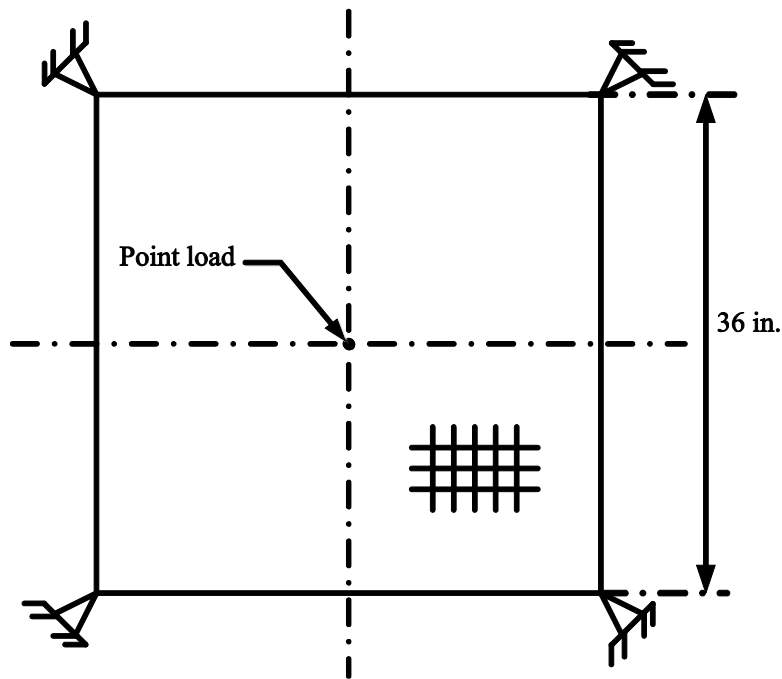


Figure 3-10: Jofriet and McNeice slab.

Table 3-9: Material properties.

Material Properties	Concrete	Reinforcing bar
Yield stress	3000 psi*	50x10 ³ psi
Ultimate stress	5500 psi	-
Young's modulus	4.15x10 ⁶ psi	29x10 ⁶ psi
Density	2.246x10 ⁻⁴ lb s ² /in ⁴	7.3x10 ⁻⁴ lb s ² /in ⁴
Cracking stress	459.8 psi	-
Plastic strain at compression failure	1.5x10 ⁻³ *	-
Tension stiffening	10x10 ⁻⁴ *	-
Ratio of ultimate biaxial to uniaxial compressive stress	1.16*	-
Poisson's ratio	0.15	0.3
Dilation angle (ψ)	32°	-
K_c	0.67	-
Eccentricity (ε)	0.1	-

(* - Assumed values)

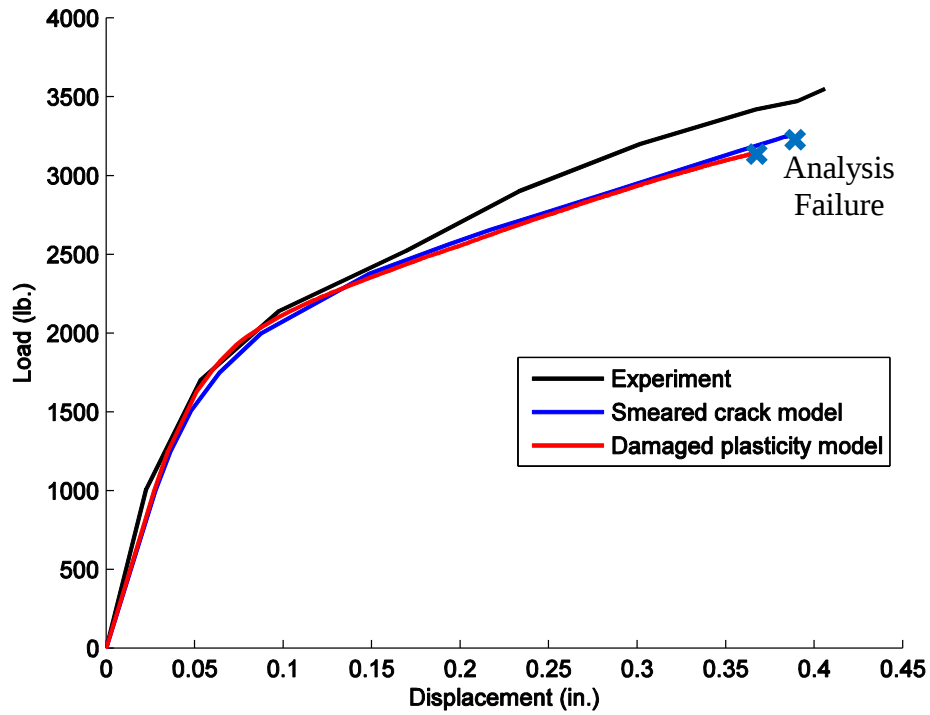


Figure 3-11: Load vs. displacement response at the center of slab.

Both the smeared crack model and damaged plasticity model predicted response close to the experimental response up to 2500 lb. Beyond 2500 lb. both concrete models predicted softer response. The model with the smeared crack plasticity model for concrete reached a marginally higher load compared to model with damaged plasticity definition (Table 3-10). Both analyses failed as a result of convergence difficulties before reaching the measured failure load of 3550 lb.

Table 3-10: Comparison between experimental and analytical response.

	Failure load (lb.)	% error	Displacement (in.)	% error
Experiment	3550		0.41	
Smeared crack model	3251	-8.4	0.39	-5.2
Damaged plasticity model	3137	-11.6	0.36	-10.3

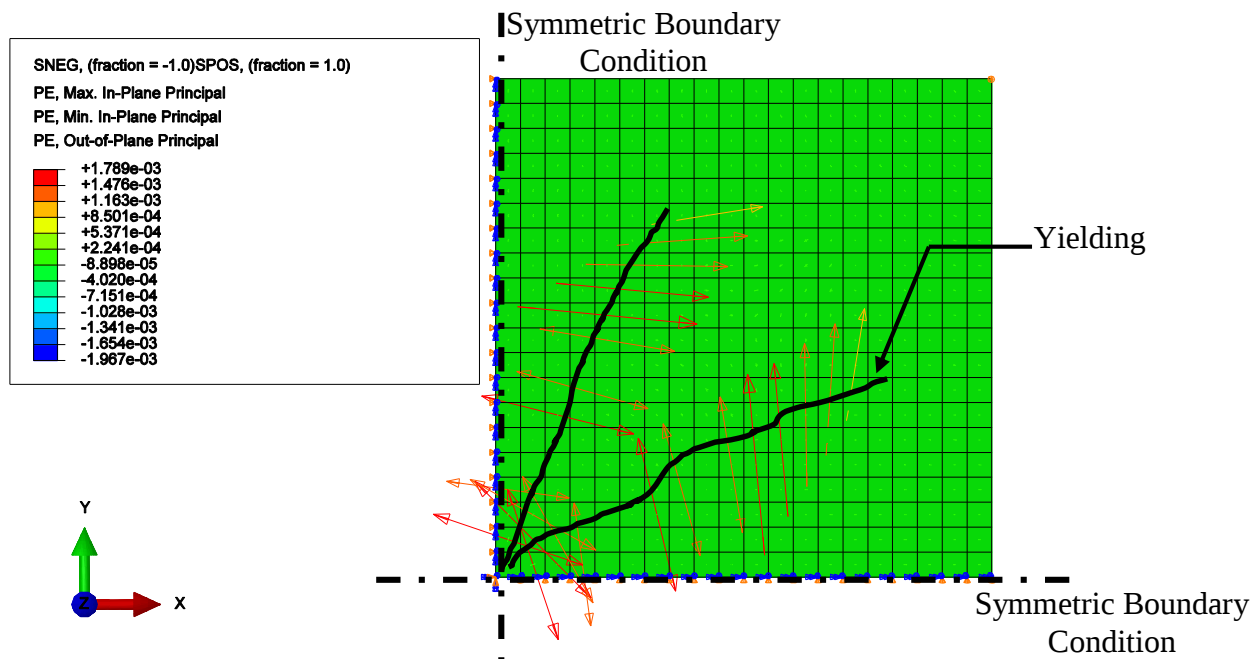


Figure 3-12: Principal plastic strain components (Model-1).

Figure 3-12 shows the vector representation of the principal plastic strain components for Model-1 at the failure load. Yielding occurs in the direction perpendicular to direction of principal plastic strain components.

3.3.1.1 Reinforcement properties

In the initial analysis performed at the start of this section, reinforcement plasticity was idealized as elastic perfectly plastic with the classical plasticity model from ABAQUS (2013). In the preliminary analysis, reinforcement with yield stress of 50 ksi was used. To understand the effect

of strain hardening and reinforcement grade on the load-displacement response, Model-1 is reanalyzed with the Grade 40 and Grade 60 reinforcement properties used by Barth and Wu (2006). Figure 3-13 shows the comparison between the experimental and analytical response with different grades of reinforcement. Use of Grade 60 reinforcement yielded response closest to the experimental response. Grade 40 reinforcement is ductile compared to Grade 60 reinforcement resulting in to ductile response of the slab. With Grade 40 reinforcement failure load is 3.3% lower than the experimental load but the displacement at the center of the slab is 74.2% higher compared to the recorded value. Grade 40 and Grade 60 analysis failed as a results of convergence difficulties before reaching the measured failure load of 3550 lb.

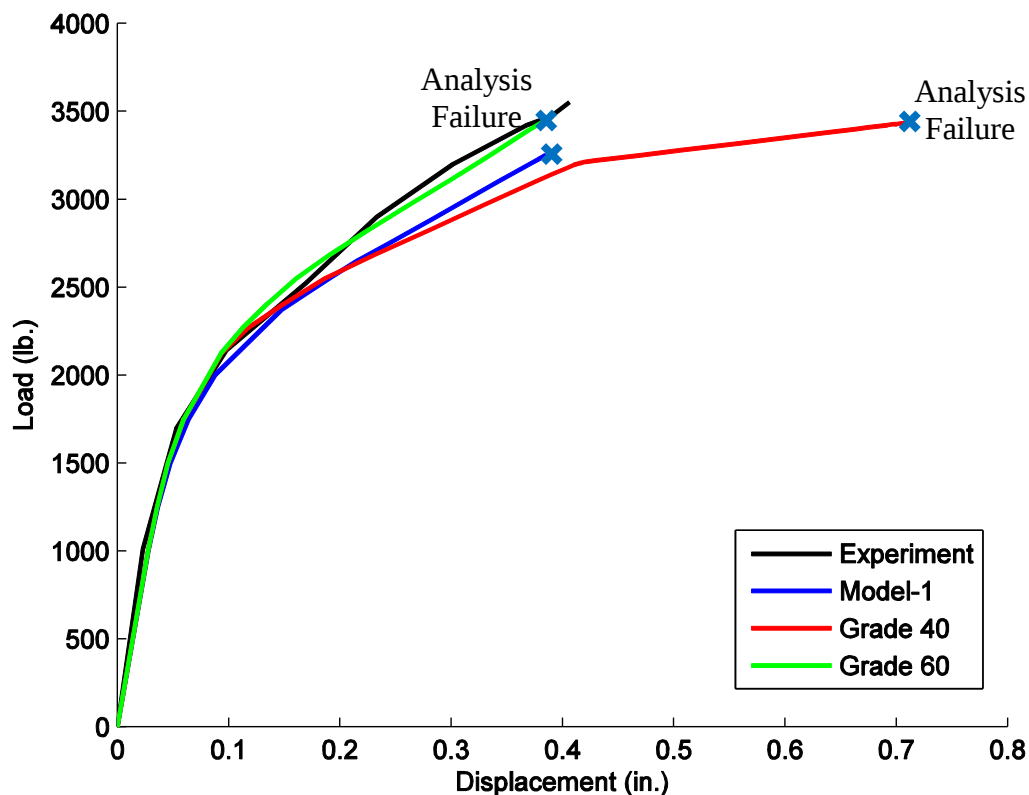


Figure 3-13: Load vs. displacement response at the center of slab for different grades of reinforcement.

3.3.1.2 Concrete properties

In the preliminary analysis, a linear relationship is used to model the post yield response of concrete in compression. The linear relationship post yield is far from the nonlinear post yield

response observed for concrete in compression. Model-1 is reanalyzed with three different uniaxial concrete stress-strain relationships in compression (Figure 3-14). The reinforcement plasticity was modeled using elastic perfectly plastic model used in the preliminary analysis. Section 3.2.2.1.1 provides a brief overview of the stress-strain relationship used for this study. The Hognestad (1951) and EN 1992-1-1 (2004) yielded nearly identical load vs. displacement response. Both of these models attained higher failure load and displacements compared to Model-1 and the CEB-FIP model (Comité euro-international du, 1993). The CEB-FIP model yielded lower strength and lower maximum displacement. Analyses with different concrete models failed before reaching the measured failure load of 3550 lb because of convergence difficulties.

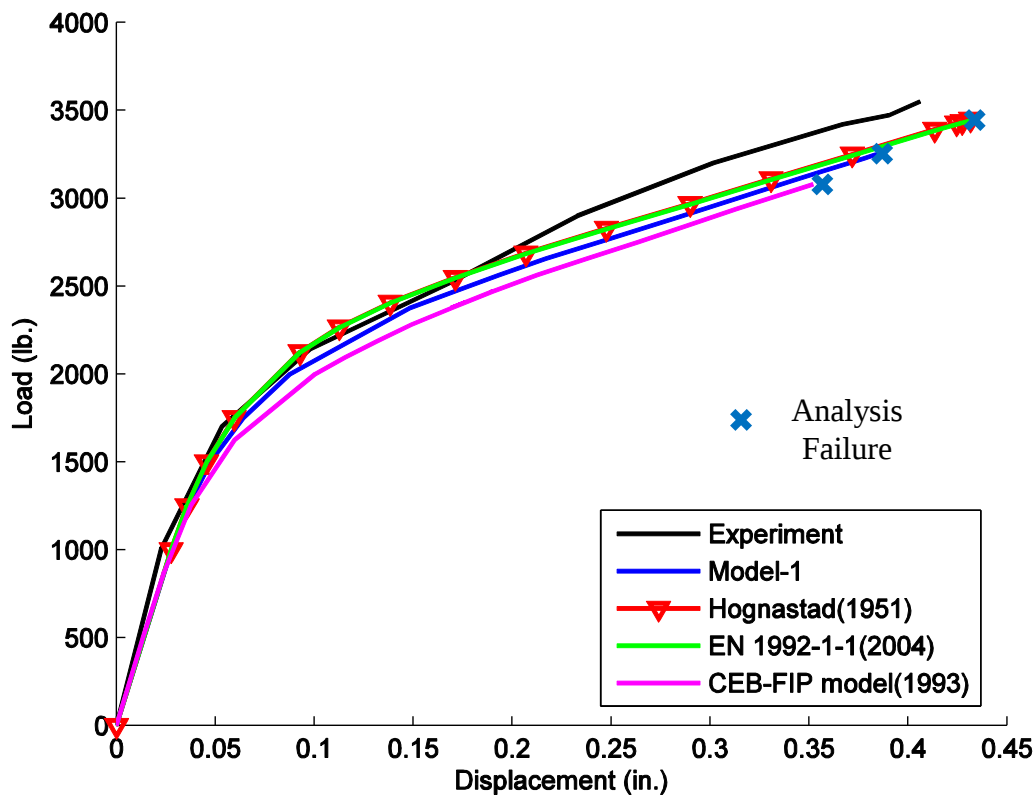


Figure 3-14: Load vs. displacement response at the center of slab for different concrete stress-strain curves.

In Model-1, the tensile strength of the concrete beyond cracking was reduced gradually to zero at a strain of 10×10^{-4} . To study the sensitivity of the value of failure strain under tension, Model-1

is reanalyzed for four different values of tensile failure strains (Figure 3-15). Increasing tensile failure strain value to 15×10^{-4} resulted in an increase in failure load of 15%. A tensile failure strain of 20×10^{-4} resulted in increase in failure load but reduced its ductility compared to the response with tensile failure strain of 15×10^{-4} . Analysis tensile failure strain of 5×10^{-4} failed as a results of convergence difficulties before reaching the measured failure load of 3550 lb.

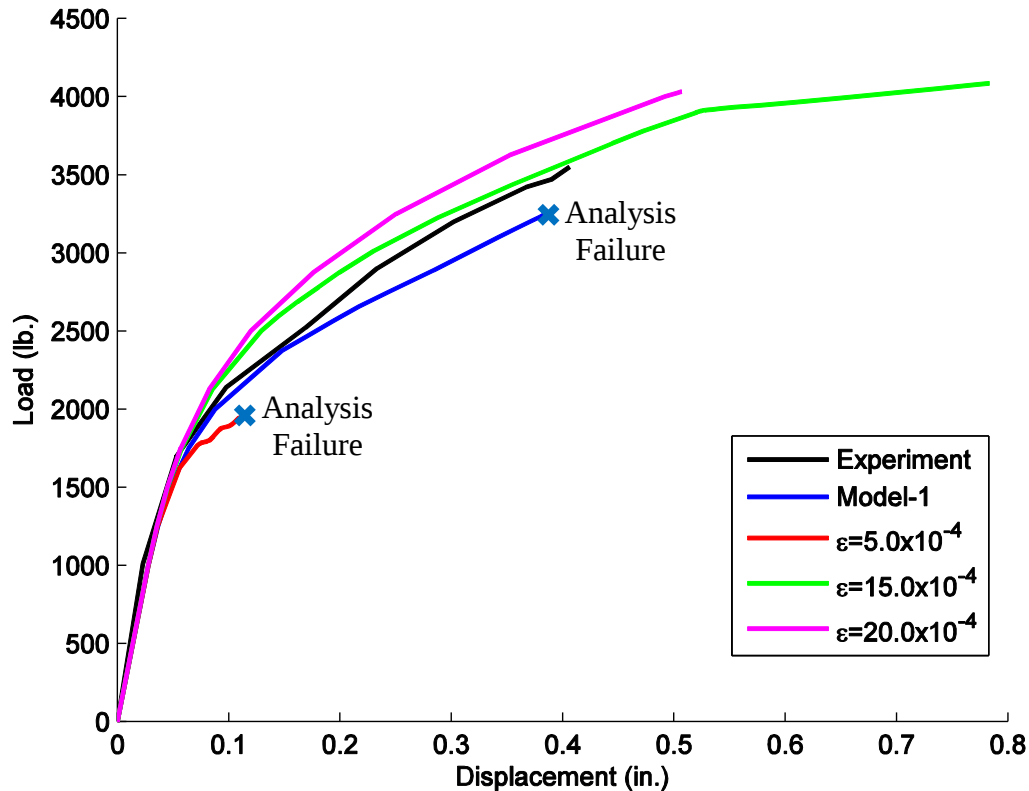


Figure 3-15: Load vs. displacement response at the center of slab for different tension stiffening values.

In the literature, several tension stiffening models are available to capture the behavior of concrete post cracking. They are discussed in detail in Section 3.2.2.1.2. To investigate their impact on the load-displacement response of the slab, Model-2 is reanalyzed with different tension stiffening models. For this analysis the compressive behavior of concrete is modeled using EN 1992-1-1 (2004) stress-strain curve. The analysis is terminated externally when the central displacement exceeded 1 in. downward displacement. The slab specimen had a single layer of reinforcement at the bottom of the slab. While the test specimens used to develop

tension stiffening models typically used two layers of reinforcements. Therefore, use of these tension-stiffening models to model post cracking behavior of concrete in the slab specimen is inappropriate and attention should be provided to the reinforcement details to select an appropriate tension-stiffening model. In spite of that, the analyses were carried with different tension stiffening modeling to study their impact on the concrete slab response. Up to 1625 lb., all the tension stiffening models showed identical load displacement response. Beyond 1625 lb., all tension stiffening models over predicted the strength and stiffness of the slab in compression. The analysis with tension stiffening model by Vecchio and Collins (1986) attained the maximum load. From these results, it is clear that the tension stiffening models allow for better convergence but results in incorrect estimation of failure load and load vs. displacement response. Therefore, tension stiffening model should be carefully selected.

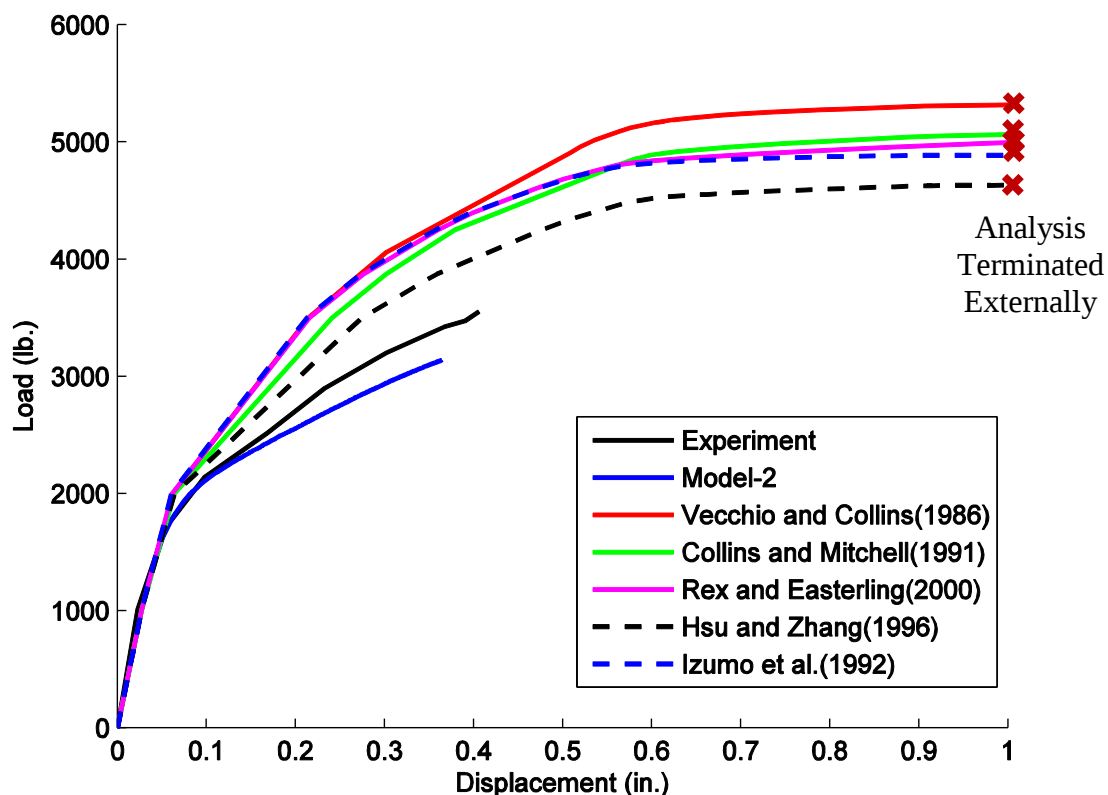


Figure 3-16: Load vs. displacement response at the center of slab for different tension stiffening models.

3.3.1.3 Concrete plasticity input parameters

In the following section, the sensitivity of input parameters for the smeared crack model and the damaged plasticity model for concrete plasticity is investigated by varying the input parameters over the typical ranges discussed previously in Section 3.2.2.1.5. The effect of failure ratios on the failure load is investigated by varying failure ratios within the ranges tabulated in Table 3-2 for Model-1 with a smeared crack model. Typical ranges for failure ratio-3 and failure ratio-4 are not found in the literature; therefore, default values of 1.28 and 1/3 set by ABAQUS (2013) are used in these analyses. Results are found to be insensitive to the failure ratio-1 (Table 3-11) indicating the absence of biaxial compressive stresses condition in the slab. In the Model-1 definition, a cracking stress of 459.8 psi is used resulting in a value of 0.0836 for failure ratio-2. Additional analyses are carried out by setting failure ratios-2 equal to 0.08, 0.09 and 0.10. An increase in failure ratio-2 showed an increase in failure load and displacement (Figure 3-17). A failure ratio-2 equal to 0.10 resulted in an increase in failure load and displacement at the center of the slab by 2.6% and 14.6% respectively compared to the experimental results. Analyses with failure ratios-2 equal to 0.08 and 0.09 failed before reaching the measured failure load of 3550 lb as a result of convergence difficulties.

Table 3-11: Impact of failure ratio on failure load and displacement.

	Failure load (lb.)	Displacement (in.)
Experimental	3550	0.41
Model-1	3251	0.39
Failure ratio 1(1.10)	3448	0.43
Failure ratio 1(1.13)	3448	0.43
Failure ratio 1(1.16)	3448	0.43
Failure ratio 2(0.08)	3172	0.37
Failure ratio 2(0.09)	3448	0.43
Failure ratio 2(0.10)	3643	0.47

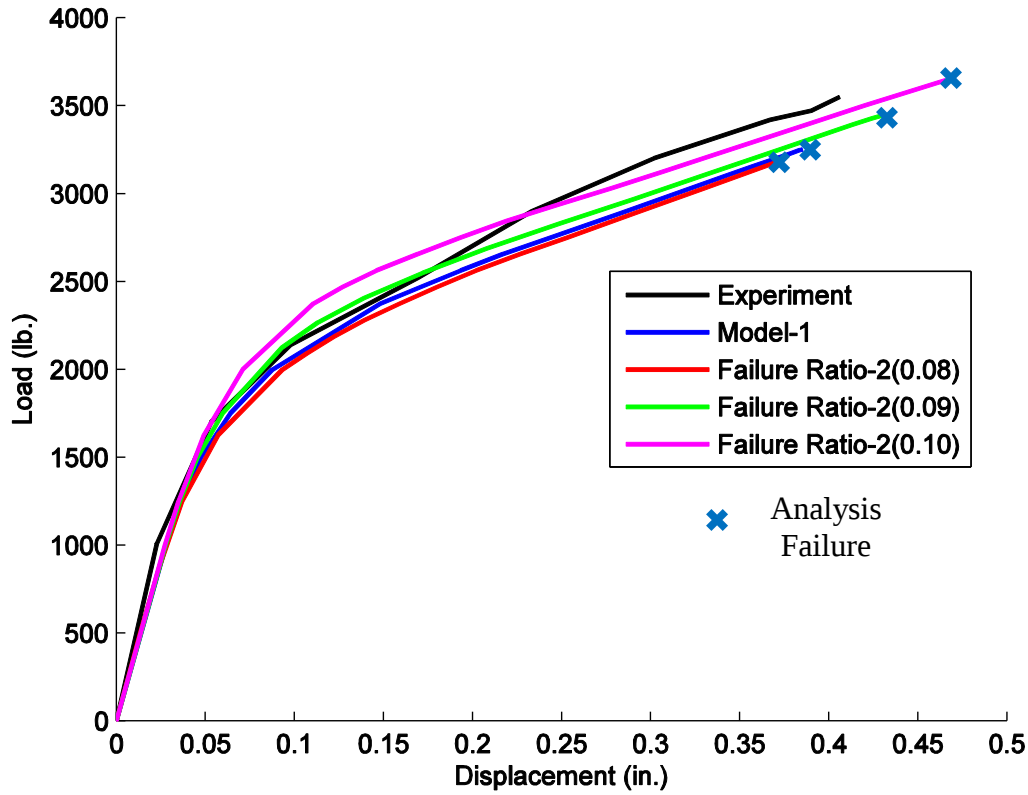


Figure 3-17: Load vs. displacement response at the center of slab for different values Failure Ratio-2.

In Model-2 with the damaged plasticity model for concrete plasticity, dilation angle of 32° is used based on the concrete strength of the slab. To evaluate effect of dilation angle on the overall response, the dilation angle is varied between 27° and 45° while keeping the concrete strength at 5.5 ksi. Results are found to be insensitive to the dilation angle (Table 3-12). Similar to Model-1, Model-2 is also reanalyzed with three different values for the ratio of ultimate biaxial compressive stress (f_{bo}) to ultimate uniaxial compressive stress (f_{co}). In addition, analyses are carried out with two eccentricity values (0.0 and 0.1) and four values for the parameter K_c . Results are found to be insensitive to all of these input parameters (Table 3-13).

Table 3-12: Impact of dilation angle on failure load and displacement.

Dilation Angle (ψ)	Failure load (lb.)	Displacement (in.)
27°	3116	0.361
32°	3126	0.363
37°	3137	0.364
40°	3144	0.365
42.5°	3149	0.366
45°	3156	0.367

Table 3-13: Impact of plasticity input parameters on failure load and displacement.

	Failure load (lb.)	Displacement (in.)
$f_{bo}/f_{co}=1.10$	3138	0.365
$f_{bo}/f_{co}=1.13$	3137	0.365
$f_{bo}/f_{co}=1.16$	3137	0.364
$\epsilon = 0.0$	3137	0.364
$\epsilon = 0.1$	3137	0.364
$K_c=0.50$	3132	0.363
$K_c=0.64$	3137	0.364
$K_c=0.67$	3137	0.364
$K_c=1.00$	3137	0.364

3.3.2 Composite plate girders (Mans, et al., 2001)

Mans, et al. (2001) tested two full-scale composite plate girders with HPS70W steel to investigate its strength and ductility in positive bending. Figure 3-18 through Figure 3-20 show the girder cross section and overall dimensions for the two girders labeled as POS1 and POS2. Both girders have a span length of 40 ft. and overall length of 42 ft. POS1 had a 60 in. wide by 7.125 in. thick concrete slab, and POS2 had 86 in. by 7.125 in concrete slab. As this research primarily focused on the flexural behavior of the specimens, Mans, et al. (2001) decided to include intermediate stiffeners to give a conservative shear strength controlled by elastic buckling. Shear studs $\frac{3}{4}$ in. in diameter by 4-1/2 in. in length were used in both specimens. Eighty pairs of shear studs were used on POS1 whereas sixty pairs were used on POS2. The concrete slab was reinforced with two layers of #4 Grade 60 reinforcement. POS1 had eight

longitudinal bars spaced at 8 in. center to center and transverse bars spaced at 8 in center to center. POS2 had eleven longitudinal bars spaced 8-1/4 in. center to center and transverse bars spaced 14 in center to center. A572 Grade 50 steel was used for transverse and bearing stiffeners. Both girders were tested by applying a point load at mid-span using a spreader beam placed perpendicular to the longitudinal axis of the composite girder. Girders were simply supported with ends braced against lateral movement. Test girder POS1 was laterally braced by attaching cables to the stiffeners at midpoint and quarter points on both sides of the girder. At mid-span, cables were attached close to both flanges whereas at quarter points only the top flange was braced. Test girder POS2 was braced using triangular braces placed at 3 ft. on either side of the mid-span. The bracing system was provided to prevent lateral movement of the deck and tension flange. Rollers were attached to the deck to allow composite girder to freely deflect downward during test. Material properties from Mans, et al. (2001) are tabulated in Table 3-14.

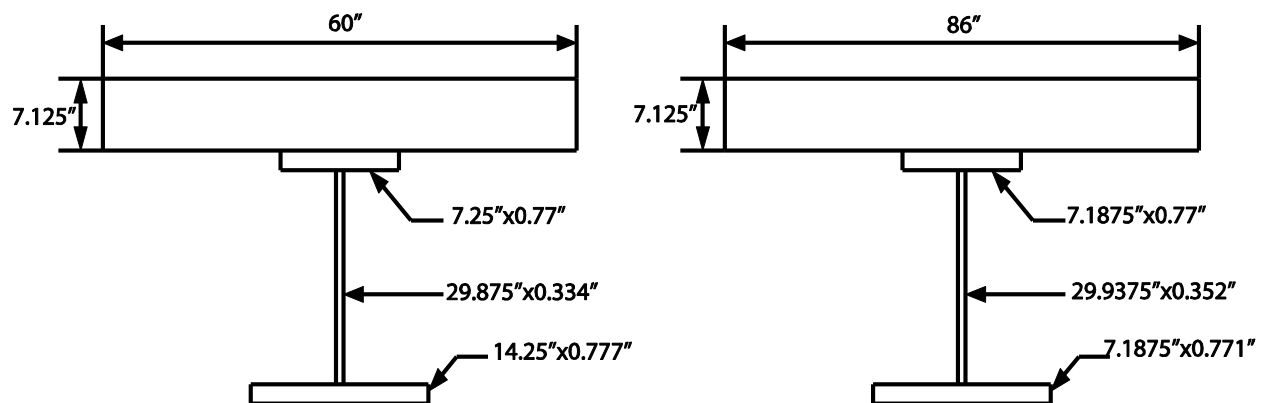


Figure 3-18: POS1 (Left) and POS2 (Right) cross section dimensions.

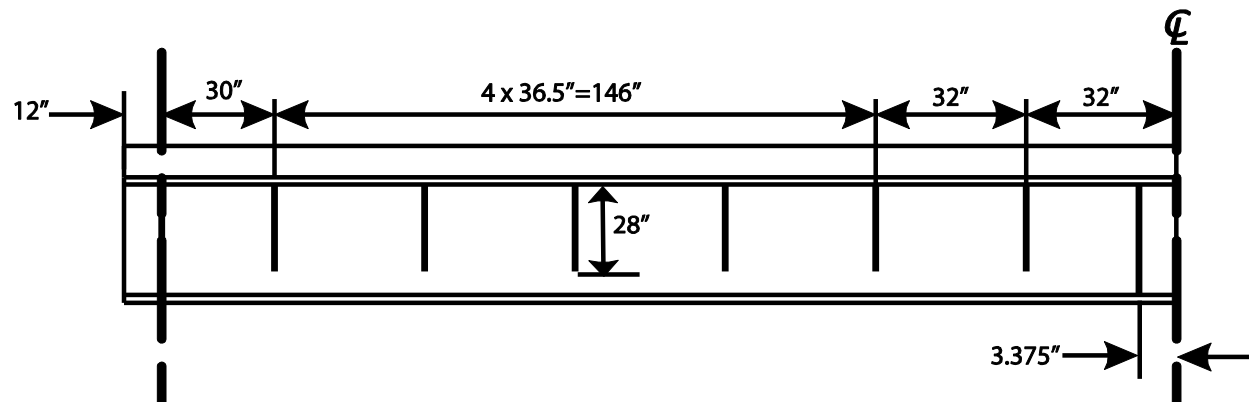


Figure 3-19: POS1 elevation.

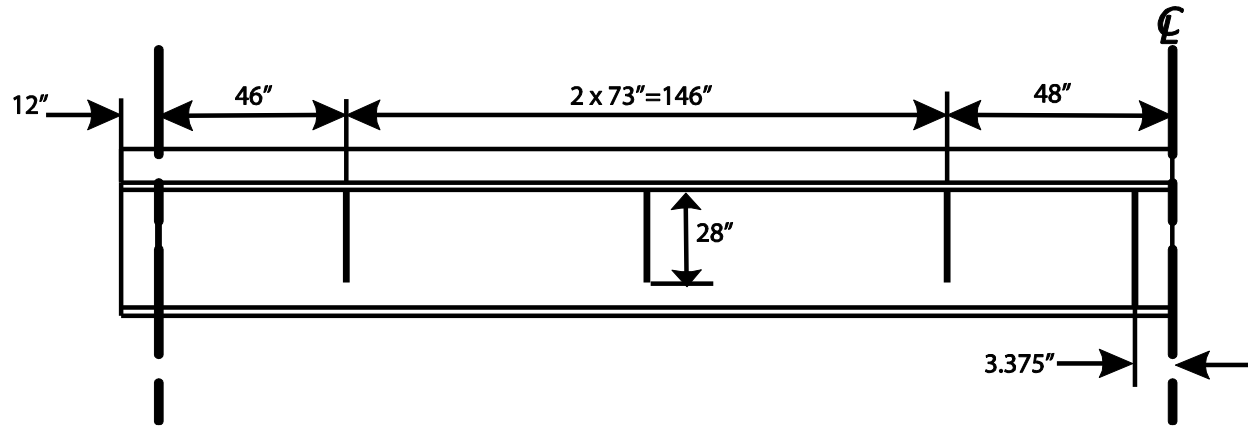


Figure 3-20: POS2 elevation.

Table 3-14: Material properties of POS1 and POS2 (Mans, et al., 2001) .

Material Properties	POS1	POS2
f_c'	4.423 ksi	7.617 ksi
f_y (Web)	84.55 ksi	
f_u (Web)	95.70 ksi	
f_y (Flange)	80.70 ksi	
f_u (Flange)	101.50 ksi	

As discussed in Section 3.2.1, the girder components (web, flange and transverse stiffeners) are modeled using general purpose four-node shell element S4R. The concrete slab is also modeled using S4R element. Longitudinal and transverse reinforcement is modeled using smeared rebar layers at the correct position through the slab thickness. Full composite action between the girder and slab is modeled using “Tie Constraints”. Boundary conditions are applied the same as the test setup. Figure 3-21 shows the finite element model of POS1. EN 1992-1-1(2004) uniaxial stress-strain relationship used to model the compressive behavior of concrete. Three-dimensional concrete plasticity is modeled using both the smeared crack model (POS1_SC or POS2_SC model) and damaged plasticity model (POS1_DP or POS2_DP model). Behavior of concrete in tension is defined using a tension stiffening model. The tension stiffening model for the preliminary analyses included a linear relationship up to cracking and post-cracking strength was degraded linearly and is reduced to zero at strain 10 times the strain at cracking. To model

Grade 60 reinforcement in the slab, the uniaxial stress-strain curve (Figure 3-7) used by Barth and Wu (2006) was implemented. HPS70W and A572 Grade 50 steels are modeled using the properties listed in Table 3-14 and typical structural steel parameters discussed in Section 3.2.2.2. Structural steel and reinforcement plasticity is modeled using classical metal plasticity model in ABAQUS (2013) that uses von-Mises yield criteria. In the damaged plasticity model, dilation angle is set equal to the angle of internal friction. Based on the relationship between the angle of internal friction and concrete strength (Nielsen and Linh, 2010) dilation angle values are selected for both test girders. Concrete plasticity parameters are summarized in Table 3-15. Analysis is carried out using Newton's (Newton-Raphson) method.

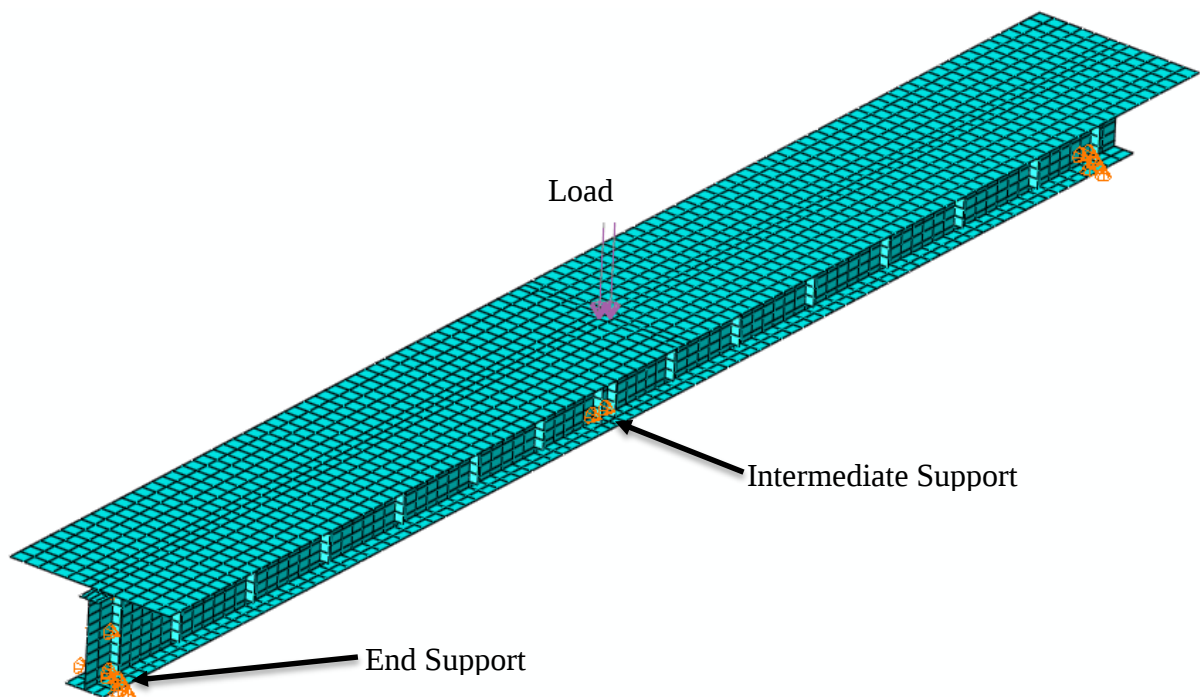


Figure 3-21: Finite element model of POS1.

Figure 3-22 and Figure 3-23 shows a comparison between the experimental and the analytical load vs. displacement response at mid-span for both girders. For both girders, analyses yielded higher load carrying capacity compared to the experimental response. The model with concrete plasticity defined using smeared crack model yielded higher failure load as well as displacement compared to model with damaged plasticity model. Crushing of concrete at mid-span results in convergence difficulties leading to failure of analyses for POS1 and POS2. Figure 3-24 shows the equivalent plastic strain (PEEQ) distribution for the POS1_DP model. Yielding is observed

on the bottom flange and on part of the web at the location of the applied load. The concrete slab yielded in compression from longitudinal bending stress. The higher analytical response can be ascribed to either the material plasticity definition or the use of tie constraints to modeling the composite action. To further understand their impact on the response POS1 girder is reanalyzed. Results from the parametric study are discussed in the following section.

Table 3-15: Concrete plasticity parameters for ABAQUS (2013).

Material Properties	POS1	POS2
Young's modulus	3791 ksi	4975 ksi
Density	$2.246 \times 10^{-4} \text{ lb s}^2/\text{in}^4$	$2.246 \times 10^{-4} \text{ lb s}^2/\text{in}^4$
Tension stiffening	10.5×10^{-4}	13.80×10^{-4}
Ratio of ultimate biaxial to uniaxial compressive stress	1.16	1.16
Poisson's ratio	0.2	0.2
Dilation angle (ψ)	33°	30°
K_c	0.67	0.67
Eccentricity (ϵ)	0.1	0.1
Cracking stress	442.3 psi	761.7 psi
Plastic strain at compression failure	0.003	0.003

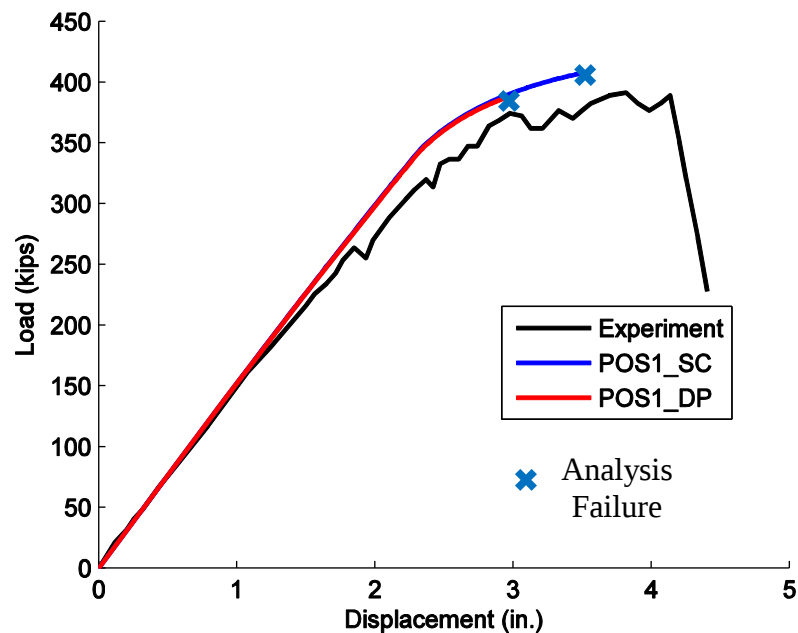


Figure 3-22: Load vs. mid-span displacement response for POS1.

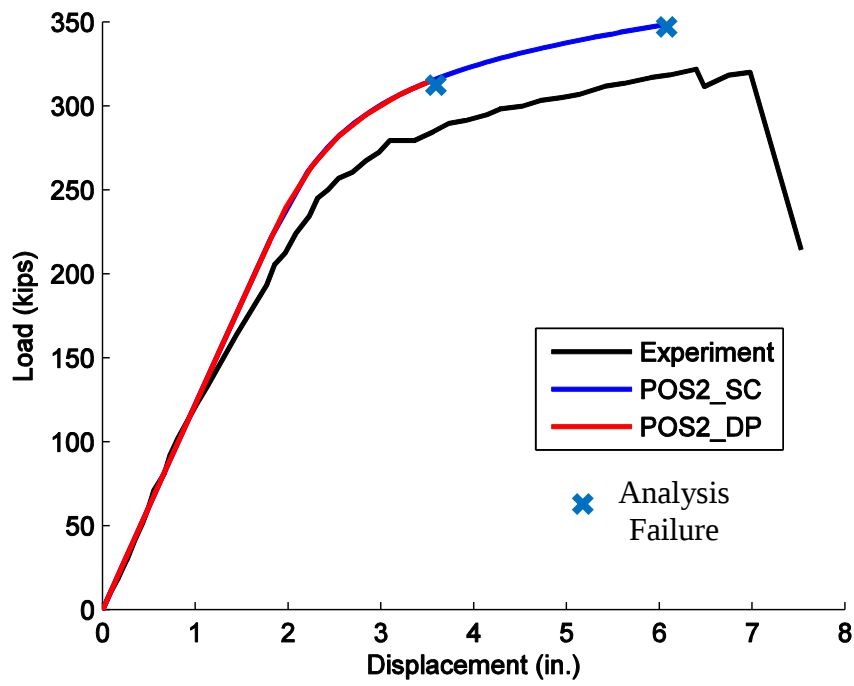


Figure 3-23: Load vs. mid-span displacement response for POS2.

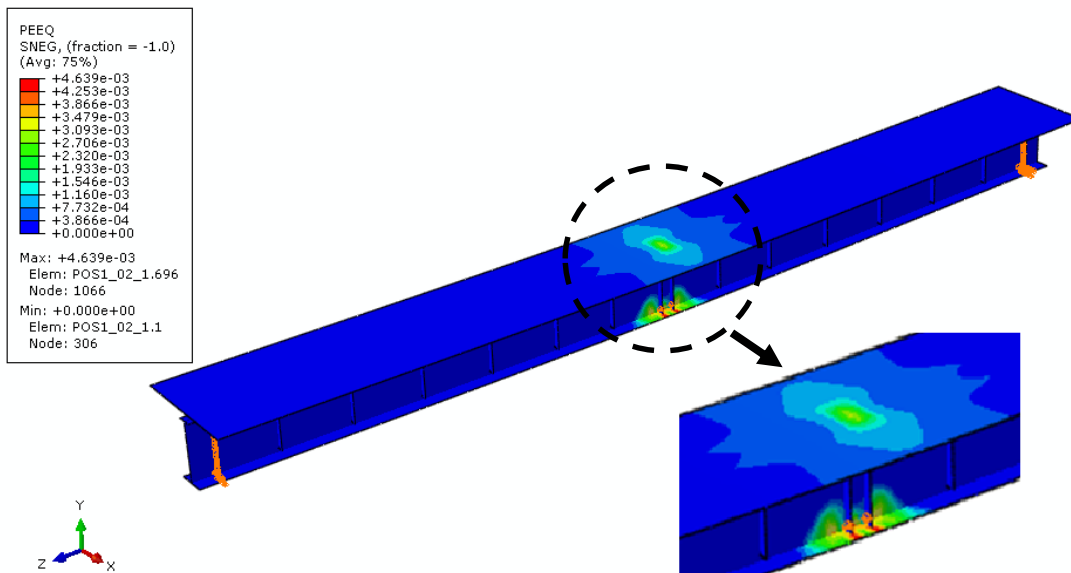


Figure 3-24: Equivalent plastic strain distribution (POS1_DP model).

For simply supported girders subjected to a point load at mid-span, the slab is under compression and use of different tension stiffening models will not affect the peak load-displacement. In spite of that, POS1_DP model is reanalyzed using the tension stiffening models discussed in Section 3.2.2.1.2. Table 3-16 shows that the peak load and mid-span displacement at peak load values are not affected by choice of tension stiffening model.

Table 3-16: Peak load and mid-span displacement at peak load for different tension stiffening models.

Model	Peak Load (kips)	Displacement (in.)
Experimental	391.05	3.82
POS1_DP	387.18	2.95
Vecchio and Collins(1986)	387.83	2.96
Hsu and Zhang(1996)	387.28	2.95
Rex and Easterling(2000)	389.87	3.02
Izumo et al.(1992)	387.83	2.97
Collins and Mitchell(1991)	387.59	2.96

3.3.2.1 Concrete plasticity input parameters

In the following section, the sensitivity of plasticity input parameters for smeared crack model (POS1_SC) and damaged plasticity model (POS1_DP) is investigated by varying the input parameters over the typical ranges discussed previously in Section 3.2.2.1.5. For POS1_SC model, the effect of failure ratios on failure load is investigated by varying failure ratios within the ranges tabulated in Table 3-2. Typical ranges for failure ratio-3 and failure ratio-4 were not found in the literature; therefore, default values (ABAQUS, 2013) of 1.28 and 1/3 are used respectively. The results are found to be insensitive to failure ratio-1 (Table 3-11) indicating absence of biaxial compressive stresses condition in the slab. POS1_SC model is reanalyzed with three values for failure ratios-2. The results are found to be insensitive to failure ratio-2 because predominant forces in concrete slab are compressive.

Table 3-17: Impact of failure ratio on peak load and mid-span displacement at peak load.

	Peak Load (kips)	Displacement (in.)
Experimental	391.05	3.82
POS1_SC	407.73	3.52
Failure ratio 1(1.10)	403.88	3.38
Failure ratio 1(1.13)	406.68	3.49
Failure ratio 1(1.16)	407.73	3.52
Failure ratio 2(0.08)	407.13	3.50
Failure ratio 2(0.09)	407.73	3.52
Failure ratio 2(0.10)	407.58	3.51

In the POS1_DP model, a dilation angle of 33° is used based on the concrete strength of the slab. To evaluate sensitivity to dilation angle, the dilation angle is varied between 27° and 45° while keeping the concrete strength at 4.423 ksi. Results are found to be insensitive to the dilation angle (Table 3-18). Similar to the POS1_SC model, the POS1_DP model is analyzed with three different values for the ratio of the ultimate biaxial compressive stress (f_{bo}) to the ultimate uniaxial compressive stress (f_{co}). In addition, analyses is carried out for two eccentricity values (0.0 and 0.1) and four values for parameter K_c . The results are found to be insensitive to all of these input parameters (Table 3-19).

Table 3-18: Impact of dilation angle on peak load and mid-span displacement at peak load.

Dilation Angle (ψ)	Peak Load (kips)	Displacement (in.)
Experimental	391.05	3.82
27°	386.72	2.95
33°	387.18	2.95
37°	387.21	2.95
40°	387.08	2.95
45°	386.53	2.93

Table 3-19: Impact of plasticity input parameters on peak load and mid-span displacement at peak load.

	Peak Load (kips)	Displacement (in.)
Experimental	391.05	3.82
$f_{bo}/f_{co}=1.10$	384.82	2.91
$f_{bo}/f_{co}=1.13$	386.18	2.94
$f_{bo}/f_{co}=1.16$	387.18	2.95
$\epsilon = 0.0$	387.18	2.95
$\epsilon = 0.1$	387.18	2.95
$K_c=0.64$	387.38	2.96
$K_c=0.67$	387.18	2.95
$K_c=1.00$	387.06	2.95

3.3.2.2 Modeling of full composite action

Until this point, the composite action between the concrete slab and steel girder is modeled using tie constraint (Tie constraint model). At lower load levels, this method would yield response similar to the experimental response. At higher loads excluding interface slip and interface stiffness degradation would result in stiffer response as seen in Figure 3-22 and Figure 3-23. To understand the contribution of composite action to the load vs. displacement response POS1_DP model is reanalyzed. In the first method (Beam connector model), composite action is modeled using beam connector element. Connector elements connect nodes at the girder flange to the web intersection with the corresponding slab nodes above. This method is similar to the tie constraint and allows for no slip at the interface. The tie constraint model ties top surface of the girder top flange to the bottom surface of the slab right above the top flange while in this method composite action is imposed using discrete connector elements along one line of nodes. In the second method (Three beam connector model), three beam connector elements were used to connect the two nodes at the flange tips and the node at the flange to web intersection with the corresponding slab nodes. In the third method (Rigid stud model), beam connector elements are spaced at an interval similar to the stud interval used during testing for POS1. In the fourth method (Shear stud model), composite action at the interface is modeled using the empirical load vs. slip relationship proposed by Ollgaard et al.(1971). Interface behavior is modeled using Beam and

(Radial-Thrust+Align) connector elements in series. Further details about this model are provided in Section 3.2.4.1. Figure 3-25 shows the comparison between the experimental and the analytical load vs. displacement response for the four models discussed above. The tie constraint model yielded stiffer response compared to all other models. The use of beam connectors yielded flexible response compared to tie constraints but stiffer response compared to the rigid stud model and shear stud model. The rigid stud model yielded response identical to the experimental response up to load of 263.5 kips. Beyond 263.5 kip load analysis yielded stiffer response. Shear stud model's initial response is softer compared to the experimental response. But, it matches the response curve beyond 270 kips better compared to other models. The maximum forces in the shear stud at the end of analysis is 16.75 kips which is lower compared to the capacity of shear stud (27.20 kips) predicted using Equation 3-76. The maximum force in the shear stud (16.75 kips) corresponds to slip of 0.02 in. which is lower compared to the maximum slip (0.4 in.) used in the connector plasticity definition ensuring validity of the analysis. The variability in response comes from flexibility at the steel-concrete interface with different composite action modeling methods.

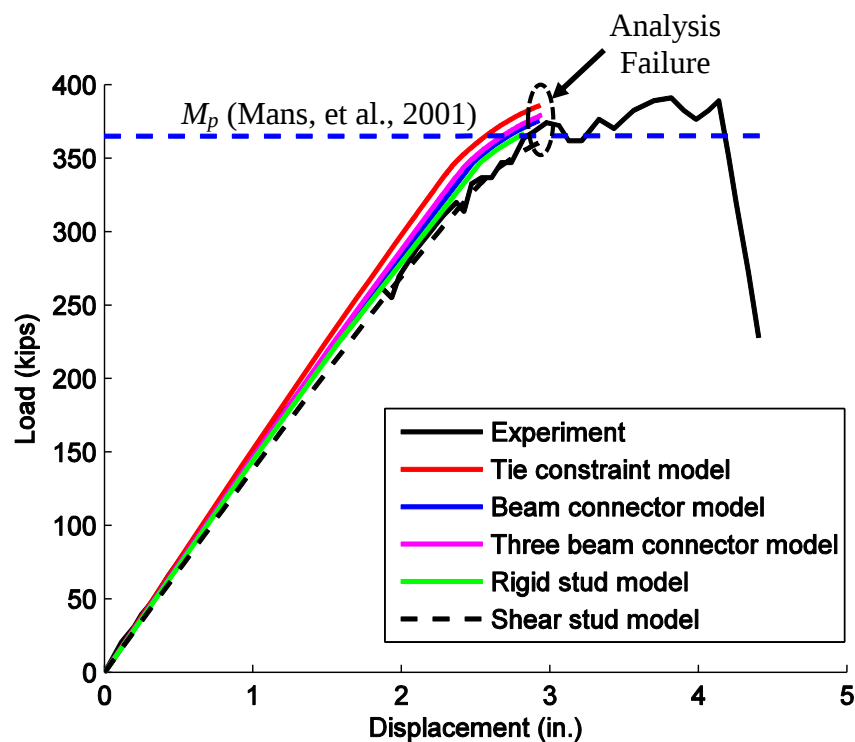


Figure 3-25: Effect of composite action modeling methods on load vs. mid-span displacement response for POS1.

Comparison of different composite action modeling techniques indicates that it is important to model the composite action at the steel-concrete interface to capture the realistic load-displacement response. To further understand its impact, both composite girders are reanalyzed with the empirical load vs. slip relationship proposed by Ollgaard et al. (1971) to model the shear studs behavior. Figure 3-26 and Figure 3-27 shows the improvement in response with inclusion of shear stud interaction at the steel-concrete interface. For both girders, inclusion of shear stud interaction yielded load vs. mid-span displacement response closer to the experimental response.

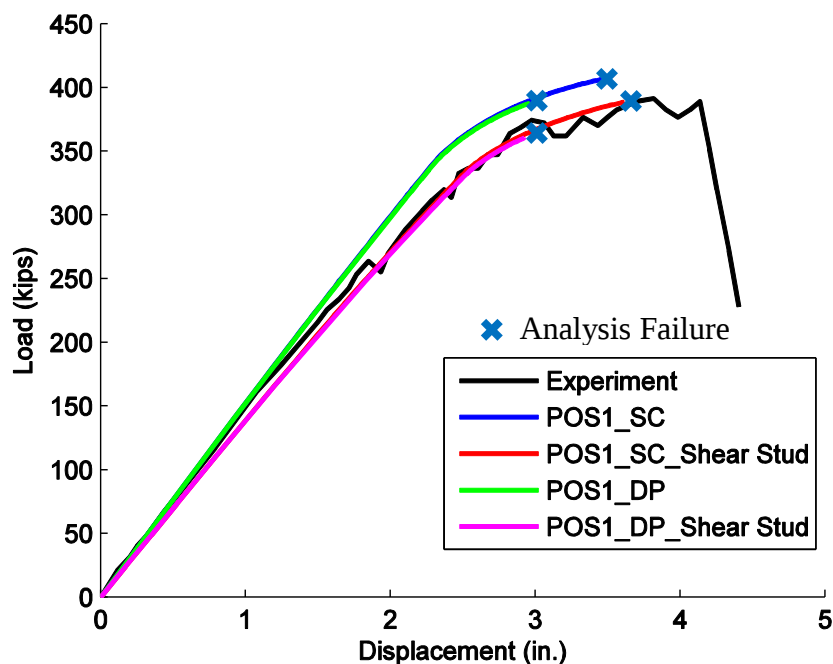


Figure 3-26: Impact of shear stud model on load vs. mid-span displacement response for POS1.

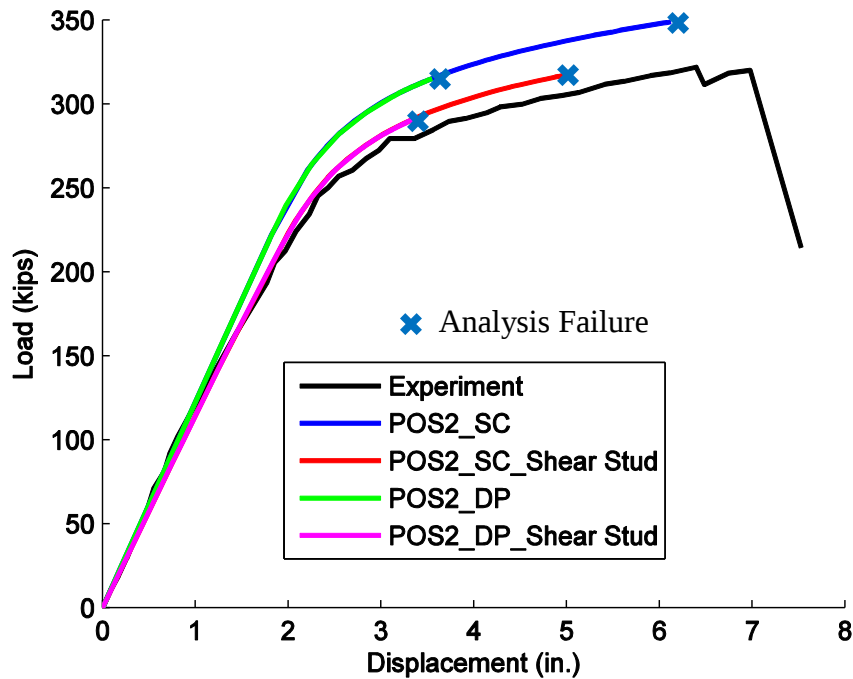


Figure 3-27: Impact of shear stud model on load vs. mid-span displacement response for POS2.

3.3.2.3 Solution techniques

There are varieties of solution techniques available in ABAQUS (2013) to solve nonlinear problems. Selection of a solution technique and solution parameters can have significant impact on the accuracy of the solution. In the following section, four solution techniques to obtain a static or quasi-static are evaluated by reanalyzing the POS1_DP_Shear Stud model.

1. *Newton's method*: The analysis is carried using *Newton's method* or *Newton-Raphson method*.
2. *Newton's method with automatic stabilization*: In case of local instabilities arising from buckling or material yielding, there will be a local transfer of strain energy from one part of the model to the neighboring parts. The *Newton's method* may not work in that case and therefore ABAQUS provides automatic stabilization process (ABAQUS, 2013). Automatic stabilization process stabilizes unstable quasi-static problem through automatic addition of volume proportional damping. A default value of 2.0×10^{-4} percentage of the strain energy is dissipated through damping. When automatic stabilization is used, it is necessary to check the viscous damping energy (ALLSD) with the total strain energy (ALLIE), and ensure that

the ratio does not exceed the dissipated energy fraction or any reasonable amount. The viscous forces (VF) can also be compared with the overall forces in the analysis to ensure that they are relatively small.

3. *Modified Riks method*: *Modified Riks method* uses the arc length method and load magnitude as an additional variable along with the displacement. This method solves for both unknowns simultaneously.
4. *Implicit quasi-static analysis*: *Implicit quasi-static analysis* technique available in *ABAQUS/Standard* uses backward Euler operator for time integration. A backward Euler operator introduces numerical damping in the solution allowing users to obtain a Quasi-static solution using an *Implicit dynamic scheme*. When a Quasi-static solution scheme is used, it is necessary to examine that the kinetic energy (ALLKE) of the model does not exceed a small fraction (typically 1-5%) of its internal energy (ALLIE).

Figure 3-28 shows the response obtained using different solvers for composite girder POS1. Up to a load of 320 kips all five-solution techniques yielded similar load vs. displacement response. Newton's method with and without automatic stabilization yielded peak load of ~360 kips with maximum downward displacement of ~2.90 in. The ratio of viscous damping energy (ALLSD) to the total strain energy (ALLIE) doesn't exceed 1% (Figure 3-29) ensuring validity of the analysis. After 320 kips, load-displacement curve obtained using Riks method flattened and terminated at a displacement of 2.91 in. With quasi-static solver, analysis continued and matched the experimental reasonably well. The solution with a quasi-static analysis continued well beyond the experimental response. Response up to a vertical downward displacement of 5 in. is plotted in Figure 3-29. The analysis is terminated externally when the vertical downward displacement reached 5 in. Figure 3-30 shows that the ratio of kinetic energy (ALLKE) to the internal energy (ALLIE) of the model does not exceed 5%. Therefore, solution can be considered as quasi-static. Out of four solution techniques, quasi-static solution method is able to continue and get converged solution for higher loads compared to other solution techniques. During testing once girder reached the maximum load, two more load stages were recorded that registered lower loads because of concrete failure at mid-span (Mans, et al., 2001). Although it is not mentioned explicitly, the testing was probably carried out using displacement control protocol while the analysis is performed as load control. With the load control analysis simulation failed to capture the descending branch of the load vs. displacement curve.

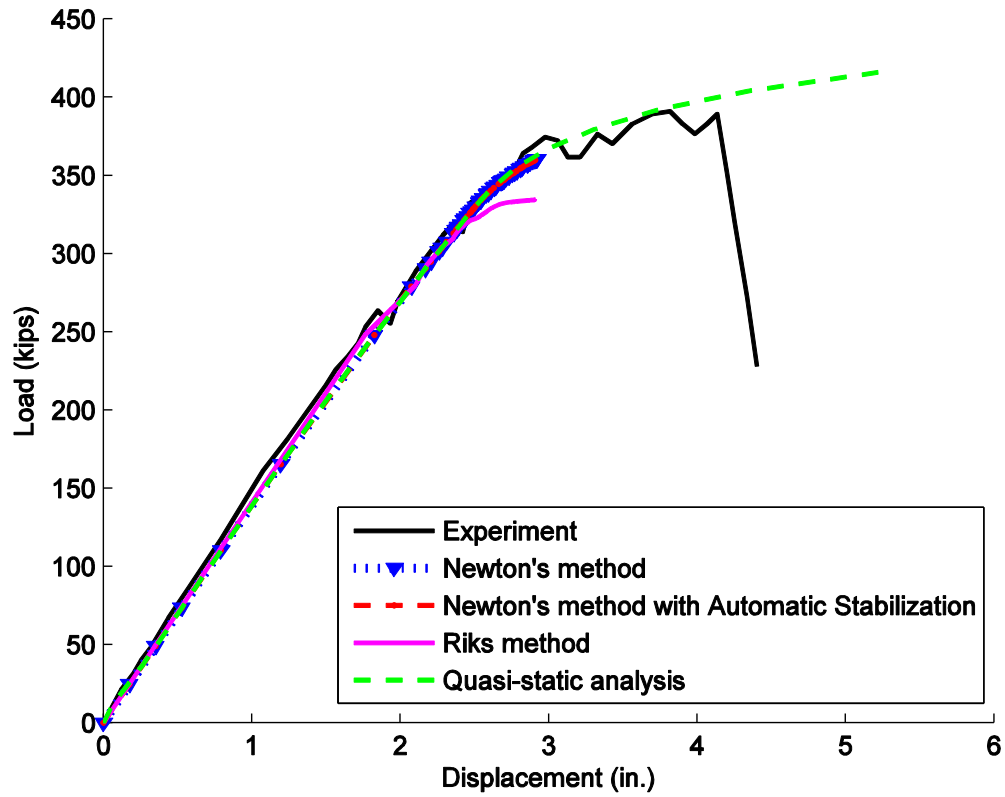


Figure 3-28: Solver comparison (POS1).

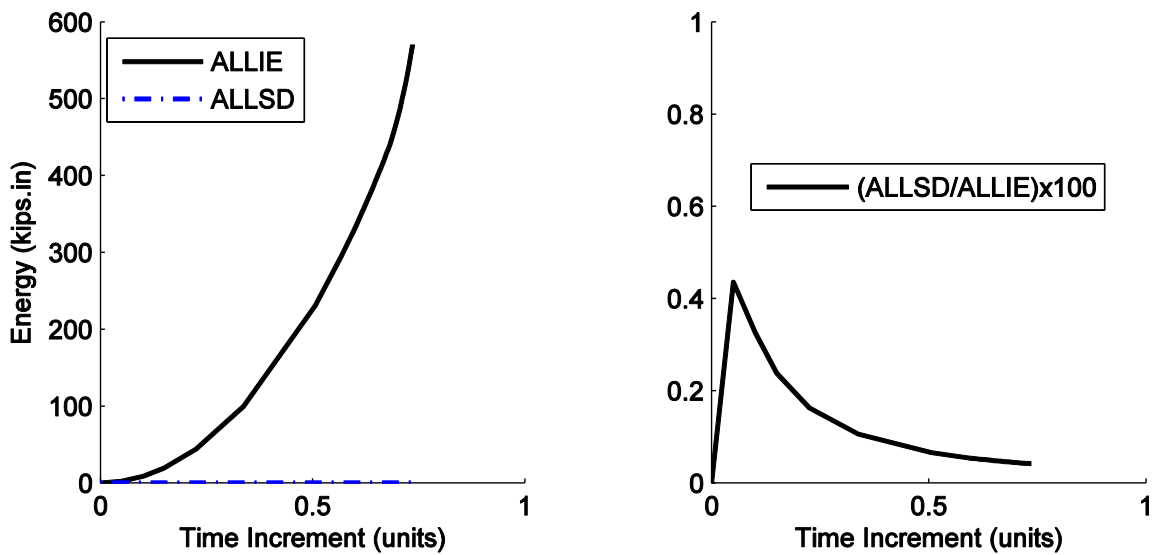


Figure 3-29: Viscous damping energy (ALLSD) and total strain energy (ALLIE) comparison.

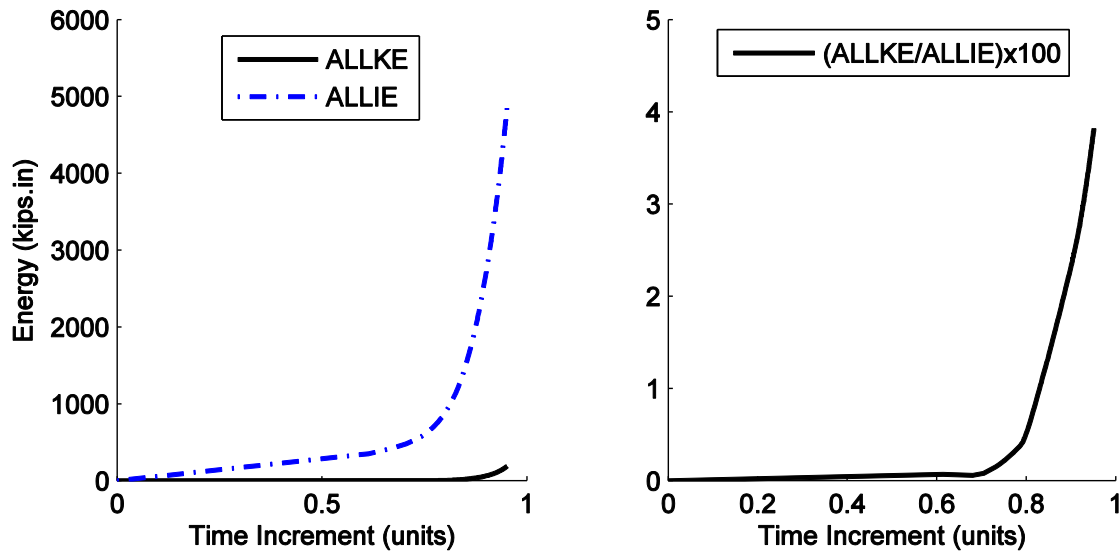


Figure 3-30: Kinetic energy (ALLKE) and internal energy (ALLIE) comparison.

3.3.2.4 Modeling of non-composite girders

To evaluate modeling options to model non-composite action at the steel girder and concrete deck interface, composite girder POS1 tested by Mans et al. (2001) is modeled as a non-composite girder. Non-composite action at the interface is modeled using the linear and nonlinear penalty option in ABAQUS (2013). Based on the recommendation by Rabbat and Russell (1985) the coefficient of static friction at the steel and concrete interface is taken as 0.57. The analysis is carried out using Newton's method with automatic stabilization. Ratio of viscous damping energy (ALLSD) to total strain energy (ALLIE) is found to be less than 1% ensuring validity of the analysis. The boundary conditions for the non-composite girder are same as the ones used during testing of composite girder POS1 by Mans et al. (2001). The analysis is carried out in two steps. In the first step, only self-weight is applied to the entire model and in the next step the point load is applied at mid-span. Application of self-weight in the first step establishes contact between the bottom of slab and the top of top flange. The non-composite girder yielded higher peak load compared to the bare steel girder but showed lower mid-span displacement. The linear penalty method yielded only 3.1% higher peak load compared to the nonlinear penalty method. The linear penalty method showed more flexible response, with mid-span displacement 19.7% higher compared to the nonlinear penalty method.

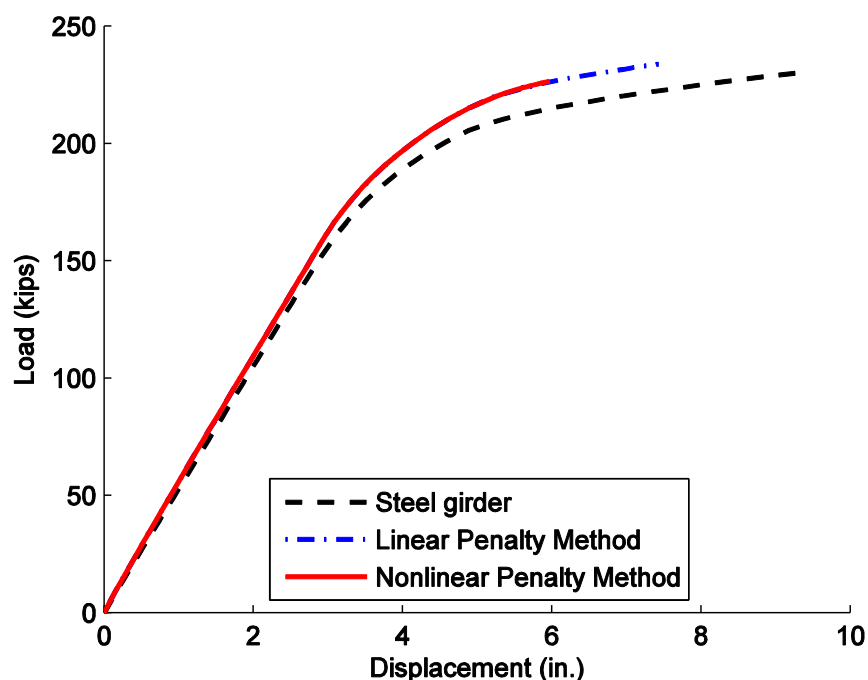


Figure 3-31: Non-composite girder response using linear and nonlinear penalty contact method.

Table 3-20: Peak load and mid-span displacement for different penalty contact method.

	Peak Load (kips)	Displacement (in.)
Steel girder only	230.29	9.37
Linear Penalty Method	233.68	7.43
Nonlinear Penalty Method	226.45	5.97

To understand the impact of interface friction on the load-displacement response, analyses are carried out for four values of the coefficient of static friction. The peak load did not show significant change for different values of the coefficient of static friction. Mid-span displacement varied but did not show any particular trend for different values of the coefficient of static friction. In an actual non-composite bridge with multiple girders and secondary members, the static friction coefficient value can possibly have more of an impact on the response.

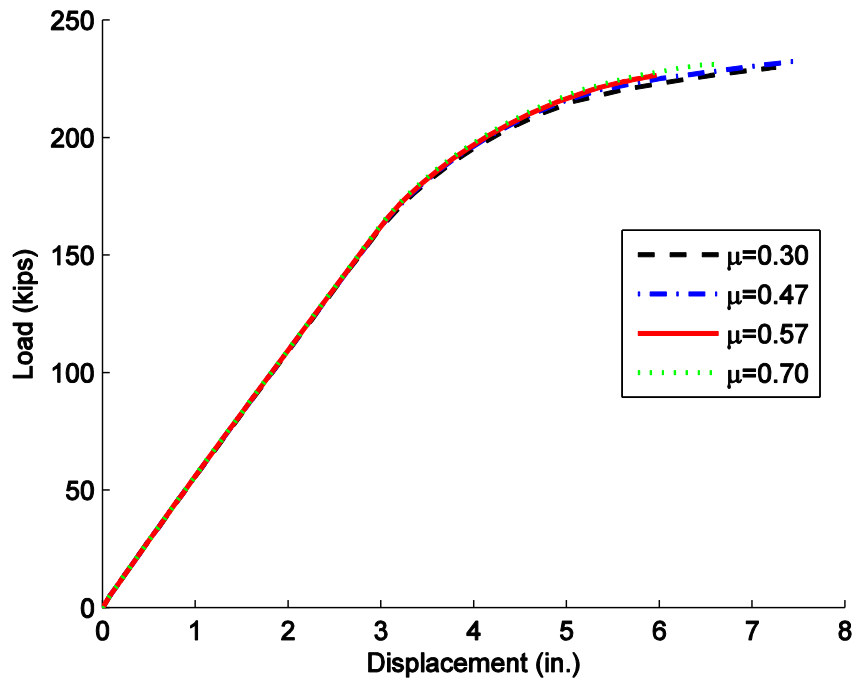


Figure 3-32: Effect of coefficient of static friction on load-displacement response of non-composite girder.

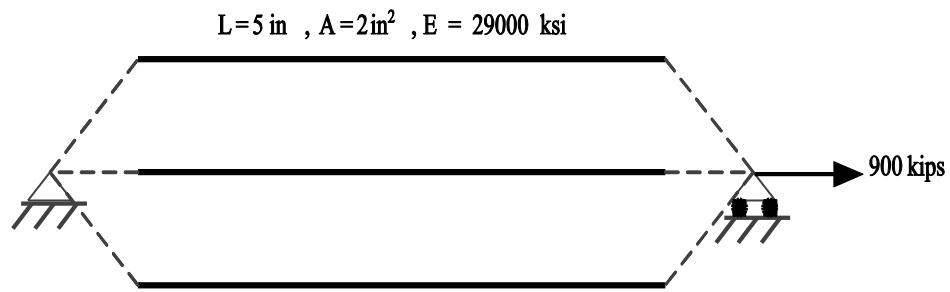
Table 3-21: Peak load and mid-span displacement for different coefficient of static friction.

μ	Peak Load (kips)	Displacement (in.)
0.30	229.87	7.25
0.47	232.39	7.46
0.57	226.45	5.97
0.70	231.13	6.58

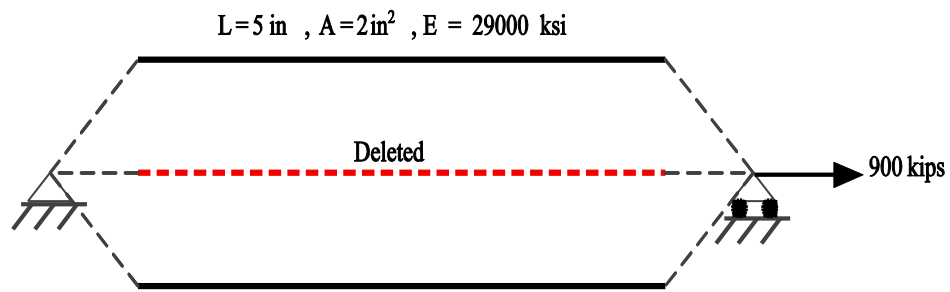
3.3.3 Verification of “*Model Change*” option to simulate fracture in ABAQUS

In this research, the “*Model Change*” option in ABAQUS (2013) is used to simulate a fracture event. As described in Section 3.2.5, the “*Model Change*” option stores the forces in the region prior to removal of connector elements and ramps down forces to zero over the step to simulate element removal. To verify the capability of this feature to simulate fracture or element deletion, a three bar model (Figure 3-33) subjected to axial load is used. Properties of the three bar model are tabulated in Table 3-22. In the first step, 900 kips load is applied to three truss bars resulting

in an axial deformation ($\delta = \frac{900 \text{ kips}}{3K_{bar}}$) of 25.86×10^{-3} in. The axial strain energy ($U_{axial} = \frac{P^2 L}{2AE}$) stored in each member is 3.879 kip-in. At end of step 1, internal work (ALLIE) is equal to external work (ALLWK) and is equal to 11.638 kip-in. In the second step, the interior truss bar is removed using “*Model Change*” option. Removal of the interior truss bar results in an instantaneous drop in axial strain energy of the system by 3.879 kip-in. Additional work ($W = \frac{1}{2} F \delta$) of 1.94 kip-in. is done to ramp down the internal force (300 kips) in the removed member to zero. At the end of step 2, two truss bars carry the 900 kips load resulting in additional displacement of 12.93×10^{-3} in. The total axial strain energy (ALLIE) stored in two truss bar is 17.457 kip-in. At the end of step 2, the external work (ALLWK) is equal to the sum of total axial strain energy stored in two truss bars (17.457 kip-in), strain energy stored in interior truss bar at end of step 1 (3.879 kip-in) and work done to ramp down the internal force in the removed member to zero (1.94 kip-in.). These hand calculation results are compared with the ABAQUS (2013) output in Table 3-23 and the difference is less than 2%. Figure 3-34 shows energy distribution recorded during analysis in ABAQUS (2013). These results imply that the “*Model Change*” option in ABAQUS (2013) is capable of simulating element removal or fracture event. Section 4.3.2 of the Hoan Bridge Study chapter provides discussion about the introduction of fracture in a dynamic analysis using ABAQUS/Standard and ABAQUS/Explicit.



a) Initial geometry



b) Removal of bar using “*Model Change*”

Figure 3-33: Three bar model.

Table 3-22: Three bar model properties.

Area per bar (A_{bar})	2 in^2
Modulus of elasticity (E_{bar})	29000 ksi
Length (L)	5 in.
Elastic stiffness (K_{bar})	$11.6 \times 10^3 \text{ kip/in.}$

Table 3-23: Comparison between hand calculation and ABAQUS (2013).

	Output	Hand calculation	ABAQUS
Step 1	Axial strain energy per truss bar	3.879 kip-in	3.920 kip-in (1.05%)
	Deformation	0.02586 in	0.0261 in (0.79%)
Step 2	Axial strain energy per truss bar	17.457 kip-in	17.732 kip-in (1.58%)
	Deformation	0.038 in.	0.039 in. (1.19%)
	External work done (ALLWK)	23.276 kip-in	23.627 kip-in (1.51%)

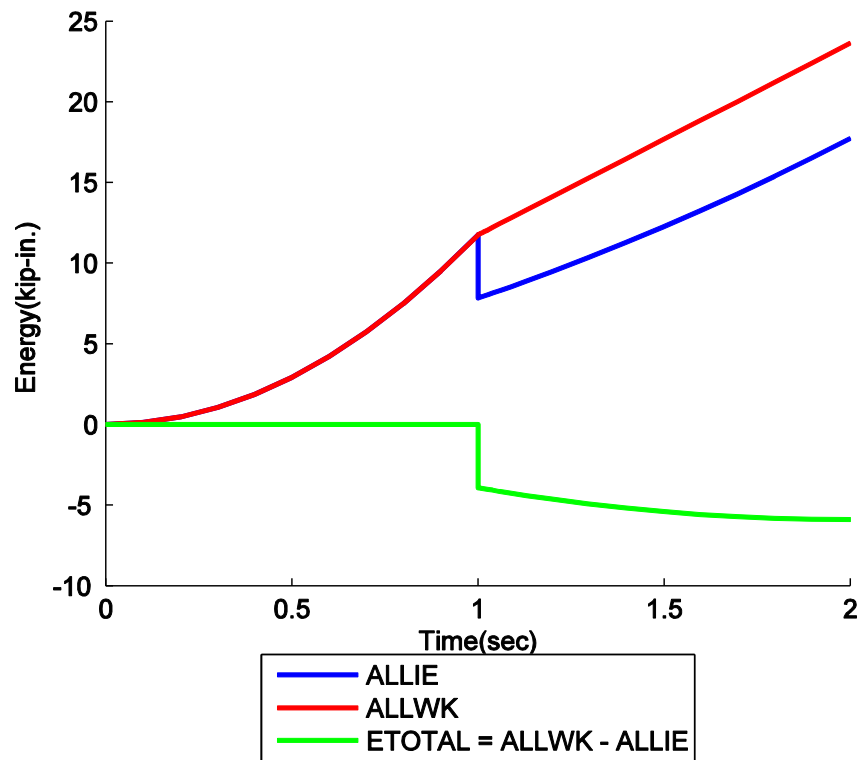


Figure 3-34: Energy distribution from ABAQUS (2013).

3.4 Conclusion

This chapter summarizes the modeling methodologies that will be used in the current research followed by verification through comparing with the experimental data. Examination of all the results leads to following conclusions:

1. The smeared crack model is suitable for concrete subjected to monotonic strain under low confining pressures, while the damaged plasticity model is suitable for concrete under monotonic, cyclic and dynamic loading under low confining pressures. Therefore, the damaged plasticity model is selected to model the plasticity in bridge slabs.
2. The EN 1992-1-1 (2004) model is selected to model uniaxial compressive behavior of concrete.
3. The tension stiffness relationship proposed by Collins and Mitchell (1991) is selected to model the post-cracking behavior of concrete because it gave an average response compared to all other tension stiffening models.
4. Analyses of two-way slab and composite girders showed that the impact of plasticity input parameters depends on the failure mode and therefore should be carefully considered based on the expected failure mode.
5. Modeling of composite action is found to be crucial to capture the experimental load-displacement response and therefore interface interaction will be included in the future analyses involving behavior of composite girders beyond elastic range.
6. In the event of fracture, shear studs are subjected to combined tension and shear near fractured region making it necessary to account for shear-tension interaction in the finite element model. The PCI Design Handbook, 6th Edition (2004) equation will be used model the shear-tension interaction in the absence of test data.
7. The contact definition in ABAQUS (2013) is selected to model the non-composite action at the steel-concrete interface. It is capable of simulating both the normal and tangential behavior at the interface.
8. The three bar model in Section 3.3.3 validated the capability of “Model Change” option to simulate element removal or fracture in ABAQUS (2013). The modeling methodology discussed here will be used in the evaluation of post fracture redundancy of steel bridges in the subsequent chapters.

4 Hoan Bridge Study

4.1 Introduction

This chapter describes one of the two cases that are used as the primary case studies in this research. The Hoan Bridge is located on I-794 over the Milwaukee River in the city of Milwaukee, Wisconsin. The bridge was opened for traffic in 1974 and carried three lanes of traffic in each direction to and from the city. The main span is a three span tied arch crossing the river. The main span has three span continuous three girder approach spans on both sides of arch.

The south approach spans, shown in Figure 4-1, consist of two parallel, three-span continuous plate girder systems with a non-composite concrete deck slab. A cross section of the bridge is shown in Figure 4-2. The structural system consists of three 10 ft deep plate girders spaced at 24 ft - 6 in. and connected with floor beams supporting intermediate stringers under the slab. On December 13th, 2000, bottom flange and full web depth fractures were found in two out of the three steel girders (Figure 4-3 and Figure 4-4). Visual inspection of the northbound bridge by WisDOT revealed a depressed area approximately 4 ft deep by 25 ft long, by about 50 ft wide across the roadway. The depression sloped out toward the outside lane. The fracture opened up several inches resulting in a kink in the girder. There was a separation between the non-composite concrete deck and steel girders. Localized concrete crushing in the deck and parapet was also observed. Partial depth web fractures were also found at three locations in Girder D as shown in Figure 4-3. Stringers are omitted from Figure 4-3 for clarity. Figure 4-4 shows the fracture in the exterior girder F and the center girder E. Figure 4-5 through Figure 4-7 shows close up of the fracture in girder D through F.

The fracture occurred around 4 AM when the ambient temperature was about -11°F and the traffic on the bridge was sparse (Fisher, et al., 2001). The load that caused the fracture is unknown but it was estimated to be a tractor-trailer salt truck with a weight of approximately 100 kips. This estimate is based on the weigh-in-motion study that was carried out following the collapse by University of Michigan (Fisher, et al., 2001). Because the large deflection was readily apparent to motorists, authorities were alerted almost immediately and the bridge was closed quickly. It is not known if any truck loads crossed the bridge in the damaged condition.

The entire roadway was immediately closed to traffic and on December 28, the damaged section was removed by explosive demolition to allow for rapid reconstruction.

All three girder web fractures initiated from high triaxial stresses at the crack-like geometric condition at the intersection of the gusset plate and transverse connection plate welded connections with intersecting and overlapping welds. Figure 4-8 shows the schematic of the connection detail at lateral gusset plate with the transverse connection plate. Investigation by Fisher et al. (2001) revealed brittle fractures at the web crack without any detectable fatigue crack extension or ductile tearing at the crack origin. All of the flange and web steel satisfied the mechanical properties and Charpy V-Notch toughness that satisfied the AASHTO requirements at the time the structure was built (Connor, et al., 2007). Investigation by Fisher et al. (2001) also revealed that plates had sufficient toughness to tolerate through plate thickness cracks under normal condition without the high constraint condition. Fisher et al. (2001) provide detailed information about the findings from the forensic investigation of the Hoan Bridge.

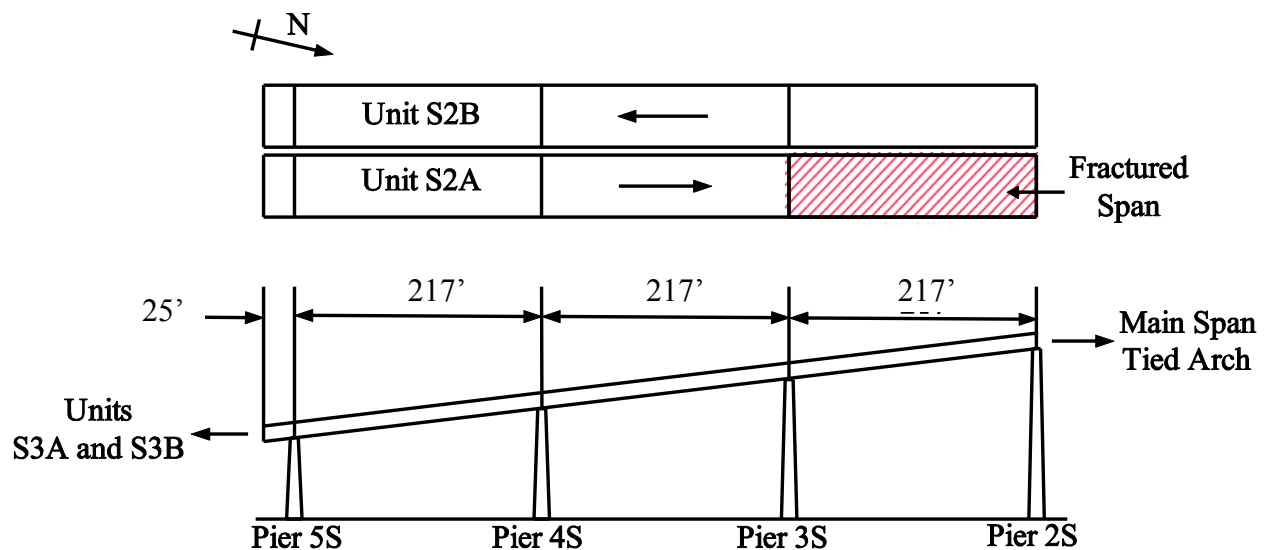


Figure 4-1: South approach Unit S2A showing location of fractured span (Not to scale).

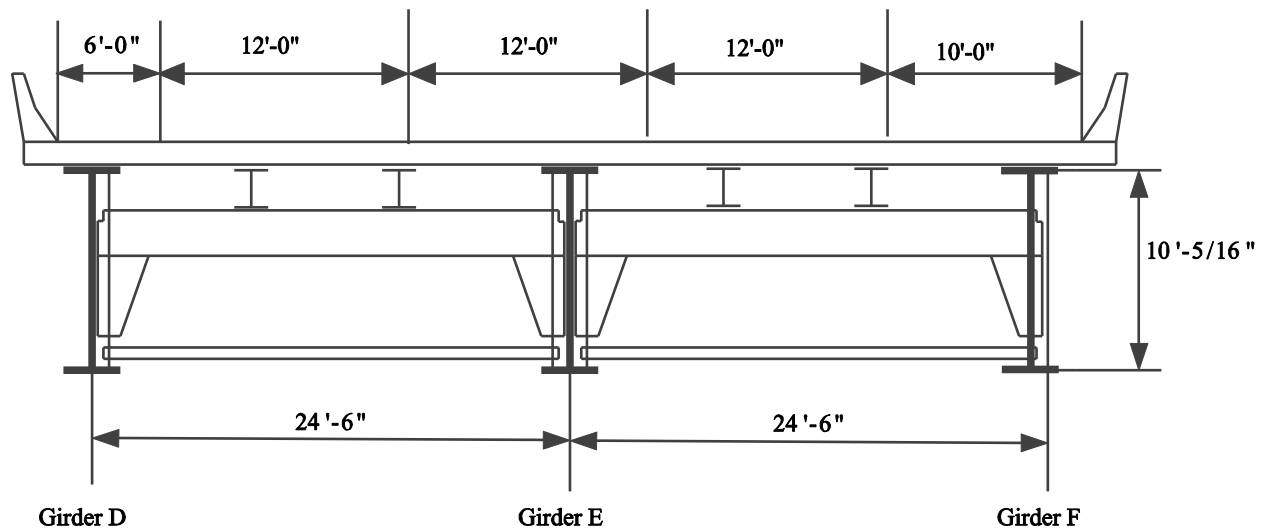


Figure 4-2: Typical Hoan bridge cross-section (Not to scale).

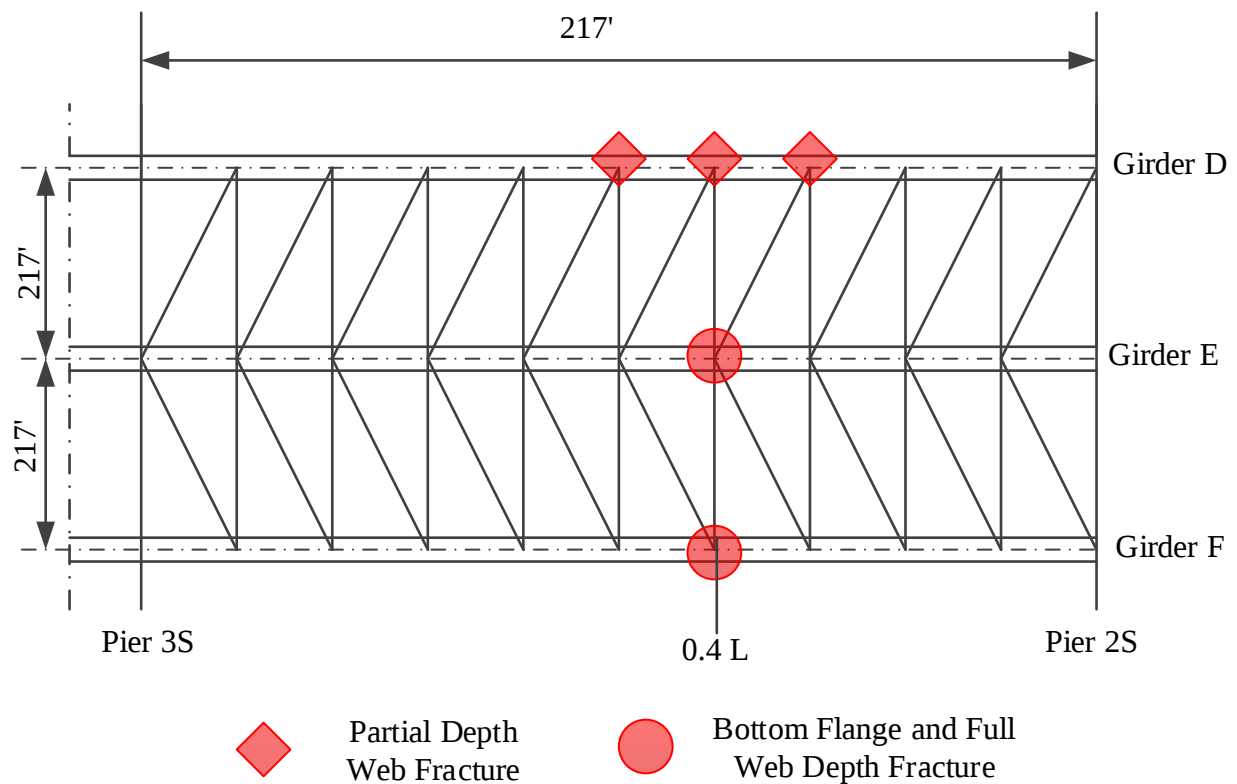


Figure 4-3: Framing plan and elevation of fractured span (Not to scale).

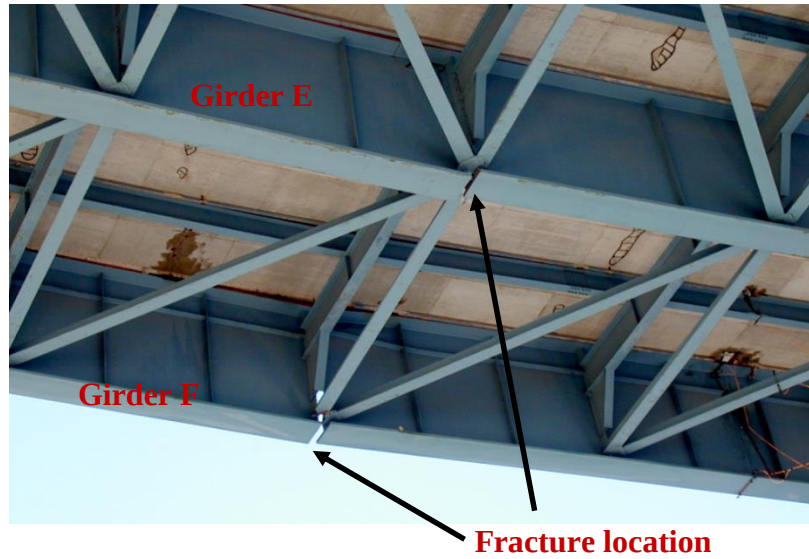


Figure 4-4: Visible fracture in exterior girder F and center girder E (Wright, 2000).



Figure 4-5: Fracture in girder D (Wright, 2000).



Figure 4-6: Fracture in girder E (Wright, 2000).



Figure 4-7: Fracture in girder F (Wright, 2000).

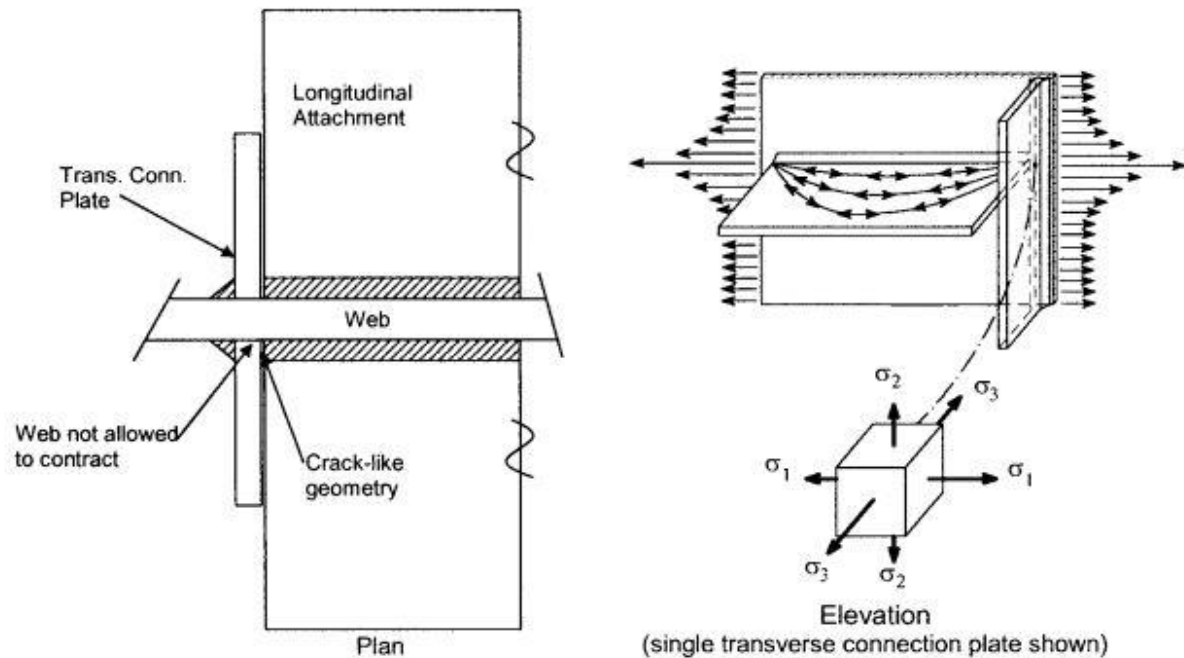


Figure 4-8: Schematic of the connection detail at lateral gusset plate with the transverse connection plate (Connor, et al., 2007), used under fair use.

In this chapter, Section 4.2 provides a detailed description of the modeling methodology used to study this damaged bridge. Section 4.3 discusses the modeling limitations that are associated with the post fracture analysis of the Hoan Bridge. Section 4.4 evaluates the impact of geometric and modeling parameters on the bridge response following simultaneous fracture in girder E and F. Section 4.6 studies the impact of fracture location and fracture sequence on the post fracture

response of the Hoan Bridge. Finally, Section 4.7 investigates suitability of simplified models to capture the post fracture response of the Hoan Bridge.

4.2 Modeling methodology

A detailed finite element model of the Hoan Bridge is constructed using (ABAQUS, 2013) where the individual bridge components are developed separately and then assembled together in the assembly module. The main girders, floor beams and stringers are modeled using general-purpose four node shell elements with reduced integration (S4R). The web is modeled using shell elements at its mid-surface, while the flanges are modeled at the flange to web connection with an offset equal to half the flange thickness. Transverse and longitudinal stiffener are modeled using S4R shell elements with linear elastic material behavior. Steel plasticity is modeled using classical metal plasticity with von Mises yield criteria. The input parameters for steel plasticity are defined as per Section 3.2.2.1.

The reinforced concrete slab is modeled with S4R shell elements with reinforcement modeled as a smeared layer at the appropriate location. Extensive review of the available documentation related to Hoan Bridge was carried out but no details about the concrete slab could be found. Therefore, the concrete slab was redesigned using the strip method in the transverse direction using AASHTO (2009). The longitudinal direction reinforcement was calculated based on distributed reinforcement requirements, and shrinkage and temperature reinforcement requirements in AASHTO (2009). A 4000 psi concrete with Grade 40 reinforcement was used for the slab design. The reinforcement in the longitudinal and transverse direction is tabulated in Table 4-1. Concrete behavior is modeled using the damaged plasticity model while rebar behavior is modeled using classical metal plasticity with von Mises yield criteria. The input parameters for concrete and rebar plasticity are defined as per Section 3.2.2.1.

Table 4-1: Slab reinforcement details (Grade 40).

Direction	Reinforcement
Transverse direction	#6 bars, 8" c/c (Top)
	#5 bars, 8" c/c (Bottom)
Longitudinal direction	#4 bars, 18" c/c (Top)
	#5 bars, 12" c/c (Bottom)

ABAQUS (2013) has a library of standard beam sections with integration points through the cross-section. The integration points are used during the section property calculations and the implementation of material plasticity in beam elements. Lower lateral bracing is modeled using beam element (B31) with predefined beam cross section. Beam elements are modeled at the section centroid.

The Hoan bridge guardrail has a geometric shape (Figure 4-9) that cannot be defined using sections defined in the beam cross-section library in ABAQUS (2013). One option is to use general beam section in ABAQUS (2013) that allows the user defined section properties and plastic behavior using hinge properties at the anticipated location of hinge formation. To define plasticity the user needs to define axial, bending and torsion behavior of the beam section. The general beam section does not account for coupling between axial, bending and torsion behavior. Another option is to idealize the guardrail cross-section as one of the cross section included in the beam cross-section library in ABAQUS (2013) and allow ABAQUS (2013) to compute sectional properties and implement plasticity during analysis. This option is used in the current research. The guardrail cross-section is idealized as a trapezoidal section with same area, base width and height as that of original cross-section. No details about the guardrail concrete and reinforcement material properties, the connection between the guardrail and the concrete slab, and the expansion joints along the bridge length are available. Therefore, for the current research it is assumed that the guardrail is composite with the concrete slab and has the same concrete properties as those of the concrete slab. In addition, the guardrail is assumed to be continuous over the bridge length i.e. with no expansion joints. In the vicinity of fracture, concrete crushing is observed at the top of the guardrail. Typically, guardrails are mildly reinforced. In this study, it is assumed that exclusion of reinforcement in the guardrail idealization will not drastically alter the global bridge response. Some variations of the guardrail modeling are addressed in Section 4.4.1 and Section 4.5.7.1. Concrete plasticity in the guardrail is modeled using the smeared crack model or the cast iron plasticity model. Section 4.4.1 provides further discussion about the impact of inclusion or exclusion of plasticity in the modeling of the guardrail.

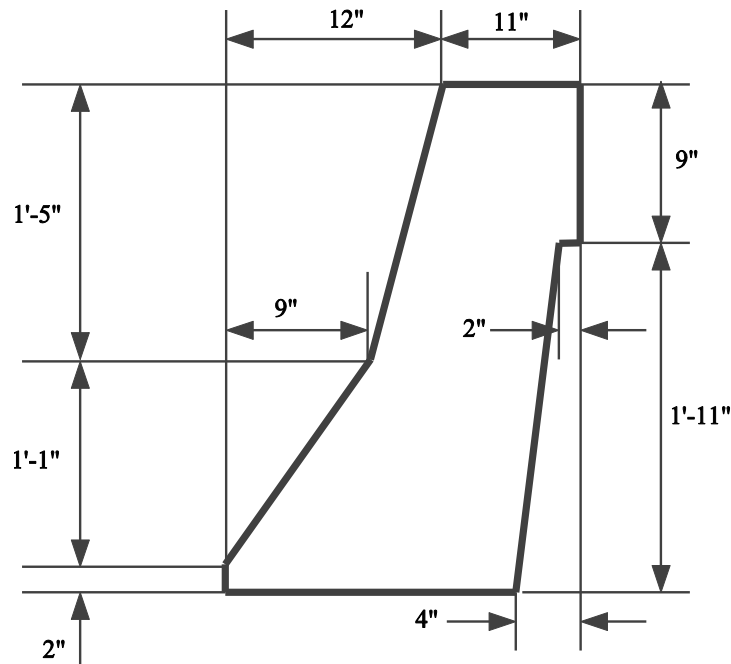


Figure 4-9: Guardrail cross-section

The floor beam assembly is connected to the girder web using tie multipoint constraints (Figure 4-10) that kinematically constrain all six degrees of freedom between the connected nodes. Similarly, the guardrail is made composite with the concrete slab using tie multipoint constraints. Beam connector elements (CONN3D2) are used to connect the lower lateral bracing to the girder web and the stringers to the floor beam. These connector elements enforce kinematic constraints by Lagrange multipliers. Connector elements are used because they provide easy and versatile ways to model and modify connection behavior between two components. In this chapter, beam connectors are used with an assumption that connections have sufficient capacity to carry additional forces following a fracture. This assumption is consistent with the field observation of the failed span. For other fracture scenarios, it will be necessary to evaluate the connection forces to ensure validity of the load redistribution path.

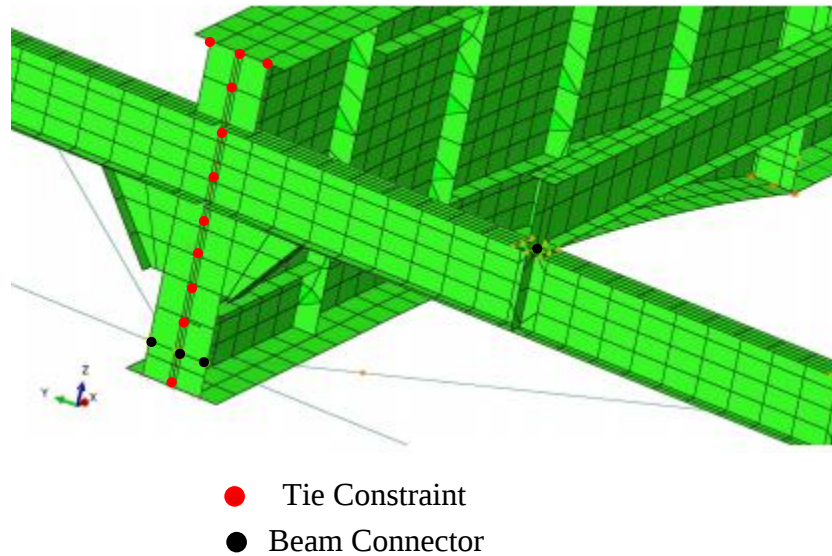


Figure 4-10: Details of the tie constraints and beam connector elements used to connect parts of the model.

The non-composite action at the steel and concrete interface is simulated using surface-to-surface contact with finite sliding. The normal behavior is modeled using hard contact with nonlinear penalty stiffness. The nonlinear penalty stiffness method is selected over the linear penalty stiffness method because it results in better convergence as per ABAQUS documentation (ABAQUS, 2013). The tangential behavior is modeled using an isotropic coulomb friction model with a coefficient of static friction of 0.57 (Rabbat and Russell, 1985).

Based on the recommendations for mesh size for modelling of steel girders by White et al. (1997), main girders and box-girder webs should be modeled with at least ten elements through the depth and four elements across the width of flange. The bottom flange of the box-girder should be treated similarly to the web and is meshed with ten elements through the width. In the Hoan Bridge model, floor beams and stringers are meshed with four elements through the web depth and two elements across the width of flange. The lower lateral bracing and cross bracing components are modeled with five beam elements along the length. The concrete slab is meshed with an average mesh size of 8 in. in the fractured span. In the remaining spans and overhang, the average mesh size in the concrete slab is gradually increased to 24 in. away from the fractured span. The guardrail is meshed with an average mesh size of 24 in. The guardrail mesh

is further refined to an average mesh size of 8 in. over the fractured region and the interior pier (0.1L on either side) to capture concrete plasticity.

Figure 4-11 shows the design boundary conditions for the Hoan Bridge. Pier 5S, Pier 4S and Pier 3S supports had fixed shoes while Pier2S supports had expansion shoes. The fixed shoe support did not allow movement in the longitudinal and transverse direction of the bridge while the expansion shoe allowed movement only in the longitudinal direction. These boundary conditions are applied at the bottom of the plate girder flanges. The Hoan Bridge had a 25 ft. overhang over Pier 5S which supported span S3A and S3B. At the joint, a roller assembly was provided allowing for transfer of vertical reaction and no moment. This assembly was treated as an internal hinge in the analyses. The reaction and vertical stiffness from span S3A and S3B was calculated by analyzing the span using line girder analysis. Span S3A and S3B were considered as non-composite bridges during this analysis. These analyses yielded a vertical reaction of 32.7 kips and a vertical stiffness of 33 kips/ft. for exterior girders D and F. For interior girder E, the analyses yielded vertical reaction of 37.65 kips and a vertical stiffness of 33 kips/ft. Figure 4-12 shows the characteristics of typical finite element model used for this study.

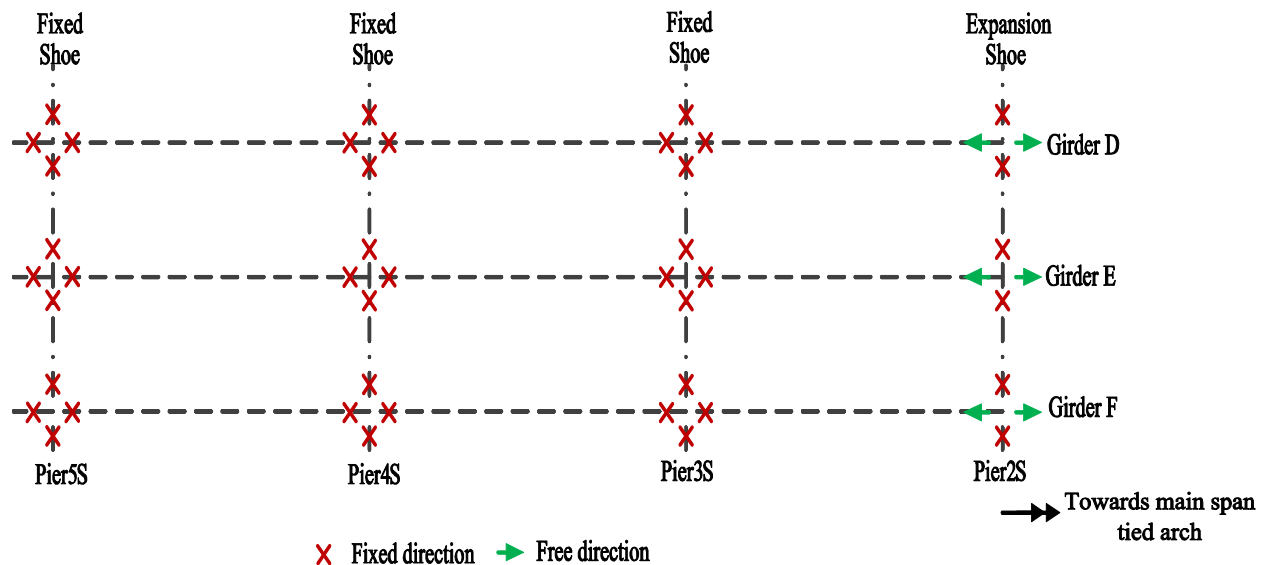


Figure 4-11: Boundary conditions on Hoan Bridge.

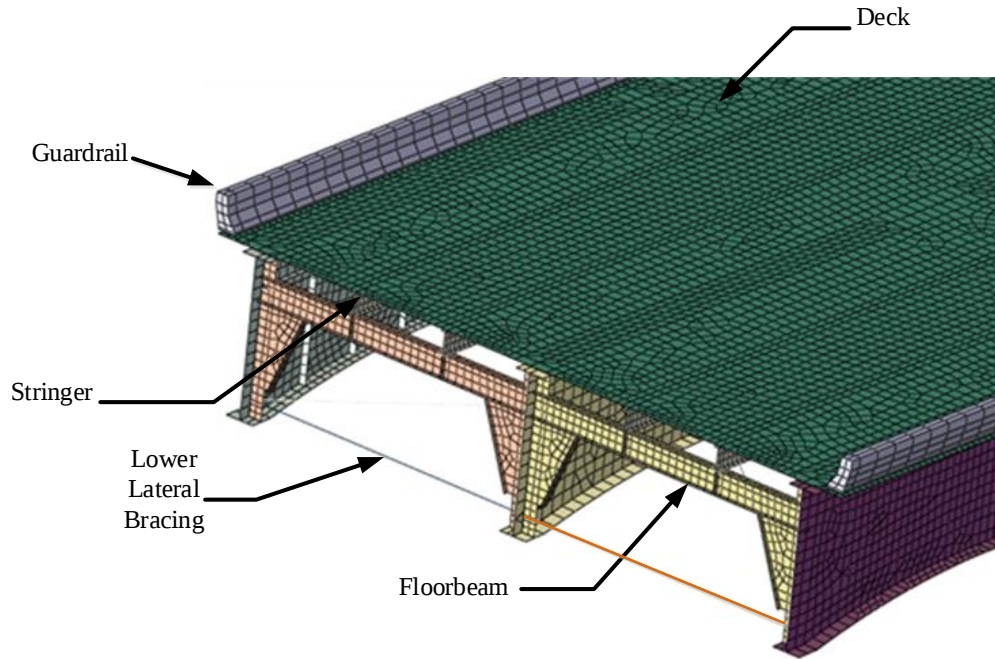


Figure 4-12: Finite element model of Hoan Bridge.

Table 4-2 tabulates the component material properties that are used in this study.

Table 4-2: Component material properties.

Bridge Component	Material	Unit Weight (lb/ft ³)	E (ksi)
Flange, flange splice and bearing stiffener plate	A588 steel	490	29000
All other structural steel	A36 steel	490	29000
Deck	Concrete	150	4326

Based on the observed conditions at the time of failure it is assumed that the interior girder E fractured first followed by exterior girder F and D (Fisher, et al., 2001). Fracture through the bottom flange and full web depth were observed in girders E and F (Figure 4-13). Partial depth web fractures were found at three locations in girder D as shown in Figure 4-3. For the following parametric study, the bridge response following simultaneous fracture of girder E and F is investigated. The partial depth fractures in girder D are ignored in this analysis because it is expected to affect only the local stress distribution in girder D near fracture and not the global post fracture response of the bridge. The validity of this assumption is further evaluated in Section 4.6.1.

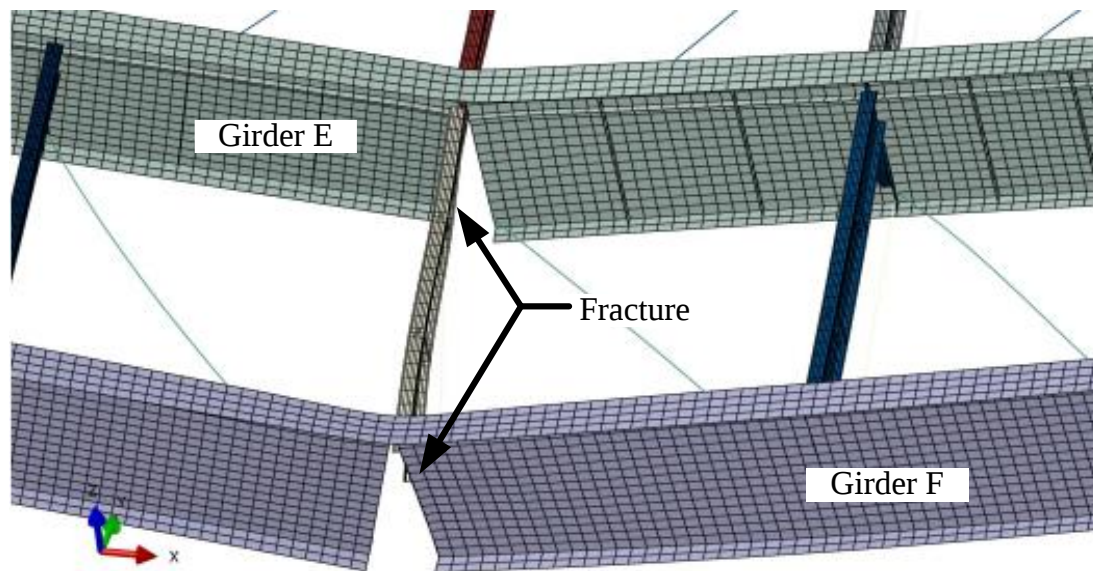


Figure 4-13: Fractured girder E and F.

4.3 Modeling caveats

The following section discusses the modeling caveats that the reader should consider as he/she assesses the results of the post fracture analysis of the Hoan Bridge.

4.3.1 Transferring results from *ABAQUS/Standard* to *ABAQUS/Explicit*

ABAQUS (2013) provides the capability to import a deformed mesh and its associated material state from *ABAQUS/Standard* into *ABAQUS/Explicit* and vice versa. In addition, new model information can be specified during the import process. This capability is useful for problems that involve several analysis stages and allows users to take advantage of features and strengths of both the *ABAQUS/Standard* and *ABAQUS/Explicit* analysis options. *ABAQUS/Standard* uses a Hilber-Hughes-Taylor time stepping scheme in which a set of nonlinear equilibrium equations are solved at each time increment. Therefore, a large number of iterations are required at each time increment to reach convergence for problems involving contact definition and material plasticity. This difficulty is observed during evaluation of different guardrail models (Section 4.4.1) and damping definition (Section 4.5.4). The *ABAQUS/Explicit* scheme uses a staggered central difference rule to calculate displacements and velocities in terms of quantities known from the previous increment. The stable time increment size may need to be very small

to satisfy the solution criteria (wave speed check in the element), but this approach is relatively inexpensive compared to the number of increments required by the *ABAQUS/Standard* approach. The dynamic equilibrium is satisfied at the beginning of the time increment. In the analysis of the Hoan Bridge, response of the intact bridge to dead and live load is simulated using *ABAQUS/Standard*. The fracture event is simulated in either *ABAQUS/Standard* or *ABAQUS/Explicit*. Section 4.3.2 provides details about the introduction of fracture in both analysis techniques. The post fracture analysis is carried out using *ABAQUS/Explicit*. Use of *ABAQUS/Explicit* to perform post fracture analysis of the Hoan Bridge resulted in significant savings in computational cost.

4.3.2 Introduction of fracture in *ABAQUS/Standard* and *ABAQUS/Explicit*

In the present study, fracture is modeled by introducing duplicate nodes at the predefined fracture path. The duplicate nodes are connected using zero length beam/weld connector elements to mimic intact condition. In Method-I, the fracture event is simulated by deactivating connector elements using the “Model Change” option in *ABAQUS/Standard*. This method is verified using a three-truss bar model in Chapter 3. In Method-II, the fracture is simulated in *ABAQUS/Explicit*, the analysis is carried out using the intact and the fractured model of the bridge. In the first step, the intact model of the bridge is analyzed and the forces in the connector elements at the location of fracture are recorded. These forces are then applied as equal and opposite nodal forces at the fracture location in the fractured bridge model to simulate intact condition. In the fracture simulation step, these forces are ramped down to zero over a short transient step to simulate fracture. Both approaches are conceptually identical and should yield the same response. Method-II is evaluated by comparing deflections at the location of fracture in all three girders following fracture in girder E (Figure 4-14). Both methods yield identical deflection response, as conceptually they are identical. Method-II used to investigate impact of fracture sequence on the Hoan Bridge response in Section 4.7.

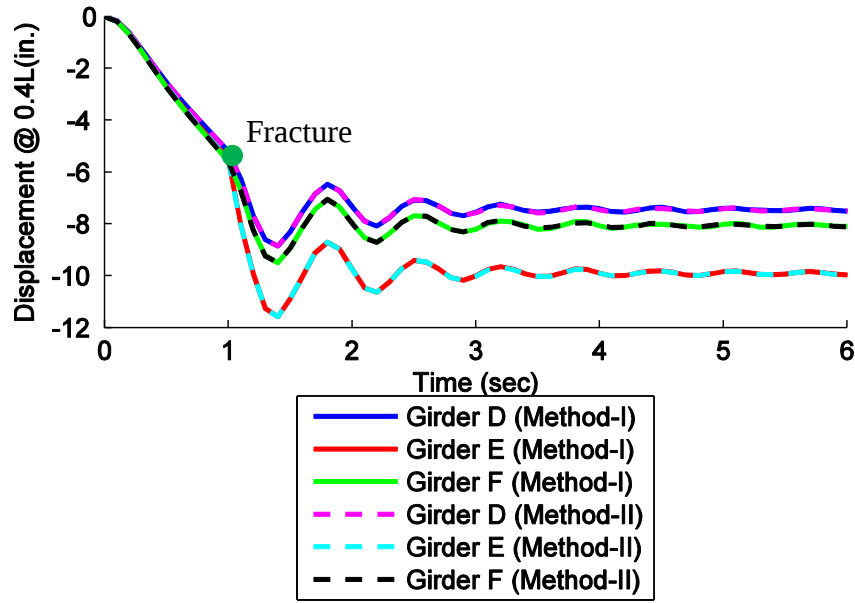


Figure 4-14: Fracture simulation method comparison.

4.3.3 Quasi-static analysis using *ABAQUS/Explicit*

ABAQUS/Standard offers four methods (*Newton's method*, *Newton's method with automatic stabilization*, *Modified Riks method*, and *Implicit quasi-static method* with backward Euler operator for time integration and Newton's method for integration within time increment) to obtain a quasi-static solution. The analyses in this section are performed using non-linear material properties. As noted earlier, with *ABAQUS/Standard* a large number of iterations are required at each time increment to get convergence for problems involving contact definition and material plasticity. Alternatively, *ABAQUS/Explicit* can be used to obtain a quasi-static solution. The size of a stable time increment in an *Explicit* scheme is small and is a function of highest frequency and critical damping ratio. The time required to simulate quasi-static analysis over the actual duration of the event can be very large. Therefore, the computational cost can be reduced by either speeding up the simulation or by scaling the mass during analysis. In this study, the quasi-static analysis is carried out by speeding up the simulation in *ABAQUS/Explicit*. Abdullah and Easterling (2009) used the fundamental time period (T_{bridge}) of the structure as an indicator for specifying the time step period in the quasi-static analysis. Similar to *Implicit quasi-static analysis* in *ABAQUS/Standard*, it is necessary to check that the kinetic energy (ALLKE) of the model does not exceed a small fraction (typically less than 5%) of its internal energy (ALLIE) in

order to eliminate inertial effects. To compare effectiveness of this technique, the response of the Hoan Bridge when fracture in girder E and F is introduced gradually using *Implicit quasi-static analysis* in *ABAQUS/Standard* is compared against *ABAQUS/Explicit* solution. Figure 4-15 and Figure 4-16 shows the displacement response of intact girder D and fractured girder F respectively. For the *Implicit quasi-static analysis* and the *Explicit quasi-static analysis* with duration of analysis equal to 0.75 sec that is very close to T_{bridge} , the ratio (Table 4-3) of kinetic energy (KE) to internal energy (IE) is higher than the typical limit of 5% used with the quasi-static analysis. Higher KE/IE ratio indicate presence of dynamic effects which leads to stiffer response for both models as shown in Figure 4-15 and Figure 4-16. Introduction of fracture over $5T_{bridge}$ and $8T_{bridge}$ time steps yielded a quasi-static solution with KE/IE ratio less than 1% throughout the analysis. The fracture introduction step duration is normalized to 1 sec for comparing results from the analyses with different durations for fracture introduction step. *ABAQUS/Explicit quasi-static analysis* with $5T_{bridge}$ and $8T_{bridge}$ yielded identical deflection response. *ABAQUS/Explicit quasi-static analysis* with T_{bridge} yielded stiffer response in the presence of inertial/dynamic effects as indicated by higher KE/IE ratio (11.1%) at the end of analysis. Therefore, quasi-static analysis with $5T_{bridge}$ or $8T_{bridge}$ can be used to simulate gradual introduction of fracture in girder E and F. This section describes the approach to select the analysis duration for quasi-static response using *ABAQUS/Explicit*. The same approach is used to select the duration for the simplified methods discussed in Section 4.7.

Table 4-3: Girder deflection comparison using *implicit* and *explicit quasi-static* analysis.

Analysis Description	Step	At fracture location (0.4L)			KE/IE*100
		Girder D (in.)	Girder E (in.)	Girder F (in.)	
Implicit quasi-static analysis	Dead and live load	-5.3	-5.7	-5.7	0.82
	Fracture introduction	-8.7	-23.8	-35.3	8.59
Explicit with $t = 0.75$ sec i.e. T_{bridge}	Dead and live load	-5.2	-5.5	-5.6	0.9
	Fracture introduction	-8.3	-21.1	-30.7	11.1
Explicit with $t = 3.75$ sec i.e. $5T_{bridge}$	Dead and live load	-5.2	-5.5	-5.6	0.9
	Fracture introduction	-8.9	-26.2	-39.8	0.4
Explicit with $t = 6.00$ sec i.e. $8T_{bridge}$	Dead and live load	-5.2	-5.5	-5.6	0.9
	Fracture introduction	-8.9	-26.0	-39.5	0.05

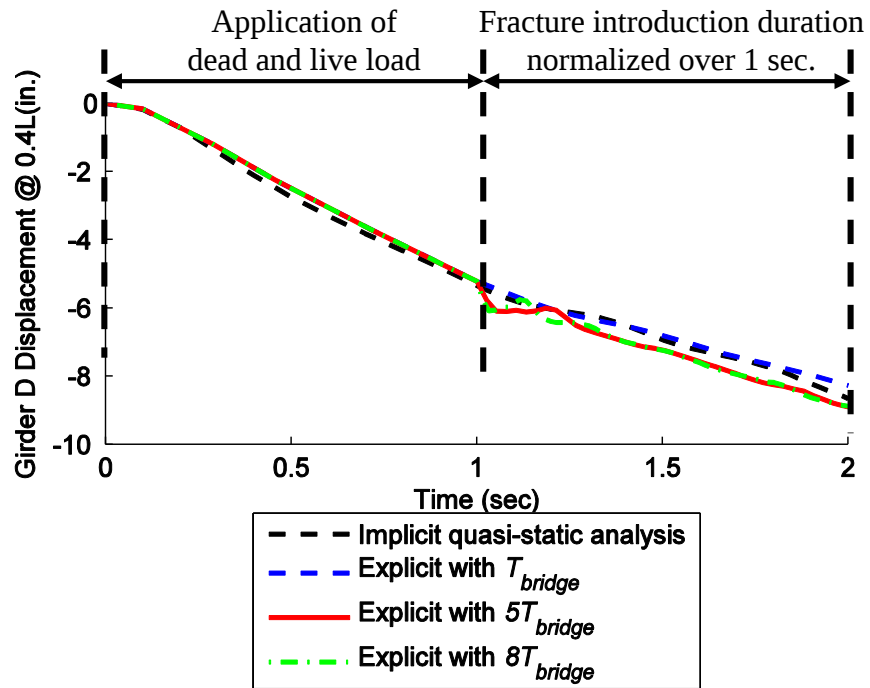


Figure 4-15: Quasi-static displacement response of girder D following fracture at 1 sec.

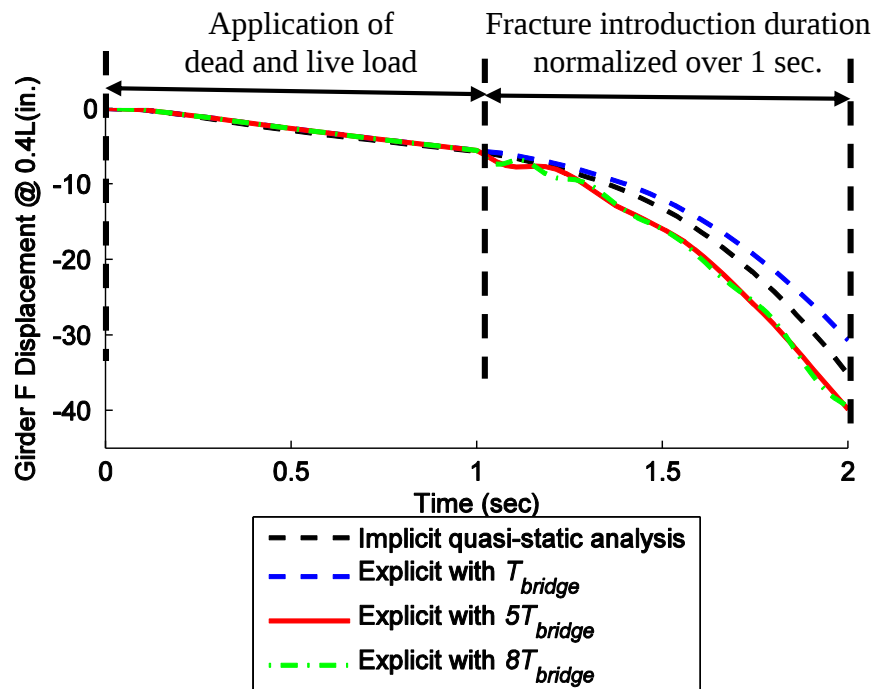


Figure 4-16: Quasi-static displacement response of girder F following fracture at 1 sec.

4.4 Finite element analysis of Hoan Bridge

The objective of this section is to determine the effect of geometric and modeling parameters on bridge response and load carrying capacity of the bridge following a bottom flange and full depth web fracture in girder E and F. For this parametric study, analyses are carried out in a series of steps in ABAQUS (2013). In the first step, dead and live load is applied to the bridge. Dead load includes the self-weight of the components, the weight of the asphalt overlay and reactions from the span S3A over the 25 ft overhang. No documents regarding the construction sequence of the bridge could be found and it is also expected that the deformation and member forces resulting from the fracture will dominate the response compared to deformation and stresses resulting from the construction sequence. Therefore, the dead load of all the bridge components is activated in a single step. Additionally, nominal material properties are used during analysis that are lower than the expected material properties. For live load, a 100 kip truck load is applied over the fracture location on the right hand lane. To calculate individual axle loads, a HS-20 (72 kips) truckload distribution is scaled to get a 100 kip truck. In the next step, fracture is simulated by deactivating beam connector elements over girder E and F using the “Model

Change” option in ABAQUS (2013). In a static analysis, the “Model Change” option introduces fracture slowly allowing structure to stabilize over the entire step. In a dynamic analysis, fracture is simulated in two steps. In the first step, fracture is introduced in a short transient step over 0.001 sec using the “Model Change” option. The step duration of 0.001 sec is based on the time it would take for a full depth fracture to propagate through a 120 in. deep Hoan girder assuming a fracture speed of 7,000 ft./s in structural steel (Barsom and Rolfe, 1999). Further details about fracture simulation using the “Model Change” option are provided in Section 3.2.5 and Section 3.3.3. In the second step, dynamic response of the structure post fracture is simulated for duration of 5 sec.

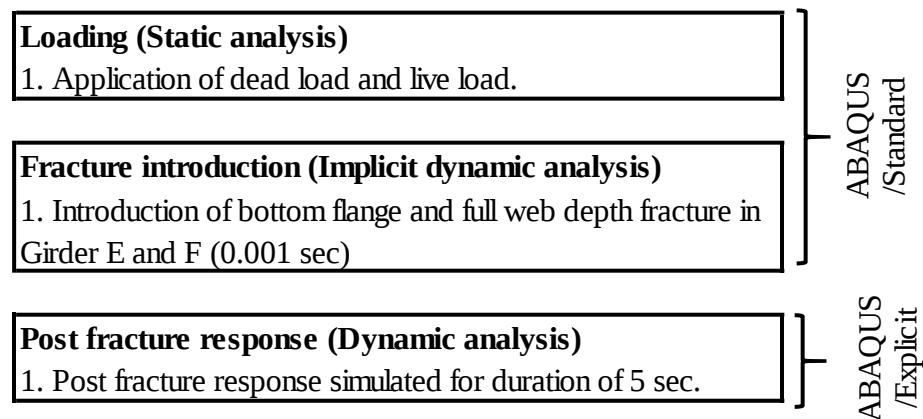


Figure 4-17: Post fracture response simulation steps.

As the first step of this study, a linear elastic model of the Hoan Bridge is analyzed using an *Implicit quasi-static analysis* to establish a baseline for the overall bridge behavior and possible yield locations. As discussed before, the analysis is carried out in two steps. In the first step, dead load and live load is applied to the bridge. In the second step, bottom flange and full web depth fracture in girder E and F is simulated over a quasi-static step. The deformation in all three girders at the location of fracture at the end of two steps are tabulated in Table 4-4 .

Table 4-4: Girder displacement (in.).

Step	Description	Girder D (in.)	Girder E (in.)	Girder F (in.)
1	Dead load and live load	-5.33	-5.69	-5.74
2	Girder E and F fracture	-8.57	-22.09	-32.75

(Negative sign indicates downward displacement)

Figure 4-18 shows the von Mises stress distribution in the steel superstructure in span 3S-2S. In the Hoan Bridge, flange plates and bearing stiffeners were made of A588 steel while all other components were made of A36 steel. Therefore, the majority of steel is A36. The display scale is set to display regions having stress higher than 36 ksi in red to visually identify regions of high stress or yield in case of A36 material. Based on the von Mises stress distribution, yielding should occur in stringers close to fracture location and in the girder web and top flange from Pier 3S up to 0.3L.

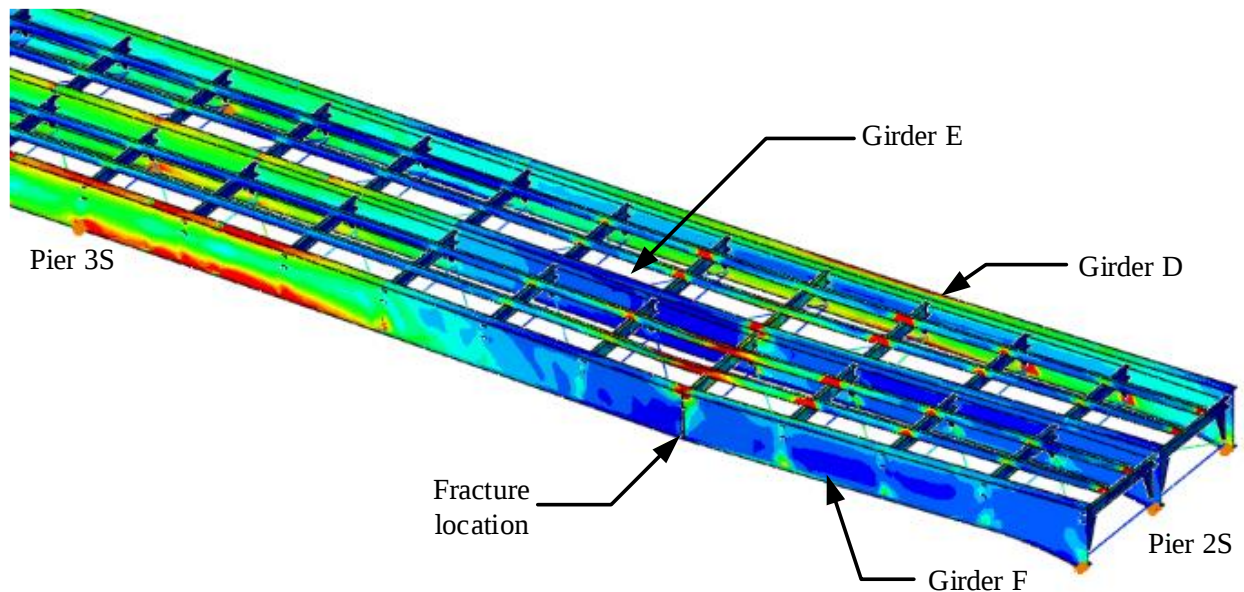


Figure 4-18: von Mises stress distribution at fracture location in Span 3S-2S (Linear elastic model).

Figure 4-19 shows the stress distribution at the top and bottom fiber of the guardrail (Element No. 44) right above the fracture location. The stresses exceeded the tensile and compressive limiting values in regions close to the fracture (Figure 4-19) but were lower than the limiting values in regions away from the fracture. As noted earlier in Section 4.2, the guardrail is assumed composite with the concrete slab, which results in lower axis neutral axis and lower tensile stresses in the bottom fiber compared to the compressive stress in the top fiber. From this analysis, it is clear that material plasticity needs to be included in the entire model to capture the post fracture response of the Hoan Bridge.

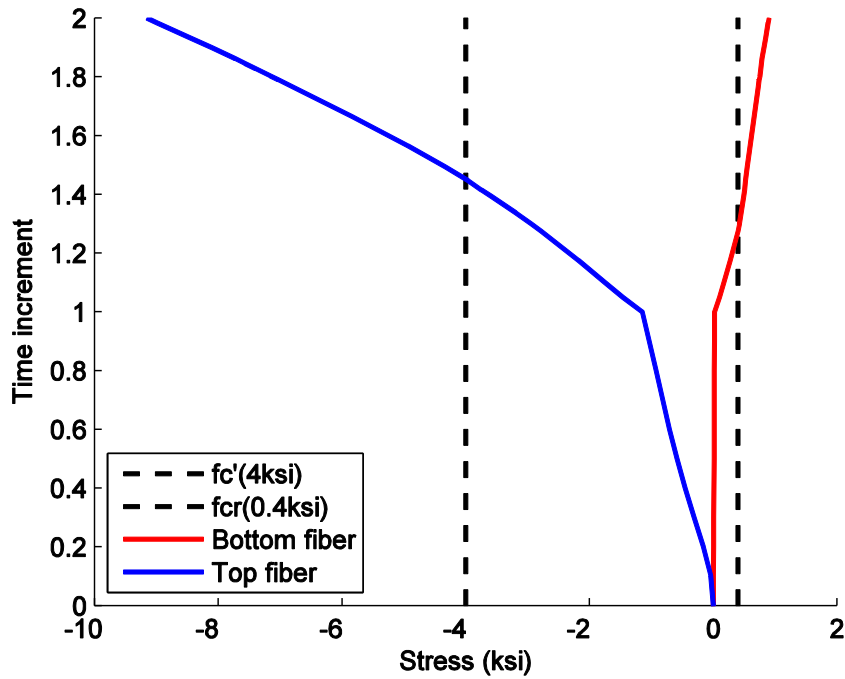


Figure 4-19: Stress distribution in guardrail (Element No. 44) for linear elastic model.

4.4.1 Selection of guardrail model

Results from the linear elastic model of the Hoan Bridge showed that it is important to understand plasticity in structural components in order to capture the load redistribution following the fracture event. Therefore, seven guardrail models (Table 4-5) are analyzed using the *Implicit quasi-static analysis* in *ABAQUS/Standard*. Model-1, Model-4 and Model-5 use linear elastic properties for guardrail concrete imposing no limit on the tensile or compressive stress. In Model-2, Model-3 and Model-6, tensile stress in concrete is held at the rupture stress for the post cracking strain region. Analyses are carried out over two steps for all the guardrail models. In the first step, dead load and live load is applied to the bridge. In the second step, the bottom flange and the full web depth fracture in the girders E and F is simulated over a quasi-static step. No additional load is applied in the second step.

Table 4-5: Guardrail model description.

Guardrail	Model description
Model-1	Elastic beam element with trapezoidal cross section.
Model-2	Beam element with trapezoidal cross section with smeared crack concrete model
Model-3	Beam element with trapezoidal cross section with cast iron plasticity model
Model-4	Elastic beam element with axial connector element at fracture location
Model-5	Elastic beam element with damage at fracture location simulated by removing beam element
Model-6	Solid elements with damaged plasticity model
Model-7	Guardrail excluded from the analysis but its weight applied as distributed load to slab.

In Model-1, the guardrail is modeled using elastic beam elements with trapezoidal cross section (Figure 4-20). The guardrail is made composite with the beam slab using beam connector elements. In Model-2, the plasticity in the guardrail is implemented using the smeared crack concrete model. The guardrail is assumed to have the same properties as the bridge slab with the exception of the post-cracking behavior of concrete. For the post-cracking behavior, tensile stress in concrete is held at rupture stress (f_{cr}) instead of including strength degradation using a tension-stiffening model. This modification was done to get around the convergence difficulties observed in the initial analysis with the tension-stiffening model. In Model-3, the concrete plasticity in the guardrail is implemented using the cast iron plasticity model in ABAQUS (2013). Cast iron plasticity model is intended for gray cast iron and it allows for different stress-strain response in tension and compression. In the cast iron plasticity model, the yield function depends on the maximum principal stress for tensile behavior and on pressure-independent behavior based on deviatoric stresses for compressive behavior. Similar to Model-2, tensile stress in concrete is held at rupture stress (f_{cr}) post-cracking.

In Model-4 (Figure 4-21), guardrail nodes near the location of girder fractures are disconnected by introducing duplicate nodes and connecting the duplicate nodes using an axial connector element rigid in compression and free in tension. In Model-5 (Figure 4-22), a discontinuity is introduced in the guardrail at the fracture location by deleting the beam element at the fracture location. In Model-4 and Model-5, plasticity in the rest of the guardrail is implemented through cast iron plasticity model.

In Model-6, the guardrail is modeled using solid elements (C3D8R), with denser mesh near the fracture location (i.e. between 0.3L and 0.5L from Pier 2S) and over the interior Pier 3S to capture the concrete crushing and cracking post girder fracture. Concrete plasticity is implemented through use of concrete damaged plasticity model. To compare the response with Model-2 and Model-3, Model-6 did not include strength degradation using tension-stiffening model; instead, the tensile stress is held at the rupture stress in the post cracking stage. In Model-7, the guardrail is excluded from the model but its mass is applied as a distributed load in the region of the slab occupied by the guardrail. This method does not account for the guardrail stiffness but accounts for its weight.

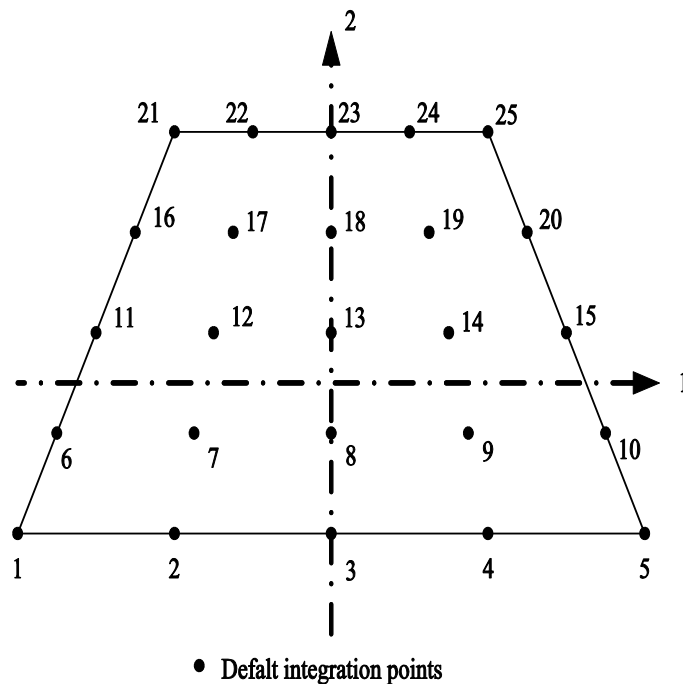


Figure 4-20: Trapezoidal beam model with default integration points.

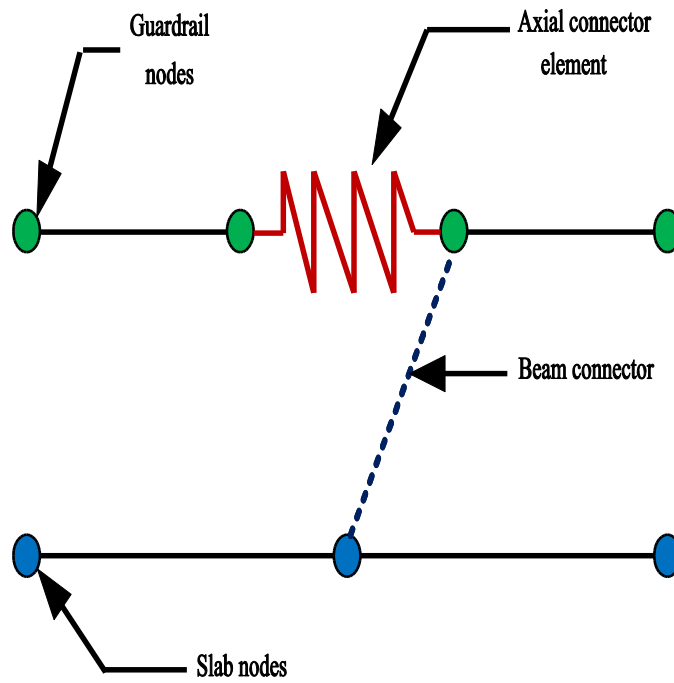


Figure 4-21: Model-4.

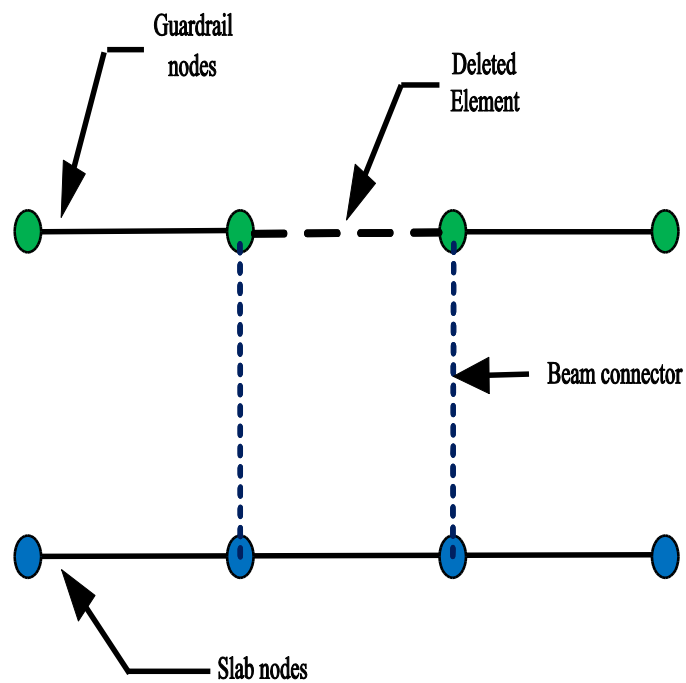


Figure 4-22: Model-5.

Model-1 and Model-6 reached a converged solution while Model-2, Model-3, Model-4, Model-5 analyses had convergence problems and the analyses terminated during the second step where fracture is introduced. Analyses failed to converge because of local failure of the concrete in the slab and guardrail near fractured location. The local failure of concrete does not imply structural failure. These analyses use “Model Change” option to simulate fracture. “Model Change” stores forces in the elements to be removed and ramps those forces down gradually to zero during the removal step. Analyses that fail to converge in the element removal step have residual forces from the removed elements. Therefore, output from last successful time increment from failed analysis can only be used to compare different guardrail models but not for the post fracture response of the bridge.

The girder displacements are recorded at the intersection of the bottom flange and web. Girder displacement in all three girders at the location of fracture are tabulated in Table 4-6. Along with the displacements at the last successful iteration, displacement at the previous time increment (taken at the nearest time interval) are also recorded to compare response between different guardrail models. Different guardrail models yielded similar displacements at intermediate time intervals as shown in Table 4-6.

The girder displacements from Model-2 and Model-3 are similar to the girder displacement from Model-6 at the intermediate time increments (Table 4-6). To evaluate the capability of Model-2 and Model-3 to capture guardrail plasticity, the axial force and the major axis bending moment are recorded at the location of fracture in the guardrail (Table 4-7). The axial force and the major axis bending moment are also recorded for Model-1. For Model-1, along with the values at the end of the analysis, the axial force and the major axis bending moment values are recorded at the intermediate time increments for comparison with Model-2 and Model-3.

The axial force values in Model-1 is similar to the axial force value in Model-2 and Model-3 at the intermediate increments. Model-1 with the elastic beam elements yields higher major axis bending moment values compared to Model-2 and Model-3 results. Use of elastic beam elements in Model-1 resulted in successful completion of the analysis but this approach can provide a false sense of safety for the bridge as the elastic guardrail will continue to carry load beyond its actual capacity instead of redistributing to other bridge members and thereby altering the bridge failure

mode. Therefore, use of elastic beam elements to model guardrail should be carefully evaluated on the case-by-case basis.

Table 4-6: Girder displacement at location of fracture (0.4L) for different guardrail models.

Guardrail	Time Increment	0.4L			KE/IE*100
		Girder D (in.)	Girder E (in.)	Girder F (in.)	
Model-1	0.70	-7.64	-15.32	-20.68	6.67
	0.80	-7.93	-18.18	-25.65	7.43
	1.00	-8.66	-24.66	-36.79	7.16
Model-2	0.70	-7.52	-15.60	-21.40	5.33
Model-3	0.70	-7.66	-15.39	-20.82	7.25
	0.80	-7.94	-18.30	-25.88	8.17
Model-4	0.71	-7.52	-15.38	-21.06	6.64
	0.78	-7.75	-17.43	-24.58	8.08
Model-5	0.72	-7.61	-16.06	-22.21	7.04
	0.76	-7.77	-17.29	-24.30	7.83
Model-6	0.70	-7.50	-14.83	-19.95	5.27
	0.80	-7.79	-17.38	-24.29	6.71
	1.00	-8.65	-23.80	-35.27	8.59
Model-7	0.72	-7.78	-15.55	-21.36	5.00
	0.78	-7.98	-16.99	-23.80	5.54
	0.80	-8.08	-17.65	-24.92	5.74
	1.00	-8.88	-23.55	-35.01	6.10

(Negative sign indicates downward displacement)

Table 4-7: Axial force and major axis bending moment in guardrail at the fracture location (Element No.44).

Guardrail	Time Increment	Axial Force (kips)	Major Axis Bending Moment (kip-in)
Model-1	0.70	598.1	7339.4
	0.80	664.8	8519.1
	1.00	851.5	13583.8
Model-2	0.70	590.9	5425.2
Model-3	0.80	656.8	5933.7

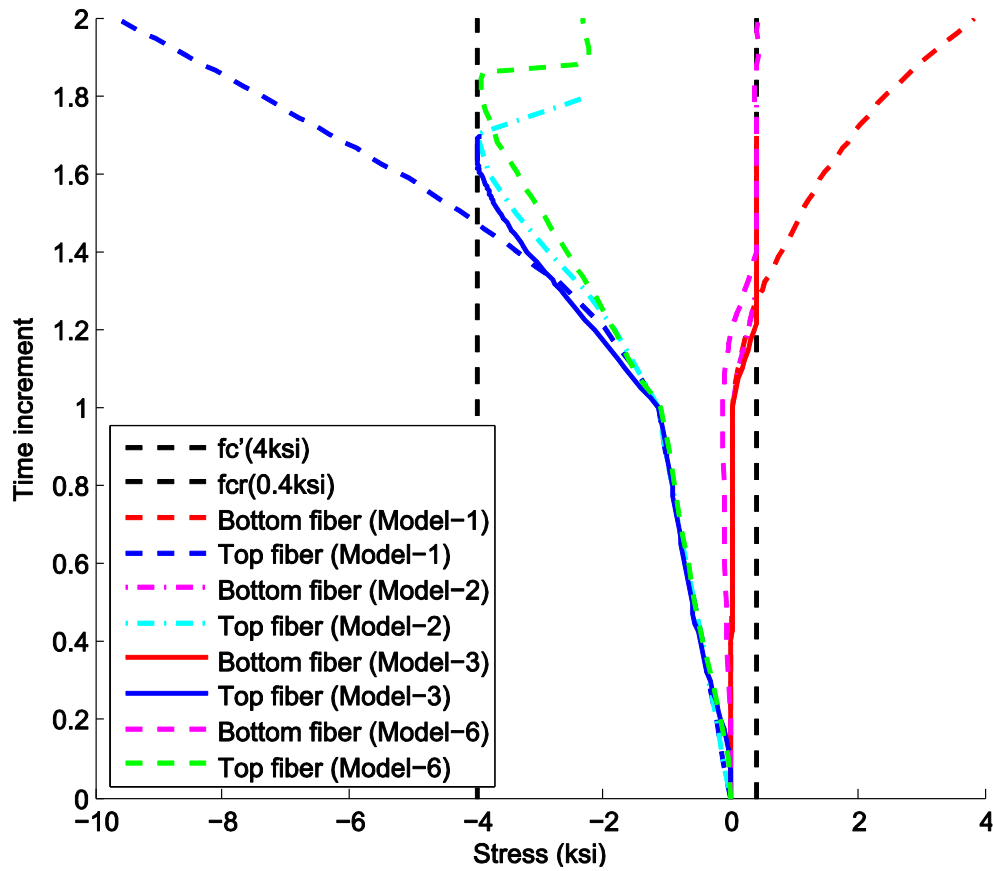


Figure 4-23: Stress distribution in guardrail at the fracture location for different guardrail models.

Model-2, Model-3 and Model-6 include a plasticity definition for the guardrail. To evaluate their capacity to capture the concrete behavior in tension and compression, the top and bottom fiber stresses at the location of fracture are recorded throughout the analysis (Figure 4-23). In addition, the top and bottom fiber stresses are also recorded for Model-1. Fracture is introduced at time increment 1.0 in step 2. At the end of step 1 where bridge is loaded with dead load and live load, the top and bottom fiber stresses are within the elastic limit for compression ($0.4 f_c'$) and tension (f_{cr}). The compressive stress in the top fiber is similar for all models while tensile stress in Model-2 is lower compared other models at the end of step 1. Lower stress at the end of step 1 for Model-2 resulted in the concrete cracking in the top fiber in later time increments in step 2. In Model-1 with the elastic beam elements, top and bottom fiber stresses exceed the compressive strength (f_c') and rupture stress (f_{cr}). Model-6 with solid elements, yielded stiffer response compared to Model-2 and Model-3. Model-2 and Model-6 are able to follow the stress-strain relationship defined for concrete in tension and compression. The cast iron plasticity model

(Model-3) captured the stress-strain curve up to the compressive strength (f_c') of the guardrail concrete correctly but then failed to follow the strength degradation beyond f_c' . The bottom fiber stress reached the rupture stress (f_{cr}) around 0.2 time increment in step 2 and all three models were able to maintain it rupture stress (f_{cr}) for the subsequent time increments. Although, the smeared crack concrete model and cast iron plasticity model used in Model-2 and Model-3 captured the guardrail plasticity at the location of fracture they failed to get a converged solution. Additionally both material models are intended for monotonic loading and will not be useful in correctly capturing the post fracture dynamic response of the bridge. Therefore, Model-2 and Model-3 will not be considered for further evaluation using dynamic analysis.

Figure 4-24 through Figure 4-30 shows the vector representation of the principal plastic strain distribution at the top and bottom of the concrete slab for all seven guardrail models. The principal plastic strain distribution at the top and bottom of the concrete slab for Model-1 and Model-6 are similar indicating that both models will result in similar yield line patterns on the slab. Model-2 and Model-3 show less plastic strain in the concrete slab as the analysis failed before reaching the full time step. Use of Model-4 (Figure 4-27) and Model-5 (Figure 4-28) resulted in more localized plasticity in the slab near the location of fracture and the plasticity did not spread out as compared to Model-6 (Figure 4-29). Therefore, Model-4 and Model-5 are excluded from further evaluation using dynamic analysis. It was expected that the girder displacements from Model-7 would be higher compared to Model-6 and Model-1 response. However, the girder displacements are similar to Model-6 and Model-1 (Table 4-6). This response can be attributed to the redistribution of the load to larger slab and steel superstructure region in the absence of the guardrail as seen in Figure 4-30. Redistribution of forces further away from the location of fracture can cause lower demand on the connections and structural components closer to fracture. Therefore, it is important to include guardrail in the analysis to capture correct global response and demand on structural components.

In Table 4-6, along with the girder displacements ratio of kinetic energy (KE) to internal energy (IE) is also tabulated at different time increment to ensure kinetic energy does not dominate the response. The KE/IE ratio in all the guardrail models is between 5-10%. The current kinetic energy contribution is higher than the typical limit of 5% used with quasi-static solution scheme. The current kinetic energy calculation includes kinetic energy from the rigid body movement of

the fractured girders. The rigid body movement does not cause any strain or stress and does not contribute to the internal energy. Therefore, kinetic energy from the rigid body movement should be excluded from the calculation of KE/IE ratio. Instead of calculating kinetic energy from the rigid body movement, response is compared with the analysis where fracture is introduced gradually. Model-2 and Model-3 are reanalyzed in Section 4.4.2 with the gradual introduction of fracture over multiple steps to assess the validity of the current analysis. Model-2 and Model-3 are selected instead of other models to see whether a converged solution can be obtained using a gradual introduction of fracture. Model-1, Model-6 and Model-7 are further analyzed using nonlinear dynamic analysis in Section 4.5.7.1.

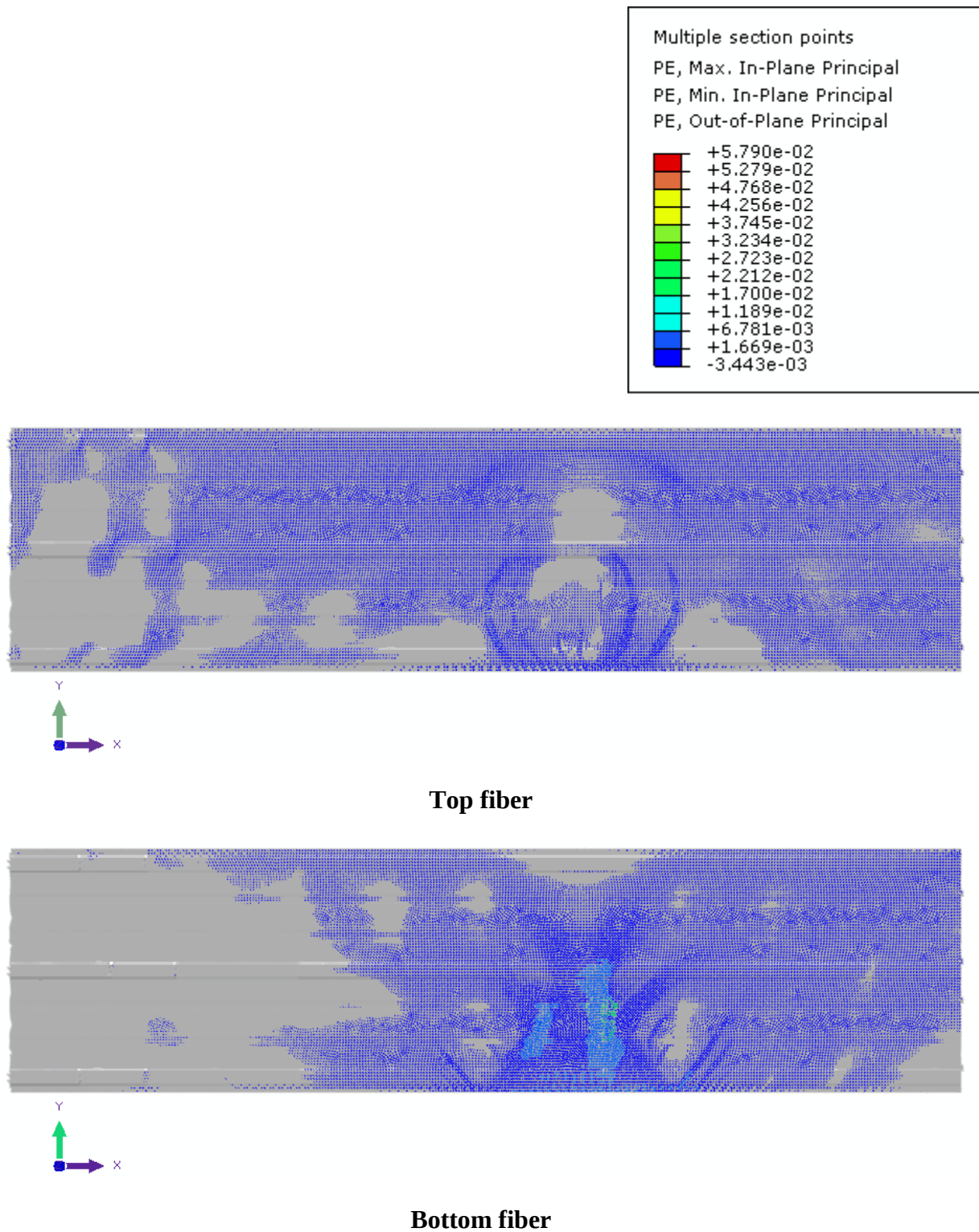


Figure 4-24: Principal plastic strain distribution on slab (Model-1).

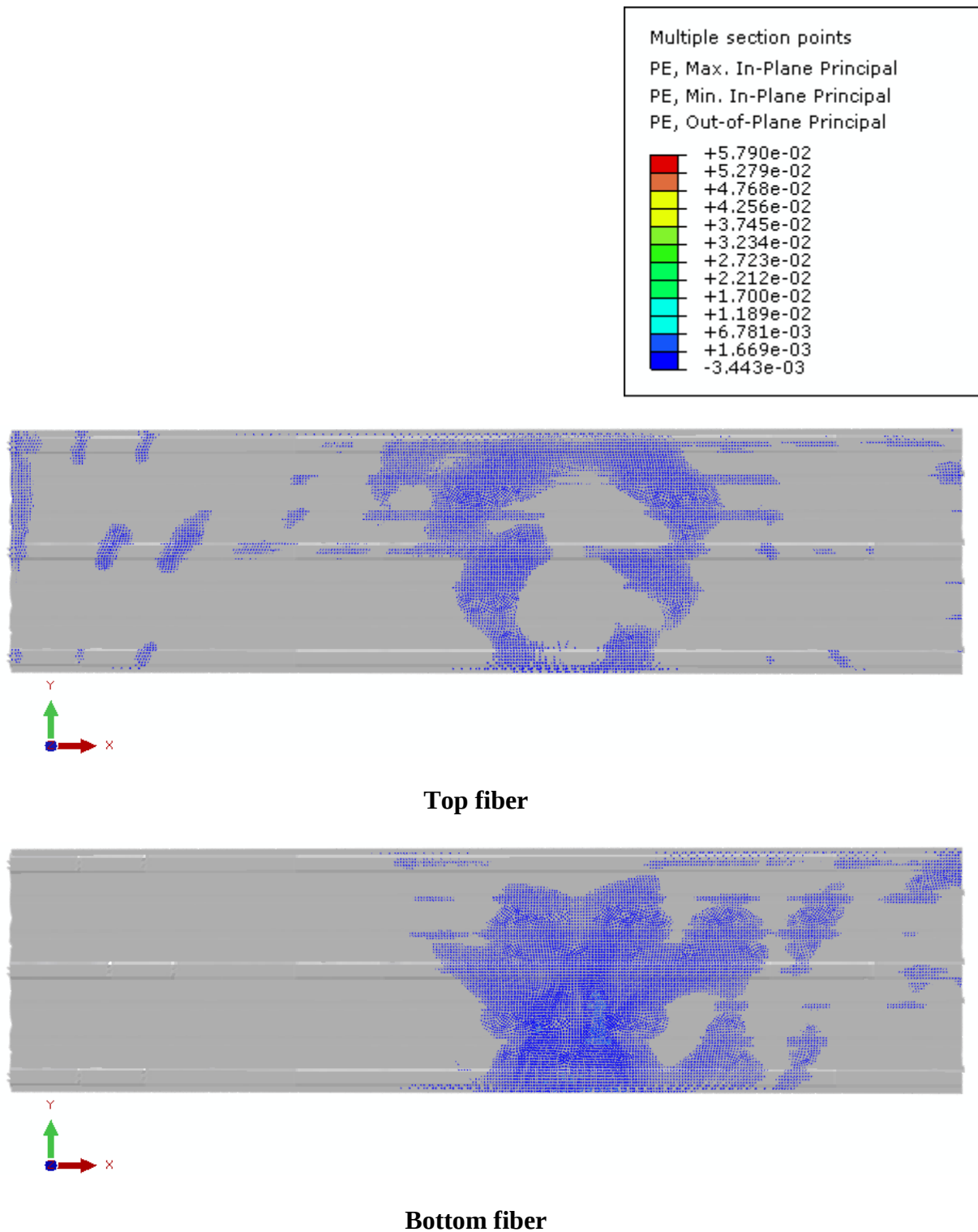


Figure 4-25: Principal plastic strain distribution on slab (Model-2).

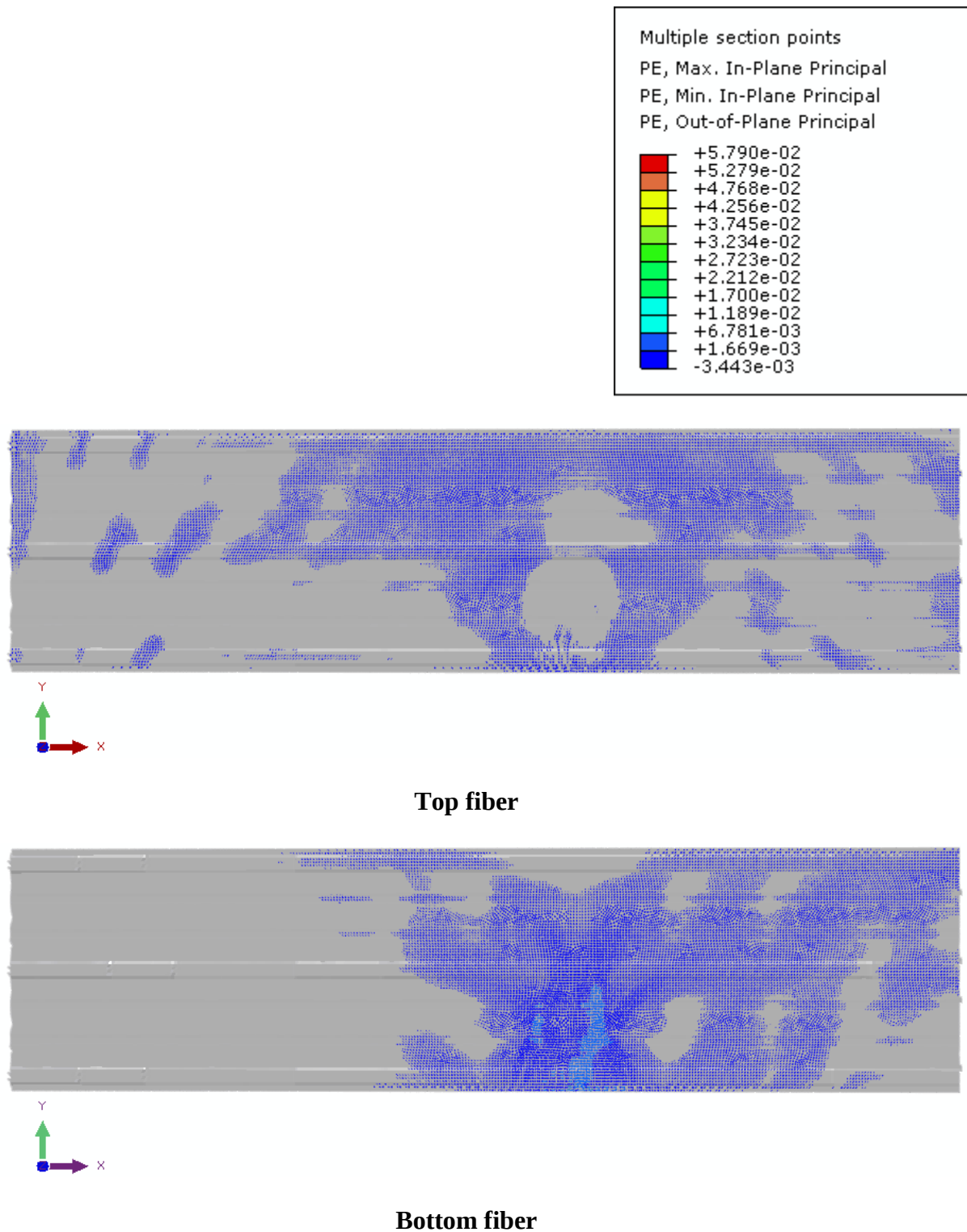


Figure 4-26: Principal plastic strain distribution on slab (Model-3).

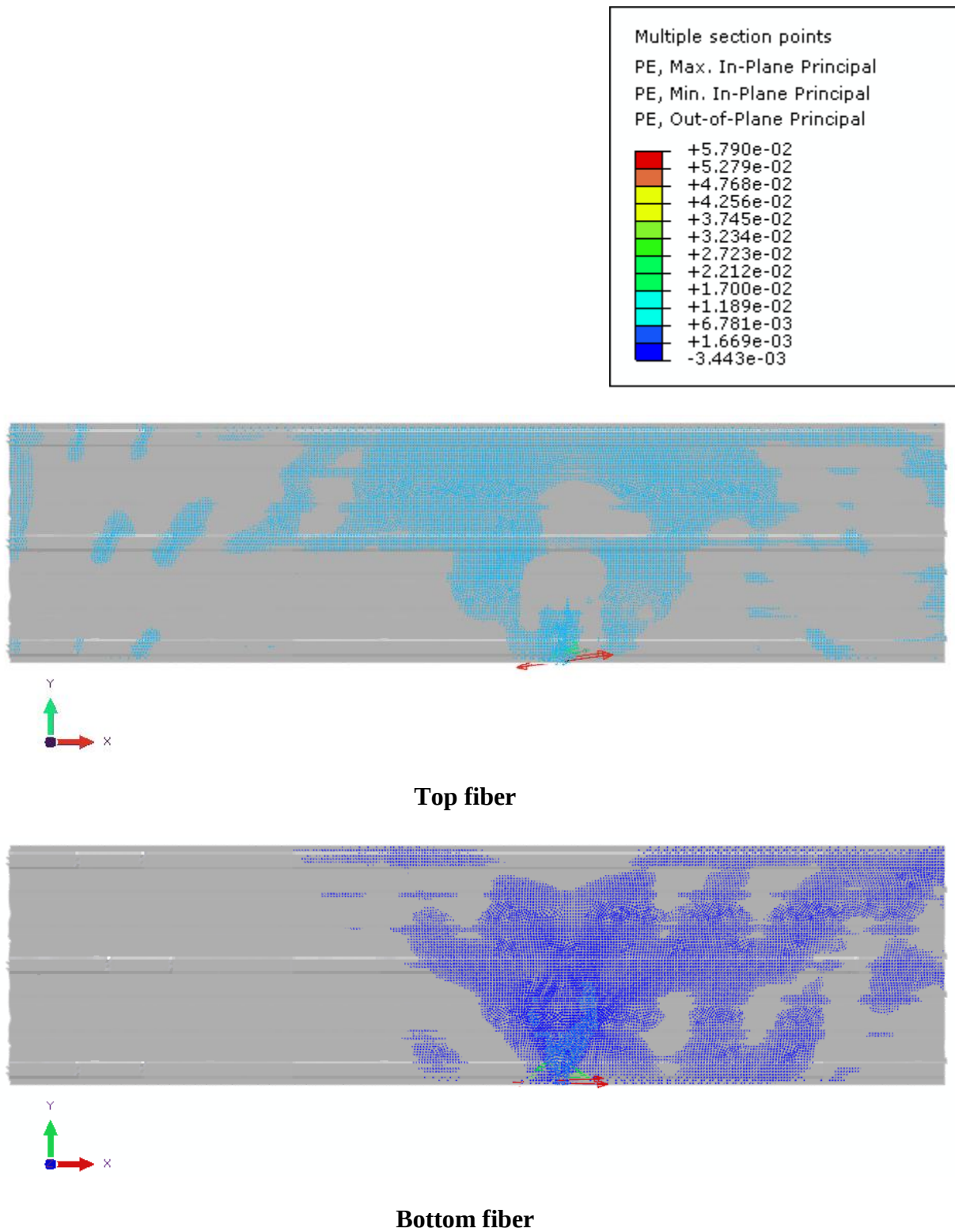


Figure 4-27: Principal plastic strain distribution on slab (Model-4).

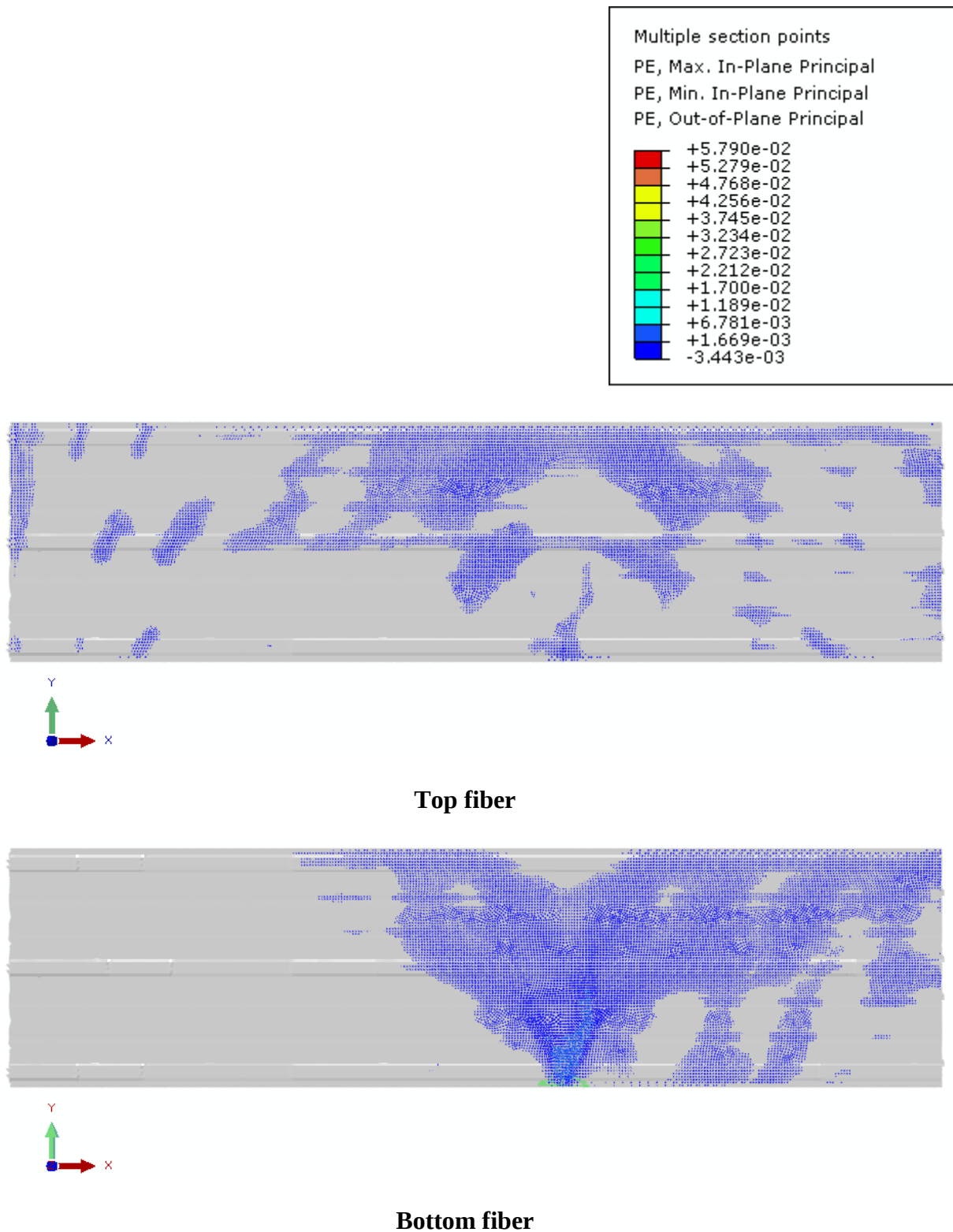


Figure 4-28: Principal plastic strain distribution on slab (Model-5).

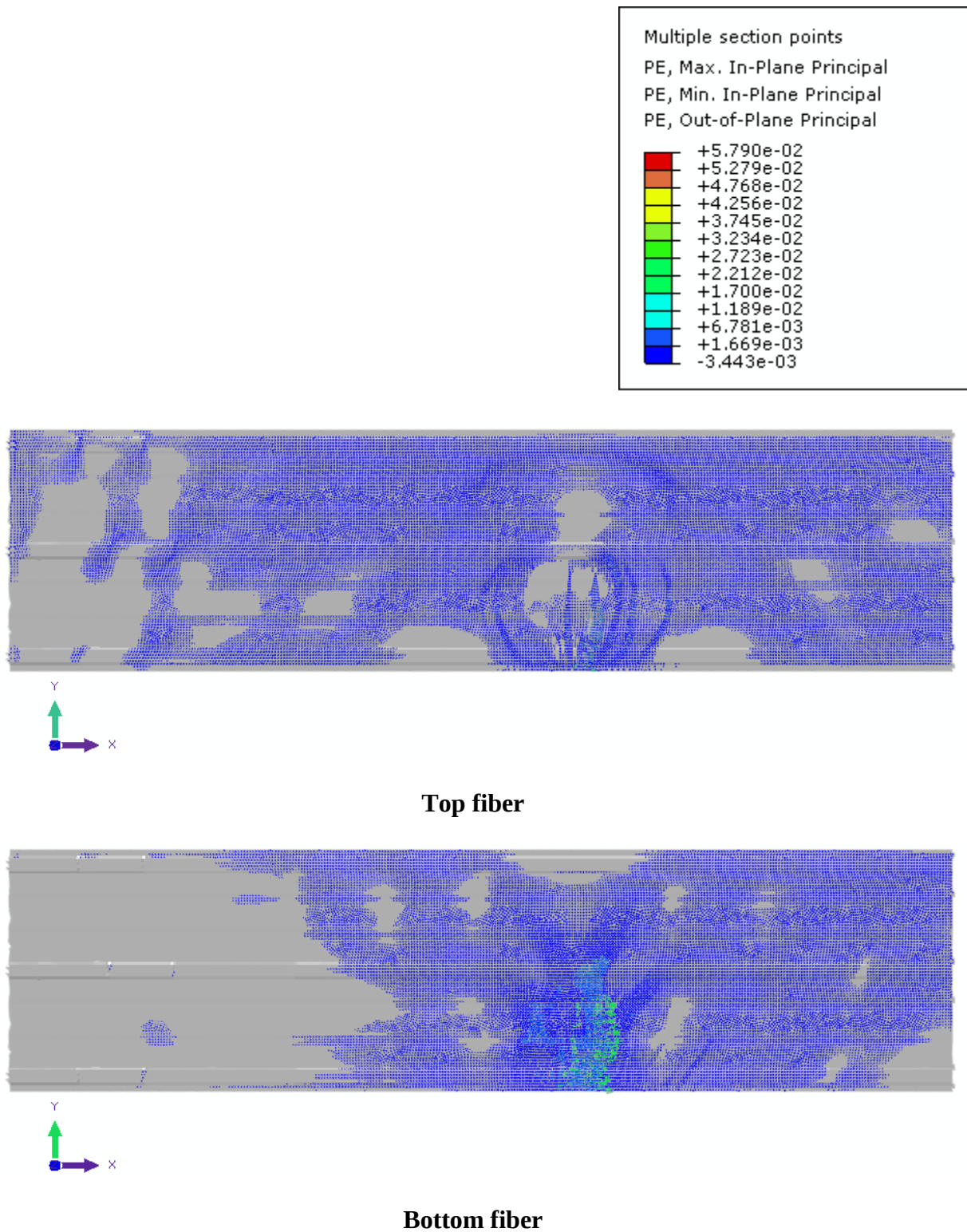


Figure 4-29: Principal plastic strain distribution on slab (Model-6).

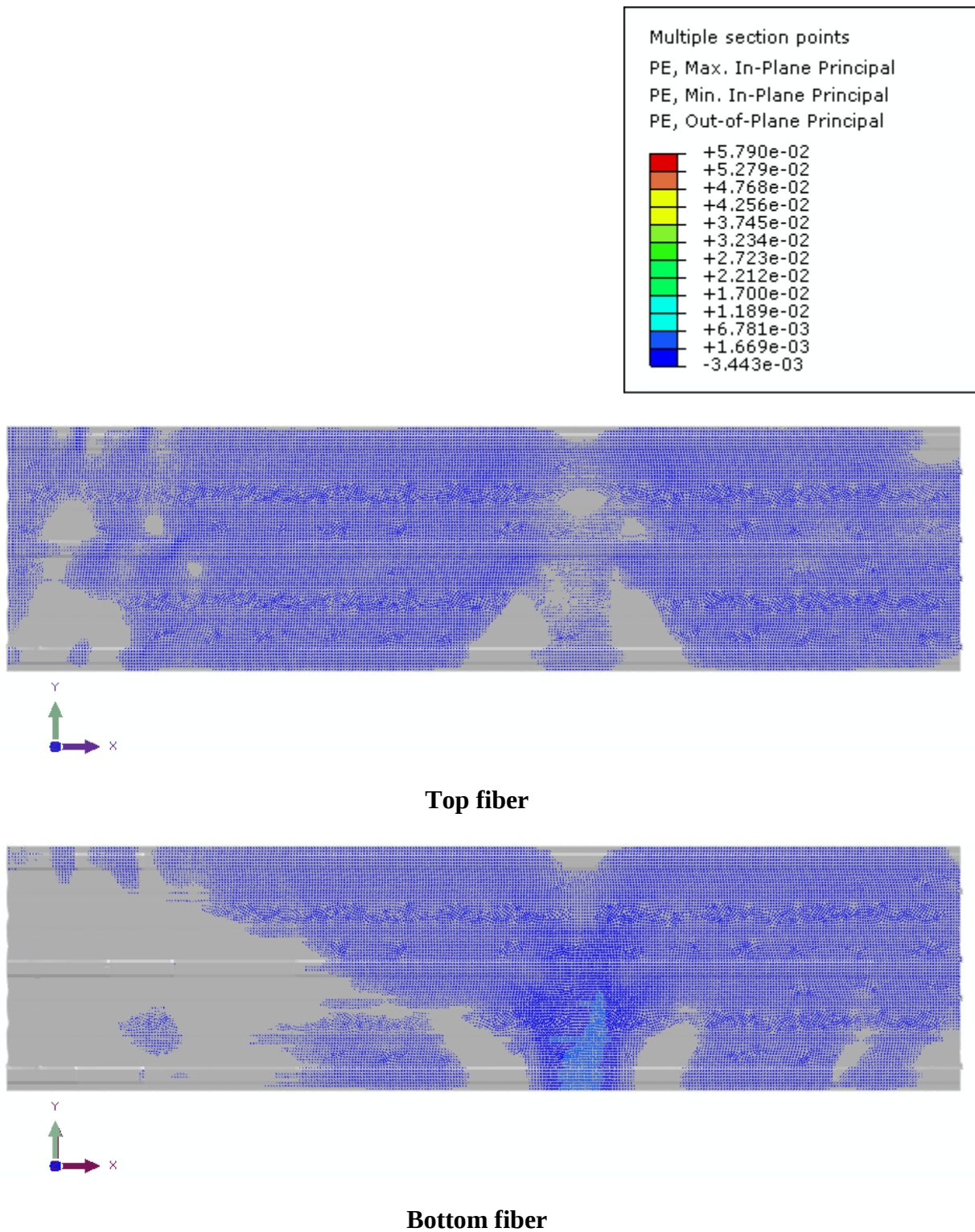


Figure 4-30: Principal plastic strain distribution on slab (Model-7).

4.4.2 Gradual introduction of fracture in quasi-static analysis

In the previous section, fractures in girder E and F are introduced simultaneously at the start of the step 2 using the “Model Change” option. For these cases, the guardrail models using either the smeared crack concrete model or the cast iron plasticity model had convergence difficulties. In addition, the Kinetic Energy (KE) to internal strain energy (IE) ratio was higher (5-10%) compared to typical limit of 5%. Therefore, to assess the validity of analyses in Section 4.4.1 and obtain a converged solution, analyses are carried out by gradually introducing fracture over multiple steps. Model-2 and Model-3 are selected for this analyses. In the first analysis, girder E and F are fractured simultaneously similar to the analyses in the previous section. In the second analysis, fracture is introduced over two steps. The interior girder E is fractured first followed by fracture of exterior girder F in the second step. In the third analysis, fracture is introduced even more gradually over four steps. In the first step bottom flange of girder E is fractured followed by girder E web fracture in the second step. Similarly, in the next two steps, girder F is fractured. The girder displacements at the fracture location at the final and intermediate steps for Model-2 and Model-3 are tabulated in Table 4-8 and Table 4-9 respectively. The KE/IE ratio is also tabulated at those time increments.

The KE/IE ratio decreased when fracture is introduced slowly over multiple steps but both models are unable to complete the full time step whether the fracture is introduced over a single step or multiple steps. The girder displacements at the intermediate and the final steps are similar between Model-2 and Model-3. The final girder displacement is insensitive to the introduction of fracture over multiple steps and therefore the first analysis where girder E and F are fractured simultaneously can be used to investigate different guardrail models in Section 4.4.1.

Table 4-8: Girder displacement at location of fracture (0.4L) for Model-2.

Analysis	Fracture description	0.4L			KE/IE*100
		Girder D (in.)	Girder E (in.)	Girder F (in.)	
1	Girder E and F (Failed at 0.7 time increment)	-7.52	-15.60	-21.40	5.33
2	Girder E	-7.15	-9.38	-7.72	0.19
	Girder F (Failed at 0.65 time increment)	-8.40	-16.55	-20.63	4.14
3	Girder E bottom flange	-6.04	-6.93	-6.46	0.03
	Girder E web	-7.15	-9.29	-7.70	0.05
	Girder F bottom flange	-7.63	-12.30	-13.15	0.61
	Girder F web (Failed at 0.44 time increment)	-8.46	-17.23	-21.84	1.40

(Negative sign indicates downward displacement)

Table 4-9: Girder displacement at location of fracture (0.4L) for Model-3.

Analysis	Fracture description	0.4L			KE/IE*100
		Girder D (in.)	Girder E (in.)	Girder F (in.)	
1	Girder E and F (Failed at 0.8 time increment)	-7.94	-18.30	-25.88	8.17
2	Girder E	-7.12	-9.34	-7.68	0.16
	Girder F (Failed at 0.80 time increment)	-8.86	-20.05	-27.06	5.46
3	Girder E bottom flange	-6.04	-6.93	-6.46	0.03
	Girder E web	-7.15	-9.29	-7.70	0.05
	Girder F bottom flange	-7.62	-12.29	-13.13	0.60
	Girder F web (Failed at 0.56 time increment)	-8.56	-18.90	-25.04	1.92

(Negative sign indicates downward displacement)

4.4.3 Eigenvalue analysis and damping coefficients

In ABAQUS (2013) damping is specified as the part of material definition using Rayleigh damping coefficients. Rayleigh damping coefficients can be determined using the frequencies of the structure. Two frequencies are required for Rayleigh damping coefficient determination, and are typically selected amongst the first few frequencies. In the following parametric study, two different models of the Hoan Bridge are utilized to understand behavior. The first model is the bare girder bridge model that only includes the steel superstructure and excludes the stiffness

contribution of the non-composite slab and guardrail to the response. The second model is the non-composite bridge model that includes the slab and guardrail explicitly and uses a frictional contact definition to model non-composite action between the slab and girder top flange. The frictional contact definition is not updated during eigenvalue analysis. Therefore, the slab in contact with the top flange at the start of the eigenvalue analysis stays in contact through the analysis. Eigenvalue analyses of both the models is carried out with undamaged and damaged (with fractured girder E and F) configurations. The first ten frequencies of bare girder bridge model and non-composite bridge model with undamaged and damaged configuration are tabulated in Table 4-10. The first mode of vibration showed significant reduction in frequency. The rest of frequencies reduced by 0.3-6.9%. Figure 4-31 and Figure 4-32 shows the mode shapes for the first ten eigenvalues for undamaged and damaged bare girder bridge model respectively. Figure 4-33 and Figure 4-34 shows the mode shapes for the first ten eigenvalues for undamaged and damaged non-composite bridge model respectively. For the bare girder bridge model and non-composite bridge model, mode shapes in Figure 4-31 through Figure 4-34 are not the same. Thus, direct comparison of eigenvalues between the bare girder bridge model and non-composite bridge model is not possible.

Table 4-10: Frequencies for the undamaged and the damaged bridge configuration.

Eigen values	Frequency (Hz)			
	Bare girder bridge model		Non-composite bridge model	
	Undamaged bridge configuration	Damaged bridge configuration	Undamaged bridge configuration	Damaged bridge configuration
1	2.298	1.077	1.907	1.336
2	2.343	2.227	2.434	2.427
3	2.744	2.707	2.780	2.587
4	2.857	2.789	2.793	2.753
5	3.189	3.093	3.018	3.001
6	3.331	3.168	3.110	3.066
7	3.335	3.243	3.235	3.126
8	3.346	3.332	3.283	3.185
9	3.370	3.335	3.293	3.235
10	3.386	3.339	3.307	3.278

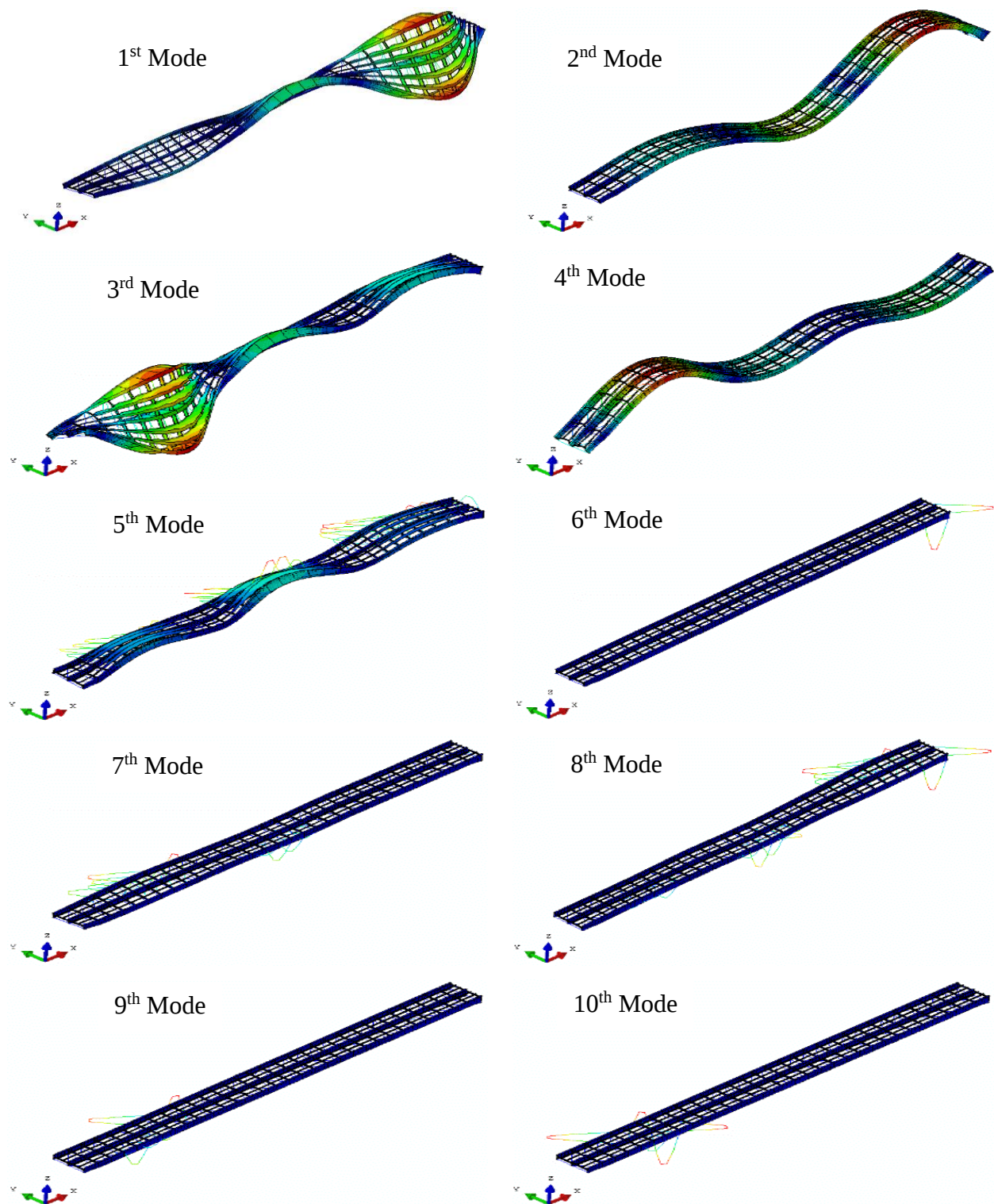


Figure 4-31: Mode shapes (Undamaged bare girder bridge model)

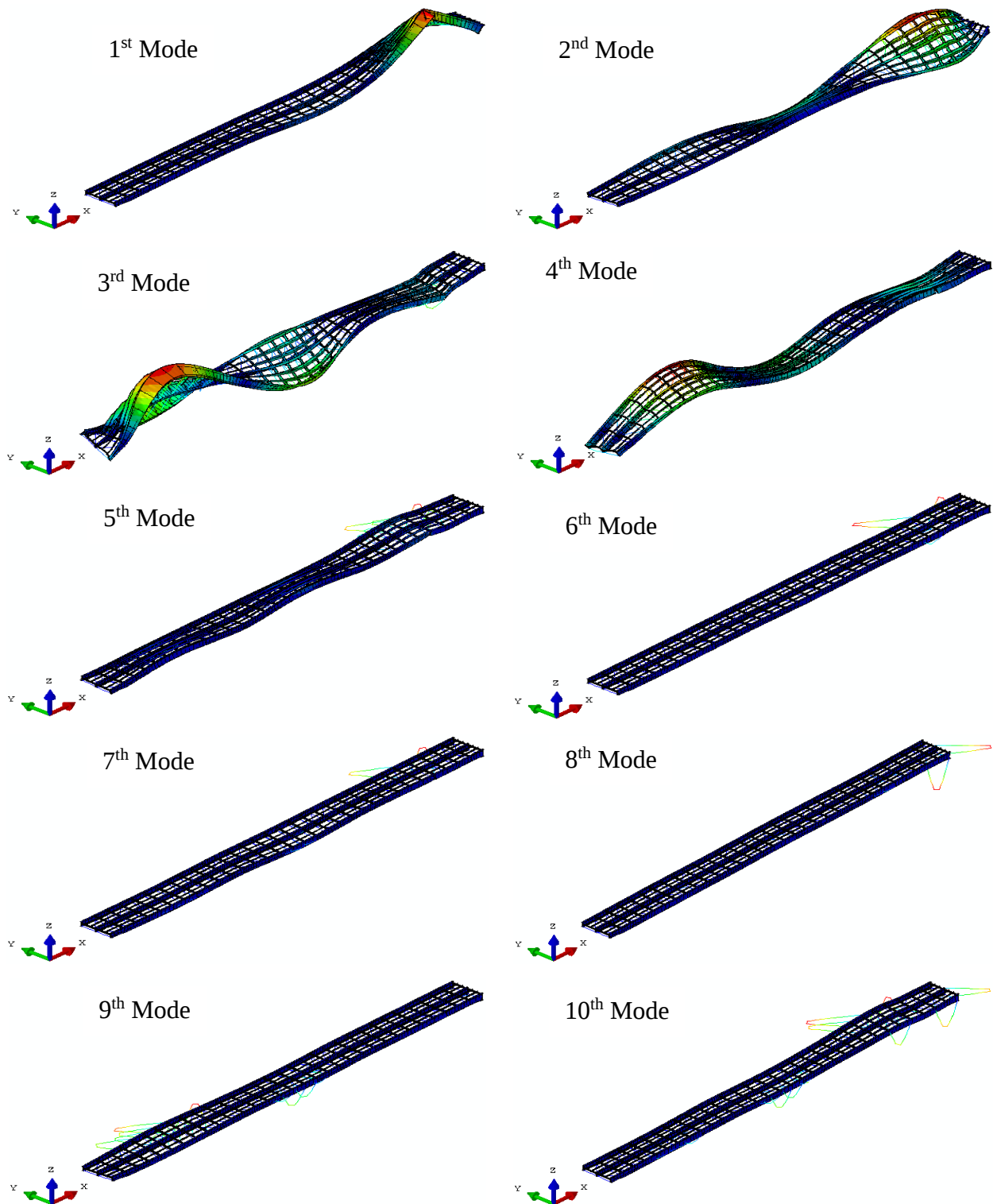


Figure 4-32: Mode shapes (Damaged bare girder bridge model)

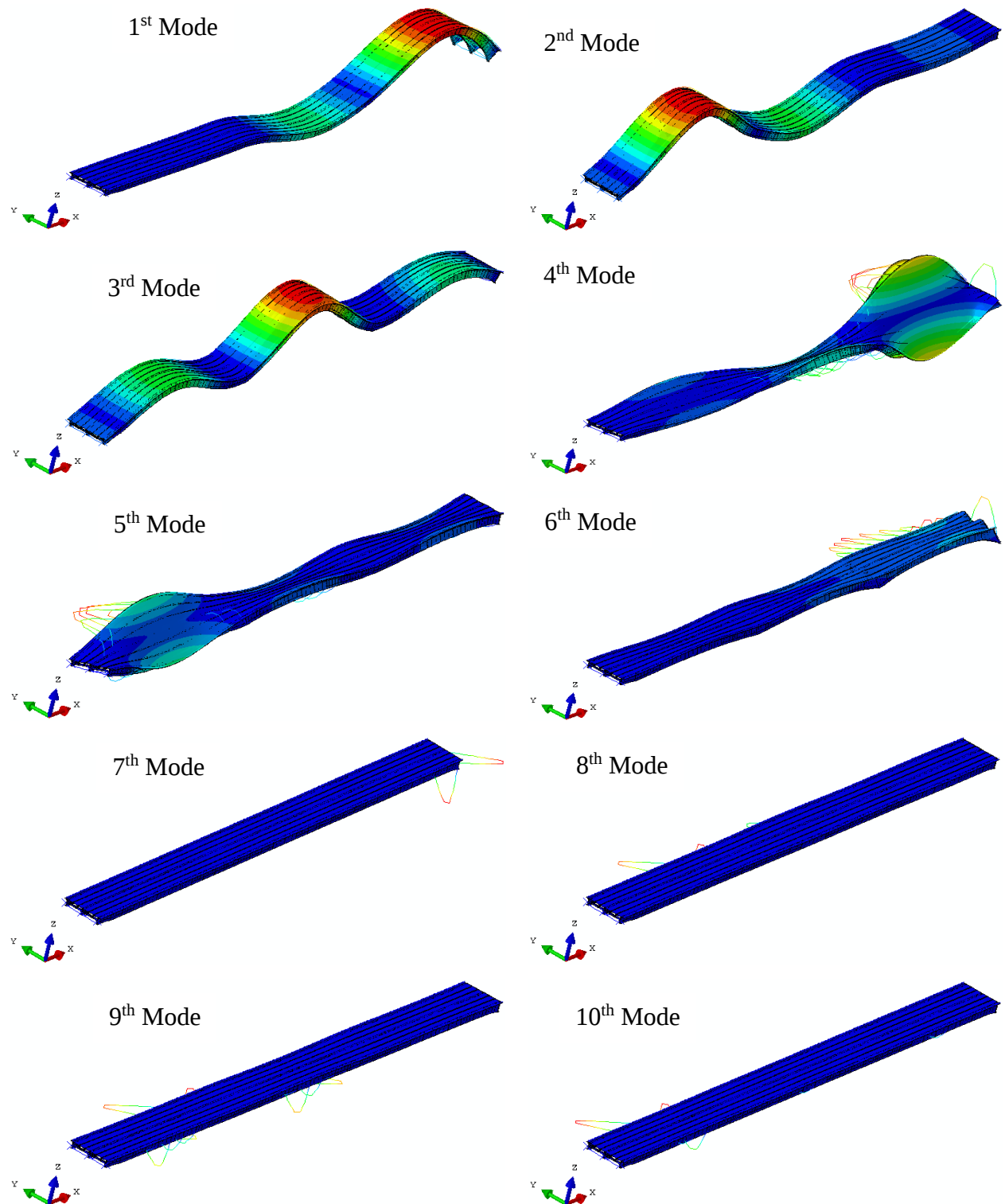


Figure 4-33: Mode shapes (Undamaged non-composite bridge model)

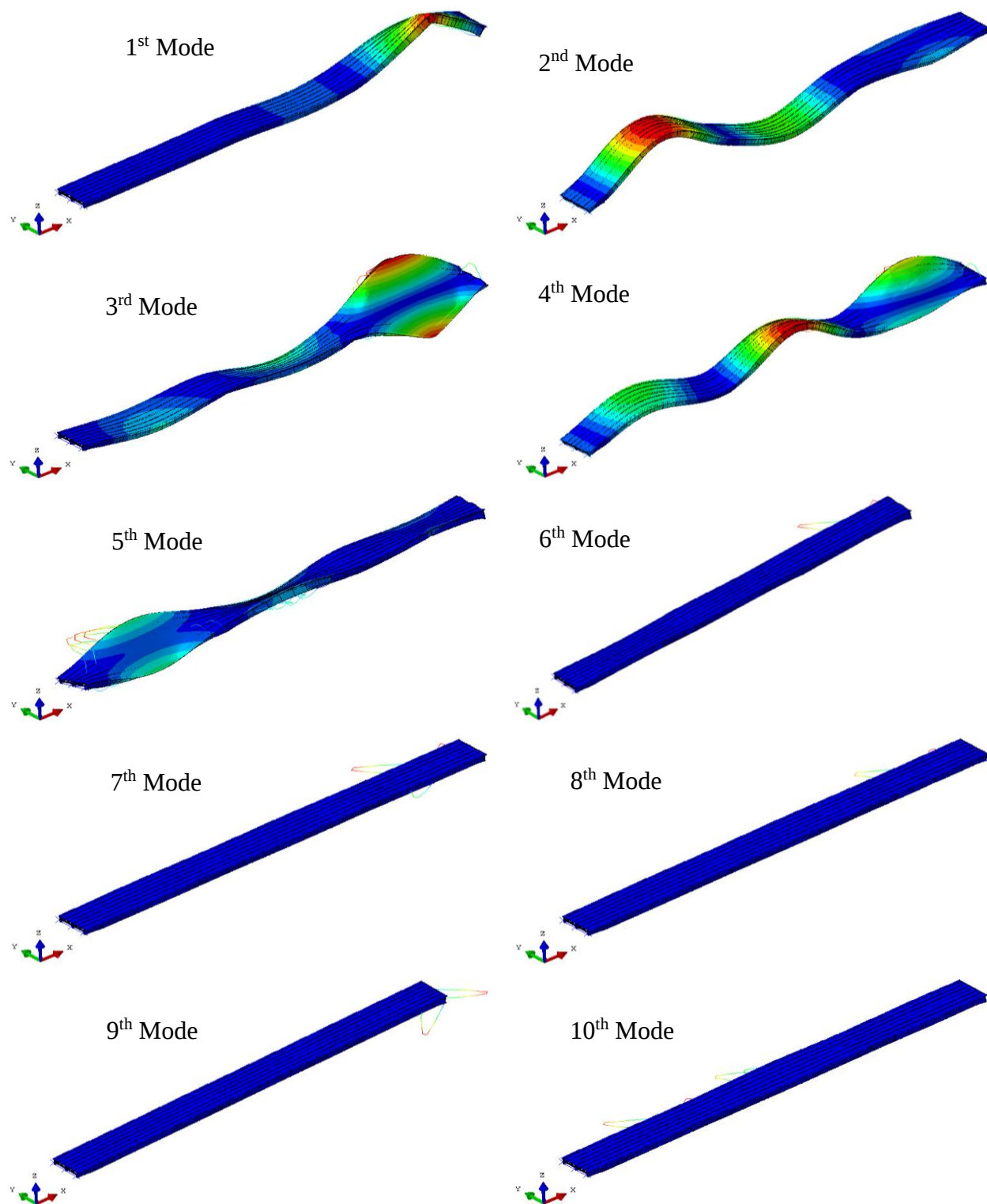


Figure 4-34: Mode shapes (Damged non-composite bridge model)

The first and fourth frequencies from the damaged bridge configuration are used to determine the Rayleigh damping coefficients at 1%, 2% and 5% critical damping and are tabulated in Table 4-11. Additionally the mass proportional damping coefficients are calculated using the first eigenvalue are also tabulated in Table 4-11.

Table 4-11: Damping coefficients using damaged bridge configuration.

Model description	Critical damping	Raileigh damping		Mass proportional damping
		α	β	α
Non-composite bridge model	1%	1.130E-01	8.000E-04	1.679E-01
	2%	2.261E-01	1.600E-03	3.358E-01
	5%	5.652E-01	3.900E-03	8.395E-01
Bare girder bridge model	1%	9.760E-02	8.000E-04	1.353E-01
	2%	1.952E-01	1.600E-03	2.706E-01
	5%	4.880E-01	4.100E-03	6.764E-01

4.5 Parameters affecting dynamic bridge response

To capture the dynamic response of the bridge following fracture event and study the effect of different geometric and analysis parameters on the bridge response, dynamic analysis is carried out using both the *Implicit* and *Explicit* schemes in ABAQUS (2013). The parameters evaluated in this study are as follows,

- Analysis parameters
 - Critical damping ratio
 - Impact of Rayleigh damping coefficient from damaged and undamaged model
 - Solution scheme (*Implicit* and *Explicit* dynamic)
 - Damping definition (Rayleigh damping and mass proportional damping)
 - Impact of coefficient of friction
- Geometric and material parameters
 - Modeling of non-composite bridge
 - Secondary elements (Guardrail and lower lateral bracing)
 - Boundary conditions
 - Concrete strength
 - Reinforcement grade

- Rate dependent material properties
- Composite action between the concrete slab and steel superstructure

Analysis is carried out in series of steps as discussed at the start of Section 4.3. In the first step, dead and live load is applied to the bridge. In the second step, fracture is simulated by deactivating beam connector elements over girder E and F using the “Model Change” option in ABAQUS (2013). In the third step, dynamic response of the structure following fracture is simulated.

4.5.1 Critical damping ratio

Critical damping ratio for bridges is typically evaluated through dynamic testing or live load testing of bridges and this kind of data is not readily available for majority of bridges in service. In the absence of test data, critical damping ratio is assumed based on engineering judgment. Therefore, it is important to understand the effect of critical damping ratio on the post fracture response of the Hoan Bridge. Post fracture response of the Hoan Bridge is simulated using 1%, 2%, and 5% Rayleigh damping with an *Implicit* scheme.

Figure 4-35 shows the response of the exterior girder F at the location of fracture (i.e. 0.4L from Pier 2S) following the fracture event. Deflections in all three girders at the end of analysis (6.00 sec) are tabulated in Table 4-12. Response of the intact girder D increased by 0-3% while fracture girder E and F response decreased by 6-20%. For the first 0.5 sec, the fractured girder F deflection is identical for different damping ratios. It begins to diverge after 0.5 sec, with analysis with lower damping ratio yielding higher deflection. Figure 4-36 shows the response of intact girder D at the location of fracture (i.e. 0.4L from Pier 2S).

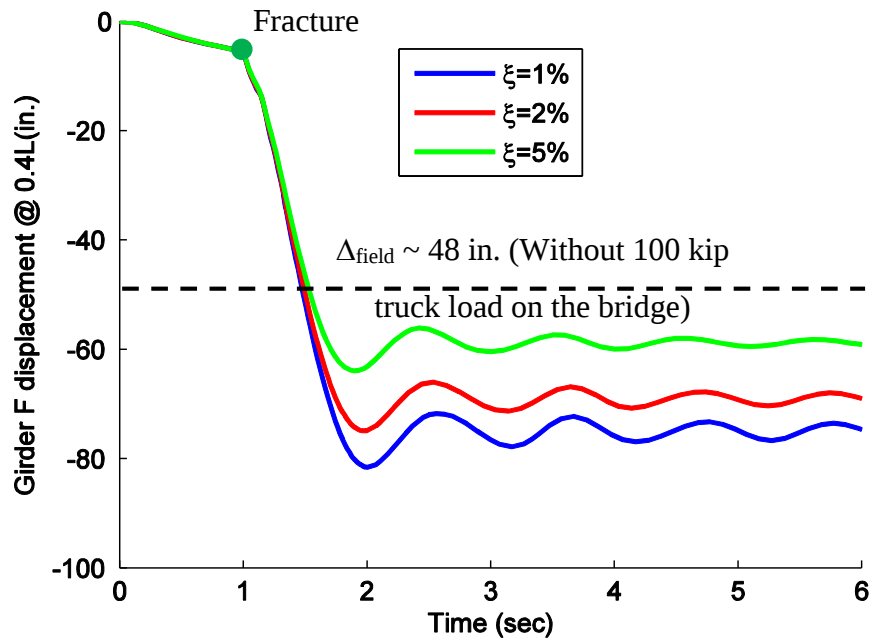


Figure 4-35: Girder-F response following fracture event at the location of fracture (0.4L from Pier 2S) for 1%, 2% and 5% critical damping using Rayleigh damping with *Implicit* scheme.

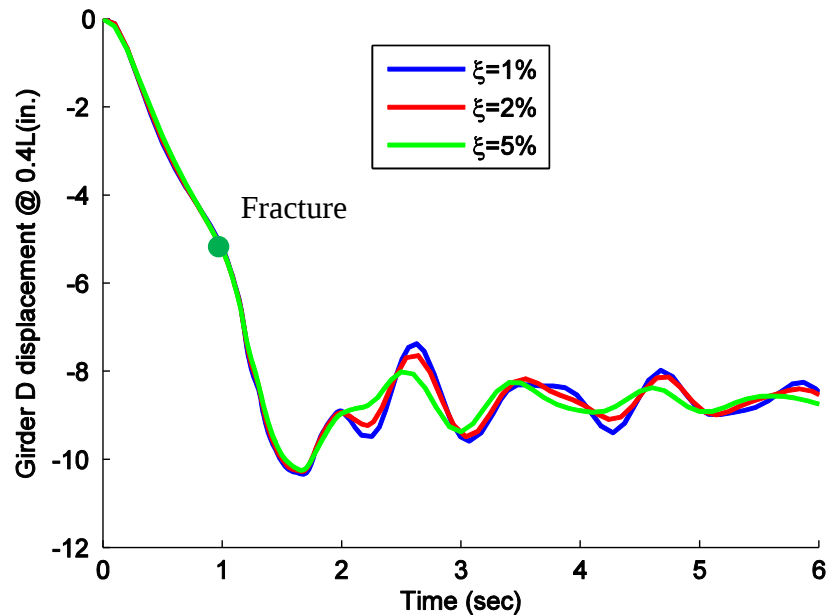


Figure 4-36: Girder-D response following fracture event at the fracture location (0.4L from Pier 2S) for 1%, 2% and 5% critical damping using Rayleigh damping with *Implicit* scheme.

Table 4-12: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis.

Critical damping	At fracture location (0.4L)		
	Girder D (in.)	Girder E (in.)	Girder F (in.)
1%	-8.47	-43.57	-74.64
2%	-8.53 (0.7%)	-40.68 (-6.6%)	-68.97 (-7.6%)
5%	-8.74 (3.2%)	-35.78 (-17.9%)	-59.10 (-20.8%)

(Negative sign indicates downward displacement)

4.5.2 Impact of Rayleigh damping coefficients from damaged and undamaged model on post fracture response

The eigenvalue analyses of the undamaged and damaged model of the Hoan Bridge showed a reduction in first frequency of 29.9% while the next nine frequencies are reduced by only 0.3-6.9% (Section 4.4.3). To investigate the impact of Rayleigh damping coefficients using frequencies from the damaged and the undamaged bridge models, an analysis is carried out with 5% critical damping using an *Implicit* scheme. Damping coefficients are calculated using the frequencies for the 1st and 4th mode for the damaged and the undamaged bridge and are tabulated in Table 4-13. In the Rayleigh damping definition, the mass proportional coefficient increased by 45.9% while stiffness proportional decreased by 17.1 %. This possibly resulted in offsetting their impact on deflection of girder F resulting in to only 1.75 % reduction in deflection when Rayleigh damping coefficient based on the undamaged bridge model is used. Although the differences in results from analyses using Rayleigh damping coefficients based on the damaged or undamaged bridge model are insignificant, damping coefficients based on the damaged bridge model will be used in parametric study.

Table 4-13: Damping coefficients for non-composite bridge model.

Model description	Critical damping ratio	Rayleigh damping	
		α	β
Damaged bridge model	5%	0.488	0.004
Undamaged bridge model		0.712 (45.9%)	0.0034 (-17.1%)

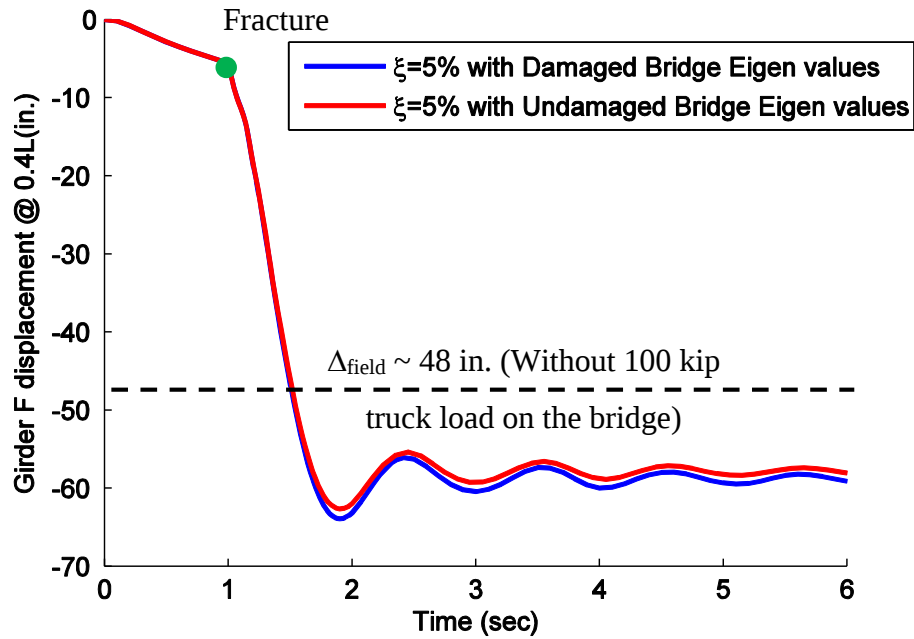


Figure 4-37: Comparison between girder F response at 0.4L (from Pier 2S) for model with Rayleigh damping coefficients ($\xi = 5\%$) based on damaged and undamaged bridge model using *Implicit* scheme.

4.5.3 Effect of solution scheme and damping definition on dynamic response

ABAQUS (2013) offers *Implicit* and *Explicit* schemes to perform dynamic analysis. The ABAQUS *Implicit* scheme uses a Hilber-Hughes-Taylor time integration scheme to step through time. A set of nonlinear equilibrium equations are solved at each time increment using Newton's method. The *Implicit* scheme is unconditionally stable and thus there is no limit on the time increment size. The ABAQUS/*Explicit* scheme uses a staggered central difference rule to

calculate displacements and velocities in terms of quantities known from the previous increment. It uses a lumped mass matrix and does not need the formation of global stiffness and mass matrices. Therefore, each increment is relatively inexpensive compared to the increment using an *Implicit* scheme. However, the size of a stable time increment in an *Explicit* scheme is small and is a function of highest frequency and critical damping ratio. A minimum value for a stable time increment can be calculated using Equation 4-1 and Equation 4-2 for analysis without and with damping respectively.

$$\Delta t \leq \frac{2}{\omega_{\max}} \quad \text{Equation 4-1}$$

$$\Delta t \leq \frac{2}{\omega_{\max}} \left(\sqrt{1 + \xi^2} - \xi \right) \quad \text{Equation 4-2}$$

The stable time increment calculated by ABAQUS (2013) for the undamped, non-composite model of the Hoan Bridge is 10^{-5} sec., which corresponds to the highest frequency ($\omega_{\max}$) of 200,000 Hz based on Equation 4-1. The present simulation is not trying to capture the response associated with the highest frequency of 200,000 Hz. The frequency of 200,000 Hz is included in the following discussion to understand the impact of mass and stiffness proportional damping coefficients on the stable time increment. The use of 5% Rayleigh damping yields a critical damping (ξ) of 2450 at 200,000 Hz. For critical damping (ξ) of 2450, the stable time increment reduces to 2.04×10^{-9} sec using Equation 4-2. Conversely use of 5% mass proportional damping coefficient yields critical damping (ξ) of 3.34×10^{-7} at the highest frequency ($\omega_{\max}$) of 200,000 Hz. This critical damping at $\omega_{\max}$ results in a stable time increment of 9.99×10^{-6} sec ($\sim 10^{-5}$ sec) using Equation 4-2. Use of mass proportional damping results in little to no reduction in the stable time increment in the *Explicit* analysis. Therefore, mass proportional damping will be used for the *Explicit* scheme in order to keep the total run time less than 24 hours.

Figure 4-38 shows the response of girder F at the location of fracture (0.4L from Pier 2S) using *Implicit* and *Explicit* solution schemes for 1%, 2% and 5% critical damping. Fracture occurs at the start of 1 sec. Girder F response using both *Implicit* and *Explicit* solution schemes is similar up to 1.4 sec. After 1.4 sec, the *Explicit* scheme response begins to diverge and results in considerably higher deflection compared to the *Implicit* scheme. With the *Implicit* scheme solution, fractured girder F deflection reduces with increase in critical damping. This behavior

can be attributed to stiffness proportional damping coefficient in Rayleigh damping that tends to overdamp higher modes. Increasing damping from 1% to 2% showed little reduction in response with *Explicit* scheme. But when damping was increased to 5% damping, the girder F deflection was reduced by 19.5 %. This behavior can be attributed to the mass proportional damping coefficient that tends to underdamp higher modes but it can generate extraneous damping forces if the total motion involves rigid body motion. The deflection at the end of analysis using the *Explicit* scheme is 25-30% higher as compared to the deflection using the *Implicit* scheme. The difference in response with *Implicit* and *Explicit* scheme is further investigated in Section 4.5.4.

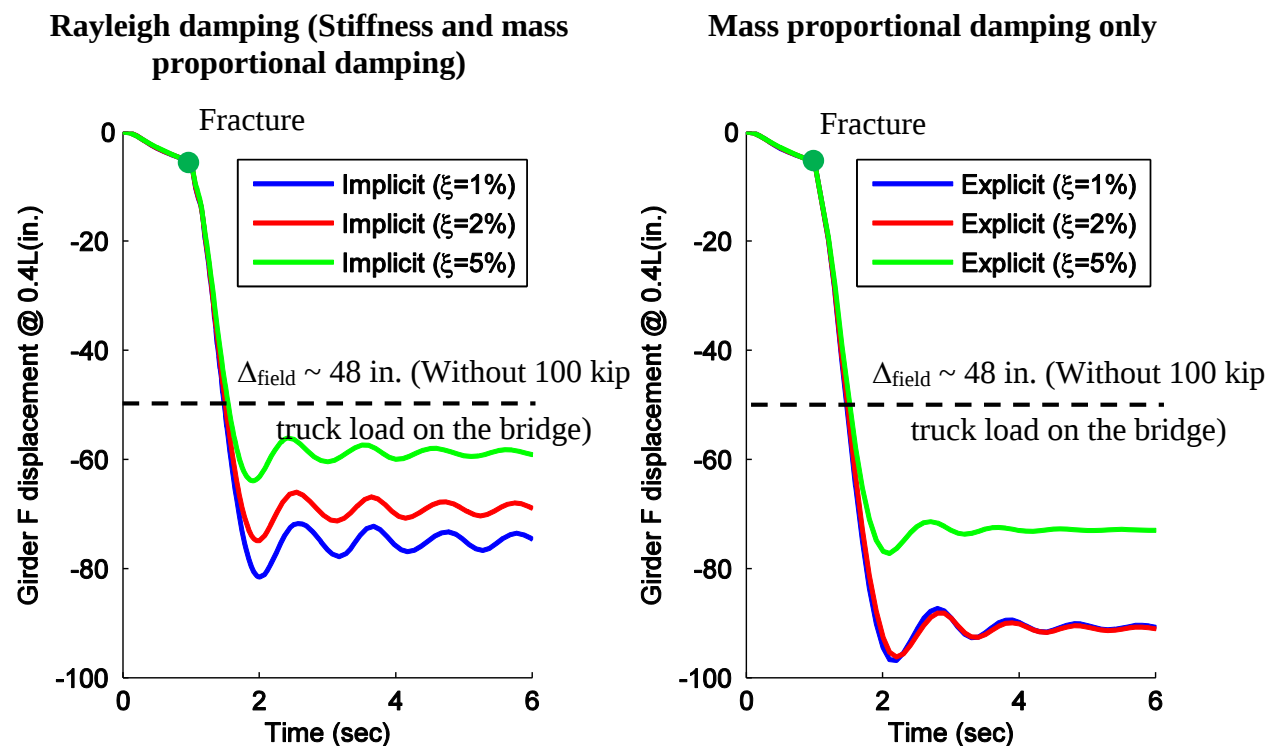


Figure 4-38: Comparison between Girder-F response at 0.4L (from Pier 2S) with *Implicit* and *Explicit* solution scheme for 1%, 2% and 5% critical damping.

Table 4-14: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of 5 sec following fracture event.

Critical damping	Analysis type	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
1%	Implicit	-8.47	-43.57	-74.64
	Explicit	-8.76	-51.86	-90.74
2%	Implicit	-8.53	-40.68	-68.97
	Explicit	-8.71	-51.85	-91.04
5%	Implicit	-8.74	-35.78	-59.10
	Explicit	-8.62	-42.77	-73.00

(Negative sign indicates downward displacement)

Figure 4-39 shows the girder F response at 0.4L (from Pier 2S) during the first second for model with 1% critical damping. The deflections begin to deviate after 0.4 sec. To understand deviation in response, the resultant velocity magnitude ($V = \sqrt{V_1^2 + V_2^2 + V_3^2}$) distributions at 0.4 sec after fracture was compared. The maximum resultant velocity from the *Implicit* analysis is 147.5 in/s compared to 165.4 in/s for the *Explicit* analysis (12 % higher). In addition, the resultant velocity magnitude distribution near fractured region is higher for the *Explicit* scheme solution compared to the *Implicit* scheme solution. Potentially this difference in velocity would affect the effective damping and explain the differences in response.

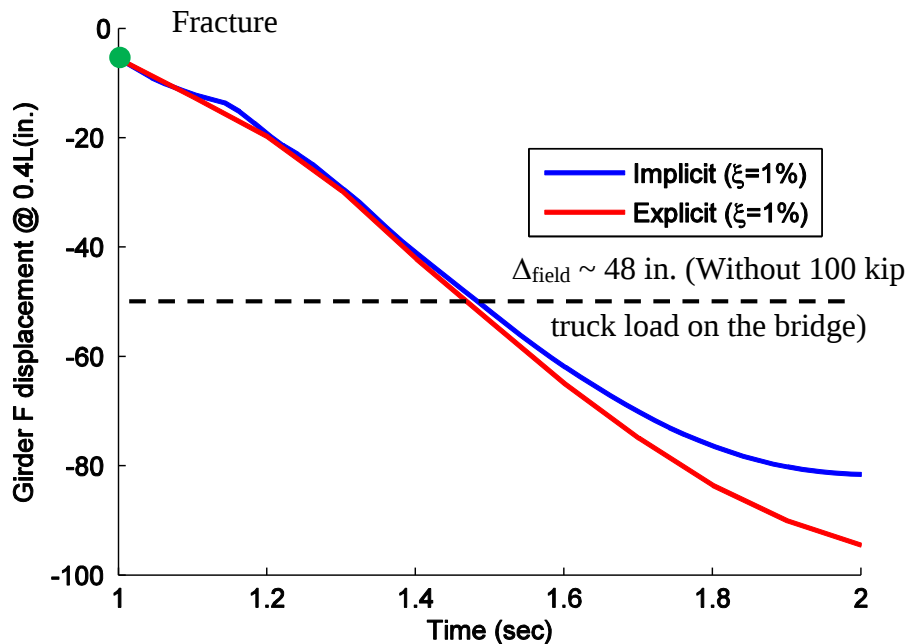


Figure 4-39 – Girder F response at 0.4L (from Pier 2S) for the non-composite bridge model using *Implicit* and *Explicit* solution scheme.

4.5.4 Impact of damping definition on post fracture dynamic response using *Implicit* and *Explicit* scheme

In the previous section, the *Implicit* and *Explicit* schemes yielded different response. Along with different solution schemes used for the analysis, the only other difference in the model definition is the damping definition. In addition, there are other sources of damping in the FEA model (hysteretic damping/energy dissipation from material yielding, coulomb damping/energy dissipation from friction at the steel-concrete interface) along with the damping definition. To understand whether the difference in response is because of the solution scheme or damping definition or both, different sources of damping are isolated and analyzed using both the *Implicit* and *Explicit* schemes. Results from this analysis are presented below.

In order to separate the sources of damping, the bare girder model (i.e. steel superstructure only model) is analyzed first because use of only mass proportional damping in the *Implicit* scheme took significantly longer to analyze in the non-composite model with slab. The longer times were probably due to iterations related to the contact definition at the steel-concrete interface. In addition, for a non-composite bridge model, there is no clean way to separate energy dissipation

from frictional contact. The bare girder analysis is carried out using both the *Implicit* and *Explicit* schemes. Displacement response of girder F at fracture location (0.4L from Pier 2S) is used to discuss the results.

Similar to previous dynamic analysis the analysis is carried out over three steps,

- Step 1: Application of dead load and live load using static analysis (0-1 sec).
- Step 2: Introduction of girder E and F fracture using dynamic analysis (1-1.001 sec).
- Step 3: Post fracture response using dynamic analysis (1.001 – 3 sec).

To isolate the effect of damping definition and material plasticity, four models are analyzed using *Implicit* and *Explicit* schemes and they are as follows:

1. Model-1 with elastic material properties and no damping
2. Model-2 with elastic material properties and 5% damping (*Implicit* scheme: Rayleigh damping and mass proportional damping; and *Explicit* scheme: Mass proportional damping)
3. Model-3 with non-linear material properties and no damping
4. Model-4 with non-linear material properties and 5% damping (*Implicit* scheme: Rayleigh damping and *Explicit* scheme: Mass proportional damping)

Figure 4-40 shows the displacement response of girder F following fracture event at 0.4L (from Pier 2S) using both the *Implicit* and *Explicit* scheme with no damping and elastic material properties. There is no difference between the *Implicit* and *Explicit* cases for this benchmark case.

To understand the effect of the damping definition and solution scheme together, three analyses are carried out with Model-2 and as follows;

1. *Implicit* analysis with Rayleigh damping ($\xi=5\%$)
2. *Implicit* analysis with mass proportional damping ($\xi=5\%$)
3. *Explicit* analysis with mass proportional damping ($\xi=5\%$)

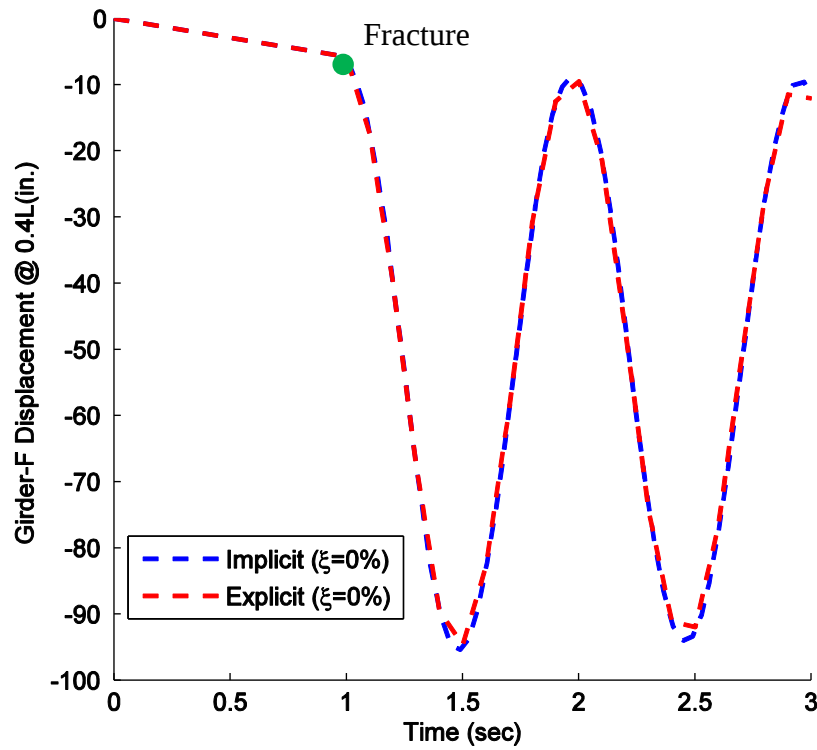


Figure 4-40: Comparison between *Implicit* and *Explicit* response for Model-1 with linear elastic material properties and no damping.

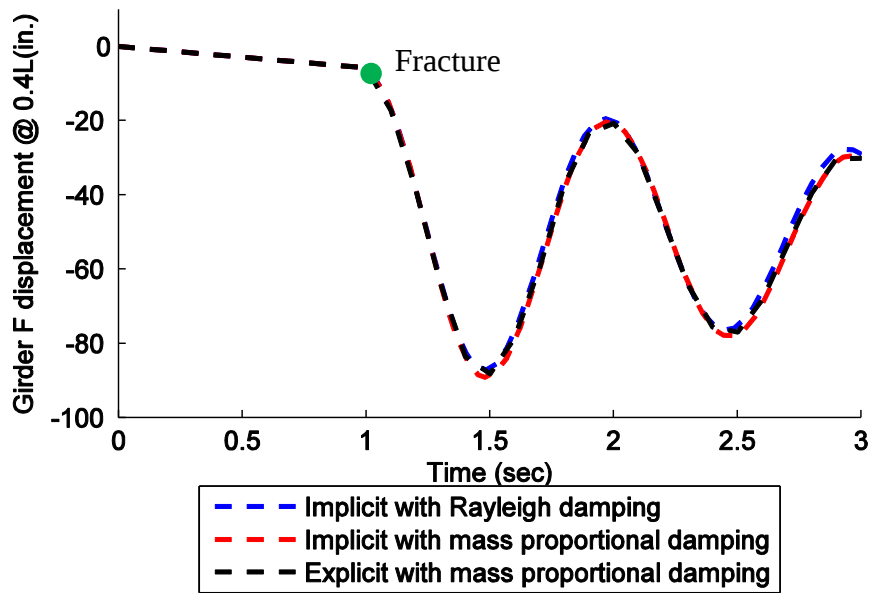


Figure 4-41: Effect of damping definition and solution scheme on Model-2.

Both the *Implicit* and *Explicit* solution schemes yielded identical response irrespective of damping definition (Figure 4-41). To understand impact of damping definition on a model with material plasticity definition, Model-3 and Model-4 are analyzed with no damping and 5% critical damping. To isolate the energy dissipation that occurs from material plasticity, the undamped bare girder model (Model-3) is analyzed first with non-linear material properties. The *Implicit* scheme took significantly longer to analyze the post fracture response. The *Implicit* scheme took approximately 20 hours to analyze the model for 0.31 sec following fracture. Therefore, it was decided to terminate the analysis at the end of 20 hours. In contrast, the *Explicit* scheme successfully completed analysis in less than 20 hours. Figure 4-42 shows the response of girder F at 0.4L (from Pier 2S). Up to 0.31 sec, the *Implicit* and *Explicit* schemes yield identical response. Model-4 includes a material plasticity definition with 5% damping. The *Implicit* scheme used the Rayleigh damping definition while the *Explicit* scheme used the mass proportional damping. Figure 4-43 shows the displacement response of girder F at the location of fracture (0.4L from Pier 2S) with the *Explicit* scheme, fractured girder deflections continued to increase (girder F reached a downward displacement of 260 in.) and analysis failed around 1.02 sec with buckling in girder E and F around 0.2-0.3L. In Model-4, use of Rayleigh damping along with material plasticity definition showed a reduction of 55% in Girder-F deflection. This behavior is consistent with the behavior observed in Section 4.5.3.

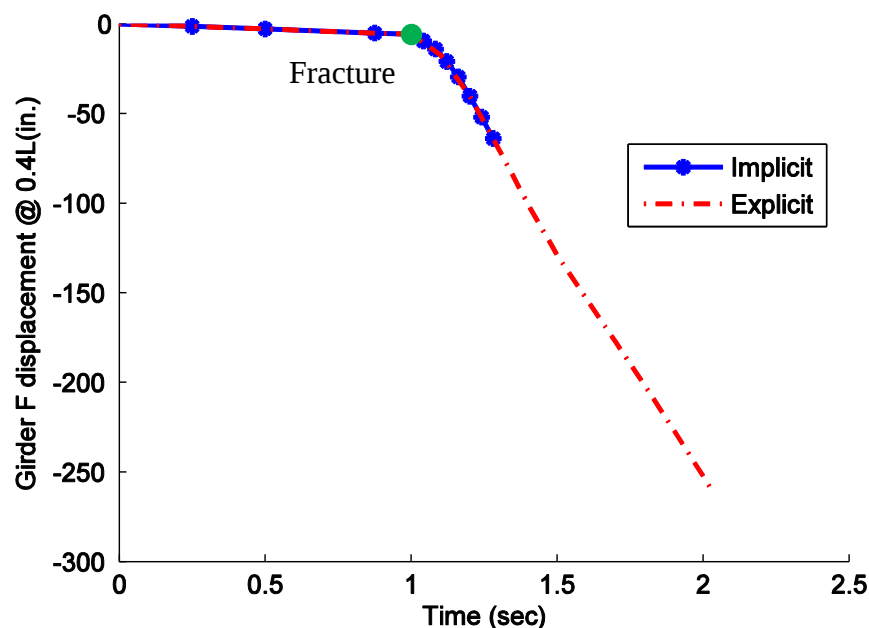


Figure 4-42: Comparison between *Implicit* and *Explicit* response for Model-3.

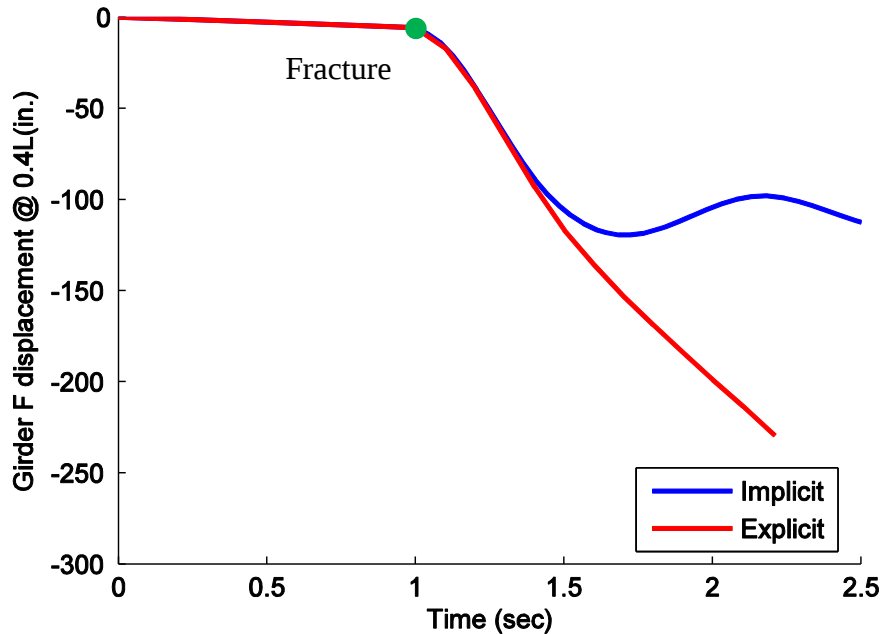


Figure 4-43: Comparison between *Implicit* and *Explicit* response for Model-4.

Results of this study show that:

1. The stiffness proportional coefficient of Rayleigh damping tends to overdamp higher modes when a material plasticity definition is included in the model and this can result in unrealistic damping;
2. Mass proportional damping conservatively underdamps higher mode effects but it can generate extraneous damping forces if the total motion involves rigid body motion;
3. In linear dynamic analysis, 2-5% Rayleigh damping can be used to account for sources of damping (hysteretic damping/energy dissipation from material yielding, coulomb damping/energy dissipation from friction at steel-concrete interface, energy dissipated at the connections) that are not explicitly modeled in the analysis. However, use of 2-5% Rayleigh damping in non-linear dynamic would yield overdamped results.

4.5.5 Impact of coefficient of friction

Non-composite action at the steel-concrete interface in the Hoan Bridge is modeled using surface-to-surface contact at the interface. The normal behavior is modeled using hard contact that minimizes the penetration of surfaces and does not allow for the transfer of tensile stresses across the interface. The tangential behavior is modeled using an isotropic coulomb friction

model. The isotropic coulomb friction definition requires a coefficient of friction at the interface. There is limited data available in the literature for this coefficient of friction and it ranges from 0.20 to 0.80 (Section 3.2.4.3). Therefore, it is necessary to understand the effect of coefficient of friction over a wide range of values on bridge response. The Hoan Bridge model is analyzed with frictional coefficient of 0.2, 0.35, 0.57, 0.70 and 0.80 to address this issue. The post fracture response is simulated using the *Explicit* scheme for 2 sec to keep the run time small. Figure 4-44 shows the post fracture displacement response of the intact girder D at the location of fracture (0.4L from Pier 2S) for different values of coefficient of friction. Increase in amplitude of oscillation of girder D in the second cycle might be because of additional slip at the steel-concrete interface. While the fractured girder F's peak displacements decreased with increase in the value of friction coefficient except for the analysis with frictional coefficient of 0.70 (Table 4-15 and Figure 4-45). Increasing frictional coefficient from 0.57 to 0.70 resulted in increase of ~1.5% in peak girder deflections at the location of fracture. Analysis with friction coefficient 0.57 and higher showed little change in peak girder deflection compared to analyses with frictional coefficient of 0.20 and 0.45. Additionally, Rabbat and Russell (1985) recommended value of 0.57 for coefficient of friction at dry interface. Therefore, a frictional coefficient of 0.57 is used for the remainder of the parametric study.

Table 4-15: Peak girder displacement at the location of fracture (0.4L from Pier 2S) for different coefficient of friction.

μ	At fracture location (0.4L)		
	Girder D (in.)	Girder E (in.)	Girder F (in.)
0.20	-10.69	-68.27	-120.47
0.35	-10.52 (-1.7%)	-61.81 (-9.5%)	-108.21 (-10.2%)
0.57	-10.08 (-5.7%)	-55.51 (-18.7%)	-96.61 (-19.8%)
0.70	-10.20 (-4.6%)	-56.10 (-17.8%)	-98.09 (-18.6%)
0.80	-10.22 (-4.4%)	-54.11 (-20.7%)	-93.97 (-22.0%)

(Negative sign indicates downward displacement)

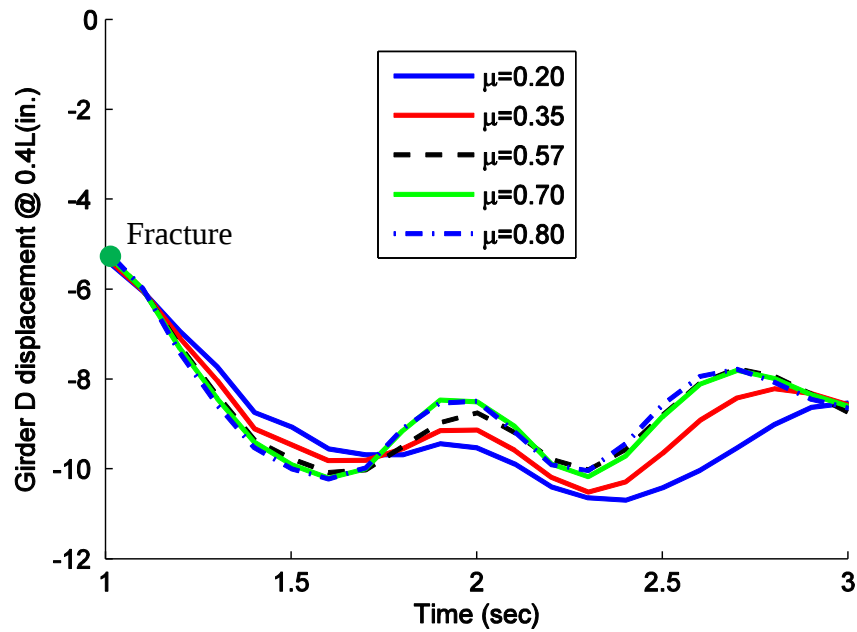


Figure 4-44: Comparison between Girder D response at 0.4L (from Pier 2S) following fracture event for different coefficient of friction using *Explicit* scheme.

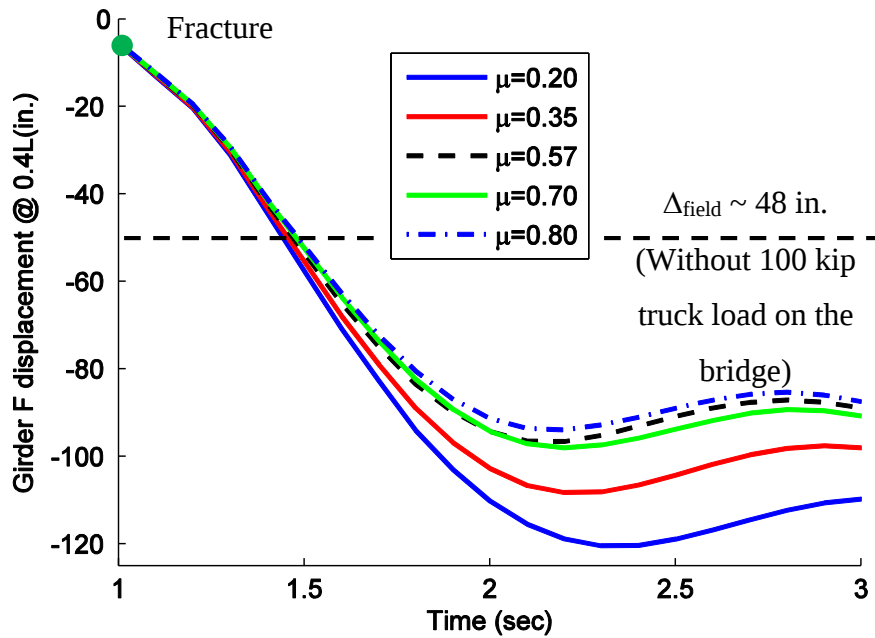


Figure 4-45: Comparison between Girder F response at 0.4L (from Pier 2S) following fracture event for different coefficient of friction using *Explicit* scheme.

4.5.6 Modeling of non-composite bridge

Two approaches are considered for modeling of non-composite bridges. In the first approach (bare girder model), the stiffness contribution of the bridge slab and the guardrail is excluded from the model. The weight of the bridge slab and the guardrail is applied as a uniformly distributed load on the top flange of girders and stringers based on the tributary width. This is a simpler approach and similar to the approach used during design of non-composite bridges. It excludes transfer of load through friction at the slab and girder interface.

In the second approach (Non-composite model), non-composite action is modeled through contact interaction at the interface as discussed in Section 3.2.4.3. In the event of fracture, the girder top flange will separate from the bridge slab close to the fracture location while maintaining contact in the regions away from the fracture. The bridge slab will carry load through plate action. The cast-in-situ guardrail will also assist with the load redistribution. Therefore, it is necessary to include stiffness contribution of non-composite bridge slab and guardrail to capture the correct structural response. To evaluate the contribution of the slab and guardrail to post fracture bridge response, the bare girder model and the non-composite model are compared using nonlinear dynamic analysis with *Explicit* scheme and 1% mass proportional damping.

In case of the bare girder model, the girder deflections continued to increase and analysis failed around 2.04 sec with buckling in girder E and F around 0.2-0.3L (Figure 4-48). The girder deflections are recorded at the intersection of the bottom flange and web. Girder deflection in the fractured girders reached unrealistic deflections. As a result of higher deflection values, significant yielding is observed in the stringers and girders in the fractured span. Figure 4-47 shows the deformed shape in span 3S-2S at the end of 2.04 sec. Rigid body motion is observed in the fractured girders. Parts of girders E and F on the right side of the fracture rotate as rigid body as shown by the linear deformed shape in Figure 4-47. The left side of the fractured girder E and F deflect like a cantilever because of the continuity over Pier 3S.

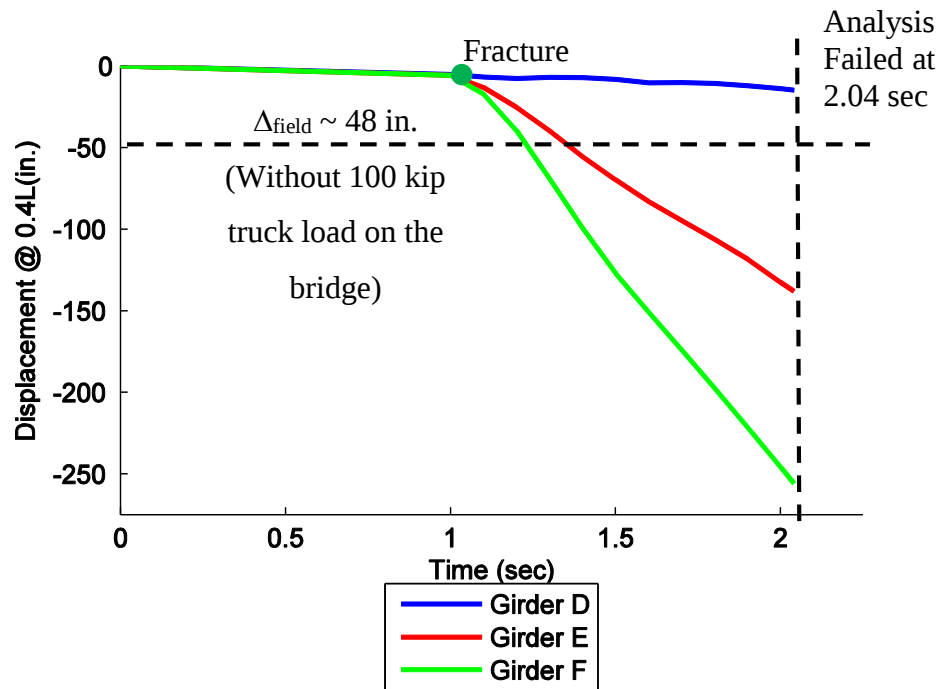


Figure 4-46: Dynamic girder deflection from fracture event at 0.4L (Bare girder model).

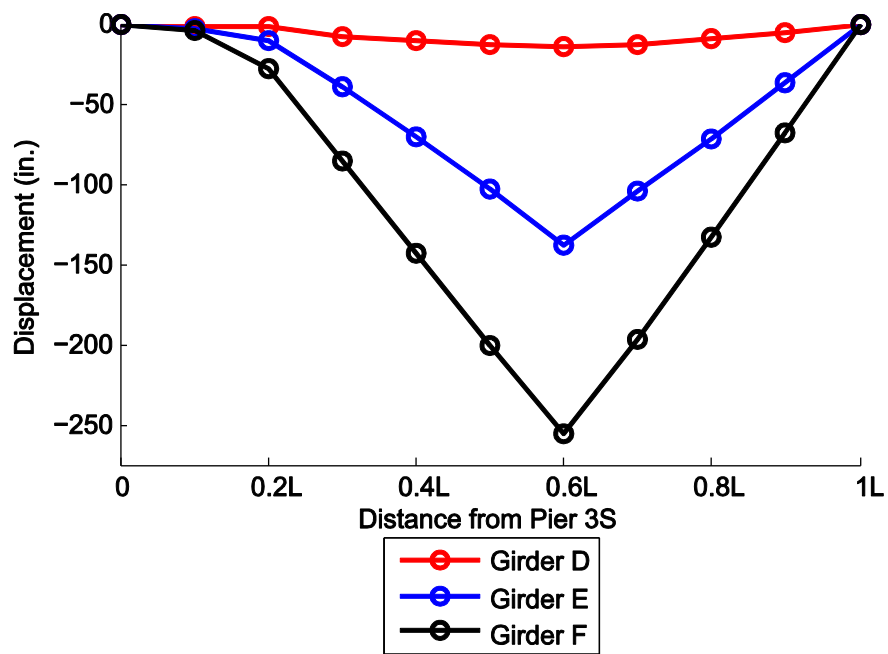


Figure 4-47: Girder deformed shape in Span 3S-2S at the end of analysis (Bare girder model).

Figure 4-48 shows the von Mises stress distribution in the steel superstructure in span 3S-2S for the bare girder model. In the Hoan Bridge, flange plates and bearing stiffeners were made of A588 steel while all other components were made of A36 steel. Therefore, majority of steel in the steel superstructure is A36. The display scale is set to display regions having stress higher than 36 ksi in red to visually identify regions of high stress or yield in case of A36 material. Significant yielding is observed in the girder E and F web between 0.2-0.3L from pier 3S. In stringers, yielding is observed at the location of stringers to cross frame connection near fracture.

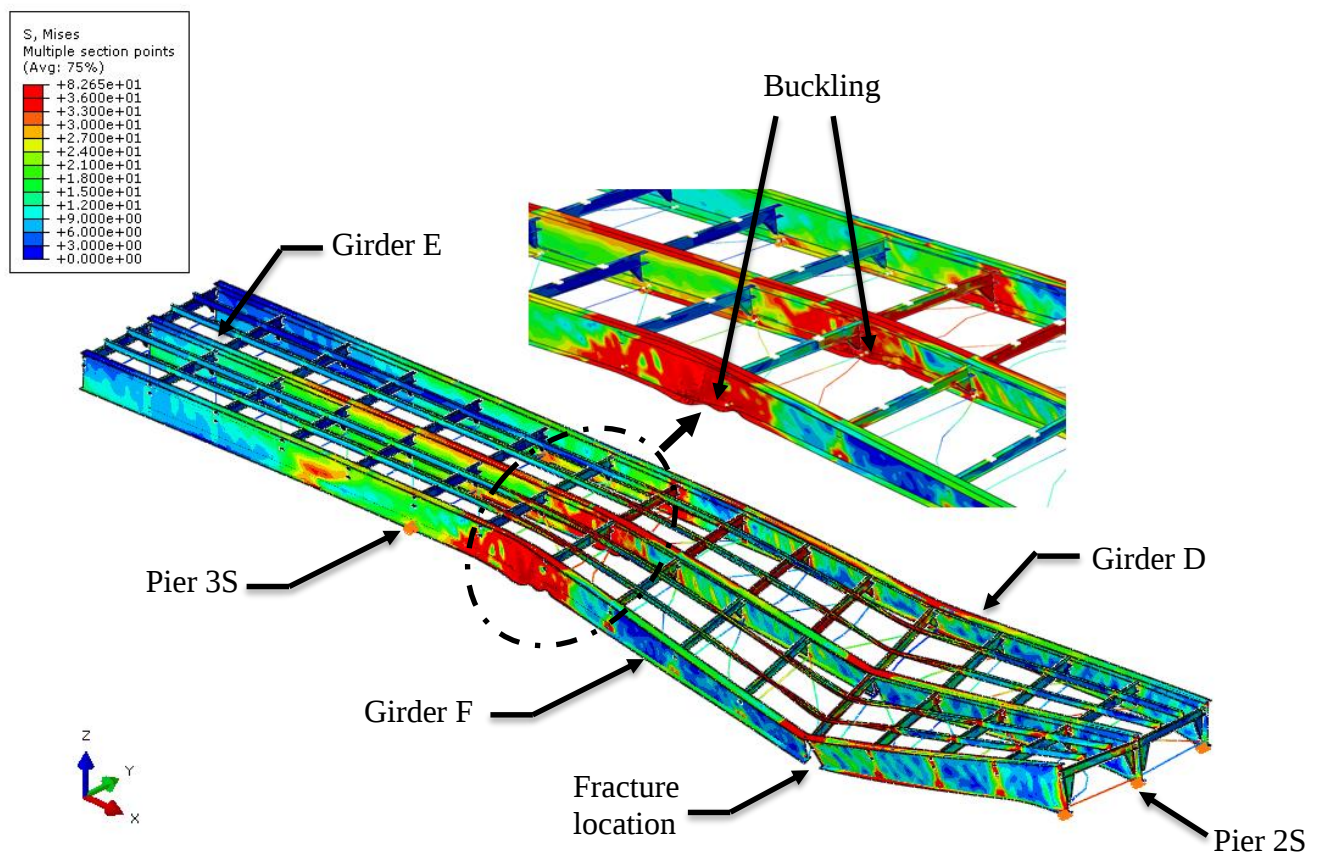


Figure 4-48: von Mises stress distribution in steel superstructure at the end of analysis ($t=2.04$ sec) in span 3S-2S (Bare girder model).

In case of the non-composite model, the presence of a non-composite slab resulted in lower girder deflections compared to the bare girder model (Figure 4-49). Concrete crushing is observed in guardrail close to fracture location, which is consistent with the field observation. Girder displacements pre and post fracture for the bare girder model and non-composite model are tabulated in Table 4-16. In the bare girder model, absence of slab resulted in large girder displacements leading to failed analysis solution. Presence of concrete slab reduced girder displacements and allowed *Explicit* scheme to obtain a converged solution.

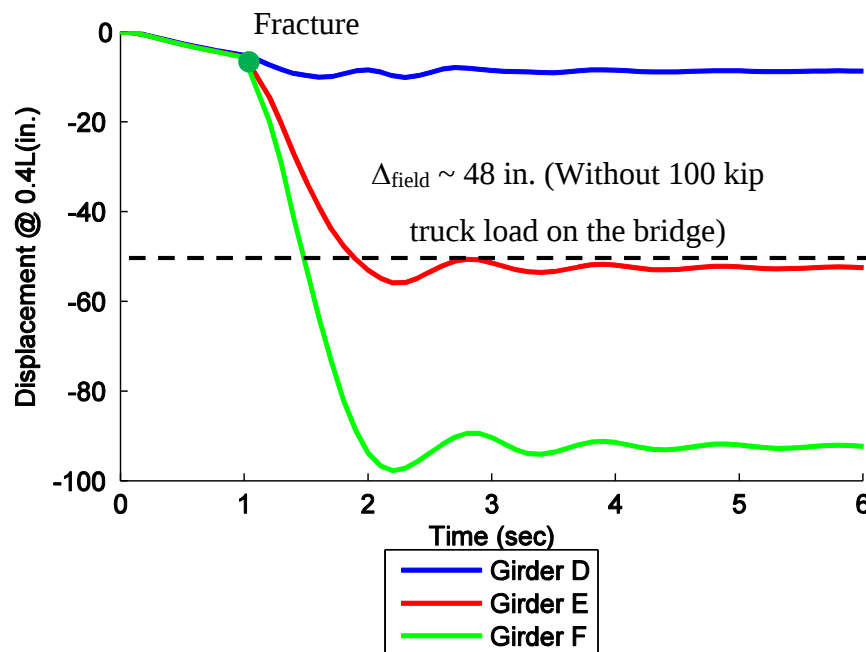


Figure 4-49: Dynamic girder deflection at 0.4L (from Pier 2S) from fracture event (Non-composite model).

Table 4-16: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis for bare girder model and non-composite model of Hoan Bridge using *Explicit* solution scheme.

Model description	Step	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
Bare girder model	Dead and live load	-5.31	-5.62	-5.7
	Post Fracture response	-14.79	-137.98	-255.75
Non-composite model	Dead and live load	-5.20	-5.52	-5.58
	Post Fracture response	-8.70	-52.49	-92.29

(Negative sign indicates downward displacement)

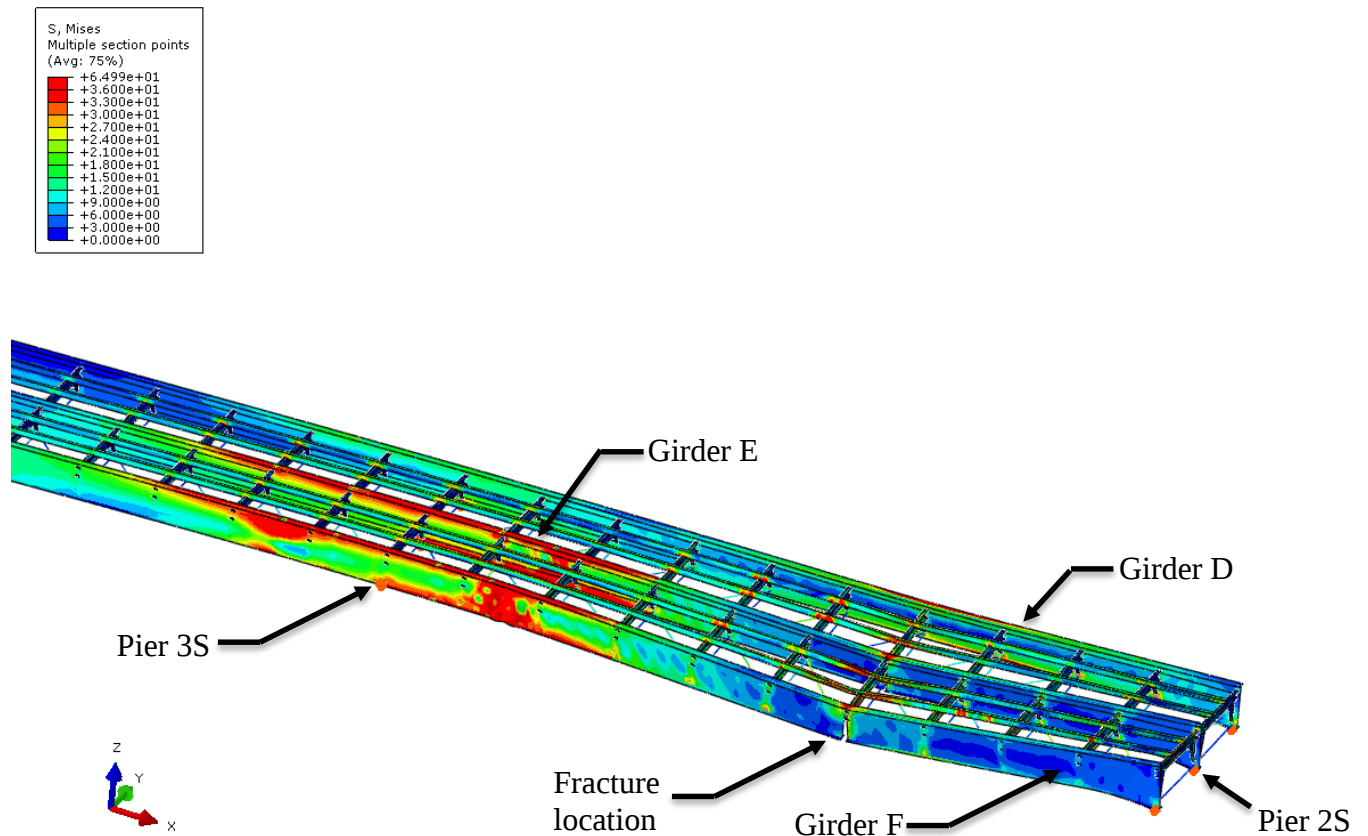
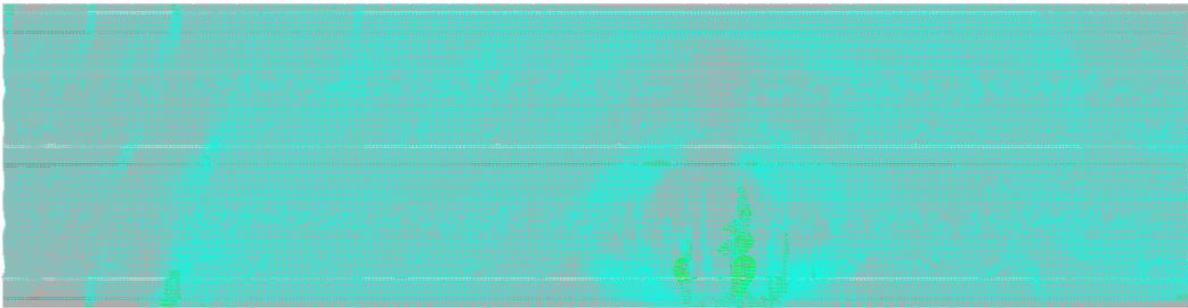
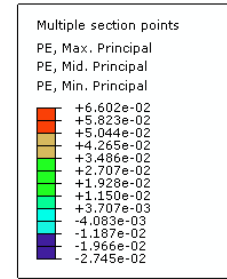


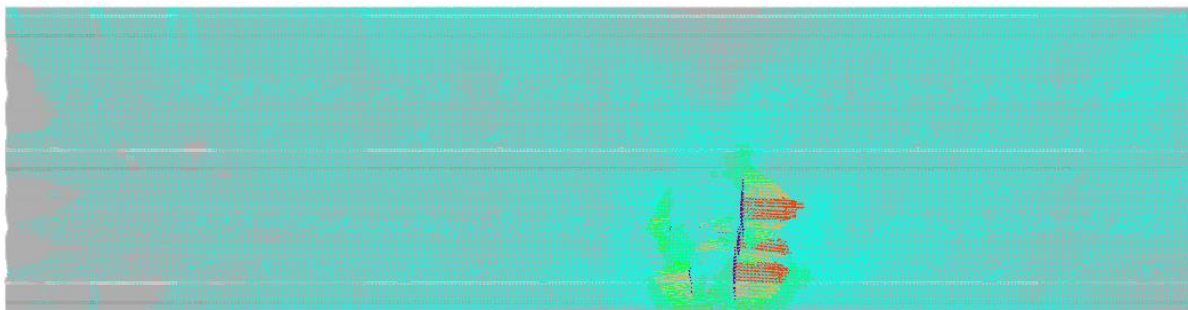
Figure 4-50: von Mises stress distribution in steel superstructure at the end of analysis (Non-composite model).

Figure 4-50 shows the von Mises stress distribution in the steel superstructure in span 3S-2S for the non-composite model. Similar to Figure 4-48, the display scale is set to display regions having stress higher than 36 ksi in red to visually identify regions of high stress or yield in case of A36 material. Significant yielding is observed in the girder E and F web between 0.2-0.3L from pier 3S. Stringers near the fractured region yield at the stringer to cross bracing connection. Along with the yielding in the span 3S-2S, yielding is observed in the fractured girders in span 4S-3S between 0.2-0.3L from pier 3S. Figure 4-51 shows the principal plastic strain distribution in the bottom fiber of the slab at the end of analysis ($t=6$ sec). Near the location of fracture, the slab separates from the top flange and carries load through plate action. This results in tensile cracking in the bottom fibers and compressive crushing in the top fibers of the slab near fracture location (Figure 4-51).

To calculate the separation at the steel-concrete interface at the fracture location, girder and stringer deflections are recorded at the intersection of top flange and web. Slab deflections are recorded at the nodes right above the girder and stringer nodes used for deflection measurement. The difference between the girder and slab deflection gives the separation at the interface. Figure 4-52 shows the separation at the steel-concrete interface with time. Separation continues to increase for first 1 sec ($t=1-2$ sec) after fracture. The region between the intact girder D and fracture girder E sees less separation compared to the region between fractured girder E and F.



Top fiber



Bottom fiber

Figure 4-51: Principal plastic strain distribution on slab (Non-composite model).

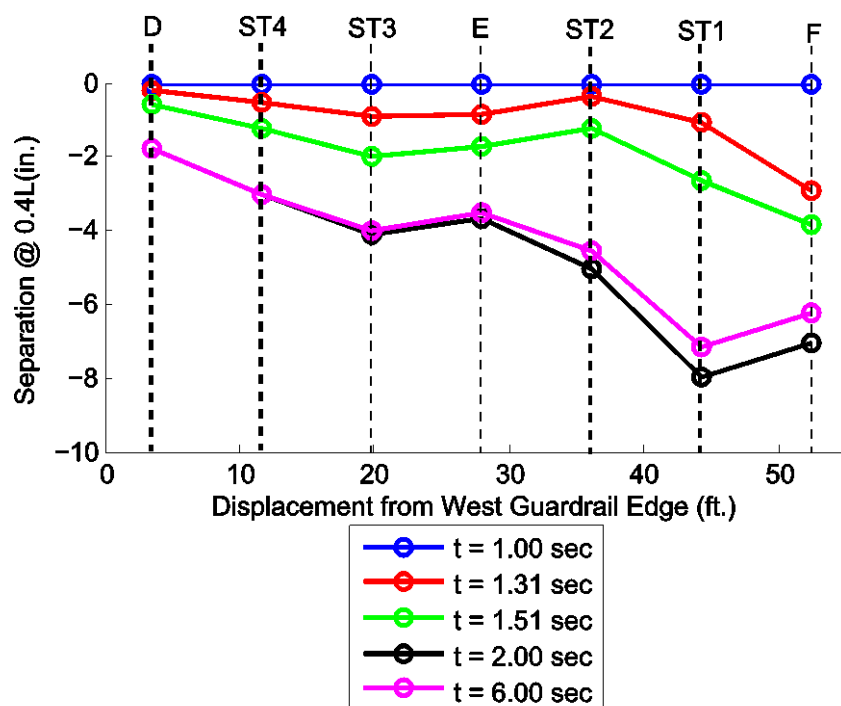


Figure 4-52: Separation at the steel-concrete interface at fracture location.

The absence of the slab in the bare girder model resulted in large girder displacements leading to analysis failure. The presence of a concrete slab in the non-composite model reduced girder displacements and allowed the *Explicit* scheme to reach a converged solution. In the Hoan Bridge, both the girder and stringer top flanges are encased in the concrete haunch, providing partial composite action, which is difficult to quantify. Results from the non-composite model provide a conservative estimate of the global response, but an excessively conservative solution is not necessarily what is desirable in this type of analyses. In addition, partial composite action on the bridge could possibly result in higher demand on the connections near the fractured region than those predicted by non-composite model.

4.5.7 Secondary elements

Secondary elements, such as bracing, diaphragms, and guardrails, are intrinsic parts of the bridge superstructures and can influence bridge superstructure response to external loads. In the event of fracture, contribution from the secondary elements to the strength and stability of bridge superstructure can be even more significant. This section investigates the effects of secondary elements on bridge response following the fracture event.

4.5.7.1 Guardrail

In Section 4.4.1, the capabilities of guardrail models is evaluated through quasi-static analysis. In this section Model-1, Model-6 and Model-7 are further analyzed using nonlinear dynamic analysis with the *Explicit* scheme and 1% mass proportional damping to simulate the post fracture response of the bridge. In Model-1, elastic beam elements are used to model the guardrail at its centroid while Model-6 uses solid elements to model the guardrail. In Model-7, guardrail is excluded from the model but the guardrail weight is applied as a distributed load in the region of slab occupied by the guardrail.

Figure 4-53 and Figure 4-54 shows the comparison in the fractured girder F and the intact girder D response with different guardrail models. Girder displacements are recorded at the intersection of the bottom flange and web at the location of fracture. Girder displacements pre and post fracture for different guardrail models are tabulated in Table 4-17. Model-1 with elastic beam element for guardrail yielded stiffer response compared to the other two models for the fractured girder F case. The intact girder D response showed little sensitivity to the guardrail model. To evaluate the impact of guardrail models on slab behavior, principal plastic strain distribution on the slab bottom fiber are compared between all the guardrail models in Figure 4-55. Model-1 showed concentrated tensile plastic strains at the location of fracture in the bottom of slab. Model-7 showed plasticity distributed to a larger slab area as compared to Model-1 and Model-6 and the plastic strain values near the fractured region are lower compared to other models. Excluding the guardrail from the analyses resulted in higher deflection and plasticity distributed over larger slab area, putting higher demand on the slab. In turn, however, this might reduce the demand on members and connections closer to fractured location. Therefore, it is necessary to include a correct representation of guardrail in the analysis. Model-6 with solid elements to represent the guardrail is selected for the rest of the parametric study.

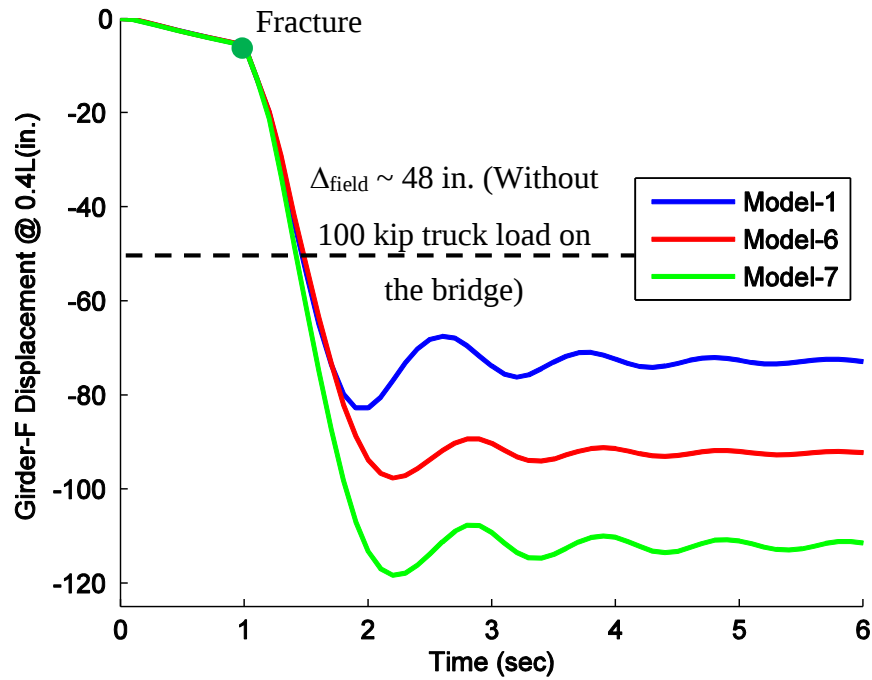


Figure 4-53: Dynamic deflection of girder F at 0.4L (from Pier 2S) for different guardrail models.

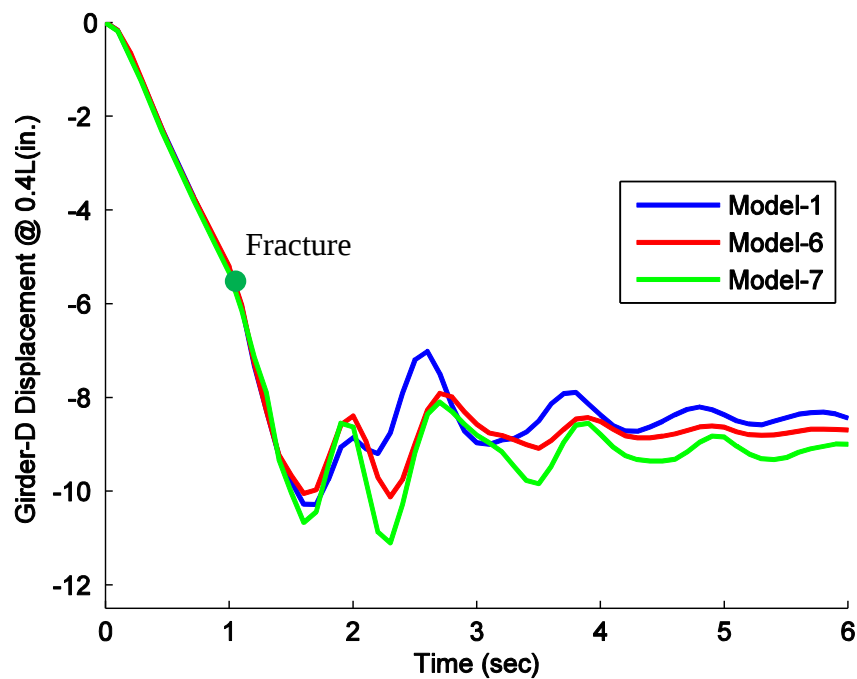


Figure 4-54: Dynamic deflection of girder D at 0.4L (from Pier 2S) for different guardrail models.

Table 4-17: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis for different guardrail models.

Guardrail Model	Analysis step description	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
Model-1	Dead and live load	-5.26	-5.58	-5.65
	Dynamic response post fracture	-8.44	-42.87	-72.93
Model-6	Dead and live load	-5.20 (-1.1%)	-5.52 (-1.1%)	-5.58 (-1.1%)
	Dynamic response post fracture	-8.70 (3.1%)	-52.49 (22.4%)	-92.29 (26.5%)
Model-7	Dead and live load	-5.34 (1.5%)	-5.65 (1.2%)	-5.73 (1.5%)
	Dynamic response post fracture	-8.99 (6.6%)	-62.73 (46.3%)	-111.47 (52.9%)

(Negative sign indicates downward displacement)

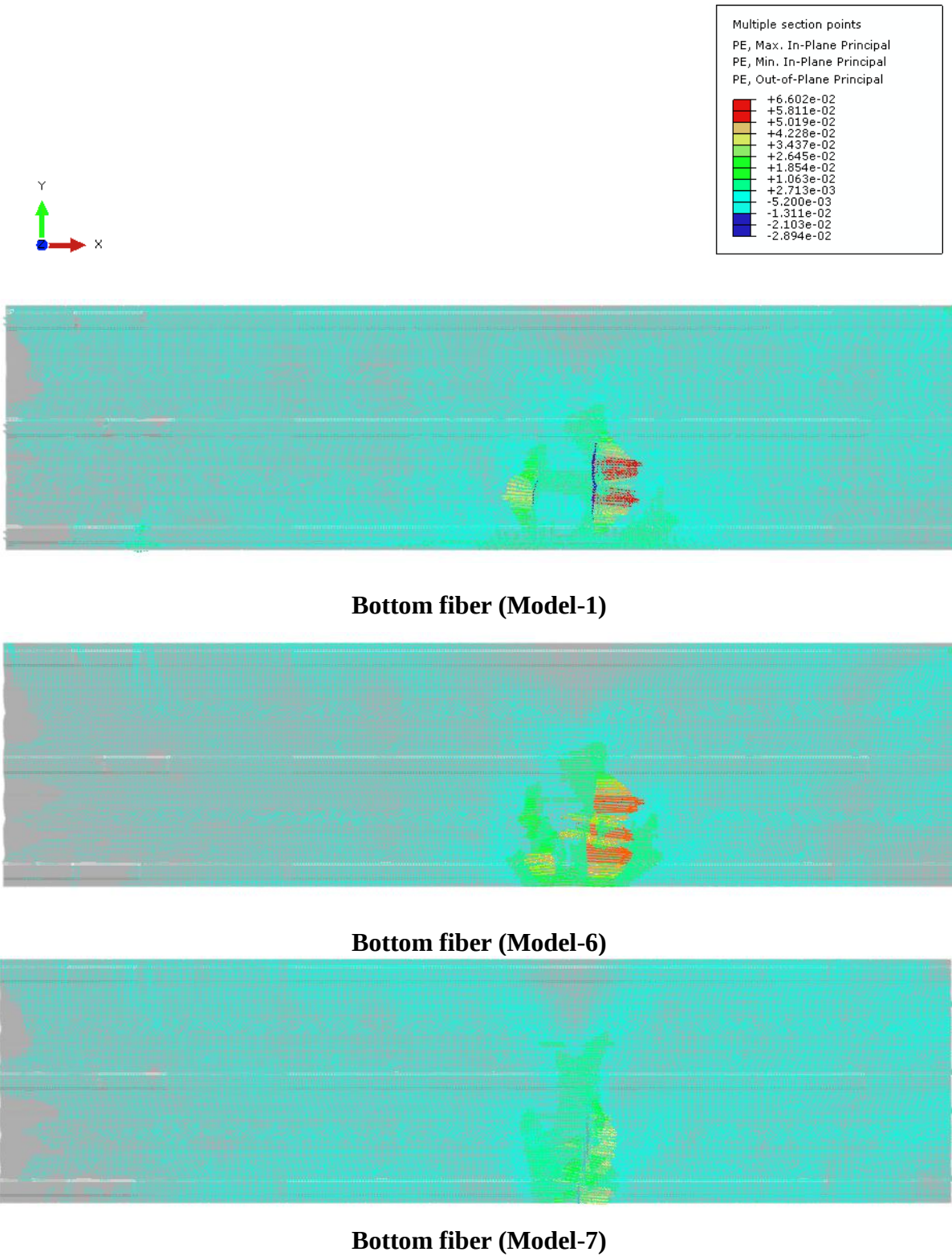


Figure 4-55: Principal plastic strain distribution on slab at the end of analysis.

4.5.7.2 Lower lateral bracing

The lower lateral bracing is connected to the girder webs through bolted connections to a welded web shelf plate. In the current finite element model, the lower lateral bracing is modeled using beam elements. These elements are connected to the girder web using connector elements that represent the bolted connection to the welded web shelf plate. It is difficult to estimate the amount of connection fixity offered by the bolted connection and the welded shelf plate. Therefore, analyses are carried out with both fixed and pinned end conditions to bound the problem. Additionally, an analysis is carried out without the lower lateral bracing to evaluate its contribution to the load redistribution following fracture event. The post fracture response of the bridge is simulated with nonlinear dynamic analysis using the *Explicit* scheme and 1% mass proportional damping.

Girder displacements pre and post fracture are tabulated in Table 4-18 for the cases described above. The model without the lower lateral brace was terminated externally when the deflection at the fractured girder exceeded 250 in. at the end of 3.2 sec. Figure 4-56 and Figure 4-57 show the displacement of the fractured girder F and the intact girder D respectively. The fractured girder F displacement increased by about 15% when the brace end condition changed from fixed to pinned, while the girder D displacement stayed the same. The absence of lower lateral bracing resulted in drastic increase in girder deflections following fracture resulting into yielding and buckling in the fractured span 3S-2S (Figure 4-58). In the absence of lower lateral bracing, the load redistributed to the intact girder D reduced, which in turn, reduced its displacement. Additionally it modified the load redistribution path putting more demand on the fractured girders F.

Table 4-18: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis for different lower lateral brace models.

Lower lateral brace model	Analysis step description	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
Fixed Brace	Dead and live load	-5.20	-5.52	-5.58
	Dynamic response post fracture	-8.70	-52.49	-92.29
Pinned Brace	Dead and live load	-5.19	-5.51	-5.58
	Dynamic response post fracture	-8.46	-58.67	-105.61
Brace Absent (Externally terminated at 3.2 sec)	Dead and live load	-4.94	-5.60	-5.48
	Dynamic response post fracture	-6.15	-143.80	-262.04

(Negative sign indicates downward displacement)

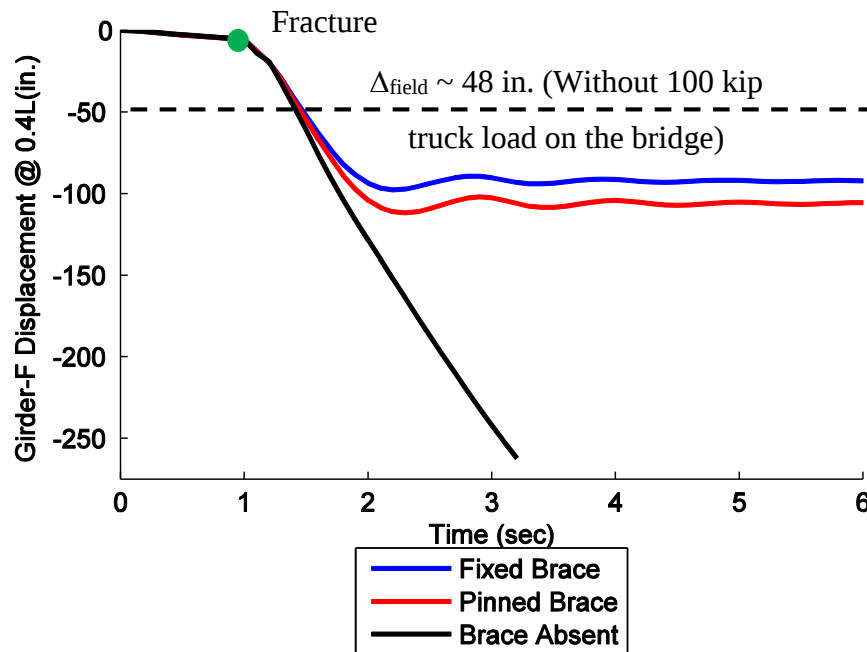


Figure 4-56: Dynamic girder F deflection at 0.4L (from Pier 2S) following fracture event.

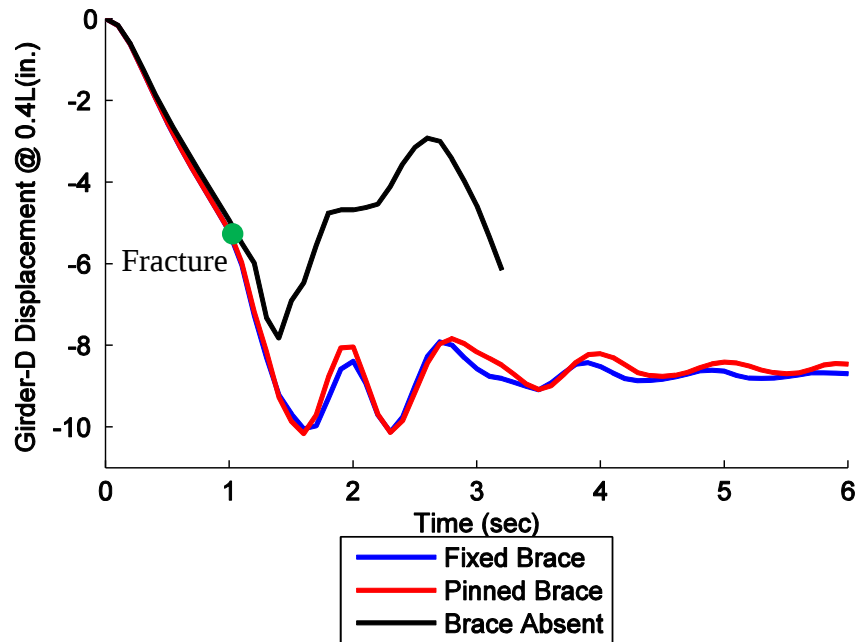


Figure 4-57: Dynamic girder D deflection at 0.4L (from Pier 2S) following fracture event.

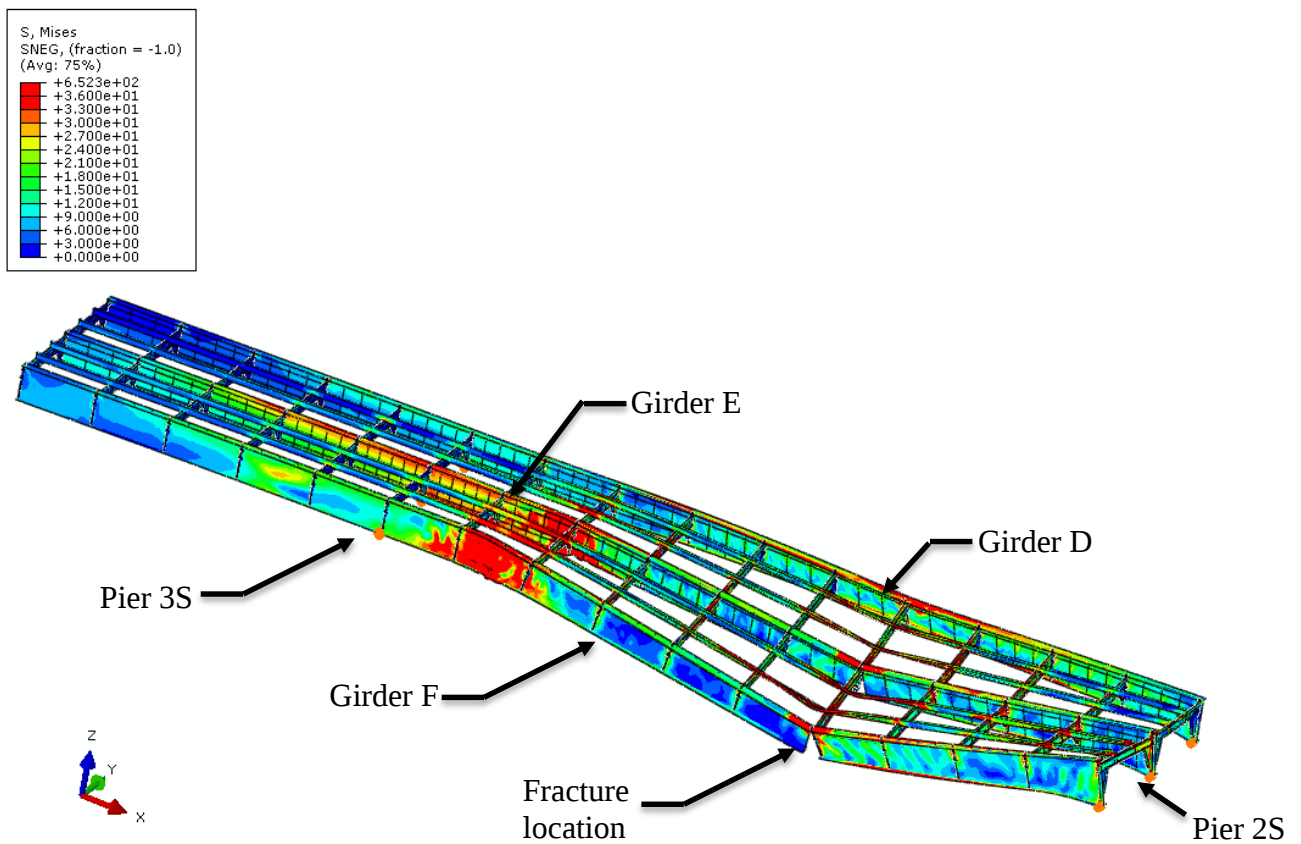


Figure 4-58: von Mises stress distribution in steel superstructure at the end of analysis (Non-composite model without lower lateral bracing).

Figure 4-59 shows the axial force in the lower lateral brace in span 3S-2S at the end of the analyses for fixed and pinned end conditions. The compressive force in the lower lateral brace connecting adjacent cross frames between girders D and E near Pier 3S increased when the brace end condition switched from fixed to pinned. Upon visual comparison of the deformed shapes with either fixed or pinned end condition (Figure 4-59), the lower lateral bracing with fixed end condition showed more pronounced out of plane deformation as the fixed end condition introduced moments at the brace ends.

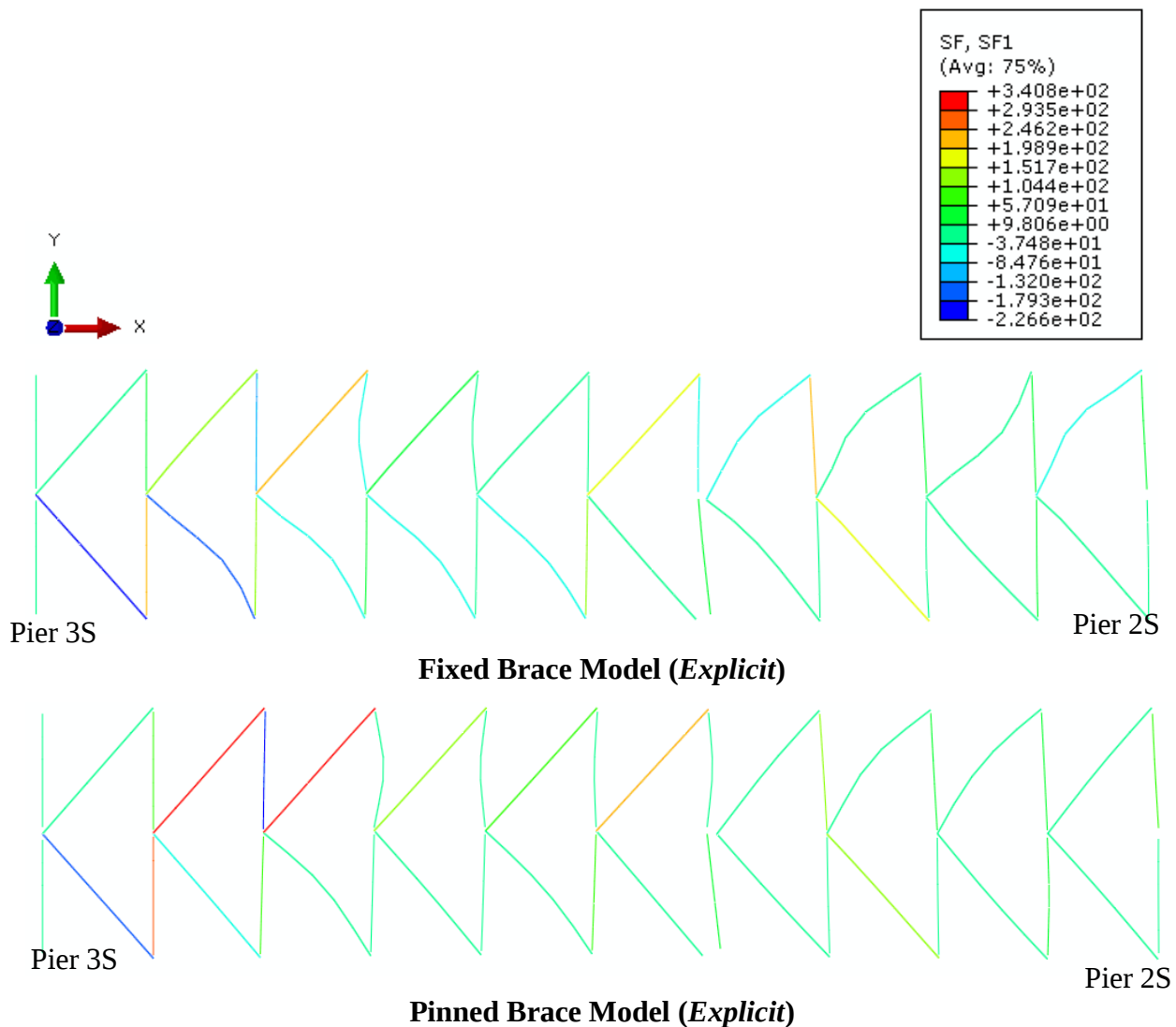


Figure 4-59: Axial force in lower lateral brace in Span 3S-2S at the end of analysis (Deformation Scale=2).

4.5.8 Boundary conditions

The Hoan bridge support configuration was unusual because it did not allow for the free expansion and contraction of the bridge superstructure from thermal cycles. Figure 4-11 show the design boundary conditions (labeled as “Design BC”) on the Hoan Bridge where Pier 5S, Pier 4S and Pier 3S supports had fixed shoes while Pier2S supports had expansion shoes. This configuration was probably selected assuming some pier flexibility but no documentation explaining the reasoning behind selection of this boundary condition was found. Figure 4-60 shows the additional support configuration labeled as “Assumed BC” analyzed in this study. The additional support configuration allows for expansion and contraction from thermal cycles. At pier 5S, girder E is restrained from longitudinal and transverse movement, while the rest of the girder E supports only restrain transverse movement. Girder D and F supports at pier 5S restrain longitudinal movement. The rest of the girder D and F supports allow movement in longitudinal and transverse direction.

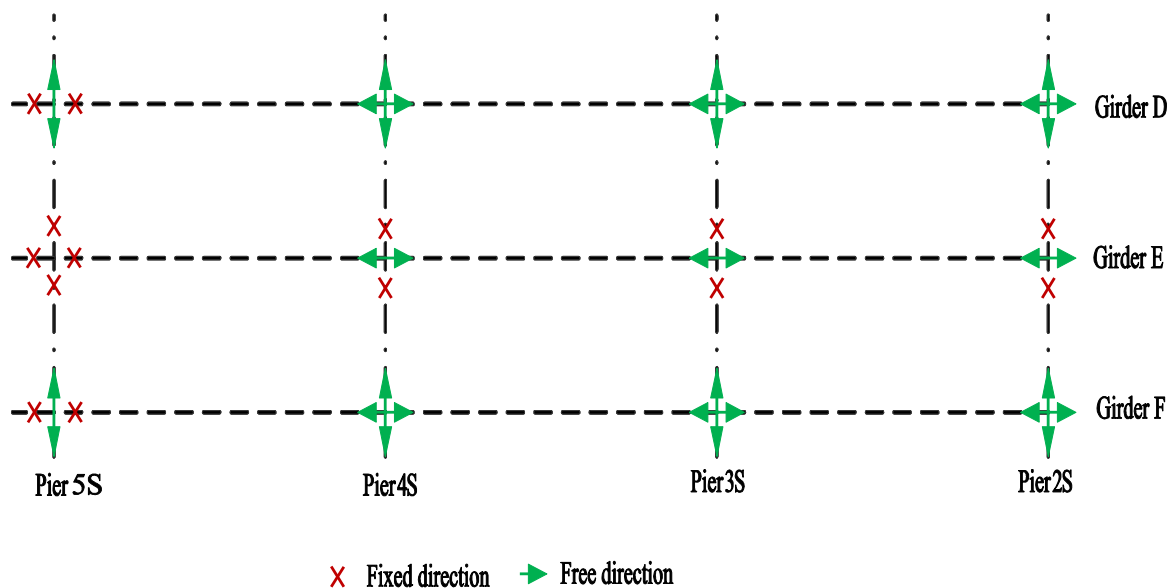


Figure 4-60: Assumed boundary condition for Hoan Bridge.

The post fracture response of the bridge with design and assumed boundary condition is simulated using nonlinear dynamic analysis with the *Explicit* scheme and 1% mass proportional damping. Figure 4-61 shows the girder deflection at the location of fracture (0.4L from Pier 2S) following fracture event for both design and assumed boundary conditions. Girder displacements are recorded at the intersection of the bottom flange and web. Pre and post fracture girder

deflections at the end of analysis for designed and assumed boundary condition are tabulated in Table 4-19. The pre fracture deflections for the “Assumed BC” model are ~8-10% higher, while the post fracture girder deflections increased by ~20-25%. Figure 4-62 shows the deformed shape of all three girders recorded at every 0.1L interval in span 3S-2S at the end of analyses. The deformed shape is similar for both boundary conditions. Rigid body motion is observed in girder E and F on the right side of the fracture for both boundary conditions, with “Assumed BC” deflecting more. The left side of the fractured girder E and F deflect like a cantilever because of the continuity over Pier 3S.

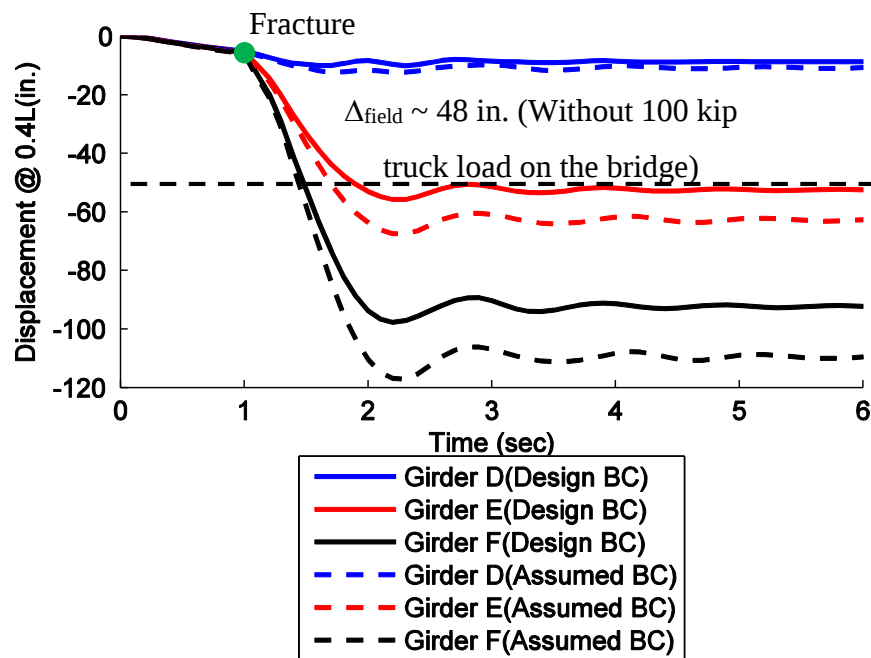


Figure 4-61: Dynamic girder deflection at 0.4L for design and assumed boundary condition.

Table 4-19: Pre and post fracture girder deflections at 0.4L (from Pier 2S) at the end of analysis for designed and assumed boundary condition.

Analysis	Step	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
Design BC	Dead and live load	-5.20	-5.52	-5.58
	Post Fracture response	-8.70	-52.49	-92.29
Assumed BC	Dead and live load	-5.67 (9%)	-5.99 (8.6%)	-6.07 (8.7%)
	Post Fracture response	-10.76 (-23.7%)	-62.74 (-19.5%)	-109.55 (-18.7%)

(Negative sign indicates downward displacement)

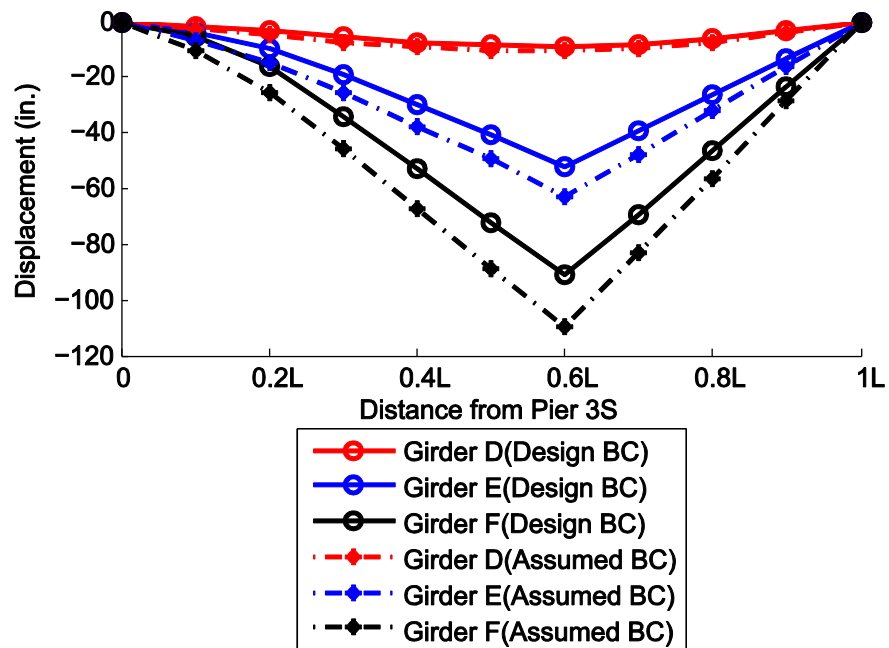


Figure 4-62: Girder deformed shape in Span 3S-2S at the end of analysis for design and assumed boundary condition.

4.5.9 Concrete strength

Slab design drawings for the Hoan Bridge were unavailable. Therefore, for the analyses models the concrete slab is redesigned using the strip method with 4000 psi concrete and Grade 40 reinforcement. Reinforcement in the longitudinal and transverse directions is tabulated in Table 4-1. Slab construction for the long span bridges involves staged construction with concrete from multiple batches. This results in strength variation over the slab area with mean strength value typically higher than the specified value on the design drawings. Additionally, concrete properties changes with time as it is subjected to external conditions. To study the impact of concrete strength on the post fracture response, analysis is carried out with 4000 psi, 5000 psi and 6500 psi concrete for concrete slab and guardrail. Analysis is carried out using *Explicit* scheme with 1% mass proportional damping. Post fracture girder displacements at the fracture location at the end of analysis are tabulated in Table 4-20. Figure 4-63 shows the post fracture response of fractured girder F for different concrete strengths. Fractured girders response decreased with increase in concrete strength while intact girder response increased by 1-2% (Table 4-20).

Table 4-20: Girder displacement at location of fracture (0.4L from Pier 2S) for 4000 psi, 5000 psi and 6500 psi concrete at the end of analysis.

fc' (psi)	At fracture location (0.4L)		
	Girder D (in.)	Girder E (in.)	Girder F (in.)
4000	-8.7	-52.49	-92.29
5000	-8.86 (1.9%)	-48.86 (-6.9%)	-84.72 (-8.2%)
6500	-8.81 (1.3%)	-47.11 (-10.2%)	-81.64 (-11.5%)

(Negative sign indicates downward displacement)

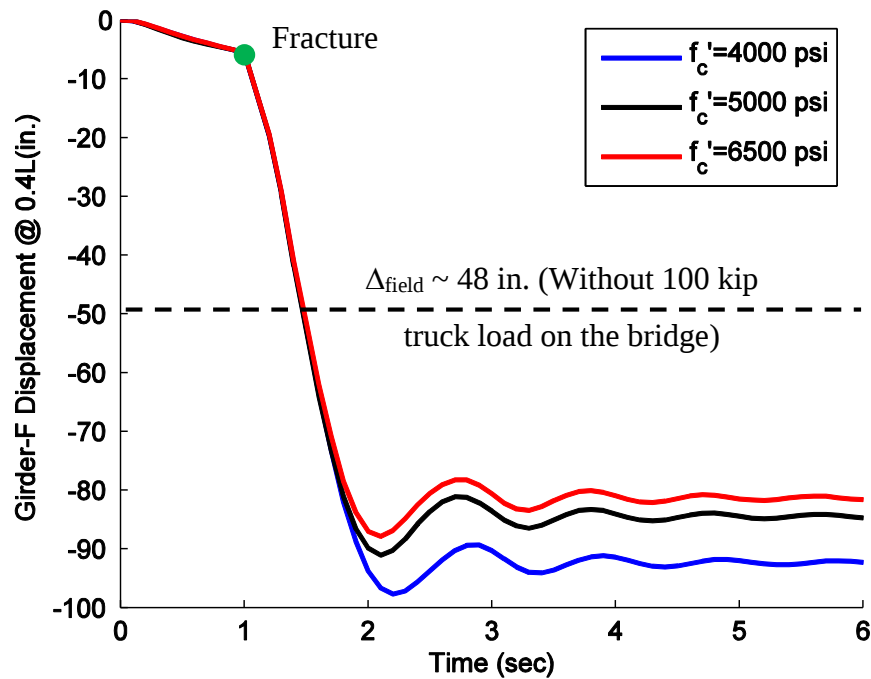


Figure 4-63: Girder-F response at 0.4L following fracture event for 4000 psi, 5000 psi and 6500 psi concrete.

4.5.10 Reinforcement

Hoan Bridge was constructed in early '70s. In the absence of slab details in the available design drawing, it was assumed that Grade 40 reinforcement was used in the slab. Nowadays Grade 60 reinforcement is typically used in bridge slab construction. Figure 4-64 shows the stress-strain curve for Grade 40 and Grade 60 reinforcement from Barth and Wu (2006). Grade 40 is far more ductile compared to Grade 60 reinforcement. Reinforcement ductility and strength can possibly have significant impact on the bridge response following girder fracture. To investigate the effect of reinforcement grade, slab is redesigned with Grade 60 reinforcement. The new reinforcement details are tabulated in Table 4-21.

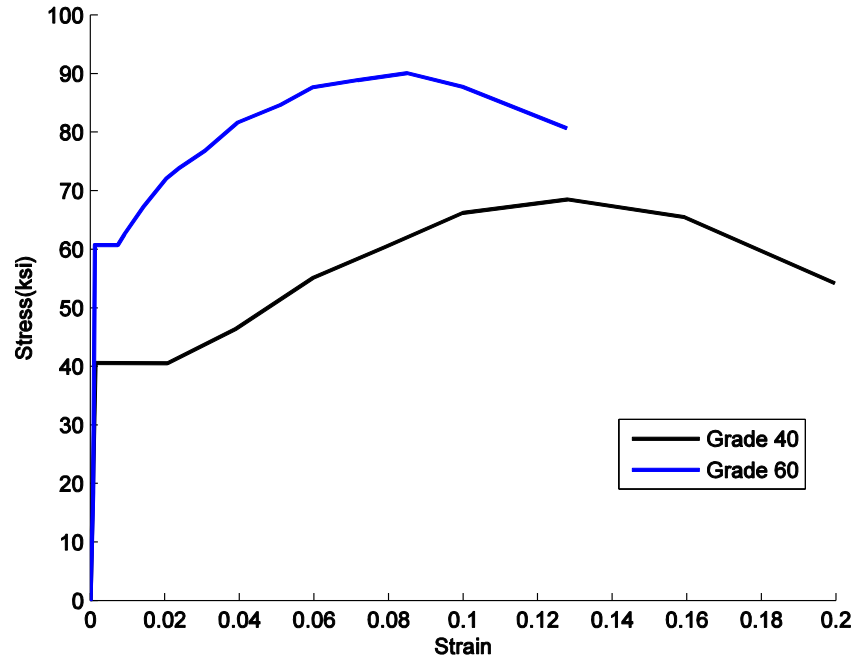


Figure 4-64: Reinforcement stress-strain curve (Barth and Wu, 2006), used under fair use..

Table 4-21: Slab reinforcement details (Grade 60).

Direction	Reinforcement
Transverse direction	#6 bars, 12" c/c (Top)
	#5 bars, 12" c/c (Bottom)
Longitudinal direction	#4 bars, 18" c/c (Top)
	#4 bars, 12" c/c (Bottom)

Girder deflection at the fracture location (Table 4-22) and von Mises stress distribution in steel superstructure (Figure 4-50 and Figure 4-65) is similar for both reinforcement grades. The Hoan Bridge model with Grade 40 reinforcement yielded ~1% higher deflections at the location of fracture compared to the model with Grade 60 reinforcement (Table 4-22). Both grades of reinforcement resulted in similar principal plastic strain distribution at the top and bottom fibers of the slab except at the fracture location. At the fracture location, the model with Grade 40 reinforcement showed higher plastic strain (Figure 4-66 and Figure 4-67). Similar to Figure 4-50, the display scale is set to display regions having stress higher than 36 ksi in red. Similarity in the overall response for Grade 40 and Grade 60 reinforcement possibly implies that the reinforcement stresses are below 40 ksi for the majority of the reinforcement in the concrete slab.

Table 4-22: Girder displacement at location of fracture (0.4L from Pier 2S) for Grade 40 and Grade 60 reinforcement at the end of analysis.

Rebar	At fracture location (0.4L)		
	Girder D (in.)	Girder E (in.)	Girder F (in.)
Grade 40	-8.70	-52.49	-92.29
Grade 60	-8.67	-51.19	-89.47

(Negative sign indicates downward displacement)

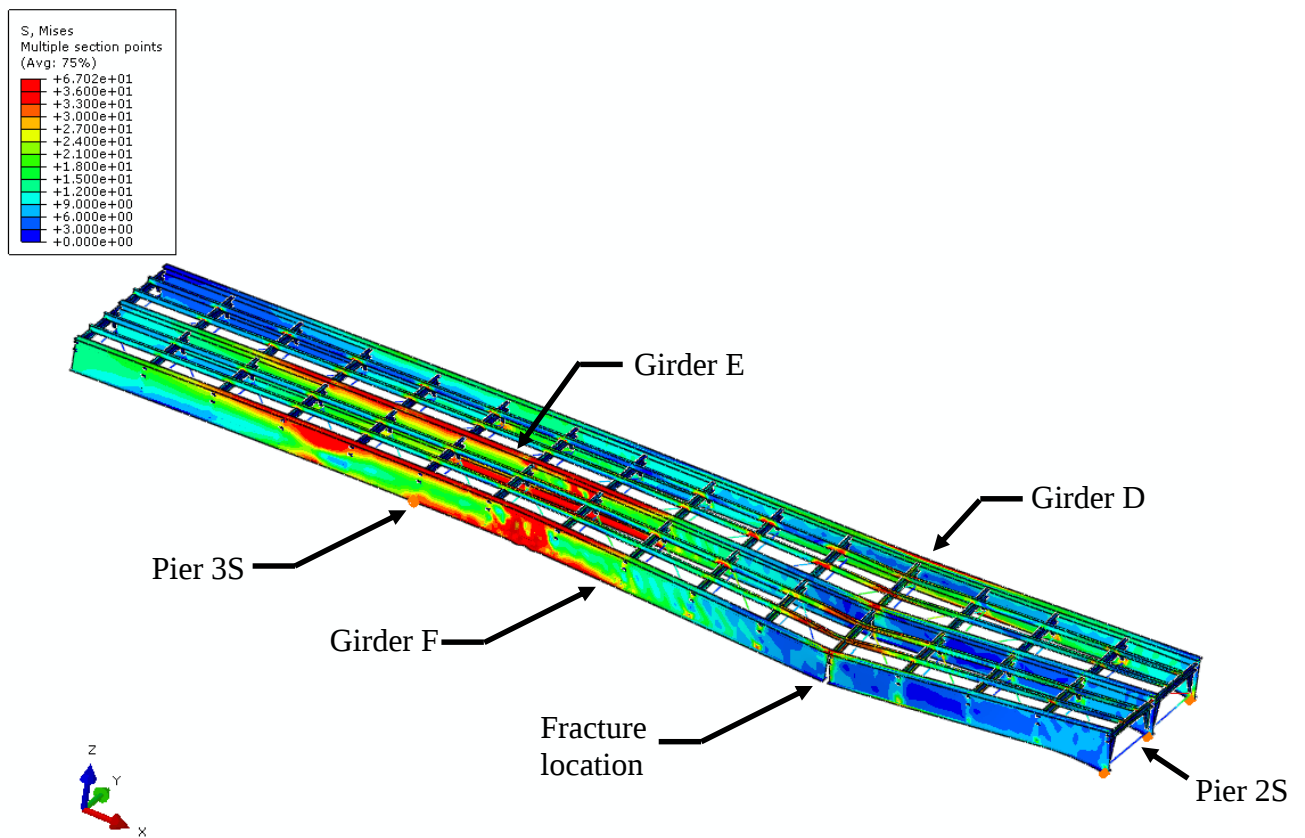


Figure 4-65: von Mises stress distribution in steel superstructure at the end of analysis for Grade 60 reinforcement in slab.

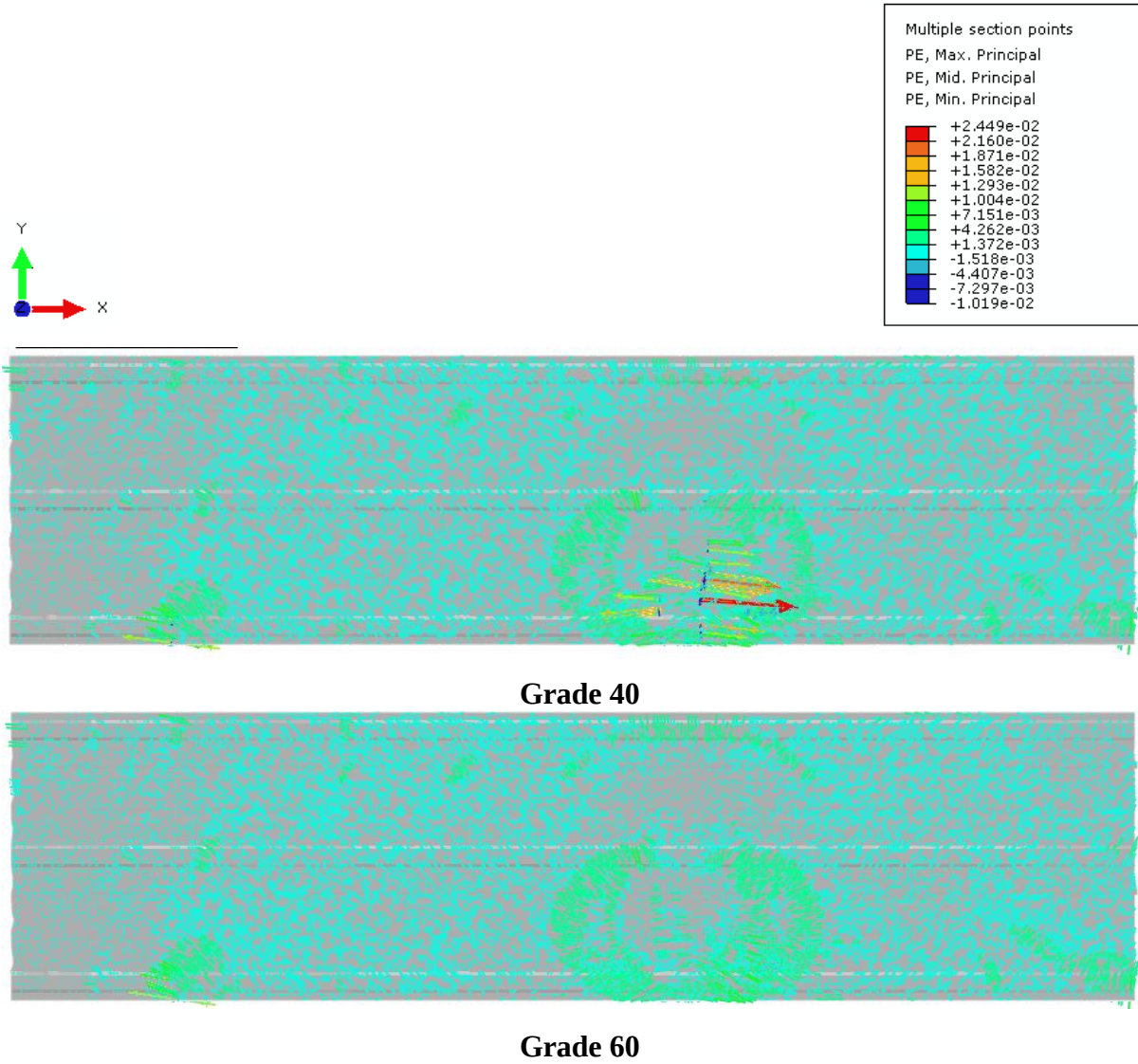


Figure 4-66: Principal plastic strain distribution on slab top fiber at the end of analysis.

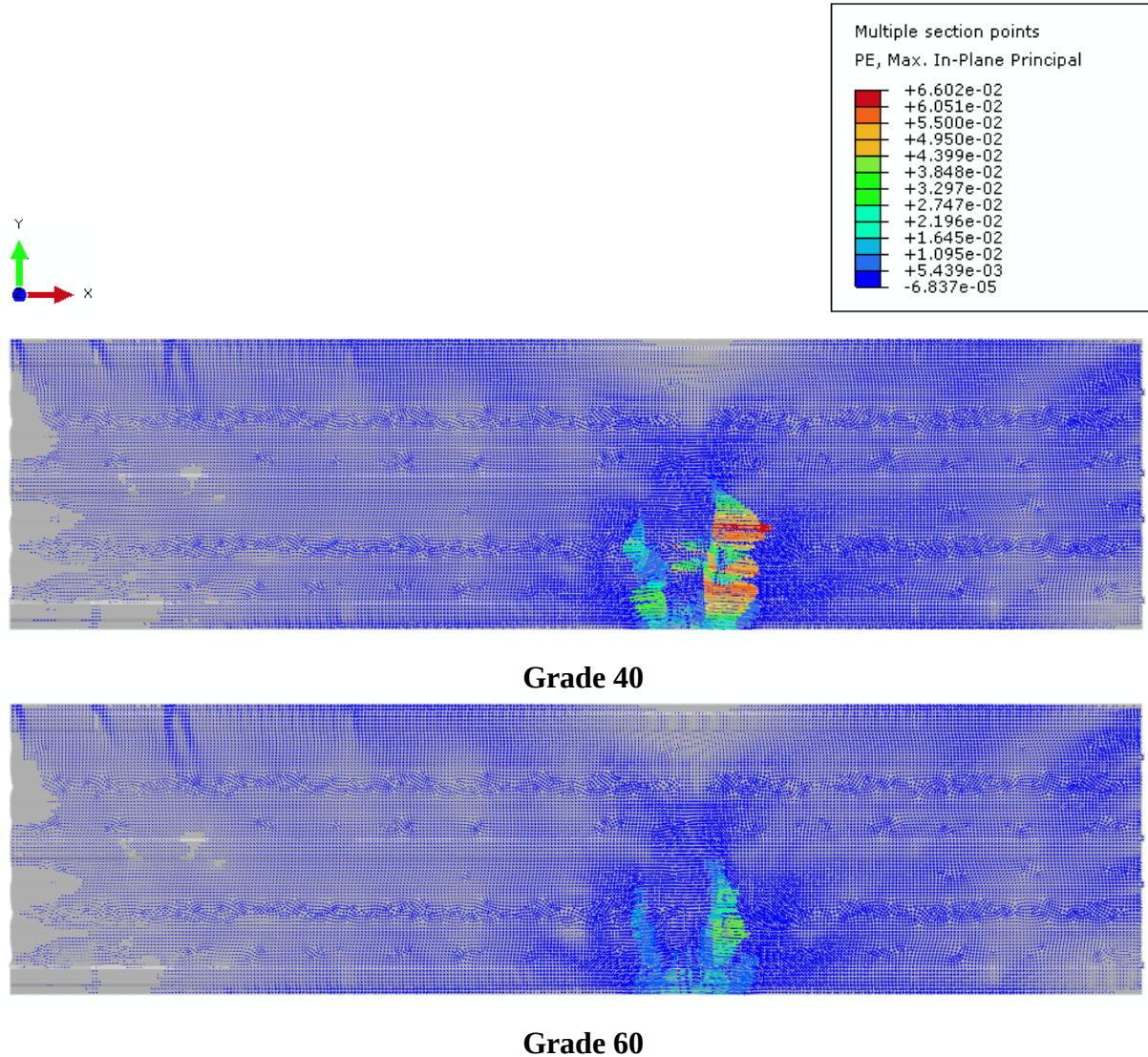


Figure 4-67: Principal plastic strain distribution on slab bottom fiber at the end of analysis.

4.5.11 Rate dependent material properties

Structural elements subjected to dynamic loading from earthquake, fracture, blast loading exhibit higher strength than similar elements under static loading. This increase in strength results from increased material strength under rapid loading rates. UFC 3-340-02 (2008) provides a discussion about the rate dependent behavior in structures subjected to accidental explosions. Rate dependent properties for structural steel and reinforced concrete from UFC 3-340-02 (2008) are used in this study. At first, the analysis with rate dependent material properties was attempted using an *Implicit* scheme with 1% Rayleigh damping. This analysis failed due to convergence

difficulties. Therefore, the model was reanalyzed with *Explicit* scheme with 1% mass proportional damping.

Figure 4-68 and Figure 4-69 show the comparisons between the post fracture response of girders D and F with rate dependent and independent material properties. Girder displacements are recorded at the intersection of the bottom flange and web. The behavior of the intact girder is found to be insensitive to the inclusion of rate dependent material properties because of the total yielded area in the intact girder is lower compared to the fractured girders, and increases in strain rate results in higher yield strength. The fractured girder F response decreased by ~20% with inclusion of rate dependent material properties. Inclusion of rate dependent material properties is necessary to capture the demands on connections close to fracture. Inclusion of rate dependent behavior in the model resulted in a reduction in plastic strains in the concrete slab near the fractured region as seen in the bottom (Figure 4-70) and top fibers (Figure 4-71) of the slab.

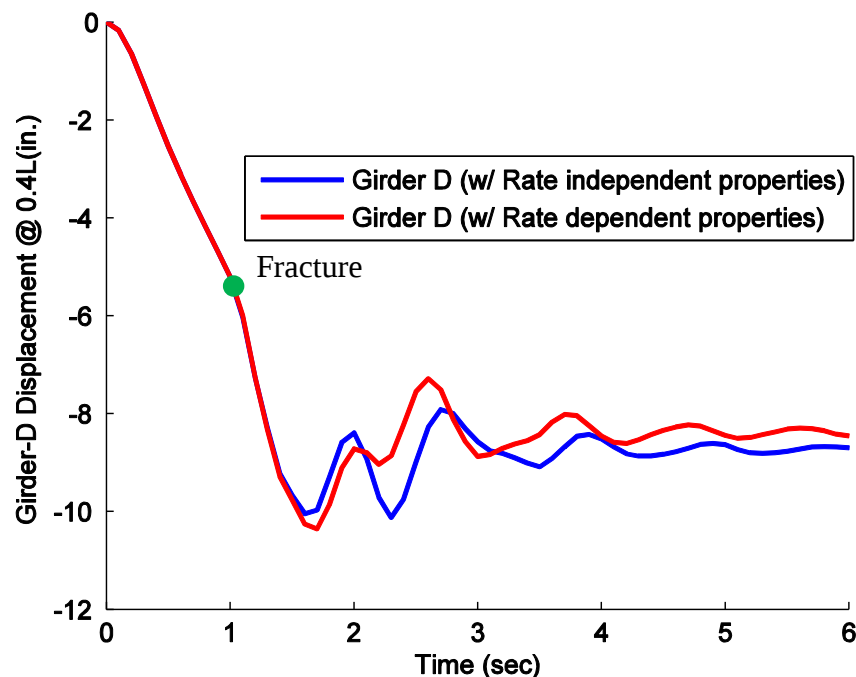


Figure 4-68: Comparison between dynamic response of girder D at 0.4L (from Pier 2S) following fracture event with rate dependent and independent material properties.

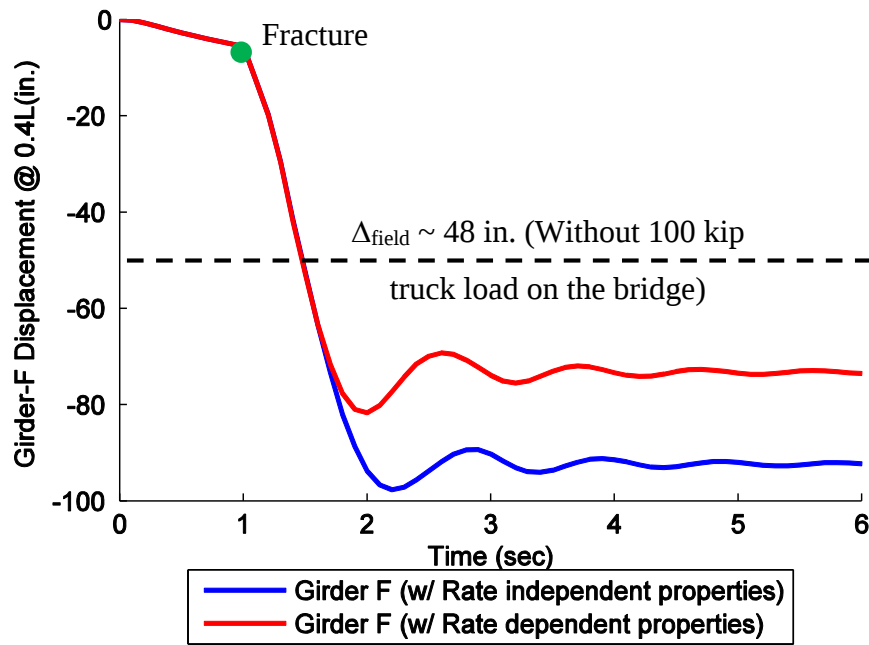


Figure 4-69: Comparison between dynamic response of girder F at 0.4L (from Pier 2S) following fracture event with rate dependent and independent material properties.

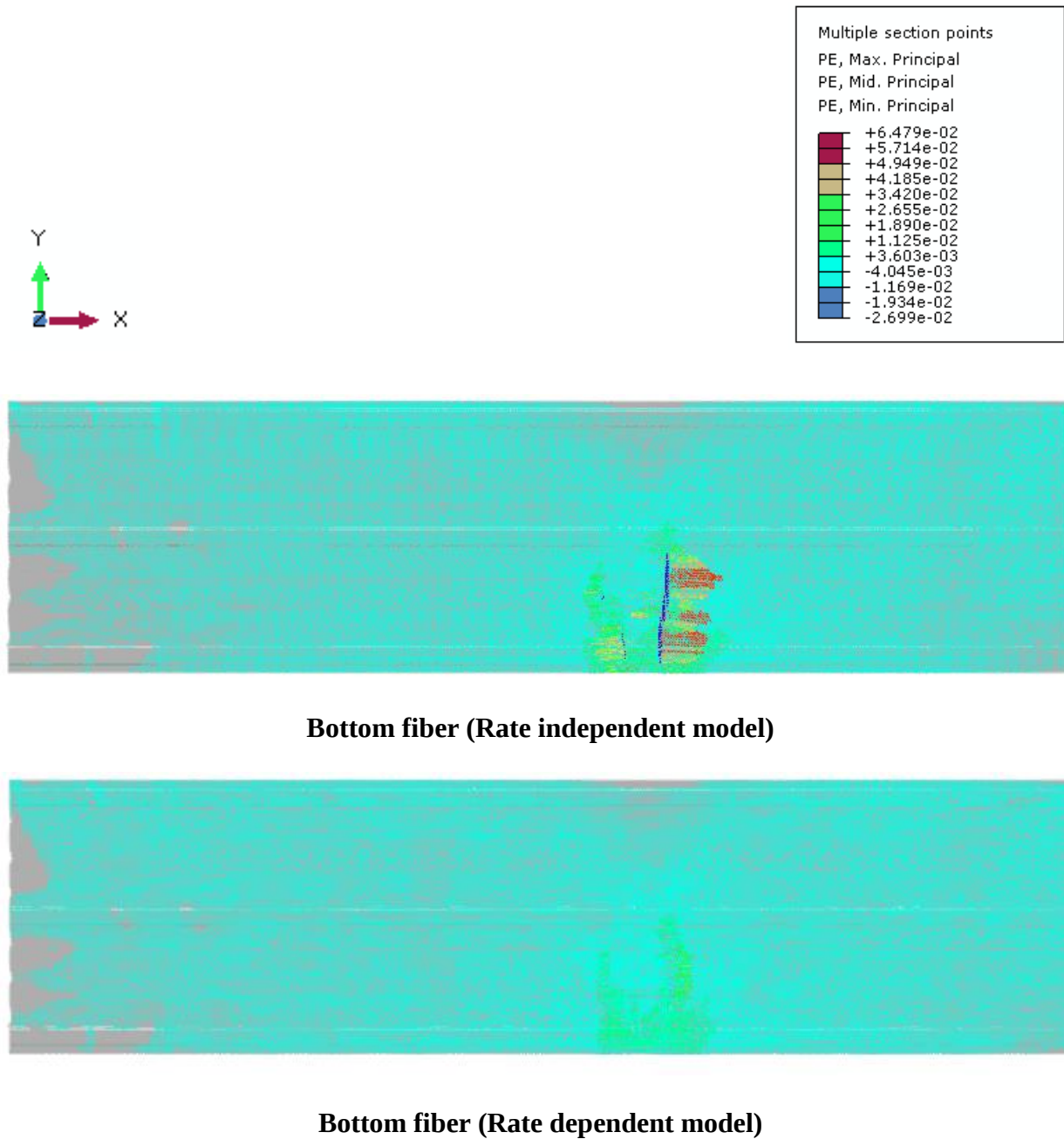


Figure 4-70: Comparison between principal plastic strain distribution at the end of analysis on slab bottom fibers for rate dependent and independent models.

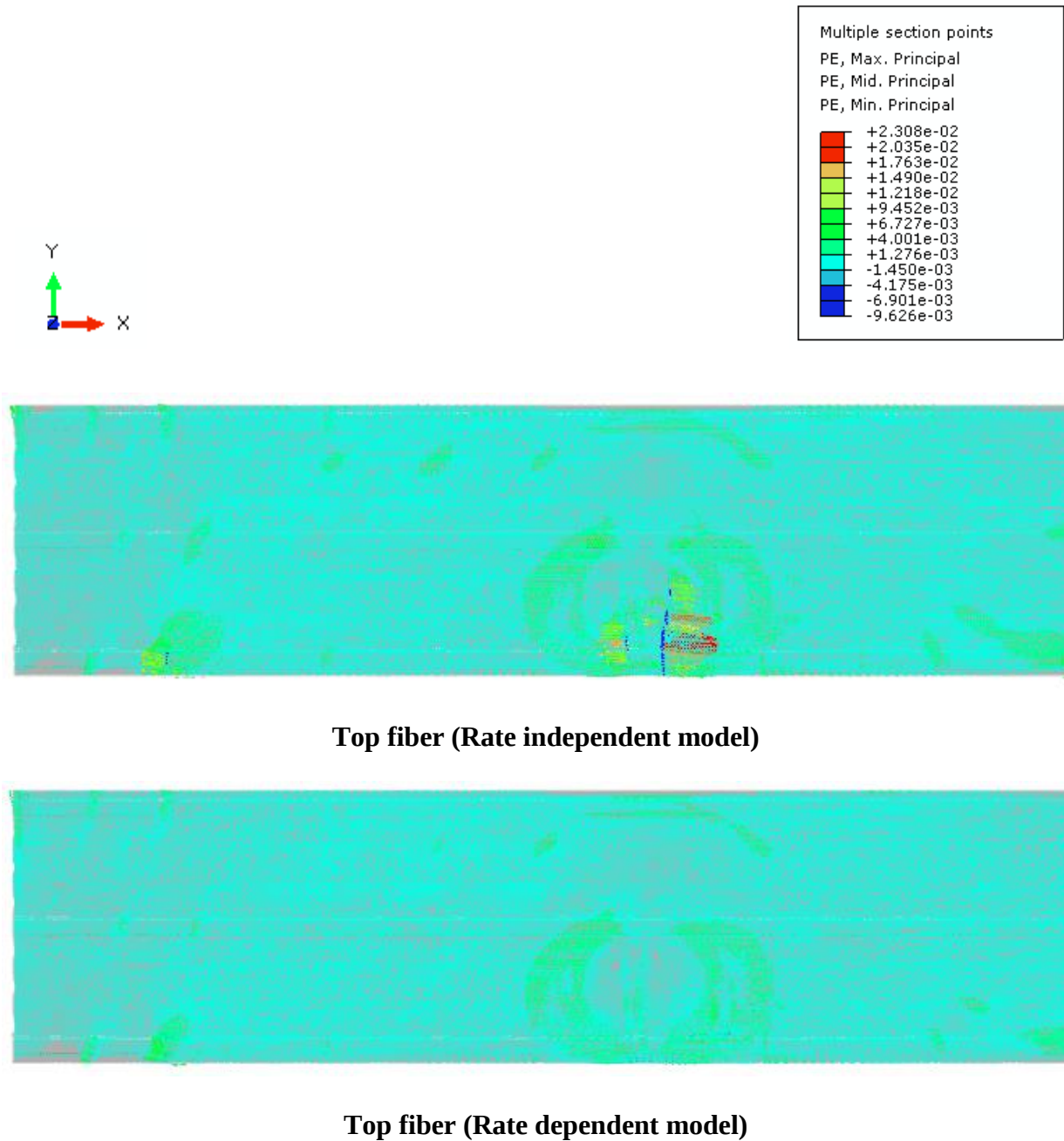


Figure 4-71: Comparison between principal plastic strain distribution at the end of analysis on slab top fibers for rate dependent and independent models.

4.5.12 Composite Hoan Bridge model

The Hoan Bridge had a non-composite bridge slab that theoretically acted independently of the steel girders. This bridge could be strengthened by making the bridge slab composite with the steel superstructure through post-installed shear studs. It is likely that composite action can significantly improve the post fracture response of the bridge. Rigid connectors (rigid links) have been widely used to model the composite action between concrete slab and steel girders. Jung (2006) compared the performance of multipoint constraints and connector elements with stud's force-slip response to predict the inelastic behavior of horizontally curved composite I-girder bridges. Load displacement curves using both methods yielded identical response except at higher load levels where second method yielded an additional vertical deflection. Multipoint constraints are similar to the rigid connectors as they do not allow any slip at the steel-concrete interface. Jung (2006) decided to use multipoint constraint for rest of the research. Use of tie constraints (i.e multipoint constraints) or rigid connector elements to predict the load-displacement response of the Nebraska test girders described in chapter 3 resulted in higher capacity and stiffer response but the increase is not significant. During redundancy evaluation of twin box-girder bridge structures for the Marquette interchange project, Elza et al. (2004) used rigid links to model the full composite action at the interface. Similarly, Hunley and Harik (2011) used rigid beam elements to model the full composite action during redundancy evaluation of twin box-girder bridges. In the event of fracture, shear studs are subjected to combined tension and shear near the fractured region making it necessary to account for shear-tension interaction. In the following section, the validity of the rigid connector element approach is evaluated using nonlinear dynamic analysis with both *Implicit* and *Explicit* schemes. In the composite Hoan Bridge model, girders and stringers are made composite with the concrete slab through rigid beam connector element.

4.5.12.1 Eigen value analysis and damping coefficients

To obtain Rayleigh damping coefficients, eigenvalue analyses of the damaged and undamaged models of the composite Hoan Bridge is carried out in ABAQUS (2013). The first ten eigenvalues are tabulated in Table 4-23. The first and fourth eigenvalues from the undamaged bridge configuration are used to determine the Rayleigh damping coefficients at 2% and 5% critical damping and are tabulated in Table 4-24. Additionally the mass proportional damping coefficients are calculated using the first eigenvalue are also tabulated in Table 4-24. Figure 4-72

and Figure 4-73 shows the mode shapes for the first ten eigenvalues for the undamaged and the damaged composite bridge model respectively.

Table 4-23: Eigen values of Composite bridge model.

Eigen values	Frequency (Hz)		% Difference
	Undamaged bridge configuration	Damaged bridge configuration	
1	1.985	1.408	29.0
2	2.539	2.538	0.1
3	2.756	2.614	5.2
4	2.836	2.768	2.4
5	2.885	2.857	1.0
6	3.061	2.987	2.4
7	3.099	3.023	2.5
8	3.100	3.062	1.2
9	3.102	3.099	0.1
10	3.103	3.103	0.0

Table 4-24: Damping coefficients using undamaged composite bridge model.

Model description	Critical damping	Raileigh damping		Mass proportional damping
		α	β	α
Composite bridge model	2%	2.935E-01	1.300E-03	4.990E-01
	5%	7.337E-01	3.300E-03	1.247E+00

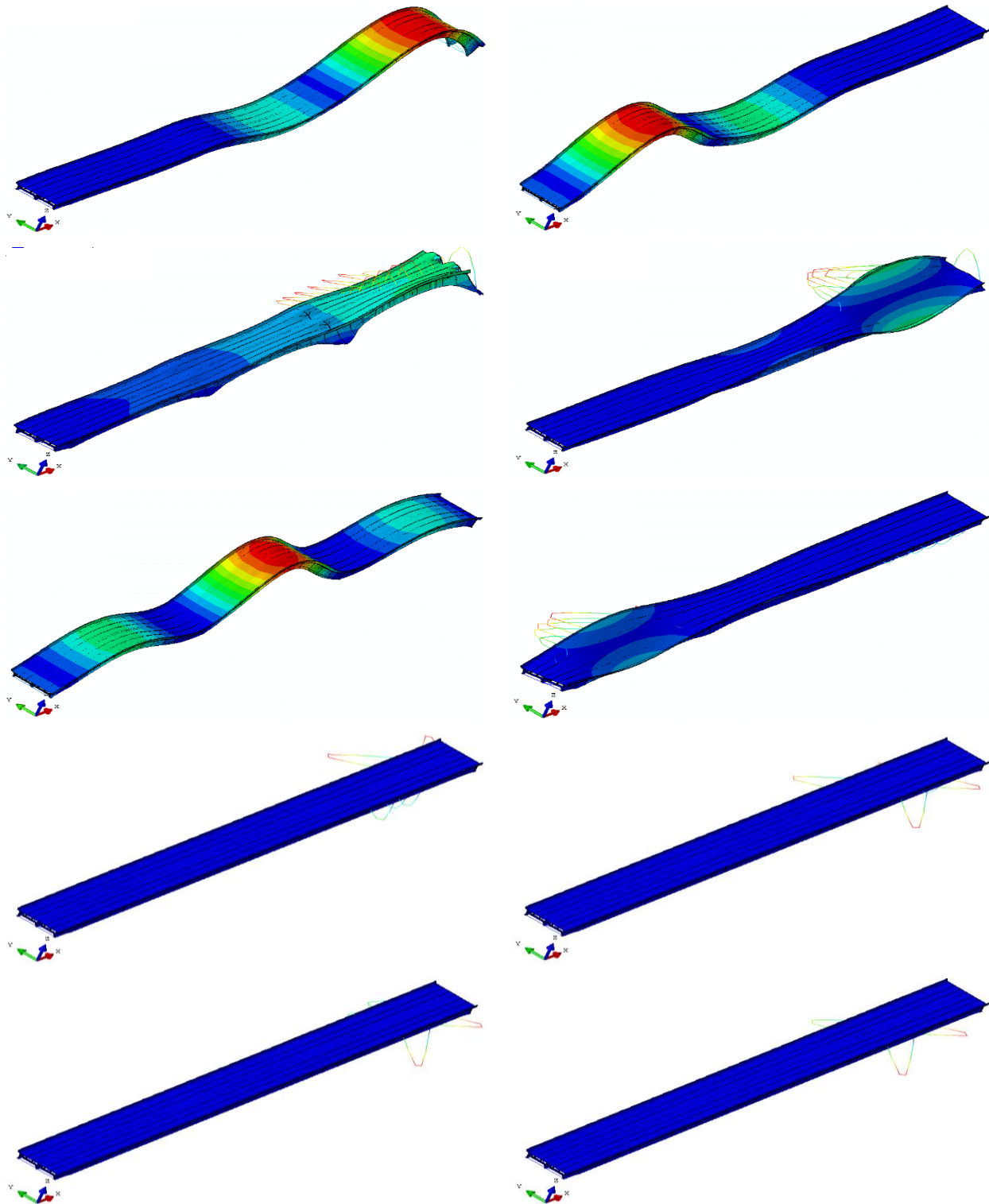


Figure 4-72: Mode shapes (Undamaged composite bridge model)

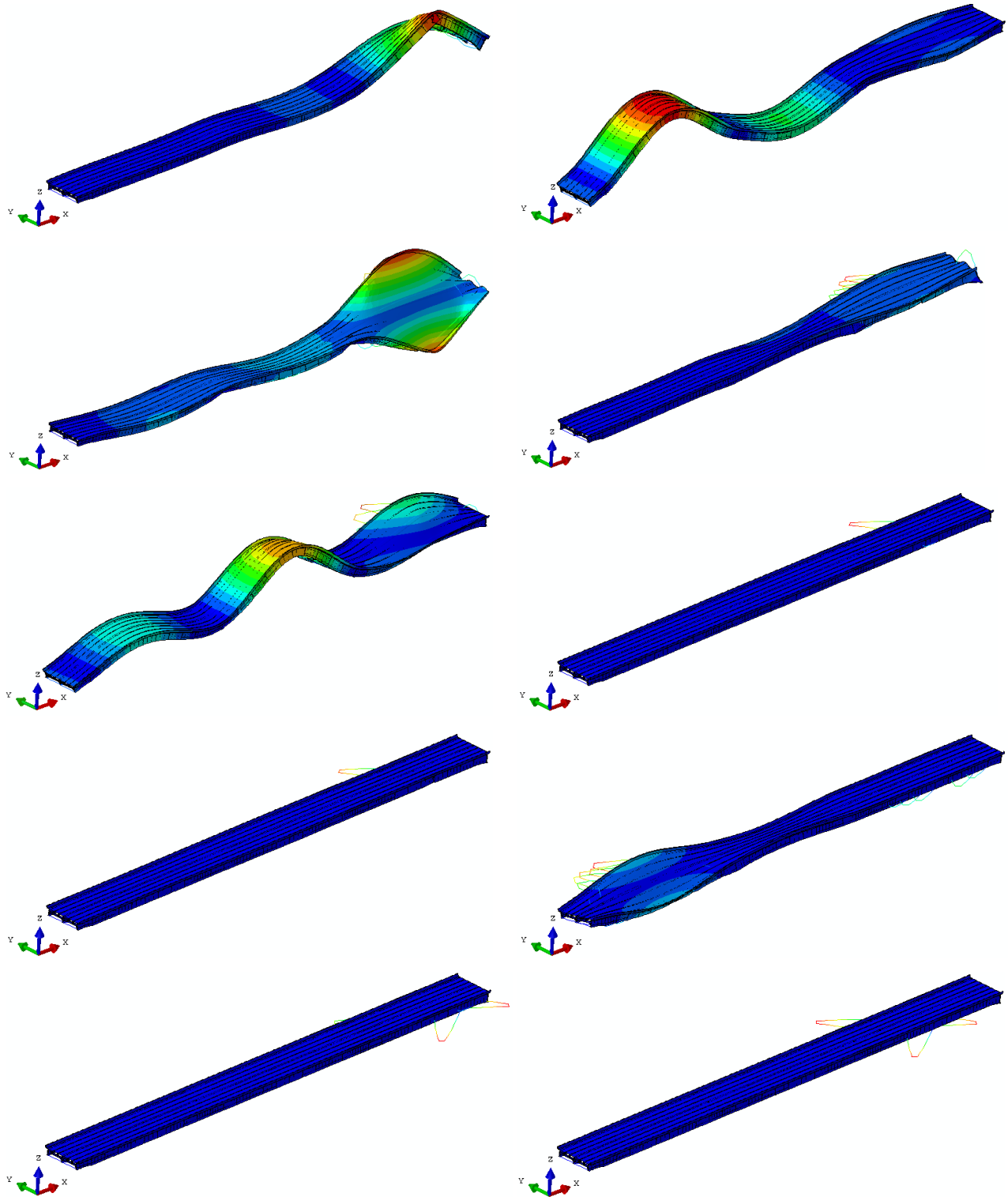


Figure 4-73: Mode shapes (Damged composite bridge model)

4.5.12.2 Post fracture response

Figure 4-74 and Figure 4-75 show the comparison for the intact girder D and the fractured girder F with 2% and 5% damping with the *Explicit* scheme. Post fracture, the fractured girder deflections in the composite model are about one fourth of the non-composite model fractured girder deflections. Intact girder deflection response reduced by ~2 in. with the composite bridge model compared to the non-composite bridge model. The intact girder deflection response is found to be insensitive to the critical damping for non-composite and composite bridge models. The fractured girder deflection reduced by ~20% and ~10% with increase in critical damping for non-composite and composite bridge model respectively. Post fracture forces in the rigid connector elements close to fracture increased drastically. This model failed to capture the unzipping of shear studs that might occur following a fracture. The behavior of shear studs under combined shear and tension, and its impact on the post fracture response of the bridge superstructure is evaluated detail in chapter 5.

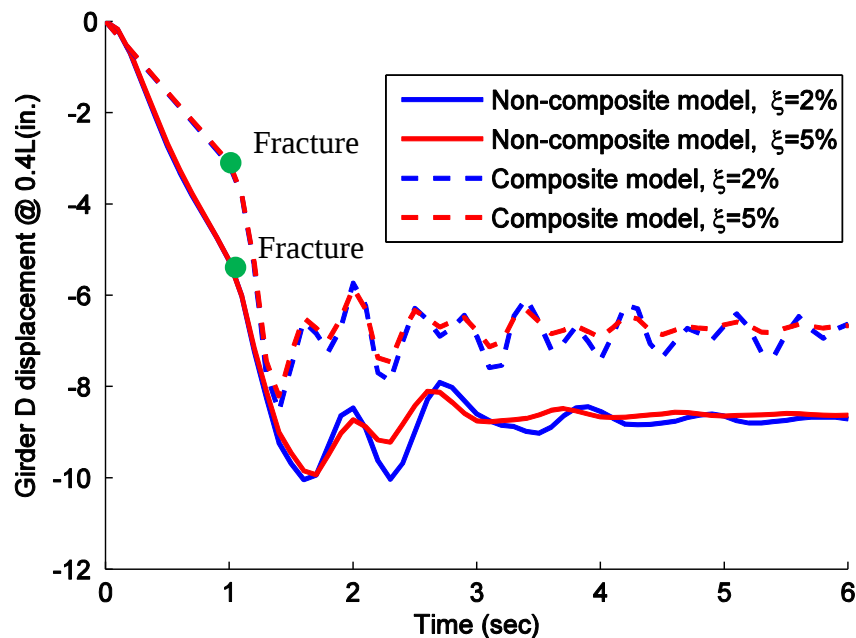


Figure 4-74: Girder D deflection at 0.4L (from Pier 2S) using *Explicit* scheme with 2% and 5% mass proportional damping for composite and non-composite bridge model.

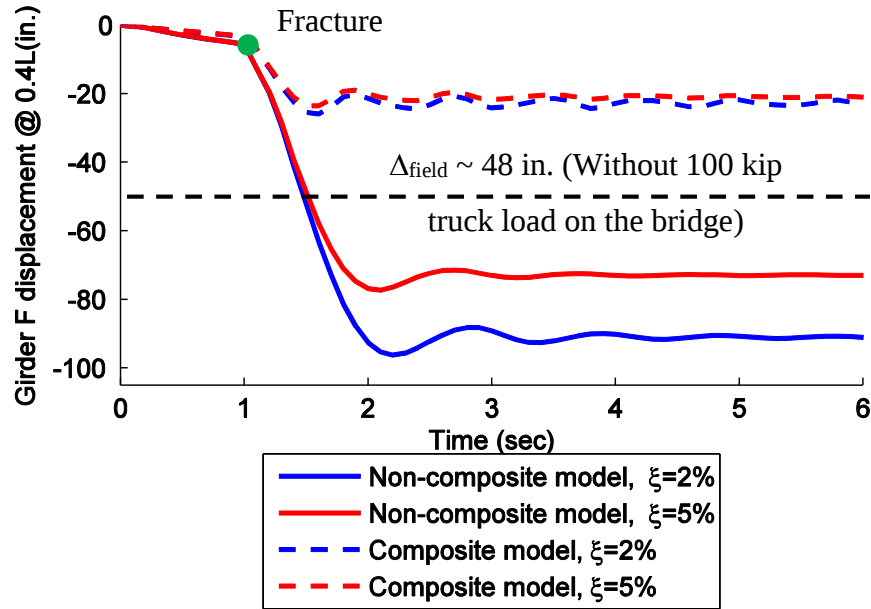


Figure 4-75: Girder F deflection at 0.4L (from Pier 2S) using *Explicit* scheme with 2% and 5% mass proportional damping for composite and non-composite bridge model.

4.6 Post fracture response

This section evaluates the impact of fracture location and fracture sequence on the post fracture response of Hoan Bridge. The nonlinear dynamic analyses in this section are performed using *Explicit* scheme with 1% mass proportional damping.

4.6.1 Post fracture response with additional partial web depth fractures in girder D

The parametric study carried out in Section 4.5 only considered fractures through the bottom flange and full web depth in girders E and F. Partial depth web fractures at three locations in girder D are excluded from the parametric study model used in Section 4.4. This decision was based on the assumption that the partial depth fractures in girder D would only affect the local stress distribution in girder D near fracture and not the global post fracture response of the bridge. In order to verify the validity of this assumption, the Hoan Bridge model is updated to include partial depth fractures at three location in girder D (Figure 4-3). In the first scenario (*Scenario-I*), partial depth fractures at three locations in girder D (Figure 4-3) are introduced simultaneously during the analysis along with the fractures in girders E and F. In the second scenario (*Scenario-II*), it is assumed that the partial depth web fracture in girder D at 0.5L was present before the events on December 13th, 2000. The bottom flange and full web depth fracture

in girders E and F, and the partial depth web fractures in girder D at 0.4L and 0.3L (from pier 2S) are introduced simultaneously during the analysis. Figure 4-76 and Figure 4-77 shows the post fracture displacement response of girder D and girder F at 0.4L (from pier 2S). Deflection response from the parametric study in Section 4.4 is also included in the graphs for comparison. From Figure 4-76 and Figure 4-77, both girders D and F yielded identical displacement response irrespective of the fracture scenarios.

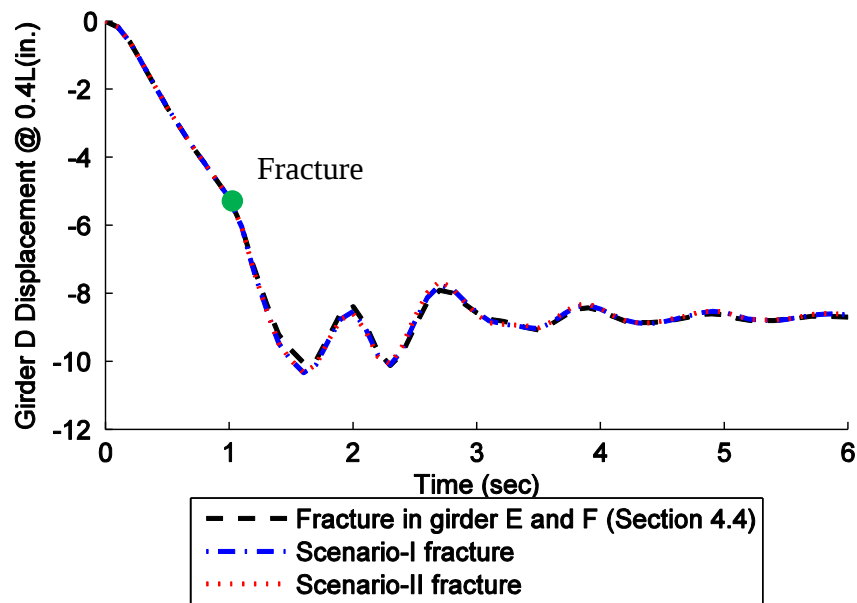


Figure 4-76: Dynamic deflection of girder D at 0.4L (from Pier 2S) following different fracture scenarios.

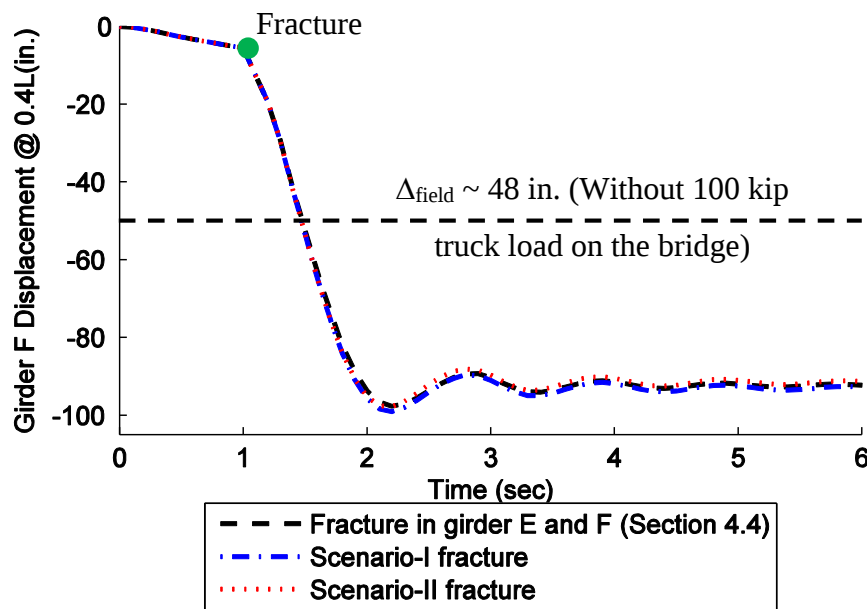


Figure 4-77: Dynamic deflection of girder F at 0.4L (from Pier 2S) following different fracture scenarios.

4.6.2 Impact of fracture sequence

On December 13th, 2000, full depth fractures were found in two of the three steel girders (E and F) and partial depth web fractures at three location in girder D. The order in which these fractures occurred is unclear. It is possible that fractures did not occur simultaneously. Results from Section 4.6.1 show that partial depth web fractures in girder D result in local change in stress distribution but insignificant impact on the global bridge response. Therefore, similar to the models used for parametric study in Section 4.4 damage in girders E and F is included while neglecting damage in girder D for the parametric study in this section. This section evaluates the impact of different fracture sequence scenarios on the post fracture response of the Hoan Bridge. Results from different fracture scenarios are compared with the results from Section 4.4 where fracture in Girder E and F is introduced simultaneously.

4.6.2.1 Scenario-I

In Scenario-I, Girder E (*Scenario-I EthenF*) or Girder F (*Scenario-I FthenE*) fractured first and the post fracture bridge response is simulated for 5 secs., damping out most of the oscillations and allowing bridge to reach its new deformed configuration. In the subsequent step, fracture is

introduced in Girder F (*Scenario-I_EthenF*) or Girder E (*Scenario-I_FthenE*) using the methodology discussed in Section 4.3.2. This fracture scenario is considered because it is possible that one of the girders fractured first and bridge stayed in service before fracturing the other intact girder.

Figure 4-78 and Figure 4-79 shows the deflection response of the intact girder D and the fractured girder F at 0.4L (from Pier 2S) respectively. Girder deflections at the fracture location at the end of analyses are tabulated in Table 4-25. The final deflection (at 0.4L from Pier 2S) of the intact girder D is found to be insensitive to sequence in which girder fractures are introduced (Figure 4-78). The analysis with *Scenario-I_FthenE* fracture sequence exhibited almost double the percentage decrease in the fractured girder response compared to the analysis with *Scenario-I_EthenF* fracture sequence (Figure 4-79 and Table 4-25).

4.6.2.2 Scenario-II

In Scenario-II, Girder E (*Scenario-II_EpeakF*) or F (*Scenario-II_FpeakF*) fractured first and the bridge is allowed to respond to the fracture event. As the fractured girder deflection reaches its peak value, fracture is introduced in Girder F (*Scenario-II_EpeakF*) or E (*Scenario-II_FpeakE*). Then the post fracture bridge response is simulated for 5 sec. This fracture scenario is considered because it is possible that as damaged bridge reached its peak deformation following fracture in girder E or F, it triggered fracture in the girder F or E.

Figure 4-80 and Figure 4-81 shows the deflection response of the intact girder D and fractured girder F at 0.4L (from Pier 2S), respectively. Girder deflections at the fracture location at the end of analysis are tabulated in Table 4-25. Overall trends in girder deflection (at 0.4L from Pier 2S) from fracture *Scenario-II* are similar to the trends observed with fracture *Scenario-I*. The final deflection (at 0.4L from Pier 2S) of the intact girder D is found to be insensitive to the sequence in which fracture is introduced (Figure 4-80). Similar to behavior under fracture *Scenario-I*, fractured girder F showed more reduction in girder deflection with *Scenario-II_FpeakE* fracture sequence (Figure 4-81 and Table 4-25).

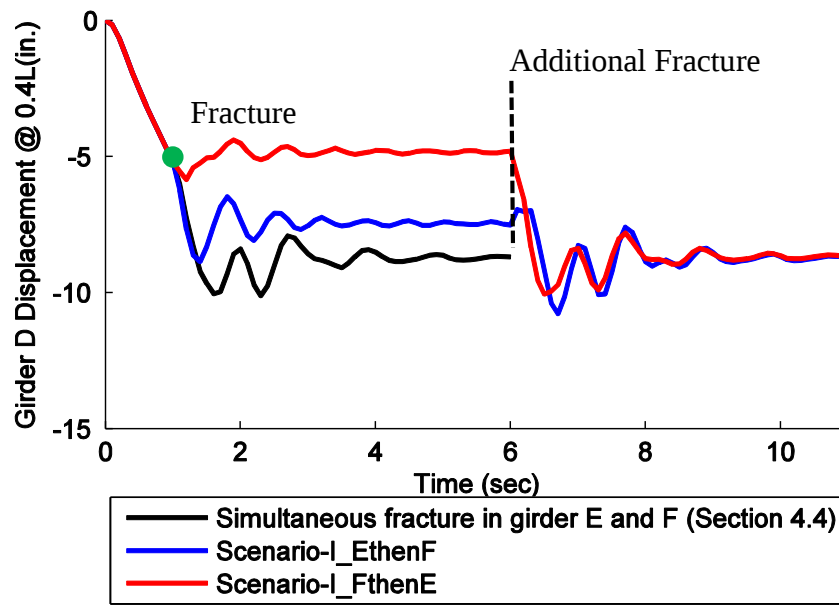


Figure 4-78: Dynamic deflection of girder D at 0.4L (from Pier 2S) following different fracture scenarios.

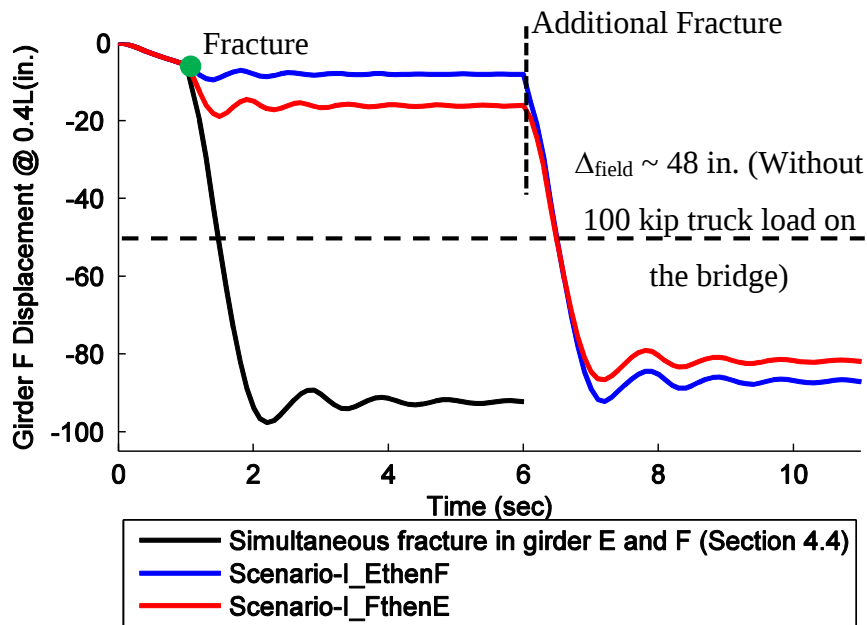


Figure 4-79: Dynamic deflection of girder F at 0.4L (from Pier 2S) following different fracture scenarios.

Table 4-25: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis.

Model description	At fracture location (0.4L)		
	Girder D	Girder E	Girder F
Simultaneous fracture in girder E and F (Section 4.4)	-8.70	-52.49	-92.29
Scenario-I_EthenF	-8.75 (-0.6%)	-50.11 (-4.5%)	-87.12 (-5.6%)
Scenario-I_FthenE	-8.70 (0%)	-47.22 (-10.0%)	-81.92 (-11.2%)
Scenario-II_EpeakF	-8.74 (-0.5%)	-48.02 (-8.5%)	-82.88 (-10.2%)
Scenario-II_FpeakE	-8.71 (-0.2%)	-45.24 (-13.8%)	-77.94 (-15.5%)

(Negative sign indicates downward displacement)

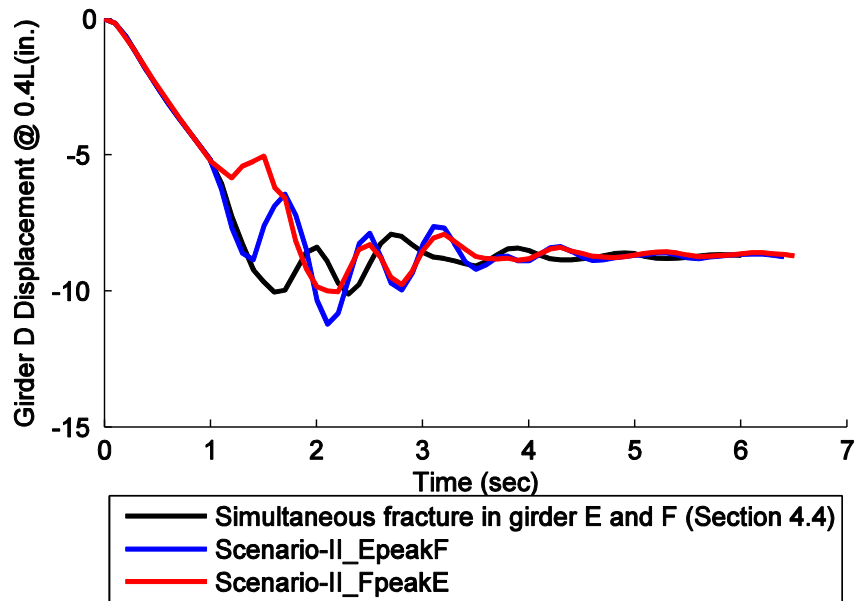


Figure 4-80: Dynamic deflection of girder D at 0.4L (from Pier 2S) following different fracture scenarios.

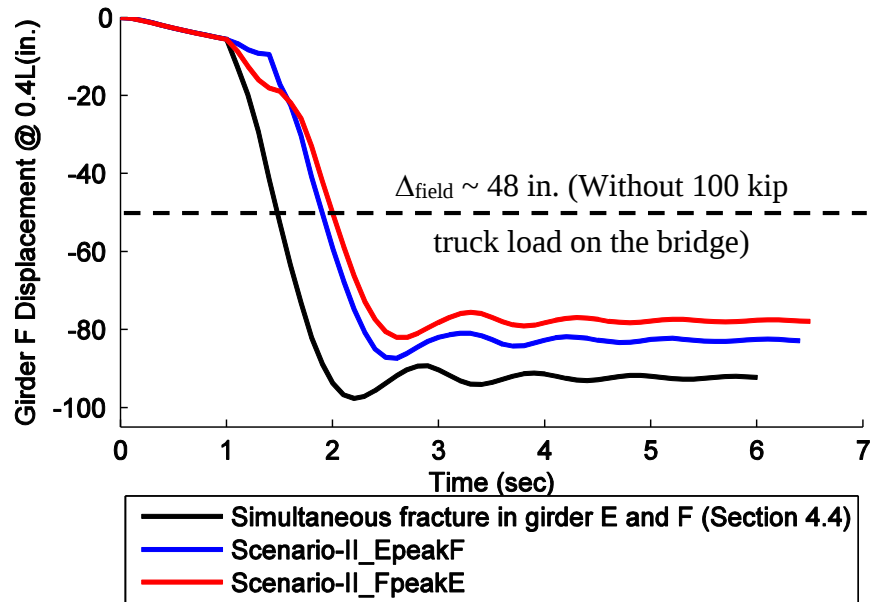


Figure 4-81: Dynamic deflection of girder F at 0.4L (from Pier 2S) following different fracture scenarios.

4.6.2.3 Impact of rate dependent material properties on post fracture response with Scenario-II fracture sequence

Scenario-II showed more reduction in deflection response of the bridge compared to *Scenario-I*. Additionally, results of the parametric study carried out in Section 4.5.11 showed reduction in overall bridge deflection with use of rate dependent material properties. Therefore, *Scenario-II_EpeakF* and *Scenario-II_FpeakE* are reanalyzed with rate dependent material properties. Rate dependent properties from UFC 3-340-02 (2008) for structural steel and reinforced concrete are used in this study. Figure 4-82 shows the deflection response of the fractured girder F at 0.4L (from Pier 2S) with and without rate dependent material properties. For analyses without the rate dependent material properties, girder F deflection at 0.4L (from Pier 2S) shows more reduction with *Scenario-II_FpeakE* fracture sequence. However, for the analyses with the rate dependent material properties, girder F deflection showed more reduction with the *Scenario-II_EpeakF* fracture sequence. Starting with the fracture in the interior girder E yielded lower girder F deflection at the start of the analysis and introduction of girder F fracture in the subsequent step resulted in larger increase in velocity and strain rate leading to more rate hardening behavior and reduction in girder F deflection in *Scenario-II_EpeakF* fracture sequence.

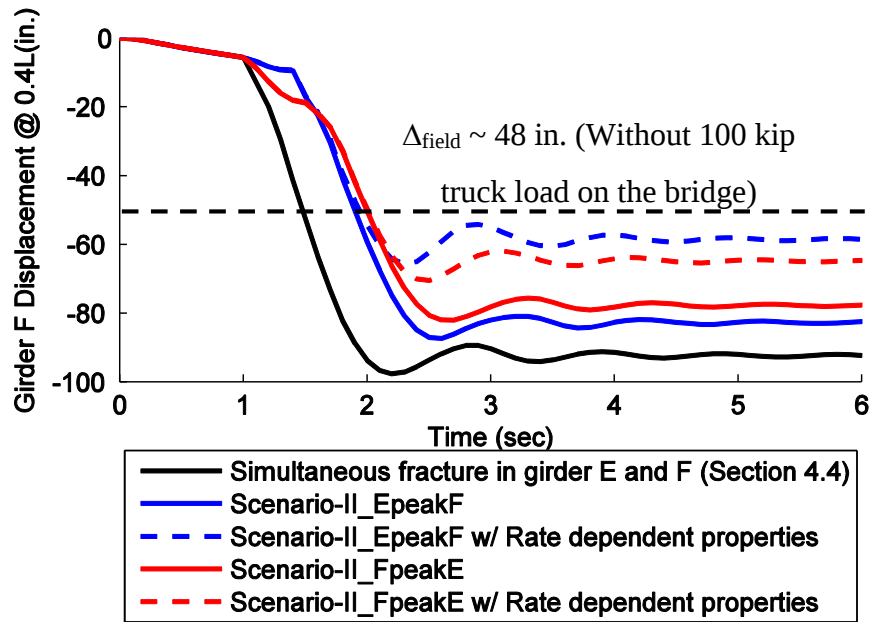


Figure 4-82: Comparison between the dynamic response of girder F at 0.4L (from Pier 2S) following fracture events in Scenario-II with rate dependent and independent material properties.

4.6.3 Impact of fracture location

This section presents the Hoan Bridge response following fracture near location of high shear (0.1L from Pier 3S) and maximum bending moment (0.4L from Pier 2S). The lower lateral bracing connection detail introduced stress triaxiality in the girder web. Stress triaxiality prevents yielding and redistribution of stress concentrations and can induce brittle failure in steel. Presence of susceptible details throughout the Hoan Bridge increased the probability of fracture at one location triggering fracture at multiple locations. In order to understand impact of fracture location and susceptibility to multiple fracture, the Hoan Bridge is analyzed with three fracture scenarios (fracture in girder E, girder F, and simultaneous fracture in girder E and F) at 0.1L and 0.4L (Figure 4-83).

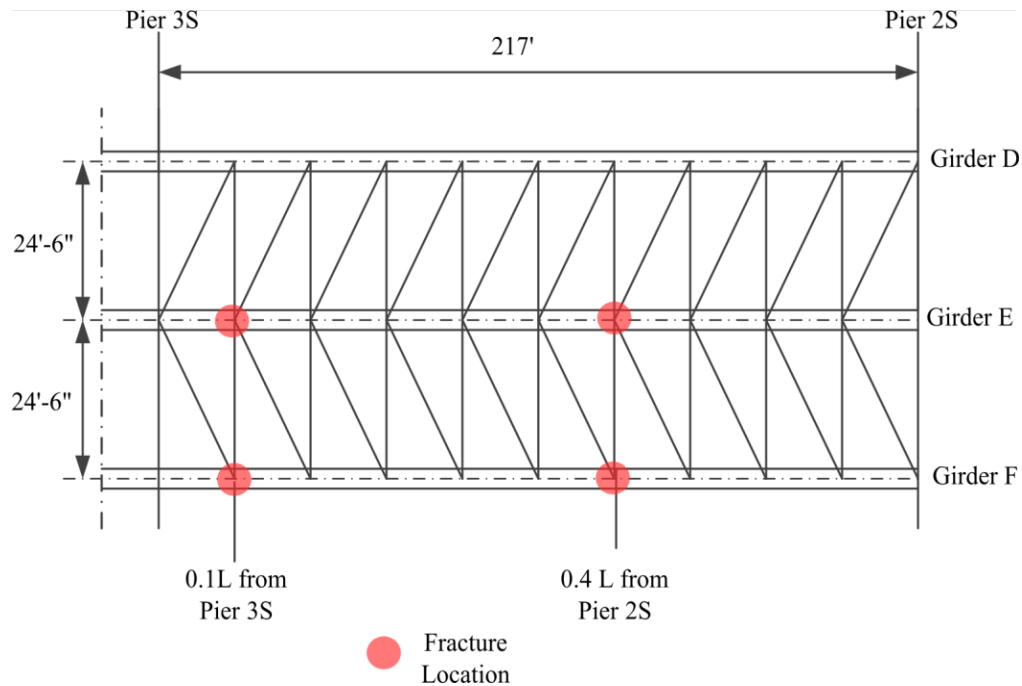


Figure 4-83: Fracture locations in end span

Figure 4-84 and Figure 4-85 shows the deflection responses of the intact girder D and the fractured girder F at 0.5L (from Pier 2S) following fracture at 0.1L from Pier 3S respectively. Figure 4-86 and Figure 4-87 shows the deflection responses of the intact girder D and the fractured girder F at 0.4L (from Pier 2S) following fracture at 0.4L from Pier 2S respectively. Simultaneous fracture at 0.4L resulted in higher deflection in both the intact girder D and the fractured girder F. Following fracture at 0.1L (from Pier 3S), load redistributed through the slab and the lower lateral bracing to the interior support (Pier 3S) yielding lower deflection compared to the deflection response following fracture at 0.4L (from Pier 2S).

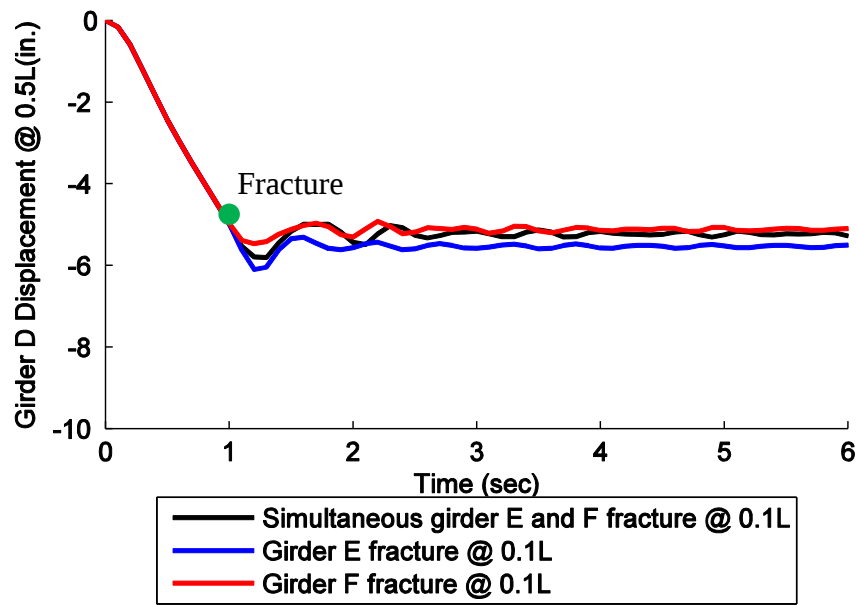


Figure 4-84: Deflection of girder D at 0.5L (from Pier 2S) following fracture at 0.1L (from Pier 3S).

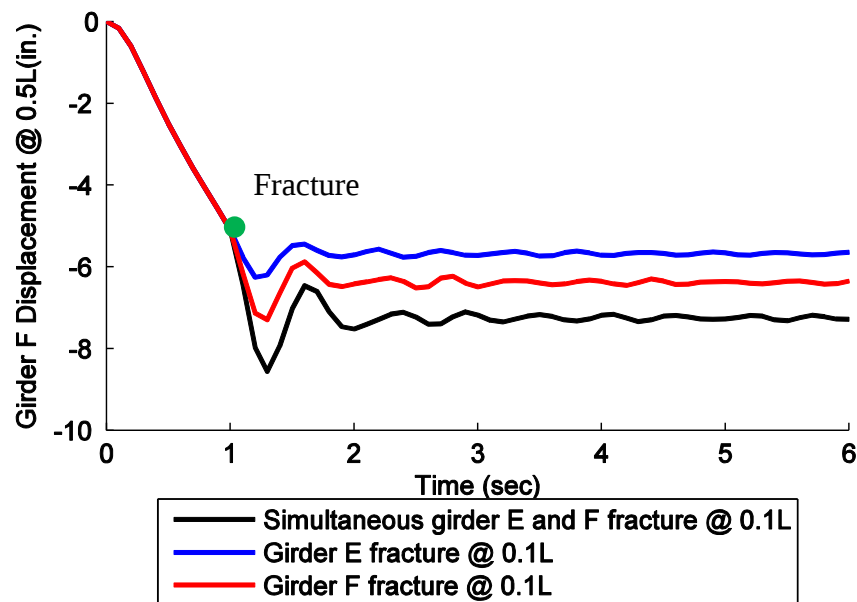


Figure 4-85: Deflection of girder F at 0.5L (from Pier 2S) following fracture at 0.1L (from Pier 3S).

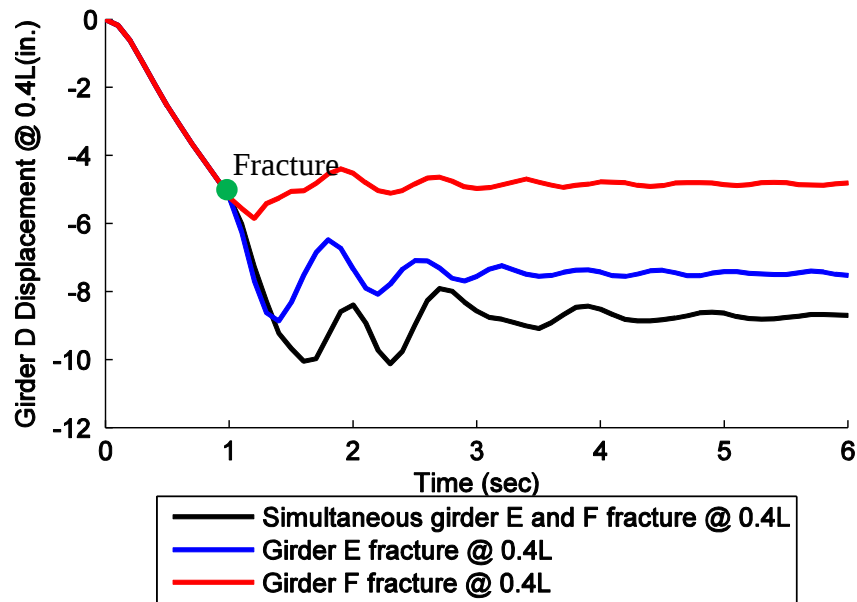


Figure 4-86: Deflection of girder D at 0.4L (from Pier 2S) following fracture at 0.4L (from Pier 2S).

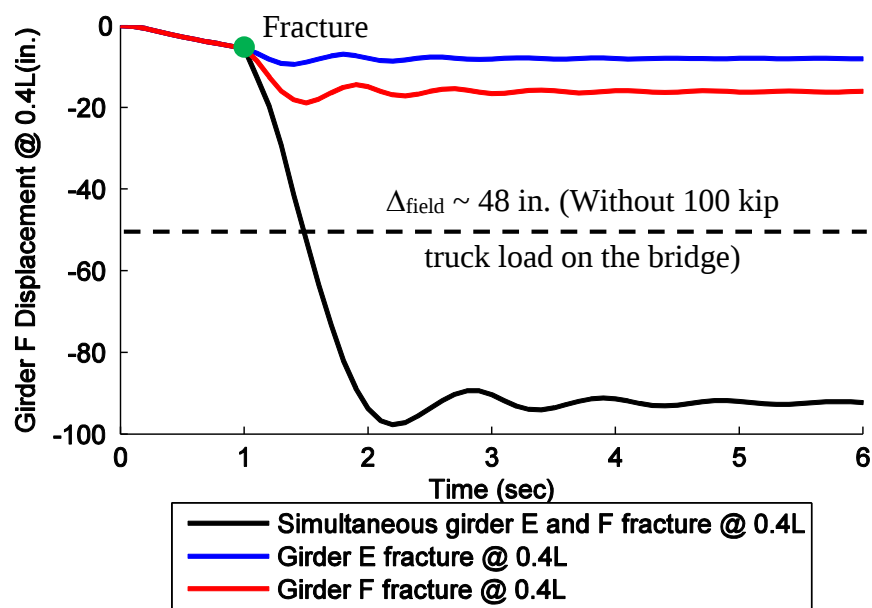


Figure 4-87: Deflection of girder F at 0.4L (from Pier 2S) following fracture at 0.4L (from Pier 2S).

4.7 Level of analysis required to capture post fracture response

Nonlinear dynamic analysis requires detailed finite element models and considerable computational resources and is not always feasible. Therefore, it is necessary to evaluate the suitability of simplified models to capture the post fracture response of the Hoan Bridge. In this section, six models with increasing level of complexity are analyzed to capture the post fracture response of the Hoan Bridge following bottom flange and full web depth fracture in girder E and F. The models are as follows:

1. *Linear_2(DL+LL)*: Linear quasi-static analysis with dead load and live load in the damaged span amplified by factor of 2, for reasons to be discussed next.
2. *Nonlinear_2(DL+LL)*: Nonlinear quasi-static analysis with dead load and live load in the damaged span amplified by factor of 2.
3. *Prefractured_Nonlinear_2(DL+LL)*: Nonlinear quasi-static analysis of the damaged bridge with dead load and live load in the damaged span amplified by factor of 2.
4. *Linear_Dynamic*: Post fracture response simulated using linear dynamic analysis.
5. *Nonlinear_Dynamic*: Post fracture response simulated using nonlinear dynamic analysis.
6. *Nonlinear_Dynamic_Rate*: Post fracture response simulated using nonlinear dynamic analysis with rate dependent material properties.

In this section, *ABAQUS/Explicit* scheme is used to perform the quasi-static analysis because *ABAQUS/Standard*'s static solvers required a very large number of iterations during each step to update the contact conditions at the steel-concrete interface and the material plasticity definition. In both the *Linear_2(DL+LL)* and the *Nonlinear_2(DL+LL)* models, fracture is introduced in *ABAQUS/Explicit* using the method discussed in Section 4.3.2. The *Prefractured_Nonlinear_2(DL+LL)* model starts with a damaged model of the bridge. Amplified dead and live load are applied gradually in *ABAQUS/Explicit* to produce a quasi-static response. Section 4.3.3 discusses the selection of the correct duration to apply load to produce quasi-static response. After a couple of iterations to obtain the correct times, the *Linear_2(DL+LL)* model is analyzed over a duration of 3.75 sec.; while the both the *Nonlinear_2(DL+LL)* and the *Prefractured_Nonlinear_2(DL+LL)* models are analyzed over a duration of 7.5 sec to obtain quasi-static response. The ratio of the kinetic energy (*KE*) to the internal energy (*IE*) is kept at less than 5%. The results from the fracture introduction step for *Linear_2(DL+LL)* and

Nonlinear_2(DL+LL) model as well as the application of the amplified dead load and live load step for the *Prefracted_Nonlinear_2(DL+LL)* model are normalized over 1 sec in . The UFC 4-023-03 guidelines (2009) used in the design of buildings to resist progressive collapse amplify gravity load combinations in the bays immediately adjacent to the removed element and at all floors above the removed element in linear and nonlinear static analysis to account for the inertial effects. In the case of a bridge superstructure, the span with the damaged member can be considered similar to the bays in a building. The maximum amplification used by UFC 4-023-03 guidelines (2009) is equal to 2.0. Therefore, the dead load and live load in the damaged span is amplified by a factor of 2.0.

The *Nonlinear_2(DL+LL)* model failed to converge during gradual introduction of the fracture, with the deflection reaching unrealistically high values. As the analysis failed during the fracture introduction step, the model has residual forces at the location of fracture. Therefore, the output from the last successful time increment from the analysis can only be used to compare different levels of analysis. The *Prefracted_Nonlinear_2(DL+LL)* model failed to converge during the application of the amplified load. As the analysis started with fractured girders E and F, there are no residual forces in the model at the location of fracture compared to *Nonlinear_2(DL+LL)* model. The dynamic analysis models were able to get a converged solution following the fracture event. The quasi-static analysis models yielded unrealistic girder deflections and failed to capture the bridge deflection response at 0.4L (Figure 4-89). If the results from the nonlinear quasi-static analysis with load amplification are used to make decisions regarding retrofit and rehab recommendations, it can lead to very high demand on certain members and connections. The load amplification factor needs to be further evaluated in order to get realistic response using simplified models. The fractured girder displacements in *Linear_2(DL+LL)* model are less than 10% off compared to the fractured girder displacement in the *Nonlinear_Dynamic_Rate* model (Table 4-26 and Figure 4-89). The mean deflection of the intact girder showed little to no sensitivity to the level of dynamic analyses (Figure 4-88).

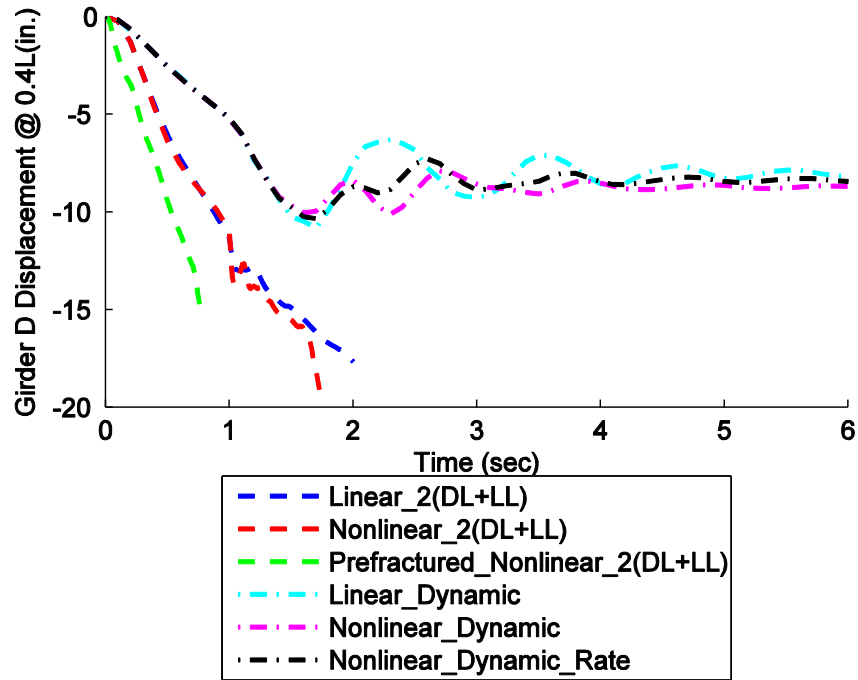


Figure 4-88: Girder D response for different levels of analysis.

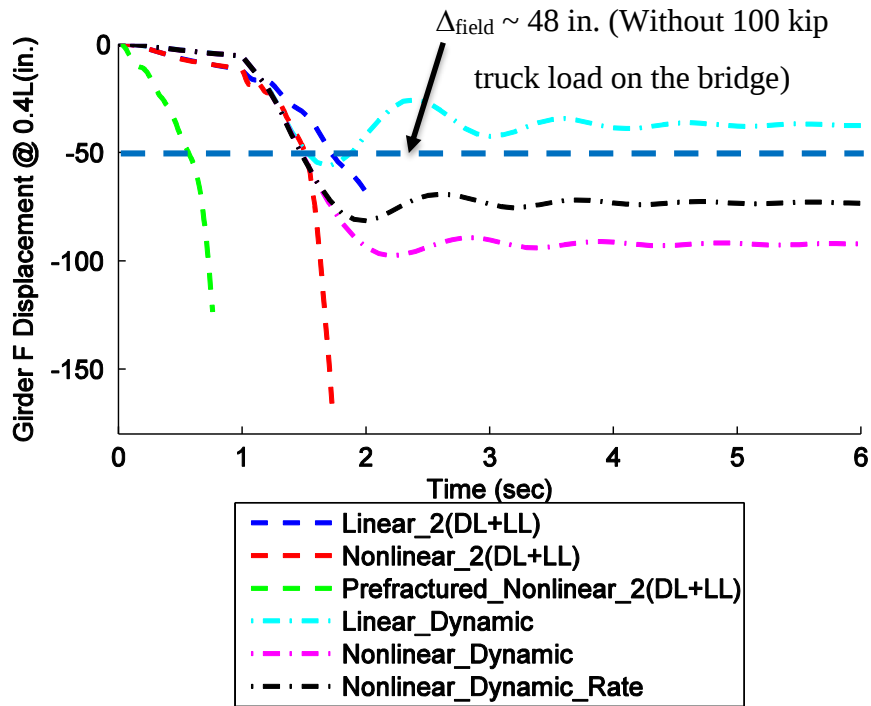


Figure 4-89: Girder F response for different levels of analysis.

Table 4-26: Girder displacement at the location of fracture (0.4L from Pier 2S) at the end of analysis for different levels of analysis.

Model	Step	At fracture location (0.4L)		
		Girder D (in.)	Girder E (in.)	Girder F (in.)
Linear_2(DL+LL)	2(DL+LL)	-11.3	-11.9	-12.1
	Post Fracture response	-17.7	-45.5	-68.4
Nonlinear_2(DL+LL)	2(DL+LL)	-10.9	-11.5	-11.8
	Post Fracture response @ t = 0.73 sec	-19.2	-96.6	-171.6
Prefractured_Nonlinear_2(DL+LL)	2(DL+LL) @ t = 0.76 s	-14.8	-72.1	-123.4
Linear_Dynamic	DL+LL	-5.2	-5.5	-5.6
	Post Fracture response	-8.2	-24.1	-37.6
Nonlinear_Dynamic	DL+LL	-5.2	-5.5	-5.6
	Post Fracture response	-8.7	-52.5	-92.3
Nonlinear_Dynamic_Rate	DL+LL	-5.2	-5.5	-5.6
	Post Fracture response	-8.5	-42.9	-73.5

4.8 Conclusion

Examination of all the results obtained so far lead to the following conclusions.

1. The stiffness proportional coefficient used by Rayleigh damping tends to over-damp higher modes and can result in unrealistically high damping, while mass proportional damping underdamps higher mode effects. In a linear dynamic analysis, Rayleigh damping can be used to account for sources of damping (hysteretic damping/energy dissipation from material yielding, coulomb damping/energy dissipation from friction at the steel-concrete interface, energy dissipated at the connections) that are not explicitly modeled in the analysis. However, use of Rayleigh damping in non-linear dynamic analyses will yield overdamped results.
2. The *Implicit* solution scheme with mass proportional damping took significantly longer to complete the analysis as compared to the *Explicit* solution scheme. Thus, it is important to determine the robustness of an *Explicit* solution scheme for collapse-type problems. While a

large number of studies have been conducted along these lines, a final conclusion has not been reached yet.

3. The absence of concrete slab in the bare girder model resulted in large girder displacements leading to lack of convergence in the solution. Presence of the concrete slab in the non-composite model reduced girder displacements and allowed the *Explicit* scheme to reach a converged solution. In the Hoan Bridge, the girder and stringer top flanges are encased in the concrete haunch, providing partial composite action that is difficult to quantify. Results from the non-composite model provide a conservative estimate of the global response, but an excessively conservative solution is not necessarily what is desirable in this type of analyses. In addition, partial composite action on the bridge could possibly result in higher demand on the connections near the fractured region than those predicted by non-composite model.
4. Secondary elements (guardrail and lower lateral bracing) play crucial role in the load redistribution following fracture. A correct analytical representation of the secondary elements is essential to capture the post fracture response. Excluding secondary elements from the bridge model would lead to modification in the load redistribution and possibly leading to false perception of safety.
5. The Hoan Bridge had an unusual support configuration that did not allow free expansion and contraction of bridge superstructure from thermal cycles. Additional support configuration that allowed free expansion and contraction of bridge superstructure is also analyzed and post fracture deflections increased by ~20-25% with reduction in external restraints.
6. The Hoan Bridge was constructed in the early '70s. In the absence of slab details in the available design drawings, slab is designed assuming 4000 psi concrete with Grade 40 reinforcement. In long span bridges such as Hoan Bridge, construction is done in stages using concrete from multiple batches. The mean strength of the slab concrete is typically higher than the specified value. Increases in concrete strength resulted in decrease in girder deflections.
7. An increase in material strength under rapid loading rates decreased the fractured girder response by ~20% while intact girder response is found to be insensitive to the rate dependent material properties. Inclusion of rate dependent material properties is necessary to capture demand on connections close to fracture.

8. Modeling of full composite action using rigid connector elements showed significant reduction in bridge deflection. The fractured girder deflections in the composite model are about one fourth of the non-composite model fractured girder deflections. This model failed to capture the unzipping of shear studs that might occur following a fracture event.
9. Partial depth web fractures in girder D affected the local stress distribution in the girder web but it did not affect the global bridge response in the analysis where fracture in all three girders is introduced simultaneously.
10. The order in which fractures occurred on the Hoan Bridge is unclear. It is possible that fractures did not occur simultaneously. Fractured girder E and F deflection response is found to be sensitive to the fracture sequence while intact girder D showed less than 1% change in deflection response. Introduction of fracture in the exterior girder F followed by interior girder E fracture (Scenario-I_FthenE and Scenario-II_FpeakE) showed higher percentage reduction in fractured girder deflection response compared to introduction of fracture in the girder E followed by girder F fracture (Scenario-I_EthenF and Scenario-II_EpeakF).
11. In the event of fracture at 0.1L (from Pier 3S), load redistributed through the slab and the lower lateral bracing to the interior support (Pier 3S) yielding lower deflection compared to the deflection response following fracture at 0.4L (from Pier 2S). However, post fracture demand on members and connections is going to be different for fracture near location of high shear (0.1L from Pier 3S) and maximum bending moment (0.4L from Pier 2S). Investigation of multiple of fracture scenarios is necessary to assess load ranges for different members and connections on the bridge.
12. *ABAQUS/Standard's* static solvers required large number of iterations during each step to update contact condition and material plasticity definition. Therefore, *ABAQUS/Explicit* scheme is used to perform the quasi-static analysis. Quasi-static analysis models yielded unrealistic girder deflections and failed to capture the bridge deflection response. Load amplification factor needs further evaluation to get realistic response using simplified models.

5 Twin box-girder test at The University of Texas at Austin

5.1 Introduction

This chapter describes one of the two cases that will be used as the primary calibration study in this research. Barnard et al. (2010) at The University of Texas at Austin investigated the capacity of a simply supported twin box-girder bridge (Figure 5-1) after fracture as part of a project funded by Texas Department of Transportation (TxDOT) and Federal Highway Administration (FHWA). This study is an important benchmark for the redundancy analysis since it is one of the few where detailed experimental results are available.

In this chapter, Section 5.2 provides a high-level description of the UT Austin test, including a description of the three major tests carried out by the UT Austin investigators. Section 5.3 provides a detailed description of the modeling methodology used for this twin box-girder simulation. Some aspects of the modeling methodology are similar to the ones used during the post fracture analysis of the Hoan Bridge described in Chapter 4. Section 5.4 provides the details about the series of steps used to simulate test sequence. Section 5.5 discusses the modeling caveats that reader should consider during assessing the results from the simulation. Section 5.6 provides a comparison between the field observations and results from the finite element analysis. Following the comparisons with the field data, the impact of geometric and modeling parameters on the response of twin box-girder tests is evaluated in Section 5.7. Section 5.8 evaluates the ability of simplified methods to estimate the post fracture capacity of twin box-girder bridge in Test 3. Section 5.9 evaluates the possibility of bridge shakedown from the Test 3 loading. Section 5.10 evaluates the effect of instantaneous fracture on the bridge post fracture response and the remaining load carrying capacity of the damaged bridge.

5.2 Bridge Description

The test bridge consists of curved simply supported 120 ft twin box-girders supporting a 23 ft -8 in. wide and 8 in. thick composite slab (Figure 5-1, Figure 5-2 and Figure 5-3). The average haunch height above the flanges was 3 in. The steel box-girders were 4 ft – 9 in deep with a 47 in. wide by 3/4 in. thick bottom flange, 1/2 in. thick web, and 12 in. wide by 5/8 in. thick top flanges. The web and flanges are made of ASTM A572 Grade 50 steel. The lateral and cross

bracing, stiffeners, diaphragms are made of ASTM A36 steel. The intermediate diaphragms were installed at 10 points along the length of the bridge. Temporary bracing was installed at 0.4L and 0.6L to provide torsional stiffness to the box-girders during construction of the slab and the guardrail. The slab was made composite with the box-girder top flanges using 64 equally spaced three-stud groups (7/8 in. diameter and 5 in. tall stud).

The reinforced concrete slab was made of TxDOT class S-type concrete with a 28-day compressive strength of 4000 psi or greater and grade 60 reinforcement. A TxDOT standard T501 rail was constructed in place after the deck had wet cured for 4 days providing as much strength as possible and while still mimicking standard practice in the field (Barnard, 2006). To maximize the effects of fracture in the exterior girder, joints in the guardrail were spaced every 30 ft to ensure that the rails would be separated at mid-span. The joints were formed using ¾ in. polystyrene foam insulation. The T501 rail was constructed using Austin class S concrete. The average compressive strength values for slab and guardrail concrete at the time of three tests are tabulated in Table 5-1. The compression test performed before Test 3 indicate an unexpected decrease in guardrail concrete strength. As per Neuman (2009), the possible reasons for reduction in concrete strength is the poor capping surface on concrete cylinders. Therefore, the guardrail concrete compressive strength from Test 2 is used for Test 3 simulation.

The bridge was supported using 22 in. long, 11 in. wide and 3 in. thick elastomeric bearing pads at both ends. The pads were reinforced with nine steel reinforcing plates (1/8 in. thick) encased in layers of elastomeric material (3/16 in. thick). An additional cross-frame (Figure 5-4) made of angle shapes was welded inside the exterior girder two feet from the fracture location on the side opposite the existing cross-frame to help provide similar resistance to twisting at both open ends of the fractured exterior girder (Neuman, 2009). Neuman (2009) doesn't provide details about the angle shapes used to make the additional cross frames. In this study it is assumed that the additional cross frames are made up of the same angle section used for temporary construction bracing.

Three separate tests, labelled Tests 1, 2 and 3, were performed on the bridge. In Test 1, a fracture was first introduced in the bottom flange of the exterior girder using an explosive shape charge to investigate the post fracture bridge response (Figure 5-5). The damaged bridge was then loaded using five concrete girders (total weight 76 kips) to represent a design truck near mid-span. The

concrete girders were positioned to induce the maximum bending moment at the fracture location in the exterior girder. The fracture in the bottom flange did not propagate into the girder webs and minimal change in deflection was observed in the fractured girder.

In Test 2, a web fracture up to 10 in. below the weld to the top flange in addition to the bottom flange fracture from Test 1 was introduced in the exterior girder by cutting the girder web while supporting it with temporary scissor jack support. The bridge was then loaded using the same five concrete girders used in Test 1. The capacity of the bridge to sustain a large release of potential energy following the girder web fracture was evaluated by severing the tension tie in the scissor jack support using explosives and returning the bridge to its natural simply-supported configuration (Figure 5-6). Extensive cracking was observed on the top surface of the concrete deck. After releasing the scissor jacks, the mid-span of the intact girder deflected downwards by additional 2.9 in. while the fractured girder deflected downwards by additional 7.0 in. The deflected shape of the fractured girder resembled that of two partially restrained cantilevers pinned at the center (Neuman, 2009). Following the fracture in Test 2, the top flanges of the fractured girder separated from the concrete slab because of concrete failure above the shear studs under combined shear and tension.

During Test 2, the bridge did not collapse when loaded with the design truck load at mid-span. Therefore, Test 3 (Figure 5-7) was planned to determine the reserve load carrying capacity of the damaged bridge and observe the sequence of failure mechanisms. In Test 3, the load is applied in three stages. In the first loading stage (Test 3a), the five concrete girders used in previous tests were rearranged with an additional girder to form a rectangular bin on the bridge deck. During rearrangement of the concrete girders to form the bin for the road base, the fracture on the outside web of the fractured girder propagated to the full web depth when total load on the bridge was 40.9 kips. The total weight of six girders was 82.1 kips. The rectangular bin was centered longitudinally along the mid-span to maximize the bending moment in the fractured girder. In the second loading stage (Test 3b), the rectangular bin was filled with 258.33 kips of the road base using a crane-mounted lift bucket. In the third loading stage (Test 3c), additional road base was deposited between the rectangular bin and the guardrail. The bridge collapsed when weight of the additional road base in Test 3c reached 22.9 kips. The total load on the bridge at time of collapse was 363.3 kips. The load on the bridge was intended to represent an

HS-20 truck load configuration although it did not have the distinct loading axle because of the manner in which the road base material was applied to the test bridge. Beyond 360.2 kips load, bridge deflection grew rapidly with additional load reaching vertical displacement of ~25 in. at a location 18 ft south of fracture in the exterior girder. The fracture in the interior web of the exterior girder propagated full depth of the web during collapse. Neuman's (2009) thesis provides detailed information about the bridge geometry, testing protocol and test results.



Figure 5-1: Twin box-girder test bridge (Barnard, et al., 2010).

Table 5-1: Bridge concrete properties.

Component	Average f_c' (ksi)			
	Average f_c'	Test 1	Test 2	Test 3
Slab	4.84	5.37	6.26	6.26
Interior railing	5.34	5.95	6.63	6.6
Exterior railing	4.74	4.9	6.27	5.49

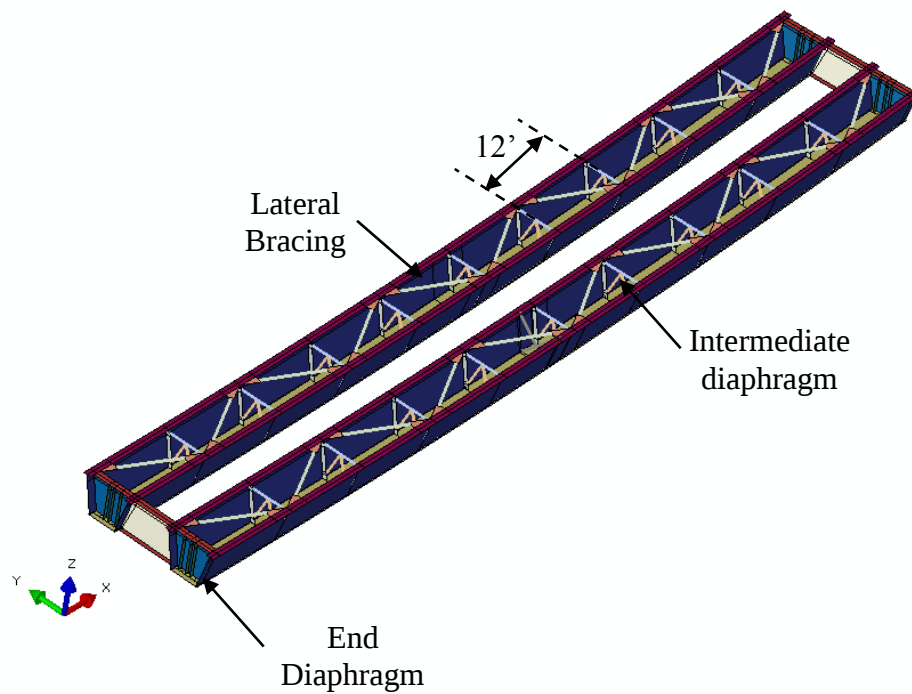


Figure 5-2: Twin box-girder layout.

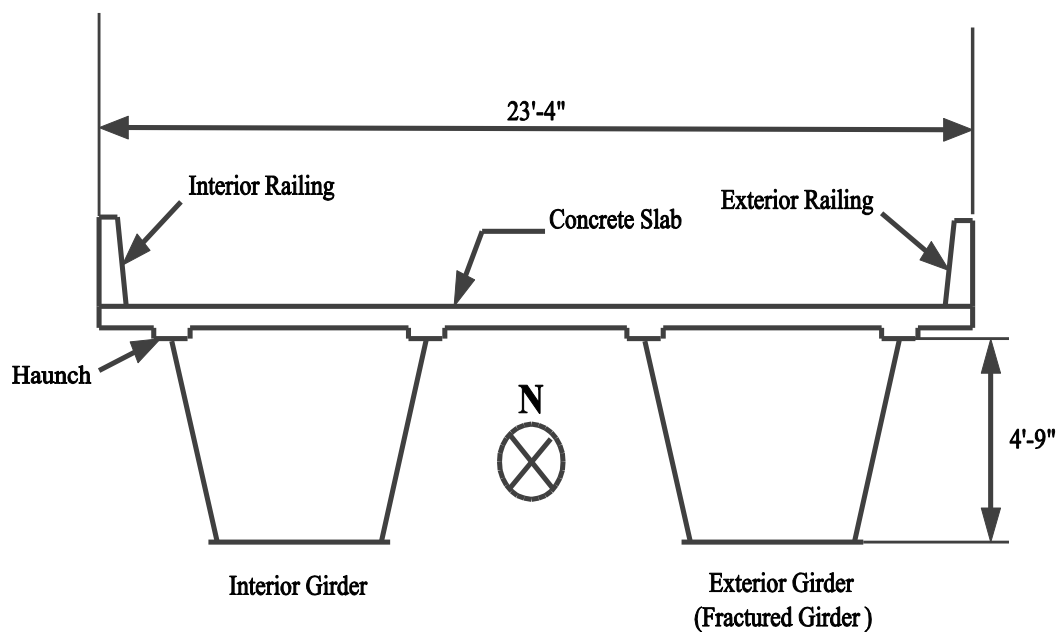


Figure 5-3: Typical twin box-girder bridge cross-section (Not to scale).



Figure 5-4: Additional cross frame installed inside the exterior girder (Neuman, 2009), used under fair use.



Figure 5-5: Bottom flange simulated fracture by explosion in Test 1 (Barnard, et al., 2010).



Figure 5-6: Test 2 (Barnard, et al., 2010).



(a) Loading process

(b) Collapsed bridge

Figure 5-7: Test 3 (Barnard, et al., 2010).

5.3 Modeling methodology

A detailed finite element model of the twin box-girder bridge (Figure 5-8) is constructed using ABAQUS (2013). In ABAQUS (2013) the individual bridge components are developed separately and then assembled together. The box-girder components are modeled using a general-purpose four node shell element with reduced integration (S4R). The web is modeled using shell elements at its mid-surface while the flanges are modeled at the flange to web connection with an offset equal to the half of the flange thickness. Steel plasticity is modeled using classical metal plasticity with von Mises yield criteria. The input parameters for the steel plasticity are defined as per Section 3.2.2.1. The reinforced concrete slab is modeled with S4R shell elements with reinforcement modeled as a smeared layer at the appropriate location. The

concrete behavior is modeled using the damaged plasticity model while the rebar behavior is modeled using the classical metal plasticity model with von Mises yield criteria. The input parameters for the concrete and the rebar plasticity are defined as per Section 3.2.2.1.

The lateral bracing and the cross bracing are modeled using beam elements (B31) with the predefined beam cross section. The guardrail is modeled using eight node solid elements with reduced integration (C3D8R). The guardrails are typically mildly reinforced. In this study, it is assumed that exclusion of reinforcement in the guardrail idealization will not drastically alter the global bridge response and therefore, reinforcement is not included in the guardrail idealization. The expansion joints in the guardrail are modeled with the actual gap of $3/4^{\text{th}}$ in. and the hard contact definition at the guardrail ends. The concrete plasticity in the guardrail is also modeled using the damaged plasticity model. The guardrail is made composite with the bridge slab using the tie constraints. Tie constraints are also used to connect the temporary external bracing to the box-girder webs.

Instead of modeling shear studs at the discrete locations along the bridge length, all the nodes at the steel-concrete interface are connected using (Cartesian + Cardan) connector elements. The stiffness and stud capacity values are adjusted to give the same interface behavior as discrete connections. The shear stud behavior is modeled using the shear-tension failure envelope using the connector damage initiation definition in ABAQUS (2013). Section 3.2.4.2 provides more details about the modeling of shear-tension failure envelope. The shear stud model used in this study does not account for increase in stud strength associated with the rate dependent behavior. Rotational degrees of freedom at the interface are assumed to be fully coupled. Coupling is done through use of rotational stiffness of 100,000 kip-in/rads.

The box-girder fractures are modeled by introducing duplicate nodes at the predefined fracture path. The duplicate nodes are connected using zero length beam/weld connector elements to mimic the intact condition. In *ABAQUS/Standard*, fracture event is simulated by deactivating zero length beam/weld connector elements using the “Model Change” option as discussed in Section 4.3.2. In *ABAQUS/Explicit*, fracture is simulated using the methodology discussed in Section 4.3.2.

The bearing pads are modeled using connector elements with the elastic stiffness values determined from the equations used for design of elastomeric bearing pads from Section 14.6.3.1-2 of AASTHO specification (2012). The connector elements connect the nodes on the bottom flange over the area in contact with the bearing pad with the fixed reference nodes modeled at the bottom of the bearing pad. The elastic stiffness values used to model the bearing pad are calculated using Equation 5-1 through Equation 5-6. The stiffness values used to model the bearing pad in this research are tabulated in Table 5-2. The axial tensile stiffness of the bearing pad is assumed to be negligible ($K_{tension} = 10^{-3} \frac{kips}{in}$) and axial compressive stiffness is calculated using Equation 5-1. In this study, shear modulus of the elastomer (G) is taken as 0.1 ksi.

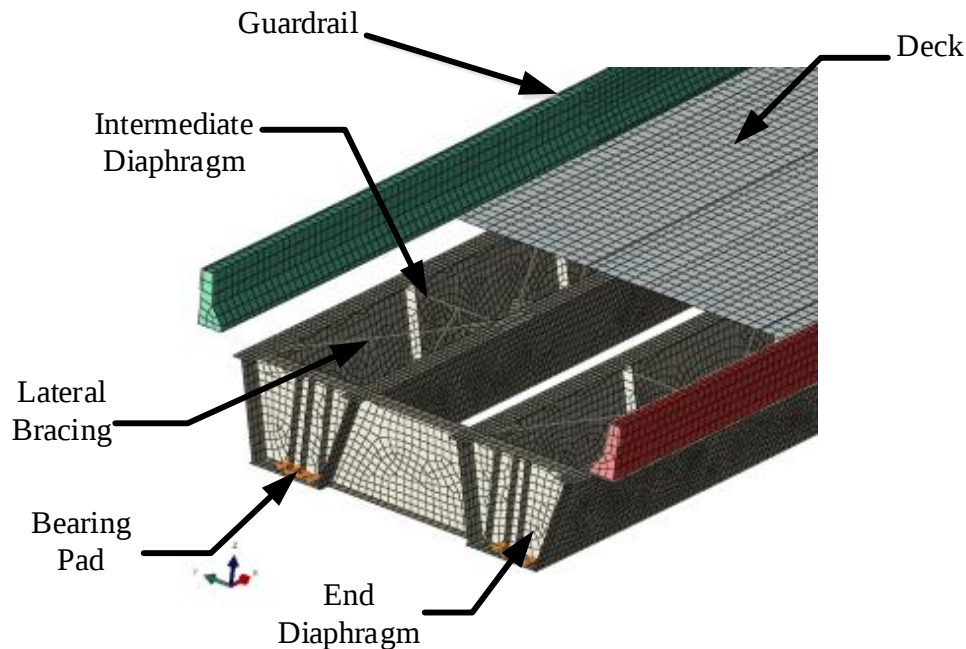


Figure 5-8: Finite element model of twin box-girder bridge.

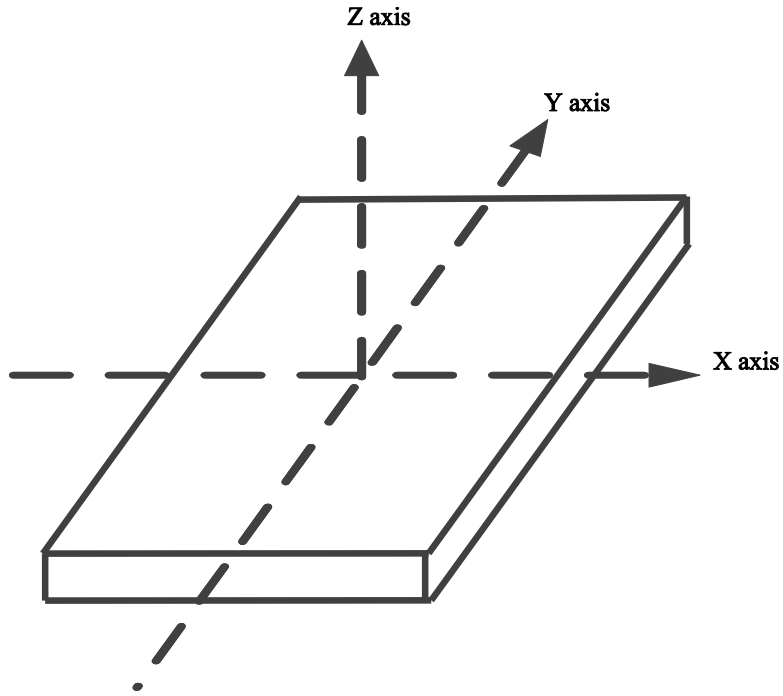


Figure 5-9: Bearing pad.

$$K_{compression} = \frac{E_c A}{h_{rt}}$$

Equation 5-1

$$K_{shear} = \frac{GA}{h_{rt}}$$

Equation 5-2

$$K_{\theta,x} = \frac{0.8E_c I_y}{h_{rt}}$$

Equation 5-3

$$K_{\theta,y} = \frac{0.8E_c I_x}{h_{rt}}$$

Equation 5-4

$$K_{\theta,z} = \frac{GI_p}{h_{rt}}$$

Equation 5-5

$$S = \frac{L \times W}{2h_{ri}(L + W)}$$

Equation 5-6

where,

$K_{compression}$	Compressive stiffness of bearing pad along z-axis.
K_{shear}	Shear stiffness of bearing pad along x and y axis.
$K_{\theta,x}, K_{\theta,y}$	Rotational stiffness of bearing pad along x and y axis
$K_{\theta,z}$	Torsional stiffness of bearing pad along z-axis
G	Shear modulus of the elastomer (ksi)
E_c	Effective modulus of elastomeric bearing in compression, $E_c = 4.8GS^2$ (ksi)
S	Shape factor of an individual elastomer layer
L	Plan dimension of the bearing perpendicular to the axis of rotation under consideration (in.)
W	Plan dimension of the bearing parallel to the axis of rotation under consideration (in.)
h_{rt}	Total elastomer thickness (in.)
h_{ri}	Individual elastomer thickness (in.)
I_x	Moment of inertial along x-axis.
I_y	Moment of inertial along y-axis.
I_p	Polar moment of inertial along z-axis

Table 5-2: Bearing pad elastic stiffness values.

	Stiffness values
$K_{compression}$	23,678.6 kip/in
K_{shear}	12.91 kip/in
$K_{\theta,x}$	764.03×10^3 kip-in/rads
$K_{\theta,y}$	191×10^3 kip-in/rads
$K_{\theta,z}$	650.72 kip-in/rads

The frequencies from the undamaged model of the twin box-girder bridge are used to determine Rayleigh damping and mass proportional damping coefficients. The first ten frequencies of the

undamaged twin box-girder bridge are tabulated in Table 5-3. In ABAQUS (2013), damping is specified as part of the material definition using Rayleigh damping coefficients. Rayleigh damping coefficients are determined using the first and the fifth frequency while the mass proportional damping coefficient is determined using the first frequency (Table 5-4). In this study, analysis is carried out using 1% critical damping.

Table 5-3: Twin box-girder bridge frequencies.

Eigen values	Frequency (Hz)
1	1.537
2	2.727
3	7.143
4	9.732
5	12.325
6	14.560
7	16.652
8	16.968
9	22.412
10	22.877

Table 5-4: Damping coefficients.

Critical damping (ξ)	Rayleigh damping		Mass proportional
	Alpha	Beta	Alpha
1%	1.718E-01	2.000E-04	1.932E-01
2%	3.435E-01	5.000E-04	3.864E-01
5%	8.588E-01	1.100E-03	9.660E-01

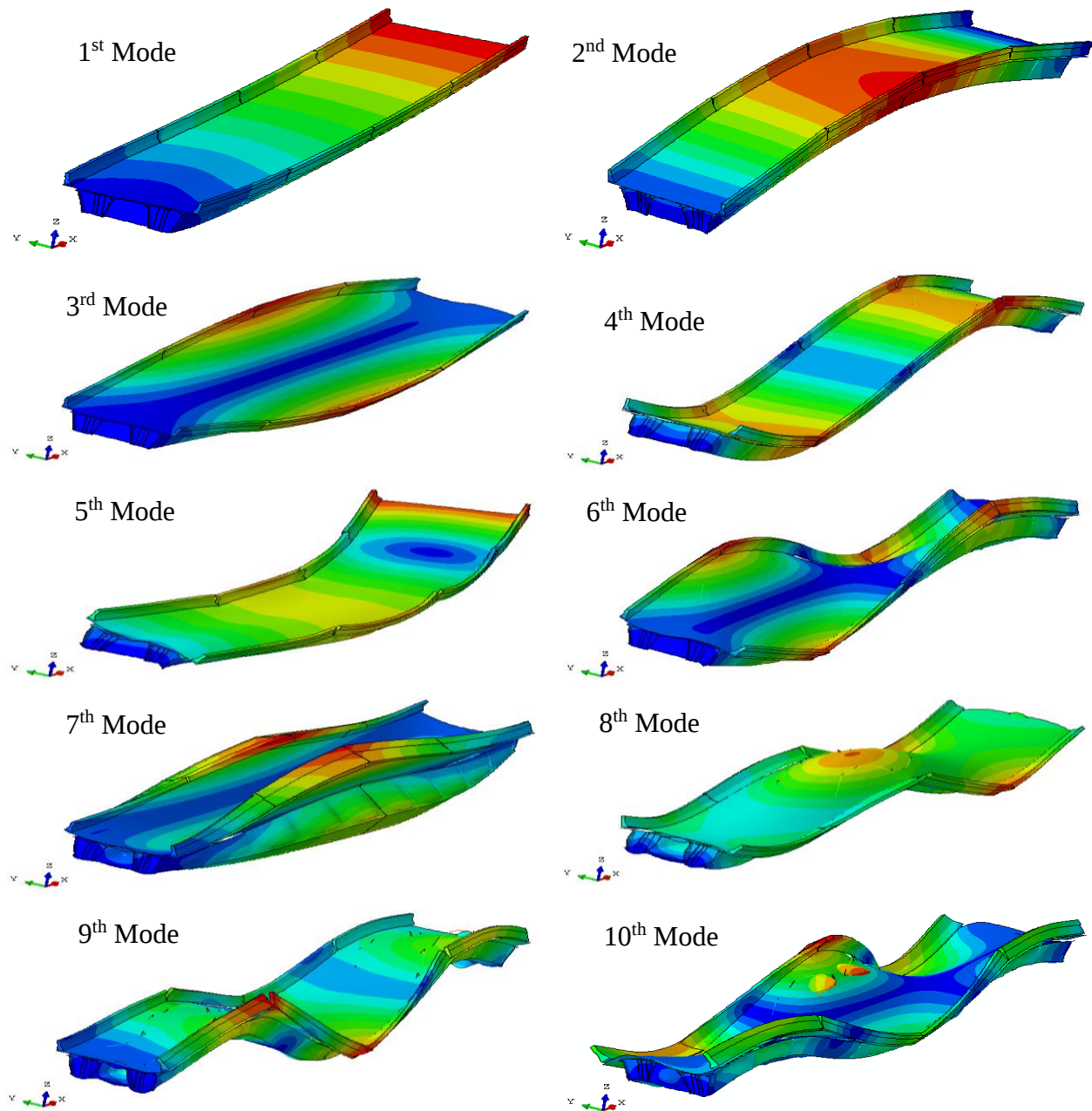


Figure 5-10: Mode shapes (Undamaged composite bridge model)

5.4 Test simulation procedure

Figure 5-11 through Figure 5-13 describes the series of steps used in ABAQUS (2013) to simulate the three tests performed on the twin box-girders tested at University of Texas at Austin. Compared to *ABAQUS/Standard*, *ABAQUS/Explicit* offers computationally efficient solution for large models with material and geometric nonlinearities; and in *ABAQUS/Explicit* analysis cost increases linearly with problem size. Therefore, after the initial steps that involve little to no nonlinearity, results are transferred to *ABAQUS/Explicit* to perform the rest of the simulation for all three tests. Girder camber is taken as the baseline for the deflection measurement for all three tests. Girder deflections are measured by taking the girder camber as zero deflection.

The Test 1 simulation (Figure 5-11) starts with the construction sequence steps followed by the application of Test 1 load (76 kips) in the form of five concrete girders in *ABAQUS/Standard*. Results from the *ABAQUS/Standard* are imported in to the *ABAQUS/Explicit* for the last phase of the analysis. In the last stage, bottom flange fracture is introduced in the exterior girder and the post fracture response is simulated for 5 sec.

The Test 2 simulation (Figure 5-12) starts with the construction sequence steps followed by the application of Test 1 load in *ABAQUS/Standard*. The field observation following Test 1 fracture showed minimal change in deflection in the fractured exterior girder. Therefore, Test 1 fracture and post fracture response following Test 1 is introduced quasi-statically in *ABAQUS/Standard* during Test 2 simulation. In the field, Test 1 load is removed followed by the application of scissor jack and jacking up the bridge by 0.25 in. at the scissor jack location (Barnard, et al., 2010). Test 2 fracture (web fracture up to 10 in. below the weld to the top flange) is introduced followed by application of Test 2 load (76 kips) in *ABAQUS/Standard*. Then results from *ABAQUS/Standard* are imported in to *ABAQUS/Explicit* for the last phase of analysis. The scissor jack boundary condition is removed and the post fracture response is simulated for 5 sec.

The Test 3 simulation (Figure 5-13) starts with the construction sequence steps followed by the application of Test 1 load in *ABAQUS/Standard*. Similar to the Test 2 simulation, Test 1 fracture and post fracture response is introduced quasi-statically in *ABAQUS/Standard*. The Test 1 load is then removed followed by application of scissor jack and jacking up the bridge by 0.25 in. at mid-span. In Test 2, web fracture up to 10 in. below the weld to the top flange was introduced by

cutting through the web. During rearrangement of concrete girders to form the bin for the road base in Test 3, the fracture on the outside web of the exterior girder propagated when total load on the bridge was 40.9 kips. The additional fracture propagated in the early stages of Test 3. Therefore, it is decided to include this additional fracture in Test 3 simulation. However, *ABAQUS/Explicit* doesn't support connector element removal during steps. Attempts to use connector damage definition to fail the connector elements connecting the girder web elements over the length of the propagated fracture were unsuccessful as the connector elements with the damage definition failed before simulation steps for Test 3. Therefore, the Test 3 fracture is introduced at the start of the Test 2 simulation steps. It is possible that this application of the additional web fracture at the start of Test 2 can affect the Test 3 response. Its impact on the Test 3 load-displacement response is evaluated in Section 5.7.2. Following introduction of Test 3 fracture, the Test 2 load (76 kips) is applied in *ABAQUS/Standard*. Then results from *ABAQUS/Standard* are imported in to *ABAQUS/Explicit* for the last phase of the analysis. The scissor jack boundary condition is removed and the post fracture response is simulated for 2 sec. Viscous pressure is applied to bring the structure to a quasi-static state. In the next step, the Test 2 load is removed slowly. To start the simulation of Test 3, the weight of six concrete girders (82.1 kips) that form the concrete bin is applied gradually to produce a quasi-static response. In the following two steps, the weight of the road base inside the bin (258.33 kips) and outside the bin (22.9 kips) is applied gradually. Section 5.5.4 provides further details about the simulation of quasi-static response in *ABAQUS/Explicit* using viscous pressure and adjusting duration of load application. Although the loading sequence used to simulate Tests 1 through Test 3 does not correspond 100% to reality, the modelling compromises made here are reasonable. Section 5.5 and other later sections discuss some of the implications of these modelling assumptions.

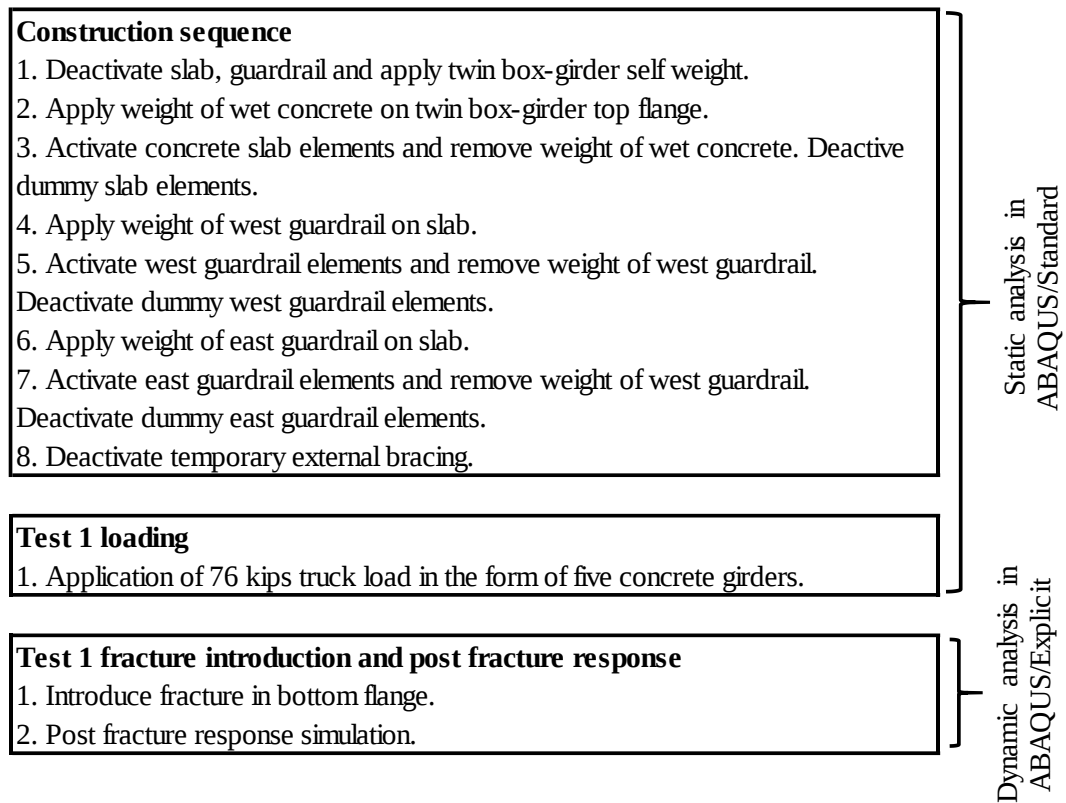


Figure 5-11: Test 1 simulation steps.

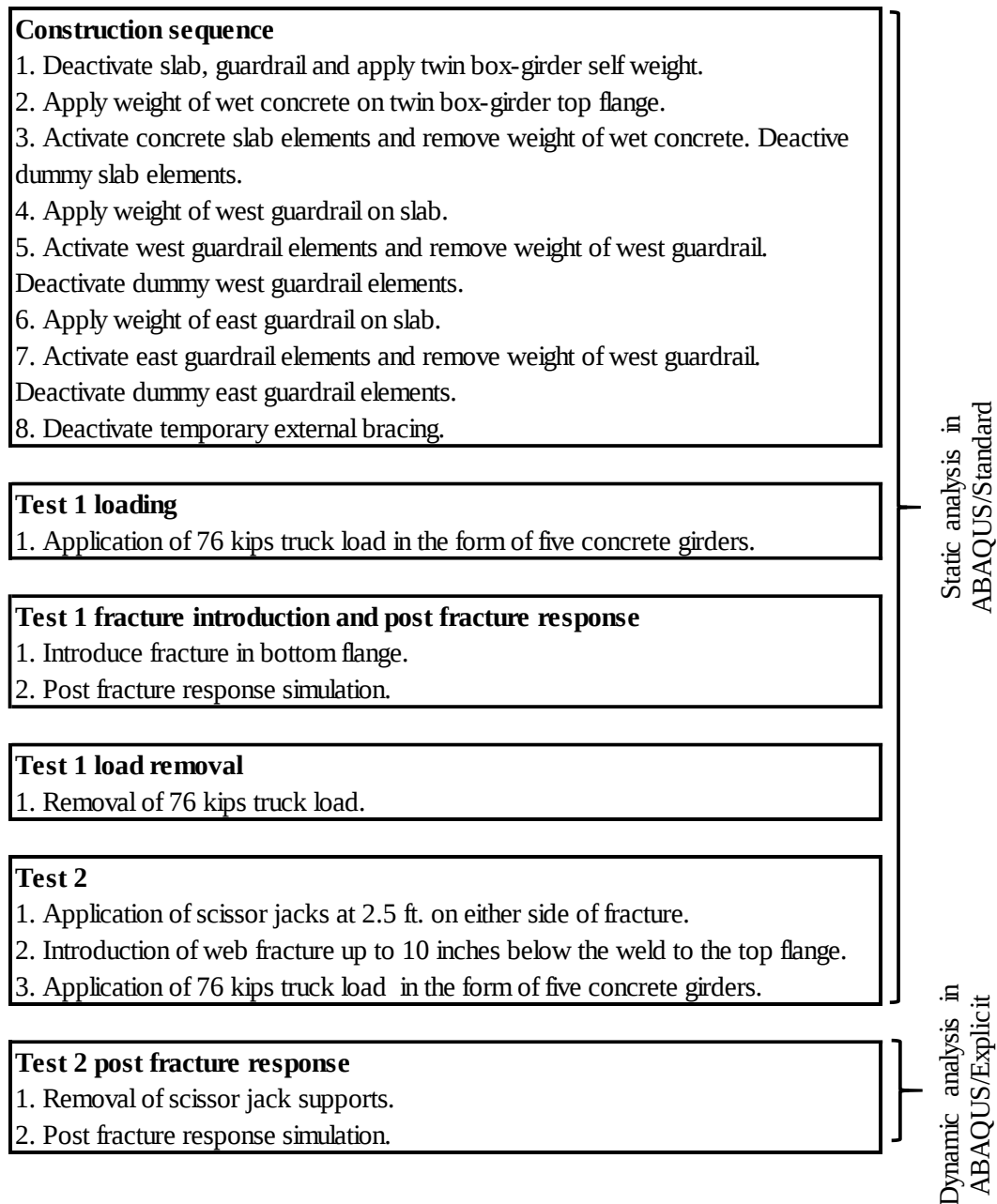


Figure 5-12: Test 2 simulation steps.

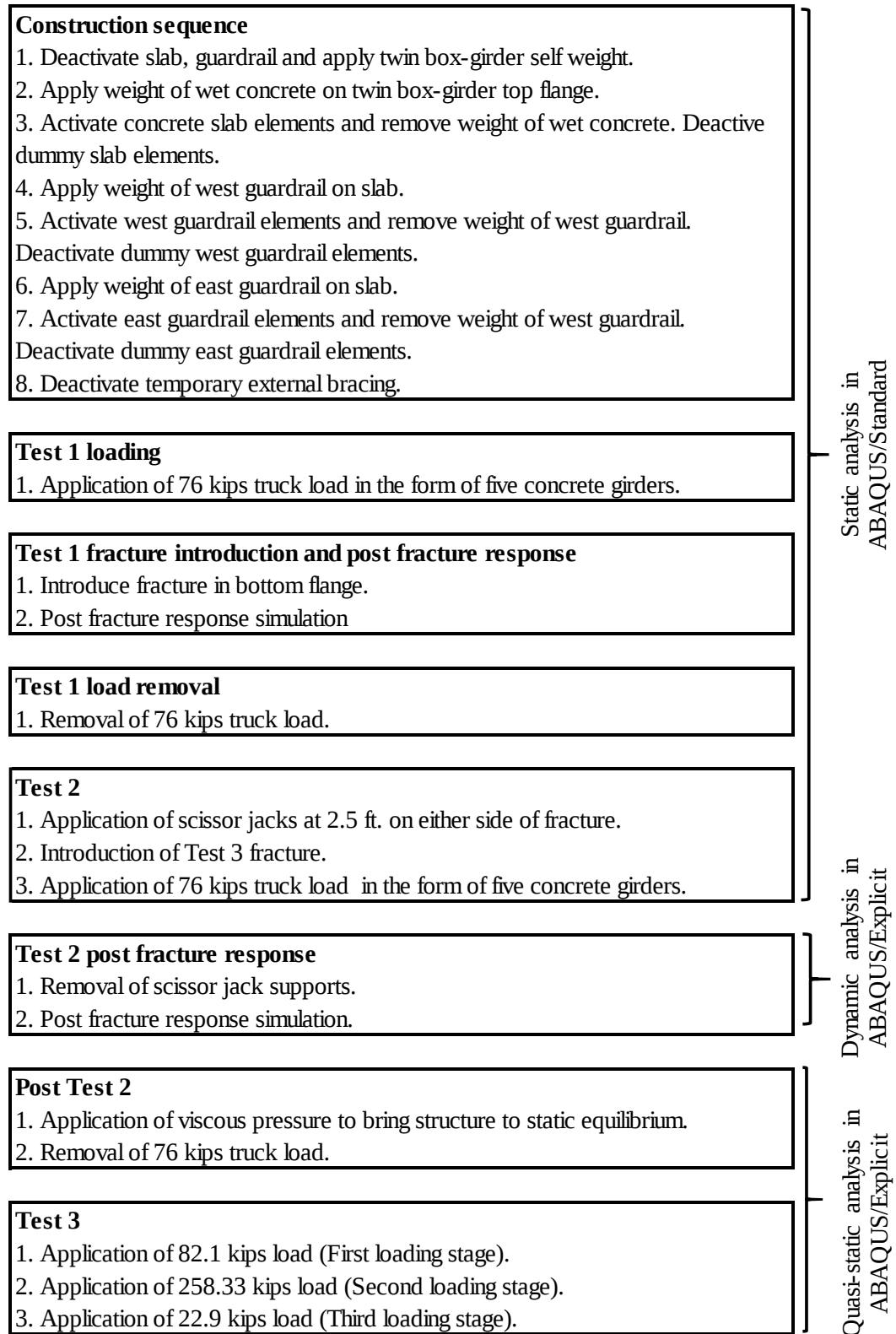


Figure 5-13: Test 3 simulation steps.

5.5 Modeling caveats

The following section discusses the modeling caveats that the reader should consider as he/she assesses the results of the post fracture evaluation of the twin box-girder bridge.

5.5.1 Construction sequence simulation using “Model Change” and dummy elements

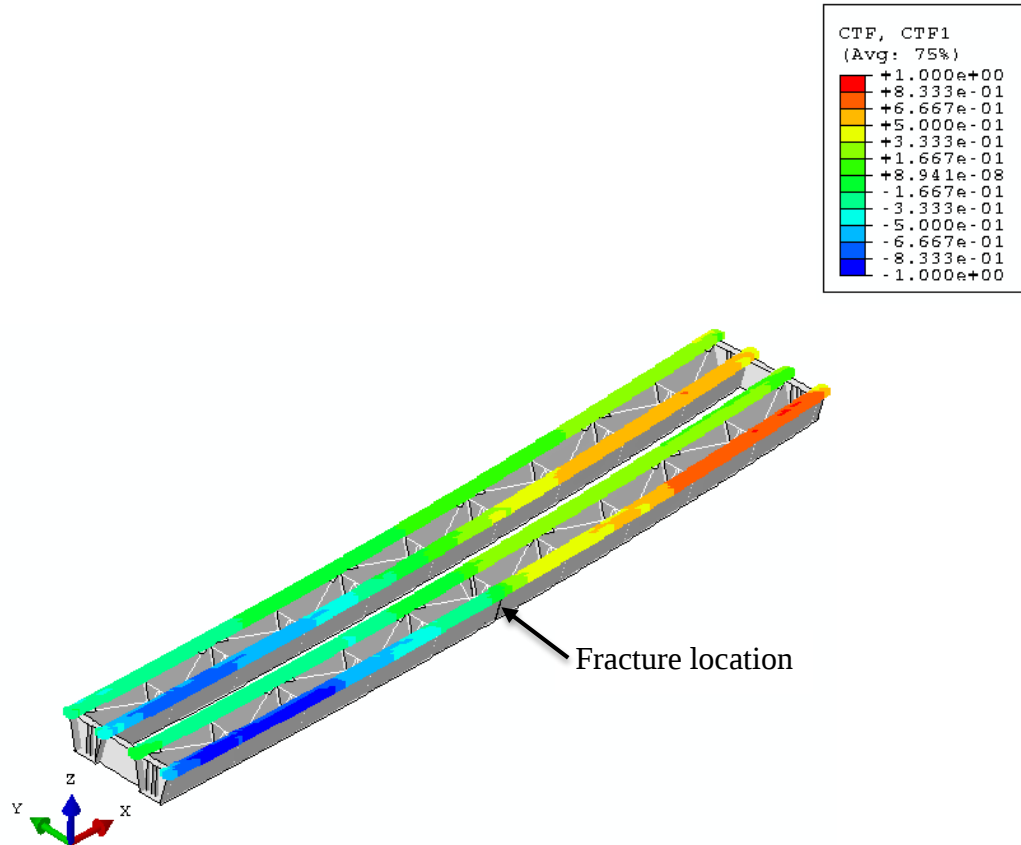
Modeling of the construction sequence is necessary to accumulate stress histories in the bridge superstructure. The construction sequence is simulated using the “Model Change” option in ABAQUS (2013) with dummy elements. Dummy elements are required to shift the nodes on the concrete slab and the guardrail compatibly with the twin box-girders. The following paragraphs describe the steps involved in construction sequence simulation and role played by dummy elements.

In the first step, all concrete components are deactivated using the “Model Change” option to simulate non-composite behavior. The weight of wet concrete slab is then applied to the box-girder top flanges as the box-girder resists slab weight before concrete hardens. The deactivated elements maintain their initial node location when they are deactivated (ABAQUS, 2013). The concrete slab nodes need to deflect compatibly with the deformed shape of the twin box-girders. This is achieved through use of dummy elements. The dummy slab occupies the same location as the concrete slab and dummy slab elements are tied together with the concrete slab elements using tie constraints. A low modulus of elasticity (10^{-10} x concrete slab modulus) and low material density (10^{-12} x concrete slab density) are assigned to the dummy slab elements in order to avoid any impact on the bridge stiffness and weight. Dummy elements are connected to the top flanges of the twin box-girders using beam connector elements. Therefore, as the twin box-girder deflects dummy slab elements deflect compatibly with the deformed shape of twin box-girders. As dummy slab elements are tied to the concrete slab elements, nodes in the concrete slab also deflect compatibly. In the next step, concrete slab elements are activated and dummy slab elements are deactivated using the “Model Change” option. The concrete slab is activated using strain-free reactivation which ensures zero strains and stresses in the concrete slab. In a similar manner, the construction sequence of the concrete guardrails is simulated using dummy guardrail parts and “Model Change” option. First the west or interior guardrail is constructed followed by construction of the east or exterior guardrail. During the guardrail construction steps, the weight of the guardrail is applied as a uniformly distributed pressure on

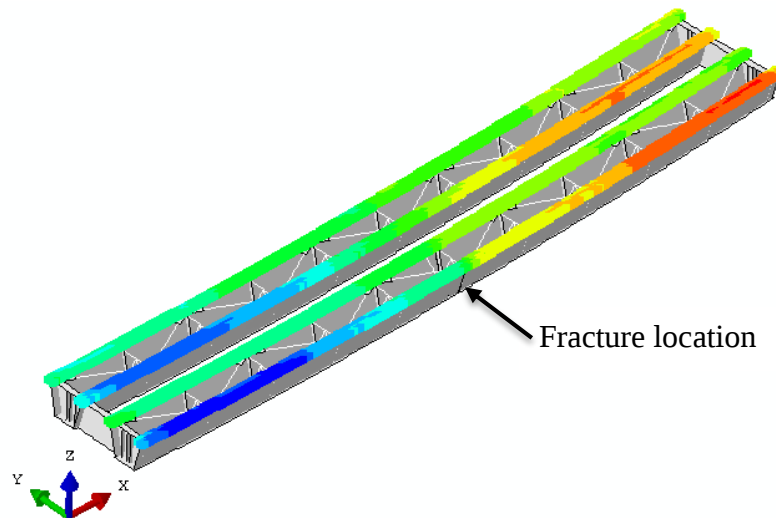
the area of the slab occupied by the guardrail. The other alternative is to develop a model for each construction step and incorporate changes in geometry and stresses in the subsequent construction step using initial conditions in ABAQUS (2013). This procedure is very laborious compared to the approach discussed above.

5.5.2 Transferring results from ABAQUS/Standard to ABAQUS/Explicit with zero velocity boundary condition

ABAQUS (2013) provides the capability to import a deformed mesh and its associated material state from ABAQUS/Standard into ABAQUS/Explicit and vice versa. However, ABAQUS allows users to import either the connector element configuration or the associated behavior definition but not both. In order to capture shear stud failure, it is necessary develop connector forces in ABAQUS/Explicit similar to the forces at the end of analysis in ABAQUS/Standard. In this study, connector elements are not imported to ABAQUS/Explicit analysis. The connector elements are redefined as part of model definition and thus are unstressed. In order to gradually develop connector forces in ABAQUS/Explicit, zero velocity boundary conditions which correspond to no change in displacement are applied to the entire bridge model. In the subsequent step, the zero velocity boundary conditions are removed and actual boundary conditions are applied. Absence of the zero velocity boundary condition in the first step of the analysis in ABAQUS/Explicit results in a rapid development of forces in the connector elements (representing shear studs) leading to their failure. Figure 5-14 shows the shear forces along x-axis (CTF1) in connector elements both at the end of ABAQUS/Standard analysis and at the end of removal of zero velocity boundary condition and application of actual boundary condition in ABAQUS/Explicit. Looking at the overall force distribution and shear forces along x-axis (CTF1), the use of zero velocity boundary condition allows for successfully transfer of connector element configuration and state to ABAQUS/Explicit.



(a) CTF1 (ABAQUS/Standard analysis)



(b) CTF1 (ABAQUS/Explicit analysis)

Figure 5-14: Shear forces in connector elements (CTF1).

5.5.3 Viscous pressure to damp-out oscillations

Section 4.3.3 of the Hoan Bridge analysis chapter discussed simulation of quasi-static response by speeding up the simulation in *ABAQUS/Explicit*. *ABAQUS/Explicit* offers viscous pressure loading that can be used to damp out dynamic effects and reach a quasi-static solution in a shorter time. The viscous pressure load (p) is defined by Equation 5-9.

$$p = -c_v (v - v_{ref}) \cdot n \quad \text{Equation 5-7}$$

$$c_v = \rho \cdot c_d \quad \text{Equation 5-8}$$

$$c_d = \sqrt{\frac{\lambda + 2\mu}{\rho}} \quad \text{Equation 5-9}$$

where,

c_v	Viscosity
v	Velocity of the points on the surface where pressure being applied
v_{ref}	Velocity of the reference node
n	Unit outward normal to the element at that point
ρ	Density of the material at the surface
λ and μ	Lame's constants

It is not desirable to absorb all the energy for typical structural problems. Therefore, ABAQUS (2013) recommends c_v to be set equal to a small percentage perhaps 1 or 2 percent of $\rho \cdot c_d$. In order to damp out dynamic effects following release of the scissor jack in Test 2, three values are evaluated for c_v as a fraction of the ρc_d value. In this study, the dynamic response following release of scissor jack in Test 2 is simulated only for 2 sec before the application of viscous pressure to keep the run time reasonable. Stud behavior is modeled using the *Stud Stiffness Only model* discussed in detail in Section 5.7.4. Use of 1.00% of ρc_d value resulted in immediate damping out of the oscillation (Figure 5-15). Use of the 0.10% of ρc_d value resulted in gradual damping out of the oscillations allowing mid-span deflection in the fractured girder to approximately reach the mean dynamic deflection (Figure 5-15). Use of the 0.50% of ρc_d value

resulted in the same final deflection but a slower convergence to the mean dynamic deflection. Therefore, 0.10% of ρc_d value will be used for C_v in the future analyses.

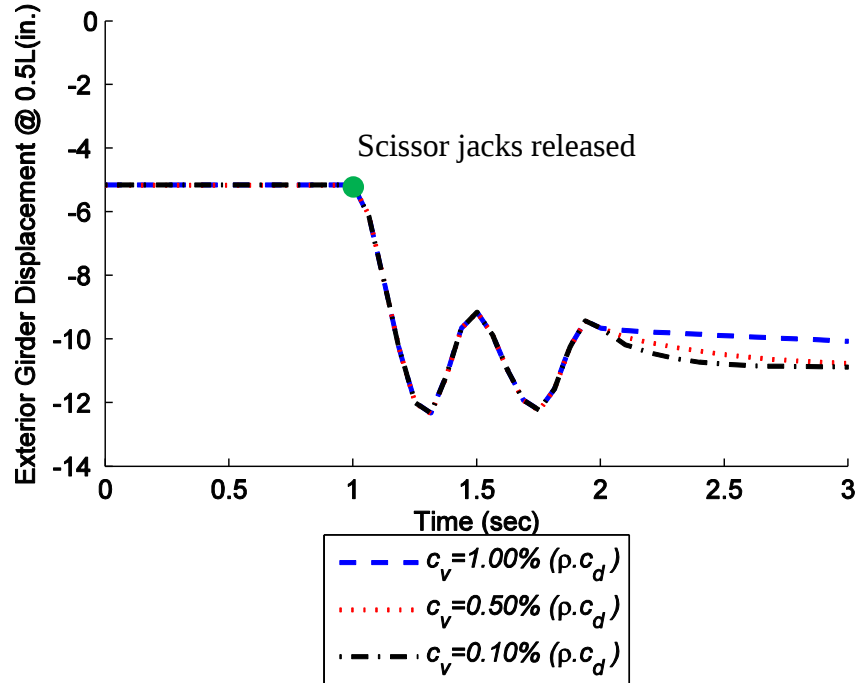


Figure 5-15: Viscous pressure to damp-out oscillations.

5.5.4 Viscous pressure and analysis duration for quasi-static analysis

The viscous pressure load discussed in Section 5.5.3 can also be used to reach a quasi-static solution in shorter time in *ABAQUS/Explicit*. However, the value of C_v needs to be carefully selected to prevent overdamping of the response. The viscous pressure value is proportional to the velocity (Equation 5-7). Therefore, the value of viscous pressure can be reduced by increasing the duration of the quasi-static analysis, which also reduces the velocity. To select C_v and the analysis duration required for a quasi-static simulation, linear elastic analyses of a simply supported composite plate girder under self-weight is carried out. The composite plate girder dimensions are tabulated in Table 5-5. Concrete slab is made from 4000 psi concrete and plate girder is made from ASTM A572 Grade 50 steel.

The results from the quasi-static analyses in *ABAQUS/Explicit* are compared against static response in *ABAQUS/Standard*. Different combinations of C_v and analysis duration are investigated to generate quasi-static response (Table 5-6). For 1 sec analysis duration, viscous pressure with C_v equal to 1.00% (ρC_d) and 0.10% (ρC_d) failed to capture the mid-span deflection and the work done by the self-weight. Increasing duration of analysis to 3 sec to reduce the effect of viscous pressure by reducing the velocity values in the model resulted in better comparison with the static analysis solution. From these results, it is clear that there is no easy way to determine the required viscous pressure and analysis duration other than actually running the analysis with combination of viscous pressure and analysis duration and evaluating the response.

Table 5-5: Composite girder dimensions.

Components	Dimensions
Slab	90"x8.625"
Top flange	16"x1"
Web	54"x0.375"
Bottom flange	16"x1"

Table 5-6: Quasi-static analysis model description.

Analysis title	Duration (sec)	C_v (ksi-sec/in)
T_1sec_1.00%(ρC_d)	1	1.00%(ρC_d)
T_1sec_0.10%(ρC_d)	1	0.10%(ρC_d)
T_3sec_0.10%(ρC_d)	3	0.10%(ρC_d)

Table 5-7: Mid-span deflection and work done.

Analysis	Mid-span deflection (in.)	Work done (kip-in)
Static analysis	0.99	30
T_1sec_1.00%(ρc_d)	0.42	5.45
T_1sec_0.10%(ρc_d)	0.92	25.56
T_3sec_0.10%(ρc_d)	0.97	28.65

To further evaluate the impact of viscous pressure and duration of the quasi-static analysis, the Test 3 from the twin box-girder tests is simulated using combination of viscous pressure and duration of the analysis. The analysis is carried out using the series of steps outlined in Figure 5-13. The finite element model is developed using the modeling methodology discussed in Section 5.3. The shear-tension interaction of shear studs is modeled using the PCI interaction equation. The initial stud stiffness in shear is calculated from the force vs. slip curve proposed by Topkaya et al. (2004). The initial axial stud stiffness is calculated based on the shear stud and the cone of concrete formed around the stud during failure. The dimensions of the concrete cone are calculated using Appendix D in ACI 318-11 (2011). Three combinations of the C_v value and analysis duration are investigated to generate quasi-static response (Table 5-8). To damp out dynamic effects following Test 2, 0.10% of ρc_d is used as the C_v value. During Test 3 loading was done in three stages. These stages are further divided for analysis purposes and are tabulated in Table 5-9. The results from the simulation are compared against the load-displacement response recorded at 18 ft. from mid-span towards the south end of the bridge.

Table 5-8: Viscous pressure and analysis duration (Test 3).

Analysis	Duration (sec)	C_v (ksi-sec/in)
T_14sec_0.10%(ρc_d)	14	0.10%(ρc_d)
T_14sec_0.01%(ρc_d)	14	0.01%(ρc_d)
T_19sec_0.01%(ρc_d)	19	0.01%(ρc_d)

Table 5-9: Loading stages and analysis duration (Test 3).

Analysis	First loading stage (82.1 kips)	Second loading stage (258.33 kips)	Third loading stage (22.29 kips)
T_14sec_0.10%(ρc_d)	2 sec (82.1 kips)	5 sec (258.33 kips)	2 sec (22.29 kips)
T_14sec_0.01%(ρc_d)	2 sec (82.1 kips)	5 sec (258.33 kips)	2 sec (22.29 kips)
T_19sec_0.01%(ρc_d)	3 sec (20.525 kips) and 3 sec (61.575 kips)	5 sec (158.34 kips) and 5 sec (99.96 kips)	3 sec (22.29 kips)

All three analyses failed to reach the bridge capacity observed in the field and displayed stiffer response initially compared to the field response (Figure 5-16). Failure occurred because of large strains in the intermediate diaphragm at mid-span of the intact girder. The strains at the beam (cross bracing) to the shell (intermediate stiffener) connection increased very rapidly resulting in the formation of a shock wave and analysis failure. This local failure is believed to be numerical and is further evaluated in Section 5.5.5. Reducing the value of C_v from 0.10% (T_14sec_0.10%(ρc_d) model) to 0.01% (T_14sec_0.01%(ρc_d) model) of ρc_d value resulted in softer response beyond 150 kips. Use of 0.10% of the ρc_d value for C_v value resulted in excess damping out of kinetic energy at the surface leading to stiffer load vs. displacement response compared to the experimental response. To further reduce the impact of viscous pressure on the load vs. displacement response of the bridge, duration of analysis is further extended for each stage of Test 3 loading as shown in Table 5-9. The T_19sec_0.01%(ρc_d) model with increased analysis duration yielded identical load-displacement response compared to the T_14sec_0.01%(ρc_d) model. Therefore, the T_14sec_0.01%(ρc_d) model or the T_19sec_0.01%(ρc_d) model can be used for Test 3 simulation in *ABAQUS/Explicit*. In the rest of analysis performed in this chapter, T_19sec_0.01%(ρc_d) model is used for simulation of Test 3 response.

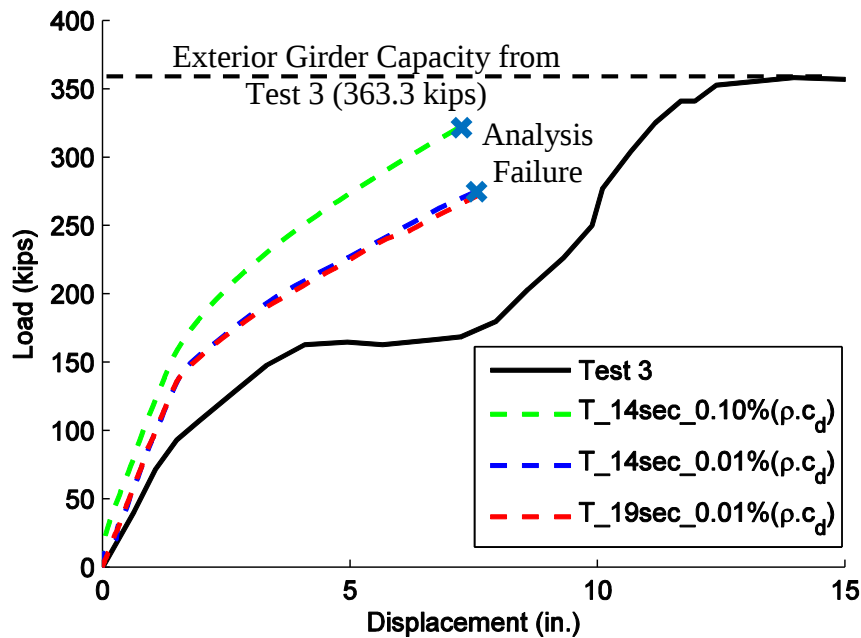


Figure 5-16: Load vs. displacement response of exterior girder (Test 3).

5.5.5 Dissipation of spurious oscillations using linear bulk viscosity

The local failure mode observed in Section 5.5.4 is believed to be the result of numerical instabilities and not representative of structural behavior. To reduce the observed rapid increase in strains at the beam-shell connection, additional shell elements (S3) are included at the interface to allow for a smoother transfer of forces at the connection (Figure 5-17). The additional shell elements are assigned gusset plate properties. Figure 5-18 shows the impact of the updated intermediate diaphragm models on the Test 3 response. The symbol ‘x’ in Figure 5-18 indicates analysis failure. The initial response from both FEA models is identical up to 161.5 kips; after that the updated FEA model (*FEA_updated*) shows stiffer response. The FEA model without the updates (*FEA_model*) failed because of rapid increase in deformation speed in the intermediate diaphragm beam elements when the applied load reached 272.5 kip. This phenomenon occurs because the finite element mesh lacks the frequencies to describe the wave propagation process correctly. ABAQUS (2013) offers the option of activating linear bulk viscosity to dissipate energy in the higher frequencies. It generates a bulk viscosity pressure that is proportional to the volumetric strain rate (Equation 5-10). Linear bulk viscosity introduces damping which results in a decrease in stable time increment.

$$p_{bvl} = b_1 \rho c_d L_e \dot{\epsilon}_{vol} \quad \text{Equation 5-10}$$

where,

b_1	Damping coefficient (default = 0.06)
ρ	Current material density
c_d	Current dilatational wave speed
L_e	Element characteristic length
$\dot{\epsilon}_{vol}$	Volumetric strain rate

The updated intermediate diaphragm model is reanalyzed with a damping coefficient (b_1) of 1.00 ($FEA_updated_b_1=1.00$). This allowed the analysis to continue beyond 272.5 kips and was able to complete the Test 3 loading. The total energy dissipated by viscous effects (ALLVD) showed negligible increase in the value with use of linear bulk viscosity when compared to the analysis without linear bulk viscosity ($FEA_updated$). This ensures reliability of the analysis approach. The analysis is continued by increasing the load applied outside the bin in the third loading stage of Test 3 until analysis failure. The simulation failed to reach a solution when the buckling on the brace was approached at total load of 414.3 kips. The load and deflection values from the three models in the exterior girder at the time of analysis failure are tabulated in Table 5-10.

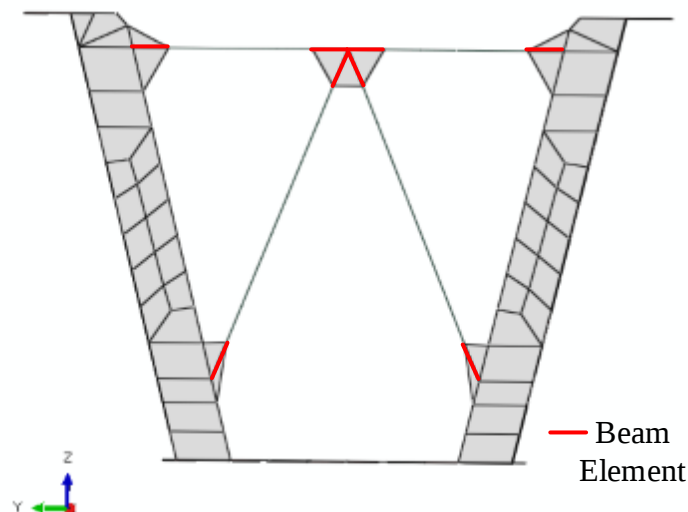


Figure 5-17: Updated finite element model of intermediate diaphragm.

Table 5-10: Peak load and displacement (Test 3).

Model description	Exterior Girder (18 ft south of mid-span)	
	Load (kips)	Displacement (in.)
Test 3	363.30	25.00
<i>FEA_model</i>	272.51	7.62
<i>FEA_updated</i>	330.40	10.06
<i>FEA_updated_b₁ = 1.00</i>	414.30	17.37

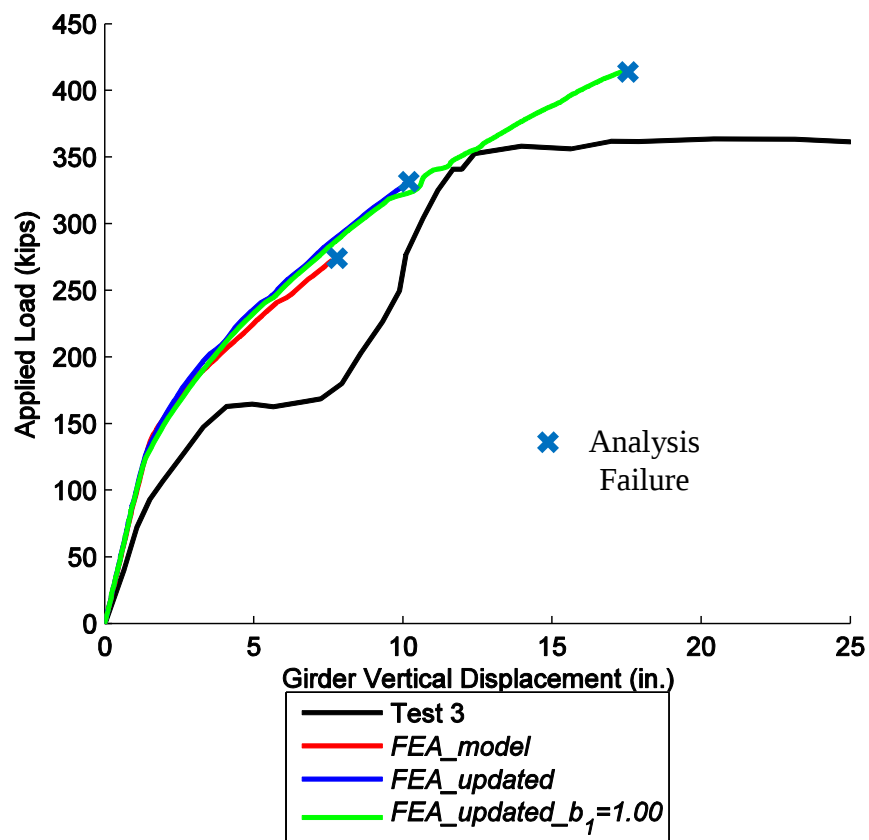


Figure 5-18: Impact of updated model on Test 3 response.

5.6 Twin box-girder FE analysis

To establish a baseline for the overall bridge behavior, the Twin box-girder test is simulated using the modeling methodology discussed in Section 5.3. The shear-tension failure envelope for the shear studs is modeled using the PCI Design Handbook, 6th Edition (2004) interaction equation. The initial shear stud stiffness (3783.9 kip/in per 3 stud group) is based on the force vs. slip relationship proposed by Topkaya et al. (2004). The initial tensile stud stiffness (47,079 kip/in per 3 stud group) is calculated assuming the stud and the surrounding concrete cone. The concrete cone dimensions are based on the concrete capacity design (CDC) method used in Appendix D of ACI-318 (2008).

5.6.1 Construction

The construction sequence simulation is carried out using the series of steps discussed in the first part of the Test 1 simulation (Figure 5-11). Figure 5-19 shows the comparison between the measured and simulated deflected shape of both girders at the end of construction. The FEA better predicted the interior girder (IG) response compared to the exterior girder (EG) response. Overall the finite element analysis matched reasonably well with the field measurements.

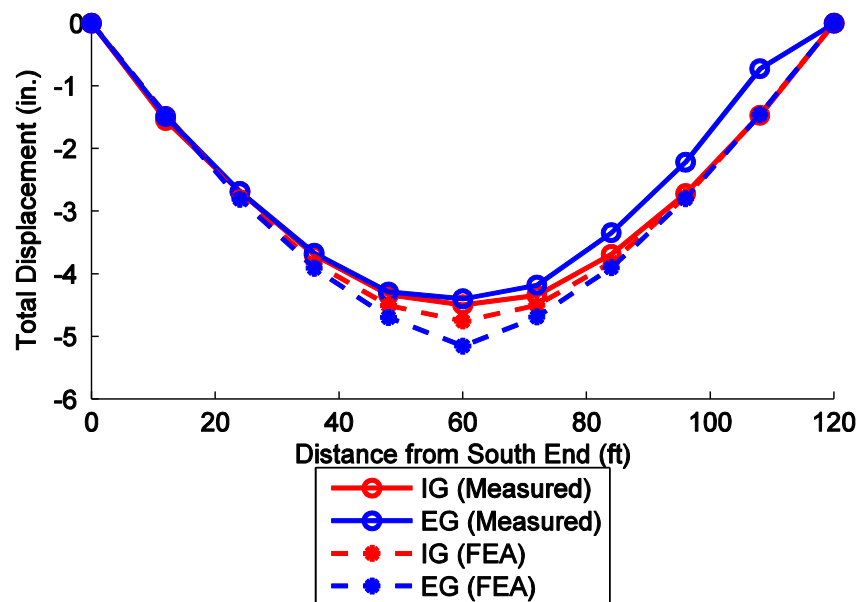


Figure 5-19: Girder deflected shape at end of construction.

5.6.2 Test 1

The Test 1 was performed to investigate the post fracture response following the bottom flange fracture using an explosive shape charge. Test 1 is simulated using series of steps defined in Figure 5-11. Field observation following the bottom flange fracture in Test 1 showed minimal change in girder deflection. Therefore, the last steps in the analysis that involve introduction of Test 1 fracture and simulation of post fracture response are also simulated using the *Implicit quasi-static analysis* in ABAQUS/Standard. Figure 5-20 shows the comparison between the field measurement and the simulated post fracture response using dynamic and quasi-static analysis. Both analyses predicted about 6% stiffer response compared to the field measurement at mid-span of the exterior girder.

Figure 5-21 shows the extent of stud damage at the end of Test 1 using connector overall damage variable (CDMG). CDMG value ranges from 0-1 with zero corresponding to intact stud and 1.0 corresponding to failed stud. Stud failure is observed at the location of fracture and damage did propagate along exterior girder flanges but it did not cause stud failure. Therefore, minimal reduction in structural stiffness occurred following Test 1 fracture which explains the minimal change in deflection observed at the end of Test 1. Neumann (2009) reported inconsistencies in the survey results for Test 1. Therefore, the values from the analysis might be closer to field measurement or they can be further apart. The deflections at the end of quasi-static analysis is less than 1% lower when compared to the mean deflection from the dynamic analysis. Figure 5-22 and Figure 5-23 show the comparison between the measured and simulated deflected shape of both the girders at the end of Test. Similar to the construction response prediction, the deflected shape of the interior girder at the end of analysis matched better compared to the deflected shape of the exterior girder. Overall the girder deflected shape from the quasi-static analysis is identical to the girder deflected shape based on the mean deflection from the dynamic analysis. Therefore, the *Implicit quasi-static analysis* is used to simulate the post fracture response in Test 1 for Test 2 and Test 3 simulation. After removal of Test 1 load during testing, bridge did not rebound to the position before application of Test 1 live load (Neuman, 2009). This indicates presence of damage in the bridge following bottom flange fracture. Surprisingly, it did not result in increased deflection following bottom flange fracture. In the finite element model, bridge rebounded to 4.72 in. after removal of Test 1 live load which is 0.1 in higher compared to the deflection before application of live load. It is interesting to notice that

frequency of the post fracture dynamic response does not correlate to the first eigenvalue. This probably because Figure 5-20 shows very local phenomenon.

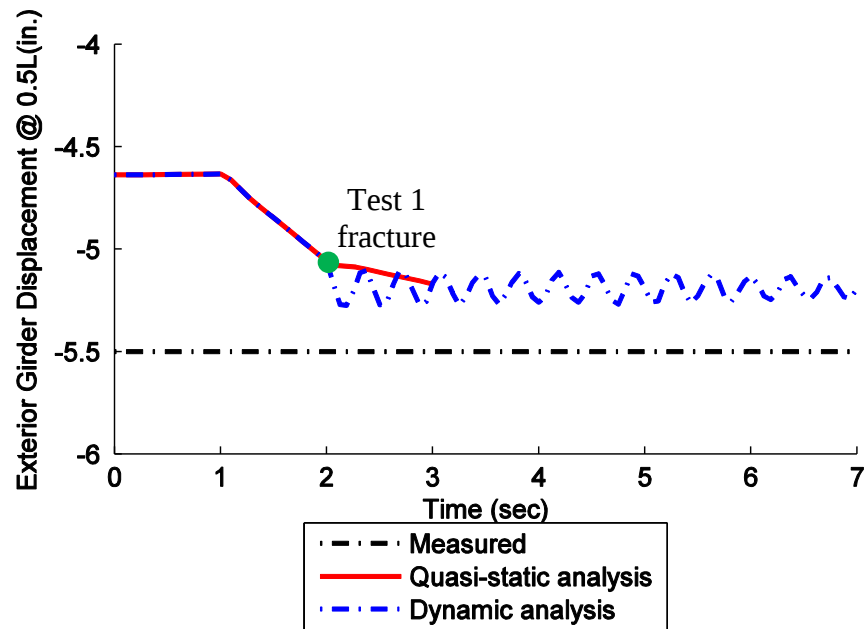


Figure 5-20: Field measurement vs. FEA (Test 1).

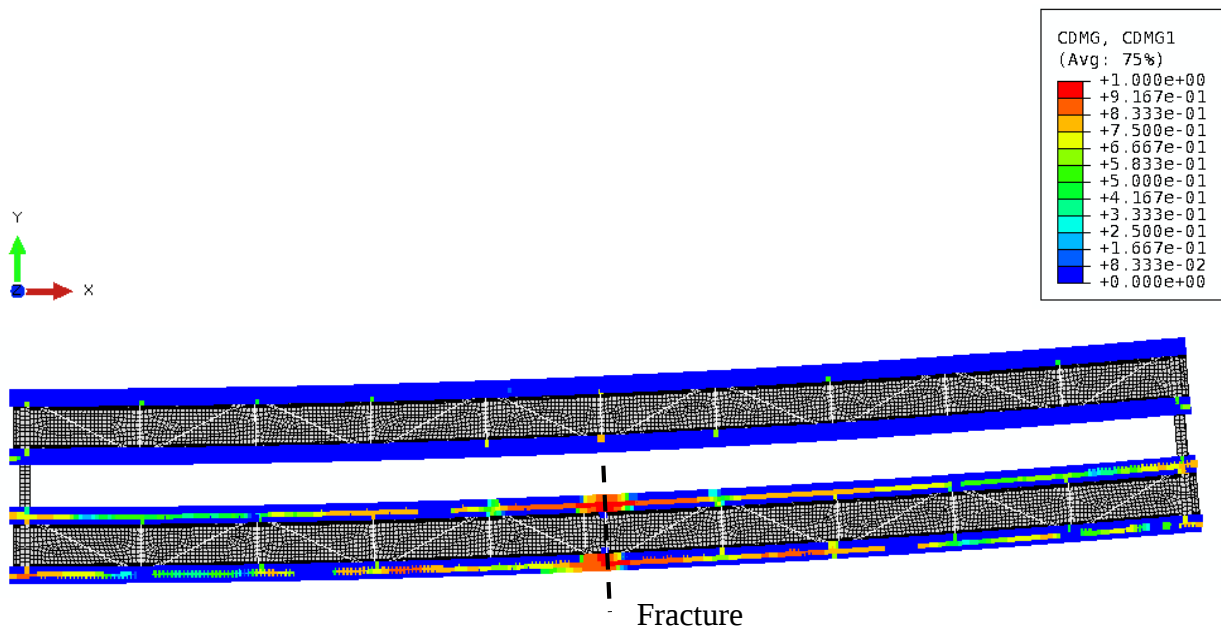


Figure 5-21: Connector overall damage variable (CDMG) at end of Test 1.

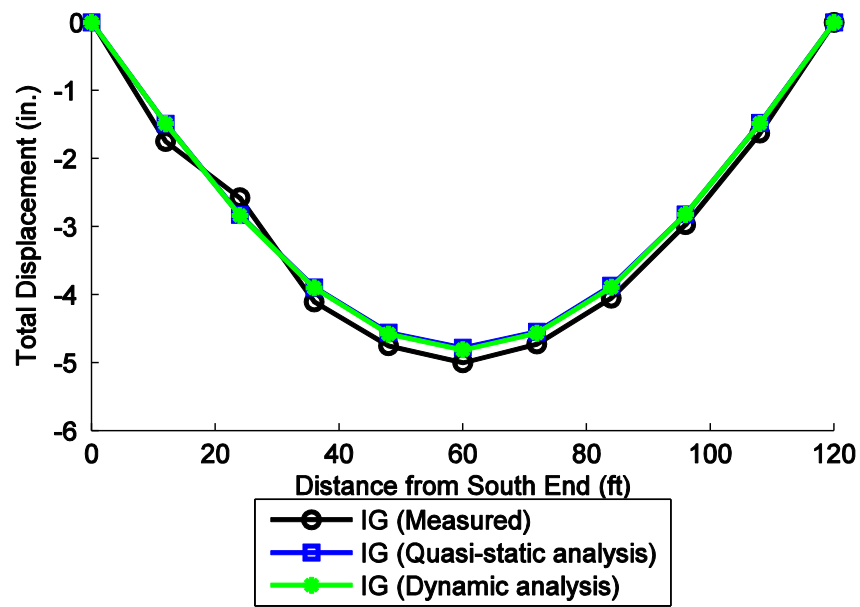


Figure 5-22: Interior girder deflected shape at the end of Test 1.

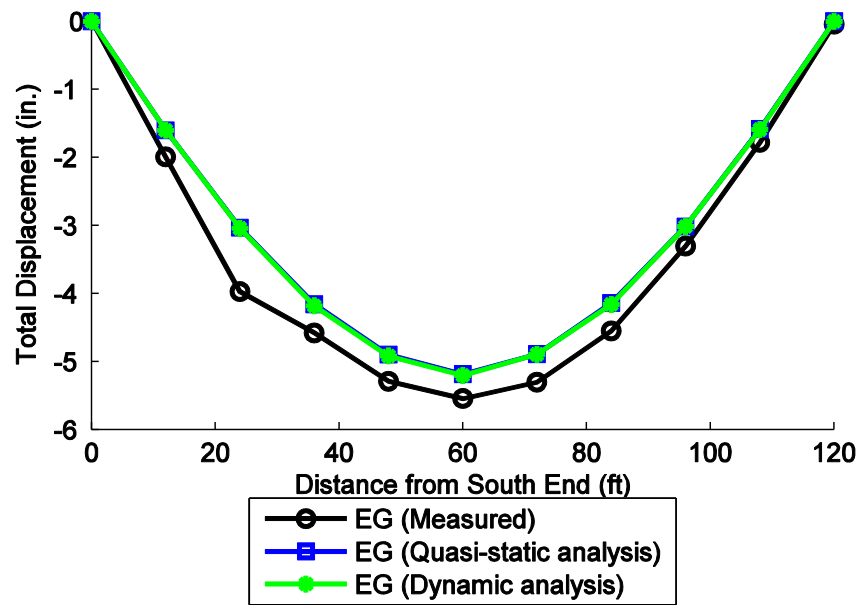


Figure 5-23: Exterior girder deflected shape at the end of Test 1.

5.6.3 Test 2

Test 2 was performed to evaluate the bridge capacity to sustain a dynamic event. The event simulated was the release of scissor jack following a nearly full-depth web fracture on the exterior girder. Test 2 is simulated using the series of steps defined in Figure 5-11. At the end of Test 1, the mid-span deflection in the exterior girder is 6% lower than the reported deflection in the field. After removal of the Test 1 load, scissor jacks were applied at 2.5 ft. on either side of the fracture location and the bridge was jacked up by 0.25 in. to hold it in the approximate position where it stood before sustaining the bottom flange fracture induced during Test 1 (Neuman, 2009). Neuman (2009) reports that deflections during Test 1 were not completely credible and it was decided to design the scissor jacks to recover 0.25 in. of vertical displacement at the mid-span of the fractured girder. The reasons for this decision remain unclear to the writer at this time. Prior to releasing the scissor jacks, the exterior girder's mid-span deflected 1.5 in. downward from the end supports (Neuman, 2009). This mid-span deflection of 1.5 in. downward from the end supports corresponds to total downward deflection of 5.1 in. when measured from the exterior girder camber.

There was approximately a 20 months gap between Test 1 and Test 2. The slab and guardrail concrete strength increased as concrete continued to cure. There is no easy way to change material properties in the middle of an analysis in ABAQUS (2013) other than switching to some form of time dependent material definition and analysis. Therefore, in the Test 2 simulation, the Test 1 response is simulated using the Test 2 concrete strengths for slab and guardrails (Table 5-1). Also, concrete shrinkage effects are not accounted for in the simulation. This resulted in stiffer response at the end of Test 1 simulation compared to the Test 1 response recorded in section 5.6.2. After removal of the Test 1 load, the mid-span deflection in the exterior girder rebounded to the total downward deflection of 4.72 in. at the mid-span of the exterior girder. In order to have the same starting position of the exterior girder as in the field, an additional deflection of 0.38 in. is imposed at the mid-span by specifying total deflection of 5.1 in. at the scissor jack location. The deformed shape of the exterior girders matches reasonably with the values measured in the field (Figure 5-23). The interior girder deflection measured in the field peaks at 48 ft. from the south end of the bridge compared to the finite element analysis that shows peak deflection at 60 ft. (Figure 5-24). Instead of imposing additional deflection at the scissor jack location, Test 2 response is simulated by applying scissor jacks without imposing

additional deflection. The displacement response and the stud damage did not match with the field observation. Therefore, Test 2 simulation is performed by imposing an additional deflection of 0.38 in.

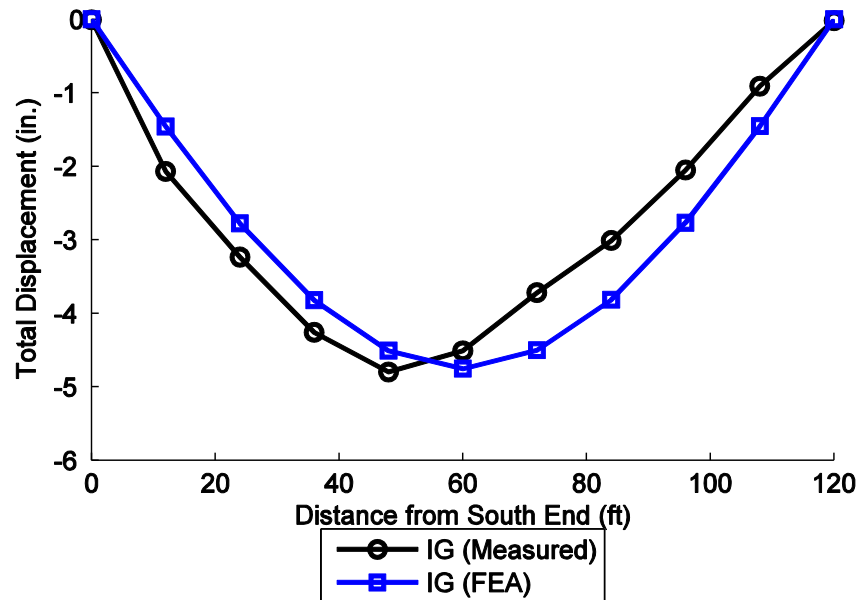


Figure 5-24: Interior girder deformed shape before scissor jack release.

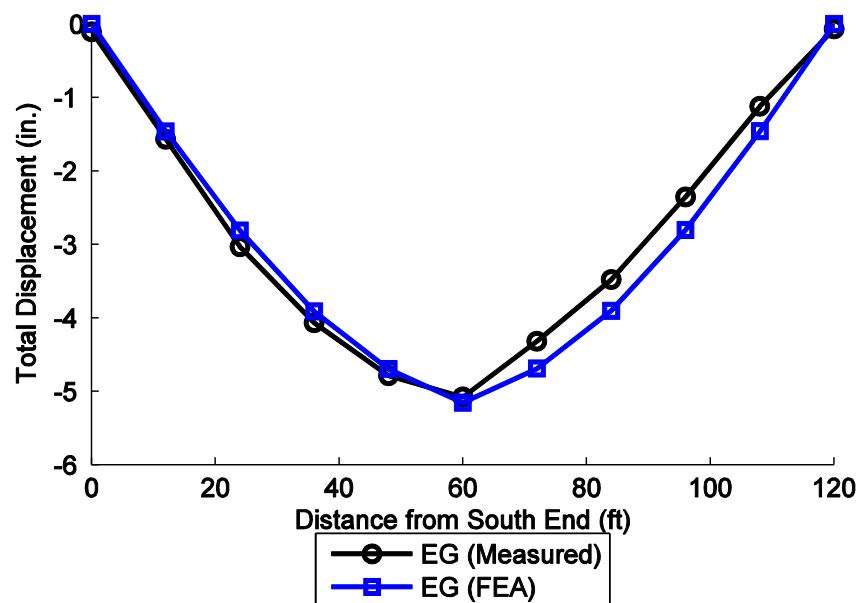


Figure 5-25: Exterior girder deformed shape before scissor jack release.

Figure 5-26 shows the post fracture response of the exterior girder at mid-span. During field test, the string potentiometer installed to measure the post fracture response at mid-span of the exterior girder was damaged from the explosion used to collapse the scissor jack. Therefore, deflection values during dynamic response following release of scissor jack are not available; instead, the deflections measured after the bridge came to rest are available. The smaller peak to peak amplitude of oscillation observed in simulation is unusual and it can possibly be attributed to the shear stud model used in this study (Figure 5-26). The impact of different stud models on post fracture bridge response is further investigated in detail in Section 5.7.3 and Section 5.7.4. The mean deflection from the dynamic analysis at mid-span is 17.4 % higher compared to the to the field observation. Temperature reading were not recorded during Test 2 (Neuman, 2009). A positive temperature gradient on a simply supported bridge would result in upward deflection reducing the downward deflection. In the absence of temperature gradient data, thermal effects are excluded from the present finite element model. It is also interesting to notice that frequency of the post fracture dynamic response does not correlate to the first eigenvalue. This probably because Figure 5-26 shows very local phenomenon.

Figure 5-27 shows the comparison between the measured deflected shape of both the girders at the end of Test 2 and the analytical response. The deflected shape of the fractured girder resembles a simply supported beam with an internal hinge at mid-span (Figure 5-27). The intact girder deflected shape resembles a simply supported beam deflected shape with the analysis predicting symmetric response about mid-span while the field measurement showed higher deformation to the left of the mid-span.

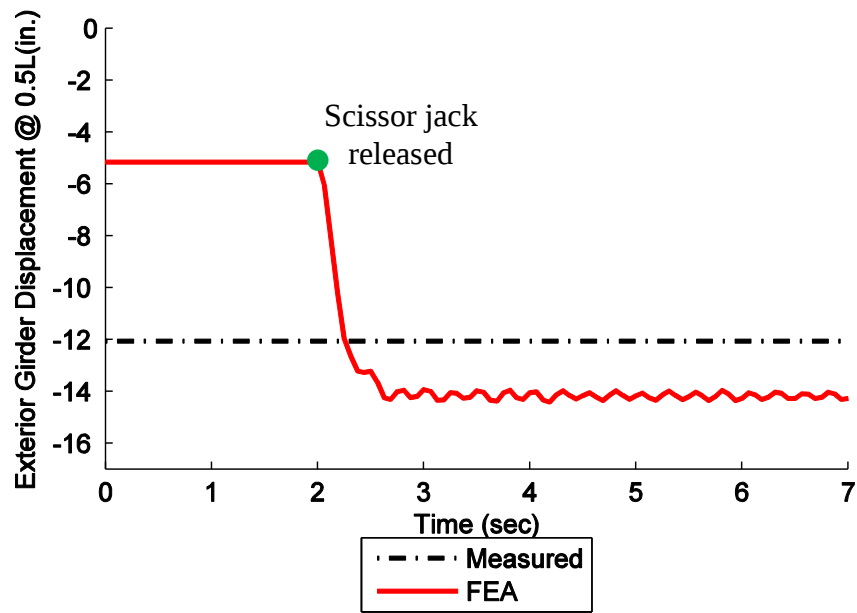


Figure 5-26: Field measurement vs. FEA (Test 2).

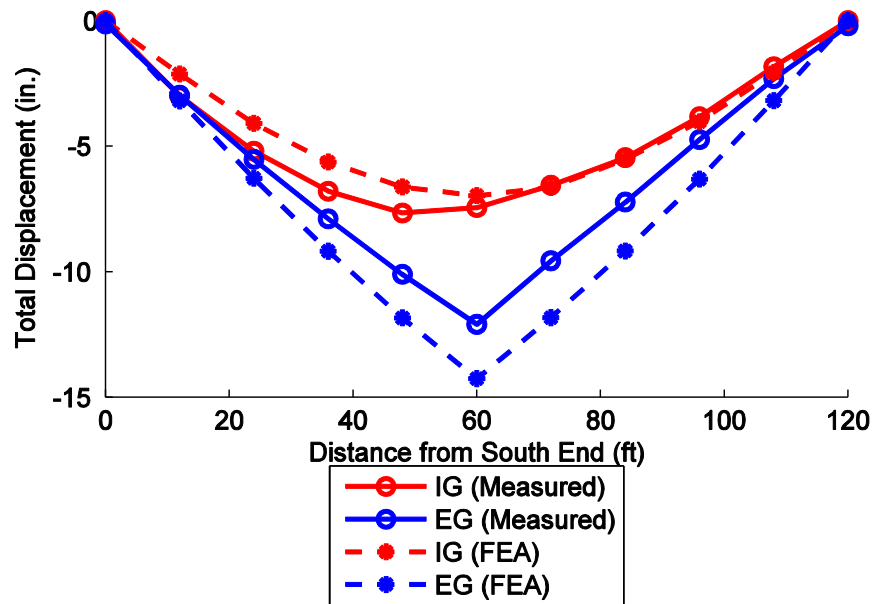


Figure 5-27: Girder deflected shape at the end of Test 2.

Figure 5-28 shows the mid-span displacement at the top of the slab across the width of the bridge at different time intervals following removal of scissor jacks. Figure 5-29 shows the evolution of stud damage using the connector overall damage variable (CDMG). CDMG value ranges from 0-1 with zero corresponding to intact stud and 1.0 corresponding to failed stud. During early stages (up to 0.06 sec) concrete slab showed slight double curvature. In the following time increments, the shear studs on the interior flange of the exterior girder failed leading to single curvature as shown by the CDMG plot in Figure 5-29-(c). Shear stud failure on the interior flange of the exterior girder overloaded the studs on the exterior flange of the interior girder leading to their failure as shown in Figure 5-29-(d).

During Test 2, unzipping failure of the shear studs under combined shear and tension results in a separation of the slab at the steel-concrete interface. Inspection of the interface at the concrete slab to the interior flange of the exterior girder connection revealed about 30 ft. of slab separation on either side of mid-span (Neuman, 2009). The finite element analysis predicted stud failure along the entire length of the interior flange of the exterior girder as shown in Figure 5-29-(e). Additional stud failure resulted in larger mean deflection as shown in Figure 5-27. Neuman (2009) does not document the stud damage and slab separation at the exterior flange of the exterior girder and both flanges of the interior girder. The finite element analysis predicted stud failure between 0.4L and 0.6L on the exterior flanges of both girders. Similar to the interior flange of the exterior girder, the interior flange of the interior girder was also predicted to suffer from stud failure along the entire bridge length.

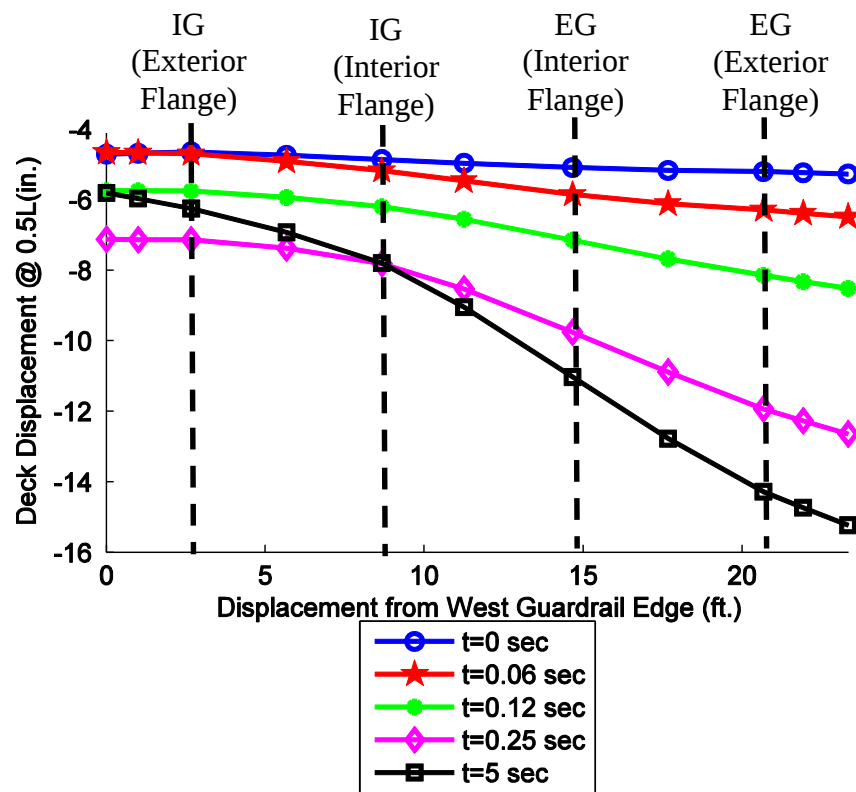
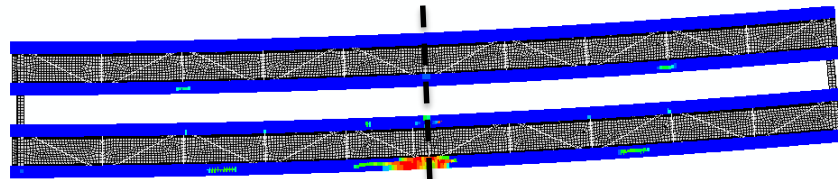
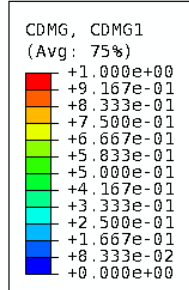
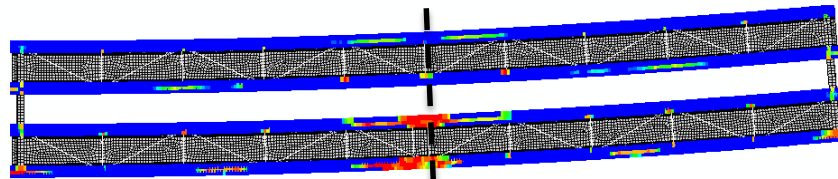


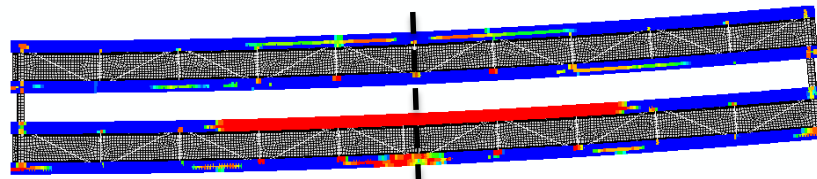
Figure 5-28: Deck profile at mid-span (in.).



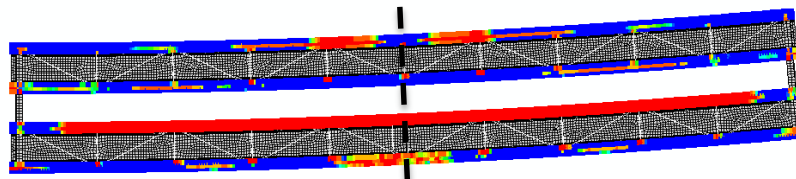
a) $T = 0 \text{ sec}$



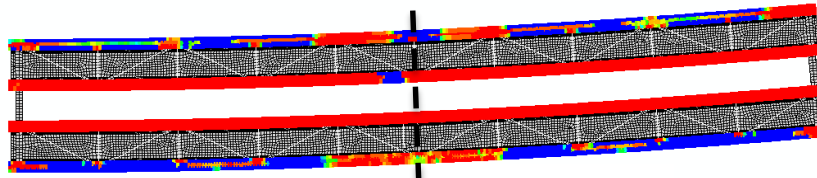
b) $T = 0.06 \text{ sec}$



c) $T = 0.12 \text{ sec}$



d) $T = 0.25 \text{ sec}$



e) $T = 5 \text{ sec}$

Fracture location

Figure 5-29: Connector overall damage variable (CDMG).

5.6.4 Test 3

Test 3 was performed to determine the reserve load capacity of the damaged bridge and observe the sequence of failure mechanisms. Test 3 is performed using series of steps defined in Figure 5-13. The Test 3 simulation requires results from the Test 2 simulation to correctly represent the testing sequence. The test 2 simulation (Section 5.6.3) predicted more stud damage, resulting in a larger exterior girder deflection response along entire length of the bridge. This alters the starting point for Test 3 simulation. The present analysis uses the analysis duration and viscous pressure values from T_19sec_0.01%(p_{cd}) model (Section 5.5.4) for Test 3 loading stages.

Figure 5-30 shows the comparison between the load vs. displacement response measured in the field to the analytical response. The symbol 'x' in Figure 5-30 indicates analysis failure. When the total load on the slab reached 161.5 kips, the exterior girder (EG) separated from the deck as a results of stud failures and displaced without additional load (Neuman, 2009). The present finite element model with shear stud model based on the initial stud stiffness and PCI Design Handbook, 6th Edition (2004) shear-tension interaction to model damage initiation of studs failed to capture the sudden separation at the interface at 161.5 kips. The shear studs are modeled by connecting all the nodes at the steel-concrete interface using connector elements with adjusted properties to have the same interface behavior as the discrete shear studs on the bridge. Distributed stud stiffness possibly resulted in gradual failure instead of rapid failure observed in the field at 161.5 kips. When the total load reached 360.2 kips, the exterior railing began to crush initiating collapse of the bridge (Neuman, 2009). The bridge failed when the total load on the bridge reached 363.3 kips. The interior girder displayed linear response throughout Test 3 (Neuman, 2009).

In the finite element analysis loading is continued by increasing the load applied outside the bin in the third loading stage of Test 3 until failure of the analysis. The finite element model failed when the total load on the bridge reached 414.3 kips. For the exterior girder, the finite element model predicted higher initial stiffness compared to the field response. The load-displacement response of both box-girders show a kink in the linear response when the total load on the bridge reached ~125 kips. The interior girder deflection in the field was 3 in. at time of collapse while the finite element model predicted a peak deflection of 7.1 in. when the total load reached 363.3 kips. When the total load on the bridge reached 414.3 kips, the exterior girder deflected by 17.4

in. while the interior girder deflected by 9 in. Large difference in response of the interior and the exterior girder deflections indicates a lack of load redistribution following the stud failures, which disconnects the slab from the exterior box-girder flanges.

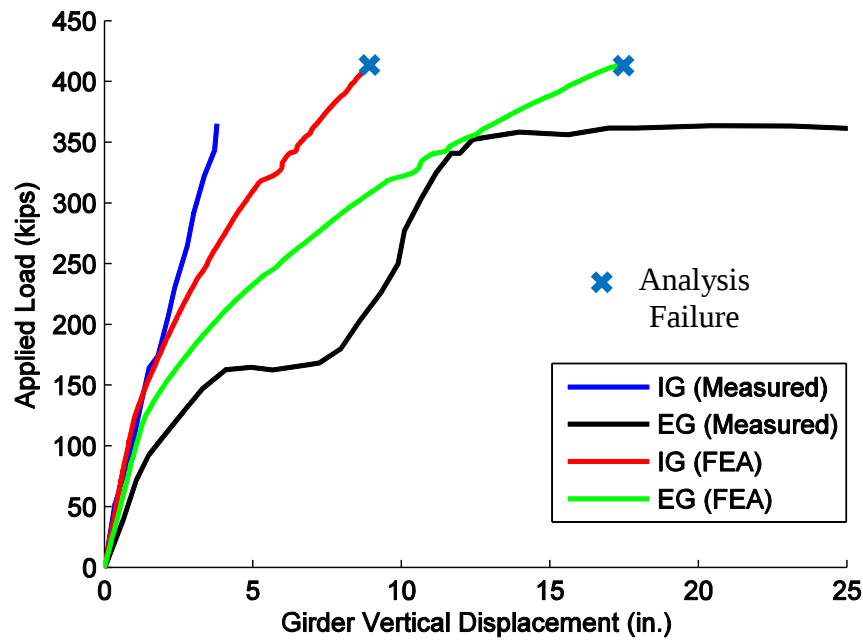


Figure 5-30: Test 3 Measured vs. FEA (18 ft. south of mid-span).

Figure 5-31 shows the von Mises stress distribution at the end of analysis. The tub-girder web and flanges are made of ASTM A572 Grade 50 steel while the lateral and cross bracing, stiffeners, diaphragms are made of ASTM A36 steel. The display scale is set to display regions having stress higher than 50 ksi in red to visually identify regions of yield in box-girder. Significant yielding is observed on the interior box-girder between 0.3L and 0.7L. The interior top flange of the fractured girder and the remaining intact web shows yielding at the end of analysis. Yielding in the interior top flange spread 0.1L on either side of the fracture. Local yielding is also observed where the lateral bracing frames into the top flange.

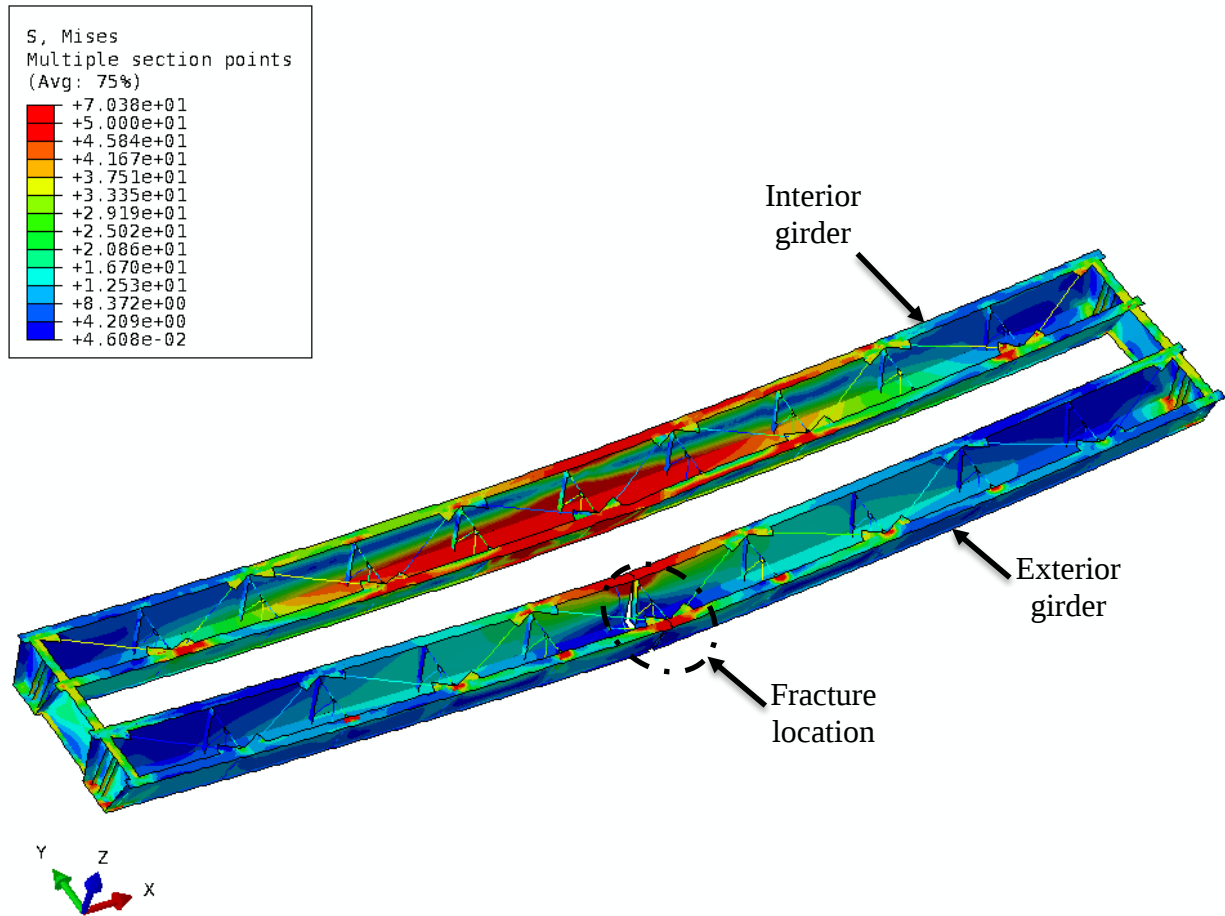


Figure 5-31: von Mises stress distribution in steel superstructure at the end of analysis.

5.7 Parameters affecting twin box-girder response

The objective of this study is to evaluate the impact of different geometric and analysis parameters on the response of twin box-girders bridges tested at The University of Texas at Austin. Results only from Test 2 and Test 3 are used in this study as following bottom flange fracture in Test 1 the bridge showed minimal change in deflection. Figure 5-12 and Figure 5-13 describes the series of steps used to simulate Test 2 and Test 3 performed on the twin box-girders. Beyond 12.5 in. of vertical deflection, the measured Test 3 load vs. displacement response flattens out and therefore the response only up to 15 in. is used for comparison. During the Test 3 simulation discussed in Section 5.5.5 and Section 5.6.4, the analysis is continued beyond Test 3 loading by increasing the load applied outside the bin until the analysis failed to converge. The analyses performed in this section does not include the additional loading beyond

the Test 3 loading of 363.3 kips. Unless mentioned otherwise the analyses in this section are carried out using the parameters and assumptions used for the baseline analysis discussed in Section 5.6. The parameters evaluated in this study are as follows:

- Modeling parameters
 - Rotational stud stiffness
 - Shear stud shear-tension interaction models
 - Rate dependent material properties
 - Steel plasticity definition
 - Boundary conditions model
 - Concrete shrinkage
- Geometric parameters
 - Extent of fracture
 - Guardrail contribution
 - Additional cross frame

5.7.1 Boundary condition

The twin box-girders were supported using reinforced elastomeric bearing pads on both ends. The baseline analysis carried out in Section 5.6 used connector elements with elastic stiffness calculated using Equation 5-1 through Equation 5-5 (*Bearing Pad model*). The simplified approach to represent simply supported boundary condition on the bridge is to apply pin-roller boundary condition at girder ends (*Pin-Roller Boundary model*). This section evaluates the effect of pin-roller boundary condition on Test 2 and Test 3 response. The pin boundary condition (all three translations restrained) is applied to the south end while roller boundary condition (only vertical motion restrained) is applied to the north end of twin box-girders. Boundary conditions are applied over the area of the bottom flange in contact with the bearing pads. Figure 5-32 shows the impact of boundary conditions on post fracture response in Test 2. The mean deflection using pin-roller boundary condition is 1.5% lower compared to the field observation while bearing pad model results in 17.4% increase in mean deflection at mid-span of the exterior girder. Figure 5-33 shows the impact of boundary condition on Test 3 load vs. displacement response. Both boundary conditions yielded similar initial response up to 125 kips. After that load, the *Pin-Roller Boundary model* yielded stiffer response. Use of pin-roller boundary

condition to model bearing pad results in additional restraint as indicated by the stiffer response in Test 2 (Figure 5-32) and Test 3 (Figure 5-33) response. Both models successfully carried the Test 3 load of 363.3 kips.

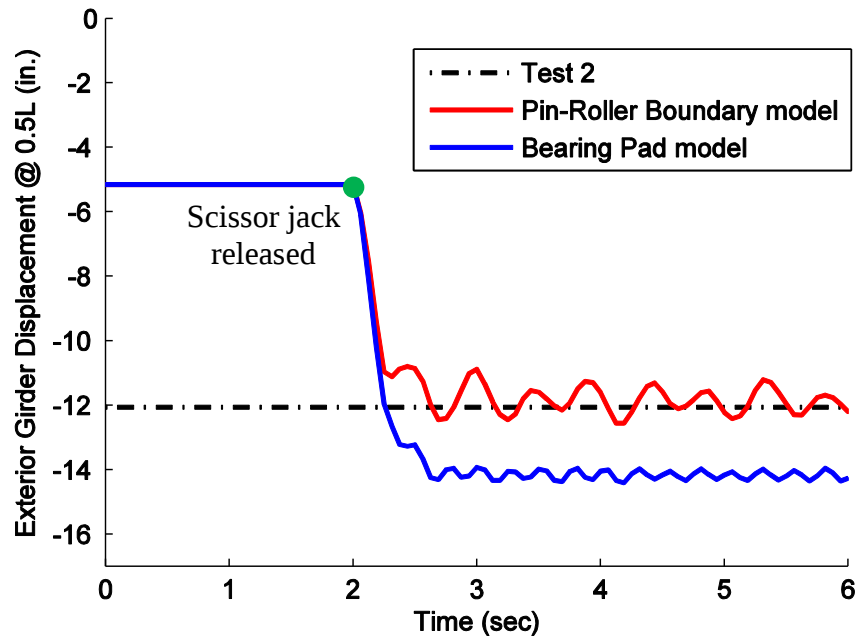


Figure 5-32: Test 2 response with boundary condition models.

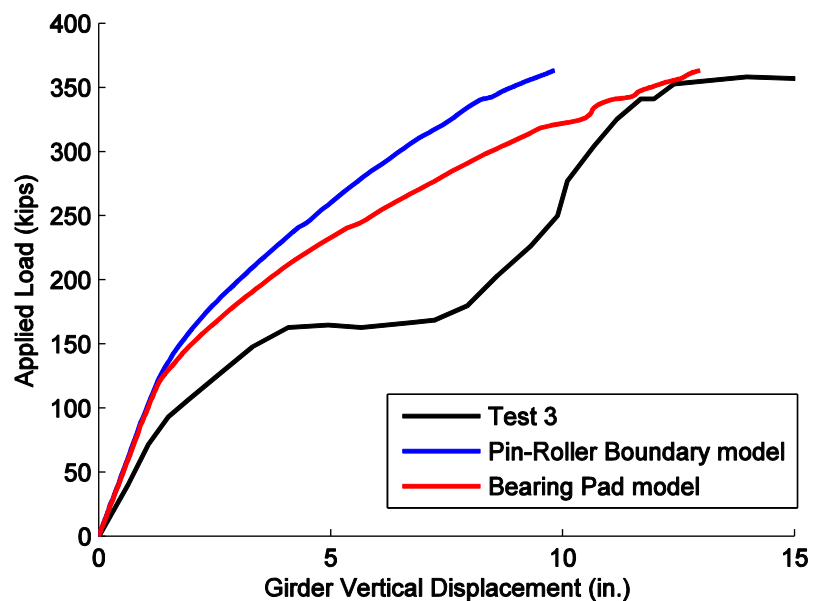


Figure 5-33: Test 3 response with boundary condition models (18 ft. south of mid-span of fractured girder).

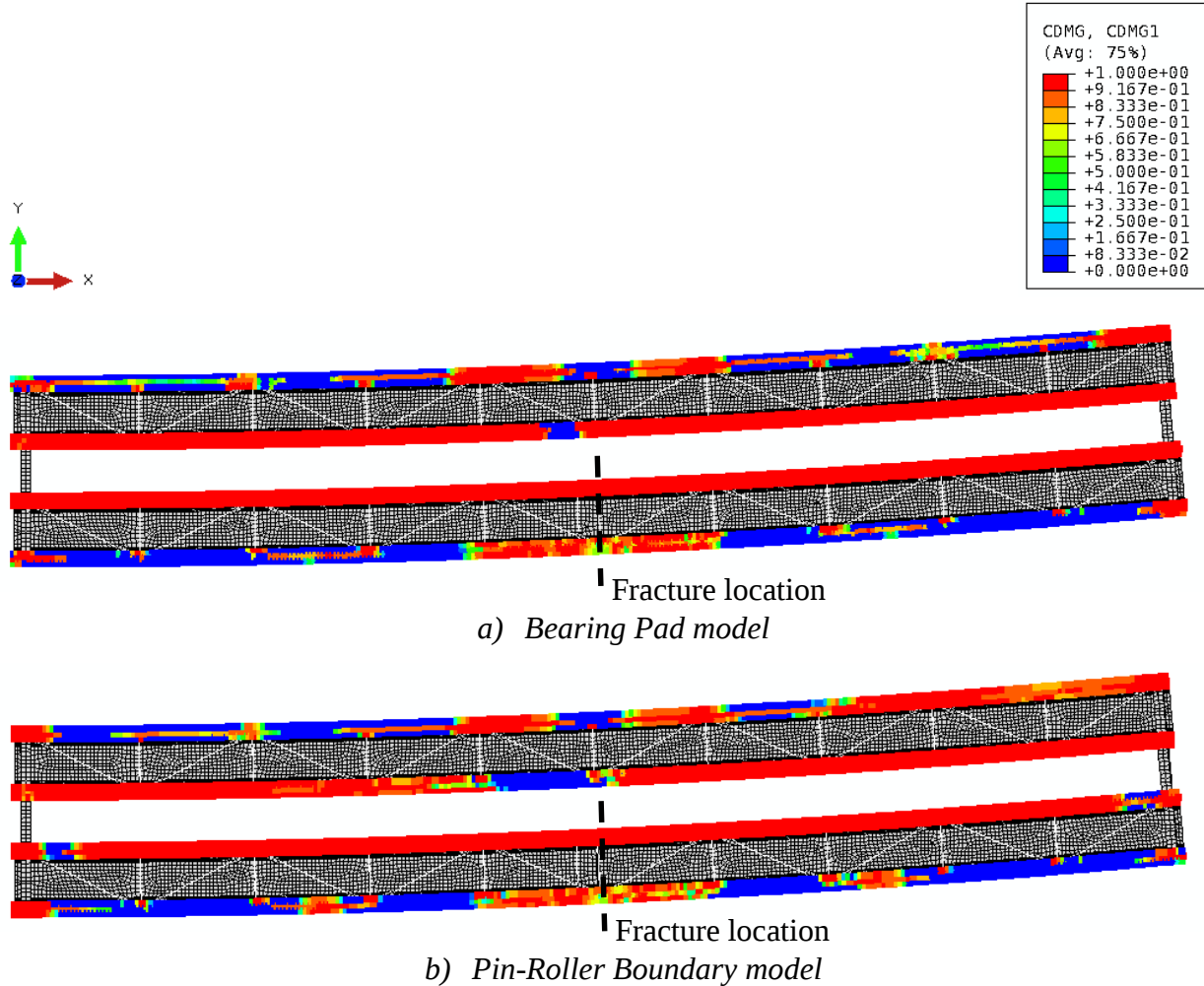


Figure 5-34: Connector overall damage variable (CDMG) at end of 5 sec (Test 2).

5.7.2 Fracture-I and Fracture-II

During rearrangement of the concrete girders in first loading stage of Test 3, the fracture on the outside web of the exterior girder propagated when the total load on the bridge was 40.9 kips. *ABAQUS/Explicit* does not support connector element removal during a step. Therefore, the analysis carried out in Section 5.6.4 included Test 3 fracture at the start of Test 2 simulation steps (*Fracture-II model*). In order to evaluate the effect of additional web fracture at the start of Test 2 on Test 3 response, the bridge model is reanalyzed without the additional web fracture (*Fracture-I model*) as shown in Figure 5-35. Both the *Fracture-I model* and *Fracture-II model* yielded similar response up to 125 kips (Figure 5-36) which is well beyond 40.9 kips load at which the fracture on the outside web of the exterior girder propagated further. Therefore, introduction of Test 3 fracture at the start of Test 2 simulation steps can be used in simulation of

Test 3 response without significantly altering the initial load vs. displacement response. Loading of the *Fracture-I model* is continued with the Test 3 loading stages to compare its load vs. displacement response with the *Fracture-II model* response. The *Fracture-I model* with less fractured web yielded stiffer response compared to *Fracture-II model* beyond 125 kips.

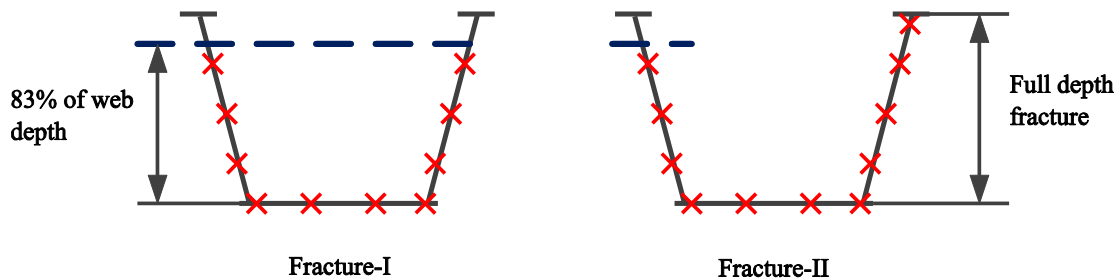


Figure 5-35: *Fracture-I* and *Fracture-II*.

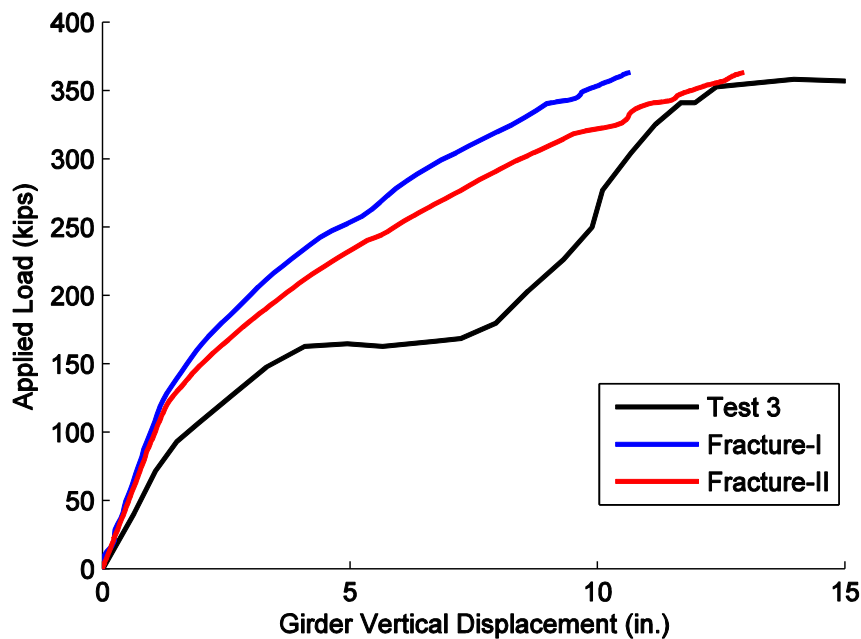


Figure 5-36: *Fracture-I* vs. *Fracture-II* Test 3 response (18 ft. south of mid-span of fractured girder).

5.7.3 Rotational stud stiffness

In the baseline analysis described in Section 5.6, the rotational degrees of freedom between the concrete slab nodes and the top flange nodes of the twin box-girders are coupled using rotational stiffness of 100,000 kip-in/rads. To ensure this does not significantly alter the simulation response, Test 2 model in Section 5.6.3 is reanalyzed with lower rotational stiffness values. Figure 5-37 shows the post fracture response of the exterior girder at mid-span following release of scissor jack with different rotational stiffness values. Reduction in rotational stiffness resulted in increase in the peak to peak amplitude but the mean deflection stayed within 5% of the mean deflection with rotational stiffness of 100,000 kip-in/rads. Therefore, a rotational stiffness of 100,000 kip-in/rads can be considered reasonable and is used in rest of the analyses.

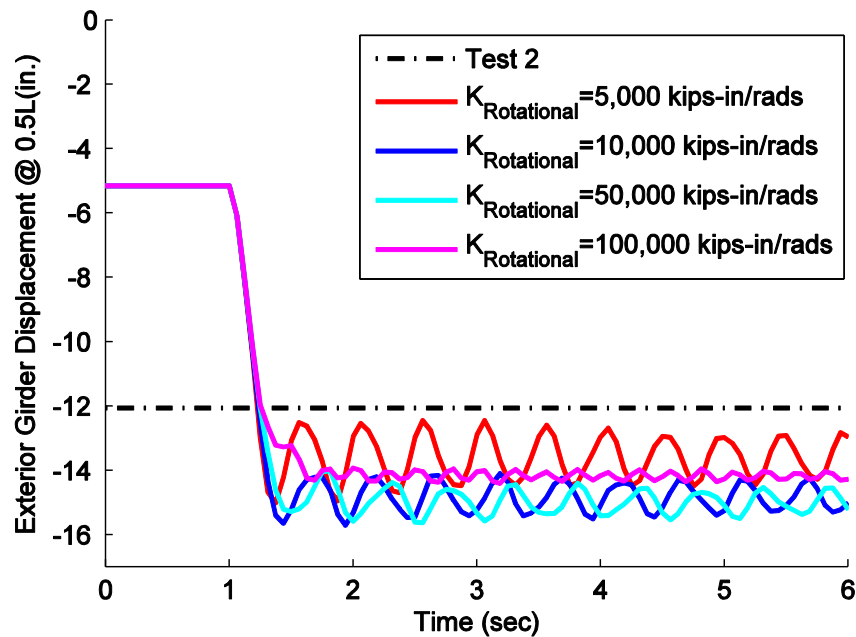


Figure 5-37: Impact of rotational stud stiffness on post fracture Test 2 response.

5.7.4 Shear-tension interaction models for shear studs

During Test 2 and Test 3, stud failure in combined shear and tension was observed resulting in a separation at the steel-concrete interface. In Section 5.6, PCI Design Handbook, 6th Edition (2004) interaction equation with the initial shear stud stiffness based on the force vs. slip relationship proposed by Topkaya et al. (2004) and the initial tensile stud stiffness based on the stud and the surrounding concrete cone is used. The shear stud model captured the overall/global trends in Test 2 and Test 3 response reasonably well but it failed to match the field response

exactly. In particular, it failed to capture the sudden separation at the steel-concrete interface observed at 161.5 kips during Test 3. The shear stud model used in Section 5.6.3 simulations is possibly one of the reasons for the difference in response. This section evaluates capability of PCI Design Handbook, 6th Edition (2004) interaction equation with different initial stiffness values and uncoupled shear-tension failure model to capture the field response.

Stud tension behavior: In the simulations discussed in Section 5.6, the initial tensile stud stiffness model (*Stud + Surrounding Concrete Stiffness model*) based on the stud and the surrounding concrete cone (47,079 kip/in per 3 stud group) is used. Alternatively the initial tensile stud stiffness based only on the shear stud stiffness (10,463 kip/in per 3 stud group) can be used during simulation (*Stud Stiffness Only model*). Both models use the same stud failure envelope based on the PCI Design Handbook, 6th Edition (2004) interaction equation. Figure 5-38 and Figure 5-39 shows the comparison between different initial tensile stud stiffness models on the Test 2 and the Test 3 response respectively. The mean dynamic deflection with the *Stud Stiffness Only model* is 9.7% lower and with the *Stud + Surrounding Concrete Stiffness model* is 17.4% higher compared to the Test 2 field measurement (Figure 5-38). Use *Stud Stiffness Only model* resulted in larger peak to peak amplitude of oscillation compared to the *Stud + Surrounding Concrete Stiffness model*. The lower initial stud stiffness draws less force during the dynamic bridge response following release of scissor jacks resulting in a lower number of stud failures. This lower number of failed studs result in a stiffer structural response as shown by the lower mid-span deflection of the exterior girder with the *Stud Stiffness Only model*.

In the Test 3 simulation, the *Stud + Surrounding Concrete Stiffness model* displayed linear response up to 125 kips; beyond 125 kips it displayed nonlinear load-displacement response. The *Stud Stiffness Only model* displayed linear response up to 300 kips while underpredicting the exterior girder deflection value at 18 ft. south of mid-span. Beyond 300 kips, the deflection grew very rapidly as a result of failure of large number of studs on the inside flange of the interior girder (Figure 5-40). Both models failed to capture the sudden separation at the interface observed at 161.5 kips. Figure 5-40 shows the evolution of stud damage using the connector overall damage variable (CDMG) during Test 3 simulation using the *Stud Stiffness Only model*. The lower initial stud stiffness in the *Stud Stiffness Only model* draws less force and engaging

more number of studs. The engaged studs reached the failure envelope simultaneously as in Figure 5-40-(b). As noted earlier, the *Stud Stiffness Only model* uses viscous pressure to reach a quasi-static solution in shorter time in *ABAQUS/Explicit*. The viscous pressure values are proportional to the velocity values in the model. As the deflection grew rapidly past 300 kips, the velocity in the finite element model increased resulting in increase in viscous pressure values. In turn, this increased viscous pressure resulted in increased damping of dynamic effects and allowed the bridge model to recover and carry Test 3 loading. The rapid increase in deflection was not observed during testing and the recovery of the finite element model is because of numerical help from viscous pressure. Therefore, results beyond 300 kips are not considered to be valid.

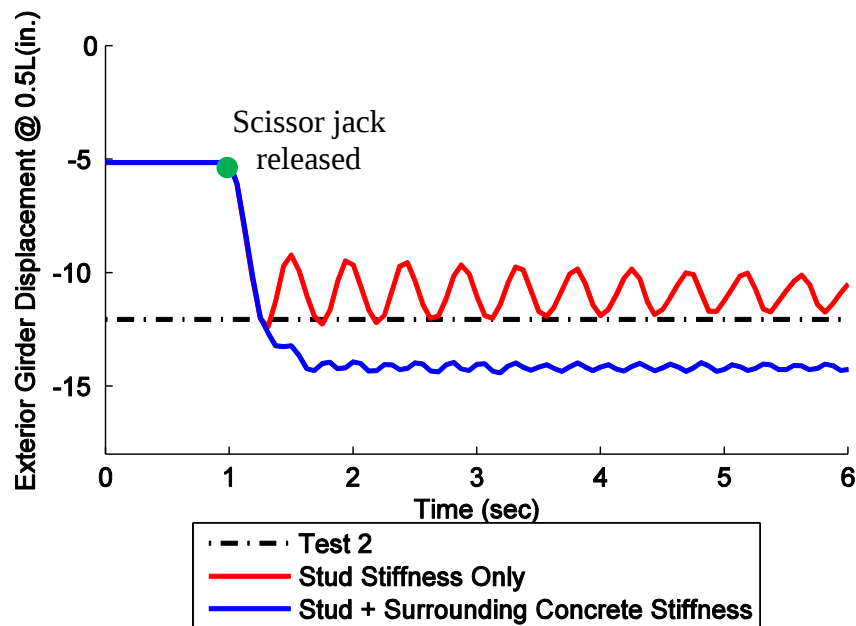


Figure 5-38: Test 2 response using different initial tensile stiffness.

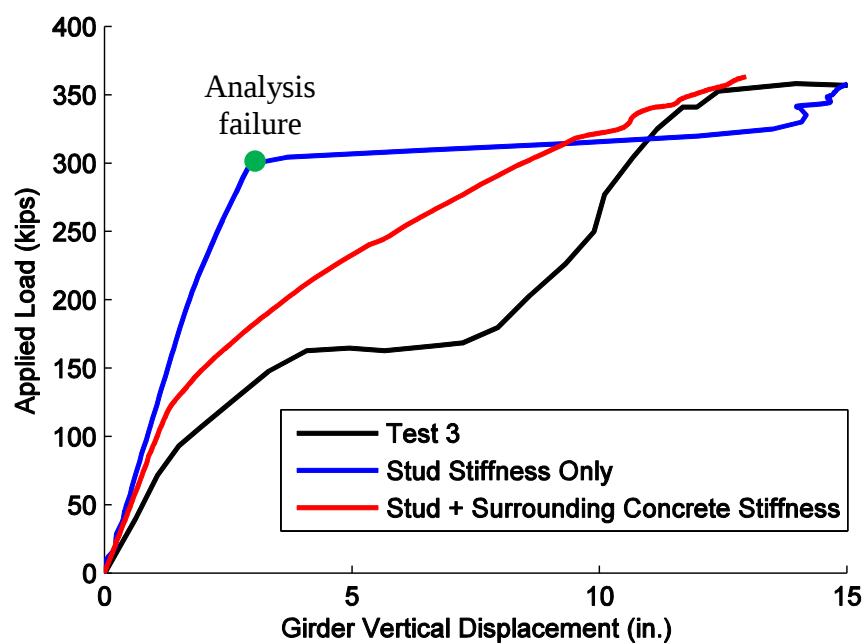


Figure 5-39: Test 3 response using different initial tensile stiffness (18 ft. south of mid-span of fractured girder).

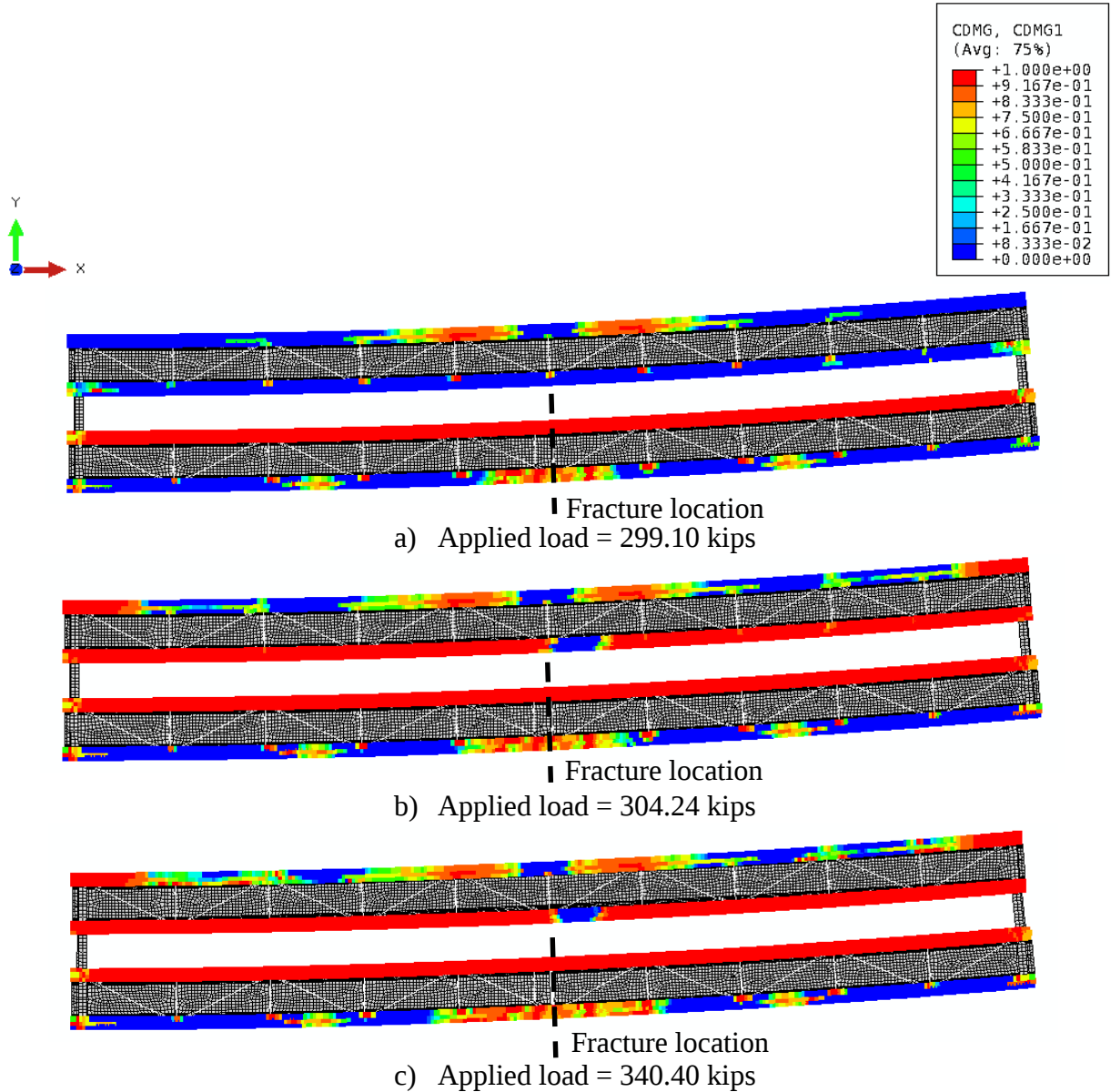


Figure 5-40: Connector overall damage variable (CDMG) evolution in *Stud Stiffness Only* model.

Stud shear behavior: In the Section 5.6 simulations, *Initial Stud Stiffness model* (3783.9 kip/in per 3 stud group) is based on the force vs. slip relationship proposed by Topkaya et al. (2004). Alternatively, it can be replaced with *Secant Stud Stiffness model* (324.2 kip/in per 3 stud group) based on the secant stiffness of the force vs. slip relationship proposed by Topkaya et al. (2004). The *Initial Stud Stiffness model* fails to capture the total interface slip while the *Secant Stud Stiffness model* captures the total interface slip but over predicts the slip before stud shear failure. Another alternative is to directly use the force vs. slip relationship proposed by Topkaya et al. (2004) as part of stud definition (*Topakaya Force-slip model*). The *Topakaya Force-slip model*

solves both the problems proposed by previous two models but it fails to capture the failure from shear-tension interaction as the ABAQUS (2013) connector definition is incapable of handling a plasticity definition along with a shear-tension failure envelope. During simulation of response using the three stud shear models discussed above, the initial tensile stud stiffness value based on the stud and the surrounding concrete cone is used. In addition to the three models discussed above, shear studs are also modeled using the beam connector elements (*Rigid Stud model*) to evaluate the validity of widely used rigid stud approach.

Figure 5-41 shows the impact of different stud shear behavior models on the Test 2 response. At the end of Test 2 post fracture response simulation, the *Initial Stud Stiffness model* showed stud damage along entire length of the interior flange of interior girder which is not observed in either the *Secant Stud Stiffness model* and the *Topakaya Force-slip model* response; and additional damage on the exterior flange of exterior fractured girder at location of fracture (Figure 5-42). Additional stud damage resulted in softening of the structure leading to larger mid-span deflection as shown in Figure 5-41. *Secant Stud Stiffness model* and *Topakaya Force-slip model* show similar stud damage at the end of Test 2 post fracture response simulation. Table 5-11 shows the mean deflection values and the percent difference in response compared to the field measurement for all four shear stud model. *Topakaya Force-slip model* yielded mid-span deflection closest to the field measurement as compared to the other stud shear models. *Rigid Stud model* under predicted the post fracture exterior girder mid-span deflection by 40%.

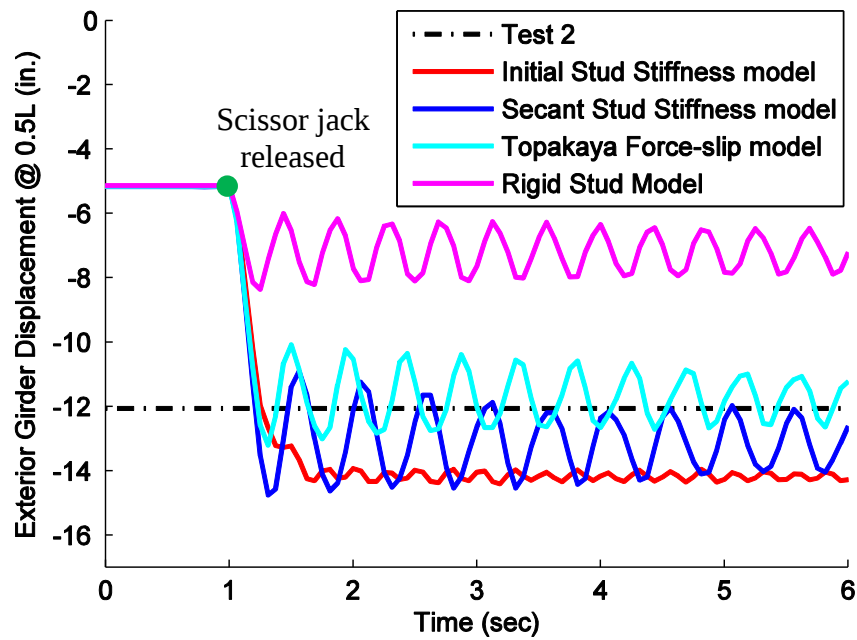


Figure 5-41: Test 2 post fracture response using different stud shear models.

Table 5-11: Mean mid-span deflection using different stud shear models.

Model description	Mid-span deflection (in.)	% Difference
Test 2	-12.07	-
Initial Stud Stiffness model	-14.17	-17.4
Secant Stud Stiffness model	-13.10	-8.5
Topakaya Force-slip model	-11.73	2.8
Rigid Stud model	-7.24	40.0

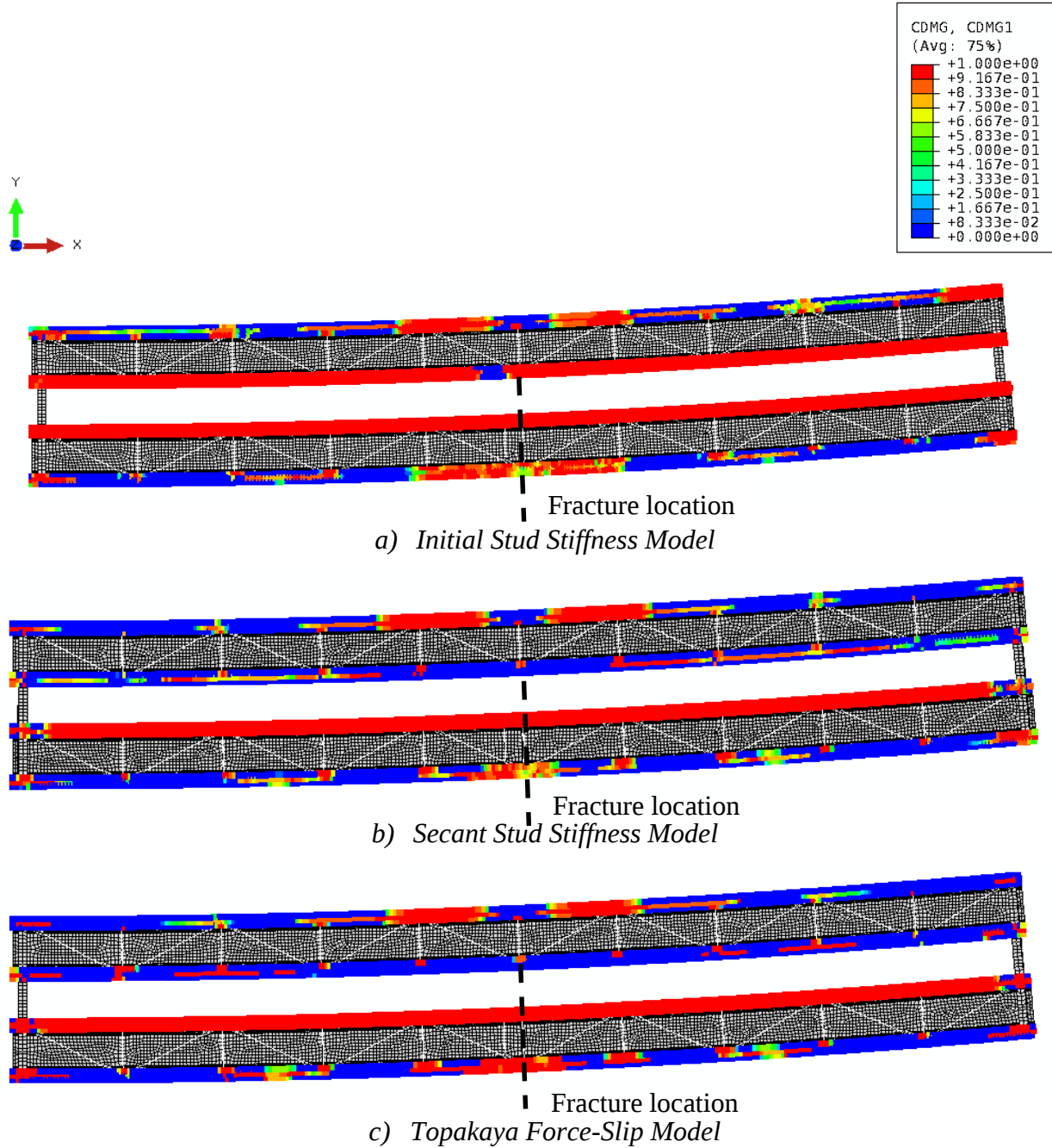


Figure 5-42: Connector overall damage variable (CDMG) values in shear studs at the end 5 sec in Test 2 simulation for different stud shear behavior models.

Figure 5-43 shows the Test 3 load vs. displacement response comparison using different stud shear behavior models as described at the start of this section. The *Secant Stud Stiffness model* and the *Topakaya Force-slip model* show fairly linear response up to ~200 kips and beyond 200 kips they display slight nonlinearity in the load-displacement response. Both models severely under predict the deflection values. In case of the *Secant Stud Stiffness model* and the *Topakaya*

Force-slip model lower stud damage at the end of Test 2 results in different starting point for Test 3. Different starting point alters the stud damage during Test 3 (Figure 5-44) resulting in to stiffer load vs. displacement response (Figure 5-43).

The *Rigid Stud model* yielded the stiffest response of all four models investigated as a part of this study. The load-displacement response with the *Rigid Stud model* showed slight nonlinearity at loads beyond ~225 kips. Results from the Test 2 (Figure 5-41) and Test 3 (Figure 5-43) clearly show that, use of *Rigid Stud model* would lead to false sense of safety. Therefore, rigid stud approach is not suitable for capturing post fracture behavior of bridges. In an attempt to develop a conservative model for stud behavior in combined shear and tension, analysis is carried out with very high initial stud stiffness (10^6 kip/in) for displacement degrees of freedom of connector elements (*Initial Rigid PCI Stud model*). This resulted in catastrophic failure of shear studs at the start of post fracture analysis leading to analysis failure. Therefore, results from this model are not include in Figure 5-43. In order to simulate reserve load carrying capacity of a damaged bridge and sequence of failure mechanisms we need realistic representation of shear studs not the conservative representation of it.

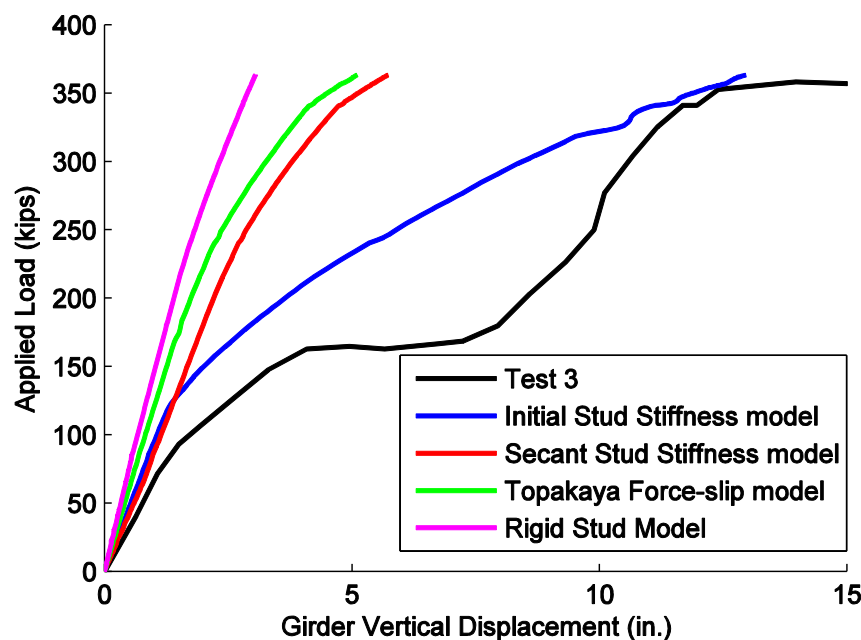


Figure 5-43: Test 3 response using different stud shear models (18 ft. south of mid-span of exterior girder).

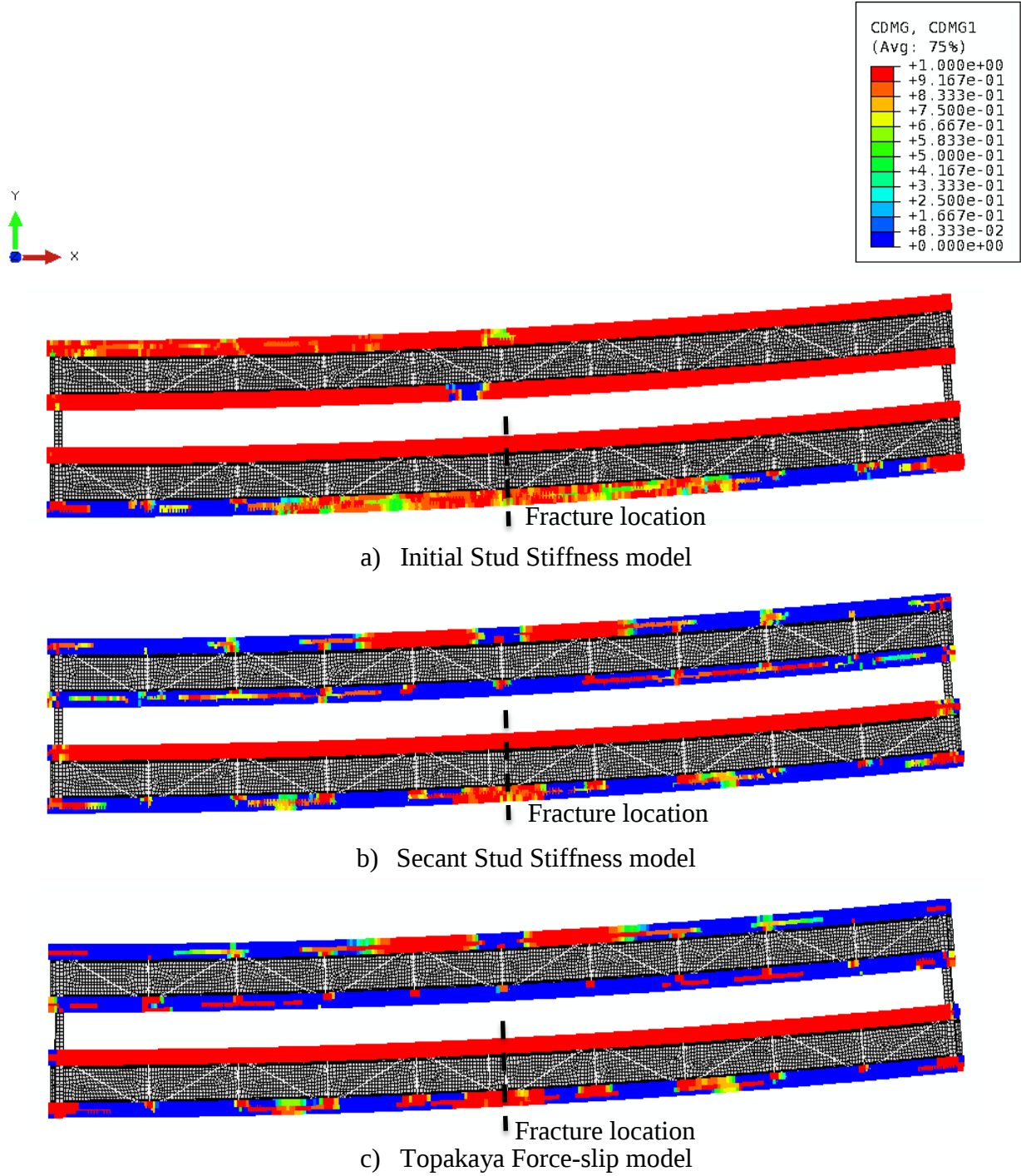


Figure 5-44: Connector overall damage variable (CDMG) at the end of Test 3 loading.

5.7.5 Rate dependent material properties (Test 2)

Structural elements subjected to dynamic loading show an increase in strength because of increased material strength under rapid loading rates. The Hoan Bridge results showed ~20% decrease in the fractured girder deflection response when the rate dependent material properties were used. Therefore, Test 2 simulation is reanalyzed using the rate dependent material properties from UFC 3-340-02 (2008) while the stud properties are assumed to stay the same in the absence of experimental data in the literature. The use of rate dependent material properties resulted in reduction of 4 % in the exterior girder deflection at mid-span (Figure 5-45). Part of the reason for the reduced sensitivity in the twin box-girder bridge as compared to the Hoan Bridge is the absence of rate dependent properties for shear studs in the twin box-girder bridge.

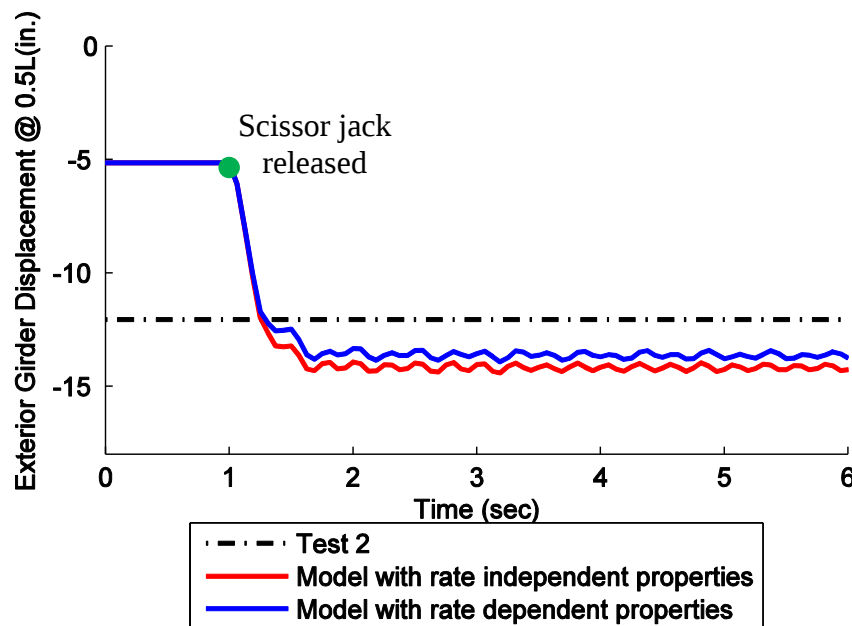


Figure 5-45: Effect of rate dependent properties on post fracture response (Test 2).

5.7.6 Steel plasticity definition (Test 2)

ABAQUS (2013) allows users to select between an isotropic hardening model, a Johnson-Cook isotropic hardening model, a linear kinematic hardening and a nonlinear isotropic/kinematic hardening models to model cyclic behavior of structural steel. In this section, the effect of the isotropic hardening model and the nonlinear isotropic/kinematic hardening model with kinematic hardening component alone on Test 2 post fracture response is evaluated. Both hardening models yielded similar post fracture mid-span deflection response in the exterior girder as shown in

Figure 5-46 with the mean deflection at mid-span about 17.4 % higher compared to the to the field observation. Similar response using both hardening models indicates absence of stress reversal. In the absence of stress reversal, the Bauschinger effect modeled using the kinematic hardening model does not influence the mid-span deflection response (Figure 5-46). Therefore, the isotropic hardening model is sufficient to model the steel plasticity in Test 2.

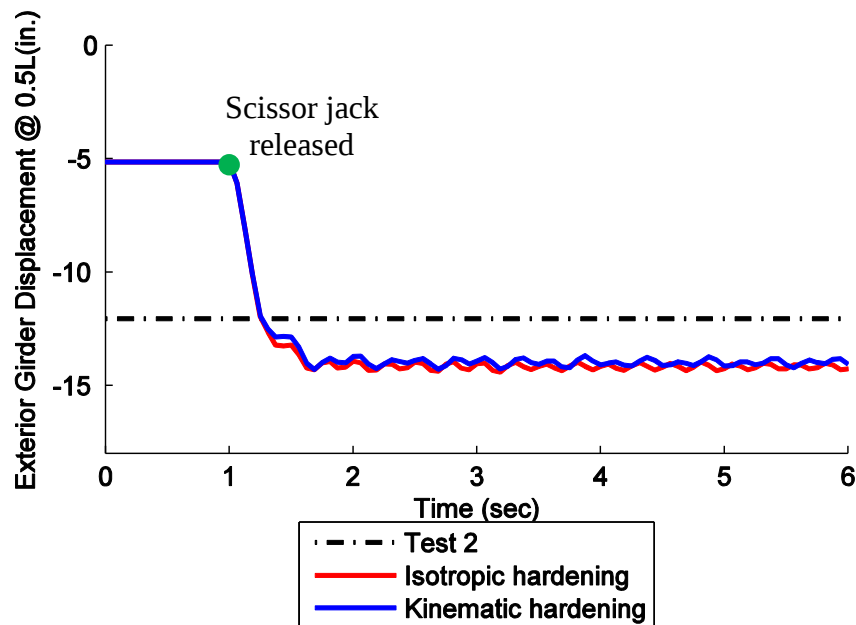


Figure 5-46: Effect of steel plasticity models on post fracture response (Test 2).

5.7.7 Guardrail contribution

The Twin box-girder bridge used cast in place TxDOT standard T501 rails with joints every 30 feet. In the event of fracture, the contribution of guardrail to the strength and stability of bridge superstructure can be significant. This section investigates the effects of including the guardrail in the finite element models for the Test 2 and the Test 3 simulation. Figure 5-47 shows the post fracture response of the exterior girder at mid-span following removal of scissor jacks in Test 2. The mean deflection without guardrail is 0.9% higher when compared to the field observation while the model with the guardrail results in a 17.4% higher mean deflection. Exclusion of the guardrail from the model resulted in less stud damage on the interior intact girder (Figure 5-49), and therefore in to stiffer structural response and lower bridge deflection.

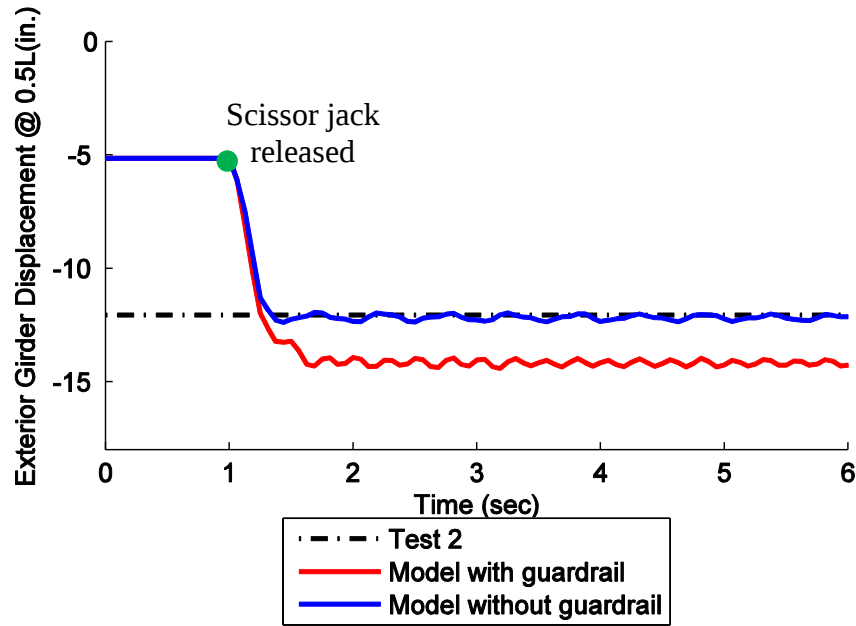


Figure 5-47: Impact of guardrail on post fracture response (Test 2).

Figure 5-48 shows the impact of the guardrail on the load vs. displacement response in Test 3. Exclusion of the guardrail from the model resulted in softer response with results matching the field response up to 100 kips of load. After 100 kips, the model without guardrail yields stiffer response compared to the measured response. The model without guardrail also fails to capture the sudden increase in displacement observed at 161.5 kips. At 276 kips the load-displacement response of the model with guardrail crosses the model without guardrail and its response begins to soften. Similar to the Test 2 response, exclusion of the guardrail from the finite element model resulted in less stud damage on the exterior flange of the interior girder (Figure 5-50). In the absence of the guardrail, the plasticity in the concrete slab at the location of fracture and overall distributed over a larger slab area as compared to the model with guardrail. This is shown by the principal plastic strain distribution in the slab for both models (Figure 5-51). Distribution of plasticity over larger area of slab engaged more studs which possibly explains the lower number of stud failures observed in the model without guardrail. The question remains whether stiffer structural response observed in the absence of guardrail is real or numerical.

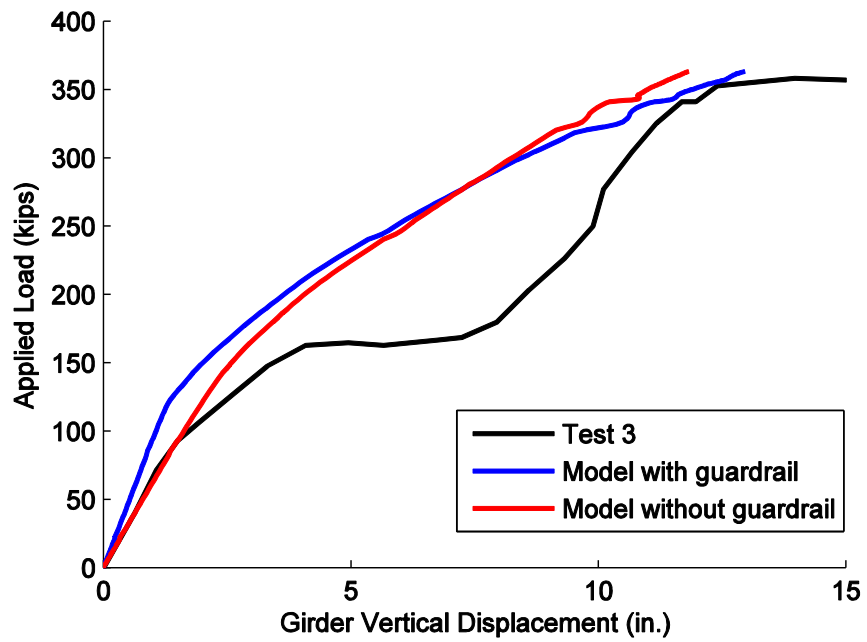


Figure 5-48: Guardrail impact on Test 3 response (18 ft. south of mid-span of fractured girder).

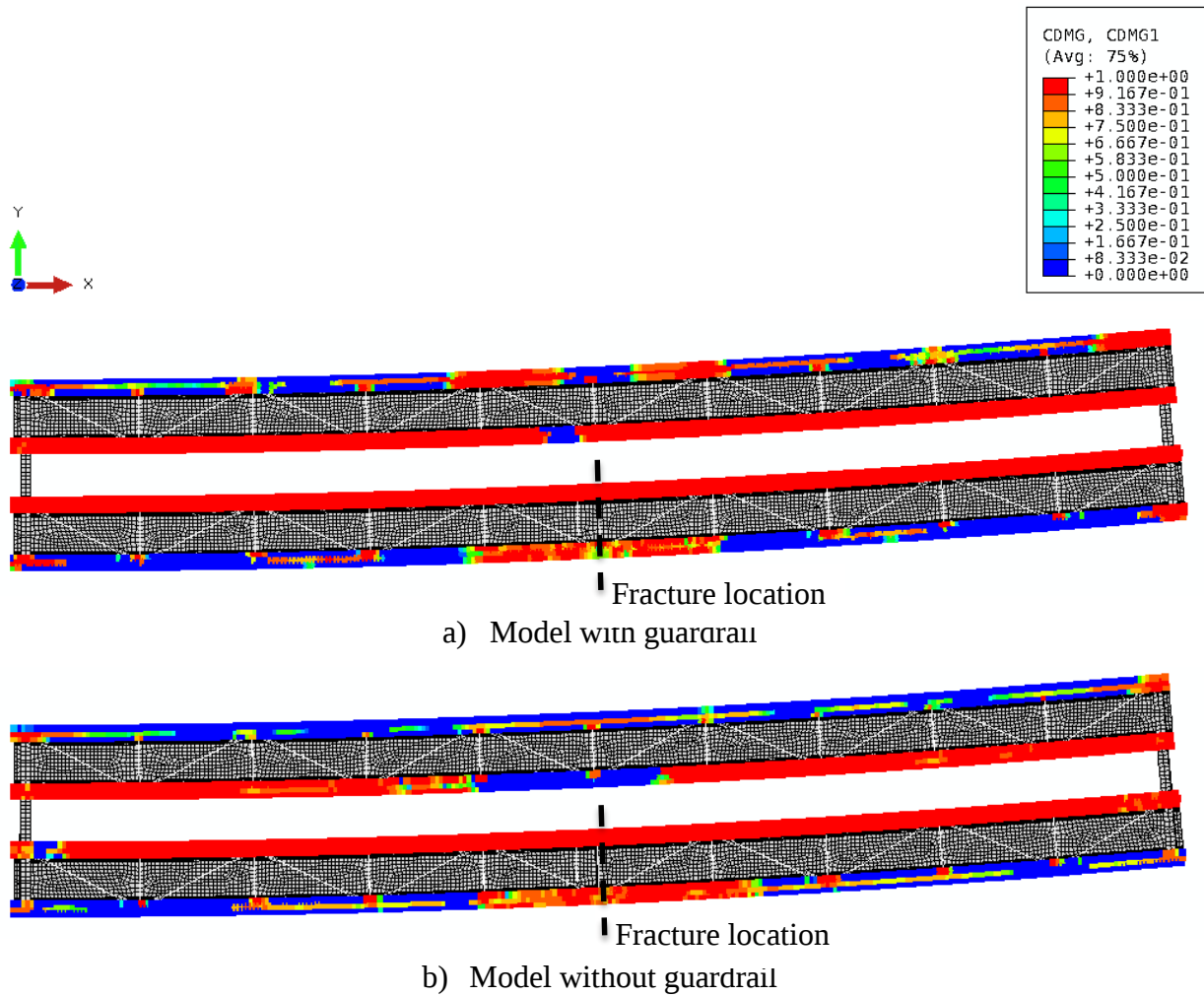


Figure 5-49: Connector overall damage variable (CDMG) at end of 5 sec (Test 2).

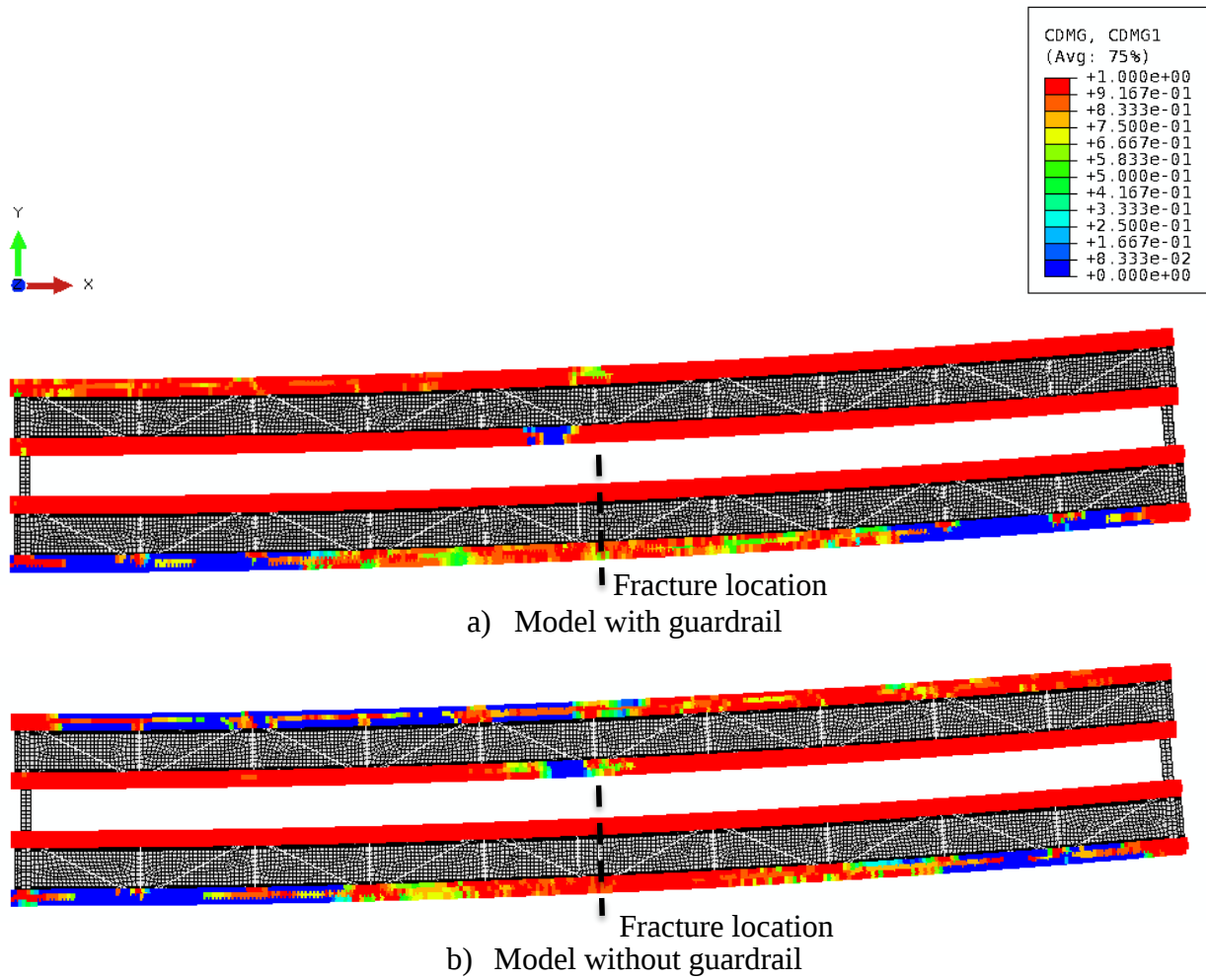


Figure 5-50: Connector overall damage variable (CDMG) at end of Test 3.

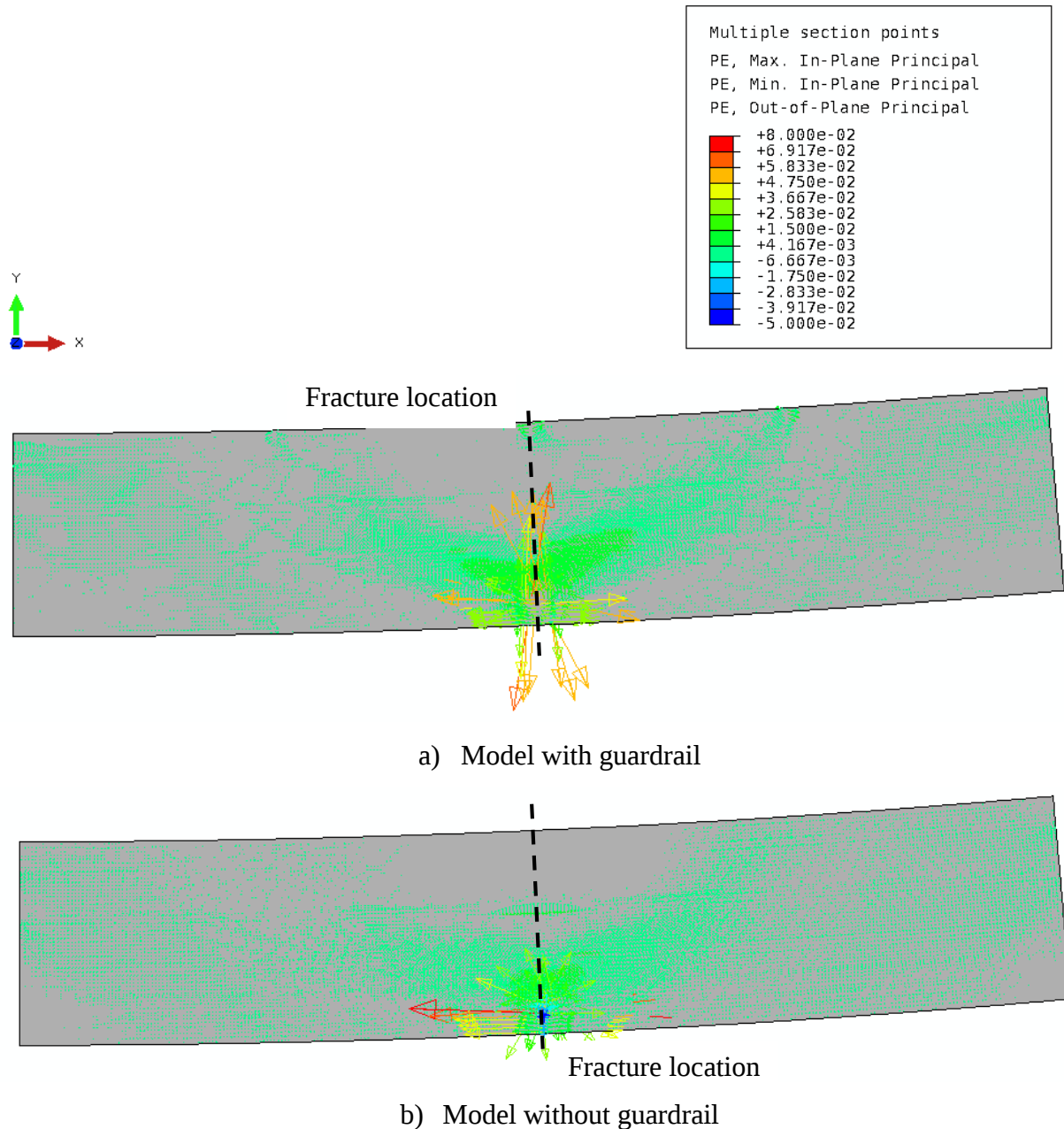


Figure 5-51: Principal plastic strain distribution in concrete slab.

5.7.8 Additional cross frame

An additional cross-frame made of angles was welded inside the exterior girder two feet from the fracture location on the side opposite the existing cross-frame to help provide similar resistance to twisting at both open ends of the fractured exterior girder (Neuman, 2009). The baseline finite element model (Section 5.6) is reanalyzed without the additional cross brace to evaluate its impact on Test 2 and Test 3. Figure 5-52 shows the post fracture response of the exterior girder at the mid-span following removal of scissor jacks in Test 2 with and without additional cross

frame. For the model without the additional cross frame, the mean deflection at mid-span of the exterior girder showed 5.3% reduction compared to the model with additional cross frame.

Figure 5-53 shows the impact of additional cross frame on the Test 3 load vs. displacement response. Up to 120 kips, model with and without additional cross frame yielded identical load vs. displacement response and beyond 120 kips model without additional cross frame yielded stiffer response. Absence of additional cross frame resulted in significant reduction in accumulated stud damage on the exterior flanges of both box-girders leading to stiffer structural response (Figure 5-54). Presence of additional cross frame stiffened the box-girder at the location of fracture which possibly resulted in the damaged box-girder carrying more load. This additional load over loaded the shear studs close to the fracture resulting in to unzipping failure of shear studs in combined shear and tension. Failure of shear studs over larger length of the bridge lead to higher vertical deflection at the end of Test 3 loading. The question remains as to whether the stiffer structural response observed in the absence of the additional cross frame is real or numerical. The behavior of structural components is coupled in the post fracture evaluation of the twin box-girder bridge and it is not possible to isolate contribution of one component from rest of the bridge.

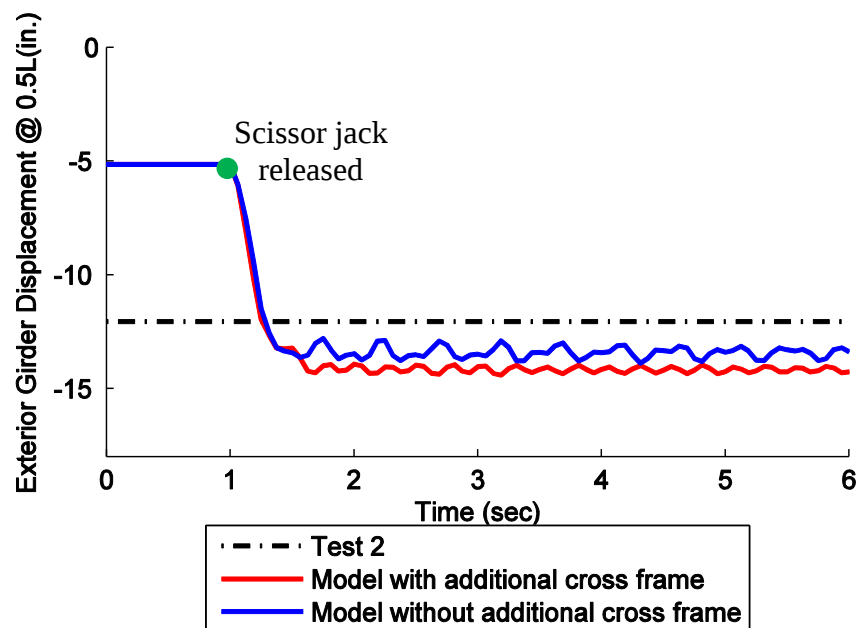


Figure 5-52: Impact of additional cross on Test 2 post fracture response.

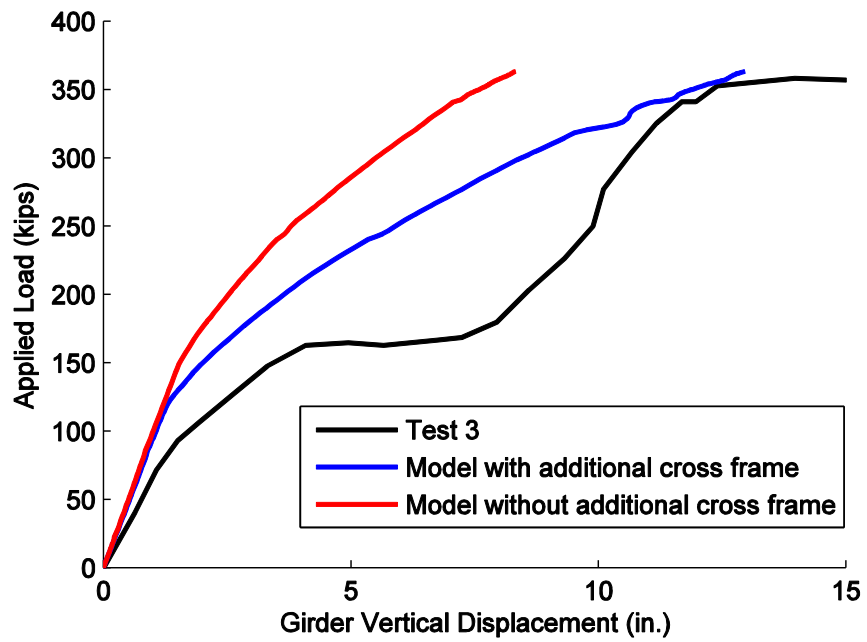


Figure 5-53: Impact of additional cross frame on Test 3 (18 ft. south of mid-span of fractured girder).

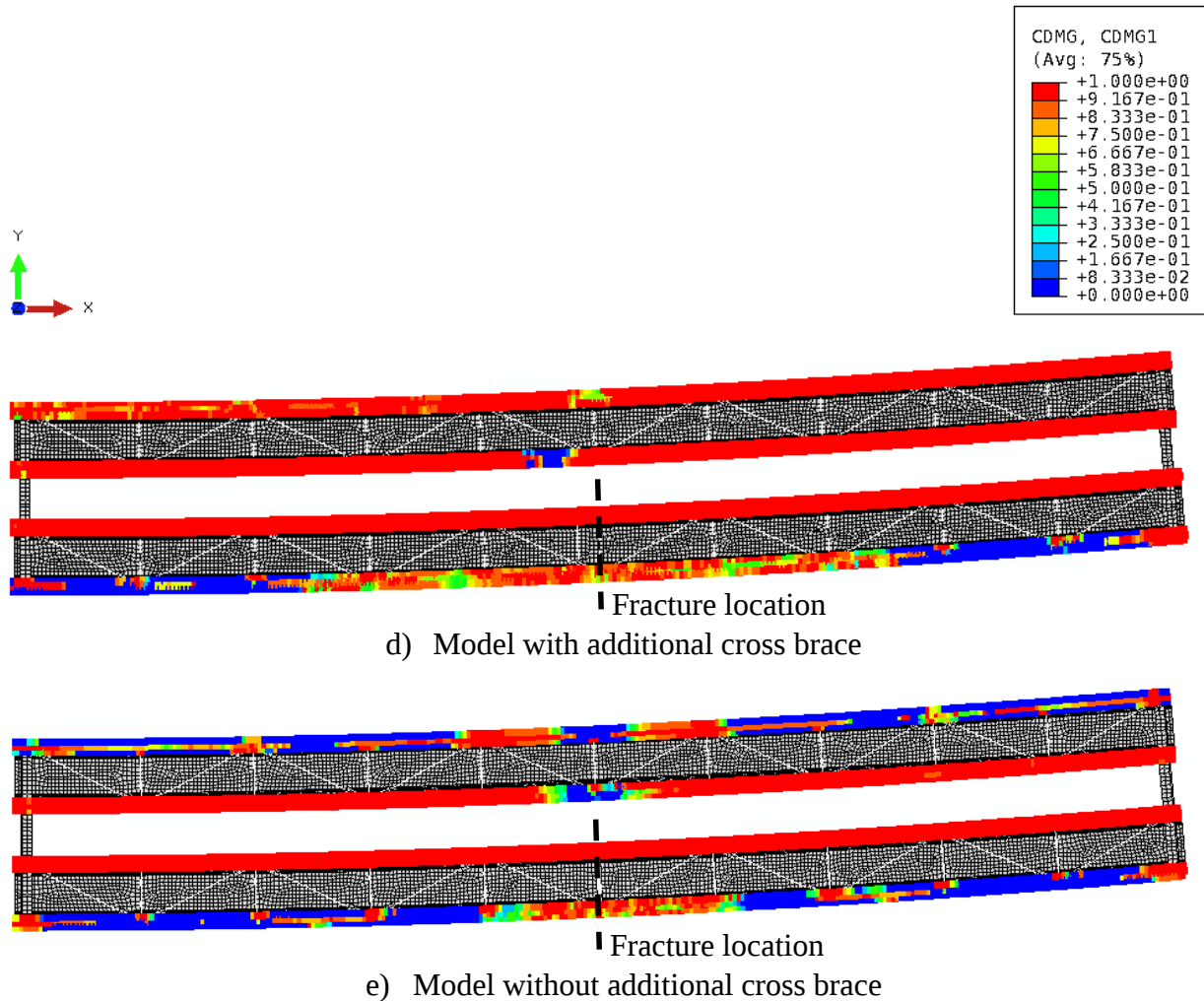


Figure 5-54: Connector overall damage variable (CDMG) at the end of Test 3 analysis.

5.7.9 Concrete shrinkage (Test 1)

Restrained shrinkage in the concrete slab can lead to cracking or stress concentrations in the concrete slab. To evaluate the impact of concrete shrinkage on the post fracture response, Test 1 simulation with *Implicit quasi-static analysis* is reanalyzed with inclusion of shrinkage strains. Test 1 was carried out on day 63 after concrete slab construction. AASHTO (2012) shrinkage model estimated unrestrained shrinkage stain of $225.5 \mu\epsilon$ at 63 days. The unrestrained shrinkage strain is applied as a negative temperature change through the concrete slab and guardrail at the end of the construction sequence simulation.

The exterior girder mid-span deflection at the end of Test 1 simulation with and without application of concrete shrinkage are tabulated in Table 5-12. The mid-span deflection increased by 0.87 in. (17% increase) as a result of the introduction of shrinkage strain in the model and it

also altered the stress distribution in the slab as shown in Figure 5-55. In the case of unrestrained shrinkage, shrinkage introduces compressive strains in concrete. Concrete shrinkage would result in increased downward deflection of a simply supported bridge. On a composite bridge, shear studs used to make slab composite with the steel superstructure offer a level of restraint introducing tensile stresses in concrete near shear stud connection (Figure 5-55). Tensile stresses from the restrained shrinkage in concrete near shear studs will possibly reduce the shear stud connection strength. The present finite element model does not capture effect of shrinkage strains on stud failure mode. Large tensile stresses are also developed at the location of expansion joints in the guardrail.

In the present study, the construction sequence and Test 1 simulation steps are performed in *ABAQUS/Standard* while Test 2 and Test 3 simulation steps are performed in *ABAQUS/Explicit*. The current version of ABAQUS (2013) does not support transferring of results from *ABAQUS/Standard* to *ABAQUS/Explicit* in an analysis where temperature values are defined at nodes of stress/displacement based elements (ABAQUS, 2013). Results can be transferred if coupled temperature-displacement analysis is carried out. The option of performing coupled temperature-displacement analysis is not pursued in this research. Looking at the Test 1 simulation results in Table 5-12, it is clear that concrete shrinkage can play an important role in capturing realistic post fracture response in Test 2 and Test 3.

Table 5-12: Exterior girder mid-span deflection (Test 1).

Model description	Mid-span deflection	% difference
Test 1 (Measured)	-5.70	-
Test 1	-5.17	9.3
Test 1 with shrinkage	-6.04	-6.0

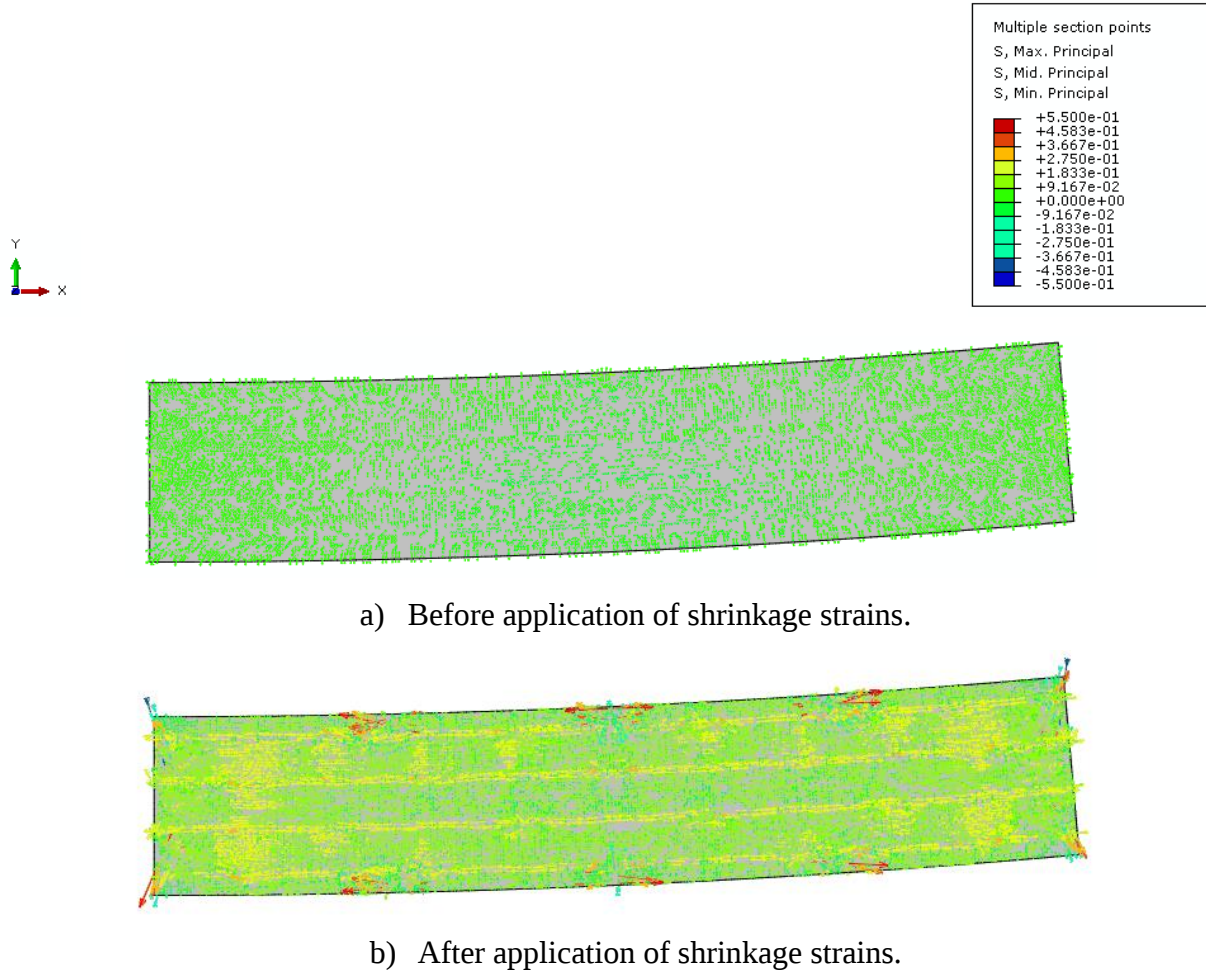


Figure 5-55: Principal stress distribution in slab after application of shrinkage strains (225.5 $\mu\epsilon$).

5.8 Simplified methods for Test 3 analysis

Section 5.6 and Section 5.7 used series of steps defined in Figure 5-13 to simulate Test 3. The steps defined in Figure 5-13 involve damage accumulation from Test 1 and Test 2, making the analysis fairly time consuming. A simplified method that does not involve damage accumulation from Test 1 and Test 2 would be very useful. This section evaluates suitability of two simplified modelling approaches to capture the reserve load carrying capacity of the damaged bridge. In the *Quasi-static Fracture model* the fracture is introduced quasi-statically at the start of the analysis followed by Test 3 load and the *Prefractured Bridge model* which starts with a damaged bridge model followed by Test 3 load. Both models are analyzed using the quasi-static analyses in *ABAQUS/Explicit* discussed in Section 5.5.4. Analysis duration and viscous pressure values used for Test 3 loading stage are the same as those used for the T_19sec_0.01%(p_{cd}) model in Section

5.5.4. The post-fracture capacity of the box-girder bridge showed particular sensitivity to boundary condition and shear stud models (Section 5.7). Therefore, the sensitivity of these parameters using simplified modelling approaches is also evaluated in this study.

5.8.1 *Quasi-static Fracture model*

The *Quasi-static Fracture model* simulation (Figure 5-56) starts with the construction sequence followed by gradual introduction of fracture in *ABAQUS/Explicit*. After the introduction of fracture over 5 secs., Test 3 loading is applied in three loading stages. Figure 5-57 shows the impact of bearing pad idealizations as described in Section 5.7.1 on the remaining load carrying capacity of twin box-girder bridge using *Quasi-static Fracture model*. The twin box-girder bridge model with *Pin-Roller Boundary model* and *Bearing Pad model* show linear response up to 250 kips and 200 kips, respectively, followed by rapid increase in girder deflection without significant increase in applied load as a result of failure of a large number of studs. The *Pin-Roller Boundary model* analysis failed with buckling of the brace in the interior intact girder at mid-span when the applied load reached 256 kips. Similarly, the *Stud Stiffness Only model* and the *Stud + Surrounding Concrete Stiffness model* for the initial tensile stud stiffness as described in Section 5.7.4 show linear response up to 252.4 kips and 200 kips (Figure 5-58) respectively followed by rapid increase in girder deflection leading to analysis failure.

The *Quasi-static Fracture models* use viscous pressure to reach a quasi-static solution in shorter time in *ABAQUS/Explicit*. The viscous pressure values are proportional to the velocity values in the model. As the deflection grew rapidly in both the bearing pad idealizations and the initial tensile stud stiffness model cases, the velocity in the finite element model increased resulting in to increase in viscous pressure values. Similar to the *Stud Stiffness Only model's* load vs. displacement response (Figure 5-39), the *Stud Stiffness Only model* recovered and successfully carried the Test 3 loading because of increased damping out of dynamic effects as a results of increased viscous pressure. In this case one cannot say with certainty that a real bridge would have shown similar recovery after sudden increase in deflection of 7.5 in. Therefore, results beyond this point are not considered to be valid.

Figure 5-59 shows the impact of different stud shear models as described in Section 5.7.4 on remaining load carrying capacity of the twin box-girder bridge using the *Quasi-static Fracture model*. The *Secant Stud Stiffness model* matches the load vs. displacement response exactly up to

100 kips. The *Secant Stud Stiffness model* and *Topakaya Force-slip model* yielded fairly linear response and display slight nonlinearity towards end of Test 3 loading.

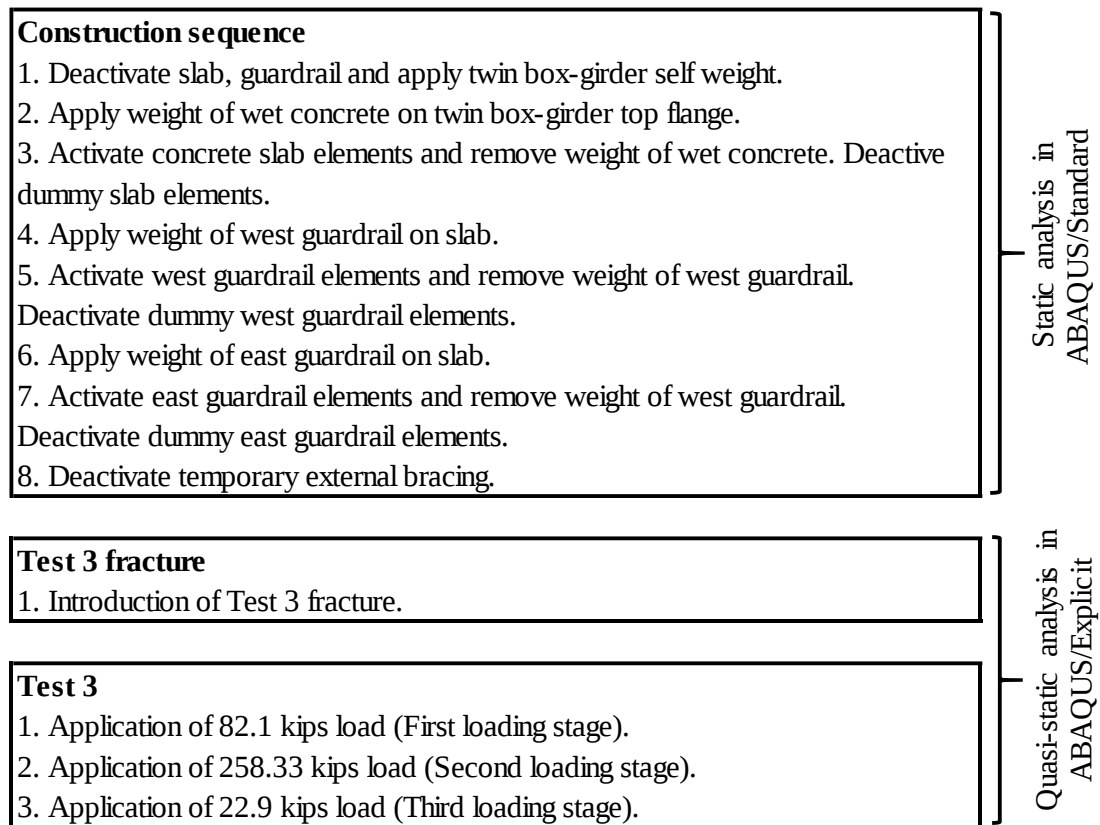


Figure 5-56: Test 3 simulation steps with quasi-static fracture introduction (*Quasi-static Fracture model*).

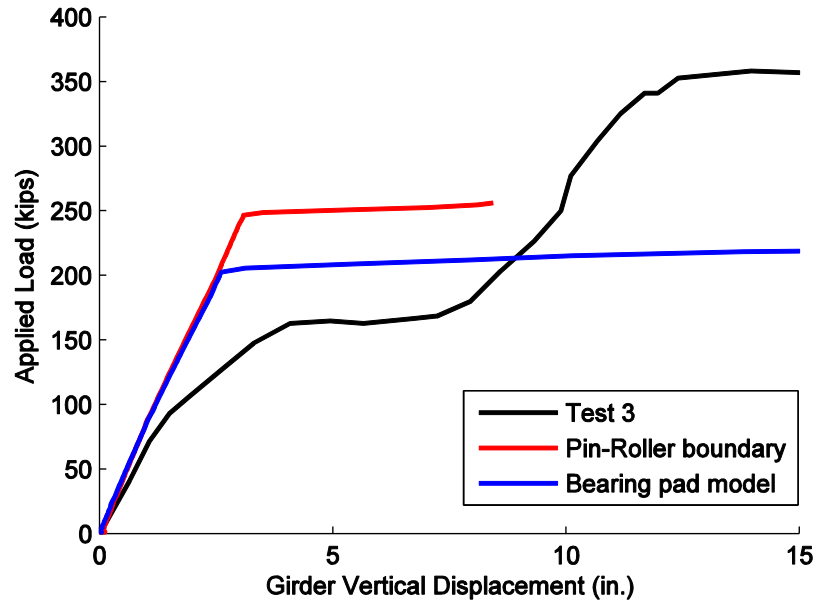


Figure 5-57: Pin-Roller boundary vs. Bearing pad model (18 ft. south of mid-span of fractured girder).

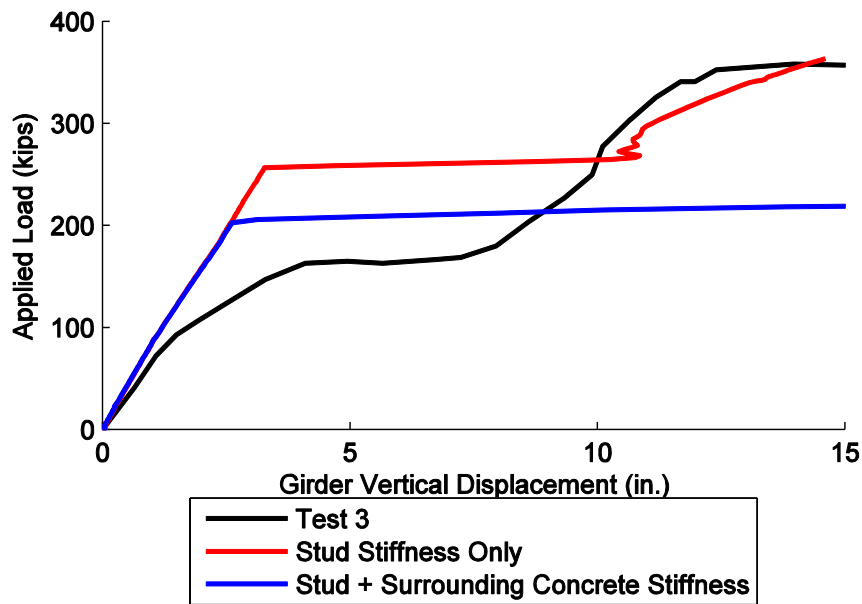


Figure 5-58: Test 3 response using different initial tensile stud stiffness (18 ft. south of mid-span of fractured girder).

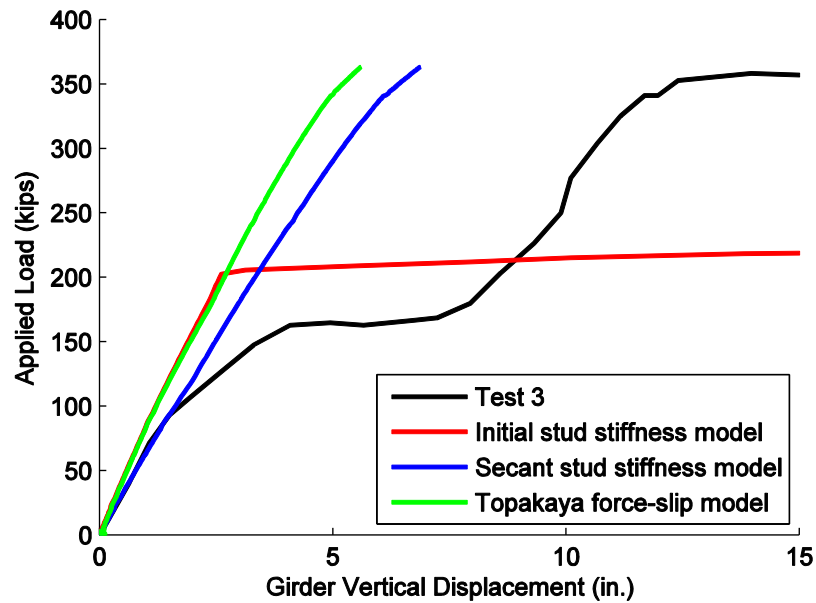


Figure 5-59: Impact of different shear stud shear models on Test 3 (18 ft. south of mid-span of fractured girder).

5.8.2 Prefractured Bridge model

The *Prefractured Bridge models* simulation (Figure 5-60) starts with a damaged bridge model with gradual application of dead load and Test 3 load in *ABAQUS/Explicit*. Figure 5-61 shows the impact of bearing pad idealizations as described in Section 5.7.1 on remaining load carrying capacity of the twin box-girder bridge using *Prefractured Bridge model*. The twin box-girder bridge model with *Pin-Roller Boundary model* and *Bearing Pad model* show linear response up to 152 kips and 169 kips respectively followed by rapid increase in girder deflection without significant increase in applied load as a result of failure of large number of studs. Similarly, the *Stud Stiffness Only model* and the *Stud + Surrounding Concrete Stiffness model* for the initial tensile stud stiffness as described in Section 5.7.4 show linear response up to 202 kips and 169 kips (Figure 5-62) respectively followed by rapid increase in girder deflection leading to analysis failure.

Figure 5-63 shows the impact of different stud shear models as described in Section 5.7.4 on the remaining load carrying capacity of the twin box-girder bridge using *Prefractured Bridge models*. The *Secant Stud Stiffness model* captured the overall structural response better compared to models analyzed as part of *Quasi-static Fracture model* and *Prefractured Bridge model*. The *Secant Stud Stiffness model* predicted slightly softer load vs. displacement response compared to

field response. The *Secant Stud Stiffness model* experienced rapid increase in deflection from 3.97 in. at total applied load of 187 kips to 7.38 in. at total applied load of 212 kips as a result of stud failure. This response is similar to the rapid increase in deflection observed during field testing when the total load on the bridge reached 161.5 kips (Neuman, 2009). The *Topakaya Force-slip model* yielded response similar to the *Secant Stud Stiffness model* but observed overall structural response is stiffer. The *Secant Stud Stiffness model* failed when the applied load on the bridge reached 346 kips. The *Force-slip model* successfully carried the Test 3 load of 363.3 kips.

Dead load 1. Application of bridge self weight to prefabricated bridge model.	Quasi-static analysis in ABAQUS/Explicit
Test 3 1. Application of 82.1 kips load (First loading stage). 2. Application of 258.33 kips load (Second loading stage). 3. Application of 22.9 kips load (Third loading stage).	

Figure 5-60: Test 3 simulation steps with *Prefracted Bridge Model*.

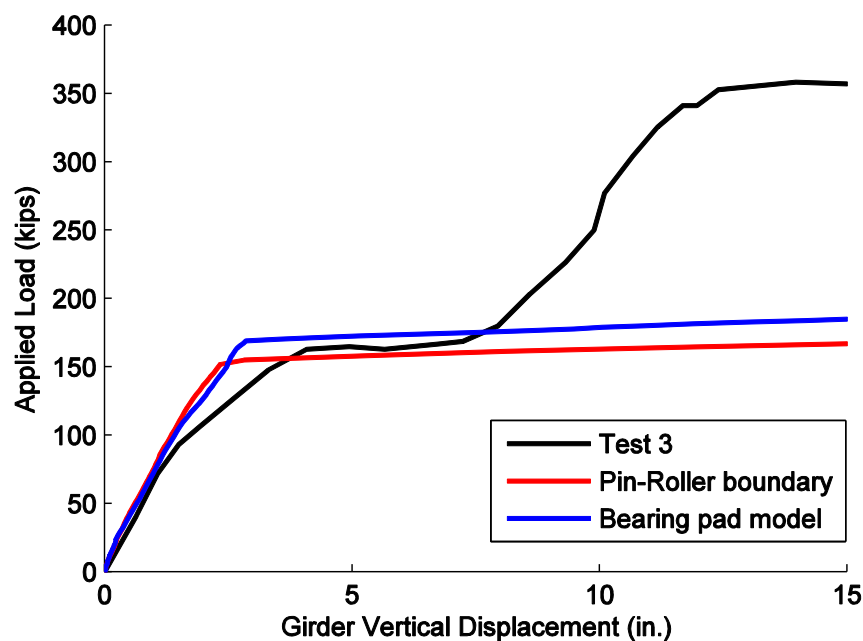


Figure 5-61: Pin-Roller boundary vs. Bearing pad model (18 ft. south of mid-span of fractured girder).

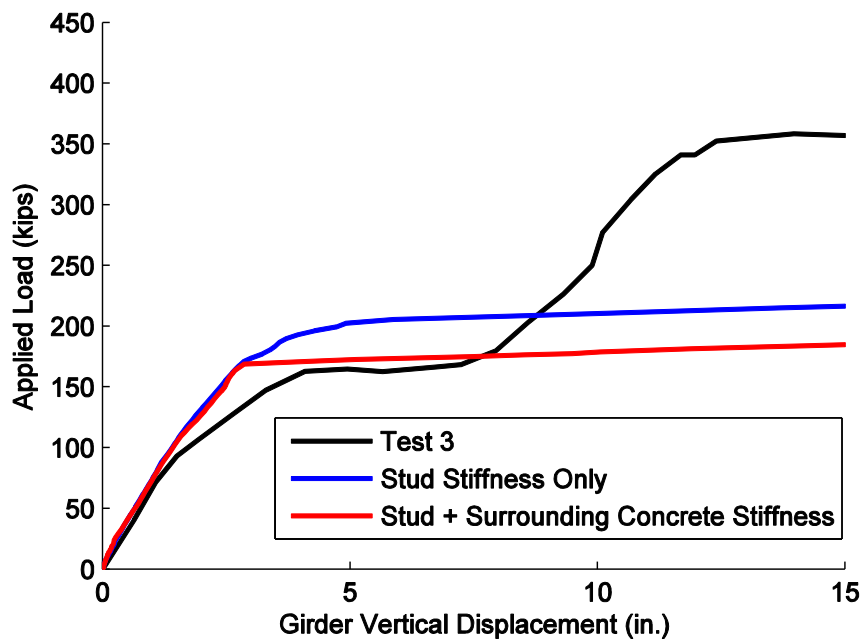


Figure 5-62: Test 3 response using different initial tensile stiffness (18 ft. south of mid-span of fractured girder).

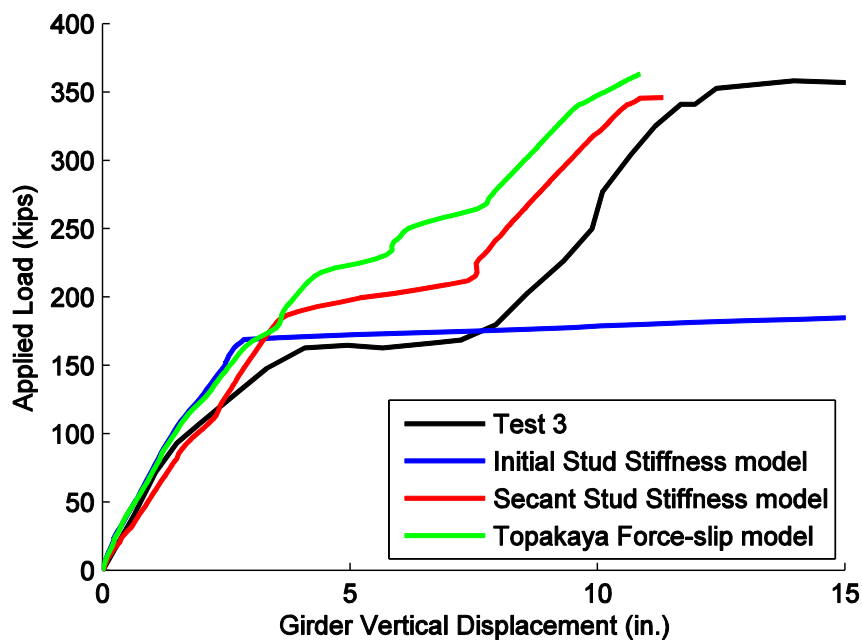


Figure 5-63: Impact of different shear stud shear models on Test 3 (18 ft. south of mid-span of fractured girder).

5.9 Shakedown or incremental collapse limit

Shakedown implies that after repeated application of a prescribed load history that exceeds the elastic limit but not the plastic collapse load of the structure, the residual deflection in the structure stabilizes (Leon and Flemming, 1997). If the difference between the shakedown limit and the collapse load for a bridge is small, then the design of the structure can be carried out using static analysis assumptions (Leon and Flemming, 1997). ABAQUS (2013) states that the kinematic hardening component used alone in case of nonlinear isotropic/kinematic hardening model is capable of predicting shakedown after one stress cycle. Therefore, the nonlinear isotropic/kinematic hardening model with kinematic hardening component alone is used to simulate plasticity in structural steel and reinforcement.

The *Secant Stud Stiffness model* captured the overall structural response better compared to the simplified models analyzed in Section 5.8. Therefore, *Prefracted Bridge model* with *Secant Stud Stiffness model* is selected for this study. The shakedown limit is evaluated by loading and unloading the bridge with fraction of Test 3 loading which is increased after every cycle. Figure 5-64 shows the comparison between the monotonic and cyclic loading. Monotonic and cyclic loading yielded identical response with both simulations failing when applied load reached ~346 kips. At each load increment during cyclic loading load is cycled only once. The shakedown analysis should be further evaluated by performing the analysis with multiple cycles at each load increment.

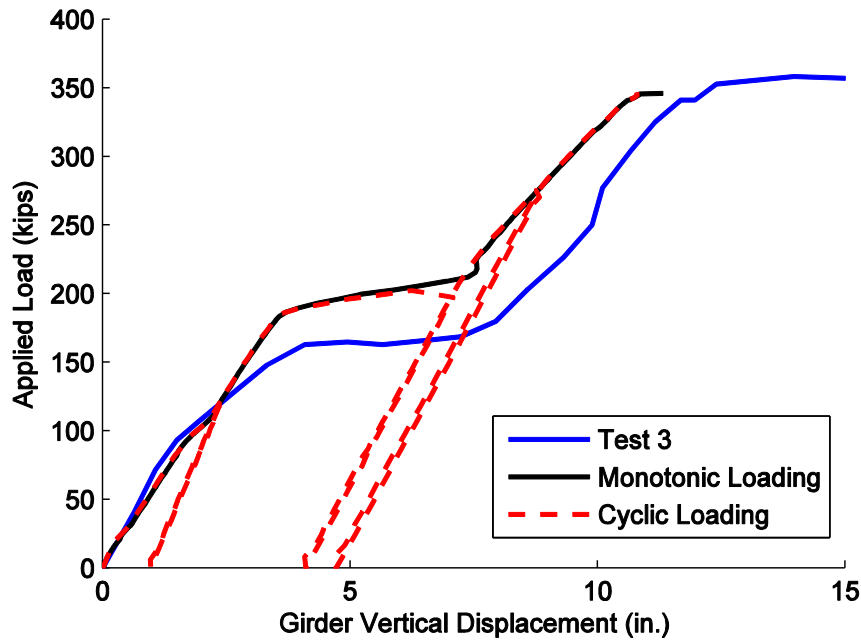


Figure 5-64: Comparison between monotonic and cyclic loading to evaluate shakedown limit.

5.10 Instantaneous fracture

At the start of Test 2, a web fracture was introduced by cutting the girder web while supporting it with scissor jack supports. The bridge was then loaded with five concrete girders (76 kips). The scissor jack support was removed by severing the tension tie to evaluate the capacity of the bridge to sustain large release of potential energy (Figure 5-6). This test does not truly represent an actual fracture event, therefore the finite element model is reanalyzed by introducing the fracture over the time it would take for the fracture to propagate assuming a fracture speed of 7,000 ft./s in structural steel (Barsom and Rolfe, 1999). The series of steps for *Instantaneous Test 2 Fracture* simulation are outlined in Figure 5-63. The extent of fracture in the *Fracture-I model* (Figure 5-33) is same as the fracture in Test 2 (*Test 2 Fracture*). The *Instantaneous Test 2 Fracture* simulation is analyzed with the *Initial Stud Stiffness model* and the *Secant Stud Stiffness model* outlined in section 5.7.4 to compare the post fracture dynamic response with the Test 2 response in section 5.7.4. Figure 5-64 shows the comparison between the post fracture responses with different stud models. The *Instantaneous Test 2 Fracture* simulation yielded lower exterior girder mid-span deflection compared to *Test 2 Fracture*. To further understand the reduction in deflection response, the stud damage pattern at the end of simulation is compared in Figure 5-65. The instantaneous introduction of fracture resulted in a reduction in the number of failed studs on

the interior flange of the exterior girder, while failing all the studs on the exterior flange of the exterior girder for the *Initial Stud Stiffness model* and failing large number of studs in case of *Secant Stud Stiffness model*. The number of failed studs on the interior intact girder stayed same irrespective of fracture introduction method.

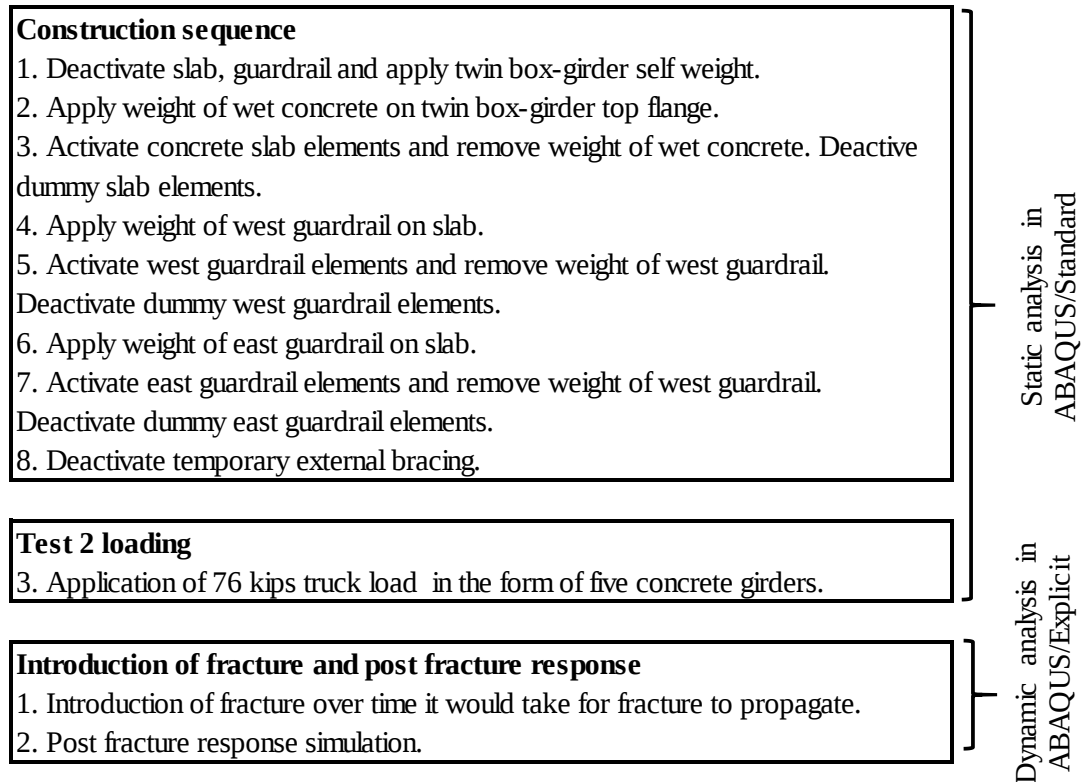
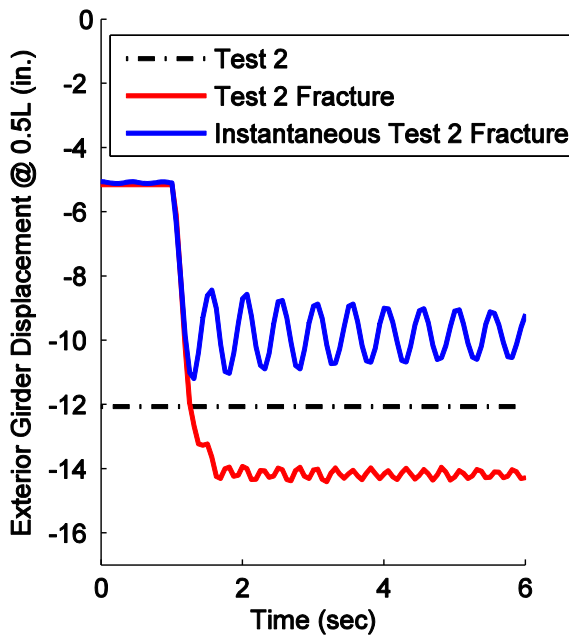
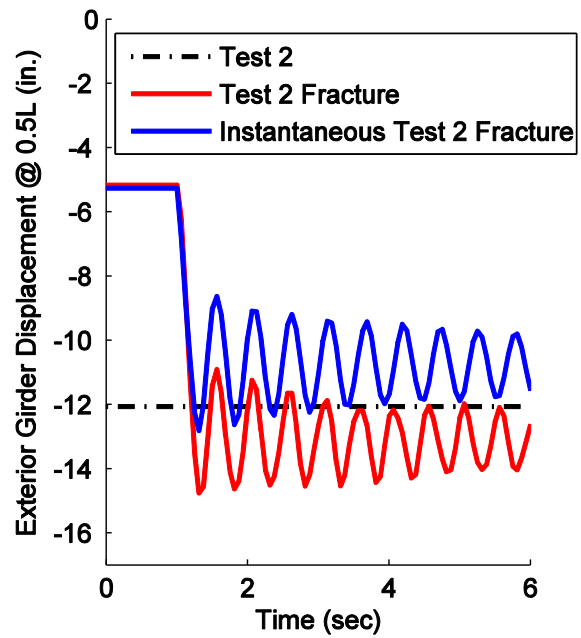


Figure 5-65: Instantaneous fracture simulation steps.



a) *Initial Stud Stiffness model*



b) *Secant Stud Stiffness model*

Figure 5-66: Post fracture response.

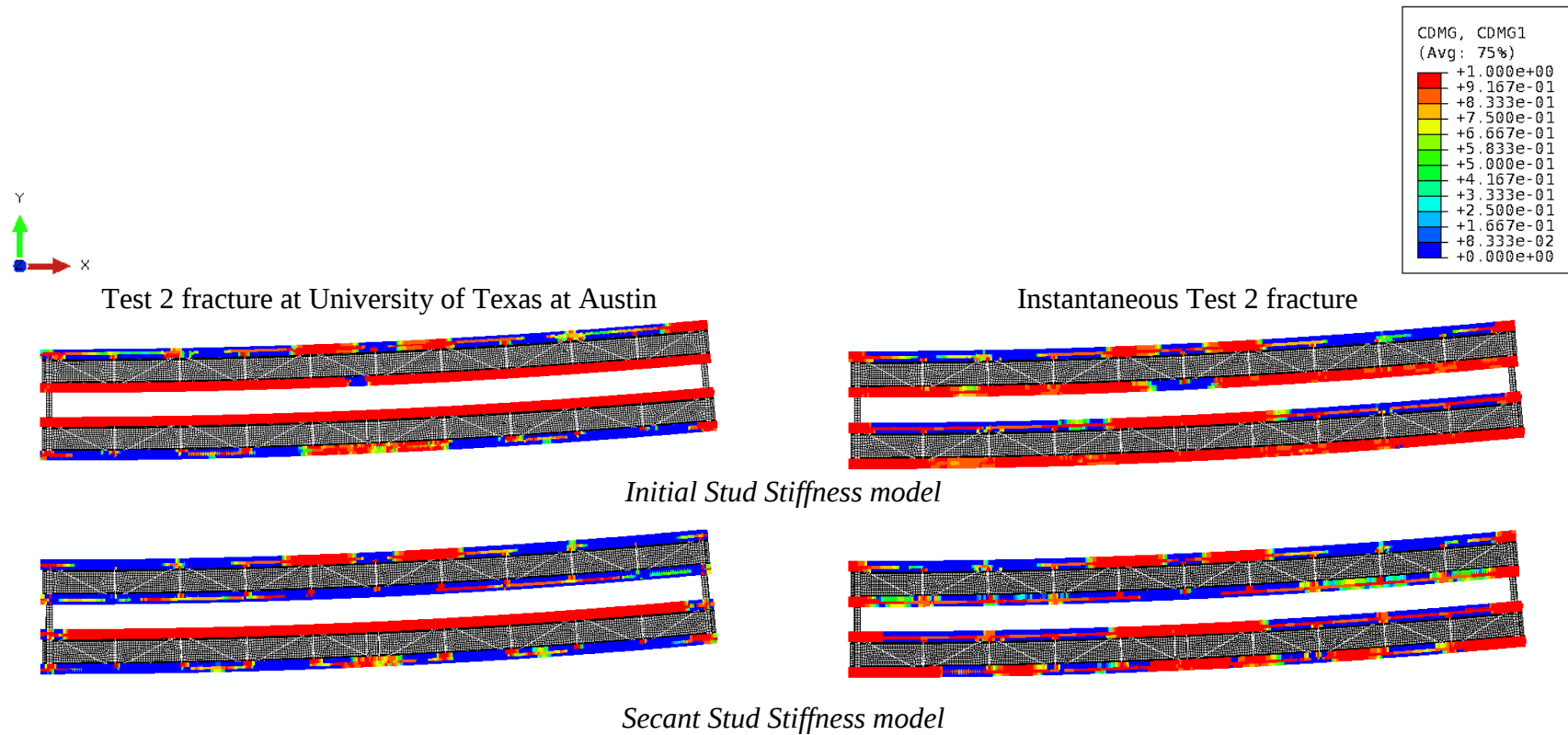


Figure 5-67: Connector overall damage variable (CDMG) comparison between *Fracture-II model* and Test 2 fracture with different stud models.

The modified stud failure pattern at the end of Test 2 alters the starting point for the remaining load carrying capacity evaluation in Test 3. In order to evaluate the impact of an instantaneous introduction of fracture on the remaining load carrying capacity of the bridge, the bridge model is reanalyzed with the *Fracture-II model* (Figure 5-33). The series of steps are outlined in Figure 5-66. During analysis the loading continued beyond the third loading stage by increasing the load applied outside the bin until the analysis stopped or the total load outside the bin reaches 122.9 kips. In addition to the *Fracture-II model*, the bridge model is reanalyzed with both a bottom flange and a full web depth fracture (*Fracture-III model*), which corresponds to fracture conditions near the end of Test 3 loading (Barnard, et al., 2010). The *Initial Stud Stiffness model* (Section 5.7.4) used to model the shear stud behavior in the *Fracture-II model* and the *Fracture-III model*.

Figure 5-67 shows the load vs. displacement response of the exterior girder at 18 ft. south of the fracture. The *Fracture-II model* and *Fracture-III model* yielded linear response up to 225 kips. The *Fracture-II model* successfully carried the Test 3 loading (363.3 kips) and the additional load of 100 kips. Meanwhile, the *Fracture-III model* failed when the total applied load on the bridge reached value of 442.51 kips. For the same applied load pattern, the *Fracture-II model* and the *Fracture-III model* yielded significantly different load vs. displacement response compared to the load vs. displacement response in section 5.6.4. The question remains whether the stiffer structural response can be only be attributed to the difference in stud failure pattern at the start of the analysis or it is a numerical phenomenon.

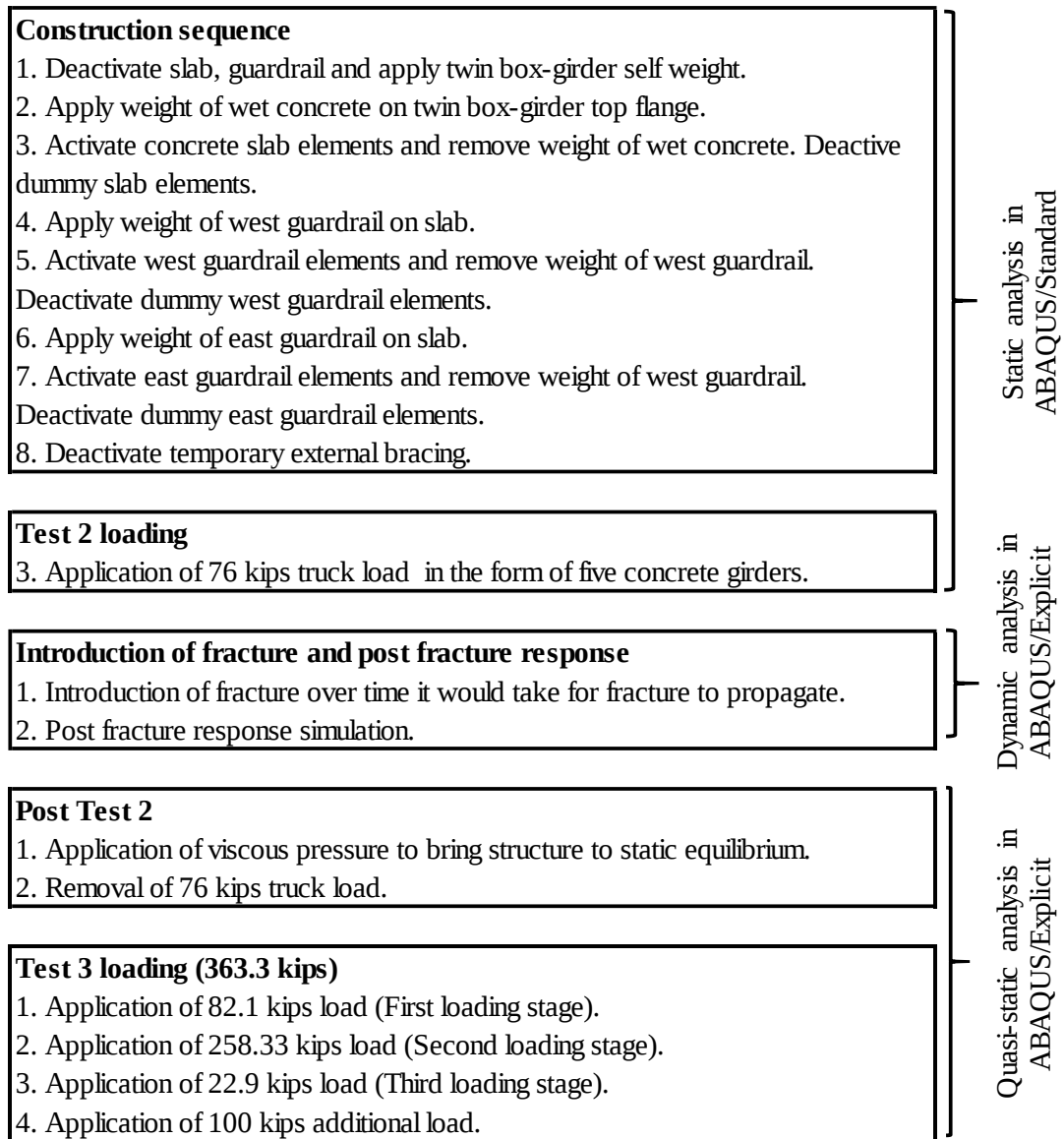


Figure 5-68: Simulation steps for remaining load carrying capacity evaluation following instantaneous introduction of fracture.

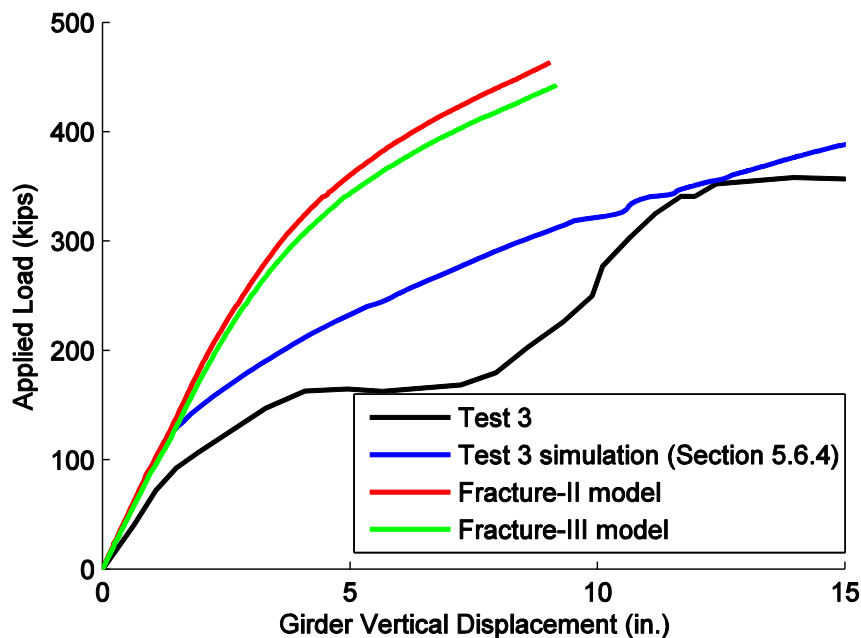


Figure 5-69: Impact of test sequence on load vs. displacement response.

5.11 Conclusion

Examination of all the results obtained so far lead to the following conclusions.

1. *The Implicit quasi-static analysis* successfully predicted the post fracture deflection response in Test 1 when compared to the field measurements and the mean deflection response from the dynamic analyses. Both analysis methods predicted about 6% stiffer response compared to the field measurement at mid-span of the exterior girder. After removal of the Test 1 load during testing and simulation, the bridge did not rebound to the position before the application of the Test 1 live load. This indicates presence of damage in the bridge following the bottom flange fracture although this did not result in an appreciable increase in deflection.
2. In Test 2, the mean deflection at mid-span from the dynamic analysis is 17.4 % higher compared to the field observation. The deflected shape of the fractured girder resembles a simply supported beam with an internal hinge at mid-span (Figure 5-27). The intact girder deflected shape resembles the deflected shape of a simply supported beam with the analysis predicting symmetric response about the mid-span while the field measurement showed higher deformation to the left of the mid-span. During early stages following release of

scissor jack, the concrete slab showed slight double curvature. In the following time increments, the shear studs on the interior flange of the exterior girder failed leading to single curvature and stud failure on the interior flange of the exterior girder. The finite element analysis predicted stud failure along the entire length of the interior flange of the exterior girder compared to the 30 ft. separation on either side of mid-span observed in the field. Additional stud failure resulted in larger mean deflection.

3. The Test 3 simulation predicted an ultimate capacity of 414.3 kips which is 14% higher than the total load (363.3 kips) on the bridge at the time of collapse. The simulation failed to reach a solution when the buckling on the brace was approached. During testing as total load on the bridge got close to the failure load of 363.3 kips, concrete crushing occurred near mid-span in the exterior guardrail. The two halves of the fractured girder rotated following failure over the exterior flange of the fractured girder until they came to rest at the fracture location on the bed of concrete blocks positioned for safety. Large difference in response results from failing to capture the failure mode in the finite element analysis. The present finite element model failed to capture the sudden separation at the interface at 161.5 kips. Instead a gradual softening in response is observed beyond 125 kips. One of the possible reasons for this behavior is the shear stud shear-tension interaction model used and the distributed stud stiffness at the steel-concrete interface compared to the discrete shear stud on the bridge. When the total load on the bridge reached 414.3 kips, the exterior girder deflected by 17.4 in. while the interior girder deflected by 9 in. Large difference in response of the interior and the exterior girder deflections indicates a lack of load redistribution from damaged to intact girder.
4. In Test 2, the *Pin-Roller Boundary model* predicted 1.5% lower mean deflection at mid-span of the exterior girder compared to the field observation while the *Bearing Pad model* resulted in 17.4% increase in mean deflection. In Test 3, both boundary conditions yielded similar initial response up to 125 kips. After that load, the *Pin-Roller Boundary model* yielded stiffer response. Use of pin-roller boundary condition to model bearing pad results in additional restraint as indicated by the stiffer response in Test 2 and Test 3 response.
5. Reduction in rotational stiffness at the steel-concrete interface resulted in increase in the peak-to-peak amplitude but the mean deflection stayed within 5% of the mean deflection with the rotational stiffness of 100,000 kip-in/rads.

6. Different shear stud models are evaluated to simulate stud tension and shear behavior. The *Stud + Surrounding Concrete Stiffness model* for the stud tension behavior better predicted the overall response when compared to the *Stud Stiffness Only model*. Out of four models used to simulate stud shear behavior, the *Topakaya Force-slip model* yielded mid-span deflection closest to the Test 2 response. While the *Initial Stud Stiffness model* better predicted the load vs. displacement response in Test 3 compared to other three models. The widely used rigid beam approach to model shear studs (*Rigid Stud model*) failed to capture the bridge response in Test 2 and Test 3. The choice of shear stud model significantly altered the bridge response.
7. When assessing the Hoan Bridge response, the use of rate dependent material properties in the Test 2 simulation resulted in a 4% reduction in the exterior girder deflection at mid-span. Part of the reason for the reduced sensitivity is the absence of rate dependent properties for shear studs in the twin box-girder bridge.
8. Similar response by both the isotropic hardening model and the nonlinear isotropic/kinematic hardening model with kinematic hardening component alone indicate an absence of stress reversal and therefore, no impact from the Bauschinger effect during Test 2.
9. The absence of a guardrail resulted in plasticity in concrete slab distributed over a larger area, engaging more studs and leading to stiffer response during simulation of both Test 2 and Test 3. Similarly, the exclusion of an additional cross-frame installed during testing also resulted in stiffer response. The question remains whether the stiffer structural response is real or numerical.
10. Application of concrete shrinkage strains to the bridge model as a negative temperature change resulted in increase of 0.87 in. (17% increase) in the total mid-span deflection. The shear studs offered partial restraint to the surrounding concrete introducing tensile stresses in the concrete. Tensile stresses from the restrained shrinkage in concrete near the shear studs will possibly reduce the shear stud connection strength.
11. Two simplified models (*Quasi-static Fracture models* and *Prefractured Bridge model*) that do not involve damage accumulation from Test 1 and Test 2 are used for Test 3 simulation. Both models are analyzed with the different shear stud models discussed in Section 5.7.4. Both simplified models with different shear stud models except the *Secant Stud Stiffness model* and the *Topakaya Force-slip model*, yielded linear response up to an applied load that

ranges from 150-250 kips followed by rapid increase in bridge deflection leading to analysis failure. The *Quasi-static Fracture model* with the *Secant Stud Stiffness model* and the *Topakaya Force-slip model* yielded linear response and displayed slight nonlinearity towards the end of Test 3 loading. The *Prefractured Bridge model* with the *Secant Stud Stiffness model* resulted in a response that matched closely with the field load vs. displacement response. The *Topakaya Force-slip* yielded stiffer response compared to the *Secant Stud Stiffness model*. Based only on the twin box-girder results, it difficult to select between the simplified modelling approaches with any level of confidence.

12. During the evaluation of shakedown limit, the Test 3 loading is applied cyclically to the *Prefractured Bridge model* with the *Secant Stud Stiffness model*. Cyclic loading yielded load-displacement response envelope that matched with the monotonic loading case indicating stabilization of residual deflection. At each load increment during cyclic loading, the load is cycled only once. The shakedown analysis should be further evaluated by performing the analysis with multiple cycles at each load increment.
13. Instantaneous introduction of fracture altered the stud failure pattern compared to the Test 2 simulation, resulting in lower mid-span deflection in the fractured girder. It also altered the starting point for the remaining load carrying capacity evaluation similar to Test 3. The *Fracture-II model* and *Fracture-III model* yielded stiffer response compared to Test 3 simulation response in Section 5.6.4.

6 Fracture-Critical System Analysis for Steel Bridges

This chapter proposes a methodology for system redundancy analysis of some fracture critical bridges, specifically three girder bridges and twin tub girder bridges. The methodology is based primarily on advanced finite element analyses. This chapter begins with a brief overview of methodologies used by researchers and practicing engineers for redundancy evaluation of fracture critical bridges (Section 6.1). Section 6.2 outlines the proposed modeling guidelines for system redundancy analysis of fracture critical bridges. Section 6.3 provides details about the system redundancy analysis methodology proposed as part of this research. The proposed methodology is evaluated through analysis of both the Hoan Bridge (Fisher, et al., 2001) and twin box-girder bridge tested at The University of Texas at Austin (Barnard, et al., 2010) in Section 6.4.

6.1 Literature review

Table 6-1 provides a brief overview of some of the models used by researchers and practicing engineers for redundancy evaluation of fracture critical bridges. These models were described in the literature review in Section 2.2.

Table 6-1: Literature overview

	Bridge type	Model description	Shear stud model
Marquette Interchange Project (Elza, et al., 2004)	Twin box-girder bridge	- Focused on the evaluation of bridge capacity to withstand fracture and the remaining load carrying capacity of the damaged bridge through finite element analysis.	Rigid studs
		- Redundancy evaluation methodology based on NCHRP Report 406.	
Hunley and Harik (2011)	Twin box-girder bridge	- Focused on the evaluation of the remaining load carrying capacity of the damaged bridge through finite element analysis.	Rigid studs
		- Redundancy evaluation methodology based on NCHRP Report 406.	
		- Evaluated impact of external bracing on the bridge redundancy.	
Park, et al. (2007, 20012)	Two girder bridge	- Focused on the evaluation of remaining load carrying capacity of the damaged bridge through testing bridge sepecimen and finite element analysis.	Not specified
		- Redundancy evaluation methodology based on NCHRP Report 406.	
		- Evaluated impact of lateral bracing on the bridge redundancy.	
Barnard et al. (2010)	Twin box-girder bridge	- Focused on the evaluation of bridge capacity to withstand fracture and the remaining load carrying capacity of the damaged bridge through full scale testing and finite element analysis.	Stud shear and tension model based on experimental data by Topakaya (2004), Sutton (2007) and Mouras (2008)
Lin et al. (2013)	Two girder bridge	- Focused on the evaluation of remaining load carrying capacity of the damaged bridge through finite element analysis.	Stud shear model by Ollgaard et al. (1971) .
		- Redundancy evaluation methodology based on NCHRP Report 406.	

6.2 Modeling guidelines for system redundancy analysis

With regard to the level of refined analysis required to demonstrate redundancy AASHTO (2012) Section 6.6.2 commentary states that:

“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.”

This section proposes modeling guidelines for system redundancy analysis of fracture critical bridges through advanced finite element analysis that aim to satisfy the AASHTO criteria above. The finite element models of the Hoan Bridge (Fisher, et al., 2001) in Chapter 4 and the twin box-girder bridge (Barnard, et al., 2010) in Chapter 5 did not fully capture the response of the structures. In the case of the Hoan Bridge, only one gross field measurement was available, and thus there was little guidance on how the model could be improved. Visual comparisons indicated that the analysis and field evidence showed similar deformation patterns but wide quantitative differences. In the case of the UT bridge, the sensitivity of the solution to the stud model, and in particular the analytical process based on a series of small steps within which equilibrium was enforced, resulted in a progressive rather than sudden unzipping of the shear connection in Test 3. The proposed modelling guidelines are in early stages of its development and there are areas that need further research before the proposed guidelines are comprehensive.

6.2.1 Evaluation of bridge capacity to withstand fracture

Bridge response following a fracture event is dynamic and can be simulated using nonlinear dynamic analysis. Nonlinear dynamic analysis requires detailed finite element models and considerable computational resources and is not feasible within typical bridge evaluation projects. Therefore, the suitability of simplified linear and nonlinear static models with dynamic load amplification factor was evaluated as part of the Hoan Bridge study in Chapter 4. However, at this point there is insufficient field and analytical data available to calibrate the dynamic load

amplification factors that can be used to amplify dead load and live load on the bridge to capture the demands on members and connections following fracture. As the possibility of using a value of dynamic load factor needs further evaluation, at this point only nonlinear dynamic analysis is recommended to evaluate bridge capacity after fracture.

6.2.2 Evaluation of remaining load carrying capacity of the damaged bridge

If a bridge shows capacity to withstand fracture, then it is necessary to evaluate the reserve load carrying capacity of the damaged bridge because it may stay in service before it is closed down. Localized structural damage following fracture can alter the overall bridge response in unexpected ways. Under that scenario, it is necessary to include deformed geometry and extant stress distributions as initial conditions for reserve load carrying capacity evaluation of the damaged bridge.

In the analysis of twin box-girder bridge tested at The University of Texas at Austin (Chapter 5), three models with decreasing order of complexity were evaluated. In the first model, damage following the fracture event (Test 2) is taken as the initial condition for the remaining load carrying capacity analysis (Test 3). This approach although comprehensive can be fairly time consuming. Therefore, two simplified approaches that do not involve damage accumulation were also evaluated. In the first simplified approach (*Quasi-static Fracture model*), the fracture is introduced quasi-statically at the start of the analysis followed by loading to evaluate the remaining load carrying capacity. In the second simplified approach (*Prefractured Bridge model*), the analysis starts with a damaged bridge model, and is followed by loading to evaluate remaining load carrying capacity. In the case of the Test 3 simulation for the twin box-girder bridge, the *Prefractured Bridge model* resulted in response that matched closely with the field load vs. displacement response. Based only on the twin box-girder results it difficult to select between the two simplified modeling approaches with any level of confidence. This choice is complicated by the choice of shear stud model (Section 5.7.4 and Section 5.8), which significantly altered the predicted bridge response. Therefore, the suitability of any simplified modelling approach cannot be determined without first establishing a stud model that is representative of the actual stud behavior. The DOT and its consultants need to decide on the choice of model for this analysis. At this point only nonlinear finite element model that accounts

for the damage following the fracture event as the initial condition for the remaining load carrying capacity evaluation of the damaged bridge is recommended.

6.2.3 Element selection

In the finite element analysis, judicious choice of both elements type and mesh size is required to ensure the accuracy of results. The choice of element type used to model each bridge component and mesh density should be agreed upon by the DOT and its consultants. Element types used to model different bridge components in the Hoan Bridge (Fisher, et al., 2001) and the twin box-girder bridge (Barnard, et al., 2010) analysis are tabulated in Table 6-2.

Based on the recommendations for mesh size for modelling of steel girders by White et al. (1997), main girders and box-girder webs are modeled with at least ten elements through the depth and four elements across the width of flange. The bottom flange of the box-girder should be treated similarly to the web and is meshed with at least ten elements through the width. In the Hoan Bridge model, floor beams and stringers are meshed with four elements through the web depth and two elements across the width of flange. The lower lateral bracing and cross bracing components are modeled with at least five beam elements along the length to capture the deformed or buckled shape with reasonable level of accuracy. The concrete slab is meshed with an average mesh size of 8 in. in the fractured span. In the remaining spans and overhang, the average mesh size in the concrete slab is gradually increased to 24 in. away from the fractured span. The guardrail is meshed with an average mesh size of 24 in. The guardrail mesh is further refined to an average mesh size of 8 in. over the fractured region and the interior pier (0.1L on either side) to capture concrete plasticity. In the twin box-girder model concrete slab, guardrails, the lateral bracing and cross bracing components are meshed with an average mesh size of 6 in. Mesh sizes and element type for different bridge components are provided here as an example only. Mesh density and element type need a case by case evaluation, particularly if geometric or stiffness irregularities exist. In the analysis of the Hoan Bridge and the twin box-girder bridge sophisticated finite element models were developed to evaluate the post fracture dynamic response and the reserve load carrying capacity of the damaged bridge under multiple fracture scenarios. Reduction in mesh density and simplification of the finite element representation of the bridge components away from the failure zone is possible and should be done. This requires some care and parametric studies of mesh density.

Table 6-2: Element type

Bridge component	Element type
Main girders, floor beams, stringers	Shell elements
Concrete slab	Shell elements with reinforcement modeled as smeared layers
Guardrail	Solid elements
Lateral bracing	Beam elements
Cross bracing components	Beam elements

6.2.4 Modeling of composite action

6.2.4.1 Shear stud modeling

In the event of a composite beam or box girder fracture, the shear studs are subjected to combination of shear and tension forces. The popular rigid stud approach used to model bridge composite action fails to capture the potential unzipping of shear studs that might occur following a fracture and lead to a false assessment of the bridge safety. There is very limited data available about both combined shear and tension stud response and the impact of haunch height on the failure mode. ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004) provide interaction equations (Section 3.2.4.2) for combined shear-tension strength check for stud group in concrete member without a haunch. The ACI 318-11 (2011) interaction equation is a trilinear approximation of the PCI Design Handbook, 6th Edition (2004) interaction equations. At the present time, PCI Design Handbook, 6th Edition (2004) interaction equations provide a good starting point to model the shear stud failure envelope. In the absence of data on shear stud behavior under combined shear and tension before failure, a variety of stud models were evaluated as part of the twin box-girder bridge parametric study but no particular stud model emerged as a clear favorite. Also, even if the studs fail in tension because of failure of the surrounding concrete, the interface still has capacity to transfer shear through interface friction and interlock. None of the models evaluated as part of the twin box-girder bridge parametric study address these other mechanisms. Based on the experience developed in this project, either the *Initial Stud Stiffness model* or the *Secant Stud Stiffness model* described in Section 5.7.4 provide a good starting point to model shear stud behavior in combined shear and tension.

6.2.4.2 *Non-composite interaction*

Typically during design of non-composite bridges, the concrete slab is excluded from the calculations as the degree of interaction at ultimate cannot be determined. Excluding the slab during system redundancy analysis eliminates the slab's capacity to redistribute load following a fracture event. Exclusion of the concrete slab from the analysis will not necessarily always lead to conservative results in a post fracture analysis because the absence of the slab might significantly alter the load redistribution path and fail to capture the demand on members and connections that would be part of the actual load redistribution path. Therefore, a realistic representation of the interface behavior is required to capture the bridge response following fracture. The interface behavior for non-composite bridges can be included through use of frictional contact definition at the steel-concrete interface.

6.2.5 Secondary members

In the bridge response following a fracture event, the contribution of the secondary elements (guardrail, cross frames, lateral bracing) to the strength and stability of the bridge superstructure can be very significant. It might seem a conservative approach to disregard secondary members when performing a redundancy analysis. But whether or not these members are included in the model, they will still attract force during a fracture event on an actual bridge. For example, exclusion of lower lateral bracing in the Hoan Bridge model resulted in rapid increase in deflection beyond realistic values (>10 ft.) resulting into significant yielding and buckling in the fractured girders E and F. With the absence of the lower lateral bracing, the load redistributed to the intact girder D reduced, which in turn, reduced its displacement. Therefore, it is necessary to include secondary members in the finite element model with appropriate connection models. It is critical to assess the connection capacity to carry forces, which might be higher than the forces connection was designed for.

6.2.6 Damping

In a linear dynamic analysis, 3-5% Rayleigh damping can be used to account for sources of damping (hysteretic damping/energy dissipation from material yielding, coulomb damping/energy dissipation from friction at steel-concrete interface, energy dissipated at the connections) that are not explicitly modeled in the analysis.

In a non-linear dynamic analysis, 3-5% Rayleigh damping would yield overdamped results. Previous work by Hall (2006) and Charney (2008) and the results from the Hoan Bridge analysis in Chapter 4 clearly show that the stiffness proportional coefficient in Rayleigh damping tends to unrealistically overdamp higher modes. On the other hand, the mass proportional coefficient in Rayleigh damping conservatively underdamps higher modes. However, mass proportional damping generates extraneous damping forces if rigid body motion is present (Hall, 2006) which can alter the post fracture bridge response. Therefore, damping values and damping model should be carefully selected during nonlinear dynamic analysis. Based on the analyses of these two bridges a critical damping ratio of less than 2% is recommended for non-linear dynamic analysis.

6.2.7 Boundary conditions

Correct representation of the bridge boundary conditions is important to capture the overall bridge response. If bridge girders are supported using elastomeric bearing pads then idealizing support condition using pin-roller boundary condition can lead unrealistic structural response. The bearing pads can be idealized as elastic springs with stiffness values based on the equations used for design of elastomeric bearing pads from Section 14.6.3.1-2 of AASTHO specification (2012). The need to include pier flexibility in the modeling of boundary conditions should be evaluated on a case by case basis. Additionally, boundary condition should be updated based on the field inspection reports to account for changes in restraint condition.

6.2.8 Material properties

Expected material properties should be used in the analysis instead of nominal material properties to capture realistic structural response and correct demand on the connections that form part of the load redistribution path. Structural elements subjected to dynamic effects exhibit higher strength than similar elements under static loading. This increase in strength results from increased material strength under rapid loading rates. UFC 3-340-02 (2008) provides rate dependent properties for structural steel and reinforced concrete. In the absence of bridge specific data, rate dependent properties from UFC 3-340-02 (2008) provide a good starting point in simulation of dynamic response following fracture event. To determine the actual material properties test coupons and cores should be taken from the bridge and tested.

6.2.9 Determination of fracture location

Selected fracture locations in the primary load carrying members should not be limited to the areas of high bending moment or shear force. Fracture locations also need to be selected based on the presence of known fracture critical details on the bridge, connection details, presence of initial defects and the current bridge condition. If there is probability of fracture propagation to other members then multiple fracture scenarios should be considered in the simulation.

6.3 System redundancy analysis methodology

AASHTO LRFD guidelines (AASHTO, 2012) defines redundancy as “ the quality of a bridge that enables it to perform its design function in a damaged state.” AASHTO (2012) recognizes three types of redundancies:

1. Load path redundancy
2. Structural redundancy
3. Internal member redundancy

Load path redundancy is based on the number of main supporting members such as girders or trusses between supports. Structural redundancy is based on the continuity in main members over interior supports or other load redistribution mechanisms. Internal member redundancy is based on the presence of alternate load path within the fractured member itself to limit fracture propagation across the entire member cross section.

On June 20, 2012 Federal Highway Administration (FHWA) published a memo with the subject “Clarification of Requirements for Fracture Critical Members” to provide clarification of the FHWA policy for the classification of fracture critical members. For the first time structural redundancy demonstrated through refined analysis is formally recognized and may also be used for identification of fracture critical members for in-service inspection protocol. This clarification does not recognize redundancy from internal built-up details to affect the classification of fracture critical members. Therefore, it requires totally fractured fracture critical member for load path redundancy evaluation. This clarification states that “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.”

This section proposes guidelines for the system redundancy analysis of fracture critical bridges based on both the lessons learned through the analysis of Hoan Bridge and twin box-girder bridge in previous chapters and the memo published by FHWA. The proposed guidelines will be used to reclassify fracture critical members as non-fracture critical for in-service inspection through advanced finite element analysis. The proposed methodology is in early stages of its development and there are areas that need further research before the proposed methodology is ready for use. The proposed methodology is described in the flowchart in Figure 6-1 and Figure 6-2. The first step is to screen bridges that can be evaluated using system redundancy analysis guidelines. If the bridge under consideration fails to satisfy the screening criteria then a decision needs to be made by DOT whether to proceed with the system redundancy analysis. Repairs or retrofits can be carried out on the bridge to satisfy the screening criteria's that are not satisfied during the preliminary screening. The second step is to perform system redundancy analysis to answer following questions:

- Will the bridge be able to withstand fracture at critical locations?
- If there is a potential to trigger fracture at other locations following or relatively soon after the first fracture occurs, does the bridge have the capacity to withstand those multiple fractures?
- If the deflection following fracture is perceivable to users, does the damaged bridge have the capacity to sustain traffic loads until it is closed for traffic?
- If the deflection following fracture is not perceivable to users, does the damaged bridge have the capacity to sustain traffic loads until next inspection?

6.3.1 Bridge screening guidelines

Bridge screening guidelines based on the material toughness, known fracture critical details on the bridge, presence of initial defects and the current bridge condition that includes but not limited to corrosion and fatigue damage to the bridge structure, repairs or modifications (for example, addition of overlay, slab replacement) done to the bridge structures, modification in travel lanes from the original design, frozen supports and expansion joints, damage from the accidents on the top or bottom of the bridge (for example, vehicle crashing in to the bridge pier

or steel superstructure), fatigue cracks, low condition rating of bridge superstructure should be agreed upon by the DOT and its consultants. Bridges that fail to meet agreed upon screening guidelines would not be eligible for a system redundancy analysis. Repairs or retrofits can be carried out on the bridge to satisfy the screening criteria that are not satisfied during the preliminary screening.

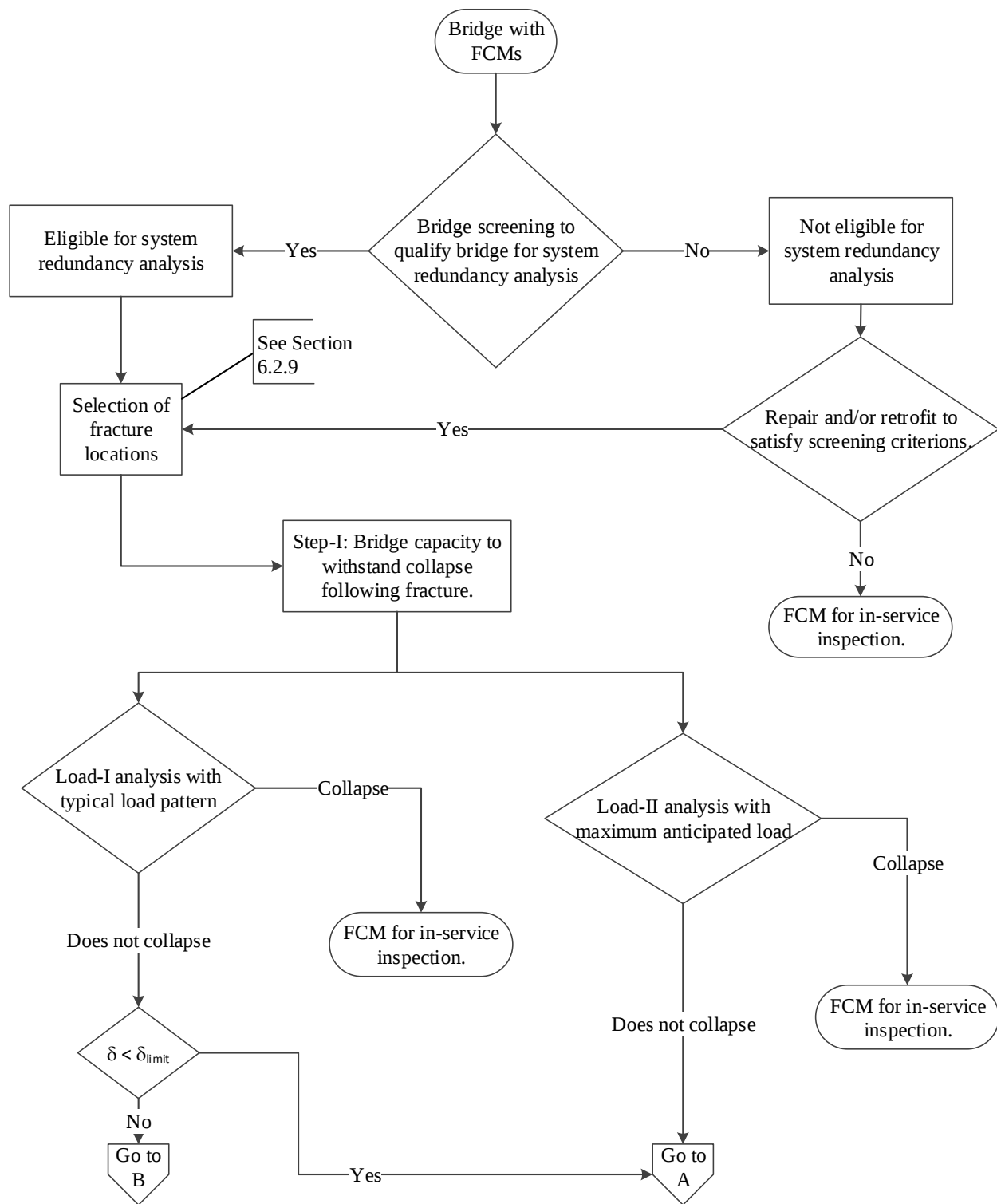


Figure 6-1: Flowchart for system redundancy analysis methodology (Part I).

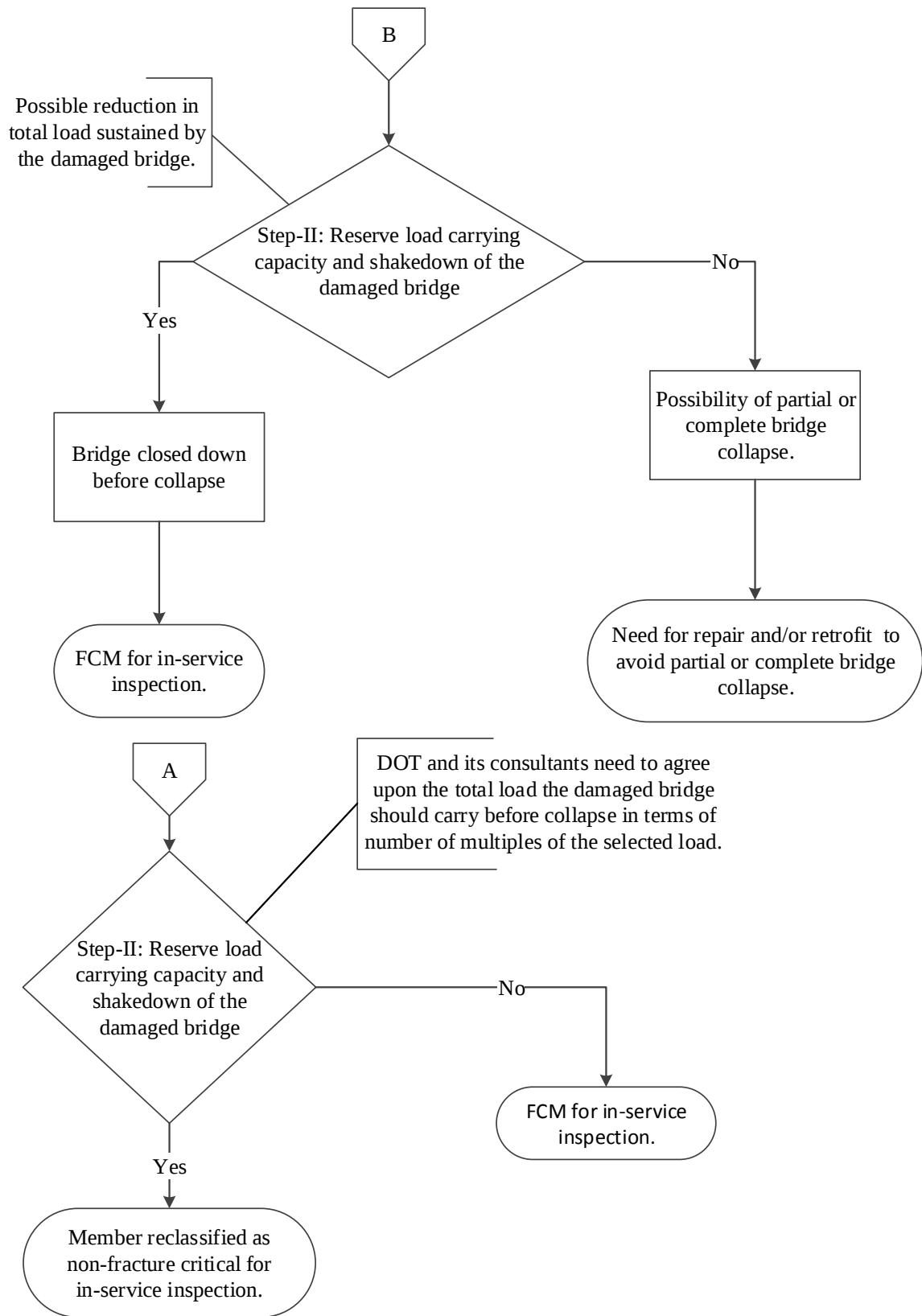


Figure 6-2: Flowchart for system redundancy analysis methodology (Part II).

6.3.2 System redundancy analysis methodology

The uncertainty associated with possibility of fracture, and estimating strength and loading on the bridge use of a probabilistic approach is required to account for uncertainties. However, at this point we simply do not have enough field and simulation data to fully develop probabilistic guidelines for system redundancy analysis of fracture critical bridges. Therefore, a deterministic approach for system redundancy analysis of fracture critical bridges is proposed in this section. If the bridge satisfies the screening criteria then bridge is eligible to be evaluated using the system redundancy analysis methodology outlined in this section. The DOT and its consultants should set the screening criteria.

6.3.2.1 Step-I: Bridge capacity to withstand collapse following fracture

The objective of the first step is not only to evaluate bridge capacity to withstand collapse following fracture but also to evaluate whether the residual deformation is perceivable to people on or off the bridge. This evaluation is performed in two steps:

- a) *Load-I*: In *Load-I*, the bridge capacity to withstand collapse following fracture is evaluated with a live load that is representative of the typical load pattern on the bridge. The live load can be determined based on the weigh-in motion data.
- b) *Load-II*: If the bridge satisfies the *Load-I* requirements, then the bridge capacity to withstand collapse following fracture is evaluated with either the maximum anticipated load on the bridge or the full HL-93 load on the bridge.

The residual deflection from *Load-I* and *Load-II* analyses should be compared against the deflection limits agreed upon by the DOT and its consultants. The limits should be set such that the bridge is closed within few hours following fracture as the deflection should be noticed by drivers as a “bump” or be visible to pedestrians nearby. The deflection limit of (span length/100) proposed by Ghosn and Moses (1998) as part of NCHRP 406 report can be used. If the residual deflection is lower than the deflection limit then the fracture might go unnoticed and the damaged bridge may or will stay in service until the next inspection. If the fracture is noticed before the next inspection, then the bridge should be repaired. Authorities can use analysis with similar level of detail to load rate and post the damaged bridge. If the residual deflection exceeds the deflection limit then, it is expected that the bridge will be closed down before collapse.

It is important to perform the analyses in two steps because *Load-I* checks the bridge capacity to withstand collapse under a typical loading while *Load-II* evaluates the bridge capacity to withstand collapse under the maximum anticipated load. For example, a twin box-girder bridge can form a part of an exit ramp. In the event of a traffic jam or heavy traffic conditions on the exit ramp, the load on the bridge can be significantly higher compared to the loading used in the *Load-I* analysis, making the *Load-II* analysis very important.

If the bridge fails to satisfy the deflection limit in the *Load-I* analysis, then it is expected that the bridge will be closed down before collapse and the member under consideration cannot be reclassified as non-fracture critical for in service inspection. If the bridge fails to satisfy the deflection limit in the *Load-II* analysis, it is still necessary to proceed with the *Step-II* analyses as the bridge might not experience the maximum anticipated load used during the *Load-II* analysis before the next inspection. If the bridge collapses in the *Load-II* analysis, then the member under consideration cannot be reclassified as non-fracture critical for in service inspection. If there is the possibility of fracture propagation to other members, then multiple fracture scenarios and fracture sequence scenarios needs to be considered in the *Load-I* and the *Load-II* analysis.

The viability of the secondary load paths needs to be evaluated based on the load redistribution following fracture. The capacity of structural connections to carry forces following fracture needs to be evaluated as force transfer though the connection changes as it deforms. For example, a brace connection that is dominated by either tension or compression before fracture can change to a connection that is dominated by tension/compression and flexure. Complicated geometric and material effects are difficult to capture using simplified analysis and will require advanced analysis to capture the correct connection behavior (Dusenberry, 2010). Additionally, a fracture triggered at the connection has the potential to propagate into the girder and subsequently cause multiple member failure.

6.3.2.2 Step-II: Reserve load carrying capacity and shakedown of the damaged bridge

If the bridge satisfies the *Step-I* requirements, then the reserve load carrying capacity of the damaged bridge needs to be evaluated. The fatigue life of the damaged bridge components needs to be evaluated using the stress ranges based on the damaged bridge configuration. The possibility of structural shakedown from subsequent load cycles also needs to be evaluated. The DOT and its consultants needs to agree upon the total load the damaged bridge should carry

before collapse in terms of the number of multiples of the selected load to reclassify the member under consideration as non-fracture critical for in-service inspection. If the *Load-I* analysis in *Step-I* yields perceivable bridge deflection (i.e. it fails to satisfy the deflection limit set in *Step-I*), then it is expected that bridge will be closed down before collapse. In this case, maximum load to be sustained during reserve load carrying capacity evaluation can be reduced in agreement with the DOT and its consultants. It is difficult to estimate the live load bridge might see before it is closed. The anticipated load can be based on the weigh-in motion data and/or engineering judgment.

In the present AASHTO LRFD provisions (2012), the dynamic effects produced by the live load is captured by increasing the static live load with a dynamic load allowance (*IM*). The AASHTO LRFD provisions (2012) use a dynamic load allowance of 33% for all limit states except fatigue and fracture limit state, which use a dynamic load allowance of 15%. In the case of a damaged bridge, the value of dynamic load allowance required to capture dynamic effects of the live load might increase as a result of softening of the bridge structure following fracture. At this point no experimental or analytical data is available in the literature to assess the change in dynamic load allowance factor for a damaged bridge. Therefore, dynamic load allowance of 33% should be used with caution in reserve load carrying capacity evaluation. Secondary load path viability also needs to be evaluated in *Step-II* analyses.

6.4 Bridge examples

The system redundancy methodology proposed in Section 6.3 is evaluated through analysis of Hoan Bridge (Fisher, et al., 2001) and twin box-girder bridge tested at The University of Texas at Austin (Barnard, et al., 2010).

6.4.1 Hoan Bridge

The Hoan Bridge is located on I-794 over the Milwaukee River in the city of Milwaukee, Wisconsin. The bridge was opened for traffic in 1974 and carried three lanes of traffic in each direction to and from the city. The main span is a three span tied arch crossing the river. The main span has three girder approach spans on both sides of arch. On December 15th, 2000, bottom flange and full web depth fractures were initiated in two of three girders and partial depth web fracture at three locations in the third girder. Multiple fractures occurred because of constraint induced fracture detail at the lower lateral brace to web connection. The Hoan Bridge

did not collapse but visual inspection of the bridge by WisDOT revealed a depressed area approximately 4-feet deep by 25-feet long, by about 50-feet wide across the roadway making it unfit for traffic. The bridge was closed within few hours following fracture. Results of the survey carried out Connor et al. (2005) as part of the NCHRP Synthesis 354 project revealed that 9 DOTs classified three girder bridges as fracture critical while 28 DOTs classified them as non-fracture critical. Although three girder Hoan Bridge will be typically considered as non-fracture critical. In the presence of constrained induced fracture detail that can cause brittle fracture, Hoan Bridge is analyzed for multiple fracture scenarios using the guidelines proposed in Section 6.3. Retrofit of the connection details is necessary to make the bridge eligible for system redundancy analysis. Details about the bridge geometry and the finite element model are provided in Chapter 4.

6.4.1.1 Step I: Bridge capacity to withstand collapse following fracture

Although FHWA clarification memo recommended a full fracture scenario for fracture critical members, the present analysis is carried out assuming a bottom flange and full web depth fractures in girder E and F which is same as the damage observed in the field. Based on the weigh-in-motion study that was carried out following the fracture by The University of Michigan, it was estimated that the load that caused the fracture was a tractor-trailer salt truck with a weight of approximately 100 kips (Fisher, et al., 2001). Therefore, the analyses in *Step-I* are carried out using 100 kip truck load with the rear and front axle spacing of 14 ft. The individual axle loads are calculated by scaling the HS-20 (72 kips) truck load distribution to get a 100 kip truck. The deflection limit of (span length/100) from Ghosn and Moses (1998) leads to deflection limit of 26 in. which is lower than the approximate 4 ft. vertical deflection observed in the field. Locations of high shear (0.1L from Pier 3S) and maximum positive bending moment (0.4L from Pier 2S) are considered as possible fracture locations during *Step-I* analysis with individual and multiple fracture scenarios.

Figure 6-3 and Figure 6-4 shows the response of girder F at the location of fracture following fracture at 0.4L from Pier 2S and 0.1L from Pier 3S respectively. The deflection response is recorded at 0.4L because that is where the maximum deflection is observed irrespective of fracture location. Simultaneous fracture in girder E and F at 0.4L from Pier 2S resulted in significantly larger deflection compared to fracture in either girder E or F at 0.4L from Pier 2S.

The mean value of deflection from simultaneous fracture scenario exceeded the deflection limit of 26 in. In case of simultaneous girder E and F fracture, use of rate dependent material properties resulted in reduction in girder deflection but the mean deflection is still higher than the deflection limit of 26 in. as shown in Figure 6-5. The results presented here are further discussed in detail in Section 4.6 of the Hoan Bridge Study chapter. The order in which fractures occurred on the Hoan Bridge is unclear. It is possible that fractures did not occur simultaneously. Therefore, fracture sequence scenarios in which the exterior girder F is fractured first followed by the interior girder E fracture and vice versa are also evaluated. Section 4.6.2 provides a detailed discussion about the impact of fracture sequence on the bridge deflection. Sequential introduction of fracture resulted in lower deflection at the fracture location and therefore only simultaneous fracture scenario is considered for evaluation in this section.

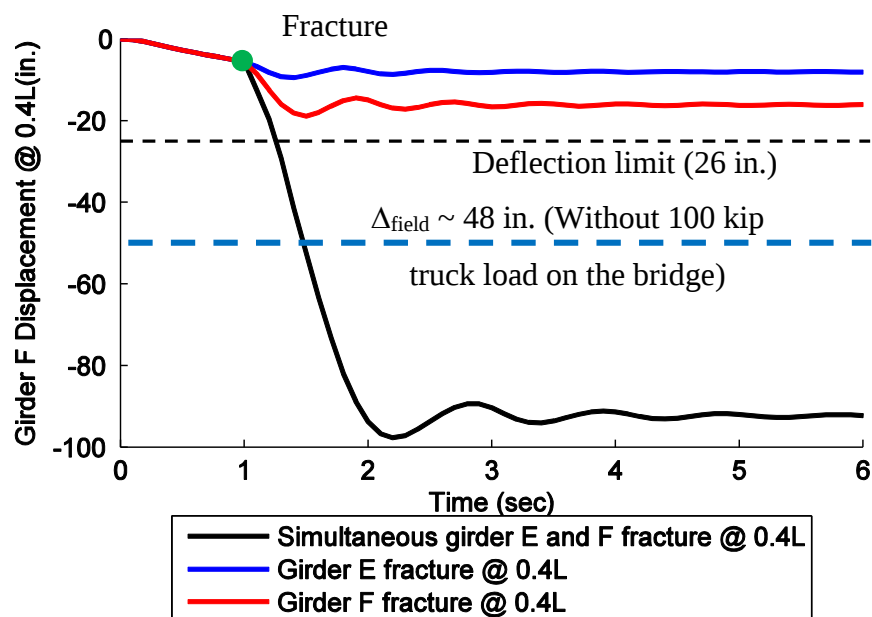


Figure 6-3: Deflection of girder F at 0.4L (from Pier 2S) following fracture at 0.4L (from Pier 2S).

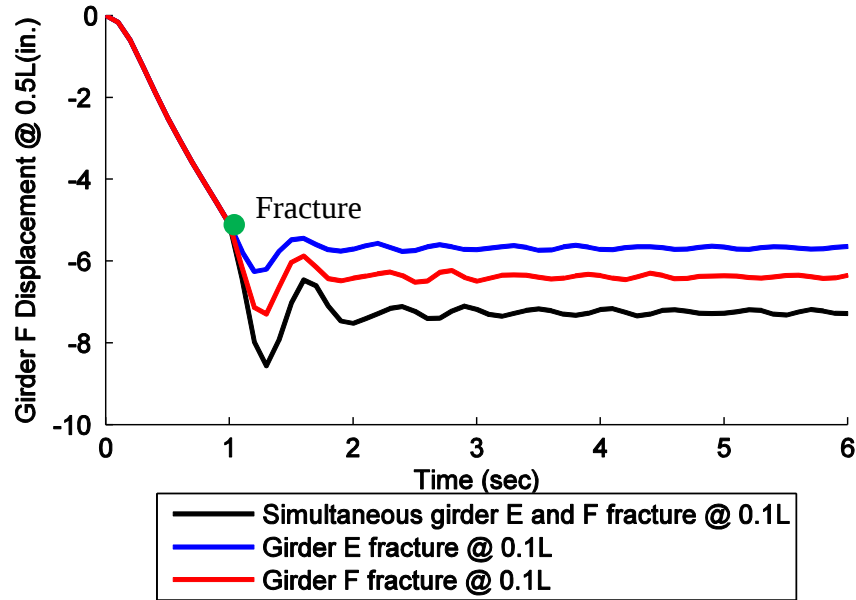


Figure 6-4: Deflection of girder F at 0.4L (from Pier 2S) following fracture at 0.1L (from Pier 3S).

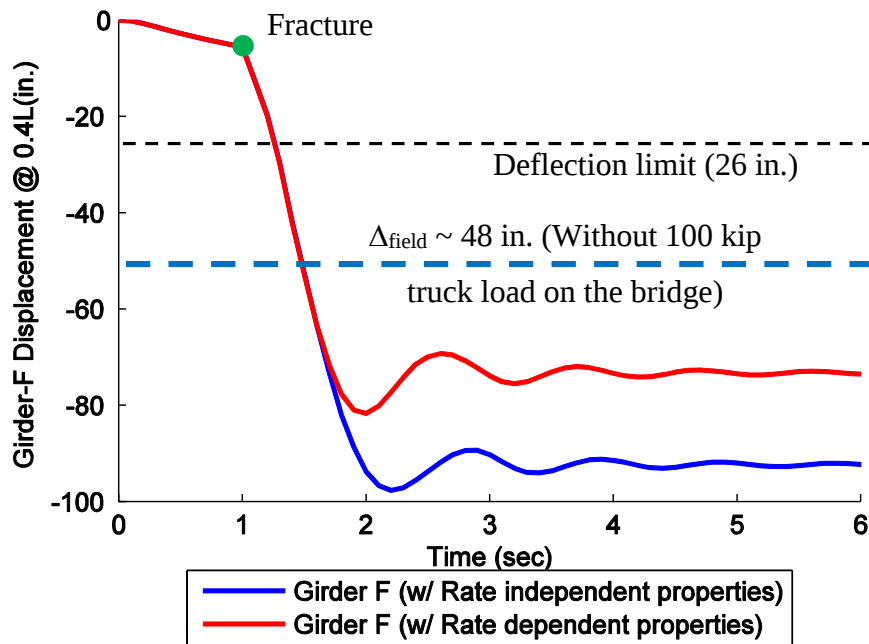


Figure 6-5: Comparison between dynamic response of girder F at 0.4L (from Pier 2S) following simultaneous fracture in girder E and F with rate dependent and independent material properties.

6.4.1.2 Conclusion

In the present system redundancy analysis, *Step-II* is not performed because deflection at the end of *Step-I* analysis and that observed in the field following fracture at 0.4L (from Pier 2S) are significantly higher compared to the prescribed deflection limit. In the presence of the predicted deflection from simulation and a 100 kip tractor-trailer salt truck still on the bridge near fracture location, it can be argued that new vehicle could not enter the fractured span without causing a catastrophic failure. Thus, the Hoan Bridge girders could not be reclassified as non-fracture critical for in-service inspection.

6.4.2 Twin box-girder bridge tested at The University of Texas at Austin

Barnard et al. (2010) at The University of Texas at Austin investigated the redundancy in simply supported twin box-girder bridge through full scale testing and finite element analysis. Details about the bridge geometry and the finite element model are provided in Chapter 5.

6.4.2.1 Step I: Bridge capacity to withstand collapse following fracture

Based on the FHWA clarification memo full fractures at two locations in the exterior box-girder are analyzed in the first step. The first fracture location is assumed at mid-span, which is the location of maximum bending moment in a simply supported bridge. The second fracture location is assumed at 3ft. from the south end of the bridge, which is close to the location of maximum shear force. The analyses are performed with rate dependent material properties for steel and concrete. The deflection values are plotted by excluding the girder camber. The *Step-I* evaluation is done in two steps:

- a) *Load-I*: In the *Load-I* step, the bridge capacity to withstand collapse following fracture is evaluated with a single HS-20 truck in the right hand design lane positioned to cause maximum moment at the location of fracture. The deflection limit of (span length/100) proposed by Ghosn and Moses (1998) is used to evaluate whether the residual deformation is perceivable to people on or off the bridge. This limit leads to deflection limit of 14.4 in. Figure 6-6 shows the post fracture response of the exterior girder following a full depth fracture at mid-span and 3 ft. from the south end of the bridge. Following full depth fracture at 3 ft. from the south end of the bridge, the maximum deflection still occurred at mid-span and it showed less than 0.1 in. change in deflection. The mean deflection following full depth

fracture at mid-span is lower than the deflection limit of 14.4 in. Therefore, the fracture might go unnoticed and the bridge might stay in service until the next inspection.

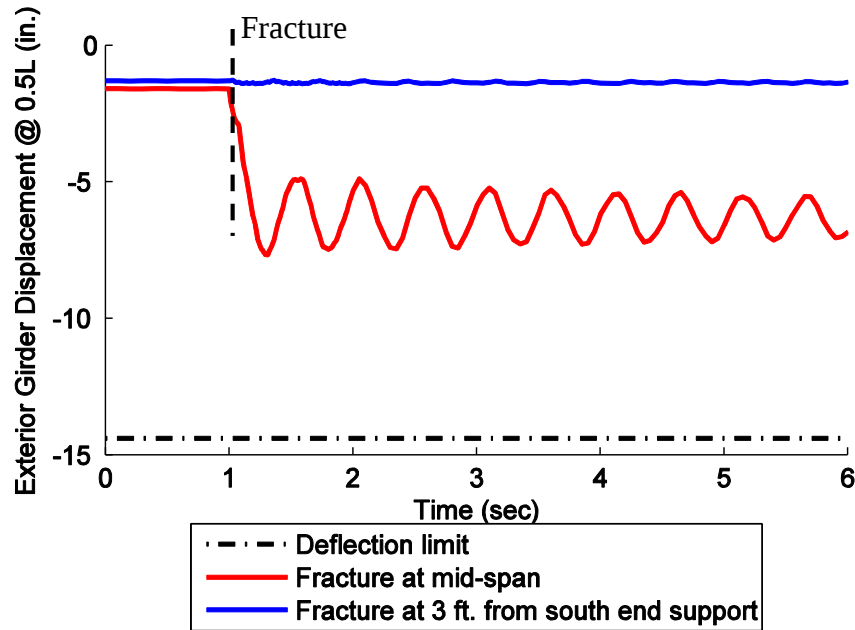


Figure 6-6: Load-I post fracture response with HS-20 truck load on the bridge.

- b) *Load-II*: If the bridge satisfies the *Load-I* requirements, the bridge capacity to withstand collapse following fracture is evaluated with the HL-93 load in both design lanes. The bridge model is reanalyzed for the two fracture scenarios discussed in *Load-I*, with the HL-93 load in both design lanes. Figure 6-7 shows the post fracture response of the exterior girder following full depth fracture at mid-span and 3 ft. from the south end of the bridge. In case of full depth fracture at mid-span, bridge deflection grew rapidly leading to analysis failure with buckling of cross frame at 0.4L from the south end of the bridge. Buckling of cross frame would not lead collapse of the actual bridge. As discussed in Chapter 5, a linear bulk viscosity definition that helps with the dissipation of spurious oscillation is included in the analysis. To further evaluate whether brace buckling is real or numerical, the bridge model is reanalyzed with linear elastic cross frames. The bridge model with linear elastic cross frames recovered, but after the first cycle, the deflection grew rapidly leading to bridge collapse. If HL-93 loading represents a possible loading on the bridge then box-girders on the bridge cannot be reclassified as non-fracture critical for in-service inspection.

The maximum anticipated load determined through the weigh in motion study can be used instead of the HL-93 load. The bridge model with the full depth fracture at mid-span is reanalyzed with the widely used two side by side HS-20 trucks load model from the NCHRP 406 report by Ghosn and Moses (1998). Two side by side HS-20 trucks are placed to cause maximum moment at mid-span. Figure 6-8 shows the post fracture displacement response at mid-span in the exterior girder. The mean deflection is lower than the deflection limit of 14.4 in.

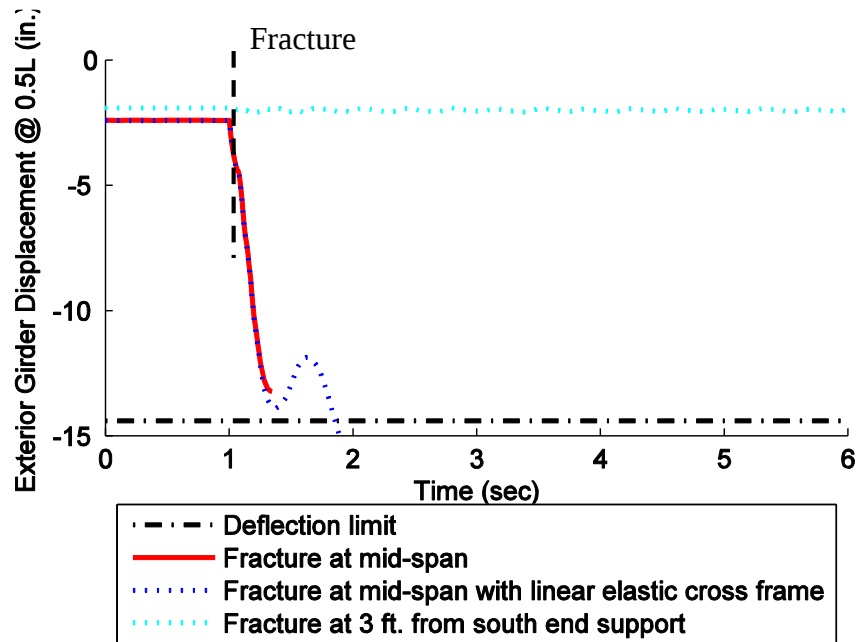


Figure 6-7: *Load-II* post fracture response with HL-93 load on the bridge.

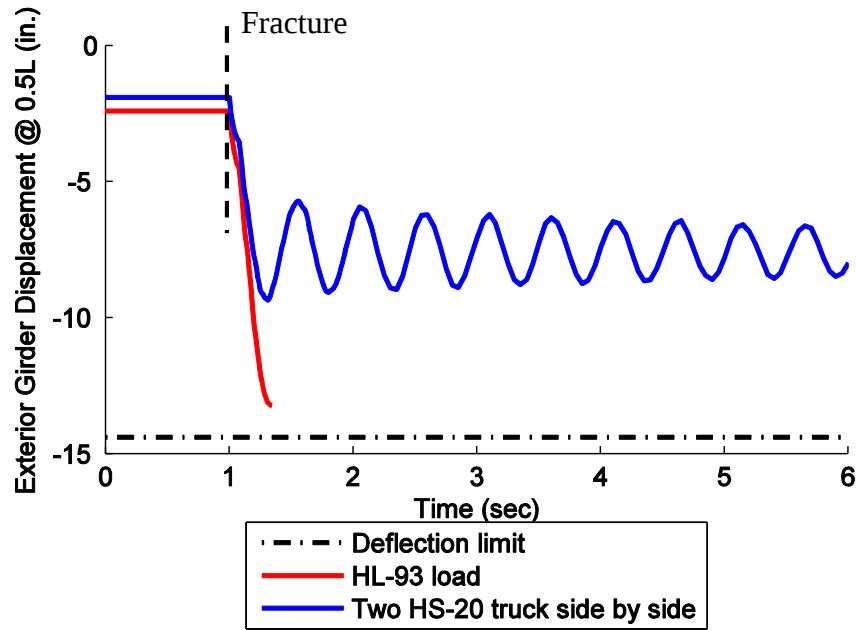


Figure 6-8: *Load-II* post fracture response.

The residual deflection from *Load-I* and *Load-II* analysis is lower when compared against the deflection limit of (Span Length/100) except in the case of mid-span full depth fracture with the HL-93 load. Therefore, the fracture might go unnoticed and the damaged bridge may stay in service until next inspection.

6.4.2.2 *Step-II: Reserve load carrying capacity of the damaged bridge*

As the bridge satisfies the deflection limit in *Step-I* for all load cases except the HL-93 load in both design lanes, the reserve load carrying capacity of the damaged bridge is evaluated by gradually increasing the load until bridge collapse. In the *Step-II* evaluation, out of the two full depth fracture scenarios evaluated in *Step-I*, only the fracture at mid-span of the exterior girder is considered for further evaluation because it yielded significantly higher deflection response compared to the full depth fracture at 3 ft. from the south end of the bridge. The reserve load carrying capacity of the damaged bridge is evaluated by taking the bridge configuration from the end of *Load-II* simulation with two HS-20 trucks side by side as the initial condition. The reserve load carrying capacity of the bridge is evaluated by first unloading the two HS-20 trucks side by side loading followed by loading the bridge with the selected load model and increasing the load until bridge collapse. This simulation is carried out using three loading cases:

- a) *HL-93 load in both design lanes*: HS-20 trucks are placed in both the design lanes at mid-span and the lane load is applied in both design lanes along the entire span length (Figure 6-9).
- b) *HS-20 trucks side by side*: Two HS-20 trucks are placed side by side at mid-span in the design lanes (Figure 6-10).
- c) *Single HS-20 truck*: HS-20 truck is placed in the right hand design lane over the fractured girder at mid-span (Figure 6-11).

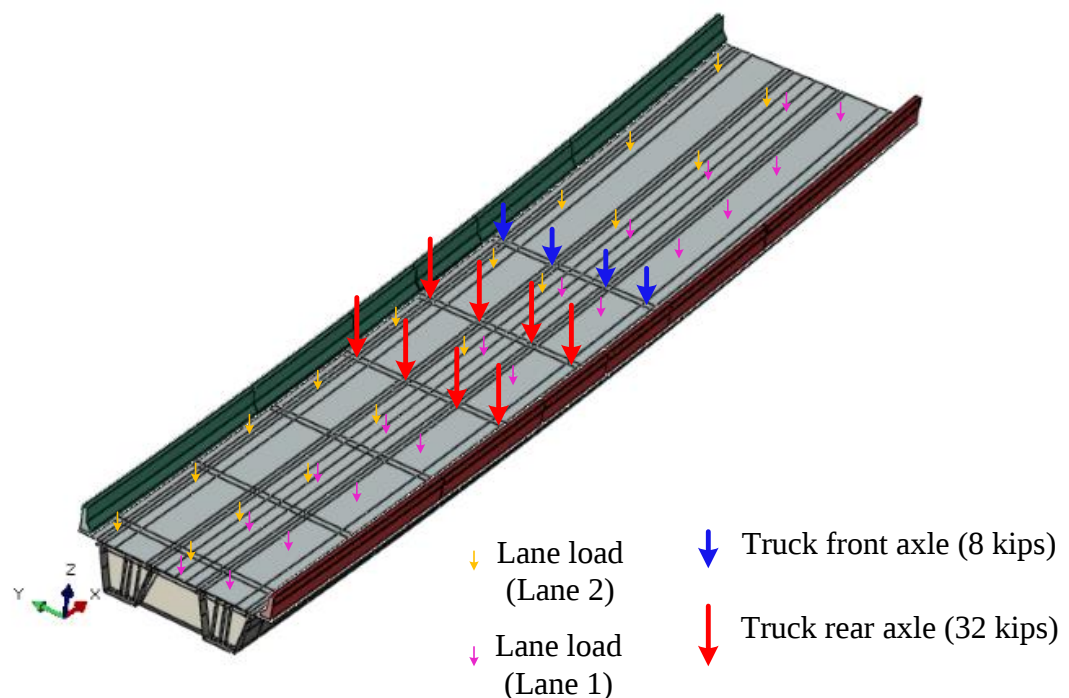


Figure 6-9: HL-93 load in both design lanes.

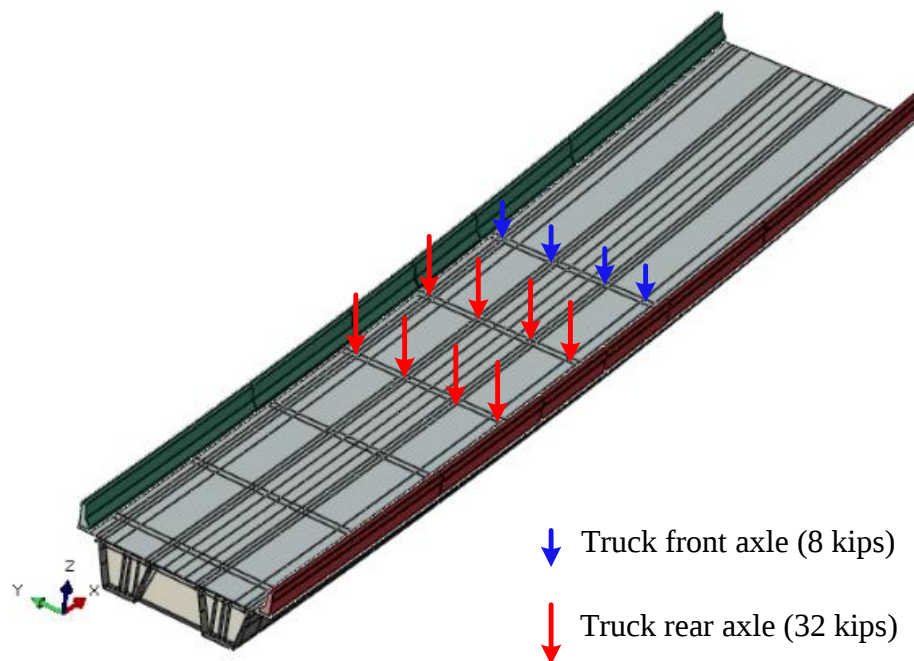


Figure 6-10: HS-20 trucks side by side.

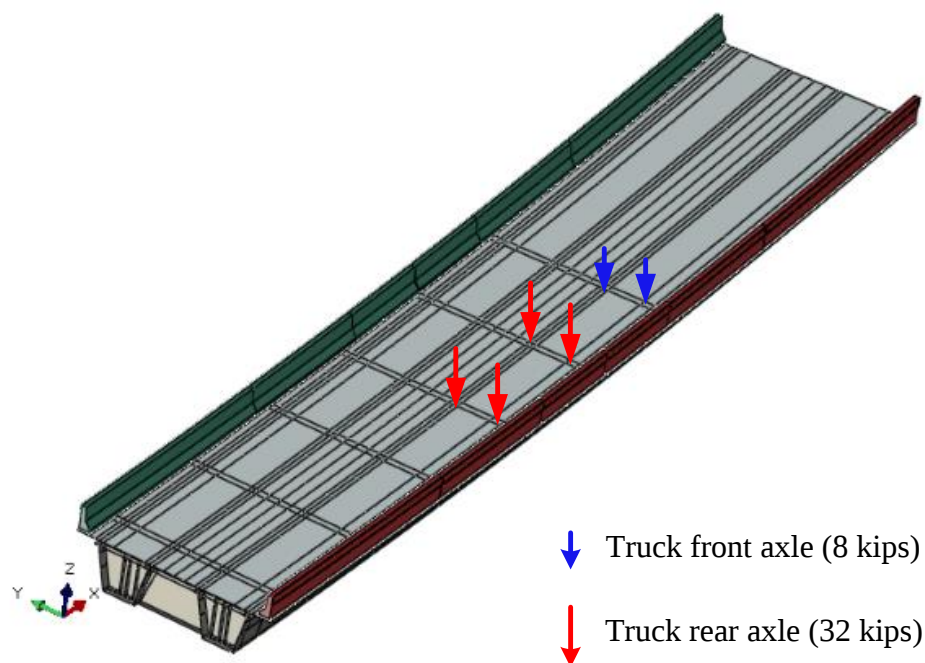


Figure 6-11: Single HS-20 truck.

Figure 6-12 shows the load vs. displacement response for the loading scenarios discussed before. The failure load, the total deflection measured from the start of the analysis excluding girder camber and the additional deflection measured from the start of remaining load carrying capacity analysis in *Step-II* are tabulated in Table 6-3. Deflection values (Figure 6-12) when the applied load on the bridge reached 72 kips for all three loading scenarios and corresponding initial bridge stiffness are tabulated in Table 6-4. The HS-20 truck in the right hand lane over the fractured girder (Figure 6-11) yielded a lower reserve load carrying capacity and a lower initial stiffness (Table 6-4) as compared to other loading scenarios with distributed load over a larger slab area. The lane load and the HS-20 truck in the HL-93 loading (Figure 6-9) puts the undamaged shear studs in both the girders in compression providing additional strength at the interface and it also engages a larger slab area possibly leading to higher reserve load carrying capacity. In the HS-20 trucks side by side loading scenario (Figure 6-10), the truck in the left design lane puts the undamaged studs in the intact girder near mid-span in compression providing additional strength at the interface and it also engages wider area of the slab possibly resulting in higher reserve load carrying capacity.

Figure 6-13 shows the lateral deflected shape (along the y-axis) of the box-girders when the total applied load on the bridge reaches 72 kips under different loading scenario. The fractured girder twists in the opposite direction, with part of the girder on the right hand side of the fracture experiencing larger lateral deflection. The *Single HS-20 truck* loading scenario resulted in larger lateral deflection of the fractured and the intact girder close to fracture location. The addition of a HS-20 truck in the left hand design lane (*HS-20 trucks side by side*) resulted in reduction in lateral movement of both the girders close to the fracture location. The application of HL93 loading in both design lanes (*HL-93 load in both design lanes*) resulted in a reduction in lateral deflection of the fractured girder close to fracture location. Compared to the *HS-20 trucks side by side* loading scenario, the lateral deflection of the intact girder increased with the application of the HL-93 loading in both design lanes. Figure 6-14 shows stud damage using the connector overall damage variable (CDMG) at the time of failure. The CDMG value ranges from 0-1 with zero corresponding to intact stud and 1.0 corresponding to failed stud. Additional stud damage is observed on the interior flange of the fractured girder with *Single HS-20 truck* loading scenario. The other two loading scenarios predicted additional stud damage on the exterior flange of the intact girder.

To evaluate reduction in structural stiffness because of full depth fracture, a linear elastic model of the fully composite bridge is analyzed under the same loading scenarios discussed before. Initial stiffness values from these analyses are compared against the damaged bridge model initial stiffness values in Table 6-5. During calculation of the initial stiffness in the damaged bridge model, the load vs. displacement response is assumed to be linear up to the applied load. This assumption is valid as the initial stiffness values tabulated in Table 6-4 and Table 6-5 are similar. The initial stiffness value using the linear elastic bridge model is three times higher compared to the damaged bridge model (Table 6-5).

Table 6-3: Failure load and deflection under different loading scenarios.

Loading scenarios	Failure load (kips)	Total deflection (in.)	Additional deflection (in.)
HS-20 truck in the right hand design lane	381.6	14.9	9.2
HS-20 trucks side by side	599.4	20.5	14.8
HL-93 load in both design lanes	732.5	23.3	17.5

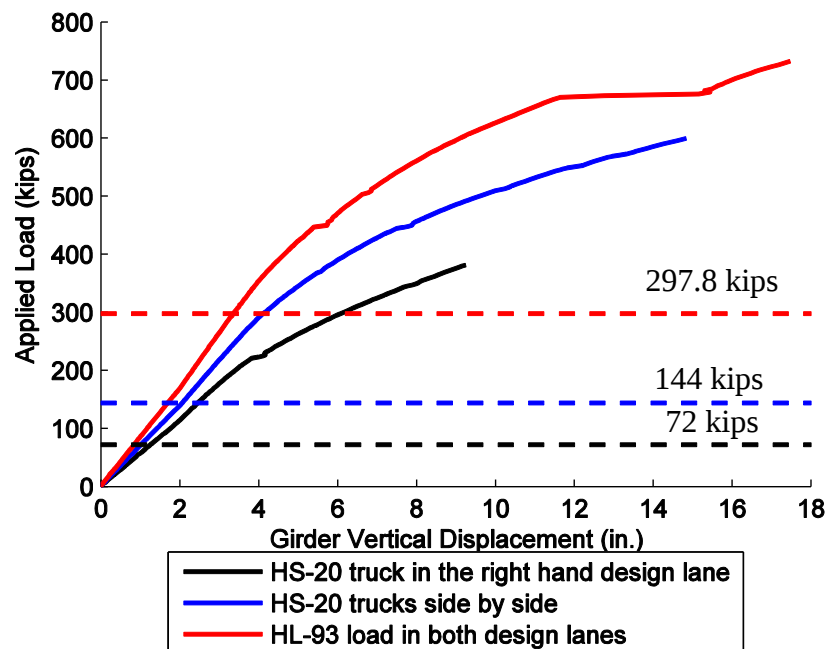


Figure 6-12: Load vs. displacement response under different loading scenarios following full depth fracture at mid-span in the exterior girder.

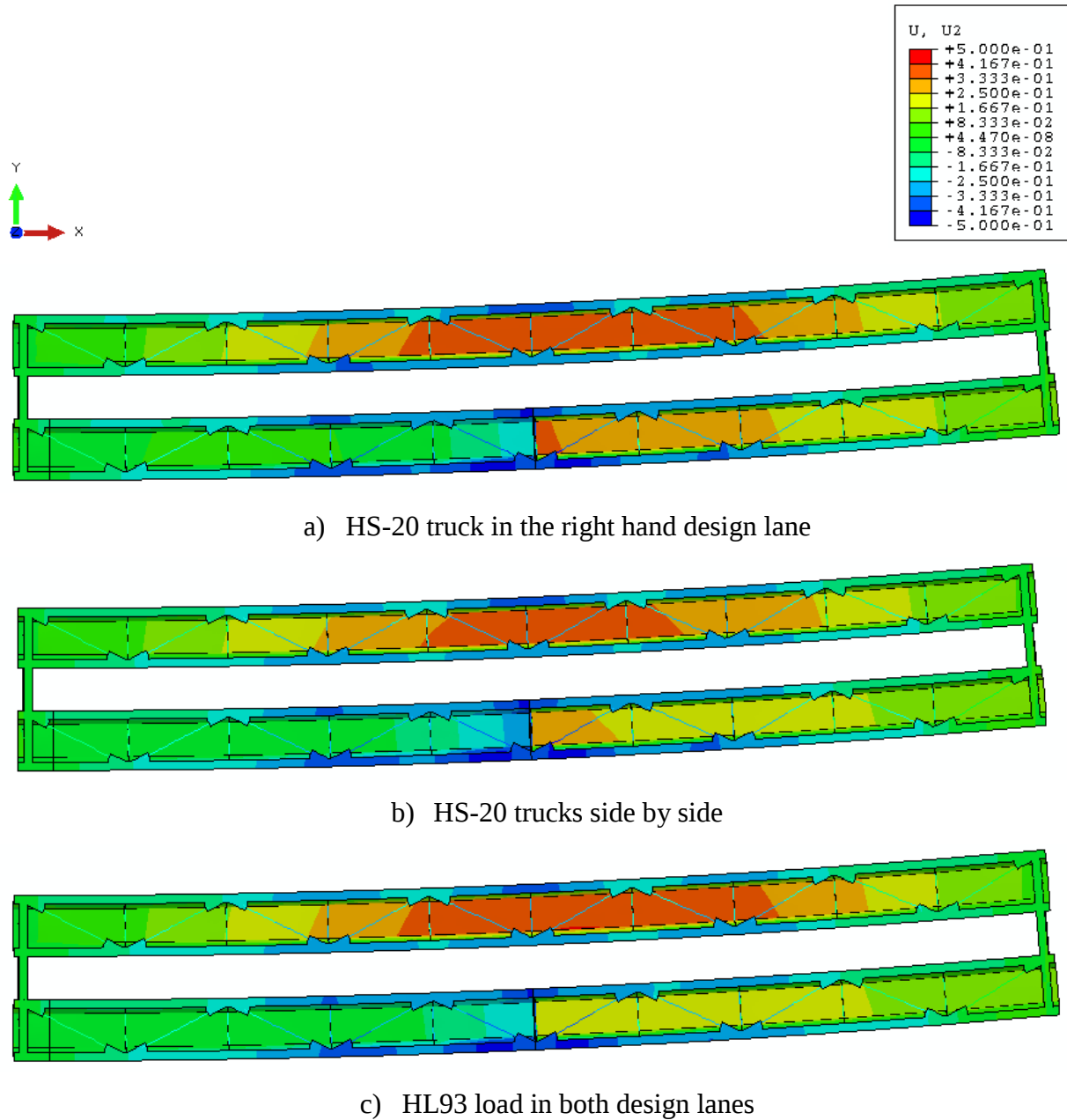


Figure 6-13: Lateral deflection (along y-axis) of twin box-girders under applied load of 72 kips.

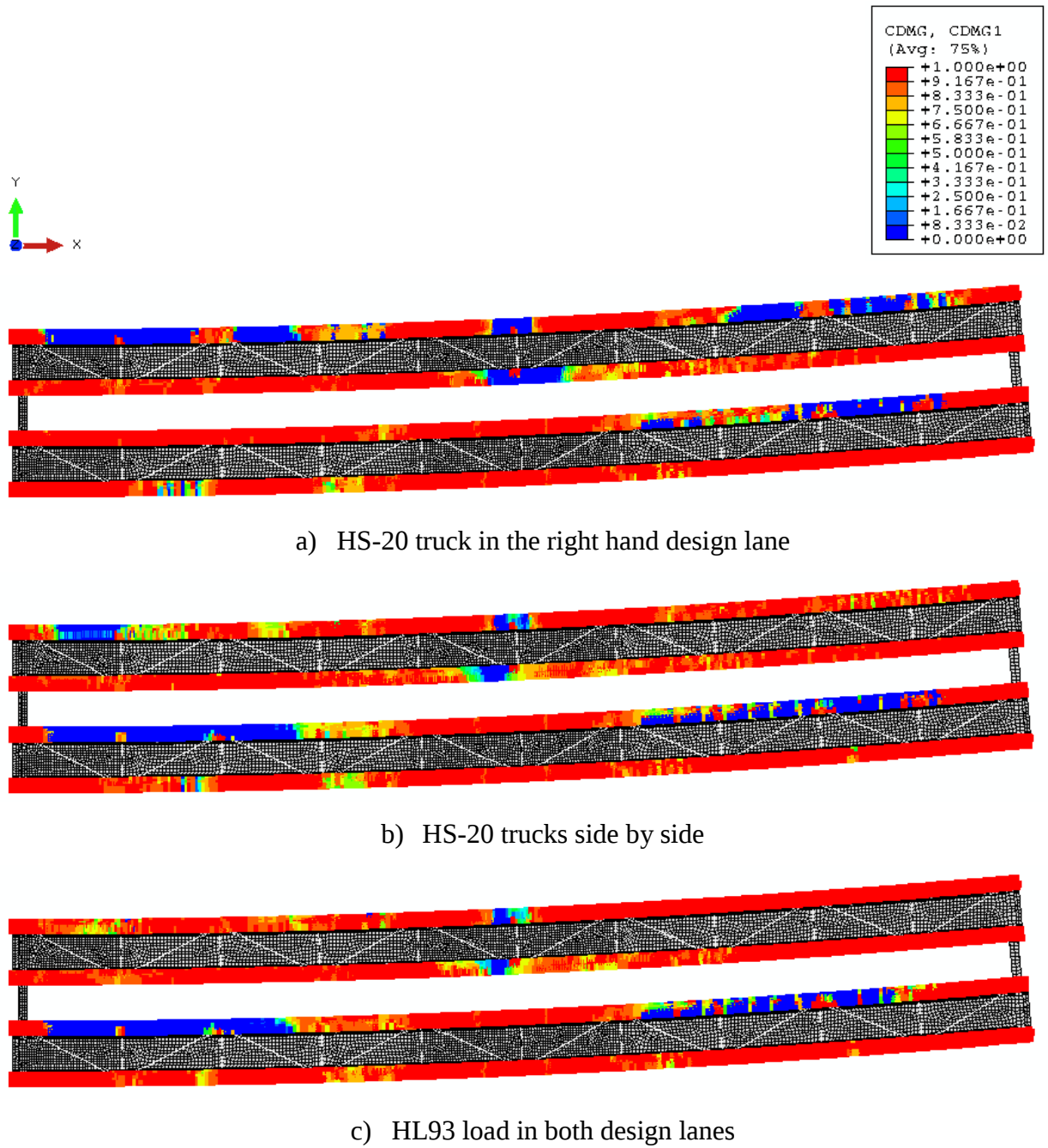


Figure 6-14: Connector overall damage variable (CDMG) at failure under different loading scenarios.

Table 6-4: Deflection at fracture location under applied load of 72 kips.

Loading scenarios	Applied load (kips)	Deflection (in.)	K=Force/Deflection (kips/in)
HS-20 truck in the right hand design lane	72	1.25	57.5
HS-20 trucks side by side		1.01	71.1
HL-93 load in both design lanes		0.83	86.3

Table 6-5: Stiffness comparison between intact and damaged bridge.

Loading scenarios	Applied load (kips)	Structural Stiffness (kips/in)	
		Intact Bridge	Full depth fracture at mid-span
HS-20 truck in the right hand design lane	72	181.1	57.6
HS-20 trucks side by side	144	202.3	69.9
HL-93 load in both design lanes	297.77	248.6	88.9

6.4.2.3 Conclusion

During *Step-I* analysis, the twin box-girder bridge resisted collapse under both the HS-20 truck in the right hand design lane and two HS-20 trucks side by side, but it failed under the HL-93 loading in both design lanes. Assuming two HS-20 trucks side by side as the maximum anticipated load pattern on the bridge, the reserve load carrying capacity of the bridge is evaluated in *Step-II*. The reserve load carrying capacity evaluation in *Step-II* showed sensitivity to the applied load pattern with *Single HS-20 truck* yielding the lowest failure load of 381.6 kips which corresponds to 5.3 times the HS-20 truck load (72 kips) assuming dynamic load allowance of 1.00. The twin box-girder bridge also successfully resisted 4.2 times the two HS-20 trucks placed side by side loading case and 2.5 times the HL-93 loading in both design lanes assuming dynamic load allowance of 1.00. If secondary load paths are viable during the two step system redundancy analysis and the HL-93 loading in both design lanes does not represent a possible loading scenario during *Step-I* evaluation, then the exterior box-girder on the bridge can be reclassified as non-fracture critical for in-service inspection.

6.5 Conclusion

The objective of this chapter is to propose and evaluate a methodology for system redundancy analysis of fracture critical bridges, specifically three girder bridges and twin tub girder bridges. The methodology is based primarily on advanced finite element analysis. The proposed methodology is evaluated through analysis of the Hoan Bridge (Fisher, et al., 2001) and twin box-girder bridge tested at The University of Texas at Austin (Barnard, et al., 2010) in Section 6.4. The Hoan Bridge yielded significantly higher deflection following simultaneous fracture in girder E and F, failing to satisfy the deflection limit set in the *Step-I* analysis. In the presence of the predicted deflection from simulation and a 100 kip tractor-trailer salt truck still on the bridge near fracture location, it can be argued that new vehicle could not enter the fractured span without causing a catastrophic failure. Thus, the Hoan Bridge girders could not be reclassified as non-fracture critical for in-service inspection. The twin box-girder bridge resisted collapse under two out of three loading scenarios in *Step-I* analysis. During reserve load carrying capacity analysis in *Step-II*, bridge successfully resisted loading from three possible loading scenarios. If the HL-93 loading in both design lanes does not represent a possible loading scenario in *Step-I* analysis and secondary load path is valid then exterior box-girder could be reclassified as non-fracture critical for in-service inspection.

7 Summary and conclusions

Cases of brittle fractures in major bridges the 1960's and 1970's prompted AASHTO to publish its first fracture control plan in 1978. It defined fracture critical members (FCMs) as “A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse”. The fracture control plan focused on material and fabrication standards, and required periodic 24 month hands-on inspection of bridges with fracture critical members. This leads to a significant increase in life cycle cost, discouraging bridge designers and owners from using bridge types that incorporated fracture critical members.

Apart from the Point Pleasant Bridge that failed in 1967, no other bridge has collapsed in the USA following a fracture. All these damaged bridges that did not collapse following a fracture showed some degree of redundancy, and therefore could be reclassified as non-fracture critical if detailed analyses are carried out. If we assume that some damage is acceptable, and then the question becomes is damaged bridge capable of resisting external loads until next inspection. These questions, which this thesis addresses, have not been studied carefully both because of the large computational resources required and the lack of robust models to track collapse of structures. The goal of this study is to provide guidance on redundancy evaluation of fracture critical bridges, specifically three girder bridges and twin tub girder bridges through advanced finite element analyses. The work was conducted within the scope of NCHRP Project 12-87. The section 7.1 summarizes the key findings from this research followed by recommendations for future work in section 7.2.

7.1 Key findings

The finite element models of the Hoan Bridge (Fisher, et al., 2001) in Chapter 4 and the twin box-girder bridge (Barnard, et al., 2010) in Chapter 5 did not fully capture the response of the structures. In the case of the Hoan Bridge, only one gross field measurement was available, and thus there was little guidance on how the model could be improved. Visual comparisons indicated that the analysis and field evidence showed similar deformation patterns but wide quantitative differences. In the case of the UT bridge, the sensitivity of the solution to the stud model, and in particular the analytical process based on a series of small steps within which equilibrium was enforced, resulted in a progressive rather than sudden unzipping of the shear

connection in Test 3. The proposed modeling guidelines and system redundancy analysis methodology in Chapter 6 are in early stages of its development and are not completely viable in the present form. There are areas that need further research before the proposed methodology and modeling guidelines are ready for use. The following sections summarize the key findings from this research categorized into three sub sections and are as follows:

7.1.1 Impact of geometric and modeling assumptions on the post fracture bridge deformation and the reserve load carrying capacity of damaged bridge

1. In the nonlinear dynamic analyses, the use of a stiffness proportional coefficient used by Rayleigh damping tends to over-damp higher modes and can result in unrealistically high damping. Conversely, use of a mass proportional coefficient underdamps higher mode effects but it can generate extraneous damping forces if the total motion involves rigid body motion.
2. In case of the Hoan Bridge, the non-composite slab modeled with frictional contact at the interface played crucial role in load redistribution following fracture. Non-composite action needs to be carefully modelled to obtain reasonable results.
3. In the event of fracture, shear studs are subjected to combined shear and tension. The popular rigid stud approach used to model the composite action fails to capture the unzipping of shear studs that might occur following a fracture event, leading to a significant reduction in twin box-girder bridge deflection and a false perception of safety. The interaction equation in ACI 318-11 (2011) and PCI Design Handbook, 6th Edition (2004) which accounts for shear-tension interaction in shear studs provide a reasonable starting point for modeling combined shear and tension interaction.
4. Secondary elements (guardrail and lower lateral bracing) play crucial role in the load redistribution following fracture, and their correct analytical representation is essential to capture the post fracture response. Excluding secondary elements from the bridge model can lead to a modification in the load redistribution path and result in erroneous results. For example, exclusion of lower lateral bracing in the Hoan Bridge model resulted in a rapid increase in deflection beyond realistic values (>10 ft.) with significant yielding and buckling in the fractured girders E and F. In the absence of the lower lateral bracing, the load redistributed to the intact girder D was reduced, which in turn, reduced its displacement.
5. Realistic material properties with rate dependent adjustments properties should be used to capture realistic post fracture response and correct demand on the connections that form part

of the load redistribution path. The rate dependent properties in UFC 3-340-02 (2008) guidelines provide a good starting point in simulation of dynamic response following fracture. In the case of the Hoan Bridge, the use of rate dependent material properties resulted in reduction in fractured girder deflection response by ~20%; in the fractured composite twin box-girder bridge this reduction was ~4%. Part of the reason for the reduced sensitivity in the twin box-girder bridge as compared to the Hoan Bridge is the absence of rate dependent properties for shear studs in the twin box-girder bridge analysis.

6. Idealization of bridge boundary conditions as pin, roller and fixed condition can lead to additional restraint for bridge near collapse. The bearing pads can be idealized using elastic springs with stiffness values determined based on the equations used for design of elastomeric bearing pads from section 14.6.3.1-2 of AASTHO LRFD design specification (2012).
7. Review of the present literature showed that the geometric imperfection and residual stresses does not play significant role in the ultimate load carrying capacity and the load vs. displacement response of the structure as long as its response is not governed by local behavior. In addition, the actual imperfection data is not available for majority of bridges and one would expect the second order effects arising from fracture to have significant impact than residual stresses on the load carrying capacity and the global response of the bridge.

7.1.2 Methods to capture the post fracture dynamic response and the reserve load carrying capacity of damaged bridge

1. Currently only a set of nonlinear dynamic analyses can be used to study post fracture response as simplified linear and nonlinear static models with dynamic load amplification factor failed to capture the post fracture response during simulation of Hoan Bridge post fracture response.
2. Based only on the twin box-girder results it difficult to select between the simplified modelling approaches to estimate the remaining load carrying of the damaged bridge with a given level of confidence. This choice is complicated by the choice of shear stud model, which significantly altered the response of the bridges studied. Therefore, the suitability of any simplified modelling approach cannot be determined without first establishing a stud model that is representative of the actual stud behavior.

3. The order in which fractures occurred on the Hoan Bridge is unclear. It is possible that fractures did not occur simultaneously. Sequential introduction of fracture resulted in lower deflection at the fracture location compared to simultaneous introduction of fracture. The fractured girder E and F deflection response is found to be sensitive to the fracture sequence while the intact girder D showed less than 1% change in deflection response.
4. The partial depth web fractures in girder D of the Hoan Bridge affected the local stress distribution in the girder web, but it did not affect the global bridge response in the analysis where fracture in all three girders is introduced simultaneously.

7.1.3 System redundancy analysis methodology

1. Given the uncertainty associated with possibility of fracture, estimating the strength and the loading on bridges, a system redundancy analysis methodology requires the use of a probabilistic approach. However, in the absence of enough field and simulation data to establish probabilistic guidelines deterministic approach for system redundancy analysis of fracture critical bridges is proposed as part of this research. It consists of two-step methodology to classify members as non-fracture critical for in-service inspection. The first step evaluates bridge capacity to withstand collapse following fracture and evaluate whether the residual deformation is perceivable to people on or off the bridge. If the bridge satisfies the first step requirements then the reserve load carrying capacity of the damaged bridge is evaluated in the second step.
2. Fracture location in the primary load carrying members is not limited to the areas of high bending and shear force. Fracture locations also need to be selected based on the presence of known fracture critical details in the bridge, presence of initial defects and the current bridge condition. Different fracture locations needs to evaluated to assess load ranges for members and connections on the bridge. If there is probability of fracture propagation to other members then multiple of fracture scenarios should be considered in the simulation.
3. The two side-by-side HS-20 truck load model has been adopted widely for redundancy evaluation. This model was based on the study done by Nowak (1999) for load and resistance models for AASHTO LRFD design provisions. This model governed for two lane bridges. For other bridge configurations, other load models governed. Therefore, the HL93 load that includes uniformly distributed lane load of 640 lb. /ft. with the HS-20 truck was proposed by Nowak (1999) to develop the maximum possible load effects on wide range of bridges. Other

possible loading scenarios that might develop severe load effects on crucial members should also be considered during system redundancy analysis.

4. The deflection limit of (span length/100) proposed by Ghosn and Moses (1998) as part of NCHRP 406 report is stringent for *Step-I* of the system redundancy analysis methodology and lower deflection limit can be used in agreement with the DOTs and its consultants.
5. During reserve load carrying capacity evaluation of twin box-girder bridge, the *Single HS-20 truck* load that concentrates loading right over the fractured girder yielded lower reserve load carrying capacity compared to the other loading scenarios (*HL-93 load in both design lanes* and *HS-20 trucks side by side*) that distributed load over larger slab area.
6. The Hoan Bridge analysis yielded deflection higher than the deflection limit of (span length/100) following simultaneous fracture in two out of three girders. Therefore, the Hoan Bridge girders could not be reclassified as non-fracture critical for in-service inspection.
7. The twin box-girder bridge not only successfully resisted collapse following full depth fracture in the exterior box-girder under different loading conditions except the HL-93 loading in both design lanes but also satisfied the deflection limit. The twin box-girder bridge displayed satisfactory reserve load carrying capacity and therefore the exterior box-girders could be reclassified as non-fracture critical for in-service inspection.

7.2 Future work

Based on the findings of the present research areas that need further investigation are as follows:

1. In case of Hoan Bridge, the top flange of the girders and stringer beams were encased in the concrete haunch, providing partial composite action, which is excluded from the non-composite model used in this study. Results from the non-composite model provide a conservative estimate of the global response, but including partial composite action in the finite element model could possibly result in higher demand on the connections and members near the fractured region. For this reason, in future research, a model that accounts for partial composite action should be developed and compared with the results discussed in chapter 4.
2. In simulation of the post fracture response following fracture and the remaining load carrying capacity evaluation of the damaged twin box-girder bridge, the choice of shear stud model in combined shear and tension significantly altered the bridge response. Further experimental and analytical research is needed to develop an accurate mathematical representation of stud

shear-tension interaction behavior and account for effect of parameters such as haunch thickness, stay in place metal formwork, reinforcement detail near studs, stud embedment length, stud spacing and positioning etc. on the stud strength and its load vs. deflection/slip response.

3. Impact of time dependent behavior (creep and shrinkage) of concrete on the post fracture response and the remaining load carrying of damaged bridge also needs further evaluation.
4. Simplified linear and nonlinear static models with dynamic load amplification factor are evaluated to capture the post fracture dynamic response as part of the Hoan Bridge study in chapter 4. The dynamic load amplification factor needs further evaluation to ensure it correctly captures the demand on members and connections following fracture.
5. As part of future research, simplified models proposed in chapter 5 to estimate the remaining load carrying of the damaged bridge should be further evaluated and compared against available field and experimental data.
6. The shakedown limit state of the twin box-girder bridge is evaluated by applying increasing magnitude of load cyclically but load was cycled only once at each load increment. For this reason, in future the shakedown limit should be further evaluated by performing the analysis with multiple cycles at each load increment.
7. Connections capacity to withstand forces following fracture plays crucial role in the validity of the load redistribution path. During analysis of the three girder bridge and the twin box-girder bridge evaluated as part of this research, possibility of connection failure was excluded based on the field and test observations respectively. In case of in service or new fracture critical bridge it is necessary to evaluate connection capacity to withstand forces following fracture. Connection capacity to withstand these forces can only be evaluated through advanced analysis accounting for complicated geometric and material effects (Dusenberry, 2010). Therefore, in future a methodology should be developed to evaluate connections capacity to withstand forces post fracture.
8. Some of the key criteria's that needs to be considered during development of bridge screening guidelines are included in the proposed system redundancy analysis methodology. In future detailed bridge screening guidelines should be developed in agreement with the Federal Highway Administration, DOTs and design engineers.

9. Additionally member and connection details that can improve redundancy of existing and new fracture critical bridges should be evaluated in future through analytical and experimental research.

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